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MATERIALS TESTING

April 2015

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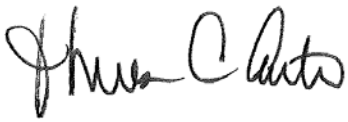
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Materials Testing

Contents

Page

PREFACE x

PART ONE SOILS

Chapter 1 FORMATION, CLASSIFICATION, AND FIELD IDENTIFICATION **1-1**
Formation 1-1
Physical Properties 1-2
Characteristics 1-6
Classification 1-7
Field Identification 1-10
Tests 1-12

Chapter 2 SOIL SURVEYS AND SAMPLING **2-1**
Survey Types 2-1
Objective 2-2
Information Sources 2-3
Field Investigations 2-5
Moisture Content Samples 2-11
Undisturbed Samples 2-11
Quartering Samples 2-18
Soil Profile 2-20

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Chapter 3 MOISTURE CONTENT DETERMINATION 3-1
Oven-Dry Test 3-1
Microwave Oven-Dry Test 3-4
Calcium Carbide Gas Pressure Test 3-6

Chapter 4 SPECIFIC GRAVITY OF SOLIDS DETERMINATION 4-1
Specific-Gravity Test 4-1
Specific Gravity (Apparent and Bulk) 4-7

Chapter 5 GRAIN SIZE ANALYSIS AND DISTRIBUTION 5-1
Sieve Analysis 5-1
Hydrometer Analysis 5-9

Chapter 6 LIQUID LIMIT, PLASTIC LIMIT, AND PLASTICITY INDEX DETERMINATION 6-1
Liquid Limit Determination 6-1
Fall Cone 6-8
Plastic Limit Determination 6-12
Plasticity Index Determination 6-15

Chapter 7 LABORATORY COMPACTION CHARACTERISTICS OF SOIL USING MODIFIED EFFORT 7-1
Compaction Test 7-1
Graph Presentation of Results 7-8
Compaction Equipment 7-13
Other Compactive Efforts 7-13

Chapter 8 IN-PLACE DENSITY DETERMINATION 8-1
Sand Cone or Sand Displacement Method 8-2
Steel Shot Volume Replacement Method 8-6
Drive Cylinder Method 8-11
Nuclear Moisture and Density Tester 8-14
Water Displacement Method 8-15
NonNuclear Density Devices 8-15

Chapter 9 CALIFORNIA BEARING RATIO TESTS 9-1
California Bearing Ratio of Laboratory-Compacted Soils 9-1
Undisturbed Sample Testing 9-10
California Bearing Ratio Data Presentation and Analysis 9-11
In-Place Field California Bearing Ratio Test With the Dynamic Cone Penetrometer 9-25
Plate Bearing Test for Determining Modulus of Soil Reaction 9-33
Soil Modulus Testing With a Light Weight Deflectometer 9-38

Chapter 10 TECHNICAL SOIL REPORT 10-1
Required Tests 10-1
Soil Technical Report 10-3

PART TWO BITUMINOUS MIXTURES

Chapter 11 BITUMINOUS PAVEMENTS AND SURFACES 11-1
Aggregates 11-1
Bituminous Materials 11-1

Chapter 12 MATERIAL SAMPLING 12-1

Bituminous Material Sampling 12-1
Aggregate Sampling 12-1

Chapter 13 FIELD IDENTIFICATION 13-1
Bitumen Field-Identification Tests 13-1
Aggregate Identification and Selection 13-4

Chapter 14 BITUMEN TESTING 14-1
Cleveland Open-Cup and Tag Open-Cup Flash Point and Fire Point Tests 14-1
Viscosity Tests 14-3
Performance-Graded Tests 14-3
Solubility Tests 14-4

Chapter 15 AGGREGATE AND FILLER TESTING 15-1
Sieve Analysis 15-1
Specific Gravity 15-1
Los Angeles Abrasion Test 15-5

Chapter 16 BITUMINOUS-MIX DESIGN 16-1
Hot-Mix Design Considerations 16-1
Optimum Bitumen Content 16-2
Superpave Mix Design 16-10
Job Mix Formula 16-11

Chapter 17 PLANT CONTROL 17-1
Plant Types 17-1
Initiating Plant Production 17-2
Plant Production Control 17-3
Asphalt Content Determination 17-5
Density Tests 17-5
Expediting the Design 17-7

PART THREE CONCRETE

Chapter 18 DESCRIPTION, TERMINOLOGY, MATERIALS, AND CONCEPTS 18-1
Terms 18-1
Materials 18-2
Concepts 18-4
Units of Measure and Conversions 18-7

Chapter 19 FIELD AGGREGATE TESTING 19-1
Stockpile Sampling 19-1
Gradation Determination 19-1
Surface Moisture 19-2
Organic Impurities, Los Angeles Abrasion, and Sulfate Soundness 19-3

Chapter 20 FIELD CONCRETE TESTING 20-1
Fresh Concrete Tests 20-1
Flexural-Strength Test (Modulus of Rupture) 20-7
Compressive-Strength Test 20-10
Nondestructive Testing 20-18

Chapter 21 CONTINGENCY FIELD TESTING AND TROUBLESHOOTING 21-1
Nonstandard Tests 21-1

Troubleshooting 21-2

PART FOUR SOILS STABILIZATION

Chapter 22 MECHANICS OF SOIL STABILIZATION AND MODIFICATION 22-1
Stabilization 22-1
Modification 22-2

Chapter 23 STABILIZING AGENTS 23-1
Stabilizer Types 23-1
Time Requirements for Testing 23-3
Stablizer Selection and Concentrations 23-3
Expedient Stabilization 23-7
Soil Stabilization in Frost Areas 23-8
Drying of Wet Soils 23-9
Pick-and-Click Tests 23-10
Wet-Dry and Freeze-Thaw Tests 23-10
Modified Mix Design for Sandy Soils 23-12

Appendix A METRIC CONVERSION CHART A-1

Appendix B UNIFIED SOIL CLASSIFICATION SYSTEM B-1

Appendix C RAPID SOILS ANALYSIS KIT C-1

Appendix D UNITED STATES MARINE CORPS SOIL TEST SET D-1

GLOSSARY Glossary-1

REFERENCES References-1

INDEX Index-1

Figures

Figure 1-1. Shapes of soil grains 1-4
Figure 1-2. Soil gradation 1-4
Figure 1-3. Suggested procedures for hasty field identification 1-13
Figure 1-4. Roll or thread test 1-19
Figure 1-5. Wet-shaking test 1-19
Figure 1-6. Breaking or dry-strength test 1-20
Figure 1-7. Ribbon test 1-22
Figure 2-1. Sample DD Form 2464 2-8
Figure 2-2. Obtaining individual bag samples 2-9
Figure 2-3. Taking a composite sample with an exposed face 2-10
Figure 2-4. Labeling bag samples 2-11
Figure 2-5. Sealing a container to retain a sample's moisture content 2-11
Figure 2-6. Taking a chunk sample from a level surface 2-12
Figure 2-7. Taking a chunk sample from a vertical face 2-13
Figure 2-8. Applying paraffin to seal a chunk sample 2-13
Figure 2-9. Wrapping a weak chunk sample before final sealing 2-13

Figure 2-10. Dipping a chunk sample into melted paraffin 2-13
Figure 2-11. Packing a chunk sample for transportation or shipment to laboratory..... 2-14
Figure 2-12. Soil trafficability sampler 2-14
Figure 2-13. Section through CBR mold 2-16
Figure 2-14. Trench excavated around cylinder 2-16
Figure 2-15. Load bar used to drive cylinder 2-16
Figure 2-16. Cylinder in position before cutting sample 2-17
Figure 2-17. Cutting off cylindrical sample 2-17
Figure 2-18. Sealing sample with paraffin after trimming 2-18
Figure 2-19. Protecting a sample in a CBR mold 2-18
Figure 2-20. Samples more than 100 pounds 2-19
Figure 2-21. Mixing a sample 25 to 100 pounds 2-19
Figure 2-22. Mixing a sample less than 25 pounds 2-20
Figure 2-23. Typical soil profile 2-21
Figure 2-24. Soil-profile scales 2-22
Figure 3-1. Sample DD Form 1205 3-2
Figure 3-2. Calcium carbide pressure moisture tester calibration chart for use with 1 1/4-inch steel balls 3-8
Figure 4-1. Apparatus for determining specific gravity of soils 4-2
Figure 4-2. Sample DD Form 1208 4-3
Figure 4-3. Calibration curve for a volumetric flask 4-4
Figure 5-1. Sieve analysis equipment 5-2
Figure 5-2. Sample DD Form 1206 5-4
Figure 5-3. Hand-operated sieve shaker 5-6
Figure 5-4. Testing sieves stacked large to small 5-6
Figure 5-5. Sample DD Form 1207 5-8
Figure 5-6. Sample DD Form 1794 5-11
Figure 6-1. Equipment for the LL and PL tests 6-2
Figure 6-2. LL device with grooving tools 6-3
Figure 6-3. Sample DD Form 1209 6-5
Figure 6-4. Leveling sample in the cup 6-6
Figure 6-5. Holding cup and grooving tool 6-6
Figure 6-6. Cutting groove with spatula in sandy soil 6-6
Figure 6-7. Soil coming into contact 6-7
Figure 6-8. Essential equipment for a fall cone test 6-8
Figure 6-9. British fall cone calibration tolerances 6-10
Figure 6-10. British fall cone data plot examples 6-11
Figure 6-11. Rolling a soil specimen 6-12
Figure 6-12. Rolled threads, crumbled and uncrumbled 6-13
Figure 6-13. PL device components 6-13
Figure 6-14. Operation of PL device 6-14

Figure 6-15. USCS plasticity chart 6-15
Figure 7-1. Sample DD Form 1210 7-5
Figure 7-2. Soil compaction in a mold 7-6
Figure 7-3. Trimming the compacted soil sample 7-7
Figure 7-4. Sample DD Form 1211 7-9
Figure 7-5. Effect of water on density 7-11
Figure 7-6. Effect of different compactive effort on density 7-12
Figure 7-7. Effect of different soil types on density 7-13
Figure 8-1. Sand displacement method equipment 8-3
Figure 8-2. Sample DD Form 1215 8-4
Figure 8-3. Steel shot volume replacement equipment with traveling case 8-7
Figure 8-4. Schematic drawing of the graduated cylinder filled with balls up to the 1000 milliliter mark 8-8
Figure 8-5. Striking off excess steel shot 8-9
Figure 8-6. Steel shot remaining after completing the test (after the second pour) 8-9
Figure 8-7. Rinsing steel shot in the No. 8 sieve 8-11
Figure 8-8. Drive cylinder components 8-12
Figure 8-9. Preparing surface for drive cylinder test 8-12
Figure 8-10. Inserting drive cylinder into soil 8-13
Figure 8-11. Removing the drive cylinder sample 8-13
Figure 8-12. Trimmed sample ready for weighing 8-14
Figure 9-1. Laboratory CBR test-set apparatus 9-3
Figure 9-2. Sample DD Form 1212 9-4
Figure 9-3. Soaking the CBR sample 9-8
Figure 9-4. Assembled CBR test penetration apparatus 9-8
Figure 9-5. Sample DD Form 2463 9-12
Figure 9-6. CBR design method density and moisture requirements 9-19
Figure 9-7. Plotted results for a free-draining soil, borrow pit A 9-24
Figure 9-8. Dual-mass DCP kit 9-25
Figure 9-9. Assembled dual-mass DCP with vertical scale and disposable cone and adapter 9-26
Figure 9-10. Excel spreadsheet used for DCP analysis 9-28
Figure 9-11. Example of penetration measurement showing a penetration of 150 millimeters 9-29
Figure 9-12. Correction of load deformation curve 9-32
Figure 9-13. Plate bearing test equipment and set-up 9-34
Figure 9-14. Chart for correction of uncorrected modulus of soil reaction for bending of the plate 9-36
Figure 9-15. PFWD with data acquisition device 9-38
Figure 9-16. Operation of the PFWD, with laptop data acquisition 9-39
Figure 10-1. Sample schedule for soils laboratory testing 10-2
Figure 11-1. Recovery and refinement of petroleum asphalt 11-3

Figure 11-2. Typical viscosity values of ACs, emulsions, and cutbacks 11-5
Figure 12-1. Sampling liquid bituminous materials from a tank car or storage tank 12-2
Figure 13-1. Identification of unknown bituminous materials 13-2
Figure 14-1. Sample DD Form 1216 14-2
Figure 15-1. Dunagan apparatus..... 15-2
Figure 15-2. Calibrated-flask apparatus for determining apparent specific gravity of fine aggregate 15-3
Figure 15-3. Manipulation of calibrated flask to remove air 15-4
Figure 16-1. Compaction of bituminous trial mix specimens..... 16-4
Figure 16-2. Weighing compacted bituminous specimen in water..... 16-4
Figure 16-3. Sample DD Form 1218 16-5
Figure 16-4. Sample DD Form 1219 16-8
Figure 16-5. Superpave gyratory compactor..... 16-10
Figure 16-6. Sample DD Form 1217 16-11
Figure 16-7. Sample DD Form 1207 16-12
Figure 17-1. Bituminous hot-mix batch plant..... 17-1
Figure 17-2. Drum mix plant..... 17-1
Figure 17-3. Probable causes of paving-mixture deficiencies detected at plant..... 17-4
Figure 17-4. Probable causes of imperfections encountered in laying pavement 17-4
Figure 17-5. Sample DD Form 1793 17-6
Figure 20-1. Sampling concrete from a truck mixer 20-1
Figure 20-2. Measuring the slump of fresh concrete..... 20-3
Figure 20-3. Apparatus for flexural-strength test..... 20-9
Figure 20-4. Hand-operated compressions tester (shown testing a concrete cylinder)20-11
Figure 20-5. Important specimen-condition features..... 20-14
Figure 20-6. Types of breaks..... 20-18
Figure 20-7. PSPA instruments..... 20-18
Figure 23-1. Gradation triangle for soil classification 23-4
Figure 23-2. Relation of cement content by weight to cement content by volume..... 23-13
Figure 23-3. Average maximum densities of the -4 fraction of soil-cement mixtures . 23-14
Figure 23-4. Indicated cement contents of soil-cement mixtures not containing material retained on the No. 4 sieve 23-14
Figure 23-5. Minimum 7-day compressive strengths required for soil-cement not containing material retained on the No. 4 sieve..... 23-15
Figure 23-6. Average maximum densities of soil-cement mixtures containing material retained on the No. 4 sieve 23-15
Figure 23-7. Indicated cement contents of soil-cement mixtures containing material retained on the No. 4 sieve 23-16
Figure 23-8. Minimum 7-day compressive strengths required for soil-cement mixtures containing material retained on the No. 4 sieve..... 23-17
Figure B-1. USCS procedures..... B-5
Figure B-2. MDD and OMC estimates for standard proctor compaction effort B-13
Figure B-3. MDD and OMC estimates for modified proctor compaction effort..... B-13

Figure C-1. RSAK contents C-1
Figure C-2. RSAK packaged contents C-1
Figure C-3. RSAK software output C-2
Figure D-1. Case 1—laboratory equipment D-1
Figure D-2. Case 2—DCP D-2
Figure D-3. Case 3—speedy moisture tester D-3

Tables

Table 1-1. Grain size groups 1-3
Table 1-2. Soil groups and symbols 1-7
Table 1-3. Sample soil description information 1-12
Table 1-4. Suggested steps for hasty field identification 1-13
Table 1-5. Sedimentation test 1-17
Table 1-6. Dry-ball strength test 1-21
Table 2-1. Methods of underground exploration and sampling 2-6
Table 3-1. Recommended minimum test specimen for reporting water content 3-3
Table 3-2. Calcium carbide pressure moisture tester conversion chart for use when not using 1 1/4-inch steel balls 3-9
Table 3-3. Determining the moisture content of the soil 3-9
Table 4-1. Relative density of water and the correction factor at various temperatures 4-6
Table 5-1. Representative soil samples for grain size analysis 5-3
Table 5-2. Dispersing agents 5-12
Table 5-3. Values of K for use in Stokes' equation for computing particle diameter 5-14
Table 5-4. Values of effective depth for hydrometer analysis 5-15
Table 5-5. Frost-susceptibility groups for typical soil types 5-16
Table 9-1. Summary of compaction requirements 9-11
Table 9-2. Summary of remolded CBR laboratory test programs 9-17
Table 9-3. Swell potential 9-21
Table 9-4. Determining CBR values for moisture-content percentages for swelling soils 9-22
Table 9-5. Determining CBR values for potential moisture content ranges for swelling soils 9-22
Table 9-6. Tabulated correlation of CBR versus DCP index 9-29
Table 11-1. AC performance, viscosity, and penetration grade 11-4
Table 11-2. Asphalt cutback composition (expressed in percent of total volume) 11-4
Table 11-3. Characteristics of bituminous materials 11-5
Table 11-4. Typical uses of bituminous materials 11-8
Table 13-1. Approximate comparison between AC grades 13-3
Table 16-1. Marshall test specifications and determination of optimum asphalt content 16-7
Table 20-1. Recommended slumps for various types of construction 20-4

Table 20-2. Requirements for unbonded caps 20-15
Table 23-1. Estimated time required for test procedures 23-3
Table 23-2. Soil stabilizers for different soil types 23-4
Table 23-3. Suggested stabilizer amounts for expedient stabilization 23-5
Table A-1. Metric conversion chart A-1
Table B-1. Soil particle size ranges B-2
Table B-2. Characteristics of soil groups pertaining to embankments and foundations . B-3
Table B-3. Terzaghi classification B-11
Table B-4. Characteristics of soil groups pertaining to roads and airfields B-19
Table C-1. RSAK contents C-3
Table D-1. Case 1 contents D-2
Table D-2. Case 2 contents D-3
Table D-3. Case 3 contents D-4

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Preface

Materials Testing 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 soils, bituminous paving mixtures, and concrete and to achieve 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 (bitumens, cements, lime, fly ash, chemical modifiers). It 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. This manual explains methods for designing bituminous paving mixtures and stabilizing soil. It also gives the procedures and tests required to control the manufacturing of these mixtures. It describes the tools and equipment needed for performing tests and contains general instructions for the care, calibration, and use of test equipment.

This manual is adopted for use by the United States Marine Corps (USMC), United States Navy (USN), United States Air Force (USAF) personnel. Certain tests and procedures prescribed differ in principle or method and are more detailed than counterpart tests that are currently required by the U.S. Navy for new construction at Navy installations (including those in forward areas). The USMC engineer units perform field identification testing only. The USMC does not possess the tools or facilities required to perform the more deliberate laboratory tests described in parts of this publication.

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, and the Portland Cement Association (PCA). The tests 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 considerations associated with arctic construction are in TM 5-349.

The principal audience for *Materials Testing* is construction trade engineers, but all engineer construction trade supervisors will benefit from reading it. Trainers, combat developers, and educators throughout the services will also use this manual.

Commanders, staffs, and subordinates ensure that decisions and actions comply with applicable United States (U.S.), international and, in some cases, host nation (HN) laws and regulations. Commanders at all levels ensure that Soldiers operate according to the law of war and the rules of engagement (see FM 27-10).

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

Appendix A contains a metric conversion chart for the measurements used in this manual. For a complete listing of preferred metric units for general use, see Federal Standard 376B.

This publication applies to Active Army, Army National Guard/Army National Guard of the United States, the U.S. Army Reserve, Marine Corps, and Marine Corps Reserve unless otherwise stated.

The proponent of *Materials Testing* is the United States Army Engineer School (USAES). The preparing agency is the Maneuver Support Center of Excellence (MSCoE) Capabilities Development and Integration Directorate; Concepts, Organizations, and Doctrine Development Division; Doctrine Branch. Send comments and recommendations on DA Form 2028 *(Recommended Changes to Publications and Blank Forms)* to Commander, U.S. Army Maneuver Support Center of Excellence, ATTN: ATZT-CDC, 14000 MSCoE Loop, Suite 270, Fort Leonard Wood, Missouri 65473-8929; e-mail the DA Form 2028 to <usarmy.leonardwood.mscoe.mbx.cdidcodddengdoc@mail.mil>; or submit an electronic DA Form 2028.

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Navy readers of this publication are encouraged to submit suggestions and changes to the Navy Warfare Development Command, ATTN: Doctrine, 1528 Piersey Street, Building O-27, Norfolk, VA 23511-2723.

Air Force readers of the publication are encouraged to submit suggestions and changes to the Air Force Civil Engineer Center–East, 139 Barnes Drive, Suite 1, Tyndall Air Force Base, FL 32403.

PART ONE

Soils

Area soil is important in structure location. 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.

Chapter 1

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 with the grain size and the voids between them, particularly what the voids enclose (water, air, organic materials).

FORMATION

1-1. 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 produce many different soils. For engineering purposes, soils are evaluated by the following basic physical properties:

- Grain size gradation.
- Bearing capacity as reflected by soil density.
- Grain shapes.

1-2. An engineer extends soil evaluation by considering the effect of water action on the soils. With a complete evaluation, an engineer can determine if the soil is adequate for the project.

ROCKS

1-3. Soil forms when rocks that are exposed to the atmosphere disintegrate and decompose as a result of 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.

1-4. Geologists classify rocks into the following basic groups:

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

STRATA

1-5. At a particular location, there 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 construction.

Note. A vertical cross section through the earth, with the depths and types of soil indicated, is called a soil profile.

PHYSICAL PROPERTIES

1-6. Physical properties of soil help determine the engineering characteristics. The discussion of the physical properties of soil focuses on the soil grains themselves. The following properties are the basis for the soil classification system used in engineering identification of soil types.

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

1-7. Physical characteristics of soil grains include size and shape. The proportions of different-size grains determine soil gradation. Density or compactness refers to the closeness of packing of soil grains. 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. The specific gravity of soil grains 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 SIZE

1-8. Soils are divided into groups based on grain sizes in the soil mass. A 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 grains to fall through and retains the larger grains 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 grain 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 (table 1-1) that are designated by the United Soil Classification System (USCS) are cobbles, gravels, sands, and fines (silt or clay) (see appendix B for more information).

Table 1-1. Grain size groups

Size Group	*Sieve Passing*	*Size Retained On*
Boulders	No maximum size[1]	10 in
Cobbles	10 in	3 in
Gravels	3 in	No. 4
Sands	No. 4	No. 200
Fines (clay or silt)	No. 200	No minimum size
[1]In military engineering, the maximum size of boulders is accepted as 40 in, based on the maximum jaw opening of a rock-crushing unit.		
Legend:		
in	inches	
No.	number	

GRAIN SHAPE

1-9. Grain shape influences soil strength and stability. The following general particle shapes (figure 1-1, page 1-4) are recognized—

- **Bulky.** 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 dived into four subdivisions that depend on the amount of weathering that has occurred. Cobbles, gravel, sand, and silt usually fall into the bulky group. These groups are discussed in the order of desirability for construction as follows:
  - **Angular.** Angular 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.** Subangular particles have been weathered to a point that the sharper points and ridges of their original angular shape have been worn off. These particles are still very irregular in shape, with some flat surfaces, and are excellent for construction.
  - **Subrounded.** 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.** 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 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 that is 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, while fine-grained particles with platy or bulky shapes are not.

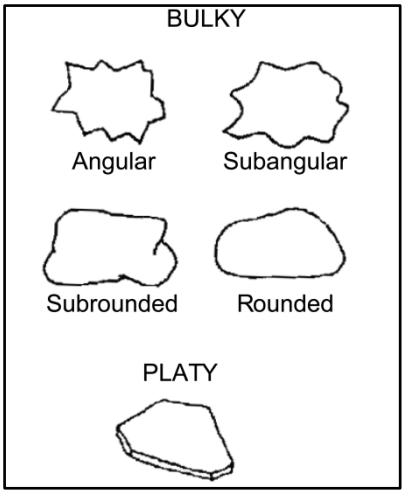


Figure 1-1. Shapes of soil grains

GRADATION

1-10. Gradation (figure 1-2) describes the distribution of the different size groups within a soil sample. There are—

- **Well-graded soils.** Well-graded soils must have a good range of all representative particle sizes between the largest and the smallest. All sizes must be represented, and no one size is overabundant or missing.
- **Poorly graded soils.** Poorly graded soils can be classified as uniformly graded or gap graded. A uniformly graded soil consists primarily of particles of nearly the same size. A gap-graded soil contains large and small particles, but the gradation continuity is broken by the absence of some particle sizes.

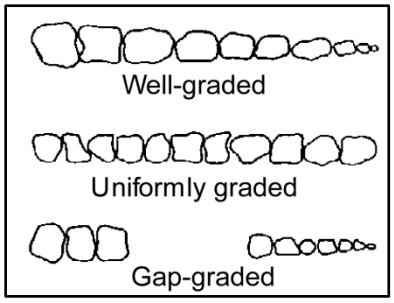


Figure 1-2. Soil gradation

DENSITY

1-11. The structure of the aggregate of soil particles may be dense (closely packed) or loose (lacking compactness). A dense structure provides the 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 are usually 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

1-12. 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. The following are ways of determining and expressing specific gravity:

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

1-13. The specific gravity of solids is the test most widely used when testing soils. The apparent and bulk specific-gravity tests are used in testing fine and coarse aggregates (see chapter 4).

MOISTURE

1-14. 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 by gravitational pull, capillary action, or hygroscopic action. This moisture influences various soils differently and usually has an effect on the behavior of fine-grained soils. The fine grains and their small voids slow water movement and tend to hold the water by surface tension.

Note. Moisture content 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 solid mineral grains (weight of dry soil) as a percentage.

1-15. 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 the 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. Moisture can enter material as or through—

- **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). The smaller the tube, the stronger the capillary action, water rises higher in 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.** 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 chemical and physical structures and relative surface area of the soil particle. 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 are also 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, 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. 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.** Plasticity is the ability of the soil to deform without cracking or breaking. Soils in which the adsorbed films are relatively thick compared to particle size (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 (clean sands, 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.
- **Cohesiveness.** Not all plastic soils are cohesive. Soil is considered cohesive if 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.

CONSISTENCY

1-16. 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.

1-17. 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 moisture content as follows:

- **LL.** The LL is the moisture content at an arbitrary limit between the liquid and plastic states of consistency of a soil. 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.
- **PL.** 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.
- **Plasticity index (PI).** The PI is used to determine whether soil is cohesive. The PI is the numerical difference in moisture contents between the two limits. It defines the range of moisture content for a soil in a plastic state. 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.
- **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

1-18. Soil having a high content of organic material is described as organic soil. It is usually very compressible and has poor load-maintaining properties.

CHARACTERISTICS

1-19. Soil characteristics are a measure of soil 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 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.

CLASSIFICATION

1-20. Soil seldom exists separately as sand, gravel, or any other single component in nature. It is usually a mixture with varying proportions of different-size particles. Each component contributes to the mixture's characteristics. Once the principal characteristics are identified within this system (by visual examination and laboratory tests), a descriptive name and letter symbol are assigned to the soil.

1-21. 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).

1-22. To start the classification process, become familiar with the USCS (see appendix B) and the classification sheet used in the classification process. The classification sheet shows the major divisions of classification and the symbols that distinguish the individual soil types.

CATEGORIES

1-23. In the USCS, soils are divided into the following major soil divisions:

- **Coarse-grained soils.** Coarse-grained soils are those having 50 percent or less material passing the Number (No.) 200 sieve.
- **Fine-grained soils.** Fine-grained soils are those having more than 50 percent passing the No. 200 sieve.
- **Highly organic soils.** Highly organic soils can generally be identified by visual examination.

1-24. The USCS recognizes 25 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 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 1-2). These symbols are also used in combination to describe borderline soils.

Table 1-2. Soil groups and symbols

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

Legend:
LL liquid limit

Coarse-Grained Soils

1-25. Coarse-grained soils are subdivided into the following divisions:

- Gravels and gravelly soils.
- Sands and sandy soils.

1-26. 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.** Less than 5 percent nonplastic fines soils may fall into the groups GW, GP, SW, or SP, where the shape of the grain size distribution curve determines the second letter of the symbol.
  - **GW and SW.** 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.** 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. Symbols M and C are 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. A C is used to indicate that the binder soil is predominantly clayey in character.
  - **GM and SM.** The GM and SM groups comprise silty gravels and silty sands with fines (more than 12 percent passing the No. 200 sieve) having low or no plasticity. For 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.** 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 these groups, the LL and PL will plot above the A line on the plasticity chart with a PI of more than 7. (See chapter 6 for more information on plasticity.)

- **Borderline soils.** Coarse-grained soils of which 5 to 12 percent of material passes the No. 200 sieve are classed as borderline and are given a dual symbol (such as 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 (such as 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.

1-27. It is best to use only a double symbol in these cases, selecting the two that are believed to be 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. 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 LL and PI of a soil were plotted in the shaded portion of the plasticity chart (such as an LL of 20 and a PI of 5), the soil would be designated SW-SC or SW-SM, depending on the engineer's judgment from the standpoint of the climatic region.

Fine-Grained Soils

1-28. Fine-grained soils are subdivided into the following divisions:

- Silts.
- Clays.

1-29. Fine-grained soils are not classified on the basis of grain size distribution, but according to plasticity and consistency. Like coarse-grained soils, laboratory classification criteria are based on the relationship between the LL and PI designated in the plasticity chart. The two major groupings of fine-grained soils are the L groups, whose LL is less than 50, and the H groups, whose LL is greater than or equal to 50. The symbols L and H have general meanings of low and high LL. Fine-grained soils are further divided by their position above or below the A line of the plasticity chart. Fine-grained soils are grouped as follows:

- **ML and MH.** 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.** In the CL and CH 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, gumbo clays of the southern United States, volcanic clays, and bentonite. Glacial clays of the northern United States cover a wide area in the CL and CH groups.
- **OL and OH.** 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 particles 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 (such as 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

1-30. A special classification is reserved for highly organic soils (such as peat), which have many characteristics undesirable for use as foundation and construction materials. These types of soils can be readily identified in the field by their distinctive color, odor, spongy feel, and fibrous textures. Leaves, grass, branches, or other fibrous vegetable matter particles are common components of these soils.

1-31. Laboratory tests are available to determine the percentage of organic matter in a soil, including the ash test given in ASTM D2974. However, specialized laboratory equipment that is not easily fielded in a military setting is required to perform organics testing. In routine military operations, this type of classification may not be necessary. If it is necessary, specialized laboratory facilities can provide testing.

SOIL GROUPS

1-32. 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 soils) 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.

1-33. Methods of general identification of soils are discussed in the following paragraphs. 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

1-34. 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.

1-35. 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.

1-36. 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 grain size and character of the soil. 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.

1-37. An approximate identification of a coarse-grained soil is made by observing—

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

1-38. 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.

1-39. 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 that is detectable by color or odor.

TEST EQUIPMENT

1-40. Field tests may be performed with little or no equipment other than a small amount of water. However, the accuracy and uniformity of results will be increased greatly by properly using the following equipment available in nearly all engineer units:

- No. 4 and No. 40 sieves (a No. 200 sieve is useful, but not required).
- A digging instrument (a pick, shovel, 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

1-41. The USCS considers the following soil properties:

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

1-42. 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

1-43. 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 (table 1-3).

Table 1-3. Sample soil description information

Color	Dark brown to white
Grain size	Course-grained (maximum particle size 3 in; estimated 60% gravel, 36% sand, and 4% fines passing through the No. 200 sieve)
Gradation	Poorly graded (gap-graded, insufficient fine gravel)
Grain shape	Subrounded to rounded gravel particles
Plasticity	Nonplastic
Predominant soil type	Predominantly gravel
Secondary components	Considerable sand and a small amount of nonplastic fines (silt)
Identification	GP (identification symbol)
Dry strength	Slightly calcareous, no dry strength, and dense in the undisturbed state
Legend:	
in	inch
No.	number

TESTS

1-44. The following tests produce observations that pertain to the USCS and permit field identification and 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 performed to establish the best identification. Figure 1-3 and table 1-4 give the suggested sequence of tests for identifying and classifying a soil sample using the hasty field procedures.

VISUAL TEST

1-45. A visual test should result in finding the color, grain sizes, grain shapes of the coarse-grained portion, some gradation properties, and some undisturbed soil properties. Determine the color, grain size, and grain shape of the material and, if possible, estimate the grain size distribution by visual examination. The following paragraphs provide methods and information concerning identification of these properties.

Color

1-46. 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 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.

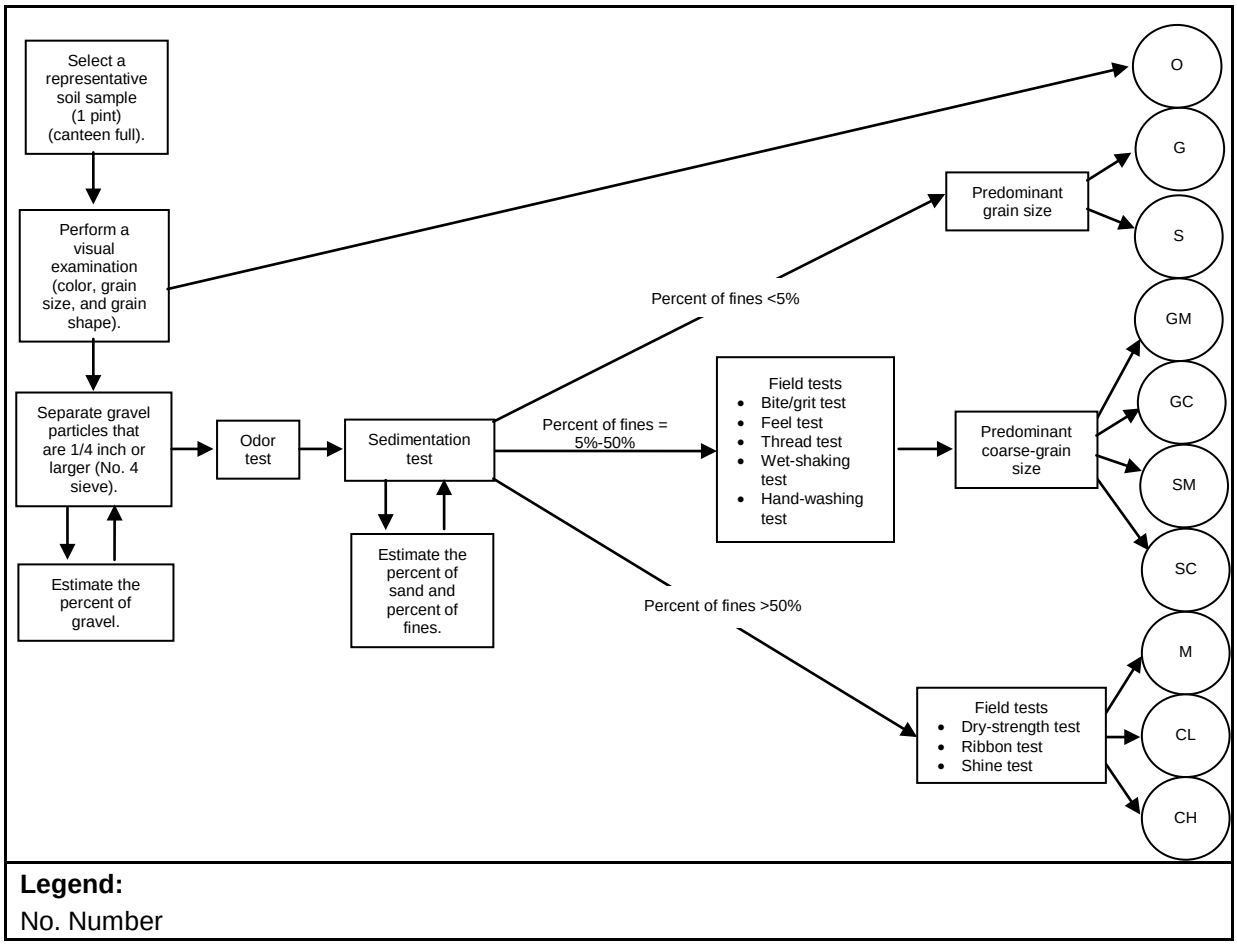


Figure 1-3. Suggested procedures for hasty field identification

Table 1-4. Suggested steps for hasty field identification

Visual Test
• Determine the color.
• Determine the grain size.
• Determine the grain shape.
• Determine the contents (leaves, grass).
• Remove particles larger than 1/4 in (No. 4 sieve) from the sample.
• Estimate the percent of gravel.
Odor Test
• Heat the sample (less gravel) with a match or open flame.
Note. If the odor becomes musty or foul smelling, there is a strong indication that organic material is present.

Table 1-4. Suggested steps for hasty field identification (continued)

Sedimentation Test
• Place approximately 1 in of the -4 material (less gravel) in a glass jar.
• Mark the depth of the sample with a grease pencil.
• Cover the sample with 5 in of water with at least 1 in of air space at the top of the jar.
• Cover and shake the mixture for 3 to 4 min.
• Place the jar on a flat surface and allow sand particles to settle for 30 sec.
• Compare the settled material after 30 sec to the grease pencil mark, and estimate the percent that has settled.
• Determine the percent of sand for the overall sample: %S = (% settled) x (100% - %G).
• Determine the percent of fines for the overall sample: %F = 100% - (%S + %G).
Bite or Grit Test
• Place a pinch of -200-sieved material between the teeth and bite.
▪ If the sample feels gritty, the sample is M.
▪ If the sample feels floury, the sample is C.
Feel Test
• Rub a portion of dry -200-sieved soil over a sensitive part of the skin (inside of the wrist).
▪ If it feels harsh and irritating, the sample is M.
▪ If it feels smooth and floury, the sample is C.
Roll or Thread Test
• Form a ball of moist soil to marble size.
• Attempt to roll the ball into a 1/8-in-diameter thread.
▪ If a thread is easily formed, it is C.
▪ If a thread cannot be formed, it is M.
Wet-Shaking Test
• Place a pat of moist (not sticky) soil in the palm of one hand (the volume is about 1/2 in^3).
• Shake your hand vigorously, and strike it against the other hand.
• Observe how rapidly water rises to the surface.
▪ If it rises fast, it is M.
▪ If there is no reaction, it is C.
• Form a moist pat 2 in in diameter by 1/2 in thick.
• Allow it to dry with low heat.
• Place the dry pat between your thumb and index finger only and attempt to break it.
▪ If breakage is easy, it is a silt.
▪ If breakage is difficult, it is CL.
▪ If breakage is impossible, it is CH.
Ribbon Test
• Form a cylinder of moist soil, approximately cigar shape and size.
• 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 it is over 8 in, it is CH.
▪ If it is 3 to 8 in, it is CL.
▪ If it is less than 3 in, it is M.

Table 1-4. Suggested steps for hasty field identification (continued)

Shine Test
• Smooth the surface using a knife blade or thumbnail of a pat of slightly moist soil.
▪ If the surface becomes shiny and lighter in texture, the sample is CH.
▪ If the surface remains dull, the sample is CL.
▪ If the surface remains very dull or granular, the sample is silt or sand.
Note. These tests are conducted only with material that passes the No. 40 sieve.

Legend:

in	inch
in^3	cubic inch
min	minute
No.	number
sec	second

Grain Size

1-47. 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 millimeters 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

1-48. 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

1-49. 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. The overabundance or lack of any size means the material is poorly graded.
- **Step 5.** Estimate the percentage of the fine-grained portion of the coarse-grained soil.
- **Step 6.** Assign a soil classification. If the—
  - Fine-grained portion is less than or equal to 5 percent of the total soil, the soil may be classified as GW, GP, SW, or SP, depending on the other information noted above.

Note. For visual examination, use 5 percent as the limiting percentage.

- Fine-grained portion exceeds 12 percent, the soil is M or C and requires further testing to identify its symbol (GM, GC, SM, SC).

Note. For visual examination, use 15 percent as the limiting percentage.

- Fine-grained portions are between 5 and 12 percent (nonplastic fines or fines not interfering with free drainage), they are borderline and require double symbols GW-GM, SW-SM, GP-SM, SP-SM. Similarly, plastic fines have GW-GC, SW-SC, GP-GC or SP-SC symbols.

Note. For visual examination, use an estimated 10 percent to receive a dual symbol.

- Fine-grained soils (greater than 50 percent) (M, C, or O) other tests are required 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 clay or silt settles.

Note. Percentages are based on laboratory weight and the volume should only be approximated if scales are not available in the field.

Undisturbed Soil Properties

1-50. 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 moisture content influences the in-place characteristics. This condition should be recognized and reported with the undisturbed soil properties.

ODOR TEST

1-51. 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

1-52. 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.

1-53. 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 of air or soil 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) (table 1-5).

Table 1-5. Sedimentation test

Approximate Time of Settlement Through 5 Inches of Water	*Grain Diameter*	*Differentiates*
2 sec	0.400 mm	Coarse sand, fine sand
30 sec	0.072 mm	Sand, fines
10 min	0.030 mm	Coarse silt, fine sand
1 hr	0.010 mm	Silt, clay
Legend:		
hr	hour	
mm	millimeter	
sec	second	
min	minute	

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.

1-54. Once this determination is complete, estimating the amount of sand and fines of the overall sample is easy, providing the approximate percentage of gravel was obtained from the visual examination. Use the following formula to determine the percent of sand for the overall sample:

$$\%S = \%\ of\ sand\ in\ jar\ \times\ (100\% - \%G)$$

where—
%S = percent of sand
%G = percent of gravel

Example. $\%S\ = 50\% \times (100\% - 40\%)$

$$\%S = 30\%$$

1-55. Use the following to determine the percent of fines:

$$\%F = 100\% - (\%G + \%S)$$

where—
%F = percent of fines
%S = percent of sand
%G = percent of gravel

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

$$\%F = 30$$

1-56. The most important use of the sedimentation test is to differentiate the coarse (0.072 millimeters) fraction from the fine fraction of a soil. Since all particles of soil larger than 0.072 millimeters will have settled to the bottom of the cup or jar 30 seconds after the mixture agitated, it follows that the particles 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.

1-57. 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

1-58. 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.

Note. This test is ideally performed on material passing the No. 200 sieve to avoid biting into sand and providing separation between silty and clayey fines.

1-59. 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

1-60. 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.

Note. This test is ideally performed on material passing the No. 200 sieve to avoid sand particles giving an erroneous reaction.

Consistency

1-61. 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

1-62. This term is applied to the fine-grained portion of the soil 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 with difficulty, but feel smooth and gritless like flour.

ROLL OR THREAD TEST

1-63. This test is performed only on 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.

1-64. 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 (figure 1-4).

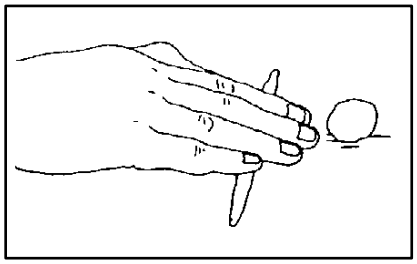


Figure 1-4. Roll or thread test

1-65. 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 degree of plasticity of the soil. Materials that cannot be rolled in this manner are nonplastic or have a very low plasticity.

1-66. 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 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 called peat.
- If the soil cannot be rolled into a thread at any moisture content, it is nonplastic soil (ML or MH).

1-67. 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

1-68. Perform the wet-shaking test (figure 1-5) only on 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).

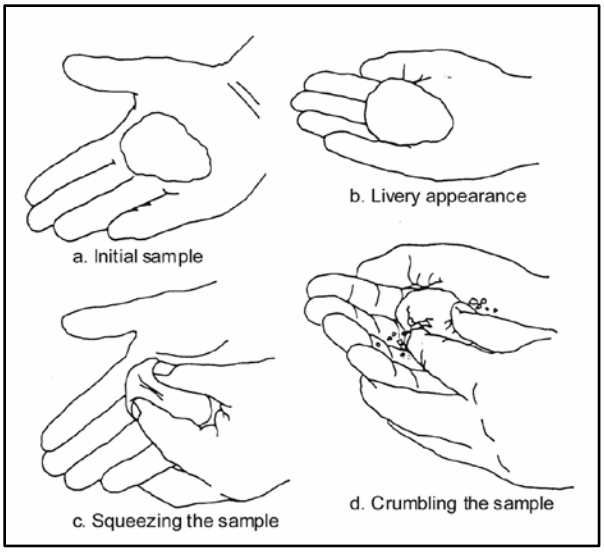


Figure 1-5. Wet-shaking test

1-69. 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. Squeeze the sample between the thumb and forefinger of the other hand and the surface water will quickly disappear. The surface becomes dull and the material becomes firm, resisting deformation. Cracks occur as pressure is continued, with the sample finally crumbling like a brittle material.

1-70. The vibration caused by shaking the soil sample tends to reorient the soil grains, decrease the voids, and force water within these voids to come to 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.

1-71. 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 slow 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 that 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

1-72. These tests are performed only on material passing the No. 40 sieve. They are used to measure the cohesive and plastic characteristics of the soil. The tests distinguish between the clayey and silty soils. Drying times for the ball test tend to be slightly shorter than those for the pat test.

Pat Test

1-73. 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 they 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.

1-74. Allow the pat to dry completely, grasp it between the thumbs and forefingers of both hands and attempt to break it (figure 1-6). If the pat breaks, powder it by rubbing it between the thumb and forefinger of one hand.

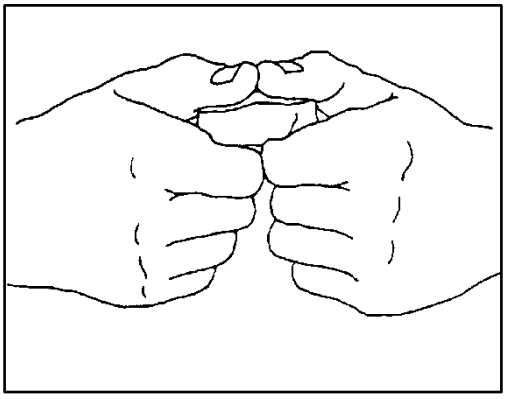


Figure 1-6. Breaking or dry-strength test

1-75. The results of this test indicate the following:

- If the pat cannot be broken or 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 a nonplastic soil (ML or MH) or (OL or OH).

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 true dry strength of the soil. It is important to distinguish between a break along such a crack and a clean, fresh break that indicates the true dry strength of the soil.

Ball Test

1-76. Separate the selected soil sample on the No. 40 sieve, and prepare enough material to mold into a ball of about 1 inch in diameter by molding it in a wet, plastic state. Mold the material until it has the consistency of putty, adding water if necessary. From the molded material, make at least three test specimens, each a ball of about 1/2 inch in diameter. Allow the balls to dry in air, sun, or by artificial means as long as the temperature does not exceed 60ºC. If the test specimen contains naturally dry lumps, those that are about 1/2 inch in diameter may be used in place of the molded balls. However, the molded balls will typically produce higher strengths that natural samples. Test the strength of the dry balls or lumps by crushing them between the fingers until they break. From this test, strength is determined to be none, low, medium, high, or very high (table 1-6).

Table 1-6. Dry-ball strength test

Dry Strength	*Criteria*	*Soil Type*
None	The dry specimen crumbles into powder with the mere pressure of the handling.	Nonplastic ML ,MH, OL, or OH
Low	The dry specimen crumbles into powder with some finger pressure.	Slightly plastic ML, MH, or CL
Medium	The dry specimen breaks into pieces or crumbles with considerable finger pressure.	Medium plastic CL
High	The dry specimen cannot be broken with finger pressure. The specimen will break into pieces between the thumb and a hard surface.	Highly plastic CL
Very high	The dry specimen cannot be broken between the thumb and a hard surface.	Very highly plastic CH

RIBBON TEST

1-77. The ribbon test is 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.

1-78. 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 (figure 1-7, page 1-22). 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.

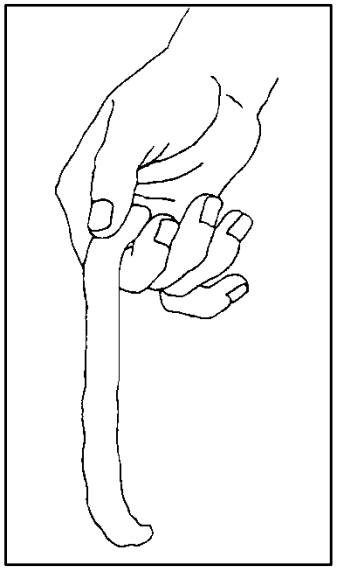


Figure 1-7. Ribbon test

1-79. The results of this test indicate the following:

- If the sample holds together for a length greater than 8 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 low plasticity (CL).
- If the soil cannot be ribboned, it is nonplastic (ML) or (MH).

SHINE TEST

1-80. The shine test is another means of determining soil plasticity. A slightly moist or dry piece of 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.

Chapter 2

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 project success.

SURVEY TYPES

2-1. A soil survey consists of gathering soil samples for examining, testing, and classifying soils and developing a soil profile.

HASTY SURVEY

2-2. A hasty survey is made under expedient conditions or when time is very limited. It is a 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.

2-3. A hasty survey should be preceded by a careful study of all available sources of information as conditions permits. 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 the natural banks of a stream, 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

2-4. 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 pits is taking soil samples 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.

RAPID SOILS ANALYSIS

2-5. The United States Army Corps of Engineers (USACE) and the USMC use specific tools to conduct rapid soils analysis.

Rapid Soils Analysis Kit

2-6. The USACE Engineer Research and Development Center (ERDC) has developed a rapid soils analysis kit (RSAK) (see appendix C and USACE report ERDC/GSL TR-08-3) that strikes a balance between a hasty and deliberate survey. This is a two-man-carry, portable soils kit that includes laboratory quality test instruments, allowing the user to measure moisture content, grain size distribution, plasticity,

and soil classification and determine the construction and strength properties of the soil within 1 hour of soil sampling. The RSAK was developed for contingency military construction operations that are conducted under compressed time frames.

2-7. The RSAK is recommended for use under conditions where limited time and resources prevent the user from conducting a deliberate survey and formal laboratory analysis as described in this manual. Test results from the kit contain errors exceeding those of ASTM test guidelines, but provide the user a best estimate of soil properties for identification and construction when no other alternative exists.

Soil Test Set

2-8. The USMC designed a soil test set (see appendix D) to be used by construction personnel and military forces for the purpose of testing soil for quality, moisture content, and compactness. Several small systems are employed to test all aspects of soil conditions in preparation for construction operations.

2-9. The components and systems of the soil test set are simple in design and operation. Power requirements consist of batteries, which are easily changed if necessary, to provide continuous testing in all fields of operation and in areas where electrical connectivity is impossible. Optional alternating current power adapters are included for some of the components in the event that power is available.

2-10. The soil test set is an invaluable tool for classifying soil types and the strength of soil on or near a project site. The kit contains three field-deployable cases that are equipped with the essential items to conduct a series of field identification tests and determine important characteristics of the soil. Case 1 contains equipment to complete the field identification, Case 2 contains a dynamic cone penetrometer (DCP) to test the California Bearing Ratio (CBR), and Case 3 contains a speedy moisture tester.

OBJECTIVE

2-11. 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

2-12. Information regarding the location, nature, and classification of soil layers is required for the 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.

SOIL CONDITION

2-13. Soil conditions (moisture content and density of a soil in its natural state) play an important part in design and construction. The moisture content of some soils in place may be so high 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

2-14. Drainage characteristics in surface and subsurface soils are controlled by a combination of factors (the void ratio, soil structure, stratification, soil temperature, depth to the water table, capillary rise height, 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 disturbed and undisturbed conditions.

GROUNDWATER AND BEDROCK

2-15. 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 can be 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.

INFORMATION SOURCES

2-16. 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 the general observation of road cuts, stream banks, eroded slopes, earth cellars, mineshafts, 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

2-17. Intelligence reports that include maps and studies of soil conditions are usually 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.

MAPS

2-18. 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.** 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.

Note. The map sheet often has a mineral resources section 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 (levees, sand dunes, beach ridges, 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 is usually classified according to its texture, color, structure, chemical and physical compositions, and morphology.
- **Digital maps.** In general, digital map data (classified and unclassified) is available from the National Geospatial-Intelligence Agency. This includes the digital, high-resolution data from satellites, which include—
  - Information on landscapes (color photography).
  - Landforms.
  - Physiography.
  - Man-made structures.
  - Geology.
  - Soil maps.

2-19. If a certain map or map data is required and cannot be obtained from sources below, contact the unit's supporting Topographic Unit who will send a request to the Theater unit responsible for the theater geospatial database. If the data requested is not in the National Geospatial-Intelligence Agency database, it can then be added.

LOCAL INHABITANTS

2-20. Local inhabitants (trained observers), contractors, engineers, 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. Local inhabitants can also provide information on the area's climate and topography.

AERIAL PHOTOGRAPHS

2-21. 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.** The landform or land configuration in different types of deposits is characteristic and can be identified on aerial photographs. For example, glacial forms (moraines, kames, eskers, terraces) are readily identifiable. In desert areas, characteristic dune shapes indicate areas covered by sands that are subject to movement by wind. In areas underlaid by flat-lying, soluble limestone, the aerial photograph typically shows sinkholes.
- **Slopes.** Prevailing ground slopes usually represent soil 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 cohesiveness of the soil. 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, cultivated and natural vegetation cover may be reliable indicators of soil type.
- **Agricultural land.** 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.
- **Soil strength.** Methods exist that allow the prediction of soil surface strength anywhere in the world based on regional soil types and climate. Guidance for use of the Soil Moisture Strength Prediction II model can be found in ERDC/CRREL SR-04-1. These tests will document the required data that must be gathered to estimate near surface strength for a given region on a soil map.

FIELD INVESTIGATIONS

2-22. A field investigation consists of sampling operations in the field.

SAMPLING METHODS

2-23. 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.

2-24. 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.

Test Pit

2-25. A test pit is an open excavation that is 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 natural condition of the soil and collecting undisturbed samples. The test pit is usually 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 EM 385-1-1.

Auger Borings

2-26. A hand auger is commonly used for digging borings. It is best suited to cohesive soils; however, it can be used on cohesionless soils above the water table if 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 2-1 shows methods of underground exploration and sampling in a condensed form.

Table 2-1. Methods of underground exploration and sampling

Method	*Materials Used*	*Method of Advancing Hole*	*Sampling Method*	*Value for Foundation Purposes*
Auger boring	Cohesive soils and cohesionless soils above groundwater 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.

SAMPLE PREPARATION

2-27. 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 pit investigations. Make soil tests on samples representing the major soil types in the area.

2-28. 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.

2-29. 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 cut areas, extend all holes 4 feet below the final subgrade elevation. In 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.

2-30. 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.

2-31. 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.

2-32. 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.

2-33. 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

2-34. The engineer in charge of the soil survey is responsible for properly surveying, numbering, and recording each auger boring, test pit, or other investigation. Coordinate references from a global positioning systems (GPS) should be included, if possible, for later geo-referencing on digital maps. Keep a log (a typical boring log is recorded on DD Form 2464 *[Report of Foundation and Borrow Investigation]* [figure 2-1, page 2-8]) 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.

REPRESENTATIVE SOIL SAMPLES

2-35. 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. Soil or aggregate samples may need to be taken from stockpiles (see chapter 4).

2-36. 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.

2-37. 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 and 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

2-38. 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 (their location in the pit, the pit location within the excavation site), in a project logbook.

Note. Ensure that digging is permitted at the testing site.

REPORT OF FOUNDATION AND BORROW INVESTIGATION

1. PROJECT		2. DATE
Engineer Center Expansion		6 Aug 14
3. JOB NUMBER	**4. TEST SITE**	**5. LOCATION**
16-P-T	North Corner	North Corner
6. TYPE OF EXPLORATION	**7. BORING NUMBER**	**8. GROUND ELEVATION**
Deliberate	5-C	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
0.75'	213.88'	1		OH	7.6	Dark brown and very plastic. Typical topsoil of the area.
5.0'	209.63'	2		SC	34	Light brown, coarse-grained sandy soil with plastic clay fines. Maximum particle size 1 3/4". High dry strength. Dense in undisturbed state.
6.0'	208.63'	3		CL	29	Brown, sticky clay. Very high plastic qualities.

SAMPLE

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

DD FORM 2464, DEC 1999 PREVIOUS EDITION IS OBSOLETE. Reset Adobe Professional 7.0

Figure 2-1. Sample DD Form 2464

Legend:	
Aug	August
CBR	California Bearing Ratio
DD	Department of Defense
Dec	December
elev	elevation
SGT	sergeant
SSG	staff sergeant
USCS	Unified Soil Classification System

Figure 2-1. Sample DD Form 2464 (continued)

Steps

2-39. Perform the following steps to obtain a representative soil sample (typically accomplished for borrow pits):

- **Step 1.** Locate the boreholes or test pit (the location of auger holes depends on the particular situation).
  - ***Step 1a.*** Draw a site sketch recording the borehole's location, elevation, azimuth, and distance from a benchmark or reference point.
  - ***Step 1b.*** Determine and record a number for each borehole and record it on the diagram.
- **Step 2.** Dig the borehole or test pit.
  - ***Step 2a.*** Remove the overburden.
  - ***Step 2b.*** Extend boreholes 4 feet below the final subgrade elevation in cut areas.
  - ***Step 3b.*** Extend boreholes 4 feet below the natural ground elevation in fill areas.
  - ***Step 4b.*** Make an effort to locate the groundwater table.

Note. The depth may be deeper depending on the depth of frost penetration.

- **Step 3.** Obtain an individual soil sample for each soil layer encountered (figures 2-2 and 2-3, page 2-10).

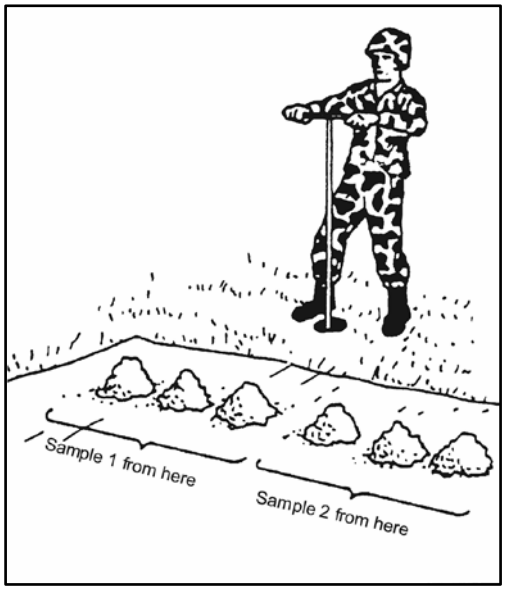


Figure 2-2. Obtaining individual bag samples

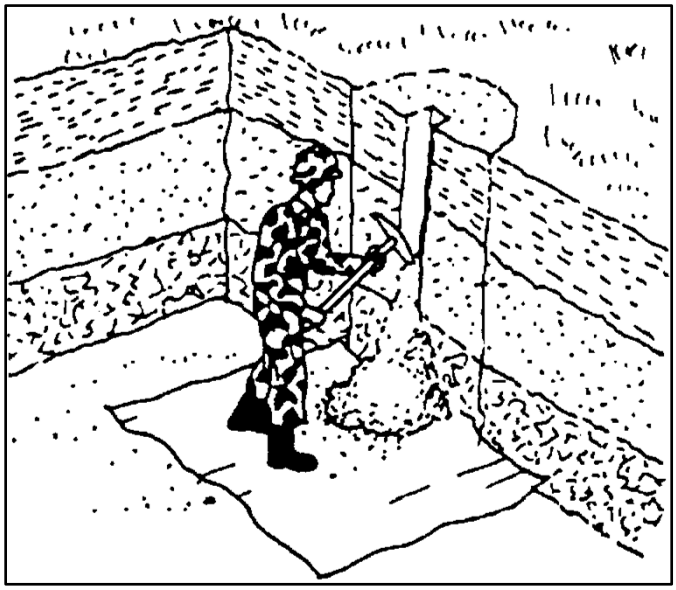


Figure 2-3. Taking a composite sample with an exposed face

- ***Step 3a.*** Place the soil from the borehole in a row in the order it was excavated, keeping soil layers separated for borings. In a test pit, obtain samples from each layer encountered.
- ***Step 3b.*** 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.
  - ***Step 4a.*** Determine the elevation or depth below the surface of the top and bottom of each soil layer encountered.
  - ***Step 4b.*** Record a description of the type of soil encountered.
  - ***Step 4c.*** Record the depth of the water table if encountered.
  - ***Step 4d.*** Record the depth of bedrock if encountered.
  - ***Step 4e.*** Record other pertinent facts (borehole number, date, noncommissioned officer in charge [NCOIC], project number, project location).
- **Step 5.** Bag individual soil samples for transportation.
  - ***Step 5a.*** 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).
  - ***Step 5b.*** Label two shipping tags for each bag, indicating the project, borehole, sample number (the order in which it was obtained), and the total number of bags included in the sample.
  - ***Step 5c.*** Place one tag inside the bag and tie the other to the outside when the bag is secured (figure 2-4).
- **Step 6.** Take a composite sample.
  - ***Step 6a.*** Remove overburden or surface soil that is to be wasted.
  - ***Step 6b.*** Shave off loose or dried material to provide a fresh face.
  - ***Step 6c.*** Spread a quartering cloth and tarpaulin at the toe of the bank.
  - ***Step 6d.*** Cut a channel of uniform cross section from top to bottom, and deposit the soil on the canvas.
  - ***Step 6e.*** Bag all material that was removed to ensure that the sample contains the appropriate proportions.

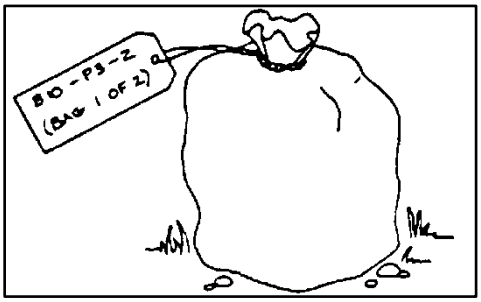


Figure 2-4. Labeling bag samples

Note. Ensure that sample numbering is recorded carefully and accurately so that the diagram borehole number, sample bags, and moisture tares correspond.

MOISTURE CONTENT SAMPLES

2-40. The natural moisture content of the soil 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 (one handful) 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 1 day after the sample is taken. If a longer time interval elapses between sampling and testing, the boxes may be sealed by sealing a container to retain a sample's moisture content (figure 2-5). Other clean containers that can be sealed adequately may be used for moisture-content samples.

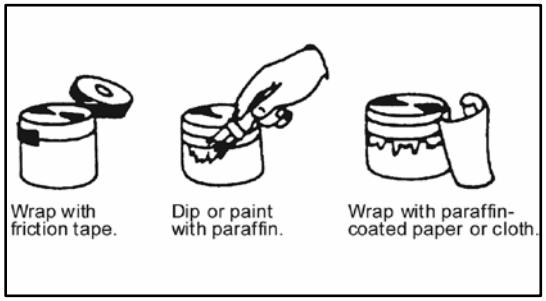


Figure 2-5. Sealing a container to retain a sample's moisture content

UNDISTURBED SAMPLES

2-41. 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 completely equipped laboratories for shear, consolidation, or other strength tests.

CHUNK SAMPLES

2-42. 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 (figure 2-6). The first step is to smooth the ground surface and mark the outline of the chunk. Excavate a trench around the chunk, then deepen the excavation and trim the sides of the chunk with a knife. Cut off the chunk at the bottom with a knife, trowel, or hacksaw blade, and carefully remove it from the hole.

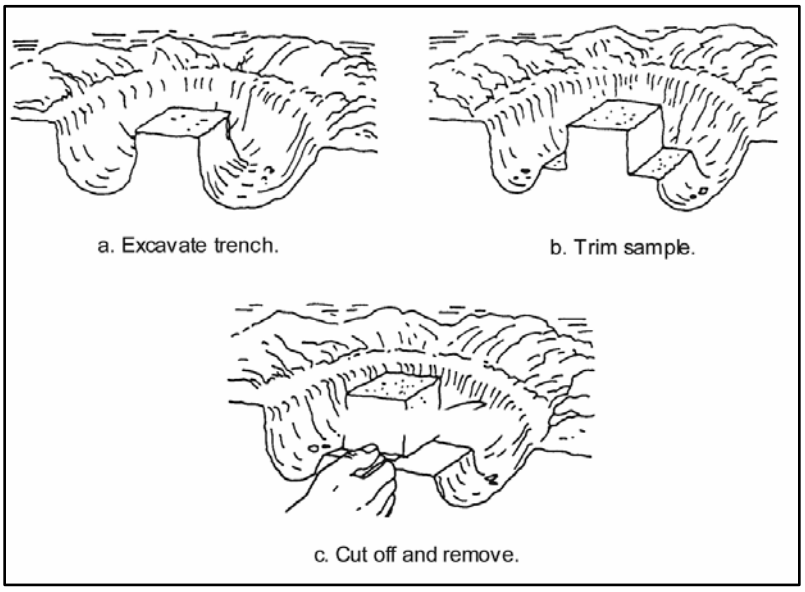


Figure 2-6. Taking a chunk sample from a level surface

2-43. To obtain a chunk sample (figure 2-7) from the vertical face of a test pit or trench, carefully smooth the surface of the face and mark the outline of the chunk. Excavate soil from the sides and back of the chunk. Shape the chunk with a knife, then cut off the chunk and carefully remove it from the hole.

2-44. Seal the chunk sample after removing it from the hole. One method is to apply three coats of melted paraffin (figure 2-8). 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 (figure 2-9). 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.

2-45. An alternate method of sealing the chunk sample is to dip the entire sample in melted paraffin after the first brushed coat has been applied and the sample has been wrapped (figure 2-10). This requires a large container and 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 (figure 2-11, page 2-14) or by applying many coats of cloth and paraffin.

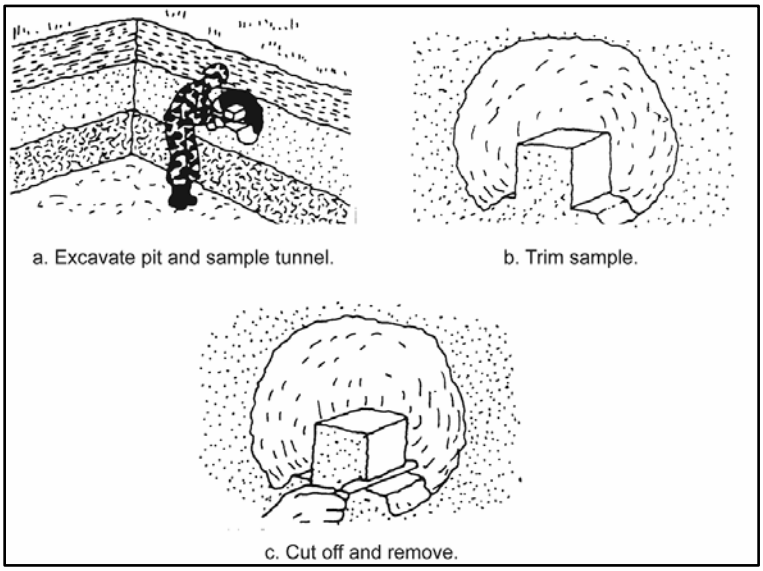


Figure 2-7. Taking a chunk sample from a vertical face

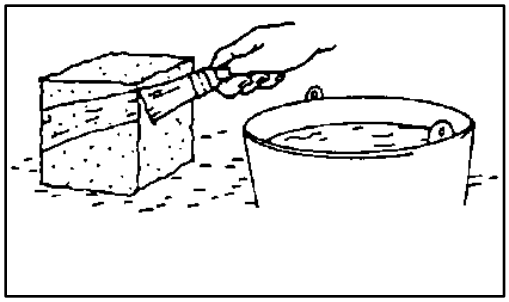


Figure 2-8. Applying paraffin to seal a chunk sample

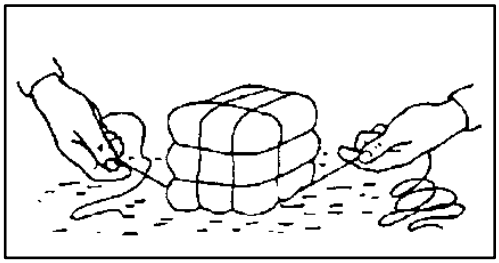


Figure 2-9. Wrapping a weak chunk sample before final sealing

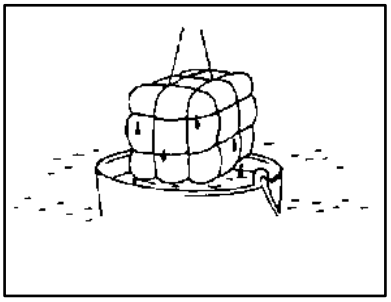


Figure 2-10. Dipping a chunk sample into melted paraffin

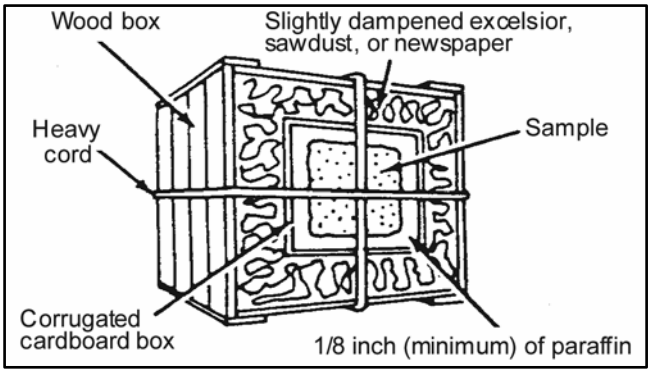


Figure 2-11. Packing a chunk sample for transportation or shipment to laboratory

2-46. 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 they are 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 the change in moisture content during sampling and shipment.

CYLINDER SAMPLES

2-47. 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

2-48. The soil-trafficability sampler consists of a cylindrical sample tube and an assembly to force the tube into the soil (figure 2-12 and FM 5-430-00-1/AFJPAM 32-8013/MCRP 3-17.7A, Volume I). 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. Long and short spacer bars 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.

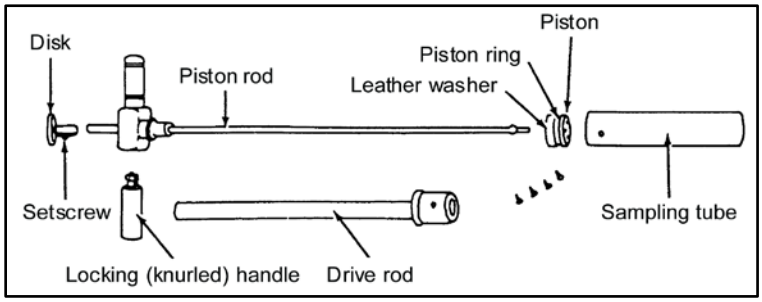


Figure 2-12. Soil trafficability sampler

2-49. Perform the following steps to take 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, 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, 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 long 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.

2-50. This sampler can be used with a hand auger to obtain cores up to 48 inches below the surface. The cores are sealed.

2-51. 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 soil type), 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.

2-52. 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.

California Bearing Ratio Mold

2-53. In soft, fine-grained soils, cylinder samples for undisturbed CBR or density tests (see chapter 8) may be taken directly in the CBR compaction cylinder by using the sampling collar (cutter) (figure 2-13). Perform the following steps using a CBR mold:

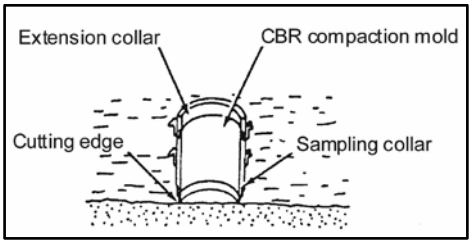


Figure 2-13. Section through CBR mold

- **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 (figure 2-14).

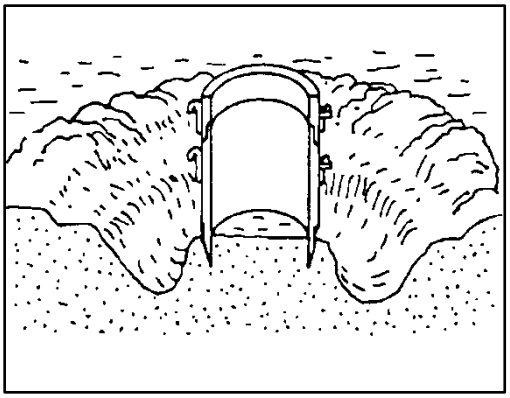


Figure 2-14. Trench excavated around cylinder

- **Step 3.** Press the mold down firmly over the soil again, using the hand driver or loading bar if necessary (figure 2-15). A loading bar may be improvised from any piece of suitable-size timber.

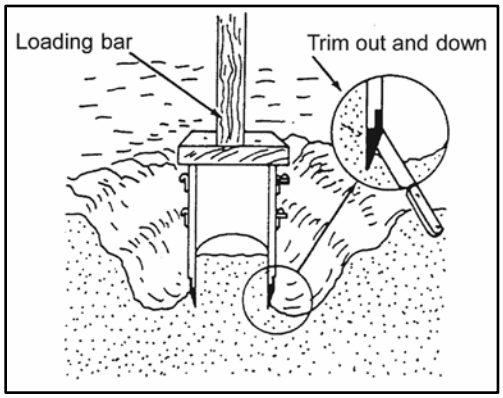


Figure 2-15. Load bar used to drive cylinder

- **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. Do not force the sampler down ahead of the trimming on the outside of the cylinder.
- **Step 5.** Excavate the trench deeper, and repeat the process until the soil penetrates well into the extension collar (figure 2-16). 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.

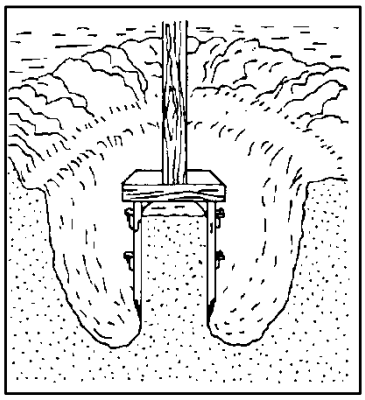


Figure 2-16. Cylinder in position before cutting sample

- **Step 6.** Cut the sample off at the bottom of the mold using a shovel, knife, or wire saw (figure 2-17).

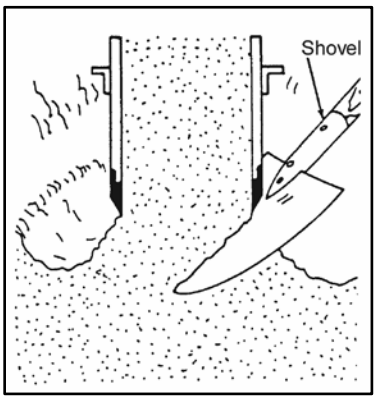


Figure 2-17. Cutting off cylindrical sample

- **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 (figure 2-18, page 2-18).
- **Step 9.** Turn the mold over, and remove the cutting edge. Trim this end down into the mold about 1/2 inch, and fill the resulting space with paraffin. If the sample is to be handled very much before testing, overfill the ends with paraffin, and trim it exactly flush with a straightedge.

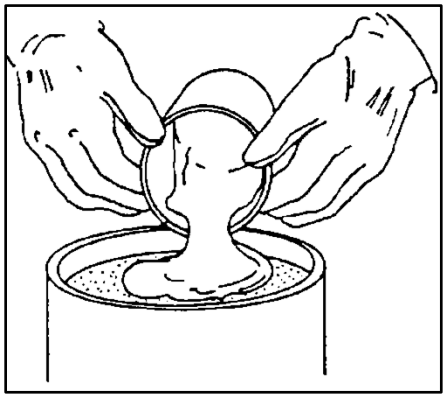


Figure 2-18. Sealing sample with paraffin after trimming

- **Step 10.** Place boards over each end, and clamp them in place using bolts, wire, or string (figure 2-19).

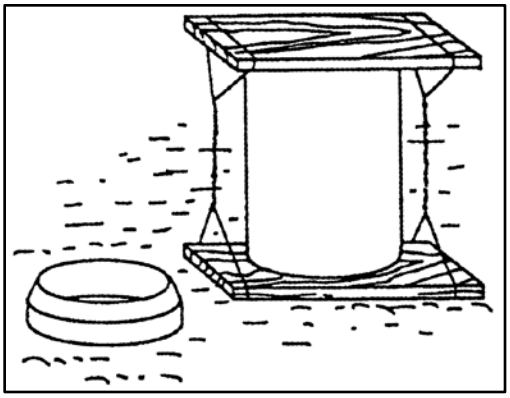


Figure 2-19. Protecting a sample in a CBR mold

- **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.

QUARTERING SAMPLES

2-54. The process of reducing a representative sample to a convenient size or 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 MORE THAN 100 POUNDS

2-55. To quarter samples weighing more than 100 pounds (figure 2-20)—

- Mix the sample and pile it on the quartering canvas.
- 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.
- 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.
- 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.

- 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.

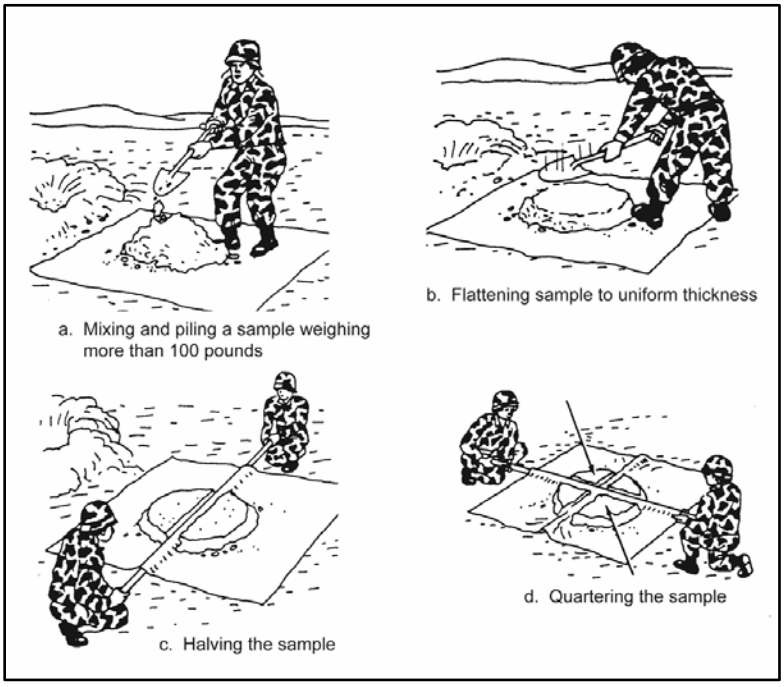


Figure 2-20. Samples more than 100 pounds

SAMPLES 25 TO 100 POUNDS

2-56. 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, and flatten it (figure 2-21).

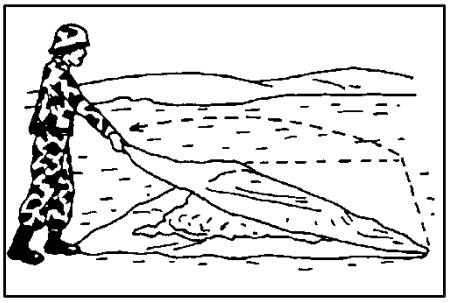


Figure 2-21. Mixing a sample 25 to 100 pounds

SAMPLES LESS THAN 25 POUNDS

2-57. The process of quartering samples less than 25 pounds is similar to the process for samples more than 100 pounds. To quarter samples weighing less than 25 pounds (figure 2-22, page 2-20)—

- 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.
- Flatten the cone by pressing downward with the trowel. Use the trowel to divide the sample into quarters.

- Set aside or discard diagonally opposite quarters.
- Repeat the process as necessary to reduce the size of the sample for testing.

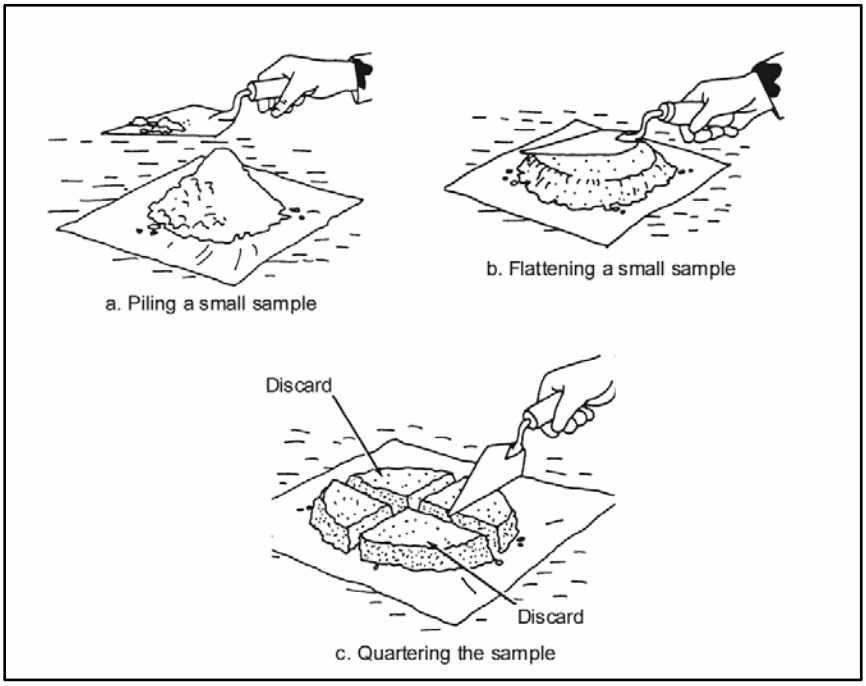


Figure 2-22. Mixing a sample less than 25 pounds

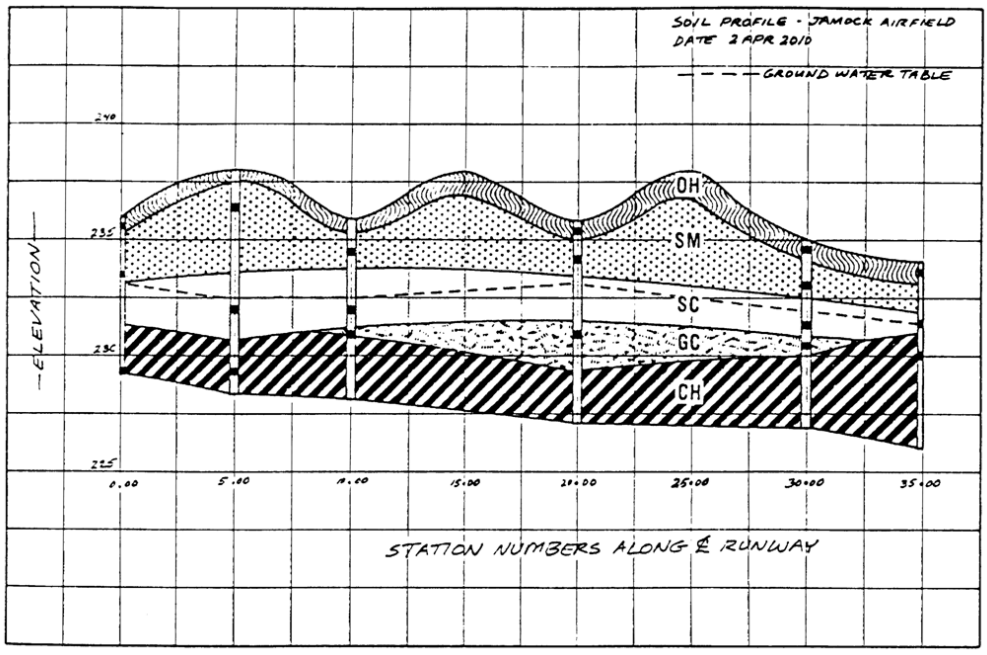
SOIL PROFILE

2-58. 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 (figure 2-23).

PURPOSE

2-59. 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 water table is high. Considerations for capillary and frost action may be particularly important when frost-susceptible soils are shown on the profile.

2-60. The soil profile, including the legend, will show each soil layer, the 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.



Legend:
Apr April

Figure 2-23. Typical soil profile

EQUIPMENT

2-61. Use the following items in a laboratory environment to obtain a soil profile:

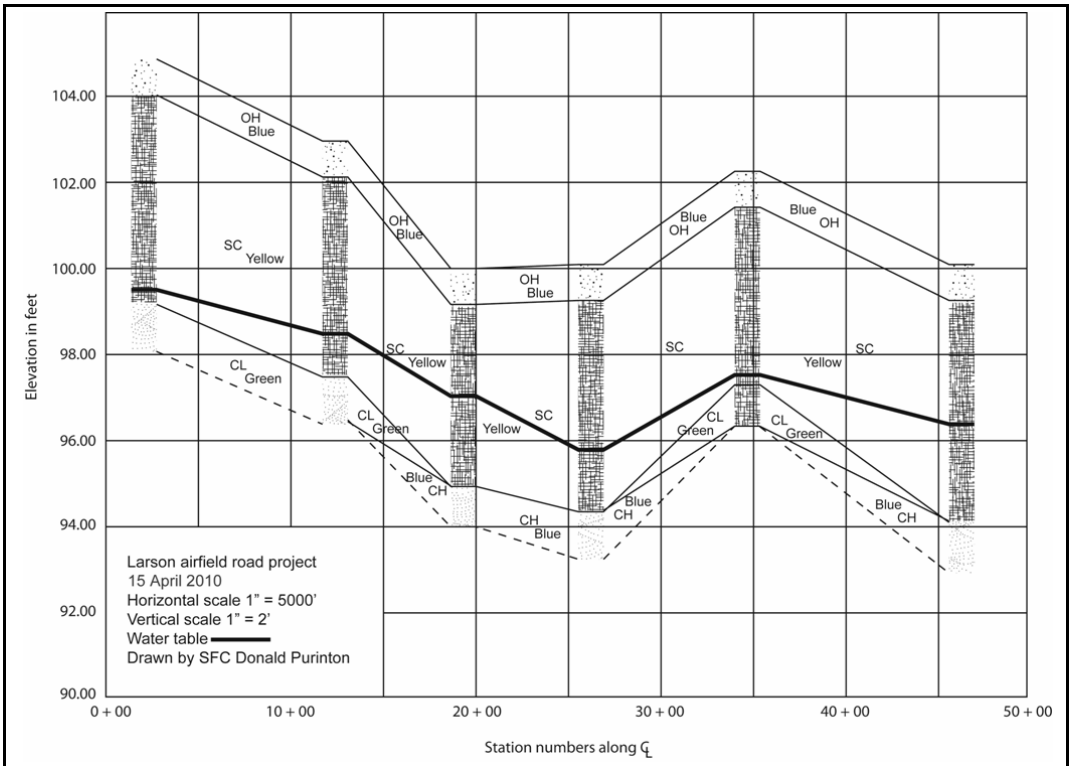
- Boring logs.
- Graph paper.
- Pencils.
- A straightedge.
- FM 5-430-00-1/AFJPAM 32-8013/MCRP 3-17.7A, Volume I.

STEPS

2-62. Perform the following steps to obtain a soil profile:

- **Step 1.** Determine the scales to be used (figure 2-24, page 2-22).
  - ***Step 1a.*** Determine and label the vertical scale (along the left side of the graph paper) representing the highest and lowest elevations found in the bore logs.
  - ***Step 1b.*** Determine and label the horizontal scale (along the bottom of the graph paper) 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/AFJPAM 32-8013/MCRP 3-17.7A, Volume I).
- **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 and a color.
- **Step 6.** Plot the depth of water at each borehole, and connect the points (with a solid, heavy line), showing the watertable profile.

- **Step 7.** Place the legend in a corner of the graph paper, and including the following:
  - The horizontal and vertical scales.
  - The line symbol for the water table.
  - The project title and location.
  - The name of the preparer and the date prepared.



Legend:

SFC	sergeant first class

Figure 2-24. Soil-profile scales

Chapter 3

Moisture Content Determination

Moisture content (also referred to as water content) of the soil 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 oven-dried solids in the sample (as a percentage). Use the following formula to determine moisture content:

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

where—
w = moisture content of the soil (as a percentage)
W_w = weight of water in the soil sample
W_s = weight of oven-dried soil solids in the sample

With many soils, the 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 tests for determining the moisture content of soils, including the—

- Oven-dry test (ASTM D2216).
- Microwave oven test (ASTM D4643).
- Calcium carbide gas pressure test (ASTM D4944).
- Nuclear moisture and density gauge test (ASTM D6938 and chapter 8).

OVEN-DRY TEST

3-1. The most accurate test of determining moisture content is the oven-dry test. This test 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 (stove, 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. This test determines the moisture content of a soil sample to within a desired percentage.

EQUIPMENT

3-2. The following items are necessary for the oven-dry test:

- 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 *(Soil Moisture-Content Determination)* (figure 3-1).
- A pencil.

STEPS

3-3. Perform the following steps to determine the moisture content:

Note. All references to form in this paragraph refer to DD Form 1205 unless noted otherwise.

- **Step 1.** Record identifying information of the sample in Blocks 1 through 5.

SOIL MOISTURE-CONTENT DETERMINATION

1. PROJECT: ENGINEER CENTER EXPANSION				2. DATE: 2 APR 2010		
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						
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)	%	%	%	%	%	%

SAMPLE

6. REMARKS

WATER CONTENT $w = \frac{Ww}{Ws} \times 100$

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

DD FORM 1205, DEC 1999 PREVIOUS EDITION IS OBSOLETE.

Figure 3-1. Sample DD Form 1205

Legend:	
Apr	April
DD	Department of Defense
Dec	December
SPC	specialist
SSG	staff sergeant

Figure 3-1. Sample DD Form 1205 (continued)

- **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. Ensure that—
  - The specimen mass stated in the test is used when conducting this test as part of another test method.
  - The values in table 3-1 are used, depending on the degree of accuracy of the reported water content when conducting this test with no minimum specimen mass provided.

Table 3-1. 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%*
2.00 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 g is used.			

Legend:	
g	gram
in	inch
kg	kilogram
mm	millimeter
No.	number

- **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°Celcius (C) ± 5° until the sample weight becomes constant. Oven drying 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 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 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 moisture content (in percent), and record it using the following formula:

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

Where—
w = moisture content of the soil (as a percentage)
W_w = weight of water in the soil sample
W_s = weight of oven-dried soil solids in the sample

3-4. 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. If none of the individual tests meet this requirement, then additional testing is required.

MICROWAVE OVEN-DRY TEST

3-5. For many field applications, the microwave is the most efficient, expedient means to determine the moisture content of soils. The microwave provides results that are within 0.5 percent of the oven-dried moisture value. The energy of the microwave allows moisture content results to be obtained within 5 to 10 minutes for nearly any soil as opposed to 4 to 16 hours in the oven-dry test.

3-6. This test is applicable for most soil types. Soils that have excessive amounts of halloysite, mica, montmorillonite, or highly organic soils may not yield reliable water content values.

EQUIPMENT

3-7. The following items are necessary for the microwave oven-dry test:

- A 700-watt microwave oven.
- A balance scale sensitive to 0.01 gram.
- Moisture determination tares.
- A grease pencil.
- Heat-resistant gloves.
- Ceramic, glass, or nonmetallic bowls.
- A stirring rod.
- A small piece of brick or concrete to use as an energy buffer.
- A timer (on microwave or a stopwatch).

STEPS

3-8. Perform the following steps to determine the moisture content:

- **Step 1.** Label and weigh a clean, dry moisture determination bowl, and record the weight as weight of the tare.
- **Step 2.** Obtain a 100-gram (recommended) soil sample. Smaller weights can be used, but introduce more error. Larger samples will require longer dry times. When conducting this test as part of another test, use the specimen mass stated in the first test.
- **Step 3.** Weigh the soil sample and the tare to the nearest 0.01 gram. Record the weight as the initial weight of the tare and the wet soil.
- **Step 4.** Place the brick in the back of the microwave oven.

WARNING

It is very important that the brick be placed in the microwave during this test. The brick acts to absorb excess microwaves during drying, preventing damage to the microwave and the soil sample.

- **Step 5.** Place the soil sample in the microwave dry for 1 minute under high power. The first minute of drying will heat the soil to the point that drying can begin. Little change in moisture content occurs during this first minute.

Note. The ASTM suggests an initial 3-minute drying time, which is acceptable for large soil samples (larger than 100 grams); however, with smaller samples or sandy soil samples, the 1 minute procedure minimizes the chance of overdrying and damaging to the soil and microwave.

- **Step 6.** Remove the sample using the heat-protected gloves, and weigh the bowl with soil. Record the weight of the bowl and dry soil.

Note. When using the balance, record only the first weight displayed as dissipation of heat within the soil will cause continual fluctuation of the balance as time passes. This behavior will stabilize when the soil reaches its final dried state.

- **Step 7.** Stir the soil with the stirring rod, being careful not to remove any material from the bowl. Reinsert the soil sample in the microwave and dry for 1 minute on high.
- **Step 8.** Remove the sample using the heat-protective gloves, and weigh the bowl with soil. Record the weight.
- **Step 9.** Calculate the percentage change in weight as the ratio between the change in soil weight during drying from step 6 to step 9 to the wet soil weight in step 3 as follows:

$$DW(\%) = \frac{(W_1 - W_2)}{(W_i - W_f)} 100$$

where—
DW = percentage change in weight
W_1 = weight of the bowl and dry soil
W_2 = weight of the bowl and soil
W_i = weight of wet soil
W_f = final weight of the bowl and dry soil

Note. If the change in weight is greater than 0.1 percent, continue to step 10.

- **Step 10.** Stir the soil with the stirring rod, being careful not to remove any material from the bowl. Reinsert the soil sample in the microwave and dry for one minute on high.
- **Step 11.** Remove the sample using the heat-protective gloves, and weigh the bowl. Record the weight.
- **Step 12**. Calculate the percentage change in weight as the ratio between the change in soil weight during drying from step 9 to step 11 to the wet soil weight in step 3 as follows:

$$DW(\%) = \frac{(W_2 - W_3)}{(W_i - W_f)} 100$$

where—
W_2 = weight of the bowl and soil
W_i = weight of wet soil
W_f = final weight of the bowl and dry soil

Note. If the change in weight is greater than 0.1 percent, repeat steps 9 through 12 until the change in weight is equal to or less than 0.1 percent. Record the final weight of the dried soil and bowl.

- **Step 13.** When the change in weight is equal to or less than 0.1 percent the test is complete. Calculate the moisture content of the soil in terms of a percentage as follows—

$$w(\%) = \frac{(W_i - W_f)}{(W_f - W_t)} 100$$

where—
w = moisture content of the soil (as a percentage)
W_i = weight of wet soil
W_f = final weight of the bowl and dried soil
W_t = weight of the tare

Note. Software is available through the USACE, Engineer Research and Development Center, Vicksburg, Mississippi to assist with the conduction of this test in the field. The software provides a stepwise interface to input the microwave data and return the final moisture content when the test is finished. All calculations described earlier are internalized and the user only needs to be responsible for the proper weighing of the sample.

MICROWAVE PRECAUTIONS

3-9. As with any electric heating element, care should be taken to protect hands and face when using a microwave. Gloves should be worn at all times to handle hot dishes or bowls as they are taken in and out of the microwave during testing. For sands and gravelly soils, eye goggles are recommended since potential overdrying of these soils can lead to bursting of the individual grains, posing a debris hazard when the samples are stirred.

3-10. If a 700-watt microwave oven is not available, drying times may vary significantly. Higher-powered microwaves (1,000 watts or greater) will dry the soil much quicker, but also pose a greater risk to overdrying during the prescribed 1 minute intervals. This can damage the soil and the microwave much faster, so care should be taken with these devices. Thirty-second intervals would be better suited to these devices. Lower-wattage microwaves (500 watts or less) will require much longer drying times (up to 10 times longer), 2- to 3-minute intervals would be better suited to these devices.

3-11. Large sample sizes (in excess of 300 grams) may require very long dry times and may not provide consistent moisture contents. In this case, taking a smaller representative sample would be preferred to determine the final moisture content. If soil must be dried for other testing purposes, divide larger samples into portions that are 300 grams or less for microwave drying.

CALCIUM CARBIDE GAS PRESSURE TEST

DANGER

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 calcium carbide pressure (speedy) moisture tester. Perform the test in a well-ventilated area, as asphyxiation could occur if performed in a confined area.

3-12. Use the calcium carbide gas pressure test to determine the moisture content of a soil sample using the 26-gram calcium carbide gas pressure test 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 to obtain the specific soil sample size and quantity of calcium carbide reagent.

3-13. Because of its small sample size, the device should not be used on soils that contain material greater than No. 4 sieve (4.75-millimeter diameter). Further, high plasticity clays or other soils not friable enough to break up may not produce representative moisture contents because the reagent cannot mix well with the soil sample.

EQUIPMENT

3-14. Use the following items for the calcium carbide gas pressure test:

- A calcium carbide pressure moisture tester to hold a 26-gram soil specimen (approximately one large tablespoon).
- 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

3-15. Perform the following steps to determine the moisture content of the soil:

- **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. Thoroughly clean the cap of carbide and soil before running another test.

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

CALCULATIONS

3-16. Determine the percentage of moisture by dry mass of the soil from the calibration curve (figure 3-2, page 3-8) or from the conversion chart (table 3-2, page 3-9) as explained below. The calibration curves, moisture content determination by the calcium carbide gas pressure test, are used for materials that need pulverizing steel balls (table 3-2). 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.

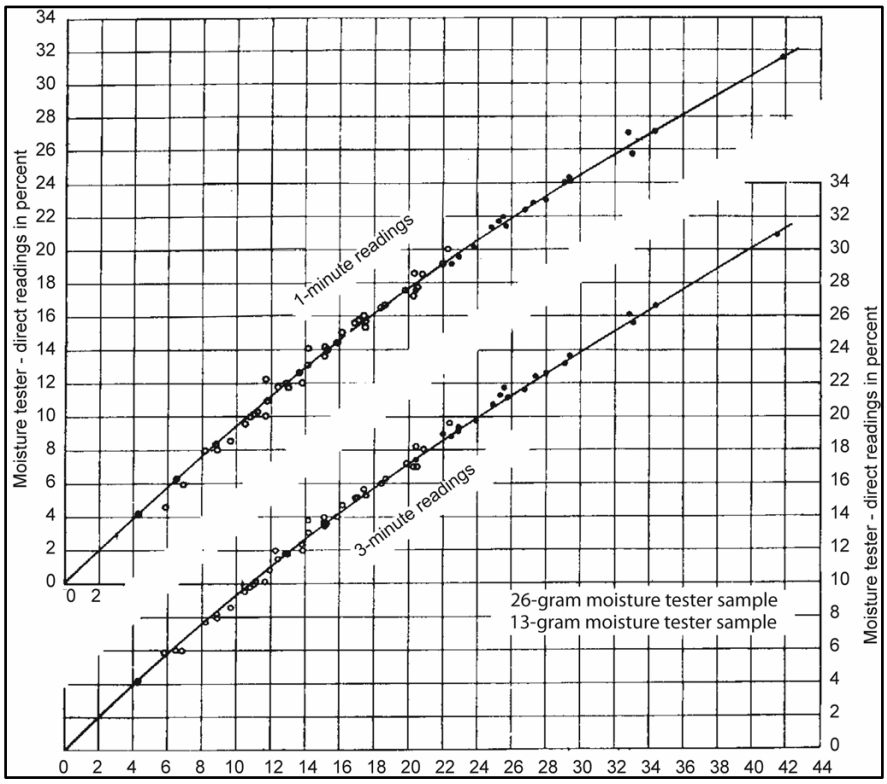


Figure 3-2. Calcium carbide pressure moisture tester calibration chart for use with 1 1/4-inch steel balls

3-17. Use the conversion chart (table 3-2) 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

3-18. A calcium carbide gas pressure 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.

3-19. Using table 3-2, the values in table 3-3, are known. To find the value of x, place the differences in the table values into a ratio. The ratio of differences is—

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

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.*

3-20. 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 to extend the conversion curve beyond 44 percent moisture content.

3-21. It may be more convenient for the field use of equipment to prepare a table of moisture tester readings versus oven-dry moisture content.

Table 3-2. Calcium carbide pressure moisture tester conversion chart for use when not using 1 1/4-inch steel balls

Wet-Weight Percentage	*Dry-Weight Percentage*	*Wet-Weight Percentage*	*Dry-Weight Percentage*	*Wet-Weight Percentage*	*Dry-Weight Percentage*
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

Table 3-3. Determining the moisture content of the soil

Wet-Weight Percentage	*Dry Weight Percentage*
3.0	3.2
3.5	x
4.0	4.3

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Chapter 4

Specific Gravity of Solids Determination

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 void ratio of the soil and for determining the grain size distribution in hydrometer analysis. The term specific gravity of soil actually refers to the specific gravity of the solid matter of the soil. 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 specific gravity of a soil to perform additional testing of that soil. The specific gravity of a soil 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 and some lateritic soils usually have a specific gravity of between 2.75 and 3.00 but could be higher.
- Sand particles composed of quartz have a specific gravity 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

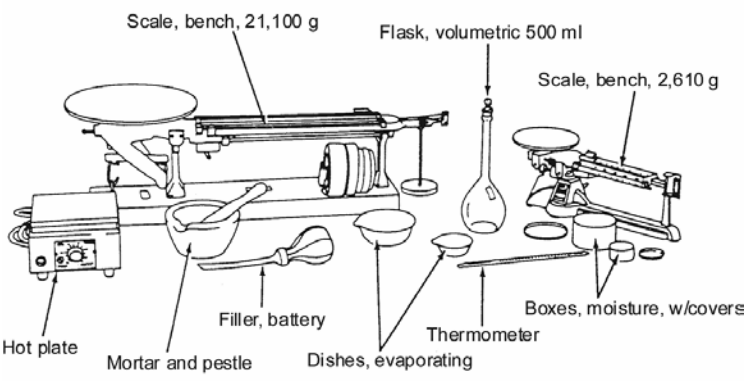
4-1. Perform the specific-gravity test to determine the specific gravity of solids (which will be used to assist in the hydrometer analysis test) and to calculate the zero air voids curve for compaction results. Take particular care to obtain representative samples for a specific-gravity test. It is 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. See ASTM C127 and ASTM C128.

EQUIPMENT

4-2. Perform the specific-gravity test in a laboratory environment, using the following items (figure 4-1, page 4-2):

- A pycnometer; volumetric flask (500-milliliter capacity).
- A laboratory oven.
- Heat-resistant gloves.
- Balance scales sensitive to 0.01 gram.
- Pudding pans.
- A water bath.
- A thermometer.
- A battery filler syringe.

- A No. 4 sieve.
- Graph paper.
- A pencil.
- A French curve.
- Cloth towels.
- Paper towels.
- A hot plate.
- DD Form 1208 *(Specific-Gravity Tests).*
- A spatula.
- An evaporating dish.
- Distilled water.
- A calculator.
- A pail.



Legend:
g gram
ml milliliter

Figure 4-1. Apparatus for determining specific gravity of soils

STEPS

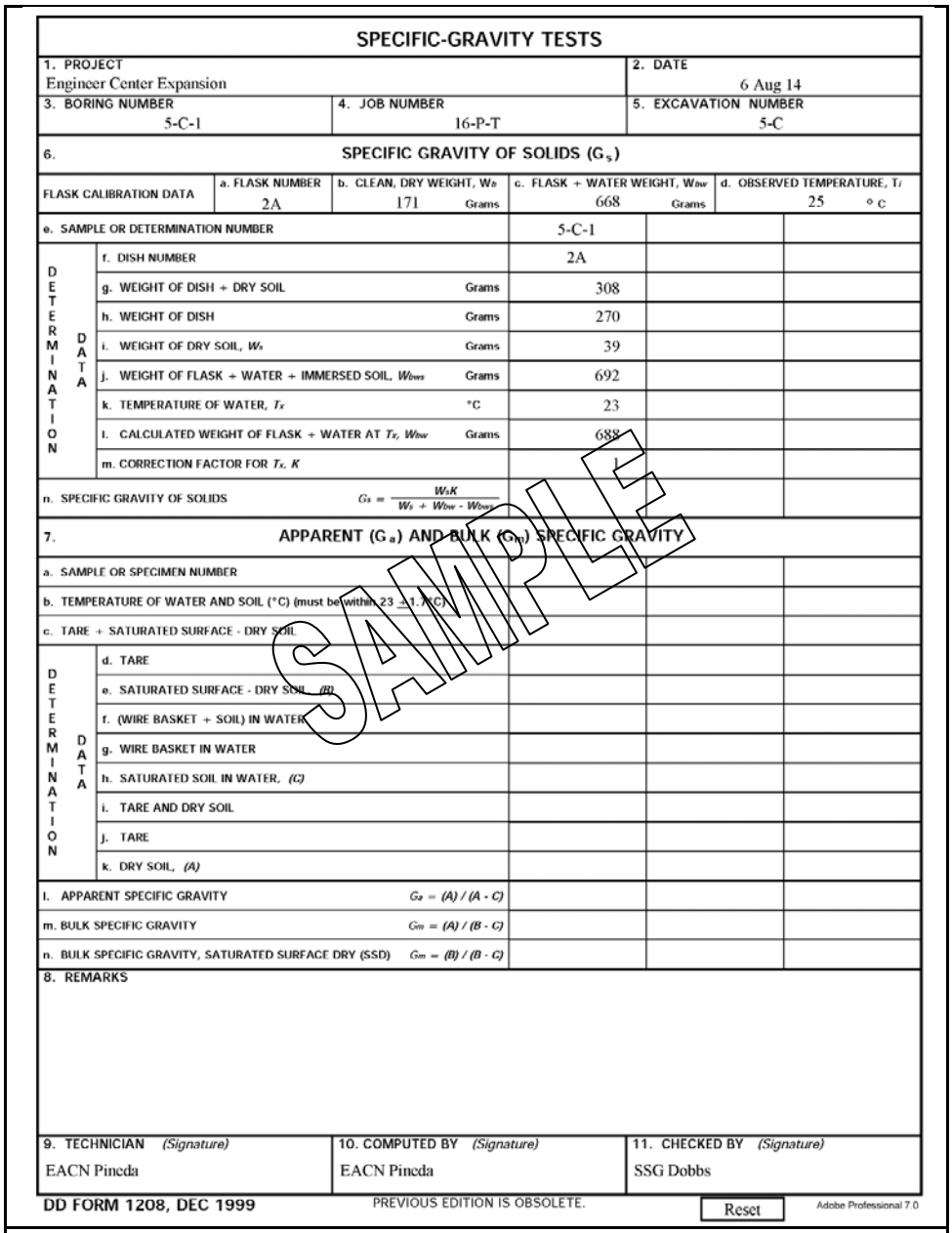
4-3. Perform the following steps to determine the specific gravity of a soil.

Note. All references to form in this paragraph refer to DD Form 1208 (figure 4-2) unless noted otherwise.

Step 1. Calibrate the Pyncometer

4-4. If a calibration graph has already been prepared for this pycnometer and will be used for the determination procedures, go to step 2. Perform the following substeps to complete step 1:

- **Step 1a.** Weigh a clean, dry pycnometer to the nearest 0.01 gram. Record this information on the form. Additionally, record the basic information concerning the specimens being tested and the pycnometer/flask number.
- **Step 1b.** Fill the pycnometer with room temperature distilled water. Ensure that the bottom of the meniscus is even with the calibration mark.
- **Step 1c.** Weigh the pycnometer plus water. Record this information on the form.



SPECIFIC-GRAVITY TESTS

1. PROJECT: Engineer Center Expansion			2. DATE: 6 Aug 14
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	b. CLEAN, DRY WEIGHT, W_b	c. FLASK + WATER WEIGHT, W_{bw}	d. OBSERVED TEMPERATURE, T_i
	2A	171 Grams	668 Grams	25 °C

DETERMINATION DATA				
e. SAMPLE OR DETERMINATION NUMBER		5-C-1		
f. DISH NUMBER		2A		
g. WEIGHT OF DISH + DRY SOIL	Grams	308		
h. WEIGHT OF DISH	Grams	270		
i. WEIGHT OF DRY SOIL, W_s	Grams	39		
j. WEIGHT OF FLASK + WATER + IMMERSED SOIL, W_{bws}	Grams	692		
k. TEMPERATURE OF WATER, T_x	°C	23		
l. CALCULATED WEIGHT OF FLASK + WATER AT T_x, W_{bw}	Grams	688		
m. CORRECTION FACTOR FOR T_x, K		1		
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

DETERMINATION DATA				
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				
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)*
EACN Pineda	EACN Pineda	SSG Dobbs

DD FORM 1208, DEC 1999 PREVIOUS EDITION IS OBSOLETE. Reset Adobe Professional 7.0

SAMPLE

Legend:

Aug	August
DD	Department of Defense
Dec	December
EACN	constructionman, engineering aide striker (naval rating)
SSG	staff sergeant

Figure 4-2. Sample DD Form 1208

- **Step 1d.** Use the thermometer to determine the water temperature to the nearest whole degree. Record the temperature on the form.
- **Step 1e.** Create a graph or table for the pycnometer being used (if additional specific-gravity determinations are to be made). This graph helps determine values for the weight of pycnometer and water for desired water temperatures. It eliminates the need to calibrate the pycnometer for

each test. The graph can be developed by using the following formula for various temperatures, plotting the temperatures against the weight of the pycnometer and water and drawing a smooth curve through the plotted points:

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

where—
$\rho_w(T_x)$ *= density of water indentified by temperature* (T_x)
$\rho_w(T_i)$ *= density of water indentified by temperature* (T_i)
W_{bw} *= weight of pycnometer and water (in grams)*
W_b *= weight of pycnometer (in grams)*
T_i *= observed or recorded temperature of water (in °C)*
T_x *= any other desired temperature (in °C)*

4-5. Figure 4-3 is a completed graph with the above formula and the following data:

- Calibration data.
  - W_{bw} *= 656.43.*
  - W_b *= 158.68.*
  - T_i *= 24ºC.*
- 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.*

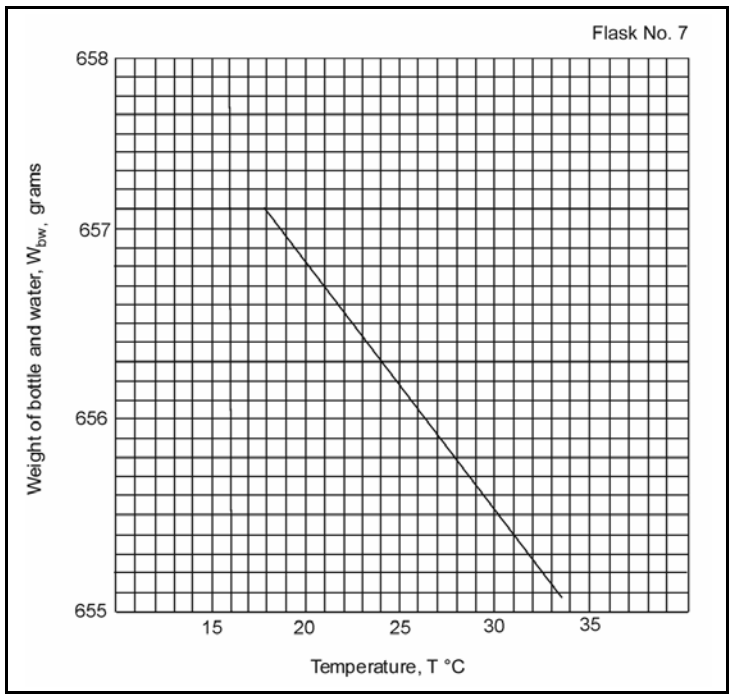


Figure 4-3. Calibration curve for a volumetric flask

Legend:	
C	Celsius
No.	number

Figure 4-3. Calibration curve for a volumetric flask (continued)

Step 2. Obtain a Soil Sample for Testing

4-6. 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. Since this test 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

4-7. 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. Perform the following substeps to complete step 3:

- **Step 3a.** Record identifying information about the sample on the form.
- **Step 3b.** Place the -4 sample into the evaporating dish.
- **Step 3c.** Perform the following procedures for soil at natural water content or moisture. If soil is not at natural water content or moisture, go to step 3d.
  - ***Step 3c(1).*** Add distilled water to the sample and mix to a slurry.
  - ***Step 3c(2).*** Transfer the slurry to the pycnometer, and add distilled water until the pycnometer is about one-third to one-half its depth with water.
- **Step 3d.** Perform the following procedures for an oven-dried soil sample.
  - ***Step 3d(1).*** Oven-dry the sample to a constant weight at a temperature of 110°C ± 5°. Allow the sample to cool, and weigh it to the nearest 0.01 gram. Record the weight on Block 6g as the weight of the dish and dry soil.
  - ***Step 3d(2).*** Transfer the dried sample to the volumetric flask. Take care to avoid the loss of any particles.
  - ***Step 3d(3).*** Fill the flask three-fourths full with distilled water, and allow it to soak for 12 hours.
  - ***Step 3d(4).*** Weigh the empty, dry evaporating dish. Record the weight on Block 6h as the weight of the dish.

Step 4. Process the Sample Through the Test

4-8. Process the sample through the test. Perform the following substeps to complete step 4:

- **Step 4a.** 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.
- **Step 4b.** Fill the pycnometer with distilled water until the bottom of the meniscus is level with the calibration mark.
- **Step 4c.** Dry the outside and thoroughly remove any moisture adhering to the neck of the pycnometer.
- **Step 4d.** Weigh the pycnometer and its contents to the nearest 0.01 gram. Record this amount on Block 6j as the weight of the flask, water, and immersed soil.
- **Step 4e.** Shake the flask immediately after weighing (putting its contents in suspension) and determine the water temperature at middepth to the nearest whole degree. Record this amount on Block 6k.
- **Step 4f.** Determine the dry unit weight for soil processed at natural moisture content.

- ***Step 4f(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.
- ***Step 4f(2).*** Oven-dry the sample to a constant weight at a temperature of 110°C ± 5°. Allow the sample to cool. Weigh and record the weight on Block 6g as the weight of the dish and dry soil.

Step 5. Compute Results

4-9. Compute the results needed to complete DD Form 1208. Perform the following substeps to complete step 5:

- **Step 5a.** Compute the weight of the dry soil by subtracting the weight of the dish from the weight of the dish and dry soil. Record it on the form.
- **Step 5b.** Determine the weight of the flask and water by plotting the temperature of the water obtained in step 4e 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.
- **Step 5c.** Determine the correction factor by locating the temperature of the water (obtained in step 4e. Using table 4-1, read across to the correction factor column and record it on the form as the correction factor.
- **Step 5d.** Compute the specific gravity of solids to two decimal places, and record the amount on the form. Use the following formula to compute the specific gravity of solids:

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

Table 4-1. Relative density of water and the correction factor at various temperatures

Temp, °C	*Relative Density*	*Correction Factor*
18	0.99862	1.0004
19	0.99843	1.0002
20	0.99823	1.0000
21	0.99802	0.9998
22	0.99780	0.9996
23	0.99757	0.9993
24	0.99733	0.9991
25	0.99708	0.9988
26	0.99682	0.9986
27	0.99655	0.9983
28	0.99627	0.9980
29	0.99598	0.9977
30	0.99568	0.9974
31	0.99537	0.9971
32	0.99505	0.9968
Note. Data obtained from ASTM. The correction factor is found by dividing the relative density of water at the test temperature by the relative density of water at 20°C.		
Legend: ASTM American Society for Testing and Materials C Celcius		

SPECIFIC GRAVITY (APPARENT AND BULK)

4-10. 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 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.

4-11. The apparent specific gravity is designated 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. The bulk specific gravity 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. These tests are applicable to testing soil with fine and coarse aggregates.

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Chapter 5

Grain Size Analysis and Distribution

The accurate determination of grain size distribution or the 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. This test is referred to as particle size or grain size distribution. This is accomplished in two steps:

- **Step 1.** A screening process (a sieve analysis, which is also called a mechanical analysis) for particle sizes retained on the No. 200 sieve.
- **Step 2.** A sedimentation process (a hydrometer analysis) for particle sizes smaller than the No. 200 sieve.

Note. Previous tests presented the sieve and hydrometer analysis as two separate tests. A combination of these analyses is a combined analysis. The ASTM employs a different test for particle size analysis which includes both tests (ASTM D422). These tests provide minor modifications to allow the end user to obtain results specific to the test purpose. The USMC uses a soil test set (see appendix D).

SIEVE ANALYSIS

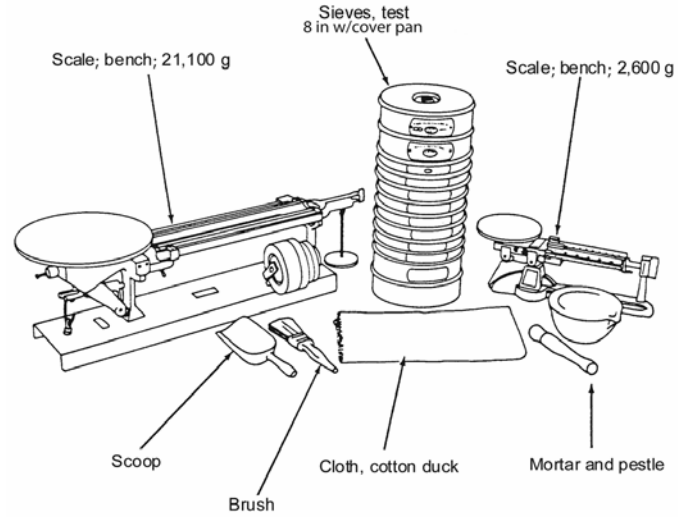
5-1. The accurate completion of the sieve-analysis test will produce the percent of gravel, sand, and fines of the material. The most accurate process for this test method is to wash the material over the sieves; this will give a more accurate percent of fines. It is possible the test will also provide sufficient information to calculate the coefficient of uniformity and the coefficient of curvature. Performing just the sieve analysis portion of this test 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 and provide necessary information required to determine the frost susceptibility of a soil.

PURPOSE

5-2. Perform the sieve analysis 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

5-3. Use the following items to perform this test (figure 5-1):



Legend:

g	gram
in	inch
w/	with

Figure 5-1. Sieve analysis equipment

- DD Form 1206 *(Grain-Size Analysis [Sieve Method])* or an equivalent form.
- DD Form 1207 *(Grain-Size Distribution Graph–Aggregate Gradation Chart).*
- A calculator.
- Beam scales.
- 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; and Nos. 4, 10, 16, 30, 40, 50, 100, and 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 or microwave.
- Heat-resistant gloves.
- A battery-filler syringe.
- Distilled water.

Note. The USACE RSAK (see appendix C) provides a method to closely approximate the particle size distribution using fewer and smaller diameter sieves. In addition, it uses an expedient dry sieving method versus wet sieving as prescribed below. In certain contingency construction scenarios, this may be a preferable test for time savings allowing a test to be completed in less than 1 hour. Case 1of the USMC soil test set includes all of the items required for grain size analysis (see appendix D).

STEPS

5-4. Perform the following steps to determine the grain size distribution:

Note. All references to form in this paragraph refer to DD Form 1206 (figure 5-2, page 5-4) unless noted otherwise.

- **Step 1.** Prepare the soil sample.
  - ***Step 1(a).*** Spread out and air-dry the soil sample.
  - ***Step 1(b).*** Break up the aggregate particles thoroughly with fingers or with the mortar and pestle.
  - ***Step 1(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 5-1.

Table 5-1. Representative soil samples for grain size analysis

Maximum Particle Size		*Minimum Sample Weight*
3/8 in (No. 4)	9.5 mm (No. 4)	500 g
3/4 in	19.0 mm	1,000 g
1 in	25.4 mm	2,000 g
1 ½ in	38.1 mm	3,000 g
2 in	50.8 mm	4,000 g
3 in	76.2 mm	5,000 g
Legend: g gram in inch mm millimeter No. number		

- **Step 2.** Record identifying information about the sample (Blocks 1 through 7).

Note. A field identification based on a visual or textural classification method as noted in Block 6 of figure 5-2, page 5-4 may differ from the actual engineering classification, depending on the individual's experience and level of detail obtained during the identification. In this example, what appears to the user to be a red, fine sandy soil by quick observation actually classifies as a sandy clay with gravel, illustrating the need to perform a sieve analysis for engineering design.

- **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.

- **Step 5.** Check *No* in Block 9 and enter *0* in Blocks 10 and 11 if only a dry sieve is to be performed, and 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 adhering and lumpy particles are completely disintegrated. This process may take 2 to 24 hours.

GRAIN-SIZE ANALYSIS (SIEVE METHOD)

1. PROJECT: Engineer Center Expansion		2. DATE: 6 Aug 14
3. JOB NUMBER: 16-P-T	4. EXCAVATION: 5-C	5. DATE COMPLETED: 8 Aug 14
6. NOTES ABOUT SAMPLE/DESCRIPTION: Red in color, very fine sands		7. SAMPLE NUMBER: 5-C-1
		8. ORIGINAL SAMPLE WEIGHT: 4,405
9. PREWASHED: [X] YES [ ] NO	10. + #200 SAMPLE WEIGHT, WASHED: 2,814	11. - #200 SAMPLE WEIGHT, WASHED: 1,570

12. SIEVE SIZE	13. SIEVE WEIGHT	14. SIEVE-SAMPLE WEIGHT	15. WEIGHT RETAINED	16. CUMULATIVE WEIGHT RETAINED	17. PERCENT RETAINED	18. PERCENT PASSING
2"	585	585	0	0	0	100
1 1/2"	714	797	84	84	1.9	98.1
3/4"	577	738	161	245	3.7	94.4
1/4"	540	1,168	628	873	14.3	80.1
#4	507	625	118	991	2.7	77.4
#16	404	827	418	1,408	9.5	67.9
#30	596	820	224	1,633	5.1	62.7
#40	402	634	232	1,865	5.3	57.4
#60	534	967	433	2,298	9.9	47.6
#80	351	655	304	2,602	6.9	40.6
#100	265	329	64	2,665	1.5	39.2
#200	347	461	114	2,779	2.6	36.6

19. TOTAL WEIGHT RETAINED IN SIEVES (Sum of column 15)	2,779	24. ERROR (8-23): 24	25. ERROR IN PERCENTAGE: $\left(\frac{24}{8}\right)$ X 100 = 0.5
20. WEIGHT SIEVED THROUGH #200 (Weight in pan)	33		
21. WASHING LOSS (8 - [10 + 11])	21		
22. TOTAL WEIGHT PASSING #200 SIEVE (20 + 11)	1,603		
23. TOTAL WEIGHT OF FRACTIONS (19 + 22)	4,381		

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

30. REMARKS: USCS SC with gravel

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

DD FORM 1206, DEC 1999 PREVIOUS EDITION IS OBSOLETE. Reset Adobe Professional 7.0

SAMPLE

Figure 5-2. Sample DD Form 1206

Legend:	
Aug	August
DD	Department of Defense
Dec	December
PV2	private enlisted two
SSG	staff sergeant
USCS	United Soil Classification System

Figure 5-2. Sample DD Form 1206 (continued)

- **Step 7.** Wash the sample over a No. 200 sieve into a 2-by-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 plus 200 material at 110°C ± 5° until a constant weight is obtained, and allow the material to cool. Record the weight to the nearest tenth of a gram (Block 10).
- **Step 9.** Process the -200 material.
  - ***Step 9a.*** Allow the -200 material to settle in the pan until the surface water becomes clear (16 to 24 hours).
  - ***Step 9b.*** Decant the surface water (using a siphon or a syringe), ensuring that the settled material is not disturbed.
  - ***Step 9c.*** Use a trowel to transfer as much of the material as possible from the pan to the pudding pans.
  - ***Step 9d.*** Rinse the remainder of the material from the 2-by-2 pans to the pudding pans with as little water as possible.
  - ***Step 9e.*** Oven dry the washed -200 material, and determine the total -200 sample weight to the nearest .01 grams. Record this weight on 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 particle size): 2 inches; 1.5 inches; 1 inch; 3/4 inch; 3/8 inch; and Nos. 4, 10, 16, 30, 40, 50, 100, and 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 the pan, and place it 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 (figure 5-3, page 5-6).

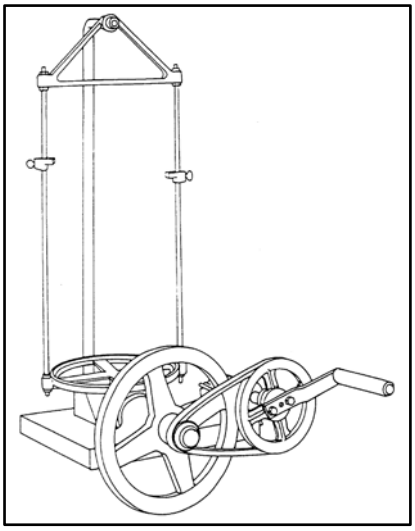


Figure 5-3. Hand-operated sieve shaker

- **Step 14.** Remove the cover of the sieve nest after the shaking is complete.
- **Step 15.** Record the weight of each sieve with the retained sample (starting with the top sieve (figure 5-4) in Column 14.

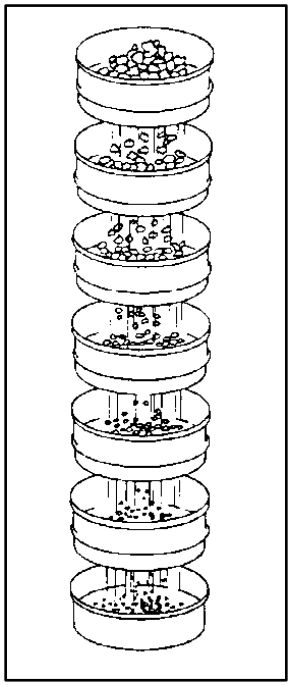


Figure 5-4. Testing sieves stacked large to small

- **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 13 and 14). 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 percentage error 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:

$$Column\ 18 = \frac{Block\ 23 - Block\ 16}{Block\ 23} \times 100$$

- **Step 23.** Determine the percentages for gravel, sand, and fines (Blocks 26, 27, and 28). Record the information on the form.

Notes.
1. Gravel is the material retained on the No. 4 sieve.
2. Sand is the material passing the No. 4 sieve and retained on the No. 200 sieve.
3. Fines are the material passing the No. 200 sieve.

- **Step 24.** Compute the decimal fines (Block 29) by dividing the percent of fines by 100 percent. This allows a smooth transition on the grain size curve between the soil sampled to conduct the hydrometer test and the total fine percentage in the sieved sample.
- **Step 25.** Complete DD Form 1207 (figure 5-5, page 5-8).
  - ***Step 25a.*** Record the identifying information for the sample in the remarks block.
  - ***Step 25b.*** Use the sieve analysis data to plot the sieve size and the percentage passing the sieve.
  - ***Step 25c.*** Using a French curve, connect the plotted points to form a smooth, free-flowing curve (the grain size distribution curve).
  - ***Step 25d.*** Determine the coefficient of uniformity.

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

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

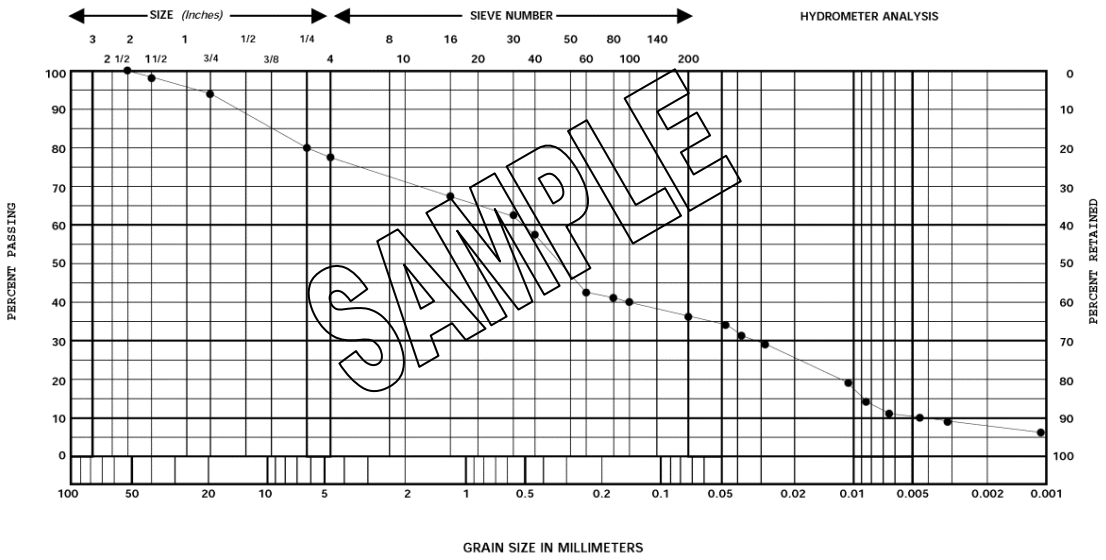
where—
Cu = coefficient of uniformity
D_{10} = 10 percent passing on the grain size distribution curve
D_{60} = 60 percent passing on the grain size distribution curve

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.

GRAIN SIZE DISTRIBUTION GRAPH - AGGREGATE GRADATION CHART

1. PROJECT: Engineer Center Expansion (16-P-T)
2. DATE: 6 Aug 14

SIEVE ANALYSIS - US STANDARD SIEVE SIZES



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)	4. PLOTTED BY (Signature)	5. CHECKED BY (Signature)
PV2 Dishman	PV2 Dishman	SSG Dobbs

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Legend:

Aug	August
DD	Department of Defense
Dec	December
LL	liquid limit
PI	plasticity index
PL	plastic limit
PV2	private two enlisted
SSG	staff sergeant
US	United States
USCS	Unified Soil Classification System

Figure 5-5. Sample DD Form 1207

- ***Step 25e.*** Determine the coefficient of curvature by using 60 percent passing on the grain size distribution curve and 10 percent passing on the grain size distribution curve, as previously discussed, and 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 DD Form 1207:

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

where—
C_c = coefficient of curvature
D_{10} = 10 percent passing on the grain size distribution curve
D_{30} = 30 percent passing on the grain size distribution curve
D_{60} = 60 percent passing on the grain size distribution curve

Note. The values for 10 percent passing on the grain size distribution curve, 30 percent passing on the grain size distribution curve, and 60 percent passing on the grain size distribution curve are obtained by going to the percent passing by weight on the left vertical scale, moving horizontally across to the right until the grain-size-distribution curve is intercepted, and then moving vertically to the horizontal axis where the diameter of the material is read in millimeters (figure 5-5).

- **Step 26.** Determine the gradation by using the abbreviated information listed below. Record the information on the form. 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. The soil is poorly graded if any of the above criteria is not fulfilled. The soil is well-graded when all of the following apply:
  - The coefficient of uniformity is greater than 4 if the soil is predominantly gravel and greater than 6 if the soil is predominantly sand.
  - The coefficient of curvature is at least 1.0 but not more than 3.0 for gravel and sand.

HYDROMETER ANALYSIS

5-5. Hydrometer analysis is based on Stoke's 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.

5-6. 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 the knowledge of specific gravity of the soil particles, determines the percentage of dispersed soil particles in suspension at the time and depth of measurement.

5-7. 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 is calculated from Stokes' equation using the corrected hydrometer reading.

PURPOSE

5-8. 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. It is not recommended to run the hydrometer if the percent passing the No. 200 sieve is less than 5 percent.

EQUIPMENT

5-9. 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 *(Grain-Size Analysis [Hydrometer Method])* (figure 5-6).
- A calculator.
- Paper.
- A pencil.
- A grease pencil.
- Graph paper.
- A straightedge.
- A mechanically operated stirring device with a dispersion cup.

STEPS

5-10. Perform the following steps for hydrometer analysis:

Note. All references to form in this paragraph refer to DD Form 1794 unless noted otherwise.

- **Step 1.** Prepare the sample.
  - ***Step 1a.*** Obtain the -200 sample as prepared in the sieve analysis by collecting the material passing the No. 4 sieve (to match that of the specific gravity test) or the No. 10 sieve. The size of the sample varies according to the type of soil being tested. Approximately 100 grams are required for sandy soils, 50 grams for silty or clayey soils. Place the sample in a dish, and add distilled water until the sample is submerged.
  - ***Step 1b.*** Determine the amount and type of dispersing agent that will be used during the test as (table 5-2, page 5-12) with sodium hexametaphosphate being the most common. Record it in Blocks 9 and 10. The dispersing agents shown in table 5-2 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 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.
  - ***Step 1c.*** 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 Aug 14
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 hexameta phosphate		10. QUANTITY: 15 ml
11. COMPOSITE CORRECTION: 0.5	12. DECIMAL FINES (Block 29, DD Form 1206): 0	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	46	26	0.01284	8.8	0.0381	93	34
0932	2	43.0	44	26	0.01284	9.1	0.0274	89	33
0935	5	38.5	39	26	0.01284	9.9	0.0181	80	29
0945	15	23.5	24	26	0.01284	12.4	0.0117	49	18
1000	30	18.5	19	25	0.01293	13.2	0.0086	39	14
1030	60	15.0	16	25	0.01293	13.7	0.0062	32	12
1130	120	13.0	14	25	0.01293	14.0	0.0044	28	10
1330	240	11.0	12	25	0.01293	14.3	0.0032	24	9
0930	1440	8.5	9	24	0.01313	14.8	0.0013	18	7

WEIGHT (Grams)		
	23. DISH + DRY SOIL	325
	24. DISH	4-A
	25. DRY SOIL (W_s)	49

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)

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

Hydrometer graduated in grams per liter (152H)

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

(a = specific gravity of solids correction factor)

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

26. REMARKS
Red in color, very fine sands

FROST GROUP:

27. TECHNICIAN (Signature)	28. COMPUTED BY (Signature)	29. CHECKED BY (Signature)
PVT Butler	PVT Butler	SFC McKinney

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SAMPLE

Legend:

Aug	August
DD	Department of Defense
Dec	December
ml	milliliters
PVT	private
SFC	sergeant first class

Figure 5-6. Sample DD Form 1794

Table 5-2. Dispersing agents

Dispersing Agent	*Stock Solution*	
	Concentration	*Grams Per Liter*
Sodium hexametaphosphate	0.4N	40
Sodium polyphosphate	0.4N	36
Sodium tetraphosphate	0.4N	31
Sodium tripolyphosphate	0.4N	29

- **Step 2.** Determine the type of hydrometer. If the hydrometer scale ranges from 1.000 to 1.038, it is Type 151H and measures specific gravity of the suspension. If the scale ranges from 0 to 60, it is Type 152H and measures grams per liter of the suspension. The dimensions for both hydrometers are the same.
- **Step 3.** Determine the composite correction. 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.

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

  - ***Step 3a.*** Place about 500 milliliters of distilled water in a graduated cylinder.
  - ***Step 3b.*** Place the amount of dispersing agent that was used in step 1 in the cylinder, and mix well.
  - ***Step 3c.*** Add additional distilled water to the cylinder to reach the 1,000-milliliter mark.
  - ***Step 3d.*** 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 the reading and 1. For the Type 152H hydrometer, the composite correction is the difference between the reading and 0.
  - ***Step 3e.*** Record the composite correction in Block 11.
  - ***Step 3f.*** 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.
  - ***Step 4a.*** Record identifying information for the sample, dispersing agent, quantity used, and composite correction on the form.
  - ***Step 4b.*** Obtain the decimal fines from the original soil sample from DD Form 1206. Record it in Block 12.
  - ***Step 4c***. Obtain the specific gravity of solids of the soil sample from DD Form 1208. Record it in Block 13.

- ***Step 4d.*** Empty and thoroughly rinse the graduated cylinder containing the dispersing solution from step 3.
- ***Step 4e.*** 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.
- ***Step 4f.*** 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.
- ***Step 4g.*** 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 the cylinder while it is in the inverted position.

- ***Step 4h.*** 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.
- ***Step 4i.*** 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).
- ***Step 4j.*** 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.

- ***Step 4k.*** 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 used for hydrometer testing (Block 25).
- **Step 7**. Compute the results the form.
  - ***Step 7a.*** Obtain the corrected reading (R) by adding the actual hydrometer reading (Column 16, R1) and the composite correction (Block 11), and record the sum in Column 17.

$$R = R^1 + composite\ correction$$

where–
R=corrected reading

- ***Step 7b.*** Obtain the temperature versus specific gravity constant (K) from table 5-3. Record it in Column 19.

where–
K = specific gravity constant

Note. Although typical specific-gravity values are listed in table 5-3, 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:

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

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

Table 5-3. Values of K for use in Stokes' equation for computing particle diameter

Temp °C	Specific Gravity of Solids (Gs)								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

Legend:
C Celsius
coeff coefficient

- ***Step 7c.*** Obtain the effective depth for each corrected reading (Column 17) (table 5-4), and record it in Column 20.
- ***Step 7d.*** Determine the particle diameter corresponding to a given hydrometer reading on the basis of Stokes' equation. Enter the information in Column 21. Calculate by using the following formula:

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

w*here—*
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 5-3 (entered in Column 19)
L = distance from the surface of the suspension to the level at which the density of the suspension is being measure (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)

Table 5-4. Values of effective depth for hydrometer analysis

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

Legend:
cm centimeter
corr corrected
hydro hydrometer

- **Step 7e.** Compute the partial percent finer. Record the results in Column 22a. 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:

Hydrometer 151H

$$partial\ percent\ finer = \frac{G_s}{G_s - 1} \times \frac{100{,}000}{W_s} \times (R - 1.0)$$

Hydrometer 152H

$$partial\ percent\ finer = \frac{R \times a}{W_s} \times 100$$

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 5-4, page 5-15) to be applied to the reading for hydrometer 152H*

- ***Step 7(f).*** Compute the total percent finer for each hydrometer reading and record it on the form using the formula—

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

RESULTS PRESENTATION

5-11. Plot the grain size distribution on DD Form 1207 using the particle diameters (grain size [in millimeters) and the total percent finer (percent passing), and connect the plotted points with a smooth curve. Read the curve on the form and determine if 3 percent or more of the particles are smaller than 0.02 millimeters in diameter; if so, the soil is frost-susceptible. Frost-susceptible soils are listed in four groups in the order of increasing susceptibility (table 5-5).

Table 5-5. 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 (b) Sands (e < 0.30) (c) Sands (e > 0.30)	0 - 3 0 - 3 3 to 10	GW and GP SW and SP SP
S-1	(a) Gravels (e < 0.25) crushed stone or rock (b) Gravelly soils	0 - 3 3 - 6	GW and GP GW, GP, GW-GM, and GP-GM, GW-GC and GP-GC
S-2	Sandy soils (e < 0.30)	3 - 6	SW, SP, SW-SM, and SP-SM, SW-SC and SP-SC
F-1	Gravelly soils	6 - 10	GW-GM, GP-GM, and GW-GC, GP-GC
F-2	(a) Gravelly soils (b) Sands	10 - 20 6 - 15	GM, GC and GM-GC SM, SC, SW-SM, and SP-SM, SW-SC, SP-SC, and SM-SC
F-3	(a) Gravelly soils (b) Sands, except very fine silty sands (c) Clays, PI > 12	>20 > 5	GM, GC and GM-GC SM, SC, and SM-SC CL, CH, and ML-CL

Table 5-5. Frost-susceptibility groups for typical soil types (continued)

F-4	(a) All silts (b) Very fine sands (c) Clays, PI < 12 (d) Varved clays and other fine-grained, banded sediments	>15	ML, MH, and ML-CL SM, SC, and SM-SC CL and ML-CL CL or CH layered with ML, MH, SM, SC, SM-SC, or ML-CL
$e=G_sY_w/Y_d-1$			
Legend:			
mm	millimeter		
NFS	not frost susceptible		
PI	plasticity index		
USCS	Unified Soil Classification System		

5-12. Soils in group F-4 have high frost susceptibility. Record the frost susceptibility group for the soil type in Block 27.

5-13. This curve can be used to determine the coefficient of uniformity and the coefficient of curvature.

5-14. The data on DD Form 1794 is plotted on DD Form 1207 to give such a curve for a mixed soil. For this soil, the diameter corresponding to 60 percent passing is 0.5 millimeters. The diameter corresponding to 10 percent passing is 0.0045 millimeters. Hence, the coefficient of uniformity is as follows:

$$Cu = \frac{D_{60}}{D_{10}} = \frac{0.5000}{0.0045} = 111.11$$

where—
C_u = coefficient of uniformity

5-15. The diameter for 30 percent passing is 0.024 millimeters. Thus, the coefficient of curvature is as follows:

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

where—
C_c = coefficient of curvature

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Chapter 6

Liquid Limit, Plastic Limit, and Plasticity Index Determination

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 and the shrinkage limit are called 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.

Note. 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 tests on samples of natural moisture content. Determine the moisture content at the end of the test.

LIQUID LIMIT DETERMINATION

6-1. The LL of a soil 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.

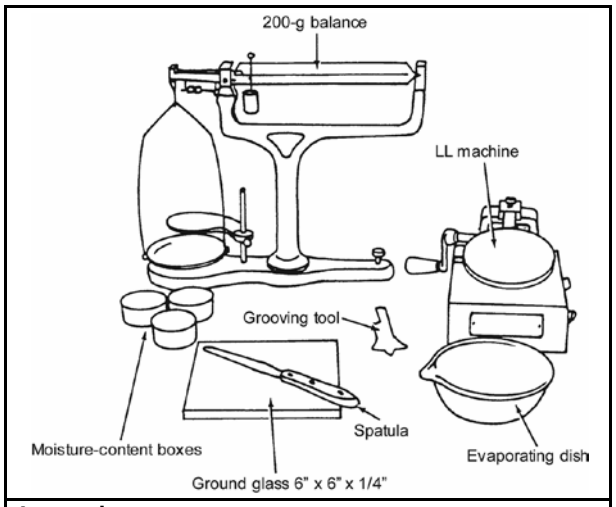
PURPOSE

6-2. Perform the LL determination test to assist in classifying the soil by determining the LL from three moisture content samples.

EQUIPMENT

6-3. Perform the LL determination test in a laboratory environment using the following items (figure 6-1):

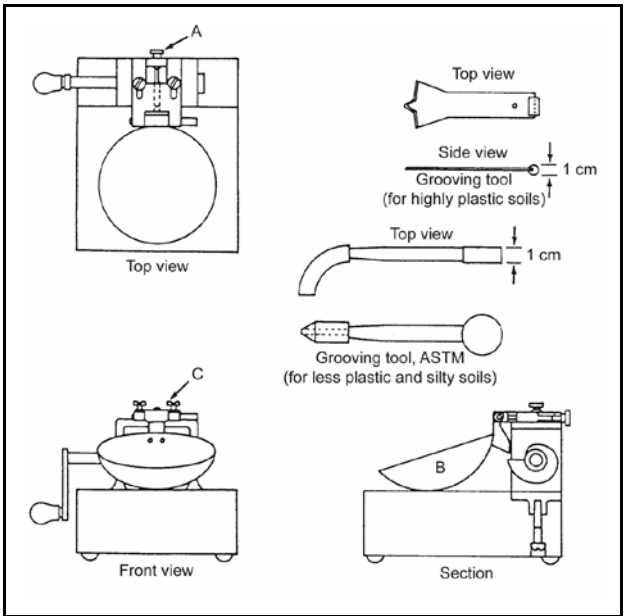
- A balance scale sensitive to 0.01 gram.
- A LL device (also known as a casagrande cup) and a grooving tool (figure 6-2).
- A No. 40 sieve.
- Pudding pans.
- A ground-glass plate (at least 30 square centimeters by 1 centimeter thick) for mixing soil and rolling PL threads.
- A plastic bag.
- A calculator.
- Moisture-determination tares.
- Paper.
- A pencil.
- A grease pencil.
- DD Form 1209 *(Liquid- and Plastic- Limits Determination).*
- Gummed labels.
- A spatula.
- A straightedge.
- A mortar with a rubber-tipped pestle.
- A laboratory oven.
- Heat-resistant gloves.
- Distilled water.



Legend:

g	gram
LL	liquid limit

Figure 6-1. Equipment for the LL and PL tests



Legend:
cm centimeter

Figure 6-2. LL device with grooving tools

STEP 1. PREPARE THE SOIL SAMPLE

6-4. Perform the following substeps to complete step 1:

- **Step 1a.** Sieve the soil sample (at natural moisture content) over the No. 40 sieve to obtain a sufficient quantity of at least 250 grams.
- **Step 1b.** Perform the following steps if little or no material is retained on the No. 40 sieve; otherwise, go to step 1c:
  - ***Step 1b(1).*** Collect 200 to 250 grams of -40 material for testing.
  - ***Step 1b(2).*** Mix the material with distilled water until the water content is slightly below the LL (peanut butter consistency). The goal is to have the material fall in the 25- to 35-blow range for the first test.
  - ***Step 1b(3).*** Place the mixture in a plastic bag, making it airtight for at least 16 hours (overnight) so that the moisture content can become consistent throughout the sample. Remix the material thoroughly before testing.
- **Step 1c.** Perform the following steps if material is retained on the No. 40 sieve:
  - ***Step 1c(1).*** Place the –40 material in a plastic bag, making it airtight to maintain its natural moisture content.
  - ***Step 1c(2).*** Soak the coarse material retained on the No. 40 sieve (the soaking time is variable).
  - ***Step 1c(3).*** Rub the colloidal material from the surfaces of the large particles until they are clean, placing the fines in suspension.
  - ***Step 1c(4).*** Pour off the suspended fines slowly into another pan, being careful not to pour off the coarse material.
  - ***Step 1c(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.

- ***Step 1c(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.
- ***Step 1c(7).*** Oven dry the coarse material that has been soaked and washed.
- ***Step 1c(8).*** Sieve the oven-dried coarse material over the No. 40 sieve.
- ***Step 1c(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.
- ***Step 1c(10).*** Place the combined mixture in a plastic bag, making it airtight for at least 16 hours (overnight) so that the moisture content can become consistent throughout the sample. Remix the material thoroughly before testing.

STEP 2. INSPECT THE LIQUID LIMIT DEVICE BEFORE TESTING

6-5. Perform the following substeps to complete step 2:

- **Step 2a.** Ensure that the pin connecting the cup is not worn (which would permit side play).
- **Step 2b.** Ensure that the screws connecting the cup to the hanger arm are tight.
- **Step 2c.** Check the cup for wear. If a groove has developed from use, replace it.
- **Step 2d.** 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.
- **Step 2e.** Check the grooving tool for wear.
- **Step 2f.** 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.
  - ***Step 2f(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.

Example. Step 2f(1) could be used to aid in checking and adjustments.

  - ***Step 2f(2).*** Attach the cup to the device, and turn the crank until the cup is raised to its maximum height.
  - ***Step 2f(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 contacted, the height of drop is approximately correct. If not, adjust the cup until simultaneous contact is made.
  - ***Step 2f(4).*** Check the adjustment by turning the crank at two 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.
  - ***Step 2f(5).*** Remove the tape after adjustments.

STEP 3. PERFORM THE LIQUID LIMIT TEST

6-6. Perform the following substeps to complete step 3:

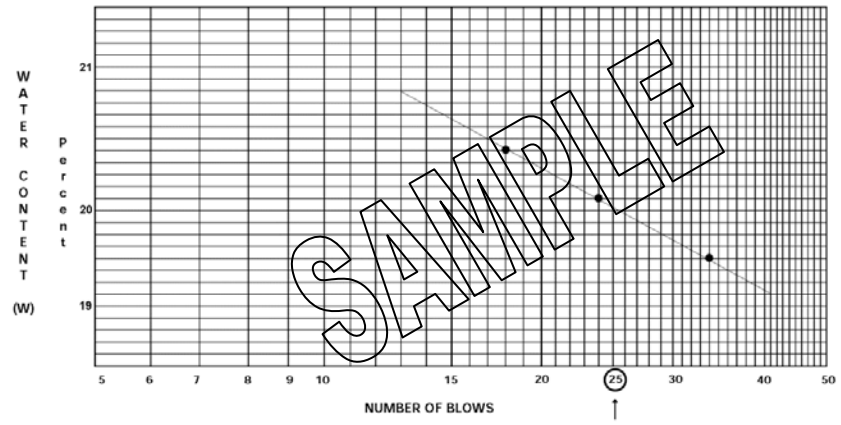
- **Step 3a.** Obtain about 50 grams of the 200- to 250-gram, prepared sample; and place in an airtight container for use in the PL test.
- **Step 3b.** Record identifying information for the sample on DD Form 1209 (figure 6-3).

LIQUID- AND PLASTIC-LIMITS DETERMINATION

1. PROJECT			2. DATE
Engineer Center Expansion			6 Aug 14
3. EXCAVATION NUMBER	4. JOB NUMBER		5. SAMPLE NUMBER
5-C	16-P-T		5-C-1

6. LIQUID LIMIT, LL

RUN NUMBER	1	2	3			
TARE NUMBER	1	2	3			
a. WEIGHT OF WET SOIL + TARE	31	24	25			
b. WEIGHT OF DRY SOIL + TARE	29	22	24			
c. WEIGHT OF WATER $W_w = a - b$	2	1	1			
d. WEIGHT OF TARE	17	15	16			
e. WEIGHT OF DRY SOIL $W_s = b - d$	11	7	7			
WATER CONTENT $w = \frac{W_w}{W_s} \times 100$	19	20	20			
NUMBER OF BLOWS	34	24	18			



7. PLASTIC LIMIT, PL

RUN NUMBER	5	6	7	8		
TARE NUMBER	5	6	7	8		
a. WEIGHT OF WET SOIL + TARE	24	23	24	24		8.3
b. WEIGHT OF DRY SOIL + TARE	23	23	23	23	Mean =	9.3 (+/- 1)
c. WEIGHT OF WATER $W_w = a - b$	0	1	1	1		10.3
d. WEIGHT OF TARE	17	16	17	17		
e. WEIGHT OF DRY SOIL $W_s = b - d$	6	7	6	6		
WATER CONTENT $w = \frac{W_w}{W_s} \times 100$	7	10	10	10		
PLASTIC LIMIT, PL (Average w)	10					

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)	10. COMPUTED BY (Signature)	11. CHECKED BY (Signature)
SGT Glasper	SGT Glasper	SSG Dobbs

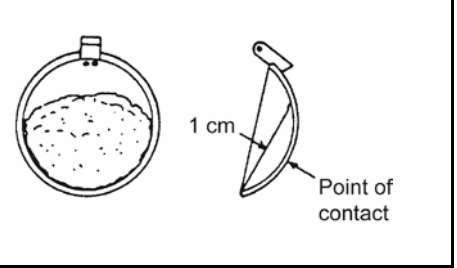
DD FORM 1209, DEC 1999 PREVIOUS EDITION IS OBSOLETE. Reset Adobe Professional 7.0

Legend:

Aug	August
DD	Department of Defense
Dec	December
LL	liquid limit
PI	plasticity index
PL	plastic limit
SGT	sergeant
SSG	stafff sergeant

Figure 6-3. Sample DD Form 1209

- **Step 3c.** Label and preweigh three empty moisture determination tares. Record the weight on the form as the weight of the tare.
- **Step 3d.** 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 (figure 6-4, page 6-6).



Legend:
cm centimeter

Figure 6-4. Leveling sample in the cup

- **Step 3e.** 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 (figure 6-5). 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.

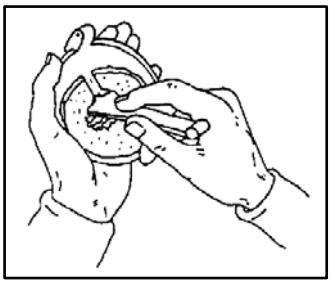


Figure 6-5. Holding cup and grooving tool

- **Step 3f.** Replace the soil sample in the cup, and regroove it if 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 (figure 6-6).

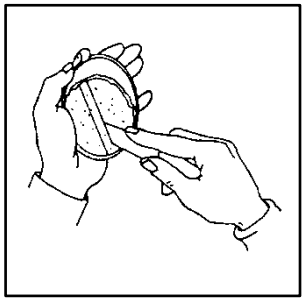
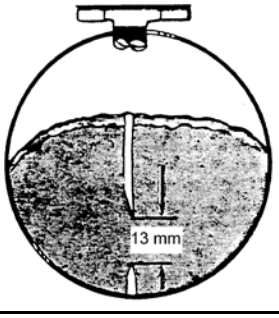


Figure 6-6. Cutting groove with spatula in sandy soil

- **Step 3g.** Attach the cup to the device, and ensure that the height of the drop is 1 centimeter.
- **Step 3h.** 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 (figure 6-7).
- **Step 3i.** Record the number of blows to close the groove for 13 millimeters.



Legend:
mm millimeter

Figure 6-7. Soil coming into contact

- **Step 3j.** 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 cover it with a lid. Weigh it, and record the weight on the form as the weight of the wet soil and the tare.
- **Step 3k.** 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.

- **Step 3l.** 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.
- **Step 3m.** 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.
- **Step 3n.** Compute the weight of the water 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.
- **Step 3o.** Compute the weight of the dry soil by subtracting the weight of the tare from the weight of the dry soil and the tare. Record the weight on the form.
- **Step 3p.** Record the water content for each specimen by using the following formula:

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

where—
w = moisture content of the soil (as a percentage)
W_w = weight of the water
W_s = weight of the dry soil

Note. Weighing should be accurate to 0.01 gram, and water contents should be computed in rounded to the nearest tenth percent.

- **Step 3q.** Plot the water content points on the semilog graph on DD Form 1209 (water versus number of blows), and draw a straight line (flow line) representative of the three or more points.
- **Step 3r.** Determine the LL by interpreting the graph where the flow line intersects the 25-blow line. Record the LL to the nearest whole number.

FALL CONE

6-7. The fall cone is provided as an alternative to the casagrande cup (LL device) technique for determining the LL. The fall cone is an international standard and is a common device used outside the United States. The fall cone also provides more reproducible results than the casagrande cup technique. It is easier to maintain in correct adjustment, involves easier calibration steps, and requires less judgment on the part of the operator to assess the soil state.

PURPOSE

6-8. Perform this test to determine the moisture content of a soil at its LL to within ± 10 percent of the casagrande cup value.

EQUIPMENT

6-9. Perform the fall cone test in a laboratory environment using the following items (figure 6-8):

- A balance scale sensitive to 0.01 gram.
- A fall cone device with stainless steel or duralumin cone tips (30° angle).
- A gauge plate for calibration.
- A flat, glass plate.
- One or more metal cups with a diameter of 55 ± 2 millimeters and a height of 40 ± 2 millimeters
- A wash bottle.
- A No. 40 sieve.
- Pudding pans.
- A calculator.
- Moisture-determination tares.
- Paper.
- A pencil.
- A grease pencil.
- DD Form 1209.
- A spatula.
- A straightedge.
- A suitable device for drying soil (oven, microwave).
- Heat-resistant gloves.
- Distilled water.



Figure 6-8. Essential equipment for a fall cone test

STEP 1. PREPARE THE SOIL SAMPLE

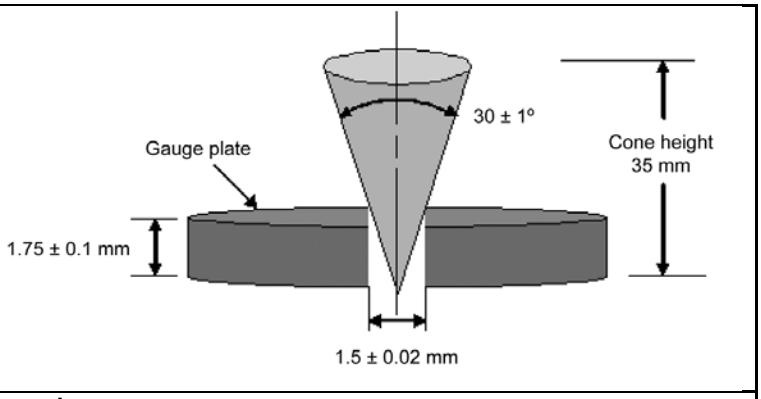
6-10. Perform the following substeps to complete step 1:

- **Step 1a.** Sieve the soil sample (at natural moisture content) over the No. 40 sieve to obtain a sufficient quantity of at least 500 grams.
- **Step 1b.** Perform the following steps if little or no material is retained on the No. 40 sieve; otherwise, go to step 1c:
  - ***Step 1b(1).*** Collect 300 grams of -40 material for testing.
  - ***Step 1b(2).*** Mix the material with distilled water until the water content is slightly below the LL (a peanut butter consistency). The material falls in the 25- to 35-blow range for the first test.
  - ***Step 1b(3).*** Place the mixture in a plastic bag, making it airtight for at least 16 hours (overnight) so that the moisture content can become consistent throughout the sample. Remix the material thoroughly before testing.
- **Step 1c.** Perform the following steps if material is retained on the No. 40 sieve:
  - ***Step 1c(1).*** Place the -40 material in a plastic bag, making it airtight to maintain its natural moisture content.
  - ***Step 1c(2).*** Soak the coarse material retained on the No. 40 sieve (the soaking time is variable).
  - ***Step 1c(3).*** Rub the colloidal material from the surfaces of the large particles until they are clean, placing the fines in suspension.
  - ***Step 1c(4).*** Pour off the suspended fines slowly into another pan, being careful not to pour off the coarse material.
  - ***Step 1c(5).*** Add clean water to the coarse material, and repeat the wash process until the water poured off is sufficiently clear to indicate that most fines that were put in suspension have been poured off.
  - ***Step 1c(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.
  - ***Step 1c(7).*** Oven-dry the coarse material that has been soaked and washed.
  - ***Step 1c(8).*** Sieve the oven-dried coarse material over the No. 40 sieve.
  - ***Step 1c(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.
  - ***Step 1c(10).*** Place the combined mixture in a plastic bag, making it airtight for at least 16 hours (overnight) so that the moisture content can become consistent throughout the sample. Remix the material thoroughly before testing.

STEP 2. INSPECT THE CONE DEVICE BEFORE TESTING

6-11. Perform the following substeps to complete step 2:

- **Step 2a.** Ensure that the tip angle has not been compromised (use the gauge plate provided with the fall cone with the specifications given in figure 6-9, page 6-10. The compromise would impede penetration of the cone resulting in too great a LL. Check for a worn tip, and if so, replace with a new cone to ensure a smooth penetration into the soil, if necessary.
- **Step 2b.** Ensure that the cone drops freely when released from the locked position.



Legend:
mm millimeter

Figure 6-9. British fall cone calibration tolerances

STEP 3. PERFORM THE LIQUID LIMIT TEST

6-12. Perform the following substeps to complete step 3:

- **Step 3a.** Mix the 300-gram paste in (step 1) for 10 minutes, using the spatula to ensure that the soil is well mixed.

Note. Highly plastic clays may require longer mixing times (upwards of 40 minutes) to achieve repeatable results.

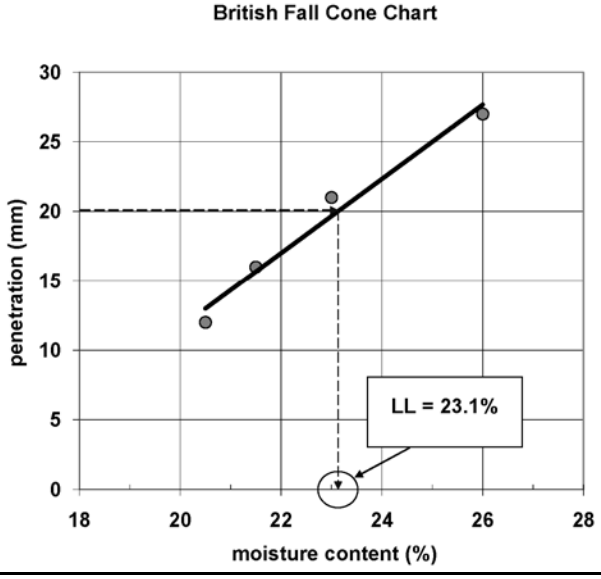
- **Step 3b.** Push a portion of the soil paste into the tin cup with a spatula or knife, taking care to not entrap air within the soil. Strike-off the excess soil above the rim of the cup with a straightedge.
- **Step 3c.** With the penetration cone locked and in the raised position, lower the supporting assembly until the tip of the cone touches the soil surface.
- **Step 3d.** Lower the stem of the dial gauge to contact the cone shaft, and take an initial height reading of the gauge to the nearest 0.1 millimeters.
- **Step 3e.** Release the cone for 4 to 6 seconds. Lock the cone in position, being careful not to jerk the apparatus and force the cone down further.
- **Step 3f.** Lower the stem of the dial gauge onto the cone shaft, and record the final height to the nearest 0.1 millimeters.
- **Step 3g.** Compute the change in height by using the following formula:

$$\Delta H = H_i - H_f$$

Where—
ΔH *= change in height*
H_i *= initial height reading*
H_f *= final height reading*

- **Step 3h.** Lift the cone carefully to avoid scratching.
- **Step 3i.** Add a little more soil to the cup to fill in the void created by the cone, taking care to not trap air, and relevel the surface with a straightedge.
- **Step 3j.** Repeat steps 3d through 3h.
- **Step 3k.** Change in height is not more than 0.5 millimeters if the difference between the two height readings record the average of the two penetrations. Go to step 3n.
- **Step 3l.** Conduct a third test if the difference between the two height readings is between 0.5 and 1.0 millimeter. If the difference between the lowest and highest readings between all three tests is still between 0.5 and 1.0 millimeter, take the average of the three tests and continue to step 3n.

- **Step 3m.** Remove the soil from the cup, and remix the soil paste with remaining soil until a better consistency is created if the difference between the two height readings is greater than 1.0 millimeter. It may be necessary to add additional distilled water at this time. Rerun the test beginning at step 3b.
- **Step 3n.** Take approximately 10 grams of soil from the penetrated area of the sample cup, and dry it to obtain the moisture content.
- **Step 3o.** Remove the soil from the cup, remold it with remaining soil, and dry it by spreading it across a flat surface.
- **Step 3p.** Repeat the test (steps 3b through 3o), trying to obtain a smaller penetration value indicating a stiffening of the soil after removing some of the moisture. Continue drying the soil and repeating the test until a value of 15 millimeters or less penetration is achieved. Ideally, four moisture content penetration points will have been obtained.
- **Step 3q.** Plot the data (figure 6-10).



Legend:	
LL	liquid limit
mm	millimeter

Figure 6-10. British fall cone data plot examples

- ***Step 3q(1).*** Identify whether the apparatus used has British or Chinese origins.
- ***Step 3q(2).*** Create an arithmetic plot with the y-axis being values of penetration and the x-axis being values of moisture content for the British fall cone device.
- ***Step 3q(3).*** Create a logarithmic plot with the y-axis being values of penetration and the x-axis being values of moisture content for the Chinese fall cone device.
- ***Step 3q(4).*** Draw a best-fit, straight line through the data points plotted on the graph.

Note. The moisture content at the LL is the point at which the penetration reaches 20 millimeters for the British cone or 17 millimeters for the Chinese cone.

- ***Step 3q(5).*** Record the moisture content at the LL to the nearest whole percentage value.

PLASTIC LIMIT DETERMINATION

6-13. The PL of a soil is the water content (as a percentage) of the weight of oven-dried soil at which the soil begins to crumble when rolled into a thread 3.2 millimeters (1/8 inch) 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

6-14. Perform the PL test to assist in classifying the soil by determining the PL moisture content to within ± 1 percent.

EQUIPMENT

6-15. Perform the PL test in a laboratory environment using the same equipment listed in the LL determination test (paragraph 6-3).

STEPS

6-16. 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 that was set aside during 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 (1/8 inch), taking no more than 2 minutes (figure 6-11).

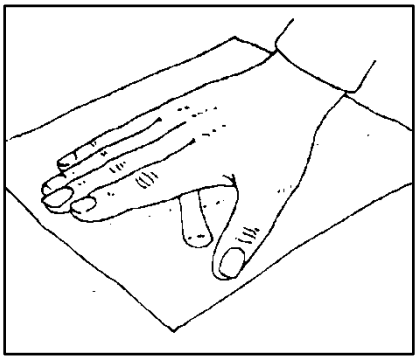


Figure 6-11. Rolling a soil specimen

Note. The rate of rolling should be between 80 and 90 strokes per minute, counting a stroke as one complete motion of the hand forward and back to the starting position. The 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 in diameter, repeating the rolling and remolding process until the sample crumbles before reaching the 3.2-millimeter thread (figure 6-12).

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.

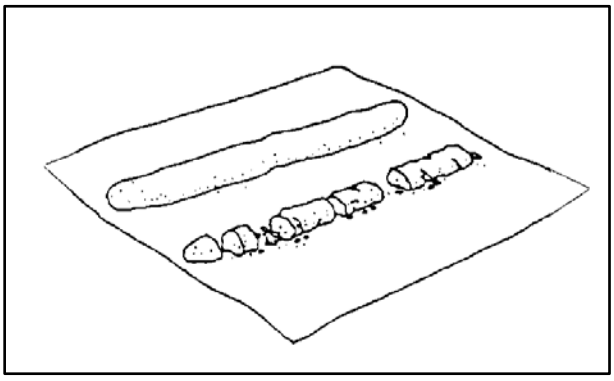


Figure 6-12. Rolled threads, crumbled and uncrumbled

- **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.
- **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 weight 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 (paragraph 6-6) 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.

PLASTIC LIMIT DEVICE

6-17. As an alternative to the manual hand-rolling method of conducting a PL, an ASTM-accepted test device is available from commercial manufacturers, known as the PL device (figure 6-13). This device streamlines the test process by using specific size soil samples to roll a thread and a calibrated box with a lip that ensures that a 3.2-millimeter-diameter thread is rolled.

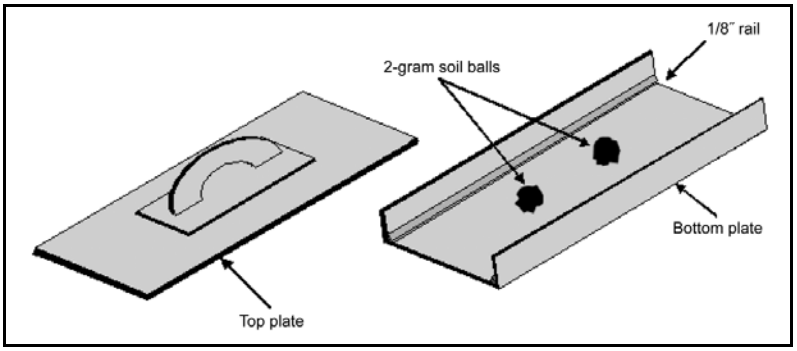


Figure 6-13. PL device components

EQUIPMENT

6-18. Use the following items in addition to the those specified in paragraph 6-3:

- PL device.
- Self-adhesive PL paper.

Note: The plastic limit roller alleviates the need for a glass plate and the use of hands to conduct the physical rolling of the soil sample. The self adhesive paper provides friction for rolling the soil masses and a drying effect to help decrease moisture content with continued rolling.

STEPS

6-19. Perform the following steps to determine the PL with the PL device:

- **Step 1.** Place one sheet of adhesive-backed PL paper on the bottom plate and one on the top plate of the PL device.
- **Step 2.** Reduce the moisture content of the soil to the approximate PL (where the soil is nonsticky).
- **Step 3.** Select four or five, 1.5-gram to 2-gram portions of soil.
- **Step 4.** Roll each portion of soil into an ellipsoidal mass.
- **Step 5**. Place one or two ellipsoidal masses about 25 to 40 millimeters apart on the bottom plate. Experience has shown that best results occur with one or two masses.
- **Step 6**. Lower the top plate gently onto the top of the soil masses, and begin gently rolling down the soil portions using a back-and-forth motion as shown in figure 6-14.

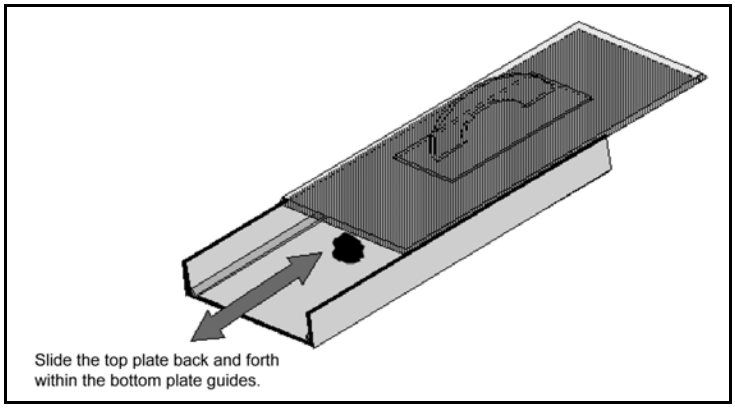


Figure 6-14. Operation of PL device

- **Step 7**. When the top plate comes into contact with 3.2-millimeter rail, check to see if the soil is at the PL.
- **Step 8**. Collect the threads and knead them together into a ball and repeat steps 3 through 7 if the soil is not at the PL. Repetitive rolling on the paper will also help dry out the soil if it is too wet.
- **Step 9**. Pour the threads off the bottom plate into a moisture tare container when the soil is at the PL, and weigh it immediately.
- **Step 10.** Determine the average water content of the samples, and record it to the nearest tenth as the PL. When determining the average water content, the individual tests must be within ± 1 percent of the mean. Individual tests that do not meet this requirement will not be used. If none of the individual tests meet this requirement, additional testing is required.

Note. To ensure smooth rolling of the threads, replace the PL paper if it becomes torn or if significant amounts of soil become stuck to the paper as to prevent.

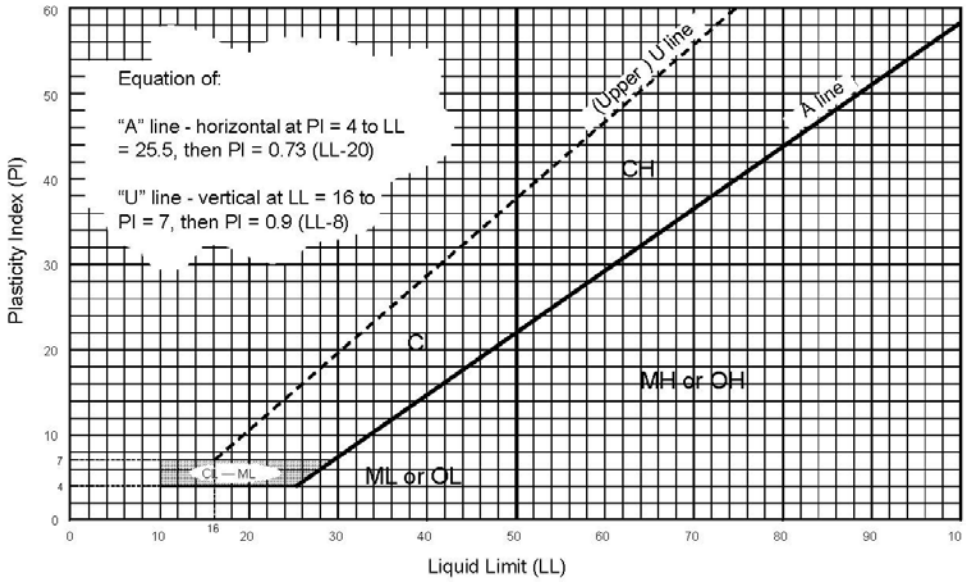
PLASTICITY INDEX DETERMINATION

6-20. Compute the PI and record it on DD Form 1209 using the following formula:

$$PI = LL - PL$$

6-21. Classify the soil by plotting the LL versus the PI on the plasticity chart as follows (figure 6-15):

- 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, and the material plotted to the left of the 50 percent line has a low LL.
- The upper line is an approximate upper boundary. Any results plotted above this line should be considered suspect, and the tests should be rechecked.



Legend:

CH	high-plasticity clay
CL	low-plasticity clay
LL	liquid limit
MH	high-plasticity silt
ML	low-plasticity silt
OH	high-plasticity soil
OL	low-plasticity organic silt
PI	plasticity index

Figure 6-15. USCS plasticity chart

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Chapter 7

Laboratory Compaction Characteristics of Soil Using Modified Effort

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 amount 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 density (dry unit weight) of the soil. Usually, soil can be compacted best and 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

7-1. In the field, compaction is accomplished by rolling or tamping with special equipment or by the passing of construction equipment. Laboratory compaction is usually 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.

7-2. Testing standards exist for varying compaction energies, but to simplify the procedure, only the compaction effort known as modified proctor is discussed. This is considered a high-energy compaction effort suitable for airfield and highway horizontal constructions and embankment and foundation preparation. ASTM D1557 defines the criteria for this procedure and is summarized in the following text.

7-3. A lower-compaction energy procedure can be found in ASTM D698, which requires slightly lighter laboratory equipment. It is not discussed here and, in some cases, may be suitable for less critical structures or when the equipment or environment prohibit the compaction of soil to high densities.

7-4. Before performing the compaction test, the grain size analysis must be determined (see chapter 5). This test method provides three alternative testing procedures. The procedure used is 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 test [see chapter 9]) and the material gradation.

PURPOSE

7-5. 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. Use the following procedures:

- **Procedure A.** This procedure uses a 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 a 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 a 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 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 D1557. 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 (compaction effort [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. If greater than 30 percent of the soil weight is retained on the 3/4-inch sieve, refer to ASTM D4718 for preparing the sample and correcting the measured moisture content and dry density.

EQUIPMENT

7-6. The following items are used for the compaction test:

- Cylinder molds (which one depends on the soil sample being processed).
  - ***Protctor mold.*** The proctor mold should be 4 inches with a 4.0-inch inside diameter, 4.584-inch inside height, 0.0333-cubic-feet internal volume, and 2.375-inch extension collar, and detachable metal baseplate.
  - ***CBR mold.*** The CBR mold should be 6 inches with a 6-inch inside diameter, 7-inch inside height, 2-inch extension collar and detachable metal baseplate. The mold should also have a metal spacer disk that is 2.416 inches thick with a 5.94-inch inside diameter and. The metal spacer disk is used 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) is 0.075 cubic feet.
- A compacting hammer or tamper. This hammer should be a sliding-weight type compacting tamper, with a 2-inch diameter steel-striking face, 10-pound mass, and 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 *(Laboratory Compaction Characteristics of Soil [Compaction Test])*.
- DD Form 1211 *(Laboratory Soil-Compaction Test Graph).*

7-7. 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:

- **Procedures A and B.** Use about 35 pounds of dry soil or at least 50 pounds of moist soil (approximately one 5-gallon bucket would be a sufficient volume).
- **Procedure C.** Use about 75 pounds of dry soil or at least 100 pounds of moist soil. (approximately two 5-gallon buckets would be a sufficient volume).

STEPS

7-8. Perform the steps in the following paragraphs to complete the compaction test:

Step 1. Determine the Test Procedure to be Used

7-9. Perform the following substeps to complete step 1:

- **Step 1a.** Do not use this method if the CBR design and tests are to be developed for the project (see chapter 9).
- **Step 1b.** Determine the test procedure by evaluating the gradation criteria of the procedures listed in the preceding paragraph.

Step 2. Prepare the Soil Sample

7-10. Perform the following substeps to complete step 2:

- **Step 2a.** 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, if the temperature of the sample does not exceed 60°C.
- **Step 2b.** Break up the sample thoroughly, but not in such a manner as to reduce the size of the individual particles.
- **Step 2c.** Sieve the sample over a No. 4 (paragraph 7-4, Procedure A), 3/8-inch (paragraph 7-4, Procedure B), or 3/4-inch sieve (paragraph 7-4, 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.
- **Step 2d.** Separate five equal portions from the sample 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 from the sample.

Step 3. Adjust the Water Content

7-11. Perform the following substeps to complete step 3:

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.

- **Step 3a.** Establish the assumed or approximate OMC.
  - ***Step 3a(1).*** Place exactly 100 grams of the excess soil sample in a dish.
  - ***Step 3a(2).*** Add 5 milliliters of water to the sample, and mix it 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 pieces shattering when bent. Usually, this will be slightly less than the PL.
  - ***Step 3a(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 the 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 8 milliliters of water was added to achieve the approximate OMC, the approximate OMC is 8.0 percent. This works in estimating because 1 milliliter of water is equal to about 1 gram. By adding weight to an original sample of 100 grams, no mathematical calculations are required.
  - ***Step 3a(4).*** Determine the moisture content range. This range is the approximated OMC ± 4. For example, if the approximate OMC is 8.0 percent, 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.
  - ***Step 3a(5).*** Use the following formula to determine the amount of water to add to each of the samples to obtain the desired, approximate moisture contents (-4, -2, OMC, +2, +4):

$$water\ (in\ milliliters) to\ add\ =$$

$$weight\ of\ sample\ (in\ grams)\ x\ desired\ percent\ (decimal\ fraction)$$

Example. To determine the amount of water to add to obtain the approximate OMC for a sample of 6,804 grams (using Procedure C), use the following calculation:

$$6{,}804\ x\ 0.08\ =\ 544.3\ milliliters$$

Perform the same calculations to determine the amount of 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/exactly the same

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

(-2) 6.0% $moisture\ for\ a\ sample\ at$ 6,800 $grams$: $6{,}815\ x\ 0.06\ =\ 408.0\ milliliters$

$(+2)$ 10.0% $moisture\ for\ a\ sample\ at$ 6,822 $grams$: $6{,}815\ x\ 0.10\ =\ 682.2\ milliliters$

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

  - ***Step 3a(6).*** Add the water figured from the formulas for each of the desired moisture contents (-4, -2, OMC, +2, and +4) and mix thoroughly to ensure the even distribution of water throughout the sample.
  - ***Step 3a(7).*** Place each sample in an airtight container and allow to stand for the minimum period of time for GW, GP, SW, and SP soil types, there is no minimum standing period of time for GM and SM soil type. 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 Identifying Information

7-12. Record identifying information (Blocks 1 through 7) on DD Form 1210 (figure 7-1).

Step 5. Prepare the Mold and Moisture Determination Tares

7-13. Perform the following substeps to complete step 5:

Note. All references to form in this paragraph refer to DD Form 1210.

- **Step 5a.** Lightly oil a mold (4- or 6-inch, depending on the procedure selected).
- **Step 5b.** Weigh the mold.

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: 6 Aug 14			
				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 (0 MC)		4-3 (-2)		5-3 (-4)					
12. WEIGHT OF MOLD + WET SOIL	Grams	11,953		12,043		11,609		11,598		11,834					
13. WEIGHT OF MOLD	Grams	7,517		7,496		7,017		7,119		7,591					
14. WEIGHT OF WET SOIL (12 - 13)	Grams	4,436		4,547		4,592		4,479		4,243					
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	308	316	313	326	333	249	273	235	258				
b. WEIGHT OF TARE + DRY SOIL	Grams	273	274	287	285	301	310	235	258	225	246				
c. WEIGHT OF WATER, W_w (a - b)	Grams	35	34	29	29	24	24	14	14	10	12				
d. WEIGHT OF TARE	Grams	62	62	61	61	62	61	61	61	62	62				
e. WEIGHT OF DRY SOIL, W_s (b - d)	Grams	211	212	226	224	239	249	175	197	163	184				
f. WATER CONTENT, $w = \frac{W_w}{W_s} \times 100$ (c / e x 100)	Percent	17	16	13	12	10	10	8	7	6	7				
17. AVERAGE WATER CONTENT	Percent	16		13		10		8		6					
18. DRY UNIT WEIGHT, $\gamma d = \frac{\gamma wet}{1+(w/100)}$	Pcf	112.0		118.7		122.8		122.4		117.3					

SAMPLE

19. REMARKS: Select material

* 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)
SPC Blest	SPC Blest	SSG Dobbs

DD FORM 1210, DEC 1999 PREVIOUS EDITION IS OBSOLETE. Reset Adobe Professional 7.0

Legend:

Aug	August
DD	Department of Defense
Dec	December
in	inch
lb	pound
SPC	specialist
SSG	staff sergeant

Figure 7-1. Sample DD Form 1210

- ***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.
- ***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.

- **Step 5c.** Attach the collar to the mold. If using the 6-inch mold, place a coarse filter paper on top of the spacer disk.
- **Step 5d.** Record the volume of the mold as 0.0333 cubic feet for the 4-inch mold or 0.075 cubic feet for the 6-inch mold with the spacer disk.
- **Step 5e.** Mark and weigh two moisture determination tares for each mold prepared. Record as the weight of tare.

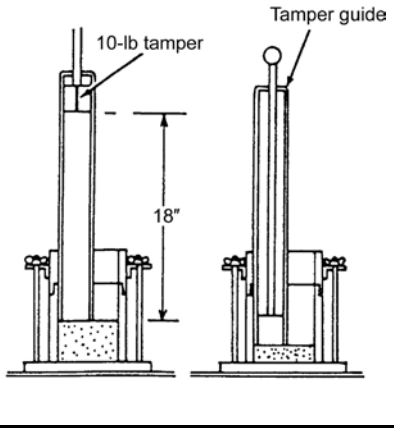
Step 6. Place Soil in the Mold

7-14. Place sufficient soil in the mold (about 1 1/2 to 2 inches) to obtain a 1-inch (approximate) compacted layer.

Step 7. Apply Compactive Effort

7-15. Perform the following substeps to apply compactive effort:

- **Step 7a.** Hold the 10-pound compaction tamper within 5° of vertical, placing its face on top of the soil.
- **Step 7b.** Raise the handle until it reaches the top (18 inches), and release it, allowing the weight to fall freely onto the soil.
- **Step 7c.** 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 (figure 7-2).



Legend:
lb pound

Figure 7-2. Soil compaction in a mold

Step 8. Trim the Compacted Layer

7-16. 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. Use the knife to roughen the compacted surface by striking shallow lines across its surface to simulate field scarification, creating a stronger bond between lifts.

Step 9. Repeat Steps 6, 7, and 8

7-17. 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.

Note. Do not perform step I on the fifth layer.

Step 10. Remove the Collar From the Mold

7-18. Perform the following steps to complete step 10:

- **Step 10a.** Cut around the inside edge of the collar to prevent shearing the compacted soil when removing the collar.
- **Step 10b.** Trim and smooth the compacted soil flush with the top of the mold (figure 7-3). 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 scrape the surface with the straightedge.

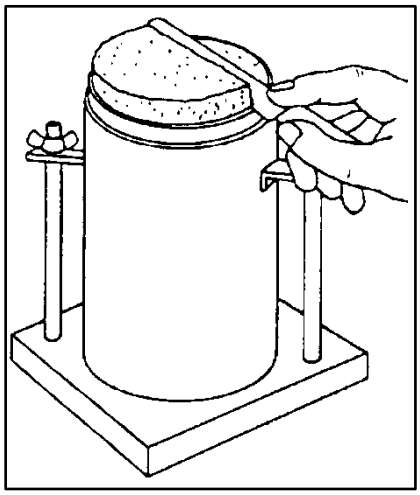


Figure 7-3. Trimming the compacted soil sample

Note. If the soil surface is indented below the lip of the mold (embedded hammer imprints), the recorded density will be too low and the test should be rerun. If rocks or materials are left above the surface of the mold, the recorded density will be too high.

Step 11. Weigh and Record Data

7-19. 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

7-20. Perform the following substeps to complete step 12:

- ***Step 12a.*** Remove the specimen from the mold.
- ***Step 12b.*** Slice the compacted specimen axially through the center, remove about 250 grams of material from one side of the cut, place it in one of the moisture determination tares, and cover it. Remove about 250 grams from the other side of the cut, place it in the other moisture determination tare, and cover.

Step 13. Repeat Steps 5 through 12

7-21. Repeat steps 5 through 12 for the remaining molds at the different moisture contents described in step 3a(5).

Step 14. Determine the Moisture Content of the Material in the Moisture Determination Tares

7-22. Determine the moisture content of the material in the moisture determination tares by performing moisture content testing of the tares as described in chapter 3.

Step 15. Perform Calculations

7-23. Record the weight of the dry soil and tare on the form. Compute the weight of water, weight of the dry soil, and the water content; and record them 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 using the following formula:

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

where—
w = moisture content

GRAPH PRESENTATION OF RESULTS

7-24. 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

7-25. The compaction curve is obtained by plotting moisture content versus dry unit weight for each test on the soil compaction test graph (figure 7-4). 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.

7-26. 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 content at which the MDD can be obtained. After plotting the compaction curve, it is possible to determine the moisture content, which will provide the MDD for that particular soil directly from the plotted curve.

7-27. 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.

7-28. The compaction curve also shows the 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 (figure 7-4); in this case, 123.2 pounds per cubic foot.

7-29. 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

7-30. 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 of 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.

7-31. 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 is equal to 100 percent. The zero air voids curve (100 percent saturation) for the soil tested is shown on the graph in figure 7-4. This curve is obtained by plotting dry densities that correspond to complete saturation at different moisture contents using the following formula:

$$w = S \times \left[\frac{\gamma_w}{\gamma_d} - \frac{1}{G_s}\right]$$

where—

w = moisture content of the soil (as a percentage)
S = degree of saturation (as a percentage)
γ_w = unit weight of water (62.43 pounds per cubic foot)
γ_d = dry unit weight of soil (in pounds per cubic foot, grams, or centimeters)
G_s = specific gravity of solids for this soil (DD Form 1211, Block 6) (chapter 4)

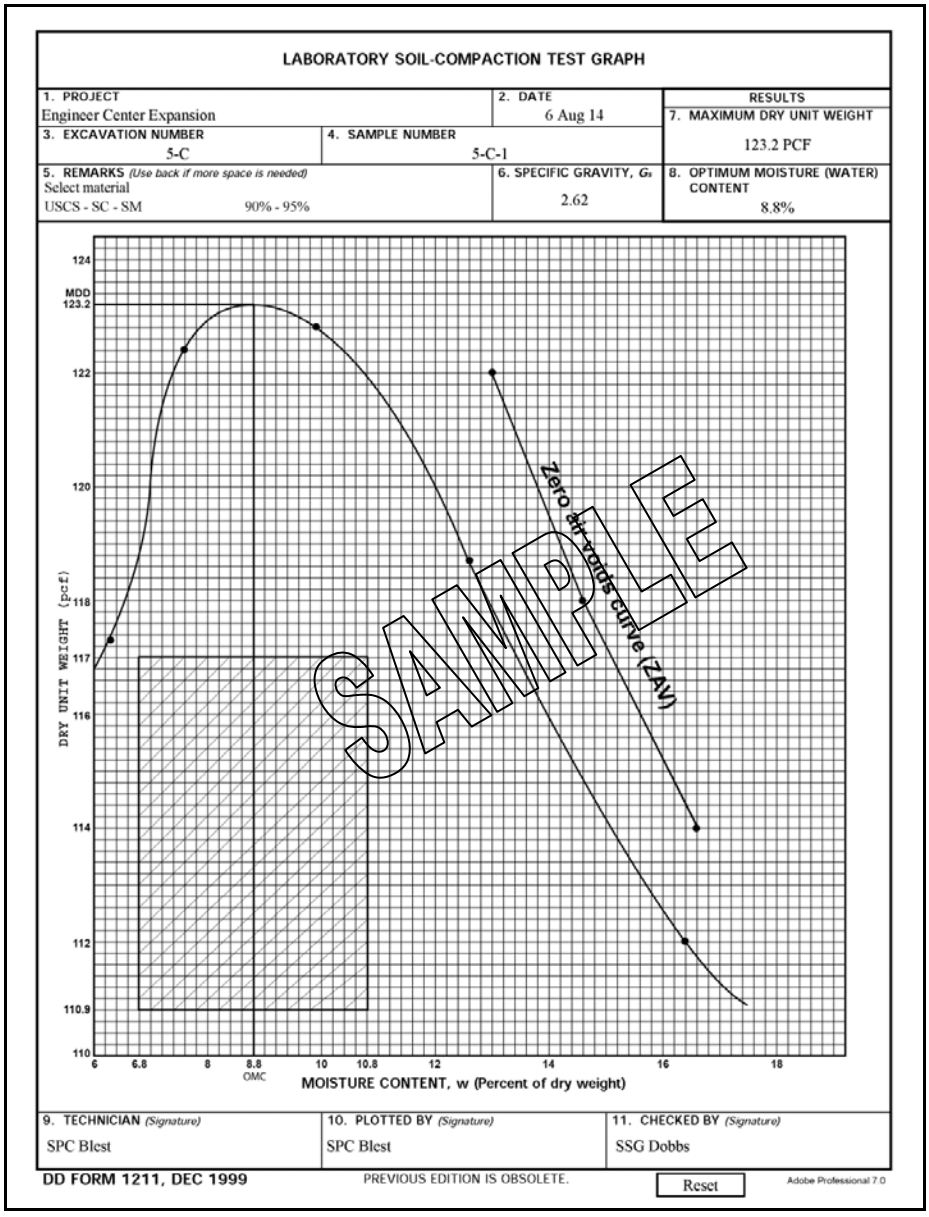


Figure 7-4. Sample DD Form 1211

Legend:	
Aug	August
DD	Department of Defense
Dec	December
MDD	maximum dry density
OMC	optimum moisture content
pcf	pound per cubic foot
SPC	specialist
SSG	staff sergeant
USCS	Unified Soil Classification System
ZAV	zero at voids curve

Figure 7-4. Sample DD Form 1211 (continued)

7-32. Use the equation above 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 them as shown above

Example. The upper-plotted point of the zero air voids curve for the soil represented in figure 7-4 is a whole number just under maximum density (122 pounds per cubic foot). The specific gravity of solids 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 using the zero air voids formula (paragraph 7-30) as follows:

$$100 \times \left(\frac{62.43}{122} - \frac{1}{2.62}\right) = 13.0$$

Other plotted points are also determined using the following formula:

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

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

PERCENT MOISTURE

7-33. To obtain the maximum dry unit weight or density in the field, it is necessary to maintain the construction moisture content of the soil 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 content 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 7-4, 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

7-34. 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 loads. 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.

COMPACTION SPECIFICATION BLOCK

7-35. 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 TM

3-34.64/MCRP 3-17.7G; FM 5-430-00-1/AFJPAM 32-8013/MCRP 3-17.7A, Volume I; or FM 5-430-00-2/AFJPAM 32-8013/MCRP 3-17.7B, Volume II. Once the range is determined, the specification block is plotted on the compaction curve, covering the specified compaction range requirement in percent for the dry unit weight within a 4 percent range of the OMC (± 2). 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

7-36. Use the following items 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

7-37. 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 percent of the OMC. The example in figure 7-4, page 7-9, represents the compaction data from DD Form 1210, 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 117.0 to 110.9 pounds per cubic foot 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 pounds per cubic foot) and the specification range of 90 to 95 percent by using the following formula:

$$123.2\ x\ 0.95\ =\ 117.0\ 123.2\ x\ 0.90\ =\ 110.9$$

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

WATER DENSITY EFFECT

7-38. Figure 7-5 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.

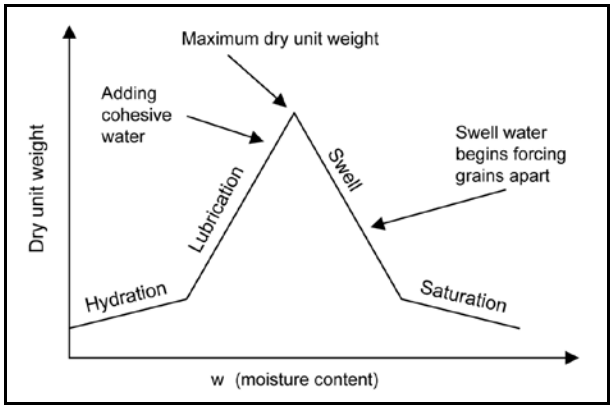


Figure 7-5. Effect of water on density

7-39. 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 dry unit weight of the soil. 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.

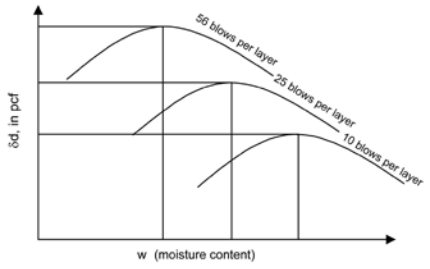
7-40. The first portion of the compaction curve to the right of OMC is known as the swell phase. The addition of water increases the film around the soil particles, forcing the soil particles apart and decreasing the dry density. With further increases in moisture content, free water added to the soil 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.

EFFECTS OF COMPACT EFFORTS ON DENSITY

7-41. 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.

7-42. 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 7-6 shows how the dry density varies with the compactive effort.

7-43. 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 (figure 7-6).

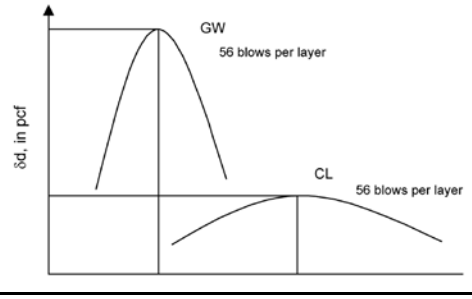


Legend:
pcf pound per cubic foot

Figure 7-6. Effect of different compactive effort on density

EFFECTS ON SOIL DENSITY

7-44. 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 (figure 7-7). 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.



Legend:
pcf pound per cubic foot

Figure 7-7. Effect of different soil types on density

COMPACTION EQUIPMENT

7-45. 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 (TM 3-34.62/MCRP 3-17.7I).

7-46. 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.

OTHER COMPACTIVE EFFORTS

7-47. 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 D1883. 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.

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Chapter 8

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. Stress deformation characteristics of the soil are directly related to the moisture content and density of the soil, 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. Moisture contents are compared with a specified range of moisture bounding 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 density and moisture content of the soil. 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 (sand, ball bearings, oil, water) and computing the volume of soil removed. The moisture content and density of the soil 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

8-1. The sand cone or sand displacement method may be used in fine- or coarse-grained materials. Calibrated sand is used to determine the volume of the hole from which a sample has been taken.

8-2. 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. 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.

8-3. 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

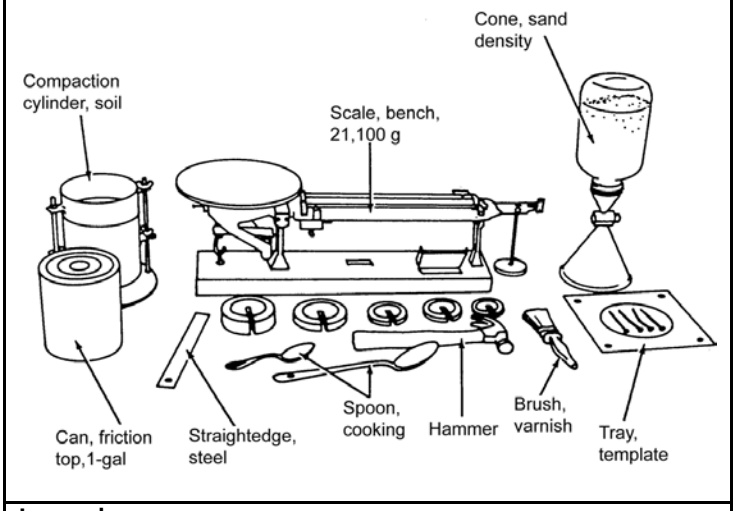
8-4. Perform the sand cone or sand displacement test to determine the in place density of a soil or to validate other in-situ density test devices.

EQUIPMENT

8-5. Use the following items to perform this test in a field environment (figure 8-1):

- 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, twenty-penny.
- 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 calcium carbide pressure moisture test set.
- 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.
- A sand recovery bucket (such as a 5-gallon bucket)

- DD Form 1215 *(In-Place Density Determination—Sand-Cone Method).*
- Paper.
- A pencil.
- A calculator.
- A grease pencil.
- Moisture determination tares.



Legend:

g	gram
gal	gallon

Figure 8-1. Sand displacement method equipment

STEPS

8-6. Perform the following steps for the sand cone or sand displacement method:

Note. All references to form in this paragraph refer to DD Form 1215 (figure 8-2, page 8-4) unless noted otherwise.

- **Step 1.** Determine the sand's density.
  - ***Step 1a.*** Weigh the mold, baseplate, spacer disk, and filter paper. Record this weight (in grams) on Line 8.
  - ***Step 1b.*** Attach the collar to the mold and place the filled sand cone apparatus, with the valve closed, on top of the mold.
  - ***Step 1c.*** 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.
  - ***Step 1d.*** Close the valve when the sand stops running into the mold and the collar.
  - ***Step 1e.*** Remove the sand cone and collar carefully from the mold.
  - ***Step 1f.*** Use a straightedge to strike off the excess sand remaining in the top of the mold.
  - ***Step 1g.*** Brush off any sand adhering to the outside of the mold.
  - ***Step 1h.*** Weigh the mold full of sand (in grams), and record it on Line 7.
  - ***Step 1i.*** Determine the weight (in grams) of the material by subtracting Line 8 from Line 7, and record the weight on Line 9.
  - ***Step 1j.*** Enter the known volume of the mold (in cubic feet) on Line 10.

IN-PLACE DENSITY DETERMINATION - SAND-CONE METHOD

1. PROJECT: ENGINEER CENTER EXPANSION					2. DATE: 2 APR 2010			
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 lb = 453.6 gm; 1 cu ft = 1728 cu in; 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)/122)*	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%		
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)	35. COMPUTED BY (Signature)	36. CHECKED BY (Signature)
SPC Carr	SPC Carr	SSG Dobbs

DD FORM 1215, DEC 1999 PREVIOUS EDITION IS OBSOLETE.

SAMPLE

Legend:

Apr	April
cc	cubic centimeter
cm	centimeter
cu ft	cubic foot
cu in	cubic inch
DD	Department of Defense
Dec	December
gm	gram
in	inch
lb	pound
MDD	maximum dry density
OMC	optimum moisture content
pcf	pound per cubic foot
SPC	specialist
SSG	staff sergeant

Figure 8-2. Sample DD Form 1215

- ***Step 1k.*** 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. Determine the unit weight of material by using the following formula:

$$unit\ weight\ of\ material\ (Line\ 11) = \frac{\left(\frac{weight\ of\ sand}{453.6}\right)}{volume\ of\ mold}$$

- ***Step 1l.*** Repeat steps 1a through 1k at least two more times. The unit weight of sand used in the calculations is 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.
  - ***Step 2a.*** Clear the overburden, and seat the template tray flush on the surface. Fasten it in place with nails.
  - ***Step 2b.*** Seal the spaces on the inside edge under the template using soil from the preparation site.
- **Step 3.** Determine the surface calibration.
  - ***Step 3a.*** Weigh the sand cone apparatus filled with sand. Record the weight (in grams) on Line 13.
  - ***Step 3b.*** Turn the sand cone apparatus over with the valve closed, and place it on the template.
  - ***Step 3c.*** 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.
  - ***Step 3d.*** Remove the sand cone apparatus from the template, and weigh it. Record this weight (in grams) on Line 14.
  - ***Step 3e.*** Determine the weight of the sand in the cone by subtracting Line 14 from Line 13. Record the difference on Line 15.
- **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.
  - ***Step 5a.*** Predetermine the weight of a friction top can. Record this weight on Line 29.
  - ***Step 5b.*** 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.
  - ***Step 5c.*** 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.
  - ***Step 5d.*** Weigh the wet soil and can. Record the weight on Line 28.
- **Step 6.** Determine the volume of the hole.
  - ***Step 6a.*** 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.
  - ***Step 6b.*** Turn the sand cone apparatus over with the valve closed, and place it on the template.
  - ***Step 6c.*** 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 the requirements for hole volume, ensure that the jars with sand have been properly weighed will be on hand during the testing procedures. Enter the initial weight of the additional jar plus sand is included on Line 16 and the final weight of the additional jar on Line 17.

- ***Step 6d.*** Close the valve when the sand stops running, and remove the sand cone apparatus. Weigh it, and record this weight on Line 17.
- ***Step 6e.*** Determine the weight of the material released by subtracting Line 17 from Line 16. Record this difference on Line 18.
- ***Step 6f.*** Determine the weight of the material in the hole by subtracting Line 15 from Line 18. Record this difference on Line 19.
- ***Step 6g.*** Recover as much sand as possible from the hole.
- ***Step 6h.*** 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:

$$volume\ of\ hole\ (Line\ 20) = \frac{\frac{weight\ of\ sand\ in\ hole}{453.6}}{average\ unit\ weight\ of\ sand}$$

- **Step 7.** Determine the average water content of the soil removed from the hole (step 5) using the oven laboratory method or calcium carbide pressure moisture tester, and record it on Line 27. If using the calcium-carbide-pressure 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 chapter 3.
- **Step 8.** Determine the unit weight (density).
  - ***Step 8a.*** 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.
  - ***Step 8b.*** 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. Use the following formula to compute the wet density:

$$wet\ density\ (Line\ 31) = \frac{\frac{weight\ of\ wet\ soil}{453.6}}{volume\ of\ hole}$$

  - ***Step 8c.*** Compute the dry unit weight (density), and record it on line 32. Use the following formula to compute the dry density:

$$dry\ density\ (Line\ 32) = wet\ unit\ weight \times \frac{100}{100 + average\ water\ content\ (\%)}$$

STEEL SHOT VOLUME REPLACEMENT METHOD

8-7. The steel shot volume replacement method is appropriate for expeditionary type military operations where the shipment of nuclear density gauges is prohibitively difficult or where the sand cone method is prohibitively slow, or requires too much user training and experience.

8-8. The steel shot volume replacement method may be used in fine-grained and sandy soils with a maximum particle size of 1/2 inches or less. The soil must have physical and mechanical properties that permit the excavation of a stable cavity with a volume on the order of 0.04 to 0.05 cubic feet. Consequently, this test method is not applicable for granular soils that are saturated or loosely deposited. The relatively conservative limitation on particle size is necessary because the volume of the cavity from which soil is removed is smaller than the volume of cavities for the sand cone test.

8-9. The steel shot volume replacement test consists of digging out a sample of the soil for which density must be estimated, determining the volume of the cavity, and determining the wet and dry weights of the sample of soil. Stainless steel balls, all with the same diameter of 3/16 inches are used to estimate the volume of the cavity from which a sample of soil has been taken. The balls are uniform in size in order to leave no opportunity for changing gradations, which could occur with segregation. The recommended balls are American Iron and Steel Institute (AISI) Type 440C as specified by ASTM A276.

8-10. The steel shot replacement method is simpler and quicker than the sand cone method because the steel shot is much less susceptible to changes in densification with changes in methods of placement. Therefore, the procedure can be more lenient in terms of how the balls are placed. In addition, the packing

of the balls in a graduated cylinder is similar to the packing of the balls in a soil cavity. Therefore, the volume of balls used to fill a soil cavity is estimated directly from the volumetric marks on a graduated cylinder.

PURPOSE

8-11. Perform this test to determine the in-place density of a soil to within 2 percent, or ± 2.5 pounds per cubic foot for soils with an average density of 125 pounds per cubic foot. There is no apparent bias in the steel shot test so the average measured soil density over multiple replicates will approach the same average measured soil density as found by the sand cone method.

EQUIPMENT

8-12. Use the following items to perform this test in a field environment (figure 8-3):

- Straightedge, 12-inch length (to smooth the ground surface for testing).
- A 12-inch-square aluminum template with a 4-inch-diameter hole, raised edges (0.5 inches), and a 3/16-inch hole near each corner for the purpose of using nails to secure the plate to the ground. The plate should be 1/8 inch thick.
- Nails, twenty-penny four or more (for securing the aluminum template).
- 80 pounds of steel shot with a loose unit weight of 284 pounds per cubic foot.
- Balance scale sensitive to 1.0 gram with a 5-kilogram capacity (not included in the photo).
- Hot plate for drying steel shot (not included in the photo).
- Plastic, graduated cylinders with a capacity of 1,000 milliliters and markings each 10 milliliters.
- Chisel, 9-inch (for digging the hole).
- Hammer, 3-pound (to be used with the chisel).
- Stainless-steel scoop, flat bottom, 3-inch scoop length, 9 inches overall (for digging the hole).
- Metal spoon, serving style, 13-inch (for digging the hole).
- Spatula, 3/4- by 4-inch blade (for trimming edges of cavity in the soil).
- Paint brush, 2 1/2 inches, soft bristle (for cleaning the hole and template surface).
- Metal bowl, 9 1/2-inch-diameter (for collecting and weighing soil).
- Stainless-steel scoop, round bottom, 5-inch scoop length, 9 inches overall (for pouring balls into the graduated cylinder).
- Straightedge, 8-inch (for leveling balls with the top surface of the aluminum template).
- Magnetic wand for retrieving steel shot from the cavity in soil.
- No. 8 sieve (for cleaning balls).
- Metal pan, 9-inch-diameter (for drying balls).
- Data sheets.



Figure 8-3. Steel shot volume replacement equipment with traveling case

STEPS

8-13. Perform the following steps for the steel shot volume replacement method:

- **Step 1.** Select a location/elevation that is relatively level and representative of the soil to be tested.
  - ***Step 1a.*** Use a flathead shovel or the 12-inch straightedge to prepare the surface of the soil at the location to be tested. The surface of the soil should be a level plane.
  - ***Step 1b.*** Seat the baseplate on the plane surface, ensuring that there is contact with the ground surface around the edge of the center hole. Secure the plate against movement by pushing or hammering a nail through each of the four 3/16-inch holes.
- **Step 2.** Dig the soil cavity.
  - ***Step 2a.*** Ensure that the test cavity volume is on the order of 0.04 to 0.05 cubic feet, which equates to digging a 4-inch-diameter cavity that is 6 inches deep. For most expedient construction operations, this depth should approximate the thickness of a compacted lift.
  - ***Step 2a.*** Dig the test cavity through the center hole in the baseplate, being careful to avoid disturbing or deforming the soil that will bound the cavity. The sides of the cavity should slope slightly inward, and the bottom should be reasonably flat or concave. The cavity should be kept as free as possible of pockets, overhangs, and sharp obtrusions since these affect the accuracy of the test. Place all excavated soil and any soil loosened during digging in a moisture-tight container that is marked to identify the test number. Take care to avoid losing any materials. Protect this material from any loss of moisture until the mass has been determined and a specimen has been obtained for a water content determination.
- **Step 3.** Estimate the volume of the cavity in the soil.
  - ***Step 3a.*** Use the round-bottom scoop to fill the graduated cylinder with steel shot up to the 1,000-milliliter mark (figure 8-4). All measurements of steel shot bulk volume will involve flattening the surface of balls in the cylinder and estimating the volume at the top surface of the balls. While filling the cylinder, it may sit vertically or be tilted slightly. The scoop should be held near the cylinder opening. The cylinder should not be subjected to excessive shaking or vibration.

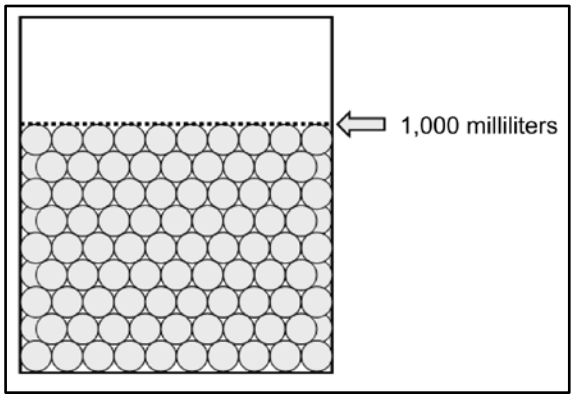


Figure 8-4. Schematic drawing of the graduated cylinder filled with balls up to the 1000 milliliter mark

  - ***Step 3b.*** Overfill the cavity slightly by pouring the steel shot from the graduated cylinder. This process will take filling the graduated cylinder twice with steel shot because the volume of the cavity should be on the order of 1,100 to 1,400 milliliters. While pouring, the open end of the graduated cylinder should be held within 3 inches of the top of the cavity. Use the 8-inch straightedge to strike off the overfilled cavity (figure 8-5) and pick up the excess steel shot with the magnet. Place these balls back into the graduated cylinder.

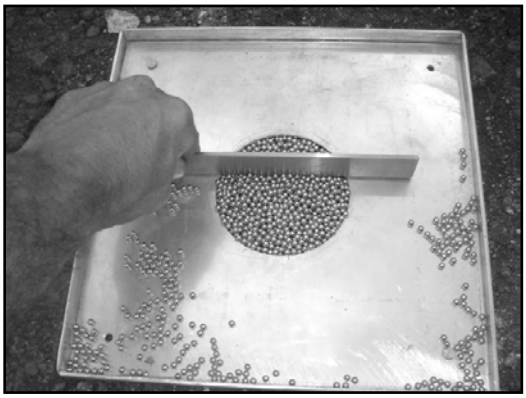


Figure 8-5. Striking off excess steel shot

- ***Step 3c.*** Flatten the top surface of the balls in the graduated cylinder and estimate the bulk volume of the remaining balls to the nearest 5 milliliters (figure 8-6). Assuming the cavity is of sufficient size and the graduated cylinder had to be filled twice use the following formula:

$$Vt\ (in\ milliliters) = 1000 + (1000 - Vb) - 25.7$$
$$Vt\ (in\ cubic\ centimeters) = Vt\ (in\ milliliters)$$

where—
Vt = total volume of the cavity
Vb = bulk volume of the remaining balls to the nearest 5 milliliters

Note. The 25.7 milliliters is subtracted to account for the volume within the thickness of the plate.



Figure 8-6. Steel shot remaining after completing the test (after the second pour)

- Retrieve the steel shot using the small scoops and/or the magnet. Balls that stayed clean during the test can be reused immediately. Balls that have touched soil need to be cleaned as described in the last section of this procedural document.

- **Step 4.** Determine the mass of soil removed from the cavity, and determine its moisture content.
  - ***Step 4a.*** Ensure that the measured mass of soil (in grams) removed from the cavity excludes the mass of the container used to transport the soil.
  - ***Step 4b.*** Mix the soil thoroughly to obtain a representative specimen for water content determination. The representative specimen should have a mass of at least 250 grams.
  - ***Step 4c.*** Determine the moisture content of the representative specimen according to ASTM D2216, ASTM D4643, ASTM D4944, or ASTM D4959. Correlations to ASTM D2216 will be performed when required by other test methods.

$$w(\%) = \frac{wet\ soil - dry\ soil}{dry\ soil} \times 100\%$$

where—
w = moisture content of soil (as a percentage)

- **Step 5.** Perform final calculations.
  - ***Step 5a.*** Using mass of soil, M_s (in grams), and total volume of cavity (in cubic centimeters), calculate the wet density of soil, ρ_s (in grams or cubic centimeters):

$$\rho_s = \frac{M_s}{V_t}$$

where—
M_s = mass of soil (in grams)
ρ_s = wet density of soil (in grams or cubic centimeters)
V_t = volume of the cavity (in cubic centimeters)

  - ***Step 5b.*** Using the wet density of soil (in grams or cubic centimeters) and the moisture content (as a percentage), calculate the dry density of soil (in grams or cubic centimeters):

$$\rho_d = \frac{\rho_s}{1 + \frac{w}{100}}$$

where—
w = moisture content of soil (as a percentage)
ρ_s = wet density of soil (in grams or cubic centimeters)
ρ_d = dry density of soil (in grams or cubic centimeters)

  - ***Step 5c.*** To convert wet or dry density to unit weight of soil use the following formula:

$$unit\ weight\ of\ soil = soil\ density \times water\ unit\ weight\ at\ room\ temperature$$

Example.

ρ_s = 1.923 grams per cubic feet
γ_s = 120 pounds per cubic feet (= 1.923 x 62.4)
ρ_d = 1.763 grams per cubic feet
γ_d = 110 pounds per cubic feet (= 1.763 x 62.4)

where—
ρ_s = wet density of soil (in grams or cubic centimeters)
ρ_d = dry density of soil (in grams or cubic centimeters)
γ_s = wet unit weight (in pounds or cubic feet)
γ_d = dry unit weight (in pounds or cubic feet)

- **Step 6.** Report information and data.
  - ***Step 6a.*** Report the test location, elevation, thickness of layer tested, or other pertinent data to locate or identify the test.
  - ***Step 6b.*** Report the test cavity volume.
  - ***Step 6c.*** Report in-place wet density or wet unit weight.
  - ***Step 6d.*** Report in-place dry density, dry density of the soil, or dry unit weight.
  - ***Step 6e.*** Report in-place moisture content of the soil.

- ***Step 6f.*** Calculate the relative density or the relative unit weight (if the in-place density or unit weight should be expressed as a percentage of a target value) using the following formula:

$$\textit{Relative density or relative unit weight} = \frac{\textit{measured value}}{\textit{target value}} \times 100\%$$

- **Step 7.** Steel balls that do not fall into contact with soil can be reused. However, any balls that do fall into contact with the soil must be cleaned prior to reuse. Cleaning the balls simply involves rinsing them with water while they rest on a sieve; a No. 8 sieve has been found to work well for this purpose (figure 8-7). A slight agitation of the balls may be required to dislodge some soil particles. Once the balls appear to be clean, they can be dried by sitting them in the sun or by using a direct heat source.



Figure 8-7. Rinsing steel shot in the No. 8 sieve

DRIVE CYLINDER METHOD

8-14. The drive cylinder density method may be used in fine- or coarse-grained materials and is especially useful in weaker soils and soils where a sand cone or steel shot replacement test cannot be used because a cylindrical hole cannot retain vertical side walls. The test method is not recommended in very hard, natural soils; heavily compacted soils; or organic soils. Soils with appreciable amounts of gravel (particles larger than 3/16 inch) may introduce significant error in the final density measurement, owing to large voids in the final extruded sample.

8-15. The drive cylinder density method consists of tamping a steel cylinder of known volume into the ground, digging the cylinder out of the ground and determining the dry weight of the sample.

PURPOSE

8-16. Perform this test to determine the in-place density of a soil to within ± 2 pounds per cubic foot.

EQUIPMENT

8-17. Use the following items to perform this test in a field environment (figure 8-8, page 8-12):

- A 3-, 4-, or 6-inch diameter drive cylinder.
- A drive head of suitable diameter to drive the cylinder.
- A drive hammer (4.5-kilogram sliding weight).
- A ruler.
- A knife.
- A steel straightedge.

- A shovel.
- A brush.
- A balance scale sensitive to 1.0 gram with a 20-kilogram capacity.
- Drying equipment for obtaining moisture content.
- A sealable container for the soil specimen.

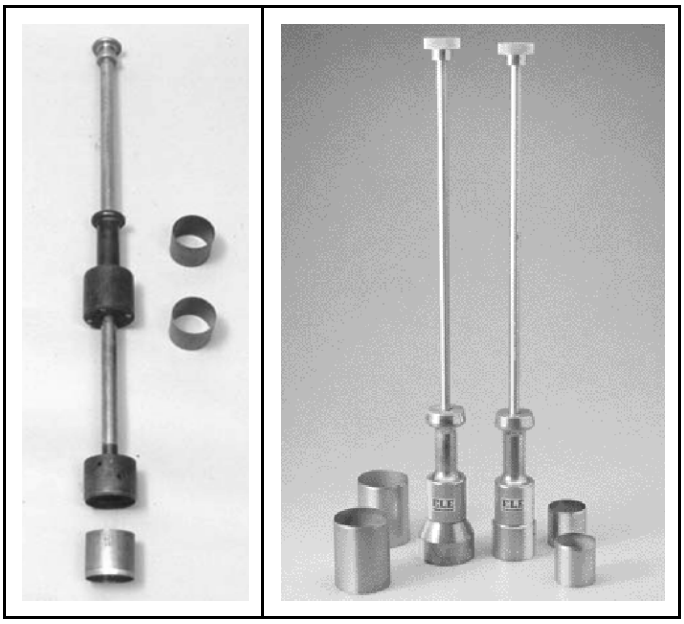


Figure 8-8. Drive cylinder components

STEPS

8-18. Perform the following steps for the drive cylinder density method:

- **Step 1.** Prepare a level surface.
  - ***Step 1a.*** Where a density test is required, obtain a fairly level surface using a shovel, straightedge, or on-site construction equipment (figure 8-9).



Figure 8-9. Preparing surface for drive cylinder test

  - ***Step 1b.*** Brush away any loose particles on the surface prior to inserting the drive cylinder.
- **Step 2.** Assemble the apparatus.
  - ***Step 2a.*** Obtain and record the mass of the empty drive cylinder.
  - ***Step 2b.*** Screw the drive head onto the drive hammer.
  - ***Step 2c.*** Insert the drive cylinder into the drive head with the sharpened edge facing the surface to be sampled.

- **Step 3.** Obtain the sample.
  - ***Step 3a.*** Drive the cylinder into the ground by raising the drop hammer and allowing it to fall or by applying a uniform force via a jack or similar device until the top of the cylinder is 1/2 inch below the original surface. Keep the drive rod steady and in a vertical position at all times (figure 8-10).



Figure 8-10. Inserting drive cylinder into soil

Note. Care should be taken to only push the cylinder as deep as its measured height (2.5 to 4.5 inches). Overdriving the sample will compress the soil within the drive cylinder and provide a false density reading. If overdriving is suspected, discard the sample and retest it.

  - ***Step 3b.*** Remove the drive head, and dig the cylinder out of the ground using a shovel, digging the soil from around the sides of the cylinder and undercutting several inches below the bottom of the cylinder before lifting it out of the ground (figure 8-11).



Figure 8-11. Removing the drive cylinder sample

Note. If the soil is noncohesive or saturated, a plate or similar level surface may need to be slid on the underside of the cylinder to prevent the leakage or soil prior to removing it from the ground.

  - ***Step 3c.*** Trim the excess soil from the end of the sample using a knife and straightedge until the sample is flush and plane with the ends of the cylinder (figure 8-12, page 8-14). Care should be taken to avoid cutting into soil contained within the cylinder.



Figure 8-12. Trimmed sample ready for weighing

Note. If the cylinder is not full or representative of the surrounding material, discard the sample and take another sample. If the drive cylinder is deformed or otherwise damaged while driving or removing the sample, obtain a new cylinder for subsequent tests and discard the current sample.

- ***Step 3d.*** Weigh the full drive cylinder and soil immediately upon completion of trimming and record the weight as mass.
- ***Step 3e.*** Extrude the soil sample, and place at least 100 grams of it into a sealable container to obtain the moisture content. Use a drying technique (oven, microwave, calcium carbide pressure moisture test) to obtain the moisture content.
- ***Step 3f.*** Calculate the dry mass of the drive-cylinder sample as follows:

$$M_3 = [(M_1 - M_2)/(100 + w)]\, x\, 100$$

where—

M_1 = mass

M_2 = mass of the empty drive cylinder

M_3 = dry mass of the drive cylinder sample

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

- Determine the volume of the drive cylinder based on its height and internal diameter.
- Calculate the dry density of the drive-cylinder sample as follows:

$$p_d = M_3/V$$

where—

p_d = dry density of the drive cylinder

M_3 = dry mass of the drive cylinder sample

V = volume of the drive cylinder

8-19. If the sample must be taken below the ground surface, auger a hole to the required depth, clean loose material from the bottom of the hole, lower the drive cylinder into the hole with sufficient rod to permit driving with a sledgehammer, and drive the cylinder until it is approximately 1 inch below the bottom surface of the augured hole. Remove the cylinder from the hole and trim it and weigh as above.

NUCLEAR MOISTURE AND DENSITY TESTER

8-20. Use this method to determine the dry density and moisture content of the soil. Individual models of equipment vary in the specific procedures, and radiation protection programs vary 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 equipment manufacturer's manual, the requirements prescribed by the Service holding the piece of equipment, and the Nuclear Regulatory Commission licensing agreement with that Service.

8-21. 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 passes 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

8-22. 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).

NONNUCLEAR DENSITY DEVICES

8-23. Currently, several manufacturers produce nonnuclear devices that are capable of measuring moisture content and soil density by measuring the resistance of electrical impulses within the soil or correlating electromagnetic field response to known soil properties.

8-24. No formal military guidance has been provided to compare the effectiveness of these nonnuclear devices to the known test methods described previously nor the limitations these nonnuclear devices have in certain soil classifications. Care should be taken by the operator when using these devices as calibration is a critical issue in obtaining useful data. It is recommended that these devices only be used for quality control when large-scale soil construction is ongoing using a uniform soil to minimize field calibration. These devices are not suitable for spot-checking density in random soils at a level sufficient for quality assurance.

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Chapter 9

California Bearing Ratio 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. 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—

- 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 existing (or those expected to occur later) soil conditions 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 that is 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 that approximates 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.

Note. The current employed standard for CBR testing is ASTM D1883.

CALIFORNIA BEARING RATIO OF LABORATORY-COMPACTED SOILS

9-1. 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 shearing resistance of the soil 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 square area piston to penetrate 0.10 inch into the soil by a standard penetration stress of 1,000 pounds per square inch. 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.

9-2. 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.

9-3. CBR values range from 0 to more than 100, depending on the type of soil and the moisture content in which they are measured. A misconception is that CBR cannot exceed 100, but in reality, some field soils in the right environment can have values exceeding 100. For the purposes of design and evaluation, the CBR will be capped at 100 even if field values exceed this.

9-4. CBR is often reported as a soaked CBR value, meaning that a prepared sample has been allowed to saturate prior to testing, providing a worst-case scenario value in case of water inundation in the field. Fine-grained soils typically have a lower CBR than coarse-grained soils, but moisture plays an important role in the final value. A wet clay can have CBRs in the 0 to 10 range, whereas a dry clay can have a CBR greater than 50. Soaked micaceous or diatomaceous silts and sands have CBRs of 15 or greater. The sand-silt-clay coarse-grained combinations range from 10 for clayey mixtures to 40 for gravelly and silty sands. Gravelly soils range from 20 for the clayey group to 80 for well-graded gravels and gravel-sand mixtures which are much less susceptible to inundation by moisture. See appendix B for the typical range for soils classified under the USCS over a wide range of compaction moisture contents.

PURPOSE

9-5. Perform this test to determine the CBR of a soil over a moisture content range of ± 4 percent of OMC in nonswelling soils and ± 8 percent of OMC in swelling soils.

EQUIPMENT

9-6. Use the following items in a laboratory environment to perform this test (figure 9-1):

- A 225-pound soil sample with known classification, PI, and OMC.
- A CBR laboratory test set consisting of—
  - Five CBR molds for each compaction effort desired.
  - Surcharge weights.
  - A penetration piston.
  - Two dial gauges reading to 0.001 inch.
- Pails.
- Plastic bags.
- 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.

Note. Asbestos gloves should not be used for any materials testing procedures. If the unit has asbestos gloves, turn them in through the supply system for proper disposal. Order heat resistant gloves to replace them.

- A pencil.
- Paper.
- A grease pencil.
- A calculator.
- DD Form 1210, DD Form 1211, and DD Form 1212 *(Laboratory California Bearing Ratio [CBR] Test Data).*
- A 3/4-inch sieve.
- A No. 4 sieve.

- Pudding pans.
- A stopwatch.
- Plans and specifications.

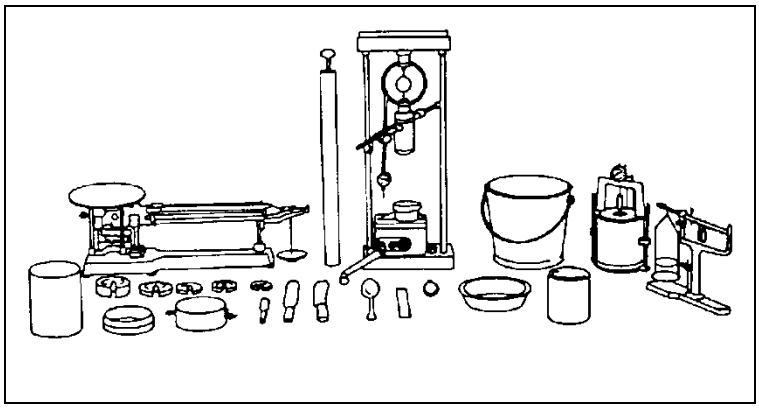


Figure 9-1. Laboratory CBR test-set apparatus

STEPS

9-7. Perform the following steps for the CBR test.

Step 1. Determine the Moisture Content Range of Investigation

9-8. Determine the moisture content range of investigation (± 4 percent of OMC) for the CBR test. If the OMC has not yet been definitively established for this soil type, perform the laboratory compaction test as described in chapter 8 to obtain the OMC.

Step 2. Prepare the Soil Sample

9-9. 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.

- **Step 2a.** 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, if the temperature of the sample does not exceed 60°C.
- **Step 2b.** Break up the sample thoroughly, but not in such a manner as to reduce the size of the individual particles.
- **Step 2c.** 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.) the amount not retained on the 3/4-inch sieve will be recorded later on DD Form 1212 (figure 9-2, page 9-4).
- **Step 2d.** 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.
- **Step 2e.** 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.

LABORATORY CALIFORNIA BEARING RATIO (CBR) TEST DATA

1. PROJECT: ENGINEER CENTER EXPANSION					2. DATE: 2 APR 2010		
3. EXCAVATION NUMBER: S-C	4. SAMPLE NUMBER: S-C-18				5. CONDITION: [X] DISTURBED [ ] UNDISTURBED		
COMPACTION DATA	6. MOLD NUMBER: 4-3 (CMC)		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	
	30 Aug 99	0930	0.00 (hr)	0.0	4.6 in	(d/e x 100)	
	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		

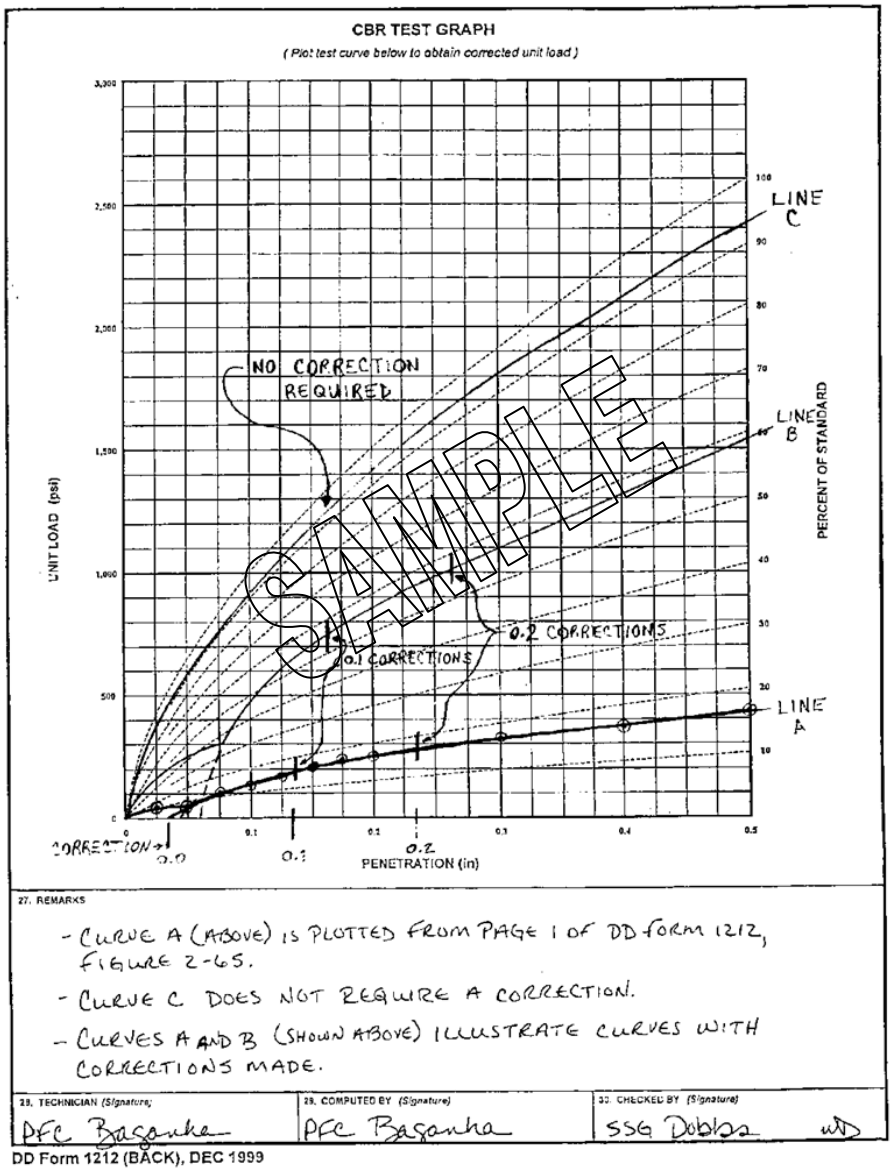
SAMPLE

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, $\gamma d = \frac{\gamma \text{ wet}}{1+(w/100)}$	Pcf	122.5		117.9	

DD Form 1212, DEC 1999 EDITION OF AUG 57 IS OBSOLETE.

Figure 9-2. Sample DD Form 1212



DD Form 1212 (BACK), DEC 1999

Legend:

Apr	April
Aug	August
CBR	California Bearing Ratio
DD	Department of Defense
Dec	December
hr	hour
in	inch
lb	pound
OMC	optimum moisture content
pcf	pound per cubic foot
PFC	private first class
psi	pound per square inch
Sep	September
SSG	staff sergeant

Figure 9-2. Sample DD Form 1212 (continued)

Step 3. Prepare and Label a 6-Inch Compaction Mold

9-10. Label a 6-inch compaction mold for each water content to be used (-4 percent, -2 percent, OMC, +2 percent, and +4 percent) at the compactive effort of 56 blows per layer as described in chapter 7.

Note. If required, prepare additional molds for each water content, using other compactive efforts of 10 and 25 blows per layer (use five molds per effort). 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

- **Step 4a.** Determine the amount of water to add to each sample using the following formula:

$$water\ to\ add\ (in\ milliliters) = weight\ of\ sample\ (grams)\ x\ (desired\ \%-floor\ moisture\ content)$$

Example. To determine the amount of water to add to a sample see the following formula:

$$6{,}800\ x\ (0.08 - 0.02) = 6{,}800\ x\ 0.06 = 408.0\ milliliters$$

where—
sample weight = 6,800 grams
OMC = 8% (0.08)
floor moisture content = 2% (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):

$6.0\%\ moisture\ for\ a\ sample\ of\ 6{,}800\ grams: 6{,}800\ x\ (0.06 - 0.02) = 272.2\ milliliters$
$4.0\%\ moisture\ for\ a\ sample\ of\ 6{,}800\ grams: 6{,}815\ x\ (0.04 - 0.02) = 136.3\ milliliters$
$10.0\%\ moisture\ for\ a\ sample\ of\ 6{,}800\ grams: 6{,}822\ x\ (0.10 - 0.02) = 545.8\ milliliters$
$12.0\%\ moisture\ for\ a\ sample\ of\ 6{,}800\ grams: 6{,}810\ x\ (0.12 - 0.02) = 681.0\ milliliters$

- **Step 4b.** Add the water figured from the formulas for each of the five desired moisture contents (-4, -2, OMC, +2, and +4), and mix thoroughly to ensure an even distribution of water throughout the sample.
- **Step 4c.** Place each sample in an airtight container, and allow it to stand for the minimum of time indicated as follows:
  - For GW, GP, SW, and SP soil types, there is no minimum standing 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 Two Moisture Determination Tares for Each Mold Prepared

9-11. For the CBR test on laboratory-compacted soil, mark and weigh two 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

9-12. 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

9-13. Place sufficient soil in the mold (about 1 1/2 to 2 inches) to obtain a 1-inch compacted layer. After compaction of all five 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

9-14. Perform the following steps to complete step 8:

- **Step 8a.** Hold the 10-pound compaction tamper within 5°, vertically, placing its face on top of the soil.
- **Step 8b.** Raise the handle until it reaches the top (18 inches) and release it, allowing the weight to fall freely onto the soil.
- **Step 8c.** 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 must be distributed evenly over the specimen's surface.

Step 9. Trim the Compacted Layer

9-15. After compacting each layer (except the fifth layer), use a knife to trim 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

9-16. 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 extends slightly into the collar, but does not exceed 1/4 inch above the top of the mold.

Step 11. Remove the Collar From the Mold

9-17. Perform the following steps to complete step 11:

- **Step 11a.** Cut around the inside edge of the collar to prevent shearing the compacted soil when removing the collar.
- **Step 11b**. 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 scrape the surface with the straightedge.

Step 12. Take Another Small Sample

9-18. Take another small sample from the remaining material after compacting each mold. Place this sample in the other moisture determination tare, and weigh it in preparation for moisture determination. Record the results on DD Form 1212.

Step 13. Place a Disk of Filter Paper on Top of the Compacted Specimen

9-19. 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 it. Raise the mold and baseplate to allow the spacer disk to slide out. Invert the mold with the baseplate, and place it flat on the table or countertop.

Step 14. Weigh and Record Data

9-20. 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 (DD Form 1212, Block 20). Do not include the collar.

Step 15. Repeat Steps 7 through 14

9-21. Repeat steps 7 through 14 for each of the compactive efforts required (10, 25, and 56 blows per layer). Compact a minimum of five molds for each compactive effort.

Step 16. Soak the Samples and Measure the Swell

9-22. Perform the following steps to complete step 16 (figure 9-3):

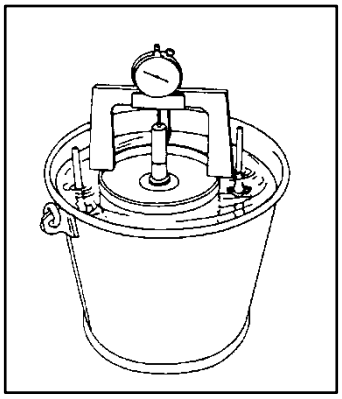


Figure 9-3. Soaking the CBR sample

- **Step 16a.** 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.
- **Step 16b.** Immerse the mold in water.
- **Step 16c.** Calibrate the adjustable stem so that the tripodal dial reads 100 and can then travel in direction. Obtain the initial dial reading, and record it on the form.
- **Step 16d.** Soak the sample for 96 hours.
- **Step 16e.** Assemble the penetration apparatus (figure 9-4).

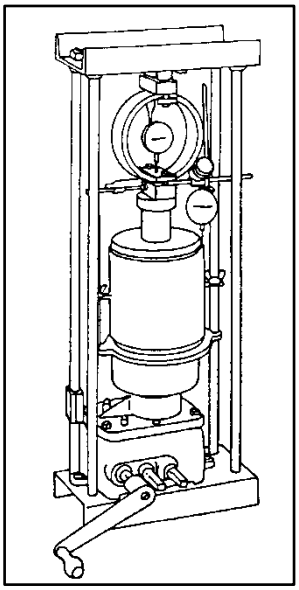


Figure 9-4. Assembled CBR test penetration apparatus

- **Step 16f.** Attach the jack to the frame.
- **Step 16g.** Attach the proving ring and dial indicator to the frame. Record the proving-ring data on the form.
- **Step 16h.** Take the dial reading at 96 hours, and record it on the form.

- **Step 16i.** Determine the amount of swell (in inches) by subtracting the initial dial reading from the final dial reading. Record the data on the form.
- **Step 16j.** Determine the percent of swell, and record it on the form using the following formula:

$$\frac{swell\ (in\ inches)\ (at\ 96\ hours)}{original\ sample\ height} \times 100$$

Step 17. Drain the California Bearing Ratio Mold

9-23. Perform the following steps to complete step 17:

- **Step 17a.** 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.
- **Step 17b.** 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.
- **Step 17c.** Determine the mass, and record it on the form as the weight of the soaked sample.
- **Step 17d.** Return a 5-pound surcharge weight to the specimen.

Step 18. Perform the California Bearing Ratio Penetration

9-24. Perform the following steps to complete step 18:

- **Step 18a.** Prepare the components.
  - ***Step 18a(1).*** Place the mold on the jack.
  - ***Step 18a(2).*** Attach and adjust the piston to the jack, and zero the dial indicator.
- **Step 18b.** 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.
  - ***Step 18b(1).*** Replace the remainder of the surcharge weights required for the mold.
  - ***Step 18b(2).*** 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.
  - ***Step 18b(3).*** Turn both dial indicators to 0.
- **Step 18c.** Apply the load. Perform the following steps to complete step 18c:
  - ***Step 18c(1).*** Crank the jack to lower the piston at a rate of 0.05 inch per minute.
  - ***Step 18c(2).*** Read the proving-ring dial indicator when the piston has reached the 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.
  - ***Step 18c(3).*** Record the proving-ring dial readings on the form.
- **Step 18d.** Determine the average moisture of the soaked samples.
  - ***Step 18d(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.
  - ***Step 18d(2).*** Weigh the tare, and record on the form.
  - ***Step 18d(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.
  - ***Step 18d(4).*** Weigh the tare and record on the form.
  - ***Step 18d(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

9-25. Perform the following steps to complete step 19:

- **Step 19a.** 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 if 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.
- **Step 19b.** Determine the unit load (in pounds per square inch) 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.
- **Step 19c.** Determine the corrected unit load (in pounds per square inch) by plotting (on the reverse side of the form) the unit load (in pounds per square inch) 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 9-3. 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.
- **Step 19d.** Calculate the CBR (in percent) for penetration at 0.100 and 0.200 inches using the following formula:

$$\frac{corrected\ unit\ load}{standard\ unit\ load} \times 100$$

Example. Since the standard unit load for each penetration is given (Block 19b), use the following formulas for each penetration:

$$CBR\ for\ 0.100 = \frac{corrected\ unit\ load}{1{,}000} \times 100$$

$$CBR\ for\ 0.200 = \frac{corrected\ unit\ load}{1{,}500} \times 100$$

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 the 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 Block 33

9-26. Ensure that information concerning the soil sample is indicated in Block 33.

UNDISTURBED SAMPLE TESTING

9-27. 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 the latter condition, duplicate samples should be tested to determine the correction necessary for the in-place tests. The reduction that occurs from 4 days of soaking is applied as a correction to the field in-place test.

9-28. 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 are 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.

9-29. 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).

CALIFORNIA BEARING RATIO DATA PRESENTATION AND ANALYSIS

9-30. 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

9-31. The test program for nonswelling soils applies to the majority of soils used in construction. 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 9-1.

Table 9-1. 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.
Legend: CE compaction effort CBR California Bearing Ratio	

Example

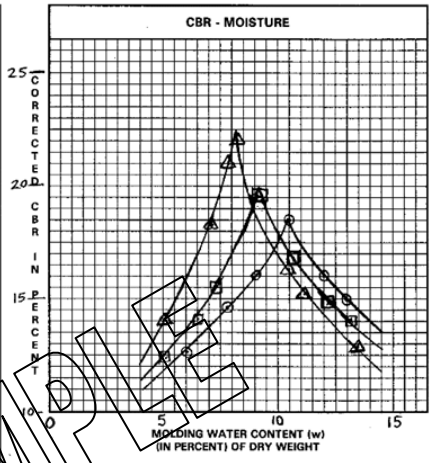
9-32. To illustrate the methods of evaluating the design CBR, the data given on the DD Form 2463 *(California Bearing Ratio [CBR] Analysis)* for the Engineer Center expansion road (figure 9-5, page 9-12) will be used. The data was taken from the subgrade along a proposed road alignment. The object of 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. It is better to evaluate other density limits that meet the minimum requirements to see if an adjustment to these limits yields greater strengths.

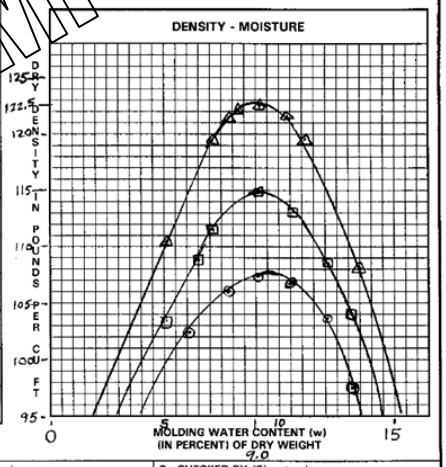
CALIFORNIA BEARING RATIO (CBR) ANALYSIS

1. PROJECT: ENGINEER CENTER EXPANSION
2. DATE: 2 APR 2010
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



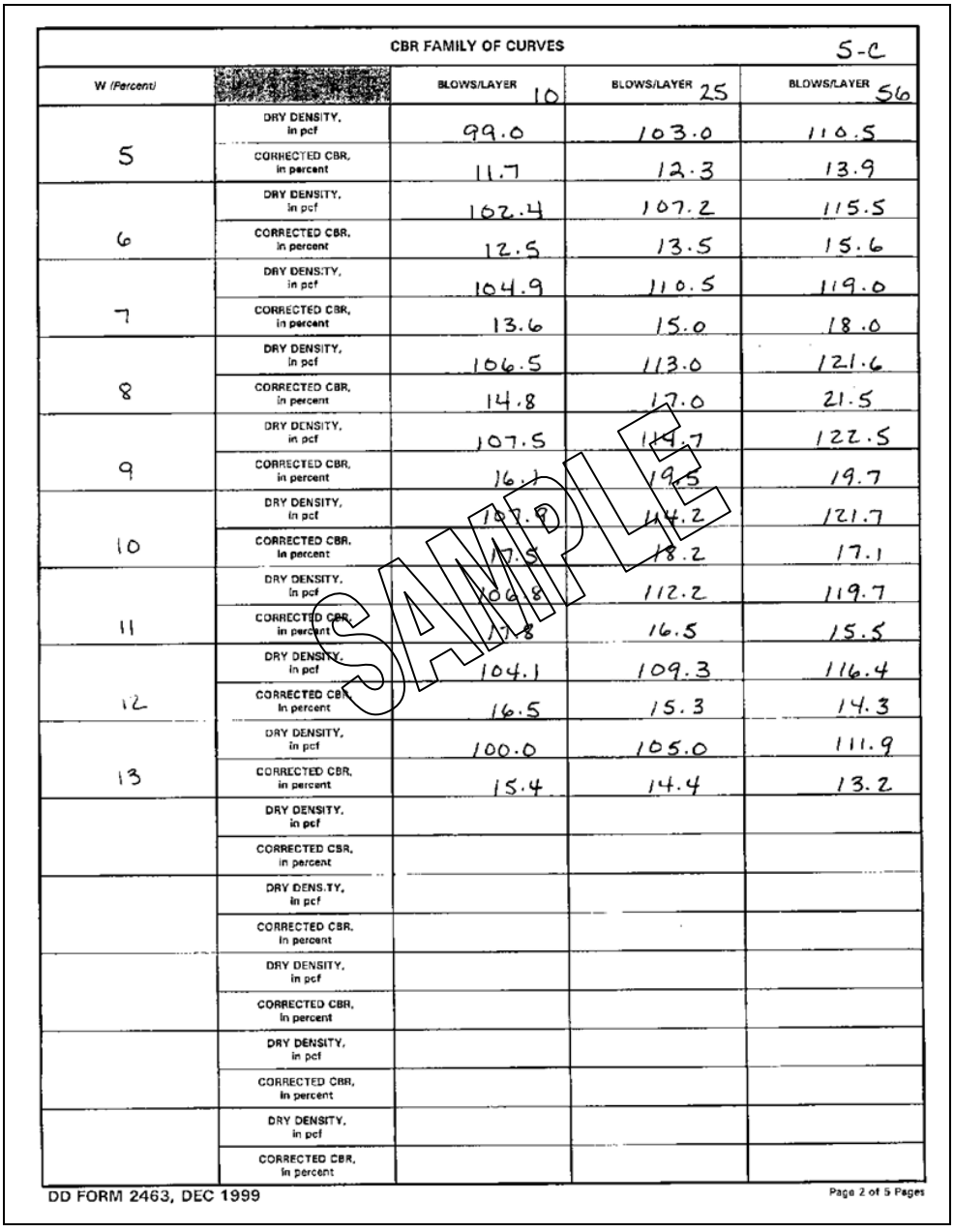


SAMPLE

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 9-5. Sample DD Form 2463



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			

SAMPLE

DD FORM 2463, DEC 1999 Page 2 of 5 Pages

Figure 9-5. Sample DD Form 2463 (continued)

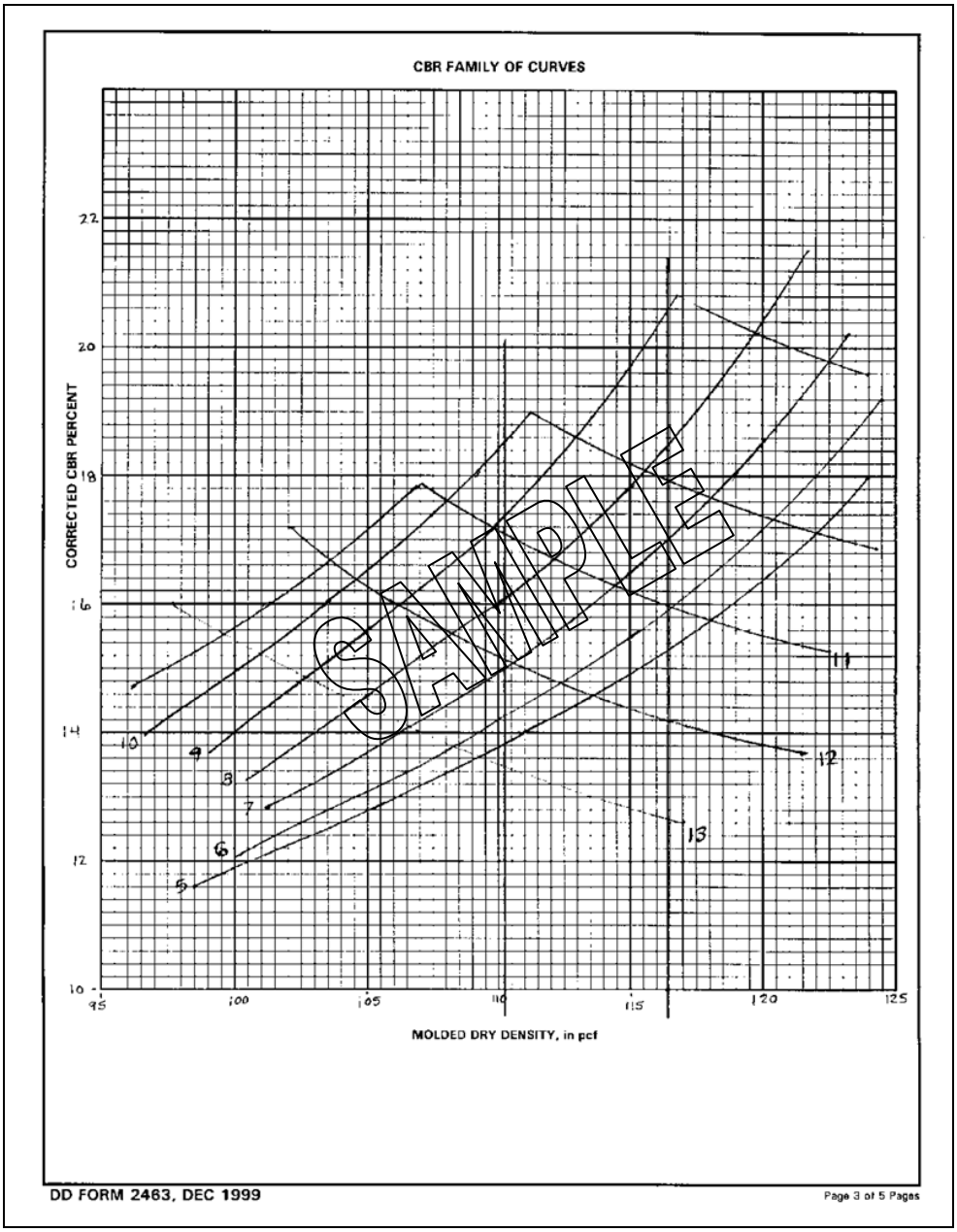
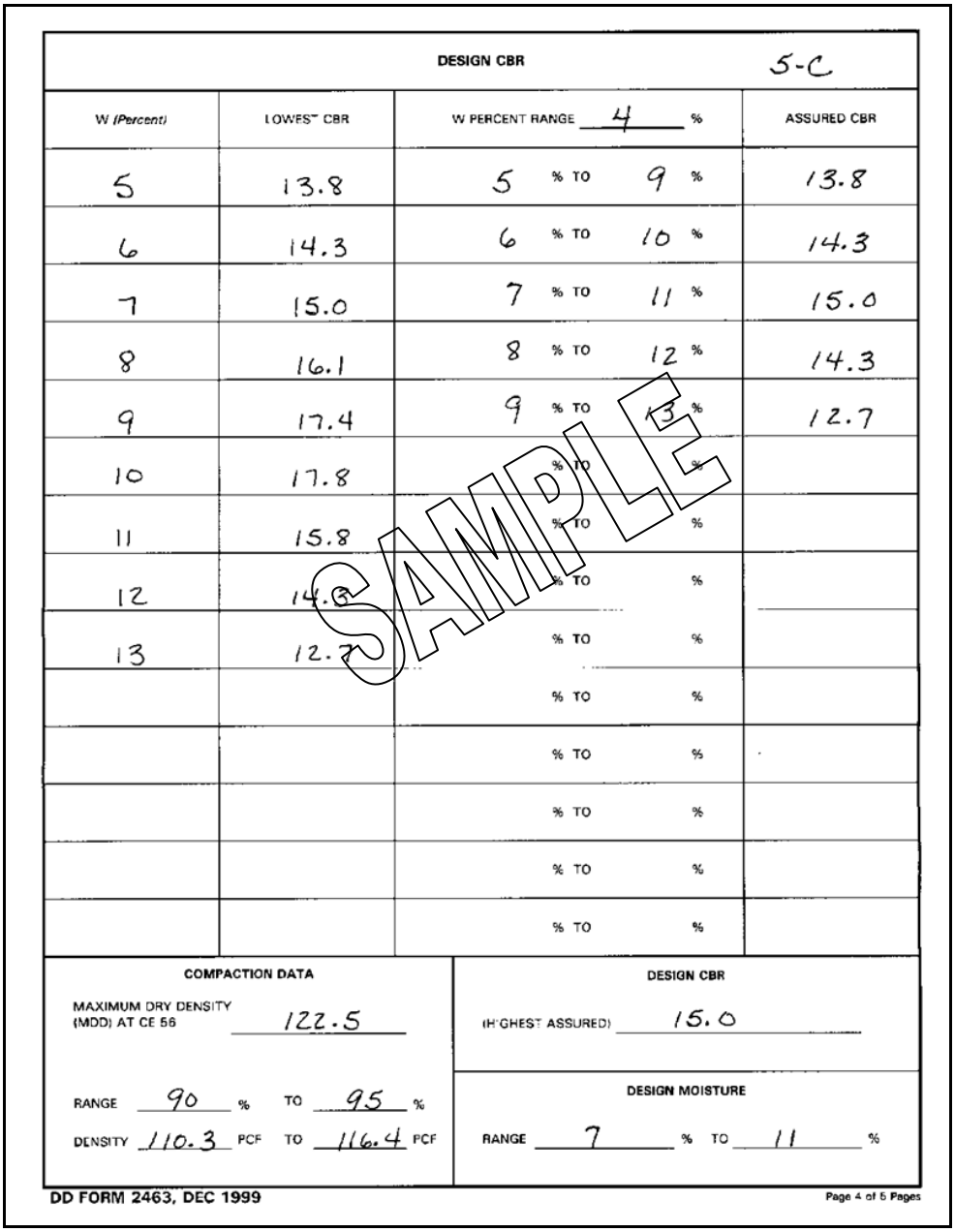


Figure 9-5. Sample DD Form 2463 (continued)



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 %	

SAMPLE

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

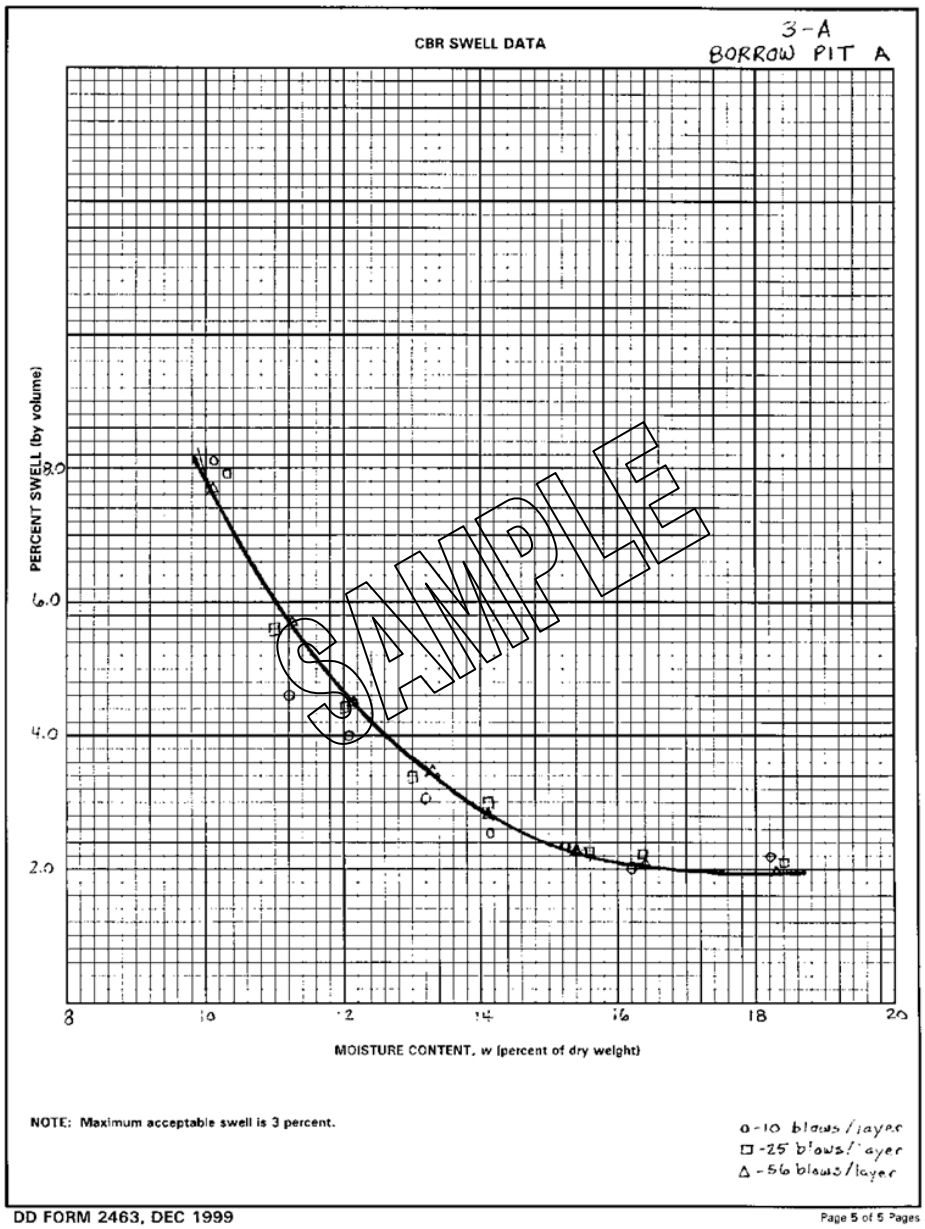
(HIGHEST ASSURED) 15.0

DESIGN MOISTURE

RANGE 7 % TO 11 %

DD FORM 2463, DEC 1999 Page 4 of 5 Pages

Figure 9-5. Sample DD Form 2463 (continued)



Legend:

Apr	April
CBR	California Bearing Ratio
cu ft	cubic feet
DD	Department of Defense
Dec	December
MDD	maximum dry density
pcf	pound per cubic foot
SFC	sergeant first class
SSG	staff sergeant

Figure 9-5. Sample DD Form 2463 (continued)

Steps for California Bearing Ratio Testing of a Nonswelling Soil

9-33. Perform the following steps for CBR testing of a nonswelling soil:

- **Step 1. Establish the optimum moisture content.** Establish the OMC of the soil at 56 blows per layer by using the data collected from the compaction test (see chapter 7). For this example, OMC equals 8.8 percent.
- **Step 2. Establish a moisture range for CBR investigation.** For step 2 of CBR testing of a nonswelling soil, 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.** For step 3 of CBR testing of a nonswelling soil, compact the samples within the moisture-content investigation range at different levels of compactive effort. This allows for evaluation of soil strength when field placement is other than 100 percent of the compaction test's maximum density. 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 less than 20), it may be placed at less than 100 percent of the compaction test's maximum density for CE 56, as indicated in table 9-2. 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 greater than 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. If desired, data can be developed at two other levels of compactive effort, encompassing the specified placement densities. In the example on DD Form 1211 the soil type is SC and the DD Form 1211 displays a bell-shaped compaction curve. This is indicative of a swelling or a nonswelling soil. A U-shaped compaction curve indicates a free-draining soil, so an assumption is made that this is a nonswelling soil.

Table 9-2. 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.			
Legend: CBR California Bearing Ratio			

- **Step 4. Soak the samples and measure the well.** Soak the samples and measure the swell.
- **Step 5. Perform CBR tests and determine the corrected CBR.** Perform CBR penetration tests and determine the corrected CBR for each sample.

Note. Accumulating the required data can involve a considerable amount of work. For this example, 21 molds (7 for each of three compactive efforts) were soaked and then penetrated.

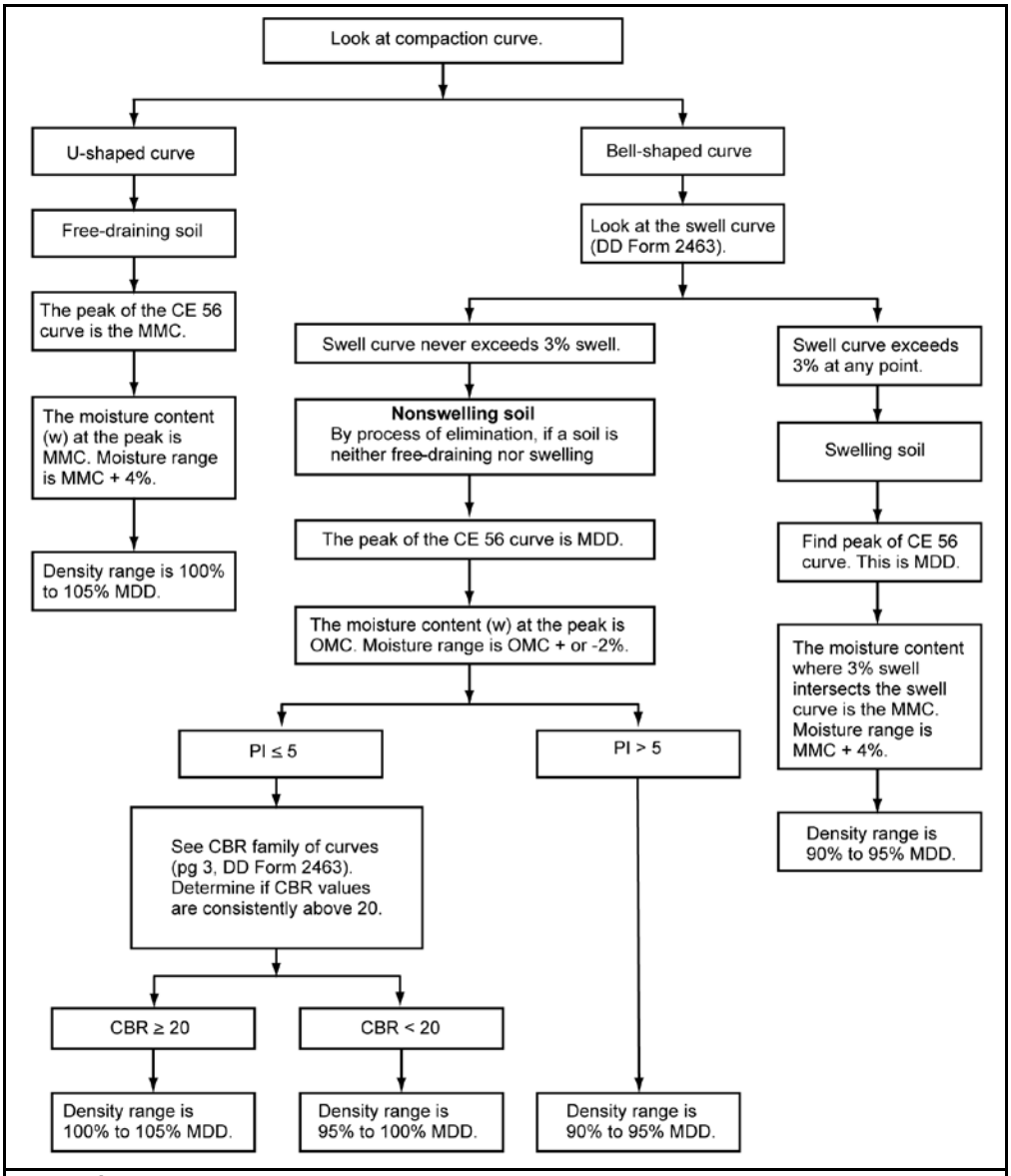
- **Step 6. Record the test results on the DD Form 2463.** Transfer the test results from each DD Form 1212 to the data summary section of DD Form 2463 (figure 9-5, page 9-12).
- **Step 7. Plot the data on graphs.** Plot the data on the graphs of dry density versus molding moisture content and corrected CBR versus molding moisture content on DD Form 2463.
- **Step 8. Determine the moisture range.** 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 data points to DD Form 2463.** Transcribe the data points to the graphs on DD Form 2463 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 graphics versus molding moisture content graphs.
- **Step 10. Plot the CBR family of curves.** Plot the CBR family of curves on DD Form 2463, page 3. 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 sample DD Form 2463 (figure 9-5, page 9-12) 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.** Proceed with an engineering analysis. After all 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 the following points of interest about the soil being used for the sample:
  - The maximum strength of this soil does not occur at OMC, but at 1 percent drier (see corrected CBR versus molding water content).
  - 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 the two preceding factors is important. If maximum strength from this soil is desired, may 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.** Establish a density range at which soil will be placed in the field. The theater of operations standard compaction range is 5 percent, unless otherwise stated. The flow chart in figure 9-6 provides the information necessary for determining the density and moisture ranges for nonswelling, swelling, and free-draining soils. To facilitate construction, it is common to specify a reasonable range of densities that can be economically obtained and 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 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, figure 6-3, page 6-5. With this information and the following figure 9-6, the soil must be placed to at least 90 percent of MDD. This would

provide a density range (in pounds per cubic foot) from 110.3 (at 90 percent MDD) to 116.4 (at 95 percent MDD). This is calculated with the following formula:

$$0.90\ x\ 122.5 = 110.3\ pounds\ per\ cubic\ foot$$
$$0.95\ x\ 122.5 = 116.4\ pounds\ per\ cubic\ foot$$

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 pounds per cubic foot and the other at 116.4 pounds per cubic foot.



Legend:

CBR	California Bearing Ratio
CE	compaction effort
DD	Department of Defense
MDD	maximum dry density
MMC	minimum moisture content
OMC	optimum moisture content
PI	plasticity index

Figure 9-6. CBR design method density and moisture requirements

- **Step 13. Determine the assured CBR values between the specified density limits.** 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 pounds per cubic foot, 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 pounds per cubic foot is 15.6. At 116.4 pounds per cubic foot, 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.
- **Step 14. Determine the CBR values for potential moisture content specification ranges.** 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 DD Form 2463.
- **Step 15. Select the moisture content range.** For step 15 of CBR testing of a nonswelling soil, 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 pounds per cubic foot.

TEST PROGRAM FOR SWELLING OR EXPANSIVE SOILS

9-34. There is a small group of soils, that 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. The expansive nature of the soil 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.

9-35. 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 (table 9-3). 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 9-3. Swell potential

PI	*Expansiveness*
0 to 14	Not expansive
14 to 25	Marginal
25 to 40	Critical
40	Highly critical
Legend: PI	plasticity index

Example

9-36. 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

9-37. Perform the following steps to complete tests on swelling or expansive soils.

- **Step 1. Establish the Optimum Moisture Content of the Soil.** Establish the OMC of the soil. 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.** 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 9-2, page 9-13) 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.** 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 it against the molding water content. The triangular data point at 10 percent moisture on the swell data curve (figure 9-6, page 9-19) was obtained from the following:
  - Initial sample height equals 4.60 inches.
  - Final dial reading equals 0.025 inches.
  - Molding water content equals 10.0 percent.
  - Level of compactive effort equals blows per layer.
  - Free-swell index or percentage of swell equals (0.025/4.60) x 100 = 0.5 percent.
- **Step 4. Plot the Points on DD Form 2463.** Plot the points on DD Form 2463. Once the points are plotted, a curve is then usually fit to the CE 56 data points only. The curve for a swelling soil shows that this soil, if placed and compacted at molding moisture contents of 14 percent or greater, will swell 3 percent or less.

Note. The sample DD Form 2463 in figure 9-6 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.

- **Step 5. Perform CBR rests.** Perform CBR penetration tests and determine the corrected CBR for each sample.

- **Step 6. Plot the data on graphs of dry density.** 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.** Reformat the data on DD Form 2463.
- **Step 8. Plot the CBR family of curves.** Plot the CBR family of curves on DD Form 2463.
- **Step 9. Establish a density range.** 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, the flow chart in figure 9-6, page 9-20, 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, DD Form 2463, by drawing two vertical lines on the CBR family of curves. For example, if the MDD was determined to be 110, use the following calculation to establish the actual density limits:

$$0.90\ x\ 110.0 = 99.0\ pounds\ per\ cubic\ foot$$
$$0.95\ x\ 110.0 = 104.5\ pounds\ per\ cubic\ foot$$

- **Step 10. Determine the assured CBR values detween the specified density limits.** Determine the assured CBR values between the specified density limits. As more than 3 percent swell is not acceptable, the 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 to 20. The CBR values of the applicable moisture contents were determined to be the amounts shown in table 9-4.

Table 9-4. 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
Legend: CBR = California Bearing Ratio; w = moisture content	

- **Step 11. Determine the CBR values for potential moisture content specification ranges.** Determine the CBR values for potential moisture content specification ranges. For this example, the values shown in table 9-5 were derived.

Table 9-5. 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
Legend: CBR = California Bearing Ratio; w = moisture content	

- **Step 12. Select the moisture content range.** 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 to 18 percent and a density of 99.0 to 104.5 pounds per cubic foot.

- **Step 13. Analyze the results.** This technique does not provide for drying the soil to a moisture content less than the amount of placement. Such extreme drying takes 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.

Note. This design value was obtained at the expense of strength.

9-38. 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; FM 5-430-00-1/AFJPAM 32-8013/MCRP 3-17.7A, Volume I; FM 5-430-00-2/AFJPAM 32-8013/MCRP 3-17.7B, Volume II; and TM 5-822-14/AFJMAN 32-1019.

TEST PROGRAM FOR FREE-DRAINING SOILS

9-39. Determining a design CBR for free-draining soils requires the least testing of the three remolded laboratory test programs.

9-40. 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. 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. 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

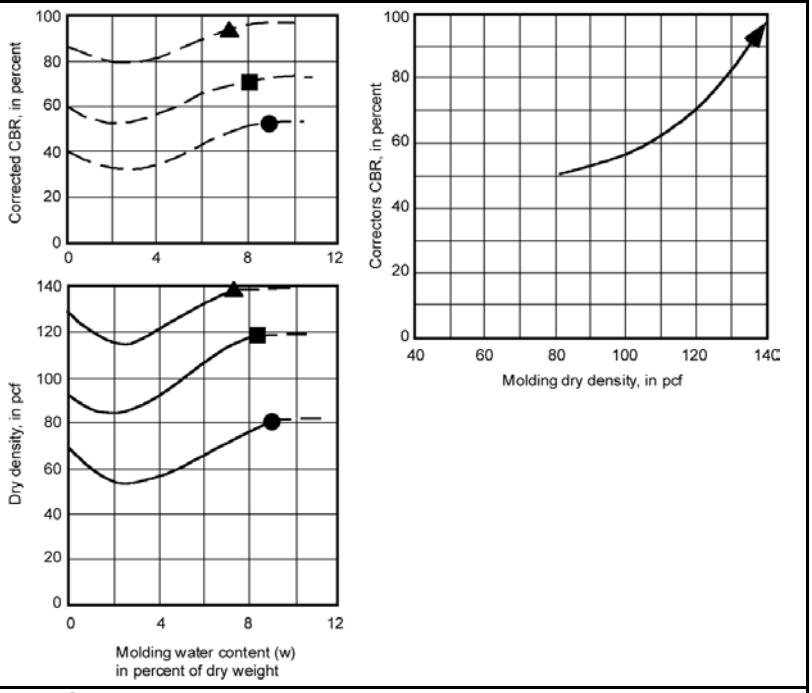
9-41. To arrive at a design CBR, the steps outlined in the previous examples are followed.

Steps

9-42. Perform the following steps for the test program for free-draining soils:

- **Step 1.** Establish the MMC of the soil 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 an MDD of 120 pounds per cubic foot. To ensure that a free-draining soil is being tested, this curve should display an 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-graded and poorly graded sands and gravels with CBR values above 20, table 9-2, page 9-17, 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.
- **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 (figure 9-7) of dry density corrected CBR versus molding moisture content. Only the plot of dry density versus molding moisture content is required. The corrected graph of 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.



Legend:	
CBR	California Bearing Ratio
pcf	pound per cubic foot

Figure 9-7. Plotted results for a free-draining soil, borrow pit A

- **Step 7.** Establish a density range at which the soil will be placed in the field. Using the criteria from table 9-2, page 9-12 and noting that the CBR value for this soil is usually 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:

$$1.00 \times 120.0 = 120.0\ pounds\ per\ cubic\ foot$$
$$1.05 \times 120.0 = 126.0\ pounds\ per\ cubic\ foot$$

- **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 pounds per cubic foot. The placement moisture content necessary to ensure that this strength is obtained is 8 percent or greater.

Note. 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.

IN-PLACE FIELD CALIFORNIA BEARING RATIO TEST WITH THE DYNAMIC CONE PENETROMETER

9-43. To overcome some of the shortfalls associated with older, in-place field CBR test methods, the ERDC has developed the dual-mass, DCP 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. The CBR is useful in determining strength values for the operation of aircraft and military vehicles on unsurfaced soils and in the design of soil in pavement systems. Three correlations currently exist for this conversion, and each depends on the soil type being tested.

Note. The procedures for testing with the DCP and the correlation of CBR values can be found in greater detail in the DCP user's manual or in FM 5-430-00-2/AFJPAM 32-8013/MCRP 3-17.7B, Volume II.

EQUIPMENT

9-44. Figure 9-8 shows a dual-mass DCP kit (national stock number [NSN] 6635-01-436-6095) designed for military engineer use. The kit includes the following items:

- Case assembly.
- Top rod, threaded and welded to the handle.
- Bottom rod, threaded and welded to the anvil.
- Dual-mass hammer.
- Vertical scale, in centimeters and inches.
- Go/no-go gauge.
- Six hardened, 60°, fixed cones.
- Three hardened cone adapters and 200 disposable cones.
- Two channel lock pliers.
- One can of light, lubricating oil.
- Thread-locking compound.
- Hexagonal wrench set (5/64 to 1/4 inch).

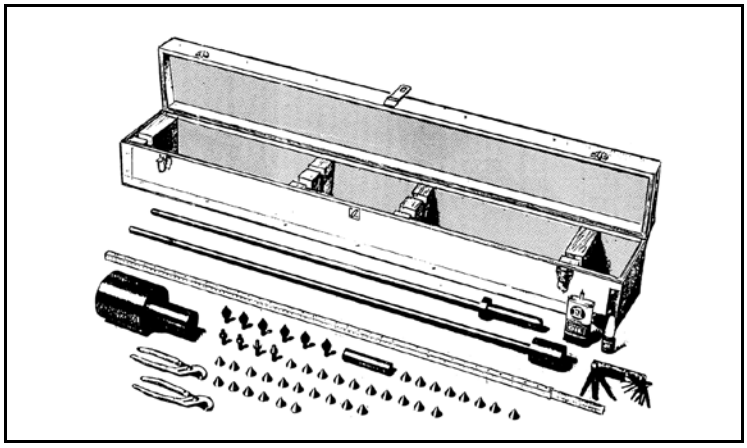
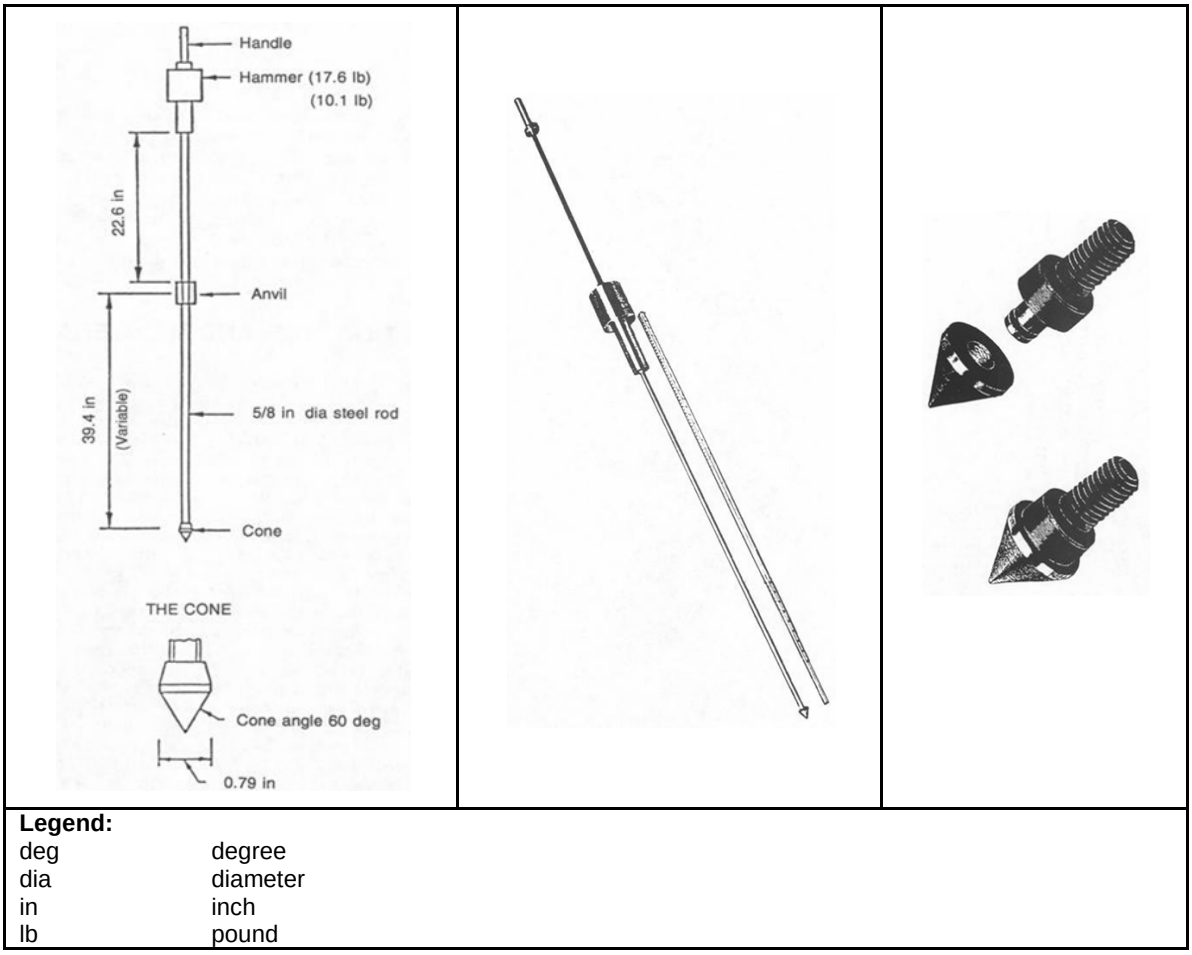


Figure 9-8. Dual-mass DCP kit

DESCRIPTION

9-45. The dual-mass DCP consists of a 5/8-inch-diameter, steel rod, approximately 36 inches in length, with a steel cone attached to one end. The rod is driven into the soil by a sliding, dual-mass hammer. Figure

9-9, page 9-26, shows an assembled DCP with vertical scale for measuring the cone penetration depth. The DCP is driven into the soil by dropping the 17.6-pound (dual-mass) or 10.1-pound (single-mass) sliding hammer. The 17.6-pound hammer is converted to 10.1 pounds by removing the hexagonal setscrew and the outer steel sleeve. The cone penetration caused by one blow of the 17.6-pound hammer is essentially twice that caused by one blow of the 10.1-pound hammer. The 10.1-pound hammer is more suitable for use and yields better test results in weaker soils having a CBR value of 10 or less. The 17.6-pound hammer penetrates high-strength soils quicker and may be preferred when these type soils are encountered. However, the 10.1-pound hammer can be used on soils up to CBR 80.



Legend:

deg	degree
dia	diameter
in	inch
lb	pound

Figure 9-9. Assembled dual-mass DCP with vertical scale and disposable cone and adapter

9-46. The depth of cone penetration is measured at selected penetration or hammer drop intervals, and the soil shear strength is reported in terms of DCP index. The DCP index is based on the average penetration depth resulting from one blow of the 17.6-pound hammer. The average penetration per hammer blow of the 10.1-pound hammer must be multiplied by two in order to obtain the DCP index value. The DCP is designed to penetrate soils up to a depth of 36 inches (limit of the rod and scale). Individual DCP index values are reported for each test depth resulting in a soil strength with depth profile for each test location.

9-47. The disposable cone is for use in soils where a standard cone is difficult to remove. The disposable cone mounts on an adapter. At the conclusion of the test, the disposable cone easily slides off the cone adapter, allowing the operator to remove the DCP device from the soil. The disposable cone remains in the soil. Using the disposable cone approximately doubles the number of tests per day that can be run by two operators.

STEPS

9-48. Perform the following steps to conduct a DCP test in soil:

- **Step 1.** Ensure that two people operate the DCP. One person holds the device by its handle in a vertical position and taps the device with the hammer until the base of the cone is flush with the surface of the soil. The second person then checks the device for a zero reading by holding the vertical scale between the soil surface and bottom of the hammer.
- **Step 2.** Ensure that the bottom of the 4-inch-diameter portion of the hammer reads 0 millimeters on the vertical scale. In weak soils, the weight of the DCP device will sink the cone past its zero reading. In this case, a zero-blow penetration reading is recorded, in millimeters, at the actual measured pretest depth.
- **Step 3.** Raise the hammer to the bottom of the handle and then drop it. Care should be exercised when raising the hammer to ensure that the hammer is touching the bottom of the handle but not lifting the cone before it is allowed to drop. The hammer must be allowed to fall freely with its downward movement not influenced by any hand movement. The operator should also be careful not to exert any downward force on the handle after dropping the hammer.
- **Step 4.** Ensure that the recorder and the operator should keep track of the number of hammer drops (blows) applied to the soil, with the recorder writing the number down between measurements on the DCP data sheet shown in figure 9-10, page 9-28. The recorder is also responsible for measuring and recording the penetration after each set of hammer blows. Penetration measurements are recorded to the nearest 5 millimeters. As an example of how to read the penetration depth, figure 9-11, page 9-29, shows a penetration depth of 150 millimeters.
- **Step 5.** Ensure that the cone penetrates at least 25 millimeters between recorded measurements. Data taken at less than 25-millimeter penetration increments are unnecessary and sometimes result in inaccurate strength determinations. The number of hammer blows between measurement recordings will generally be 20, 10, 5, 3, 2, or 1, depending on the soil strength and the cone penetration rate. The operator and recorder should be alert to sudden increases in the cone-penetration rates during the test. Any noticeable increase in the penetration rate indicates a weaker soil layer. The operator should stop and allow the recorder to record the blow count and penetration depth whenever a weaker soil layer is encountered.
- **Step 6.** Extract the cone from the soil by driving the hammer upward against the top handle, after the cone has been driven to the desired test depth (maximum 36 inches). Caution must be exercised during this operation to prevent damage to the DCP device. The hammer must be raised in a vertical direction (rather than in an arcing motion), or the rod can be bent or broken where it connects to the anvil. In soils where great difficulty is encountered in extracting the DCP device, disposable cones should be used.
- **Step 7.** Calculate the CBR from the recorded DCP values using table 9-6, page 9-29. The correlation of DCP index with CBR requires some knowledge of the soil condition. Three correlations exist depending on whether the soil is—
  - A fat clay, CH.
  - A weak clay, CL with a CBR below 10.
  - All other soils (table 9-6 shows the tabulated correlation of DCP index with CBR for the three soil conditions).

DCP TEST DATA

File Name:

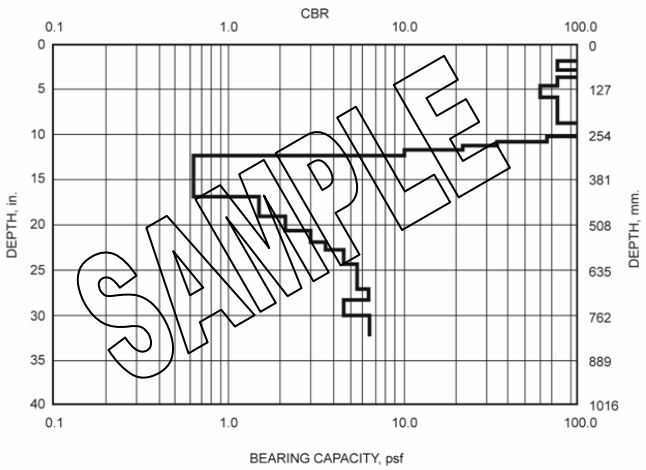
Project: Test DCP2000 Date: 02-Apr-10

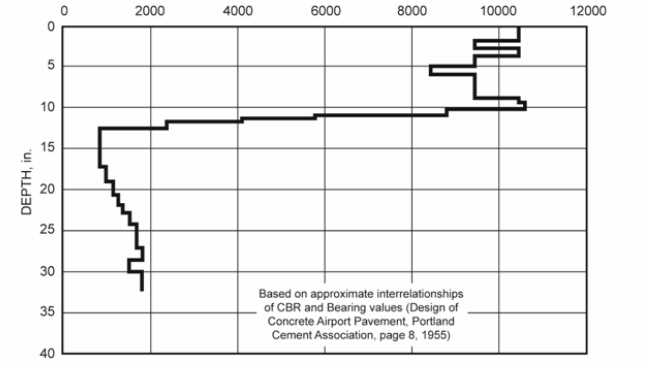
Location: Location 1 Soil Type(s): Type in the soil type

Hammer: ● 10.1 lb ○ 17.6 lb ○ Both hammers used

Soil type: ○ CH ○ CL ● All other soils

No. of Blows	Acuumulative Penetration (mm)	Type of Hammer
0	0	2
15	20	2
15	40	2
15	65	2
15	85	2
15	110	2
15	140	2
15	165	2
15	190	2
15	215	2
15	235	2
15	250	2
8	265	2
3	275	2
3	290	2
1	300	2
1	310	2
1	430	2
1	485	2
1	525	2
1	555	2
1	580	2
2	620	2
2	655	2
2	690	2
2	720	2
2	760	2
2	790	2
2	820	2
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		2
		2
		2





Legend:

Apr	April
CBR	California Bearing Ratio
DCP	dynamic cone penetrometer
in	inches
lb	pound
mm	millimeter
No.	number
psf	pound per square foot

Figure 9-10. Excel spreadsheet used for DCP analysis

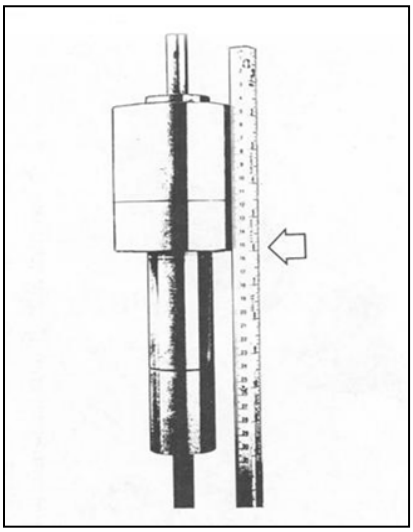


Figure 9-11. Example of penetration measurement showing a penetration of 150 millimeters

Table 9-6. Tabulated correlation of CBR versus DCP index

CH Soils		*CL Soils: CBR <10*		*All Other Soils*			
DCP Index (mm/blow)	*CBR (%)*	*DCP Index (mm/blow)*	*CBR (%)*	*DCP Index (mm/blow)*	*CBR (%)*	*DCP Index (mm/blow)*	*CBR (%)*
17	20.0	19	10.0	<3	100.0	51	3.6
18	19.0	20	9.0	3	80.0	52	3.5
19	18.0	22	7.0	4	60.0	53-54	3.4
20	17.0	24	6.0	5	50.0	55	3.3
22	16.0	26	5.0	6	40.0	56-57	3.2
23	15.0	27	4.7	7	25.0	58	3.1
25	14.0	28	4.4	8	30.0	59-60	3.0
27	13.0	29	4.1	9	25.0	61-62	2.9
29	12.0	30	3.8	10-11	20.0	63-64	2.8
32	11.0	31	3.6	12	18.0	65-66	2.7
35	10.0	32	3.4	13	16.0	67-68	2.6
39	9.0	33	3.2	14	15.0	69-70	2.5
43	8.0	34	3.0	15	14.0	72-74	2.4
50	7.0	35	2.8	16	13.0	75-77	2.3
58	6.0	36	2.7	17	12.0	78-80	2.2
69	5.0	37	2.5	18–19	11.0	81-83	2.1
72	4.8	38	2.4	20–21	10.0	84-87	2.0
75	4.6	39	2.3	22–23	9.0	88-91	1.9
79	4.4	40	2.2	24–26	8.0	92-96	1.8
83	4.2	41	2.1	27–29	7.0	97-101	1.7
87	4.0	42	2.0	30–34	6.0	102-107	1.6
92	3.8	43	1.9	35–38	5.0	108-114	1.5
97	3.6	44	1.8	39	4.8	115-121	1.4
102	3.4	45	1.7	40	4.7	122-130	1.3

Table 9-6. Tabulated correlation of CBR versus DCP index (continued)

CH Soils		*CL Soils: CBR <10*		*All Other Soils*			
DCP Index (mm/blow)	***CBR (%)***	***DCP Index (mm/blow)***	***CBR (%)***	***DCP Index (mm/blow)***	***CBR (%)***	***DCP Index (mm/blow)***	***CBR (%)***
110	3.2	46	1.6	41	4.6	131-140	1.2
115	3.0	48	1.5	42	4.4	141-152	1.1
125	2.8	50	1.4	43	4.3	153-166	1.0
135	2.6	51	1.3	44	4.2	167-183	0.9
145	2.4	54	1.2	45	4.1	184-205	0.8
152	2.3	56	1.1	46	4.0	206-233	0.7
158	2.2	59	1.0	47	3.9	234-271	0.60
165	2.1			48	3.8	272-324	0.5
175	2.0			49-50	3.7	>324	<0.5

Legend:
CBR California Bearing Ratio
DCP dynamic cone penetrometer
mm millimeter

Dynamic Cone Penetrometer Maintenance

9-49. The DCP should be kept clean, and all soil should be removed from the penetration rod and cone before each test. A light application of spray lubricant or oil should be applied to the hammer slide rod before each day's use. All joints should be constantly monitored and kept tight. Loose joints will lead to equipment failure. Any problem joints should be treated with a joint locking compound. The lower penetration rod should be kept clean and lubricated with oil when clay soils are tested.

9-50. Using disposable cones will save wear and tear on the device and the operator. In soils with large aggregate, the DCP may try to penetrate the soil at a slant rather than from a vertical direction. The operator should not apply force to the handle of the DCP in an attempt to force it to penetrate the soil vertically. Lateral force on the handle in an attempt to make the DCP penetrate the soil vertically will cause the upper handle rod to fatigue and break at the point where it screws into the anvil. Instead, the test should be stopped when the handle deviates laterally 6 inches or more from the vertical position and a new test should be attempted at another location.

9-51. The go/no-go gauge is used to ensure the cone's base diameter is within proper tolerance. Each new cone should be checked before use and at selected usage intervals to ensure that the cone's base diameter is within a proper tolerance of 0.78 to 0.80 inch. The cone must be replaced if its base diameter fits into both ends or neither end of the go/no-go gauge. The cone is within proper tolerance when it fits into only one end of the gauge

SOIL STRENGTH EVALUATION WITH THE DYNAMIC CONE PENETROMETER

9-52. The following are important aspects of soil strength evaluation with the dynamic cone penetrometer:

- **Number of measurements.** The number of measurements to be made, location of the measurements, depth of measurements required, and frequency of recording data with depth vary with type of road or airfield pavement operation and with the time available for conducting the tests. For this reason, hard-and-fast rules for the number of tests required in evaluating roads and airfields are impracticable. Soil conditions are extremely variable. The strength range and uniformity of the soils or existing pavement materials will generally control the number of measurements necessary. In all cases, it is advisable to first test those spots that appear to be weakest. Since the weakest condition controls the pavement evaluation, penetrations in areas that

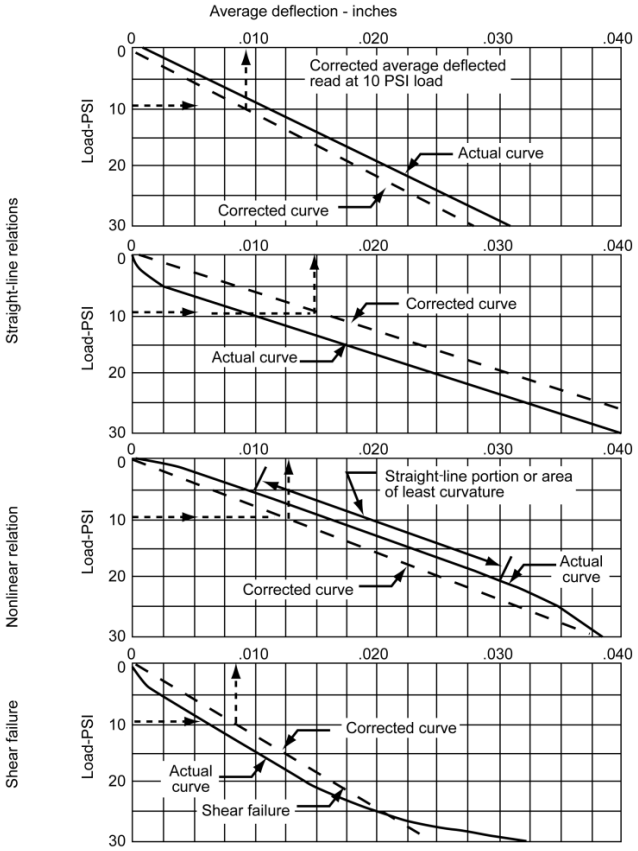
appear to be firm and uniform may be few and widely spaced. In areas of doubtful strength, penetration tests should be more closely spaced.

Note. No less than three penetration tests should be made in each area having similar type soil conditions.

- **Reading depths in soil.** Soil strength usually increases with depth; but in some cases, a thin, hard crust will overlay a soft layer or the soil will contain thin layers of hard and soft material. For this reason and the fact that many aircraft and some military vehicles will affect the soil to depths of 36 inches or more, it is recommended that each penetration be made to a depth of 36 inches unless prevented by a very hard condition at a lesser depth. Soil test depths may be reduced when required traffic operations are known and the thickness requirements indicate that a reduced thickness above the subgrade controls the evaluation.
- **Data tabulation.** The data can be tabulated in spreadsheet format with the number of hammer blows, hammer weight, and cone penetration recorded to the nearest 5 millimeters after each set of hammer blows.

Note. Figure 9-10, page 9-28 shows a printout from the Excel spreadsheet program for use in analyzing the DCP test data. The Excel DCP program may be included in the DCP kit or is available from ERDC-WES or the DCP manufacturer. To obtain the latest software, contact ERDC-WES.

Example. The DCP data shown in figure 9-12 was obtained on a test road consisting of 10 inches of crushed gravel over a very weak CH subgrade. A geogrid over geotextile was installed over the soft subgrade to separate and reinforce the aggregate layer. The CBR versus depth plot shows a 10-inch layer of base material with an 80 to 100 CBR over a very weak subgrade soil having a CBR of 0.6 to 0.7. The stair step increase in subgrade strength below the 15-inch depth is typical in plastic clay soils. This occurs due to the plastic clay sticking to the DCP penetration rod at test depths below 12 inches. The strength of the CH soil may not actually be increasing as indicated at the 17-inch depth. The geogrid/geotextile layer was easily located by the DCP as it took several hammer drops to penetrate the geotextile at the 10-inch depth.



Note. Corrected curve may lie below or above actual curve.

Legend:

psi pound per square inch

Figure 9-12. Correction of load deformation curve

SPECIAL CONSIDERATIONS

9-53. The following are special considerations for in-place field CBR testing with the DCP:

- **Weather.** Because soil conditions are immediately and significantly affected by weather, an evaluation is valid only for the period immediately after measurements are made for unsurfaced pavements. However, it can usually be assumed that the evaluation will remain constant as long as no rain occurs. Gravel-surfaced pavements will be affected to a much lesser extent by rain.
- **Clay soils.** DCP tests in highly plastic clays are generally accurate for depths to approximately 12 inches. At deeper depths, clay sticking to the lower rod may indicate higher CBR values than the actual values. Oiling the penetration rod will help prevent the clay from sticking to the penetration rod; however, it will not significantly improve the test results. A 2-inch-diameter (or larger) auger can be used to open the test hole after each 12-inch DCP test penetration. This will eliminate lower-rod friction problems and allow the test to accurately measure the clay soil strength for an additional 12 inches.

  Sands. Many sands occur in a loose state. When relatively dry, loose sands show no DCP index values for the top few inches and may show increasing DCP index values with depth. The confining action of aircraft tires will increase the strength of the sand. Generally, dry sand or gravel will be adequate for aircraft in the C-130 class, regardless of the DCP index values. All sands and gravels in a quick condition (water percolating through them) must be avoided. The evaluation of moist sands should be based on the DCP tests as described above.

- **Cone penetration refusal.** If the cone does not penetrate 25 millimeters after 10 blows with the 17.6-pound hammer of 20 blows with the 10.1-pound hammer, the test should be stopped and the CBR noted at that depth as 100. If this firm material is a stabilized soil or high-strength aggregate base layer, it should be cored or drilled with an auger to allow access of the DCP cone to underlying layers. The DCP test can then proceed through the access hole after the depth of the material layer has been recorded. The drilled material layer is assigned a CBR value of 100 if penetration was possible. However, if a core or auger drill is not available, the 17.6-pound DCP hammer might still be used to drive the lower rod and cone through the firm material. If a large rock or other object stopped the cone penetration, the DCP should be extracted and another attempt made within a few feet of the initial test.

Note. The DCP is generally not suitable for soils having significant amounts of aggregate greater than a 2-inch sieve.

PLATE BEARING TEST FOR DETERMINING MODULUS OF SOIL REACTION

9-54. This test is used to determine the modulus of reaction of soils by means of the plate-bearing test and for determining the corrections to be applied to the field test values by means of laboratory tests. The modulus of soil reaction is required in rigid pavement design and evaluation.

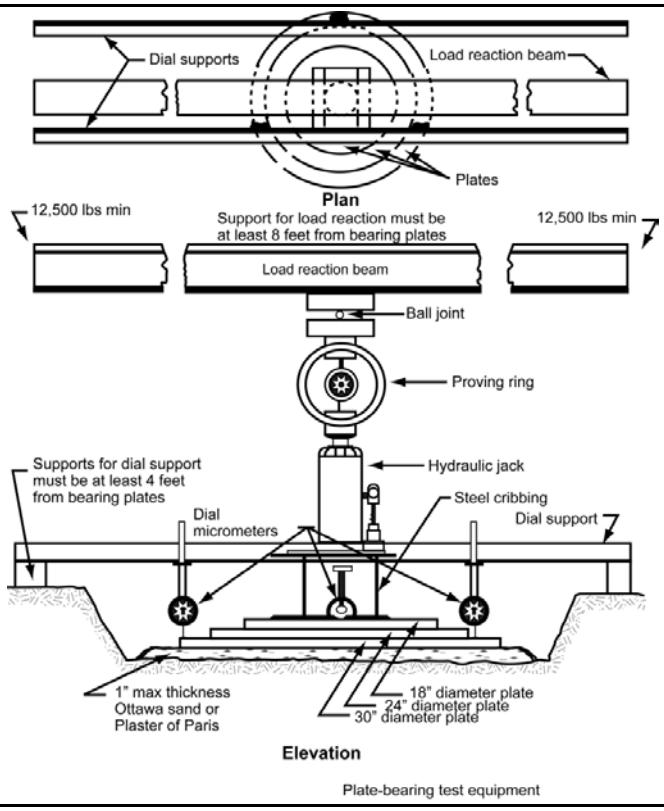
STEP 1. STRIP AN AREA OF SOIL TO BE TESTED

9-55. Strip an area of soil to be tested to the proposed elevation of the subgrade surface. The stripped area should be twice the diameter of the largest bearing plate. If the subgrade is to be fill material, construct a small embankment at least 30 inches in height, using the desired soil compacted at the moisture and density conditions specified in the field. Clear the area to be tested of any loose material and make it level. Apply and level a thin layer of sand or plaster of Paris, not to exceed 1 inch in thickness on the test area.

Note. Care should be taken to not disturb the soil around the test, especially in granular soils.

STEP 2. SEAT THE 30-INCH-DIAMETER BEARING PLATE

9-56. Seat the 30-inch-diameter bearing plate on the sand or plaster of Paris. Turning or working the plate back and forth will help to provide uniform seating of the plate. Center the 24-inch- and 18-inch-diameter plates on the 30-inch-diameter plate, and center the hydraulic jack on the 18-inch-diameter plate. If cribbing is needed, crib between the top plate and the jack. A steel proving ring, if used to measure load, should be placed on top of the jack; and the ball joint should be used between the proving ring and load reaction device. The load reaction device must be long enough so that its supports will be at least 8 feet from the bearing plate. A steel beam between two loaded trucks provides a good load reaction device. Place the three micrometers so that the dial stems rest on the bottom 30-inch-diameter plate not more than 0.25 inches from the outer edge, spaced 120° apart. Fasten the micrometers to a frame whose supports are at least 4 feet from the edge of the 30-inch-diameter plate. A typical apparatus setup is shown in figure 9-13. Place the loading plate and the sensors so that they are resting on a firm and stable test surface.



Legend:	
lb	pound
min	minimum

Figure 9-13. Plate bearing test equipment and set-up

STEP 3. SEAT THE LOADING SYSTEM AND BEARING PLATE

9-57. Seat the loading system and bearing plate by applying a load of 707 pounds (1 pound per square inch) when the thickness of the pavement will be less than 15 inches or a load of 1,414 pounds (2 pounds per square inch) when the design thickness of the pavement is 15 inches or more. Allow the seating load to remain in place until nearly all deformation has taken place. Then, take a reading on all three micrometers, which will be used as the zero reading. The seating load is also considered to be the zero load. Cyclic loading under the seating load may be used to assure good seating of the apparatus and bearing plate.

- **Step 3a. Cohesive Soils (PI greater than 0).** Do not release the seating load. Apply two load increments of 3,535 pounds (5 pounds per square inch) each with each load increment being held until the rate of deformation is less than 0.0001 inch/minute. Read all three micrometers at the end each load increment. Following the completion of the 7,070-pound (10 pounds per square inch) load increment, determine the average deflection (in inches) by averaging the total movement between the 0- and 10-pounds per square inch increment for each dial.
- **Step 3b. Noncohesive Soils (PI equals 0)**. Proceed after the seating load deformation has practically ceased and the zero dial readings taken. Do not release the seating load; apply load increments of 3,535 pounds (5 pounds per square inch) each until a total load of 21,210 pounds (30 pounds per square inch) is reached, allowing each load increment to remain until the rate of deformation is less than 0.0001 inch per minute. Read all three micrometers at the completion of each load increment (in inches).

STEP 4. COMPUTE A VALUE FOR UNCORRECTED MOLDULUS OF SOIL REACTION

9-58. Compute a value of k'_u (uncorrected modulus of soil reaction) using the following formula:

$$k'_u = \frac{10\ pounds\ per\ square\ inch}{average\ deflection}$$

where—
k'_u = uncorrected modulus of soil reaction

9-59. If the value of the uncorrected modulus of soil reaction is less than 200, the test is considered complete and the load is released. If the value of the uncorrected modulus of soil raction is 200 or greater (in cohesive soils), apply additional load increments of 3,535 pounds (5 pounds per square inch) until a total load of 21,210 pounds (30 pounds per square inch) is reached, allowing each load increment to remain until the rate of deformation is less than 0.0001 inch per minute. Read all three micrometers at the completion of each load increment (in inches).

STEP 5. OBTAIN AN UNDISTURBED SAMPLE

9-60. Obtain an undisturbed sample of the foundation material for laboratory testing to determine the saturation correction to apply to the field test value. The undisturbed sample must be large enough to obtain two consolidometer specimens side by side (at least 5 inches across). Take the sample in a container suitable for sealing to preserve the moisture content until the laboratory correction tests can be performed. When the plate-bearing test is performed directly on cohesive subgrade material, obtain the undisturbed specimen from the foundation at the same elevation at which the test is performed, but alongside rather than under the plates. When the test is performed on a granular base course material that is underlain by a cohesive material and when the base course is less than 75 inches in thickness, take the undisturbed sample from the cohesive material at the bottom of the base course.

STEP 6. DETERMINE THE CORRECTED VALUE OF THE MOLDULUS OF SOIL REACTION

9-61. To determine the corrected value of the moldulus of soil reaction from the higher load (30 pounds per square inch), plot a load deformation curve being the unit load (pounds per square inch) on the plate plotted against the average deflection for each load increment. The average deflection is the average of the three dial readings between the zero and the end of each load increment. If the load deformation relation plots as a straight line passing through the origin, no correction is necessary. However, if the load-deformation relation results in a curve or a straight line not passing through the origin, the curve is corrected as shown in figure 9-12. Generally, the load deformation curve will approximate a straight line between the unit loads of 10 and 30 pounds per square inch. The correction consists of drawing a straight line, parallel to the straight-line portion of the plotted curve, and through the origin. When correcting the load deformation curve, good engineering judgment will be required. If the curve is nonlinear throughout its length, the straight-line correction will be based on the average slope of the curve through at least three points in the region of the curve having the least curvature.

STEP 7. RECOMPUTE THE UNCORRECTED MODULUS OF SOIL REACTION

9-62. Using the same equation in step 4, take the average deflection from the corrected curve from step 6 at a load of 10 pounds per square inch. The value of uncorrected modulus of soil reaction computed must then be corrected for bearing plate bending and soil saturation as outlined in steps 8 and 9.

STEP 9. CORRECT FOR SOIL SATURATION

9-63. The design of pavement is generally based on the modulus of soil reaction when the soil is saturated. It is not feasible to saturate the soil in the field prior to the field test, and seldom will the soil be in a saturated condition in its natural state. Therefore, the field test value must be corrected to reflect the value that will be obtained when the soil becomes saturated. For cohesionless soils, no correction is necessary. The correction is obtained for cohesive soils through the use of the consolidation test. The correction test will be made on undisturbed specimens of the soil from the location of the field test taken in step 5. If the

field test is performed on the surface of a cohesionless base course material that is underlain by a cohesive soil, the saturation correction will be determined by tests on the underlying cohesive material (ASTM D2435 and EM 1110-2-5027).

STEP 8. CORRECT FOR BEARING PLATE BENDING

9-64. Bending in the bearing plates results in a greater deflection at the center of the plate than at the rim where the deflections are measured. The correction of The value of uncorrected modulus of soil reaction is made by entering figure 9-14, page 9-36, with the computed value of uncorrected modulus of soil reaction on the ordinate and projecting horizontally to the intersection of the plotted curve. The corrected value for the modulus of soil reaction, is then determined by projecting vertically to the abscissa of the plot and reading the value as shown.

STEP 10. CONDUCT CONSOLIDATION TESTING

9-65. The saturation correction factor is the ratio of the deformation of the consolidation specimen at the natural moisture content to the deformation in a saturated specimen under a 10-pounds-per-square inch load. Two specimens of the undisturbed material are placed in a consolidometer. One specimen will be tested at the in-situ moisture content, and the other specimen will be saturated after the seating load has been applied. Each specimen is then subjected to the same seating load (1 or 2 pounds per square inch) that was used for the field test (step 5). The seating load is allowed to remain on the in-situ moisture content specimen until all deformation occurs, at which time a zero reading is taken on the vertical deformation dial. The other specimen is allowed to soak in the consolidometer under the seating load (1 or 2 pounds per square inch). After the specimen is saturated, a zero dial reading is obtained.

9-66. Without releasing the seating load, an additional 10-pounds-per –square-inch load is applied to each specimen and allowed to remain until all deformation has occurred. A final reading is then taken on the vertical deformation dial. Record the final deflection of the saturated specimen.

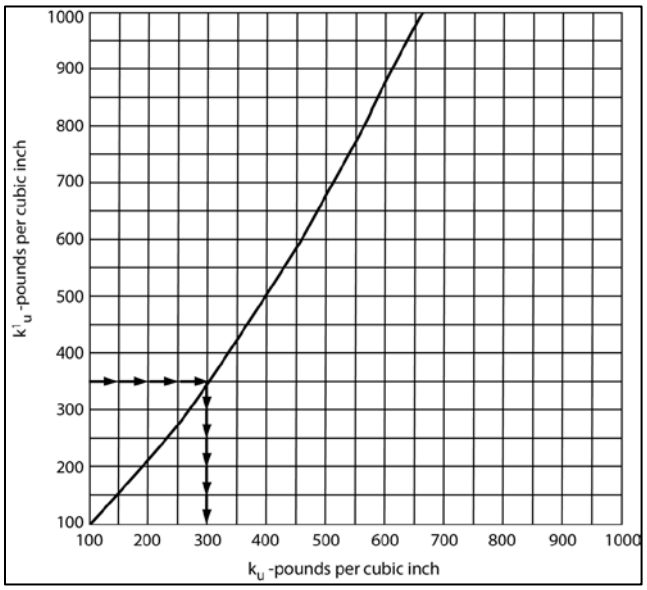


Figure 9-14. Chart for correction of uncorrected modulus of soil reaction for bending of the plate

9-67. For soils of certain types, the specimen may swell under the seating load as it becomes saturated. Swelling of the material will result in extrusion of material above the top of the consolidometer ring, so that when the 10-pounds per square inch load is applied, the material may squeeze out over the ring rather than consolidate, which will lead to erroneous results. To prevent this when dealing with a swelling type soil or one that is suspected of being a swelling type soil, the consolidometer ring will not be completely filled

with soil. This can be accomplished by trimming the top of the specimen a sufficient amount, generally 0.06 inch, to allow for the swelling. When the specimen for saturation is trimmed to allow for swelling, the specimen to be tested at the in-situ moisture content will also be trimmed an equal amount so that the heights of the specimens will be equal at the beginning of the test.

STEP 11. COMPUTE THE CORRECTED MOLDULUS OF SOIL REACTION

9-68. The correction for saturation will be applied in proportion to the deformation of the two specimens under a unit load of 10 pounds per square inch as follows:

$$k = k_u\left(\frac{d}{d_s} + \frac{b}{75}\right) - \frac{d}{d_s}$$

where—
k = corrected modulus of soil reaction (in pounds per cubic inch)
k_u = modulus of soil reaction uncorrected for saturation (in pounds per cubic inch)
d =deformation of a consolidometer specimen at in-situ moisture content under a unit (in pounds per square inch)
d_s = deformation of a saturated consolidometer specimen under a unit load of 10 pounds per square inch
b = thickness of base course material (in inches)

Note. In no instance, will a value of d/d_s greater than 1.0 be used in the above formula. This formula is applicable for the computation of the corrected *k* value, whether there is a granular base course or not. However, if the base course is 75 inches or more in thickness, a saturation correction for the cohesive subgrade material will not be made.

PURPOSE

9-69. The plate bearing test is used to obtain the modulus of soil reaction required for rigid airfield pavement design and evaluation. figure 9-15, page 9-38, displays the typical setup of the plate bearing test. The test is designed to incrementally load soil up to 25,000 pounds of force on top of a rigid plate and measure the resulting deflection under the plate. The test measures the stiffness of the soil under loading that is designated as the modulus of subgrade reaction.

EQUIPMENT

9-70. The essential components used for this test are (figure 9-15)—

- Load reaction equipment. A truck or trailer, a tractor-trailer, an anchored frame, or another structure having a dead load of at least 25,000 pounds.
- Hydraulic jack capable of applying 25,000 pounds of force with a spherical bearing attachment.
- Bearing plates. These should be 1-inch-thick steel or 1.5-inch-thick aluminum plates of 18, 24, and 30 inch diameters to be stacked in a pyramid fashion.
- Load-measuring device, consisting of a hydraulic gauge on the jack, a steel proving ring, or a load cell.
- Three dial micrometers reading 1/10,000 inches.
- Deflection beam to mount the micrometers.
- Cribbing, consisting of short pieces of hardwood or steel H- or I-beams.
- Sand or plaster of Paris for ground leveling.
- Stopwatch.
- Containers to collect soil samples.
- Cutting tools for extracting soil samples.

SOIL MODULUS TESTING WITH A LIGHT WEIGHT DEFLECTOMETER

9-71. This test method covers the use of a light weight deflectometer (LWD) also known as a portable falling weight deflectometer (PFWD). This device is used to measure small deflections occurring on compacted soil layers and bound paving materials. These deflections are converted to stiffness by back-calculation analysis techniques to determine the bearing capacity of the soil for vehicle and airfield trafficking.

9-72. This device acts as a miniature plate-bearing test. A weight is manually lifted to a prescribed height and dropped onto a plate resting on top of the soil or pavement (figure 9-15). The peak vertical deflection of the soil surface and the peak force occurring at the center of the plate are recorded on the LWD as the falling weight strikes the plate and stored in a laptop or handheld computer.

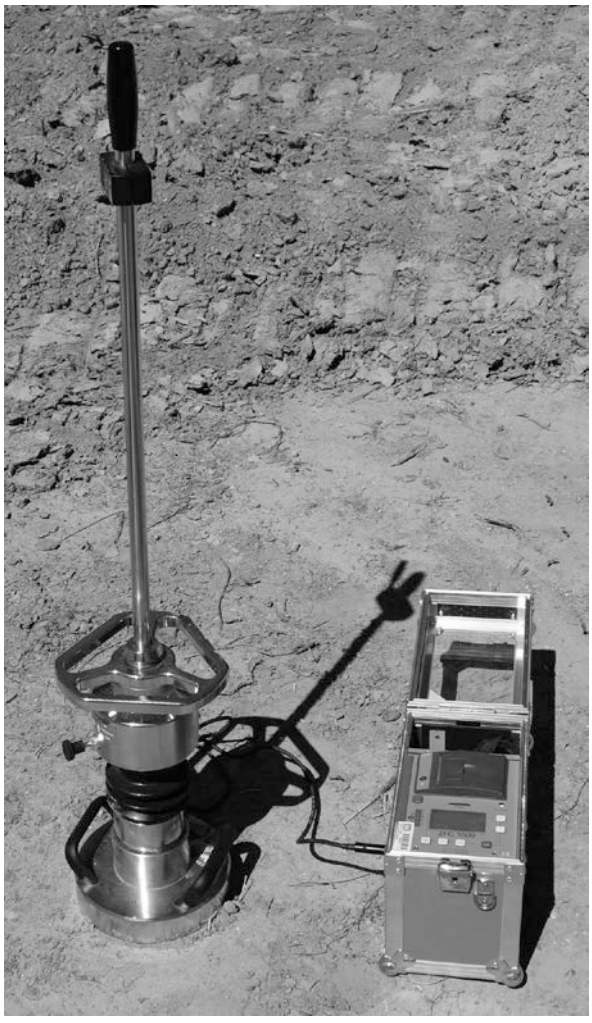


Figure 9-15. PFWD with data acquisition device

PURPOSE

9-73. The purpose of soil modulus testing is to estimate the soil or pavement layer stiffness for use in assessing bearing capacity. The device will provide an assessment of the soil modulus for depths of 280 to 380 millimeters in weak soil layers and less than 280 millimeters in strong soil layers.

EQUIPMENT

9-74. The essential components of this test are (figure 9-16):

- Transport case.
- PWFD equipped with a load cell and deflection gauge.
- Laptop or handheld computer for data acquisition.
- Rubber base pad for hardened pavement surfaces.
- Sand to provide a level surface on rocky or gravelly soil.



Figure 9-16. Operation of the PFWD, with laptop data acquisition

STEPS

9-75. Perform the following steps to operate an LWD:

- **Step 1.** Position the PFWD over the desired test point. Ensure that the test surface is as clean and smooth as possible, with loose granules and protruding material removed. If the soil surface is uneven, the placement of sand may be required to provide a smooth solid surface on which to rest the baseplate.
- **Step 2.** Place the loading plate and the sensors to ensure that they are resting on a firm and stable test surface (figure 9-16).
- **Step 3.** Raise the falling weight to the desired height, and allow it to fall freely.
- **Step 4.** Record the resulting peak surface deflections and peak load.
- **Step 5.** Repeat at least once to provide two falling weight sequences for comparison. If the difference is greater than ± 3 percent for any sensor, note the variability. The variability of stiffness measurements depends on soil type. As the soil becomes more fine-grained, the accuracy of the stiffness measurement decreases. This suggests that this device is best suited for granular soils, particularly gravels that are commonly found in base layers of pavement systems. The reported coefficient of variation for typical field conditions or the major soil types is as follows:
  - GM, GC and GP soils: 10 to 20 percent.
  - SM, SW and SP soils: 15 to 35 percent.
  - ML and CL soils: 40 to 60 percent.

9-76. Data from this device is correlated to the modulus obtained from a large-scale falling weight deflectometer. Published studies differ on the relationship of the modulus correlations, but USACE provides the following guidance as best current practice:

$$EFWD = 0.97\ x\ EPFWD\ for\ 12.5\ MPa < EPFWD < 865\ megapascals$$

9-77. It is difficult to conduct this test on weak soils, those with a recorded modulus of less than 12.5 megapascals. The test works best with stiffer soils, primarily those materials that are being improved through compaction effort.

Chapter 10

Technical Soil Report

A good program for soil 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.

REQUIRED TESTS

10-1. The tests required by any program depend on the type of construction being planned. However, there are a number of tests that are consistently run on road and airfield construction programs. A complete testing program should include the following tests:

- Soils exploration (see chapters 1 and 2).
- Compaction (see chapter 7).
- Plasticity index (see chapter 6).
- Particle size analysis (see chapter 5).
- CBR (see chapter 9).
- Trafficability (see FM 5-430-00-1/AFJPAM 32-8013/MCRP 3-17.7A, Volume I and FM 5-430-00-2/AFJPAM 32-8013/MCRP 3-17.7B, Volume II).

10-2. The soil tests listed above are specific tests that are 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.

10-3. Each test supplies the necessary data to answer questions based on the engineering evaluation of scientific data rather than a meaningless guess.

10-4. 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 the time needed for soaking or drying samples.

10-5. An example of the daily activities in a complete soil-testing program are listed in figure 10-1, page 10-2. Some testing programs may not include all of 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.

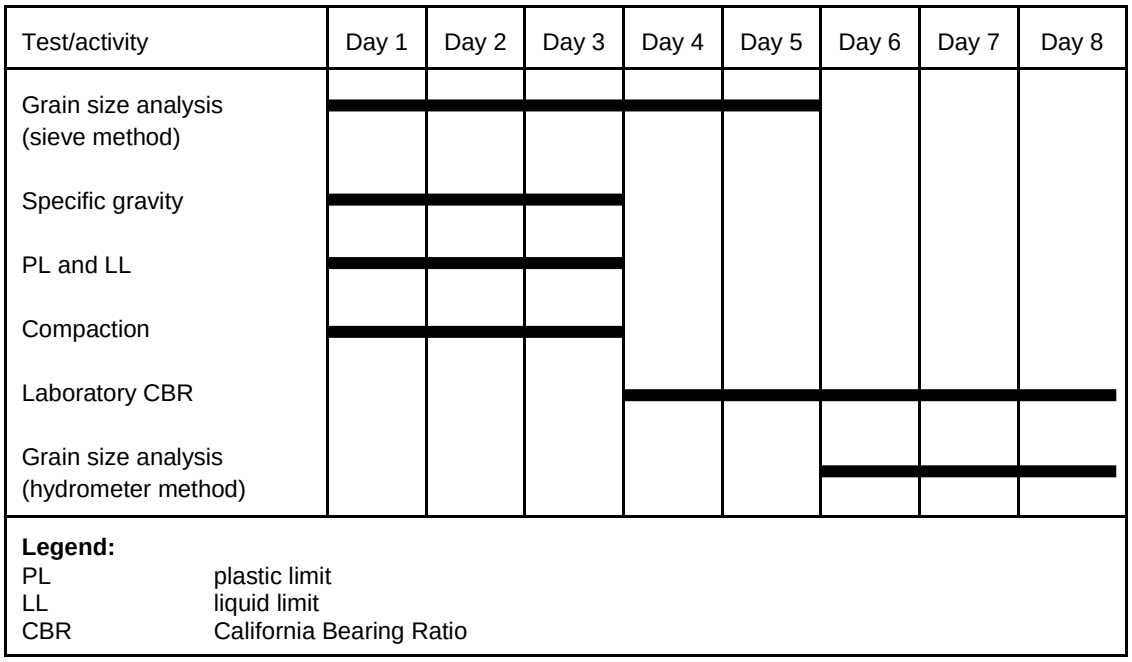


Figure 10-1. Sample schedule for soils laboratory testing

10-6. The following cite some considerations that may be helpful in setting up a continuous soil-testing program:

- **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 anytime 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 4-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.

- **In-place, field CBR test.** The latest addition to equipment used for determining the CBR value of a soil is the dual-mass DCP (FM 5-430-00-2/AFJPAM 32-8013/MCRP 3-17.7B, Volume II).

SOIL TECHNICAL REPORT

10-7. 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.

10-8. 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.

10-9. Use the following items and information to write a soils technical report:

- Soil boring logs.
- The profile of the soil.
- The field identification of soil.
- 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/AFJPAM 32-8013/MCRP 3-17.7A, Volume I.

10-10. Use the following recommended outline to organize a technical soils report:

- **Project.**
  - ***Description.*** What type of construction is being performed (Class D road, base camp, airstrip)?
  - ***Purpose and scope.*** Who is requesting construction? What is the extent and why is it being performed?
- **Site description.**
  - ***Location.*** All map references and directions, including county or province (if available).
  - ***Existing facilities.*** Old logging roads, dirt paths, patrol roads, buildings, overhead power lines.
  - ***Topography, cultivation, and drainage.*** Forests, farmland, well-drained areas, swampy areas, hills.
  - **Climate.** Temperature extremes, seasonal precipitation, average wind speed.

- **Geology.**
  - ***Overburden.*** Main depositing force (river, glacial,), rock classes (sedimentary, igneous, metamorphic), and depth of overburden.
  - ***Bedrock.*** Average depth to bedrock and state of bedrock (faulted, fractured, folded).
- **Site conditions.**
  - ***Field explorations.*** Details of performed tests, location of test sites, and explanation of sample tags.
  - ***Field tests.*** Type of field tests performed and field sample results. Enclose copies in Annex G.
  - ***Laboratory tests.*** Type of laboratory tests performed. (Do not include MDD or laboratory CBR results.)
  - ***Test results.*** Synopsis of test results and classifications. (Refer to forms in Annex F.)
- **Fill and borrow materials.**
  - ***Field explorations.*** Details of performed tests, location of test sites, and explanation of sample tags.
  - ***Field tests.*** Type of field tests performed and field sample results. Enclose copies in Annex G).
  - ***Laboratory tests.*** Type of laboratory tests performed. (Do not include MDD or laboratory CBR results.)
  - ***Test results.*** Synopsis of test results and classifications. (Refer to forms in Annex F.)
- **Conclusions and recommendations.**
  - ***Final site selection.*** Exact location of finished project. (Refer to plans enclosed in Annex C.)
  - ***Economical design.*** Low-cost or resource alternatives to current suggestions (soil cement, asphalt binder, chemical stabilization methods, mats, or fabrics).
  - ***Minimum specifications.*** Design CBR, MDD, and OMC specifications. (Refer to laboratory forms in Annex F.)
- **Annexes.**
  - Project directive and all directives involving this project.
  - General plan drawings (geology should be indicated).
  - Location plan drawings (existing and proposed features).
  - Profiles and cross sections.
  - Boring logs.
  - Laboratory testing data.
  - 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 may 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 *Not Applicable* across the form.

PART TWO

Bituminous Mixtures

Part Two provides information on the materials used in constructing bituminous surfaces, the methods of testing these materials, the design considerations and the mixes prepared from them. Mixing and placing operations and considerations when preparing the mixes, are described in UFC 3-250-03.

Chapter 11

Bituminous Pavements and Surfaces

Bituminous pavements and 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

11-1. Mineral aggregates may consist of crushed rock, crushed or uncrushed soils (including gravels and sands although their content may be limited), slag, mineral filler, or a combination of these materials. Other materials that may be used as aggregate in certain geographical areas include vesicular lava and coral. Another source of aggregate that is sometimes used is reclaimed asphalt pavement (RAP). 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 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 skid-resistant surface. A portion of the aggregate extends slightly above the normal surface of the wearing mat, thereby providing texture for tires to grip.

BITUMINOUS MATERIALS

11-2. A bituminous material is the adhesive agent or binder in a bituminous mixture. This material provides two functions:

- It binds the aggregate together, holds it in place, and prevents displacement.
- It fills approximately 75 to 80 percent of the voids in the mineral aggregate, they providing a waterproof mixture that keeps surface water from seeping into and weakening the base material.

11-3. 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 composed mainly of bitumen—a black semisolid that provides the black color, cementing ability, and waterproofing properties. Bituminous materials are classified into two main groups—asphalts and tars. Due to environmental issues,

tar is seldom used. The asphalt binder is available in several forms that are 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 are used for cold mixes or are applied as sprays in building pavements. Cutback asphalts are no longer used in some locations due to environmental issues as a result of the added solvent. Prior to considering the use of cutback asphalts, confirm that it is allowed to be used in the location.

ASPHALTS

11-4. Asphalt is normally obtained from crude petroleum, but may also occur as a natural asphalt. There are two general classes of asphalt—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 (figure 11-1). A military engineer is seldom concerned with natural asphalts because they are not usually available in areas of interest. Therefore, this chapter discusses the uses and testing of manufactured asphalts. All asphalt cements (ACs) are solid or semisolid at room temperature (77°Fahrenheit [F]) and, prior to use, must be converted to a fluid state by heating, emulsifying, or dissolving in a petroleum solvent.

Grading

11-5. There are three classification systems that can be used for identifying AC—penetration graded, viscosity graded, and performance graded. The penetration grade is determined by the distance that a standard needle under a standard load will penetrate a sample in a given time, under a given temperature condition. The viscosity grade is determined at 140°F using a standard viscometer under standard conditions. The performance-graded (PG) asphalts are graded by determining the maximum and minimum temperature °C at which the binder is expected to perform satisfactorily. Table 11-1, page 11-4, lists some typical grades for each of the three classification systems, along with the relative consistency.

Cutback Asphalts

11-6. When heating equipment is not available or is impractical to use, AC 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 AC to perform its functions. In many areas, cutback asphalts are no longer used due to the environmental issues caused by the solvents. In these areas that do not use cutback asphalt, use asphalt emulsion instead. Asphalt emulsions do not cause any particular environmental issues so they are often preferred. Prior to using cutback asphalts, check with the state or other local government environmental authorities to determine if the use of cutback asphalts are allowed and to see if there are any rules that must be followed when using these materials.

11-7. 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 heavy petroleum oils in the SC grade of liquid asphalt. Table 11-2, page 11-4, shows the percentage of components by grade for the three types of asphalt cutbacks.

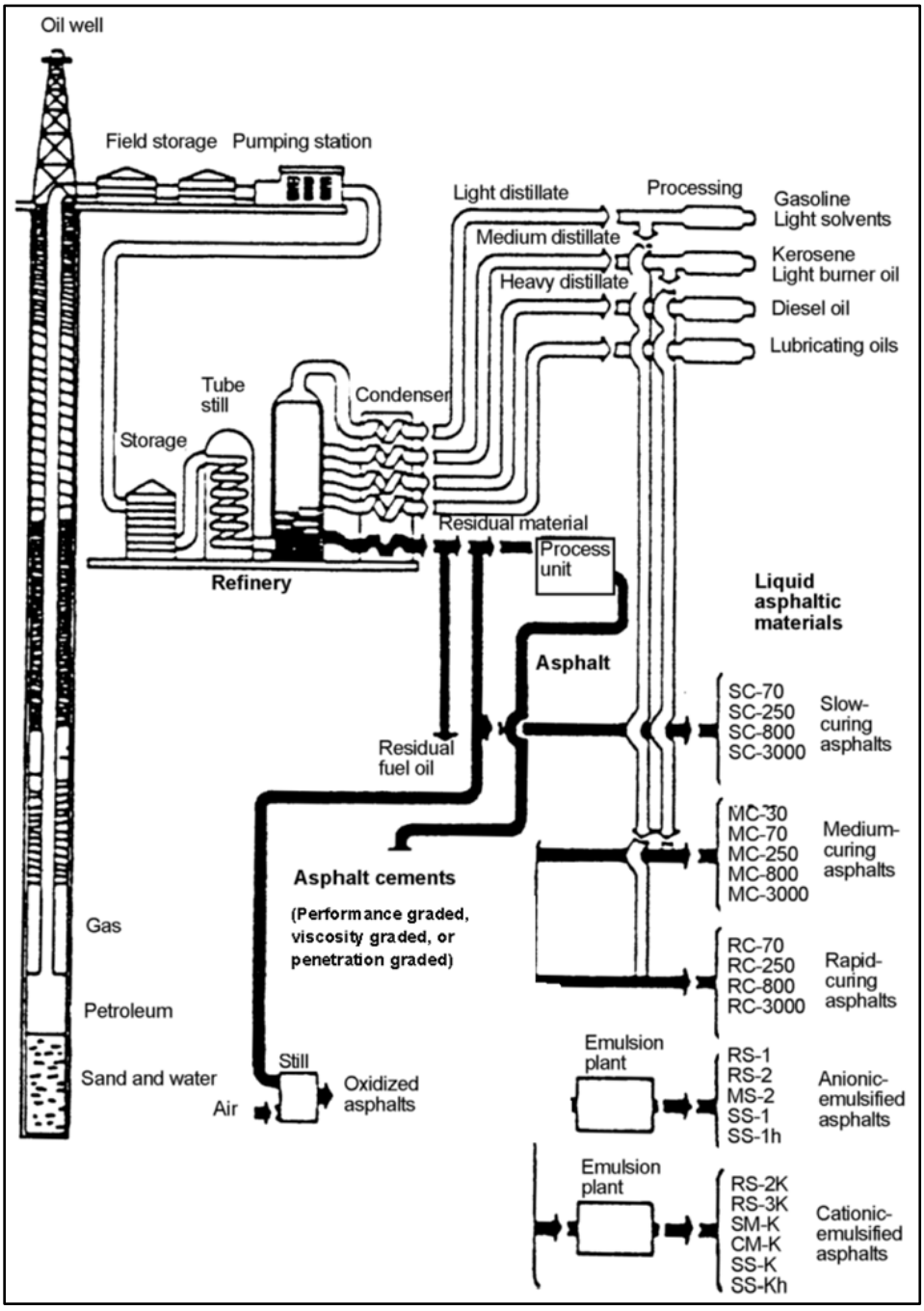


Figure 11-1. Recovery and refinement of petroleum asphalt

Table 11-1. AC performance, viscosity, and penetration grade

PG (Some Typical Grades)	*AC Viscosity Grade*	*Penetration Grade*	*Relative Consistency*
70-22	AC-40.0	40 to 50	Hard
64-22	AC-20.0	60 to 70	
58-28	AC-10.0	85 to 100	Medium
52-22	AC-5.0	120 to 150	Soft
46-28	AC-2.5	200 to 300	
Legend: AC asphalt cement PG performance-graded			

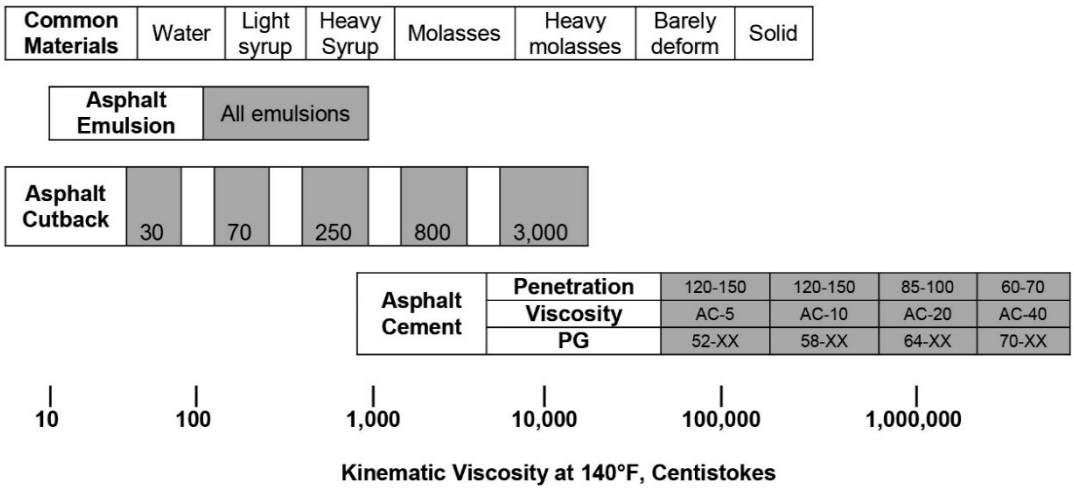
Table 11-2. Asphalt cutback composition (expressed in percent of total volume)

Viscosity Grade	*Amount of Asphalt by Volume, %*		
	SC[1]	*MC*	*RC*
30		50 min	
70	50 min	55 min	55 min
250	60 min	67 min	65 min
800	70 min	75 min	75 min
3000	80 min	80 min	80 min
[1] Based on residue of 100 penetration.			
Legend: min minimum			

11-8. As more cutter stock is mixed with a given amount of AC, a thinner liquid results. In practice, different amounts of cutter stock are added to a given amount of AC to obtain various viscosities, or grades of cutbacks. The ASTM has specified a set of specifications for cutbacks based on kinematic viscosity (ASTM D2026, ASTM D2027, and ASTM D2028). The number assigned to each grade, as shown in figure 11-2 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.

11-9. 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 centistokes), 800 (800 to 1,600 centistokes), and 3,000 (3,000 to 6,000 centistokes). In addition, the MC has a 30 grade (30 to 60 centistokes). Figure 11-2 shows the scale of viscosity grades. The higher the viscosity, the thicker the liquid, and the less amount of solvent in the cutback.

11-10. 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 (table 11-2). 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 are often used as a tack coat and can be specifically designed to be used as a prime coat for bituminous pavements. In fact, most state DOTs use asphalt emulsions for prime coats due to their environmental advantage over cutback asphalts. The grades of asphalt emulsions are found in ASTM D977 and ASTM D2397. These documents provide a list of the various grades of asphalt emulsion and the specifications for the different grades.



Legend:
AC asphalt cement
F Fahrenheit
PG performance graded

Figure 11-2. Typical viscosity values of ACs, emulsions, and cutbacks

SAFETY PRECAUTIONS

11-11. Be extremely cautious when handling bituminous materials. AC, 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 it comes 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 100°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.

Note. The spraying and mixing temperatures in many cases are above the flash point (table 11-3). Use extreme caution when handling these mixtures. Do not expose their vapors to an open flame. Cutbacks may also be classified as an environmentally hazardous material. Check with unit and installation environmental representatives for further guidance as to the proper storage, use, and disposal of these substances. If the unit's environmental representative is unknown, consult the commander for assistance. Emulsified asphalts can replace cutback asphalts for almost any purpose, and there are no significant environmental issues with emulsions.

Table 11-3. Characteristics of bituminous materials

Material	Form	Grade Designation	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**

Table 11-3. Characteristics of bituminous materials (continued)

Material	Form	Grade Designation	Temperature of Application Ranges				Flash Point (Min)		Remarks
			Spraying**		Mixing				
			°F	°C	°F	°C	°F	°C	
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 (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 (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.

Table 11-3. Characteristics of bituminous materials (continued)

Material	Form	Grade Designation	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 RS-2 RS-2K RS-3K	50–140 50–140 50–140 50–140	10–60 10–60 10–60 10–60	Nonmixing 50–140 50–140 50–140	10–60 10–60 10–60 10–60			Freezing destroys emulsion. Use for road and plant mixes with coarse aggregates (SS). All emulsions with *K* suffix are cationic
Asphalt emulsions (MS)	Liquids—asphalt particles held in an aqueous suspension by an emulsifying agent	MS-2 SM-K CM-K	50–140 50–140 50–140	10–60 10–60 10–60	50–140 50–140 50–140	10–60 10–60 10–60			
Asphalt emulsions (SS)	Liquids—asphalt particles held in an aqueous suspension by an emulsifying agent	SS-1 SS-1h SS-K SS-Kh	50–140 50–140 50–140 50–140	10–60 10–60 10–60 10–60	50–140 50–140 50–140 50–140	10–60 10–60 10–60 10–60			
Road tars	Liquids	RT-1 RT-2 RT-3	60–125 60–125 60–125	15–52 15–52 27–66					Priming oils. RT-4 through RT-12 not generally used.
RTCBs	Liquids	RTCB-5 RTCB-6	60–120 60–120	16–49 16–49					Patching mixtures. **Caution: Flammable**
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-centistoke viscosity.**

Legend:

C Celcius
F Fahrenheit
MC medium-curing
min minute
RC rapid-curing
RS rapid-setting
RT road tar
RTCB road tar cutback
SC slow-curing
SS slow-setting

ADVANTAGES AND DISADVANTAGES

11-12. Advantages and disadvantages of the bituminous materials used in construction are as follows:

- **Cutback asphalts are flammable**. Asphalt pavements are susceptible to damage by the blast from jet planes, and they can be softened by petroleum products that may be spilled on them (such as during refueling at an airfield). Tars, on the other hand, resist damage from petroleum derivatives since they are products of coal. Tars are not used much anymore due to their environmental issues and, especially, the emissions during placement that may be harmful to the workers. About the only place that tars are used is for sealers of asphalt pavements to provide some resistance to fuel and oil spillage. For this reason, tars are not discussed in any detail in this document.
- **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 that allowed for cutbacks. Additional water may be added to the emulsion (maximum 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. In recent years, the use of emulsions has far exceeded the use of cutback asphalts due to the environmental issues with cutbacks. Table 11-4 gives typical uses of bituminous materials.

Table 11-4. Typical uses of bituminous materials

Purpose	*Cutback Asphalt*	*Emulsified Asphalt*
Dust palliative	MC-30, MC-70, MC-250, SC-70, SC-250	SS-1, SS-1h, CSS-1, CSS-1h
Prime coat	MC-70, MC-250	SS-1, SS-1h, CSS-1, CSS-1h
Tack coat	RC-250, RC-800	SS-1, SS-1h, CSS-1, CSS-1h, RS-1, RS-2, CRS-1, CRS-2
Surface treatment	RC-250, RC-800	RS-1, RS-2, CRS-1, CRS-2
Mixed in place	MC-70, MC-250	SS-1, SS-1h, CSS-1, CSS-1h
Patching material	MC-70, MC-250	MS-2, MS-2h, CMS-2, CMS-2h
Note. In many locations, including most of the United States, cutback asphalts are not allowed due to environmental reasons. In these cases, some type of asphalt emulsion is generally used. Weather conditions may affect the grade and type of asphalt selected. In some weather conditions, it is desirable for the mixture to cure earlier than in others.		
Legend:		
min	minimum	
MC	medium curing	
MS	medium-setting	
RC	rapid-curing asphalt cutback	
RS	rapid-setting	
SC	slow-curing	
SS	slow setting	

Chapter 12

Material Sampling

When conducting tests, use the materials that represent those that will be used in construction; otherwise, test results will be misleading. The 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 or with a sample splitter. The 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.) Testing is done by the refineries; hence, it is good to get a copy of the certified test results for the binder material being produced. It is common practice to use the test results provided by the supplier and to take additional samples to be tested in case there is a problem. Take samples for analysis to identify properties of bituminous materials if records are not available. Obtain sufficient quantities of materials at the time of sampling to determine if materials meet specification requirements and to provide sufficient material for laboratory mix design. Normally, aggregates that produce 150 pounds of the desired gradation and 2 gallons of bituminous material are sufficient to conduct the mix design.

BITUMINOUS MATERIAL SAMPLING

12-1. Use clean, dry containers for sampling bituminous materials. Keep the containers tightly closed and properly marked. A sample for a routine laboratory examination should be at least one quart.

LIQUID MATERIALS

12-2. When sampling liquid bituminous materials from nonagitated vertical tanks, take samples from near the top, middle, and bottom. Consider testing the samples from the three levels separately to detect stratification. However, this additional testing may significantly increase the test time and cost. To reduce testing time and cost, combine the three samples and conduct one set of tests. 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 (figure 12-1, page 12-2). 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

12-3. When sampling solid or semisolid materials in drums, barrels, cartons, or bags, take samples at least 3 inches below the surface and at least 3 inches from the side of the container. Use a clean hatchet on hard material and a stiff putty knife on soft material.

AGGREGATE SAMPLING

12-4. Aggregate particles vary in size from large stones and rocks to gravels and sands and even to dust-size materials. These materials for paving may still be in their natural deposits or may be in individual stockpiles.

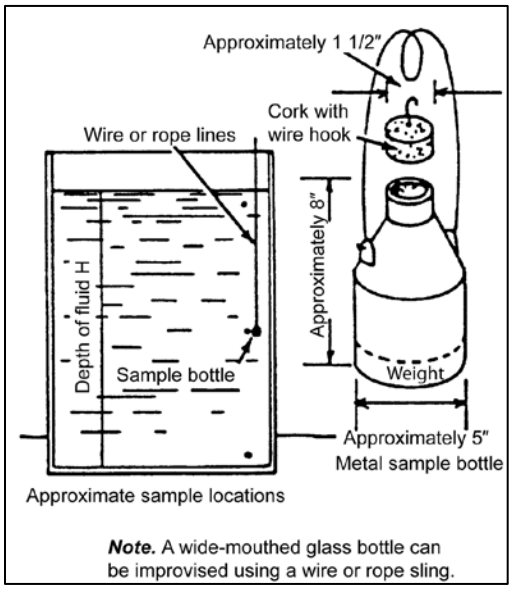


Figure 12-1. Sampling liquid bituminous materials from a tank car or storage tank

LEDGE AND QUARRY STONE

12-5. 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

12-6. 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, represents 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 samples for each change in strata from the pit representative. If the material being sampled is all sand, about 25 pounds is sufficient for tests. If it consists of sand and gravel, a larger sample (about 75 pounds) is required for preliminary tests. The coarser the gravel portion, the larger the sample required.

STOCKPILES

12-7. If the material has been stockpiled previously, take care in obtaining samples. There is a natural tendency toward segregation of materials in the stockpile. The material near the outer edges and near the base of the pile is likely to be coarser than the average material. Cut a face into the stockpile about halfway up the stockpile, and take a sample there. (Use care to ensure that material from above this point does not flow into the sampling area. This may require that a plate be used immediately above the sampling areas.) Do this at several points around the stockpile, and combine all of these samples into one large sample. This process should provide a representative sample of the stockpile. Combine samples from at least three different sections of the pile. Always follow good, safe practices when working around stockpiles.

COMMERCIAL AGGREGATES

12-8. 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 choose samples that are representative.

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Chapter 13
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 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 determines the proper equipment and the necessary safety procedures. The aggregate materials must also be tested for acceptable bituminous construction use.

BITUMEN FIELD-IDENTIFICATION TESTS

13-1. Perform field tests to identify the bituminous paving materials as AC, asphalt cutback, or asphalt emulsion. In addition, identify the viscosity grade of the bitumen. To distinguish among the several asphalt products, it is necessary to know something of their origin, physical properties, and the manner in which they are normally used. The identification procedure outlined in figure 13-1, page 13-2, is based on a consideration of the physical properties of these materials.

13-2. 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

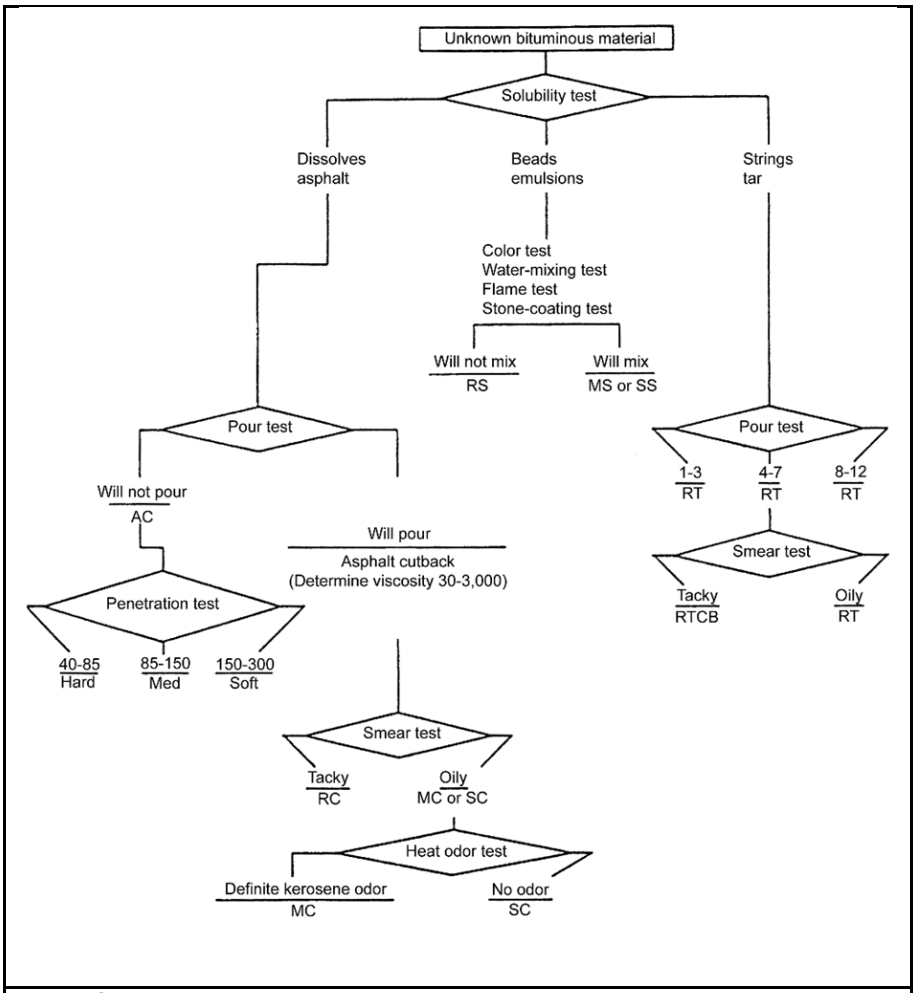
13-3. The first procedure in identifying an unknown bituminous material is to determine, by a solubility test, whether it is an asphalt product. Attempt to dissolve an unknown sample (a few drops, if liquid, or enough to cover the head of a nail, if solid) by stirring it in any petroleum distillate. 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. Tar products will not dissolve. If the sample is an asphalt, the sample distillate mix will consist of a dark, uniform liquid. ACs or cutbacks dissolve completely.

13-4. 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. While tars have been used in the past for paving surfaces, it is not recommended that they be used, due to environmental issues and the effects of emissions on the workers.

ASPHALT CEMENT AND CUTBACKS

13-5. Perform the following procedure to determine if the specimen is an AC or a cutback. The various grades of AC are solid at room temperature, while cutbacks are liquid; 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 AC. If it pours, it is a cutback or an emulsion.

Note. At 77°F the softest AC will not pour or deform if the container is tilted.



Legend:

AC	asphalt cement
MC	medium curing
med	medium
MS	medium setting
RC	rapid-curing asphalt cutback
RS	rapid-setting
RT	road tar
RTCB	road tar cutback
SC	slow curing
SS	slow setting

Figure 13-1. Identification of unknown bituminous materials

13-6. The various grades of AC 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 AC. 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 AC is present. If penetration is made with some difficulty, a medium AC is present. If penetration is made with ease, it is a soft AC in the high-penetration scale. Even the highest penetration will not pour or deform at 77°F if the container is tilted (table 13-1).

Table 13-1. Approximate comparison between AC grades

	Penetration Grade	*Viscosity Grade*	*Performance Graded*
Hard	60–70	AC–40	PG 70–16
Medium	85–100	AC–20	PG 64–22
Soft	120–150	AC–10	PG 58–28
Legend: AC asphalt cutback PG performance graded			

CUTBACK ASPHALT TESTS

13-7. The following tests are used to determine the grade of a cutback asphalt:

- **Pour test.** As stated previously, an AC will not pour at 77°F, but a cutback will. The pour test can be used to determine whether the unknown material is a cutback asphalt. If the material pours, it is a cutback asphalt. The approximate viscosity grade number of the cutback is found by comparing the flow to well-known materials (water, syrup). 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:
  - Water: 30.
  - Light syrup: 70.
  - Syrup: 250.
  - Molasses: 800.
  - Barely deform: 3,000.

Note. After the pour test, the approximate viscosity grade of the cutback is known, but the type (RC, MC, or SC) is not.

- **Smear test.** The smear test is used 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 cutback 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. A prolonged smear test is used 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 RC-3,000, it will cure completely in 3 hours, whereas RC-800 will take about 6 hours to cure. Even after 24 hours, an MC or SC will still be sticky.

ASPHALT EMULSION (ANIONIC) TESTS

13-8. Another asphalt material used in paving is asphalt emulsion, which is a mixture of asphalt, water, and an emulsifying agent. The anionic emulsion specifications cover three types of asphalt emulsion—RS grades 1 and 2; MS grades, 1, 2, and 2h; and SS grades 1 and 1h. There are other grades, but these are the grades that are most frequently used.

SOLUBILITY TEST

13-9. 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 other bituminous materials are black. The following are solubility tests:

- **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. However, mixing with excessive water may cause the emulsion to break, resulting in the asphalt and water separating. This can cause significant problems when trying to use an emulsion.
- **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). Use a metal spoon to mix a small amount (6 to 8 percent by weight) with damp sand do not saturate it. An RS emulsion will break so quickly that it will not be possible to mix it with sand. It breaks immediately, gumming up the spoon with the relatively hard original AC. If the sample is an MS or SS emulsion, the material will mix easily and coat the particles completely (including the mixing spoon) with a uniform coating of asphalt.

AGGREGATE IDENTIFICATION AND SELECTION

13-10. 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

13-11. The aggregate in a pavement must transmit the traffic load to the base, usually by 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, individual particles will not slip or slide over one another, but will lock together. However, more binder may be required since angular aggregate will not pack as closely together as rounded aggregate, hence requiring more asphalt to provide the desired air void level in the mixture. Although angular particles are desired, the aggregate should not contain an excessive number of flat or elongated particles, as these particles cause bridging and make compaction difficult.

13-12. Angular aggregates very seldom occur in nature, so it is necessary, in most cases, to crush the aggregate to obtain the desired angular particles.

HARDNESS AND DURABILITY

13-13. 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, which is the preferred method if it is available. The Mohs hardness scale may be used to determine the hardness of the aggregate. This scale is fully explained in TM 3-34.64/MCRP 3-17.7G. 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

13-14. 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 and adherence of bitumen and result in stripping of the binder. If aggregate particles are coated with silt or clay particles, the aggregate should be washed before being used to produce an asphalt mixture. The aggregate should be made as clean as possible with the equipment and manpower available. Some dust-size particles are needed in the mixture, but this dust should not coat the aggregate. The dust should come from the crushing of aggregate, and it should not stick to the coarser aggregate. Typically, most mixtures will need at least 3 percent dust in the mix and can have up to 6 to 8 percent dust.

MOISTURE SUSCEPTIBILITY

13-15. 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. The most common test for the detrimental effect of water on a bituminous mix is ASTM D4867.

13-16. A rough estimate of the stripping potential can be determined by following ASTM D3625. This test involves placing a sample of asphalt mixture in water, boiling it, and then making a visual examination to determine the percentage of exposed aggregate surface.

13-17. Asphalt 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 clean, sound stone; slag; or gravel aggregate and mineral filler produced from crushed stone will show test values of swell of less than 1.5 percent of the original specimen thickness.

GRADATION

13-18. The following designations help identify aggregates:

- Uniform gradation occurs when all particles are about the same size, normally less than 1 inch. These mixtures tend to have high void levels, and permeable, and mixtures tend to be weak.
- Macadam gradation consists of uniformly sized particles, 1 inch or larger.
- Gap-graded mixtures have a small amount of certain-size materials. For instance, most gap-graded mixtures have a deficiency of material on the middle sieve sizes. These mixtures typically result in open mixtures that may be permeable to air and water.
- Dense gradation occurs when there is a good representation of all particle sizes, including coarse, fine, and mineral fillers. These mixtures tend to be strong, with little permeability.

PARTICLE SIZE

13-19. 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. The following are particle sizes:

- **Coarse aggregate.** Coarse aggregate is material larger than a No. 4 sieve.
- **Fine aggregate.** Fine aggregate passes the No. 4 sieve, but is retained on the No. 200 sieve. However, fine aggregates always contain some percentage of material passing the No. 200 sieve. In bituminous paving, the fine aggregate may consist of natural sand or crushed fine aggregate (ASTM D1073).
- **Mineral filler or mineral dust.** Mineral filler or mineral dust is nonplastic materials which pass the No. 200 sieve. Most clays are too plastic and aggregates containing clay particles are normally unacceptable. Generally, crushed rock dust, agricultural mineral filler, lime, or portland cement may be used as mineral filler (ASTM D546). Generally, fine aggregates will contain sufficient mineral filler and no additional filler has to be added.

Chapter 14

Bitumen Testing

The field identification tests for bitumen 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 an AC, cutback, or emulsion. Bituminous materials are manufactured to meet specifications established by the American Association of State Highway and Transportation Officials (AASHTO) and the ASTM. These specifications define the limits permitted in the manufacture of the material and assure the user that the material will possess definite characteristics and fulfill the project requirements. The tests used to specify AC vary considerably depending on the grading system used. These tests are not normally conducted in the field laboratory but there may be some lab equipment available that will allow an estimate of the asphalt grade. The equipment for performing these tests is not included in the asphalt test set and is not normally available to the materials technician. However, the test procedures are discussed below. Specific gravity of a 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 to convert weight to volume. Space is provided on DD Form 1216 *(Specific Gravity of Bituminous Mix Components)* to make this determination (figure 14-1, page 14-2). The specific gravity of ACs will usually be in the range of 1.00 to 1.06, with most ACs being around 1.03. 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. In acceptance and control testing on a job, it is used as a check on the uniformity of succeeding shipments of asphalt.

CLEVELAND OPEN-CUP AND TAG OPEN-CUP FLASH POINT AND FIRE POINT TESTS

14-1. These tests are applicable to asphalt 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. The following are flash point and fire point tests—

- **ASTM D92.** Perform the ASTM D92 test on all petroleum products, except fuel oils and those having open-cup flash points above 175°F.
- **ASTM D1310.** Perform the ASTM D1310 test on RC- and MC-asphalt cutbacks having a flash points below 325°F.

Note. Either test can be used when the flash point is between 175° and 325°F.

SPECIFIC GRAVITY OF BITUMINOUS MIX COMPONENTS — DATE 2 APR 2010

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 = (Line 1) / (Line 3)	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 = (Line 4) / (Line 8)	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 = (Line 9) / (Line 13)	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 = (Line 18) / (Line 21)	9.9402 / 9.7429 = 1.020			

TECHNICIAN (Signature): SP4 Jones — COMPUTED BY (Signature): SSG Schmick — CHECKED BY (Signature): SGT. Allen

DD FORM 1216, DEC 65 — PREVIOUS EDITION OF THIS FORM IS OBSOLETE. — USAPPC V1.00

Legend:

Apr	April
DD	Department of Defense
Dec	December
No.	number
SGT	sergeant
SP4	specialist four
SSG	staff sergeant
USAPPC	United States Army Publications and Printing Command

Figure 14-1. Sample DD Form 1216

SAMPLE

PENETRATION TEST

14-2. The penetration test is sometimes used to specify the grade of an AC. 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 in one hundredths of a centimeter. Other conditions of temperature, load, and time are sometimes used for special testing purposes, but these are exceptions and should only be used in special cases.

DUCTILITY TEST

14-3. 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 (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.

14-4. 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 the 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 (specimen imperfections or impurities).

SOFTENING-POINT TEST

14-5. For the softening-point test (ASTM D61), position a 3/8-inch-diameter steel ball on a brass ring filled with asphalt. Place the assembly in 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.

14-6. 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 the effects of aging. If asphalt shows an unusual increase in the softening point, considerable aging and hardening have occurred.

VISCOSITY TESTS

14-7. There are two viscosity tests for identifying the qualities of bitumen:

- **Saybolt-Furol test.** 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 the field application of the bitumen.
- **Capillary tube viscometer.** This test, like the Saybolt-Furor test, measures the time that a given amount of liquid asphalt will flow through a tube of standard dimensions under rigidly controlled conditions of temperature and pressure (or vacuum). At higher temperatures, the flow through the tube is caused by gravity and no vacuum is needed. This test is good from 6 to 100,000 centistokes so it can be used for liquid asphalts and ACs at higher temperatures (such as 275°F). At lower temperatures (140°F) a vacuum has to be used to create flow in the material (ASTM D2171).

PERFORMANCE-GRADED TESTS

14-8. There are four tests that are primarily conducted as a part of the PG system. These tests include the pressure aging vessel, dynamic shear rheometer, bending beam rheometer, and direct tension tests. These

tests are conducted at various temperatures to establish the grade of the asphalt being used. These tests are fairly complicated and are almost certainly not used in the field. The standard specification for PG asphalts is provided in ASTM D6373. The pressure aging vessel test is described in ASTM D6521. The dynamic shear rheometer test is described in ASTM D7175. The bending beam rheometer test is described in ASTM D6648. The direct tension test is described in ASTM D6723.

SOLUBILITY TESTS

14-9. The solubility of AC 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 the installation environmental office for further guidance as to the safe handling and disposal of trichloroethylene.

14-10. 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.

14-11. The solubility test is a quality control test used in specifications to ensure that the AC is not contaminated with mineral matter or is not improperly refined.

14-12. 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. A penetration, viscosity, or one or more of the PG grading tests may also be conducted on the sample after the heating period to evaluate changes in the asphalt. The primary purpose of the thin-film oven test is to estimate the property changes of the asphalt binder that will take place when it is mixed with the hot aggregates during production of the asphalt mixture.

Chapter 15

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 (shape, surface, cleanliness) are determined visually. Durability and hardness cannot be seen, but require knowledge based 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 is typically established by the specification requirements. In bituminous paving, the aggregate constitutes the bulk of the mixture. Common practice subdivides the bituminous aggregates into coarse aggregate, fine aggregate, and mineral filler. The No. 4 sieve separates coarse from fine aggregate, and the No. 200 sieve is the lower limit for fine aggregate. The distribution of the different sizes determines how many voids will remain in the aggregate 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

15-1. A sieve analysis of the aggregates to be used in a paving mixture is required to determine the particle size distribution.

MINERAL FILLER

15-2. In bituminous paving, particles finer than the No. 200 sieve are referred to as a mineral filler. To measure the amount of filler and percentages passing each sieve in a selected sample, perform a washed sieve analysis (ASTM C136).

FINE AND COARSE AGGREGATE

15-3. Specific size designations of coarse and fine aggregates are specified as shown in ASTM D448. Materials are generally ordered from the aggregate supplier using the designated numbers. For example, a No. 6 aggregate would have 90 to 100 percent passing the 3/4-inch sieve and 0 to 15 percent passing the 3/8-inch sieve. The sieve analysis with prewashing must be made using suitable sieves. When conducting gradation tests, record the results on DD Form 1206 and plot the results as a gradation curve on DD Form 1207. When testing aggregates, obtain a sample as described in ASTM D75 and reduce the sample down to the proper size by using a sample splitter (if available) or by quartering if necessary. The minimum required size of the sample depends on the maximum size of particles as specified in the testing methods.

SPECIFIC GRAVITY

15-4. The bulk specific gravity of aggregates and mineral filler used in bituminous paving mixtures is required to compute the percent of voids in the mineral aggregate. The air voids in the asphalt mixture are determined from the density of a compacted mixture and the theoretical maximum specific gravity (ASTM D2041) of the mixture.

APPARENT AND BULK SPECIFIC GRAVITY OF COARSE AGGREGATE

15-5. Apparent and bulk specific gravity can be determined using the method described for apparent and bulk specific gravity in ASTM C127. The apparent specific gravity is much easier to measure, and it can be determined using the Dunagan apparatus furnished with the concrete test set.

Equipment

15-6. Use the concrete test set to determine the apparent specific gravity. The test set includes—

- The Dunagan apparatus (figure 15-1).
- Sieves (2-, 1 1/2-, 1/2-, and 3/8-inch and Nos. 4, 10, 40, 60, 80, 100, and 200).
- Evaporating dishes.
- An electric oven.
- Pans.

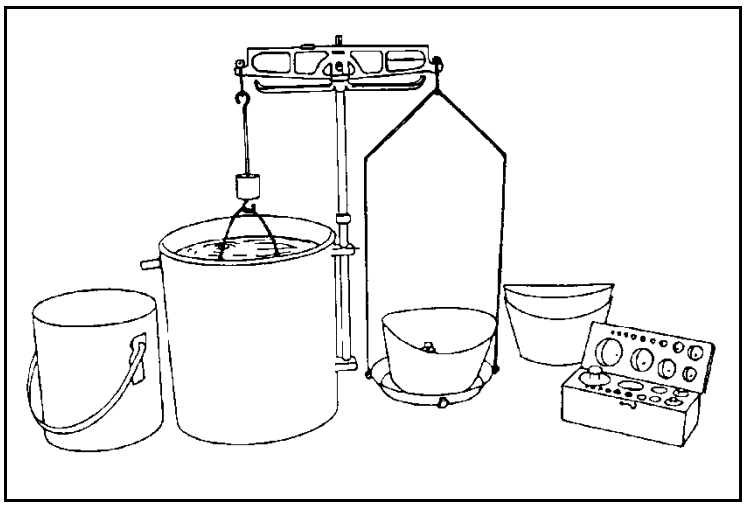


Figure 15-1. Dunagan apparatus

Steps

15-7. Perform the following steps to determine the apparent specific gravity:

- **Step 1.** Select (by splitting or quartering) about 5,000 grams of aggregate from the sample, do not include particles smaller than the 3/8-inch sieve.
- **Step 2.** Wash the aggregate to remove dust or other coatings, and dry it to a constant weight in the oven. Record the total weight of the oven-dried aggregate on DD Form 1216 (figure 14-1, page 14-2).
- **Step 3.** Immerse the aggregate in water at 59° to 77°Farenheit for 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 of the sample. Record the weight.

Calculations

15-8. The calculations required to determine the apparent specific gravity of coarse aggregate are shown on DD Form 1216 and are self-explanatory (figure 14-1).

BULK AND APPARENT SPECIFIC GRAVITY OF FINE AGGREGATE (CALIBRATED FLASK)

15-9. The fine aggregate bulk and apparent specific gravity can be determined according to ASTM C128. It is much easier to determine the apparent specific gravity than the bulk specific gravity. The procedure discussed below can be used to determine the apparent specific gravity when a calibrated flask is available.

Equipment

15-10. Use the following items (figure 15-2) to determine the apparent specific gravity of fine aggregate (calibrated flask):

- A balance, 2,000-gram, 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°F gradations).
- An absorbent paper or cloth.

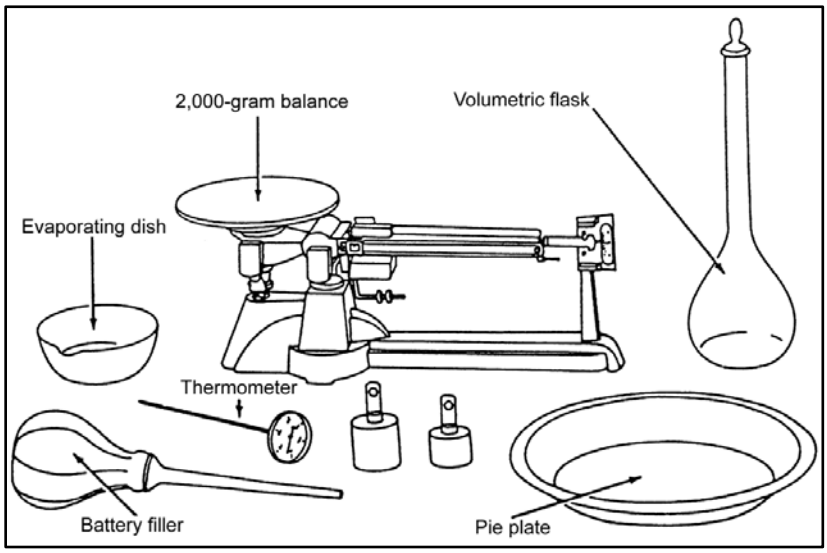


Figure 15-2. Calibrated-flask apparatus for determining apparent specific gravity of fine aggregate

Steps

15-11. Perform the following steps for particles finer than the No. 4 sieve (the data is recorded on DD Form 1208 [figure 4-2, page 4-3]):

- **Step 1**. Calibrate a 500-milliliter volumetric flask.
- **Step 2**. Dry a representative sample, weighing about 500 grams, to a constant weight in the electric oven.
- **Step 3.** Determine the oven-dried weight of the cooled sample, and record in on DD Form 1208 (figure 4-2)
- **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 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 (figure 15-3).

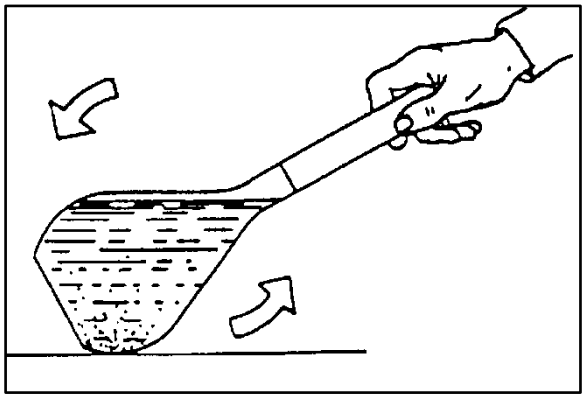


Figure 15-3. Manipulation of calibrated flask to remove air

- **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 outside of the flask.
- **Step 8.** Determine the weight of the flask, aggregate, and water. Record the weight on DD Form 1208.
- **Step 9**. Measure and record the temperature.

Calculations

15-12. The calculations for apparent specific gravity of fine aggregate are the same as those indicated for soil in figure 4-2, page 4-3.

APPARENT SPECIFIC GRAVITY OF FINE AGGREGATE (UNCALIBRATED FLASK)

15-13. Perform the procedure below to determine the apparent specific gravity when a calibrated flask is not available.

Equipment

15-14. 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

15-15. 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 plus 9°F.
- **Step 2**. Obtain and record the dry weight after the sample has air cooled.
- **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 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.
- **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 68°Farenheit. 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 and 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°Farenheit, 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°Farenheit on DD Form 1216. This needs to be done only once for a given flask, and the value can be tabulated and used in subsequent tests. The weight of the flask filled with only water at 68°Farenheit must be known.

Calculations

15-16. Indicate the calculations necessary to determine the apparent specific gravity of fine aggregate using an uncalibrated flask on DD Form 1216.

SPECIFIC GRAVITY OF MINERAL FILLER

15-17. The specific gravity of mineral filler used in bituminous mixes is required for calculation of the voids in the mineral aggregate. Most of the time the mineral filler will come from the fine aggregate. In this case there is no additional mineral filler added so this test is not needed. The methods described in the specific-gravity test apply (including procedures, calibration, testing, and calculations). Details can be found in ASTM D854.

LOS ANGELES ABRASION TEST

15-18. 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 by an electric motor. Inside the drum, there is a shelf that picks up the aggregate sample and a charge of steel balls and drops them together on the opposite side of the drum. Details can be found in ASTM C131. The LA abrasion test is used to evaluate the ability of the aggregate to resist degradation during handling and when exposed to traffic after the pavement is constructed. Materials with a higher LA abrasion will tend to break down more than aggregates with lower abrasion values.

15-19. Job specifications usually require that the loss in weight as determined in the Los Angeles abrasion test is not greater than 40 percent for pavement aggregates and 50 percent for base and subbase aggregates.

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Chapter 16

Bituminous-Mix Design

Hot-mix bituminous concrete for pavements is a mixture of blended aggregate that is coated with bituminous cement binder. The materials are heated when mixed so that the bitumen becomes fluid and thoroughly coats the aggregate particles. The design of a bituminous concrete mix is performed to determine an economical blend of aggregates meeting the specification requirements and to identify the proper bituminous cement content. 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. The testing of ingredients and the mix is started before and continued throughout the paving operations. (FM 5-430-00-1/AFJPAM 32-8013/MCRP 3-17.7A, Volume I covers the design considerations in more detail. The paving operations, blending, and bitumen content criteria are explained in UFC 3-250-03. The testing phases are described in this manual.) The selection of the mix ratios of materials is tentative and will likely need to be changed during mixture production. The aggregate and bitumen used for mix design should be the same as that used during construction. The aggregates and binder must meet definite requirements. At times, it is 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

16-1. 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

16-2. Perform the following steps to determine the most economical blend of components:

- **Step 1**. Sample the available aggregates and binder materials.
- **Step 2**. Perform a sieve analysis of the available aggregates.
- **Step 3**. Determine the aggregate blend that will achieve the specified gradation (UFC 3-250-03). Plot the selected blended gradation on a graph, with the allowable limits, to see that the blend conforms to the specification requirements.
- **Step 4**. Determine the specific gravity of the components.
- **Step 5.** Use selected percentages of bitumen (UFC 3-250-03), make trial mixes, and determine the test properties.
- **Step 6**. Plot the test properties on individual graphs, using the selected bitumen percentages. Draw smooth curves through the plotted points.

- **Step 7**. Select the bitumen content that provides 4 percent air voids in the mixture.
- **Step 8**. Determine mixture properties at the selected bitumen content. If the properties are within the specification requirements, this is the optimum asphalt content. If one or more properties do not meet the specification requirements, adjust the binder content slightly so that the mixture meets the specification requirements. If the properties do not all meet with some slight binder content adjustment, the aggregate proportions or sources must be changed and the mix design performed again.

VARIABLES

16-3. Gradation specifications are based on the limits established as satisfactory by USACE. 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 grade (AC).
- Loading (low tire pressure [100 pounds per square inch and under] or high tire pressure [more than 100 pounds per square inch]).
- The maximum size of the aggregate to be used on the project.

BLENDS

16-4. 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 UFC 3-250-03.

OPTIMUM BITUMEN CONTENT

16-5. The determination of optimum bitumen content is based on the Marshall test method or some other method of mix design (such as Superpave) (ASTM D6925).

MARSHALL TEST

16-6. The purposes of the Marshall test 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 pounds per square inch of pressure and 75 blows on each end of the specimen for tires with greater than 100 pounds per square inch of pressure.
- **Find the optimum bitumen content by determining the asphalt content that produces 4 percent air voids in the laboratory-compacted mixture.** This method of picking the optimum asphalt content does not work very well for sand asphalt mixtures, but does work well for dense graded asphalt mixtures having maximum aggregate sizes of approximately 1/2 inch and larger. Typically, sand mixtures will require a higher void level in the mixture.
- **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.

16-7. The Marshall test is applicable for hot mixes containing penetration, viscosity, or performance graded of AC and containing aggregate with not more than 10 percent of the aggregate larger than the 1-inch sieve. The procedure for handling large aggregate and for cold mixes is described later in this chapter.

Steps

16-8. The following paragraphs are procedures and examples to determine the optimum asphalt content for one particular blend of aggregates.

Step 1. Select the Bitumen Content

16-9. Start the laboratory tests by estimating the optimum amount of bitumen for the aggregate to be tested. Continue the tests until the 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. One percent incremental changes of bitumen content may be used for preliminary work. However, use increments of 1/2 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 UFC 3-250-03.

Step 2. Prepare the Aggregate

16-10. The 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 a 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 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 particles will exceed 1 inch. (Special handling of oversize aggregates is covered later.)

Step 3. Prepare the Hot Mixture

16-11. Heat the bitumen and aggregates to specified temperatures for mixing. 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°F 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 thoroughly as possible to ensure that the bitumen is uniformly distributed throughout the aggregate.

Step 4. Compact the Hot Mixture

16-12. 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 the time of compaction should be no more than 25°F lower than the mixing temperatures provided in table 11-3, page 11-5.

16-13. Apply the required number of blows with the compaction hammer (figure 16-1, page 16-4). Remove the base plate 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 pressure of 100 pounds per square inch or less should be compacted by 50 blows on each end of the specimen. If the pavement is being designed for aircraft, whose tires carry pressure greater than 100 pounds per square inch, the compactive effort should be 75 blows per side.

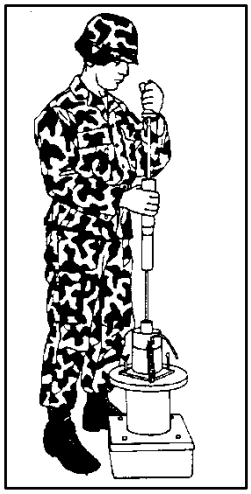


Figure 16-1. Compaction of bituminous trial mix specimens

Step 5. Cool the Hot Specimen

16-14. After compacting, remove the baseplate and collar; air-cool the mold and specimen (normally overnight), or place them in cold water for at least 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.

Step 6. Weigh the Specimen in Air and in Water

16-15. 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 (figure 16-2). These tests should be conducted according to ASTM D2726.

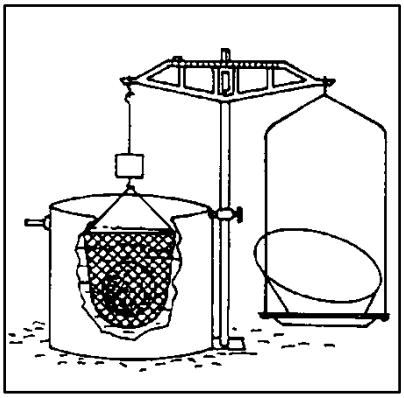


Figure 16-2. Weighing compacted bituminous specimen in water

16-16. DD Form 1218 *(Marshall Method–Computation of Properties of Asphalt Mixtures)* (figure 16-3) provides space for recording the measurements made at room temperature. A direct weight in water of open-textured or porous specimens will give erroneous results because of water penetration and absorption. For such specimens, other means must be used to determine the volume. Two means of measuring the volume of the porous specimens include the use of paraffin (ASTM D1188) and the use of automatic vacuum sealing methods (ASTM D6752).

Step 7. Measure Stability and Flow

16-17. The purpose of the stability and flow test is to determine the potential for the rutting or shoving of the mixture under traffic. If the stability is too low or the flow is too high, this is an indication of a potential for rutting or shoving. The data is recorded, and calculations are made as shown in table 16-1, page 16-7.

MARSHALL METHOD - COMPUTATION OF PROPERTIES OF ASPHALT MIXTURES

DATE OF COMPUTATION: 2 APR 2010

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) IN AIR	WEIGHT (Grams) IN WATER	VOLUME CC	SPECIFIC GRAVITY ACTUAL	SPECIFIC GRAVITY THEO-RIZED	AC BY VOLUME (Percent)	VOIDS (Percent) TOTAL MIX	VOIDS (Percent) FILLED	UNIT WEIGHT TOTAL MIX (Lb./Cu. Ft.)	STABILITY (Pounds) MEASURED	STABILITY (Pounds) CON-VERTED	FLOW UNITS OF 1/100 IN.
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))	(f)/(f+j)	(g x 62.4)		*	
A-1	3.5		1228.3	716.3	512.0	2.399						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	2.413						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	9.4	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

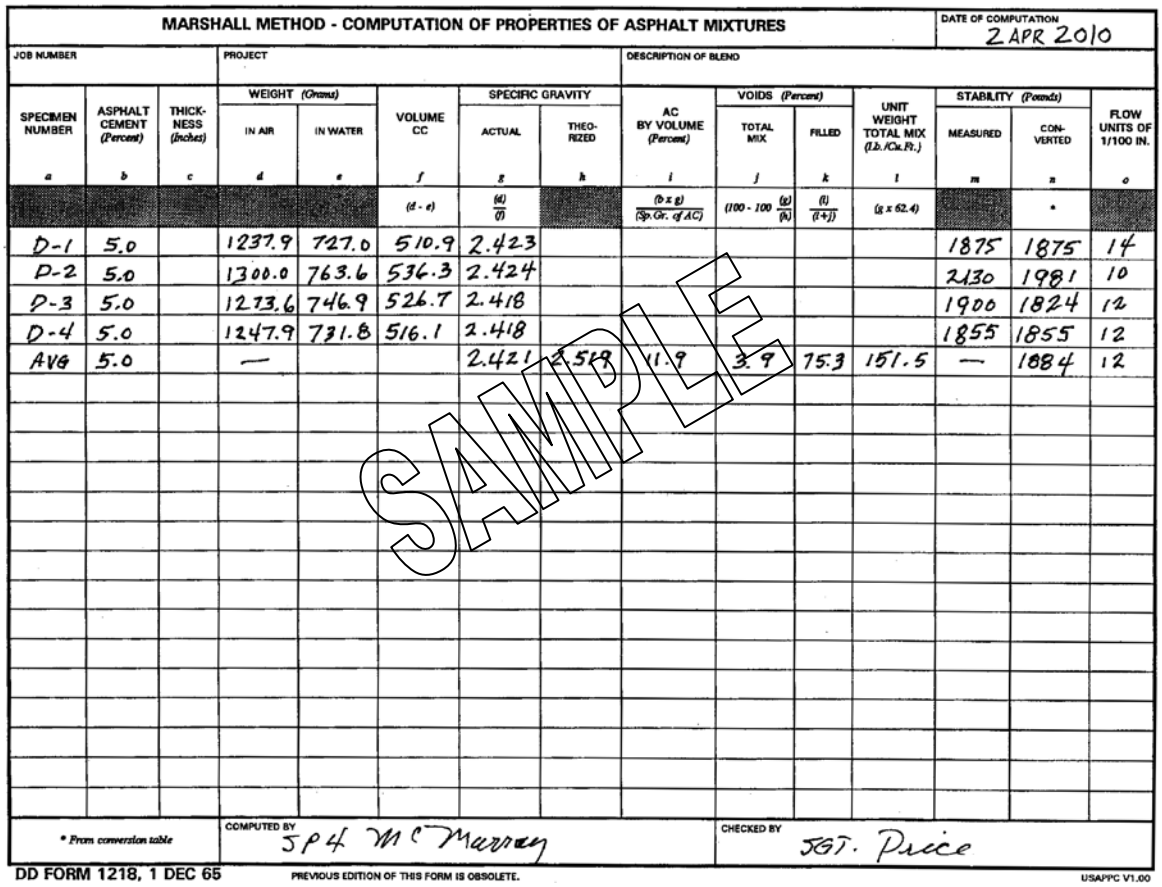
* From conversion table

COMPUTED BY: SP-4 Alexandria M. Murray | CHECKED BY: SGT Arthur Price

DD FORM 1218, 1 DEC 65 — PREVIOUS EDITION OF THIS FORM IS OBSOLETE. — USAPPC V1.00

SAMPLE

Figure 16-3. Sample DD Form 1218



MARSHALL METHOD - COMPUTATION OF PROPERTIES OF ASPHALT MIXTURES

DATE OF COMPUTATION: 2 APR 2010

JOB NUMBER | PROJECT | DESCRIPTION OF BLEND

SPECIMEN NUMBER	ASPHALT CEMENT (Percent)	THICKNESS (Inches)	WEIGHT (Grams) IN AIR	WEIGHT (Grams) IN WATER	VOLUME CC	SPECIFIC GRAVITY ACTUAL	SPECIFIC GRAVITY THEORIZED	AC BY VOLUME (Percent)	VOIDS (Percent) TOTAL MIX	VOIDS (Percent) FILLED	UNIT WEIGHT TOTAL MIX (Lb./Cu.Ft.)	STABILITY (Pounds) MEASURED	STABILITY (Pounds) CONVERTED	FLOW UNITS OF 1/100 IN.
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)		*	
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	2.589	11.9	3.9	75.3	151.5	—	1884	12

SAMPLE

* From conversion table

COMPUTED BY: SP4 Mc Murray

CHECKED BY: SGT. Price

DD FORM 1218, 1 DEC 65 PREVIOUS EDITION OF THIS FORM IS OBSOLETE. USAPPC V1.00

Legend:

AC	asphalt cement
AGG	aggregate
Apr	April
AVG	average
cc	cubic centimeter
cu ft	cubic foot
DD	Department of Defense
Dec	December
g	gram
gr	gravity
h	hour
in	inch
lb	pound
SGT	sergeant
sp	specific
SP-4	specialist 4
SP4	specialist 4
USAPPC	United States Army Publications and Printing Command

Figure 16-3. Sample DD Form 1218 (continued)

Table 16-1. Marshall test specifications and determination of optimum asphalt content

		Criteria		Determination of OAC	
Property	***Course***	***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	Not used	Not used
Unit weight	Surface	N/A	N/A	Not used	Not used
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	Not used	Not used
Stability	Binder	1,800 or higher	500 or higher	Not used	Not used
Unit weight	Binder	N/A	N/A	Not used	Not used
Flow	Binder	16 or less	20 or less	Not used	Not used
Percent voids total mix	Binder	3 to 5 percent	3 to 5 percent	6 percent	5 percent
Percent voids filled w/AC	Binder	70 to 80 percent	75 to 85 percent	Not used	Not used
Stability	Sand asphalt	*	500 or higher	*	Not used
Unit weight	Sand asphalt	N/A	N/A	*	Not used
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	*	Not used
*The criteria for sand asphalt to be used in designing pavement for high-pressure tires have not been established.					
Legend:					
AC	asphalt cement				
ASTM	American Society for Testing and Materials				
OAC	optimum asphalt content				
N/A	not applicable				
w/	with				

Step 8. Calculate and Present Results

16-18. DD Form 1218 is used to summarize the measured and calculated Marshall test properties. The specimen numbers, placed on each specimen with a marking crayon, are given for identification.

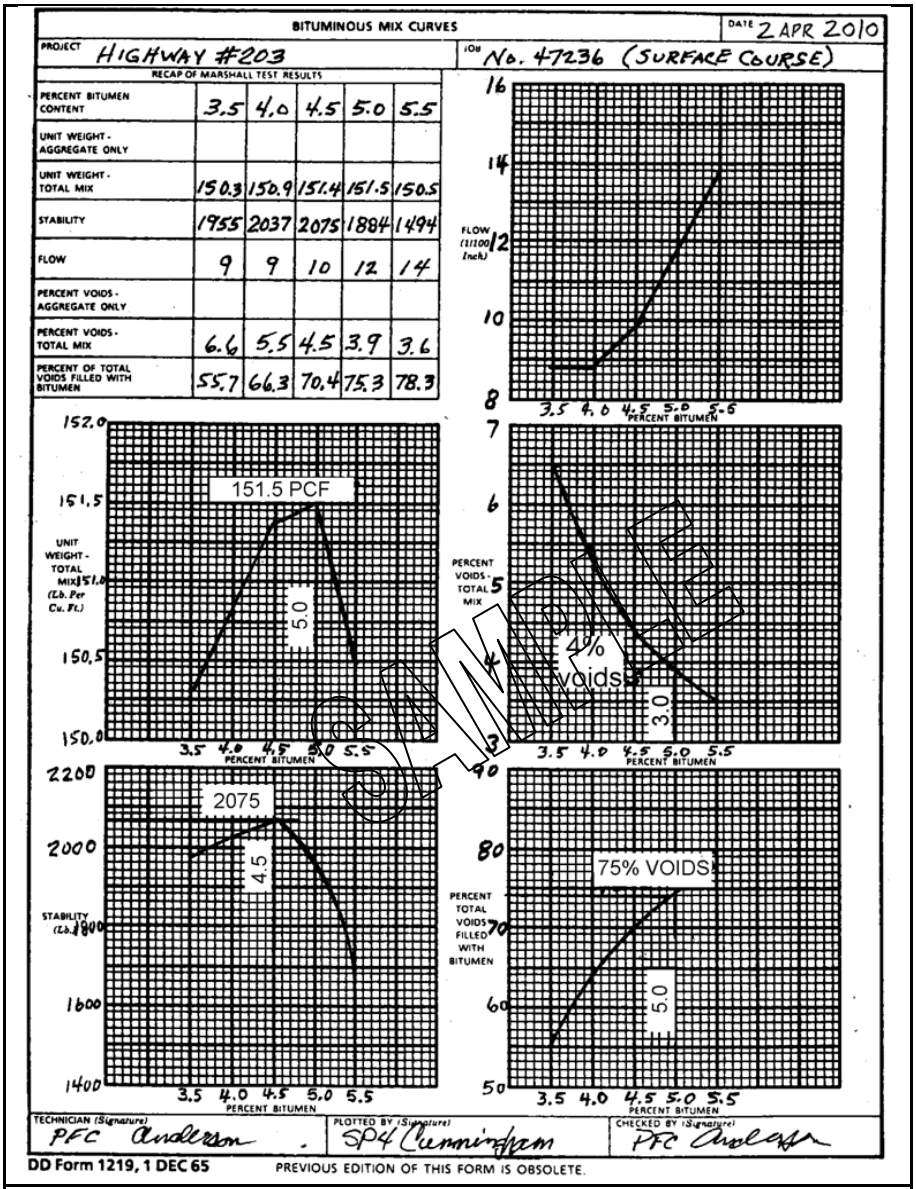
> ***Note.*** There are four duplicate tests for each bitumen content and these four test values are averaged in each instance. Note also that the specimen's thickness is not indicated, since the volume can be used to find the stability correlation ratio.

16-19. The theoretical specific gravity is determined according to ASTM D2041 and is used to calculate the voids. Note that the stability value is shown directly in pounds. 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 as presented in the standard. All other calculations are indicated directly on the form.

Step 9. Complete a Graphical Presentation

16-20. The average Marshall test properties from the tabulation for each bitumen content are shown graphically on DD Form 1219 *(Bituminous Mix Curves)* (figure 16-4, page 16-8). 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.



Legend:

Apr	April	No.	number
cu ft	cubic foot	pcf	pound per cubic foot
DD	Department of Defense	PFC	private first class
Dec	December	SP4	specialist four
lb	pound		

Figure 16-4. Sample DD Form 1219

Step 10. Apply the Marshall Test

16-21. Table 16-1, page 16-7, lists the criteria for determining the optimum bitumen content along with the Marshall specifications for a satisfactory mix. Once the optimum bitumen content is obtained, the Marshall properties at this average are read from curves representing the mix. These values are compared with the specification limits to evaluate the mix.

Step 11. Apply the Modified Marshall Test for Cold-Mix Pavement

16-22. 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.

TEST VARIATIONS

16-23. These variations apply to aggregates with 10 percent or more larger than 1 inch. 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 as follows:

$$G_{\mathrm{A}} = \frac{100}{\left[\frac{A}{C} + \frac{B}{D}\right] \times f}$$

where—
G_A = adjusted specific gravity
A = weight of dry, 1-inch material (as a percentage) of the total batch weight (bitumen + 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.

16-24. When using emulsions, it is better to design the mix with some compactive effort and select the asphalt emulsion for some void level, typically 4 to 6 percent. However, if samples cannot be compacted, use the following formula to determine the amount of 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

Note. Express all percentages as whole numbers. Absorptive aggregate (slag, lime rock, vesicular lava, coral) will require additional asphalt to fill the voids in the aggregate particles.

SUPERPAVE MIX DESIGN

16-25. Superpave mix design is used by most state departments of transportation and many locations outside the United States The Surperpave mix design method has also been used some for airfields. This procedure for preparing and compacting samples is described in ASTM D6925. The mix design procedure is the same as described for the Marshall test, but the method of compaction is different. A gyratory compactor is used for compaction as shown in figure 16-5. The Superpave gyratory compactor compacts samples that are 6 inches in diameter; hence, the aggregate can be up to 1.5 inches without causing any problem.

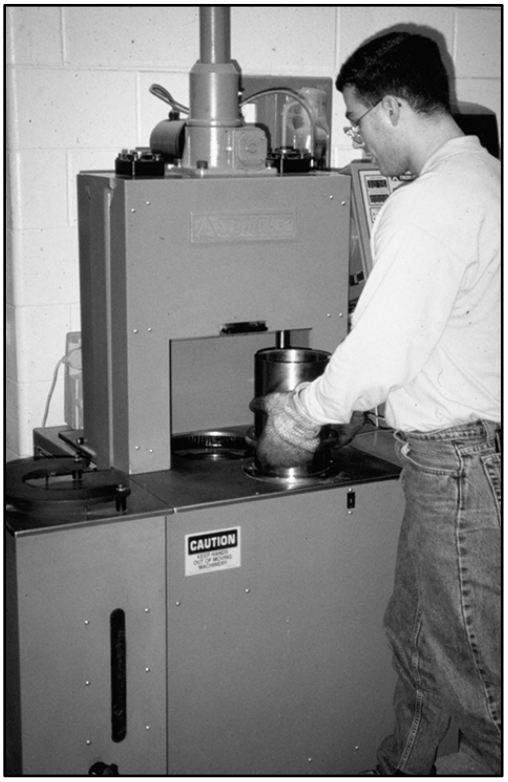


Figure 16-5. Superpave gyratory compactor

16-26. The samples are compacted in the gyratory compactor and tested for volumetric properties similar to that for Marshall test. All volumetric testing of the Superpave samples is the same as for Marshall; however, there are no Marshall stability and flow tests conducted on the Superpave gyratory-compacted samples. The asphalt content for the Superpave designed mixtures is selected to provide 4 percent air voids

just like with the Marshall procedure. The number of gyrations to be used for compaction has to be determined and provided in the specifications. Mixtures designed for high traffic will need a higher number of gyrations, and mixtures designed with lower traffic will need a lower number of gyrations. $N_{initial}$ and $N_{maximum}$ requirements are normally part of the Superpave design procedure, but these properties are not used as part of the procedure for designing military airfield mixtures. Only N_{design} is used for military airfields.

JOB MIX FORMULA

16-27. 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 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 Form 1217 *(Bituminous Mix Design–Aggregate Blending)* (figure 16-6) and DD Form 1207 (figure 16-7, page 16-12), the portions were 45 percent coarse aggregate, 30 percent fine aggregate, 20 percent fine river bar sand, and 5 percent limestone dust or mineral filler. Usually, there is enough dust in the crushed aggregates so that mineral filler does not have to be added to the mixture. So, adding mineral filler separately is not typically done. Also, the amount of natural sand is 20 percent, which is satisfactory for road projects, but exceeds the 15 percent that is allowed for airfields.

BITUMINOUS MIX DESIGN - AGGREGATE BLENDING

DATE: 2 APR 2010

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										
COARSE 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 16-6. Sample DD Form 1217

Legend:	
Apr	April
CA	course aggregate
DD	Department of defense
Dec	December
FA	fine aggregate
FRBS	fine river bar sand
LSD	limestone dust
No.	number

Figure 16-6. Sample DD Form 1217 (continued)

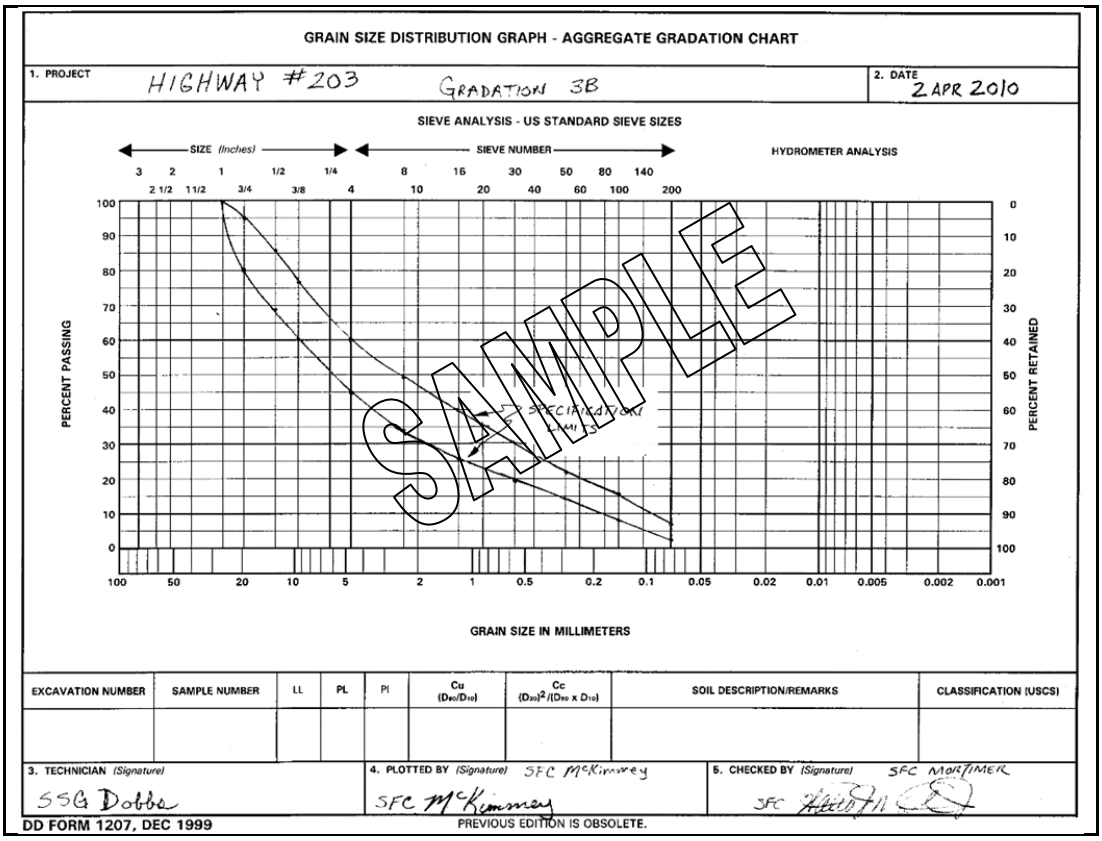
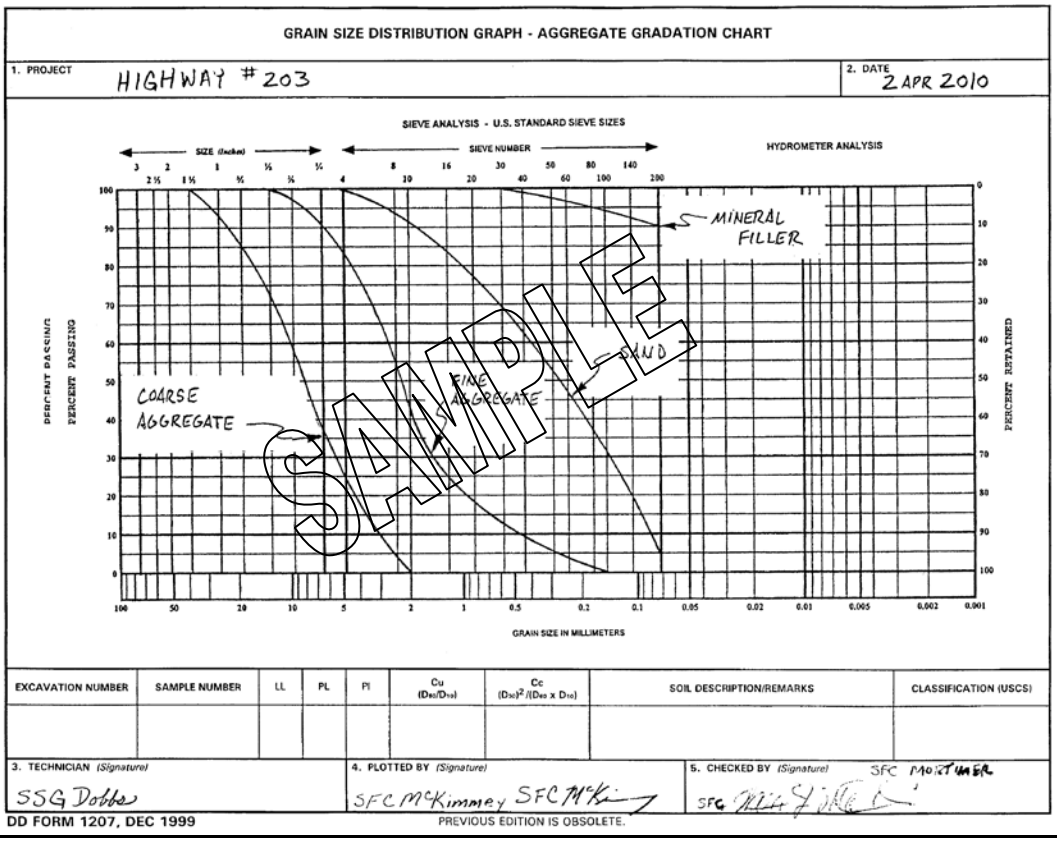


Figure 16-7. Sample DD Form 1207



Legend:

Apr	April
DD	Department of Defense
Dec	December
SFC	sergeant first class
SSG	staff sergeant
USCS	Unified Soil Classification System
LL	liquid limit
PL	plastic limit
PI	plasticity index
U.S.	United States

Figure 16-7. Sample DD Form 1207 (continued)

16-28. The job-mix formula is then computed as follows:

$$Coarse\ aggregate\ =\ 95.3\ \times\ 45\ =\ 42.9\ percent$$
$$Fine\ aggregate\ =\ 95.3\ \times\ 30\ =\ 28.6\ percent$$
$$FRBS\ =\ 5.3\ \times\ 20\ =\ 19.0\ percent$$
$$Mineral\ filler\ =\ 95.3\ \times\ 5\ =\ 4.8\ percent$$
$$Bitumen\ =\ 4.7\ percent$$
$$Total\ =\ 100.0\ percent$$

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Chapter 17

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

17-1. Figures 17-1 and 17-2 are schematic drawings of a batch plant and a drum-mix plant.

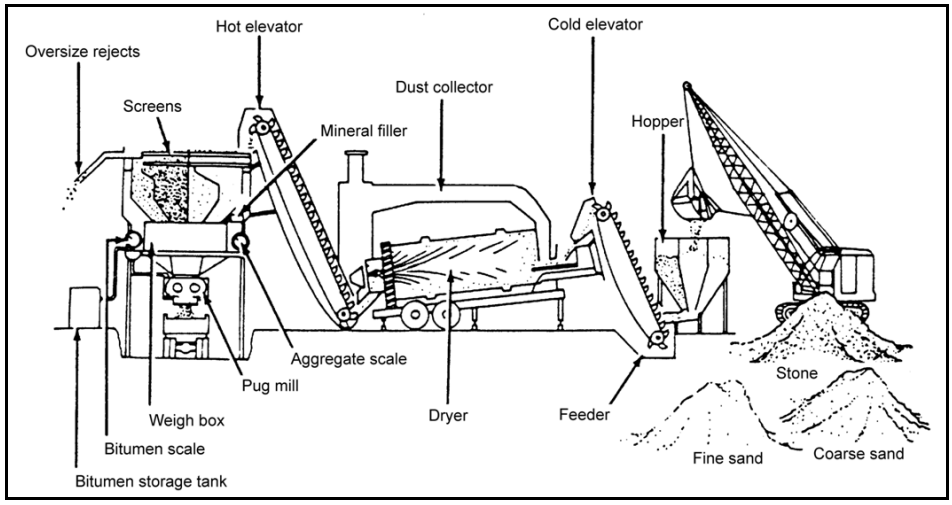


Figure 17-1. Bituminous hot-mix batch plant

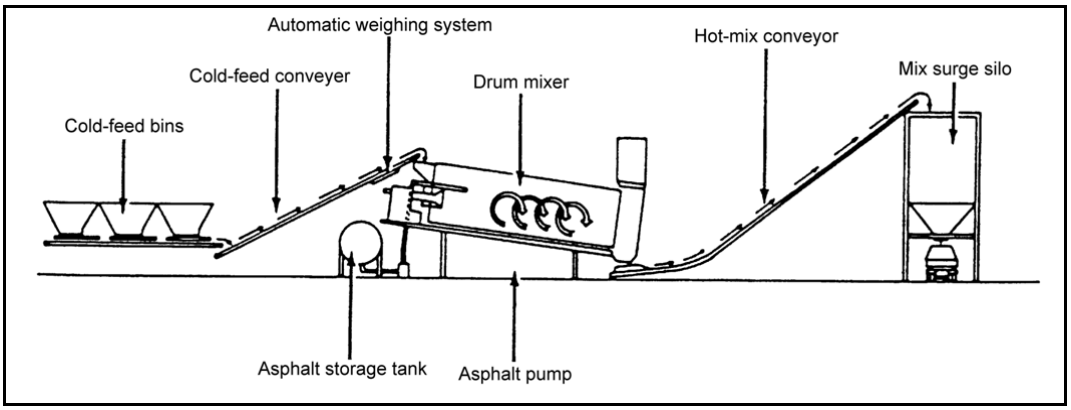


Figure 17-2. Drum mix plant

BATCH PLANT

17-2. In the operation of a high-type bituminous paving plant, aggregates from two or more sources (typically at least three) are fed into the aggregate dryer in the approximate proportions required to produce the desired gradation. This initial proportioning is usually accomplished by a hopper-type feeder, operating from individual bins for each aggregate being used in the mixture. The aggregates are fed onto a collector belt, which feeds the aggregates into a cold elevator that delivers them to the dryer. The mechanical feeder is typically loaded with a front-end loader that picks up the material from the stockpile and loads the feeder bin.

17-3. The aggregates are heated to the desired temperature. Upon leaving the dryer, the combined aggregate passes 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). The fine aggregate is screened into the third bin. An additional screen is provided for separating the coarse aggregate into another component in a four-bin plant. Additional mineral filler, if required, is usually stored and weighed or proportioned into the mix separately. It may be obtained from the plant's dust collector system or 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.

17-4. The aggregates must be fed into the plant uniformly, preferably with a mechanical feeder, to obtain efficient plant operation and to produce a desired mixture. It is necessary that the portion of material taken from each hot bin be selected so that the combined gradation of the material coming from the hot bins be approximately the same as the gradation of the material going into the plant. If the gradations are not approximately the same, the plant will be unbalanced and result in one or more bins running full of material and one or more bins running low on material, leading to an overflow of the full bins.

DRUM MIX PLANTS

17-5. Drum mix plants have not been available for use as long as batch plants. In fact, drum mix plants have only been around since the late 1970s. These plants have the ability to produce mixtures of equal quality to that of a batch plant. However, these plants are much more portable and easier to use. There is no rescreening of the aggregates; hence, the gradation of the aggregate entering the plant should be the same as the gradation of the aggregate entering the plant, except for some breakdown of materials that will generally increase the amount of material passing the No. 200 sieve and some of the larger sieves. It is estimated that more than 90 percent of the new plants being sold in the United States are drum mix type plants.

INITIATING PLANT PRODUCTION

17-6. 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 aggregate feeder. The gradation of aggregate supplied by the plant may not precisely reproduce the desired gradation. The gradation of the plant-produced aggregates approximates the one used in design, within reasonable tolerances, if initial sampling were performed properly and if the plant were 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

17-7. A sieve analysis is made on material recovered from the asphalt mixture and material fed into the plant prior to mixing with AC. Aggregate samples that are recovered from the hot-mix asphalt (HMA) mixtures for sieve analyses are never obtained at plant start-up. One should always wait for a couple of trucks to load out before testing begins since there are typically some minor issues with the first mix

produced at start up. Final adjustments to the mixture are based on these sieve analyses and other testing that is performed.

MIX REDESIGN

17-8. Sometimes, significant changes have to be made in the mixture design to produce acceptable properties. 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 design. This allows the determination of optimum asphalt content and ensures that a satisfactory mix is designed and produced by the plant. Sufficient additional tests should be performed to establish optimum asphalt content requirements and to ensure that the mix meets applicable criteria.

PLANT PRODUCTION CONTROL

17-9. When production begins, obtain samples from one of the first few truckloads of asphalt mixture, but typically not the first two trucks. Prepare three or four specimens from this sample, and compact and test them according to standard procedures described previously. On the first day of construction, the operation should be stopped until the test results are completed and adjustments in the mix are made as needed. After the mix has been verified to meet the requirements, additional work can be continued and testing done during production. When samples are taken, conduct the tests as rapidly as possible so that the results are available prior to production of too much additional asphalt mixture. Be careful in making adjustments since there is always variability in testing, and this normal variability may make it appear that the test results are slightly different than desired when this is actually a result of normal testing variability and not material or mixture issues. It is always good to look at the last few tests that were conducted when making changes in the mixture components.

17-10. Probable causes of paving-mixture deficiencies for batch and drum mix plants are shown in figure 17-3, page 17-4. These deficiencies are observed at the plant. Other imperfections and their causes that may be encountered in placing the mix in the pavement are shown in figure 17-4, page 17-4.

MARSHALL TEST CONTROL

17-11. The flow value of the individual test specimens should never exceed the specification limits and should not vary more than two points between specimens. If a polymer-modified asphalt is used, the flow test will not provide reasonable values so, in this case the flow is not part of the criteria. The average value of the other test properties for the 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 pounds per cubic foot.
- Marshall stability, ± 10 percent.
- Voids total mix, ± 1 percent.

Note. No limit is placed on voids filled since control is ensured by the limitations on the voids total mix.

PLANT CONTROL EXECUTION

17-12. 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 do not usually 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.

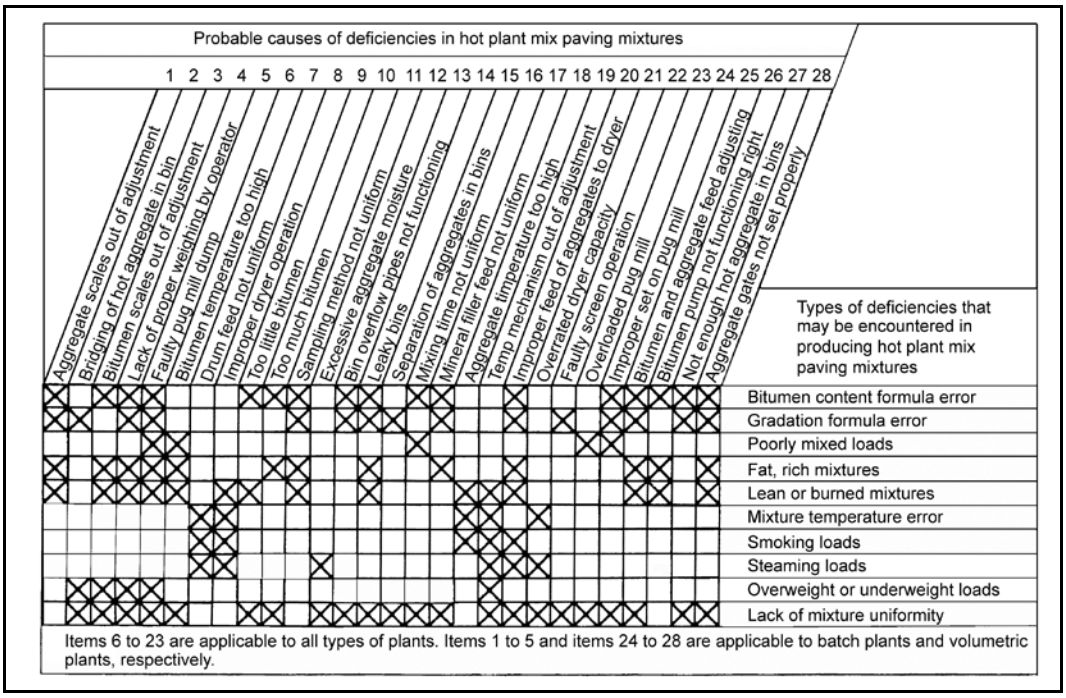


Figure 17-3. Probable causes of paving-mixture deficiencies detected at plant

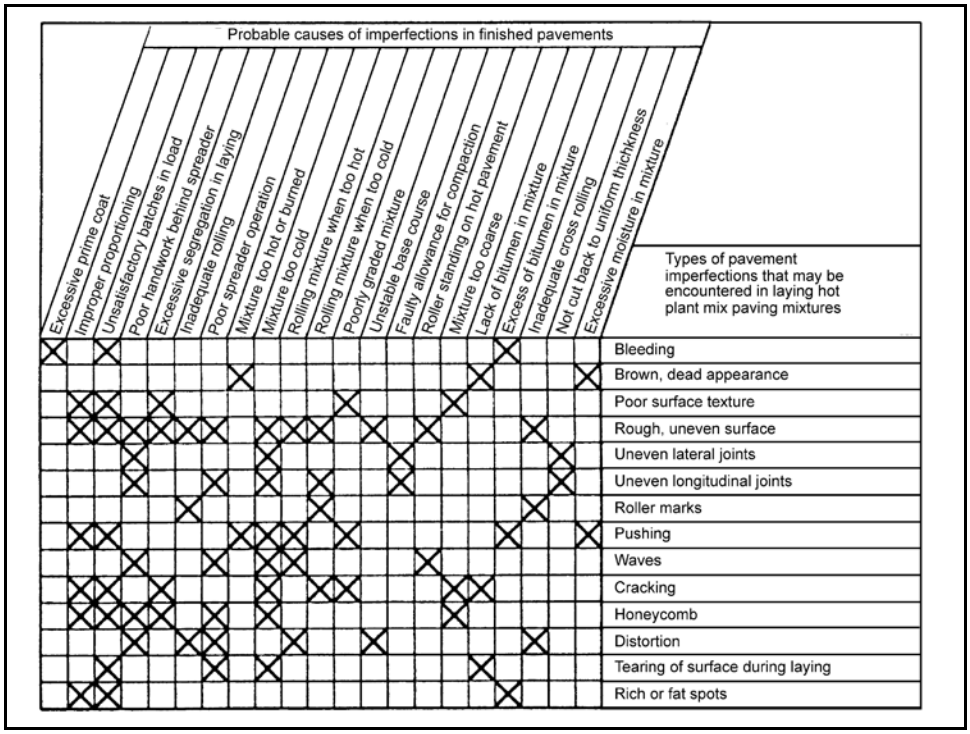


Figure 17-4. Probable causes of imperfections encountered in laying pavement

17-13. 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 additional samples for testing. The amount of sampling and testing required depends on the specifications for the project. If there is no guidance available

on the amount of testing required, then two sets of tests should be conducted per day of operation. The testing should include, as a minimum, sampling the asphalt mixture from a loaded truck; compacting samples in the laboratory; and measuring the air voids, stability, and flow. Additional testing includes asphalt content measurement and gradation measurement.

17-14. It is very important that the mixture be controlled during production so that the air voids in laboratory-compacted samples is approximately 4 percent.

ASPHALT CONTENT DETERMINATION

17-15. The procedure for controlling plant production (based on the test properties obtained using Marshall or Superpave procedures) may be supplemented by the determination of asphalt content using a centrifugal extraction apparatus or the ignition test. 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 or ignited, 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.

17-16. The ignition test is much more user-friendly since it does not require the use of solvents that are difficult to obtain, store, and discard. Both tests, if correctly conducted, will provide a reasonably accurate measure of the asphalt content and allow for the gradation of the aggregate to be determined. Perform calculations on DD Form 1793 *(Determination of Asphalt Content [Dullin - Rotarex Extractor])* (figure 17-5, page 17-6) for the solvent extraction test.

17-17. The clean aggregate recovered from the solvent extraction test or ignition test may be subjected to sieve analysis. When using the solvent extraction test, 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.

DENSITY TESTS

17-18. 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. Take the sample for testing from a random location on any portion of the bituminous pavement.

17-19. A coring machine or concrete saw may be used for cutting out 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 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. Using a coring machine is the preferred method most often used for obtaining samples for density measurement.

17-20. 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. It usually requires that a sharp object and hammer be used to remove the top layer. The two layers can be separated by sawing. Determine the bulk specific gravity for each specimen, and use the following formula to calculate the specimen's density:

$$D = G_m x\ 62.4$$

where—
D = density of specimen (in pounds per cubic foot)
G_m = bulk specific gravity of specimen
62.4 = density of water (in pounds per cubic foot)

DETERMINATION OF ASPHALT CONTENT
(Dulin - Rotarex Extractor)

PROJECT: HIGHWAY NO. 206 — DATE: 2 APR 2010

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	$(\frac{10}{11} \times 14)$	g	1.5	
16.	Total amount of aggregate in sample	(6-15)	g	189.0	
17.	Percentage asphalt	$(\frac{3-16}{6} \times 100)$	%	5.5	

REMARKS

SAMPLE 78

SPECIFICATION 55% CHECK, MIX OK

TECHNICIAN: SP4 Cunningham — COMPUTED BY: F. Dawnis — CHECKED BY: S. Mayo

DD Form 1793, FEB 71

SAMPLE

Legend:

Apr	April
DD	Department of Defense
Feb	February
No.	number
SP4	specialist four

Figure 17-5. Sample DD Form 1793

17-21. Compare the calculated density of the bituminous pavement with the densities obtained during the laboratory compaction of samples of the same mixture. Normally, the field density must be at least 95 to 98 percent of the laboratory-compacted density. 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 compaction procedures so that satisfactory density is obtained. When constant difficulty is experienced in meeting the specified density, check the capability of the rollers and the acceptability of the rolling pattern.

The most common method used to specify density requirements is to specify it in terms of theoretical maximum density (TMD) (ASTM D2041). The typical requirements when using this requirement is approximately 4 percent lower than that required for the percent of laboratory density. A typical requirement is 91 to 94 percent of TMD.

EXPEDITING THE DESIGN

17-22. When military expediency demands it, the preliminary laboratory mix design can be eliminated and the mix 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). The most important requirement to meet is to have 4 percent air voids in the laboratory compacted mixture.

17-23. The manufacture of the paving mix can then be initiated. If the available aggregate must be used, regardless of whether it produces a mix that meets all of the design criteria at OAC, the asphalt content should be selected to give a mix that meets the criteria for stability, flow, and percent of voids in the total mix.

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PART THREE

Concrete

Part Three discusses the testing of both fresh and hardened concrete and the materials from which it is made. Fresh concrete properties are performed to ensure that the concrete mixture proportions meet design requirements before it is placed. Hardened properties tests are performed to ensure that the concrete meets basic design performance criteria. Specifications are developed around results of standard test methods; therefore, it is important to comply with the details of these test methods which are based on ASTM standards. These methods may not necessarily represent the placing environment or conditions. Information on contingency field testing and troubleshooting are also included. Further information on concrete types, components, design, and uses in military construction can be found in TM 3-34.44/MCRP 3-17.7D.

Chapter 18

Description, Terminology, Materials, and Concepts

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 of the construction. When first mixed, fresh concrete varies from thick fluid to a putty-like material, depending on the mixture proportions. The mixture can then be placed in a form or extruded (usually in the case of pavements), where it is allowed to harden until it reaches the required strength for service loads. The finished and cured concrete is a hard, stone-like material. It is used for pavements, foundations, dams, retaining walls, bridges, and buildings of all types.

TERMS

18-1. The following are some useful terms, some of which are commonly misunderstood:

- **Air.** In the context of concrete, air refers to the air-filled voids that exist in concrete. Air is classified as two types: entrapped air and entrained air. Field tests indicate total air, which includes both entrained and entrapped air.
  - ***Entrapped air.*** Entrapped air refers to air-filled voids that occur unintentionally, but as a natural part of the mixing, transporting, placing, and consolidation of concrete. Values of 1 to 2 percent are common.
  - ***Entrained air.*** Entrained air refers to air-filled voids that are intentionally introduced in fresh concrete by using air-entraining admixtures. Properly entrained air voids range in diameter from 0.01 to 1.00 millimeter. Whether concrete is air entrained or not is difficult to determine without magnification.

- **Cement.** Cement is one of the components of concrete. When it reacts with water, it functions to bind the other components of concrete together. Concrete structures are often incorrectly identified as made of cement (a cement runway).
- **Concrete.** Concrete is a mixture of cement, sand, coarse aggregate, and water.
- **Grout.** Grout is a material that is used to fill spaces. Grouts normally have a high level of fluidity so that they can be pumped into spaces. They may be cement paste, mortar (commonly called sanded grout), or concrete. Grouts are commonly used in repair applications and in some new construction where there are voids or gaps that need to be filled.
- **Hydraulic cement.** Hydraulic cement is any cement whose chemical basis for cementing depends on reacting with water.
- **Mortar.** Mortar is a mixture of cement, sand, and water. Mortars are most commonly used in masonry construction.
- **Paste.** Paste is a mixture of cement and water. Cement pastes are sometimes used for repair and grouting of very small cracks and features and is the binder phase of concretes and mortars.

MATERIALS

18-2. Materials used in making concrete are discussed in the following paragraphs.

CEMENT

18-3. In the United States, cements are sold for general construction under three specifications: ASTM C150, ASTM C595, and ASTM C1157. ASTM C150 covers portland cements; ASTM C595 covers blended cements; and ASTM C1157 covers a series of hydraulic cements that may have a wide variety of chemical compositions that are significantly different from portland cements. All three specifications require similar performance properties. Some cements comply with more than one of these specifications, so bags will sometimes be marked as complying with more than one specification.

18-4. Portland cements are cements that are made primarily of calcium silicates that conform to a relatively narrow set of requirements on composition and performance. ASTM C150 recognizes the following types:

- **Type I.** Type I is for general-purpose use. This is the predominant cement used in general construction.
- **Type II.** Type II is for application where moderate amounts of sulfate are found in soil or groundwater, and for use when lower heat evolution than typical of Type I cement is needed.
- **Type III.** Type III is for use when more rapid strength gain than offered by Type I cement is needed.
- **Type IV.** Type IV is for use when very low heat evolution is required (mostly in structures with thick sections).
- **Type V.** Type V is for use when high levels of sulfate are present in soils and groundwater.

18-5. Optional limits for early workability, air entraining, and chemical compatibility with certain aggregates can be invoked for dealing with special conditions.

18-6. Blended cements consist principally of portland cement, but will have significant amounts of supplemental cementing materials (SCM) blended in with them. Typically, coal fly ash and slag are the most common materials used. The amounts of these blended SCMs is large enough that the material is no longer called a portland cement. Specification ASTM C595 recognizes two basic types based on whether the cement uses pozzolan (coal fly ash, clay, or volcanic ash) as the SCM or uses slag from steel manufacture as the SCM. There are a number of options that deal with the same performance properties that the 5 types of portland cement described in ASTM C150 are designed to address.

18-7. ASTM C1157 on generic hydraulic cements recognizes six types of hydraulic cements similar problems to those covered by ASTM C150:

- Type GU (general use).
- Type MS (moderate sulfate exposures).

- Type HS (high sulfate exposures).
- Type MH (moderate heat evolution).
- Type LH (low heat evolution).
- Type HE (high early strengths).

18-8. Many nationalities have standard specifications that cover the cements available in their locality. The names and type designations will usually differ considerably from U.S. specifications, but performance properties are designed to deal with the same issues that are behind the various requirements of U.S. cements. Specific requirements are typically designed around strength, time of setting, length of workability, amount of heat evolved, and resistance to certain chemical degradation problems (exposure to sulfates, chemically unstable aggregates).

18-9. Testing hydraulic cements for specification compliance requires more in the way of laboratory capability than usually exists in field construction laboratories. Hydraulic cements are normally supplied with a mill certificate that describes the relevant properties. Mill certificates often represent typical properties and do not always accurately represent the cement in hand. Under critical construction conditions, these properties may need to be verified by an independent laboratory that is competent to execute the relevant tests.

SUPPLEMENTAL CEMENTING MATERIALS

18-10. SCM is a class of materials that are not usually used alone as cements, but rather as a partial replacement for portland cement. Most only SCM develop cementing properties in the chemical environment of portland cement. When blended with cement at a cement manufacturing facility, these are called blended cements and covered by ASTM C595. Blending often occurs during the batching of concrete, which allows the user to control the proportions.

18-11. Typical materials and their associated specification are fly ash from coal combustion in coal-fired electric power plants (ASTM C618), blast furnace slag from steel manufacture (ASTM C989), volcanic ash, some clays and shales (ASTM C618), and silica residues from the manufacture of silicon metal, called silica fume (ASTM C1240). Other national specifications recognize these same materials, but names and designations will differ from U.S. specifications.

18-12. The testing of SCMs requires a capability that is not usually available in a construction laboratory. Manufacturer's certificates can normally be secured for these materials, but they usually represent typical properties rather than specific properties of the material on hand.

AGGREGATES

18-13. Aggregates are the sand (fine aggregate) and stone (coarse aggregate) components of concrete and mortar. Both are specified in ASTM C33. Aggregates may be derived from natural deposits or by crushing quarried rock. Fine aggregate normally contains particle sizes ranging from about 0.2 to 5.0 millimeters in diameter. Coarse aggregate particle diameters range from about 1 to 75 millimeters for paving, and up to 150 millimeters for other applications. In general, the maximum size used depends on the dimensions of the structure and on the spaces between reinforcing steel. General guidance is that the smallest dimension of a structure and the minimum openings in reinforcing steel should be at least three times larger than the maximum aggregate size.

18-14. Several aggregate properties have significant impact on concrete. These include absorption of mixing water, particle shape, particle strength and durability to freezing, and absence of any tendency to chemically react with cement paste in a damaging way (alkali aggregate reaction). Aggregate sources are normally evaluated for these properties before construction begins and the source approved for use.

18-15. Aggregate grading (distribution of particle sizes) has major effects on workability and the amount of cement and water required to produce good concrete, consequently, is an important test regularly executed during construction. Other properties that must be frequently verified are moisture content, absorption, density, and the amount of extremely fine material (dirt).

WATER FOR MIXING AND CURING

18-16. Properties of water that is used for mixing and curing concrete can be critical. In general, however, water that is suitable for drinking should work well for making and curing concrete. Most natural water sources that are not used for drinking water can also be used without a problem. Even seawater can be used to make concrete as long as there is no steel reinforcement that can be corroded by the chlorides present in seawater. When problems develop from a particular water source, they are usually related to chemical contaminations. Specifically problematic are significant contaminations with organic materials and salts. ASTM C1602 is the standard specification for mixing water. This specification basically uses strength development and time of setting as indicators of major water quality problems. There are also limits on certain chemical properties that may have implications for long-term durability and staining.

18-17. Full ASTM C1602 testing of water is beyond the capability of most construction laboratories; however, the strength and time of setting requirements for mixing water can usually be determined in field construction laboratories. Chemical properties testing must be done in a specialized laboratory.

CHEMICAL ADMIXTURES

18-18. Chemical admixtures are used to generate properties in concrete that usually cannot be achieved without them. Chemical admixtures are often an optional ingredient for ordinary concrete applications. Through much of the 20th century, most concrete was made without chemical admixtures. One exception is air entraining admixture for use in concrete that is exposed to cycles of freezing and thawing when wet. ASTM C260 and ASTM C494 are the common specifications for admixtures used in the United States. Commonly used chemical admixtures are:

- **Water-reducing admixtures (WRAs).** WRAs are normally used to achieve better workability without increasing the mixing water amount or allowing mixing water reduction to obtain higher strengths without loss of workability. Materials that allow major reductions of mixing water are called high-range WRAs (or superplastizers).
- **Accelerators.** These materials cause concrete to set more quickly and to achieve higher early strengths than normal. These are usually used to compensate for the slow setting times and strength gain caused by cold weather.
- **Retarders.** These materials cause a retardation of setting time. These are usually used to compensate for the accelerated setting time caused by hot weather. Sugars are common and strong retarders. Some concrete operations use table sugar as a retarder. This is effective, but it is easy to overdose the concrete and to retard the setting time for many days.
- **Air-entraining admixtures (AEAs).** These materials cause a system of fine (less than 1 millimeter) air bubbles to form in the cement paste fraction of concrete. The primary purpose is to protect against the effects of freezing and thawing cycles of concrete while in a water-saturated condition. Concretes are sometimes air-entrained to enhance workability.

CONCEPTS

18-19. The following paragraphs discuss concepts involved in concrete.

AIR ENTRAINING

18-20. Air entraining is the development of a system of very fine air bubbles in the cement paste fraction of concrete and mortar. Bubble sizes range from about 0.01 to 1.00 millimeter. All but the largest sizes are not easily seen without the use of magnification. The primary purpose is to protect concrete from cycles of freezing and thawing when saturated with water, so air entraining is usually a requirement in environments in which the concrete will experience these conditions. The practice was first developed in the early 1940s, so structures in cold and wet environments constructed earlier than this often suffer significant deterioration.

ALKALI AGGREGATE REACTION

18-21. AAR refers to a process in which the pore fluid of concrete reacts chemically with certain aggregates resulting in the formation of an expansive material that can destroy concrete. The most common form of AAR involves silica-containing aggregates and is called alkali silica reaction (ASR). Most aggregates are not reactive; but for those that are, the visible evidence of the damage does not appear for many years. Reactive aggregates can only be determined through detailed analysis and testing by a laboratory that is specifically qualified for that purpose.

BLEEDING

18-22. Bleeding is the accumulation of mixing water on the surface of concrete as a result of the solid particles settling. Bleeding starts immediately after the end of consolidation and ends no later than the time of initial setting. The maximum rate of bleeding typically occurs in the first hour after finishing. A small amount of bleeding is normal and sometimes useful for buffering concrete against early age drying. Excessive bleeding (more than a few millimeters) often indicates a problem with the proportions of materials in the concrete. Excessive bleeding is often recognized by the presence of small mounds of fine material that is brought to the surface along bleed channels. This can result in a weak surface.

CRACKING (PLASTIC SHRINKAGE)

18-23. Plastic shrinkage cracking is a form of drying shrinkage cracking that sometimes occurs while concrete is in the plastic state (before the time of initial setting). This is likely to occur when the rate of evaporation of mixing water is higher than the rate at which the bleed water forms. This is typically caused by windy conditions and/or high concrete temperatures. Air temperature and relative humidity also contribute but are of less importance than wind speed and concrete temperature. The exposed surfaces of fresh concrete are particularly prone to this because water is easily evaporated. Concrete is more resistant to loss of water to evaporation after time of setting. Plastic shrinkage cracks vary in depth from a few millimeters to full depth, depending on the conditions.

CRACKING (DRYING SHRINKAGE)

18-24. After the time of setting, concrete may continue to dry out and shrink, although at a slower rate than before setting. This shrinkage may eventually cause cracking. It is important to cure the concrete for several days after setting to allow for most of the water to be consumed by a reaction with the cement so that it will not be available to evaporate. Also, the concrete will be stronger and better able to withstand the shrinkage without cracking. Long-term drying shrinkage is very difficult to completely prevent.

CURING

18-25. Curing is the maintenance of moisture and temperature conditions necessary for concrete to develop physical properties. Concrete gains strength because of the chemical reaction between cement and water. This reaction stops if the concrete dries out. Project specifications will normally specify a time period (usually a few days) during which concrete must be kept wet by the direct application of water (misting, wet burlap, wet sand or soil) or by preventing the evaporation of the water already in the concrete (mixing water). The latter is accomplished by using waterproof sheeting or by applying a suitable paint-like membrane called curing compound.

18-26. Concrete only gains strength when reasonably warm. Strength gain effectively stops with temperatures approach the freezing point of water. If the concrete freezes during the first 24 hours, irreversible damage usually occurs. Temperatures between 10° and 40°C are ideal for proper strength gain rates. Strength gain is faster at higher temperatures. Temperatures of fresh concrete are normally limited to a maximum of about 35°C because there is a tendency for premature loss of workability and setting at higher temperatures and because of plastic shrinkage cracking issues.

DURABILITY

18-27. Durability refers to any number of conditions that cause concrete to deteriorate. In the absence of aggressive chemical or overloading physical conditions, concrete will slowly change chemical composition due to the interaction with atmospheric conditions and groundwater, but will last for centuries. Conditions that cause deterioration include physical overload, abrasion, cycles of freezing and thawing while saturated with water, and chemical reactions within the concrete that cause expansion (cracking) or cause decomposition of cement and resultant loss of strength.

SHRINKAGE

18-28. Shrinkage is a normal phenomenon in concrete due to large temperature changes, drying, and chemical reactions with atmospheric carbon dioxide. Excessive shrinkage will cause cracks to form. Shrinkage is minimized if concrete is properly proportioned and cured and if standard design and construction practices are followed. Excessive shrinkage is a common field problem.

STRENGTH

18-29. Concrete strength starts to develop after the time of initial setting and continues until all of the cement in the concrete has reacted with the water or until the hydration consumes all of the mixing water. Low temperatures will also essentially stop strength gain. Most modern concretes reach a large fraction of their ultimate strength between 28 and 90 days (depending on materials, mixture proportions, and temperature) but may continue to slowly gain strength for years.

TEMPERATURE EFFECTS

18-30. Temperature has the following major effects on concrete:

- **Rate of strength gain and the time of setting.** Strength gain is severely limited when temperatures drop below 5°C for a long time, although no permanent damage occurs and normal strength ultimately results. Setting time is also significantly extended at such low temperatures. Significant damage occurs to concrete if it freezes during the first 24 hours.
- **Damage from freezing.** If concrete is not properly air-entrained, even strong concrete will suffer damage from repeated freezing and thawing cycles when wet.
- **High temperature.** The major problem with high placing temperatures is that working time can become severely restricted, and it may contribute to excessive evaporation of mixing water and subsequent shrinkage cracking. Very high temperatures in newly hardened concrete can cause cracking if the concrete cools rapidly. This is particularly a problem when wet concrete is exposed to extreme drying conditions, resulting in the rapid cooling of the concrete surface.

TIME OF SETTING

18-31. The time of setting is determined as the point at which concrete ceases to be a fluid and starts to take on properties of a solid. Two setting conditions are defined: time of initial setting and time of final setting. These are not sharply distinguishable events, but the time of initial setting is usually taken as the time at which concrete will start to support some load, while time of final setting is the time at which concrete can tolerate light service loads (foot traffic) without damage. The final finishing and application of final curing measures (plastic sheeting, wet burlap, water, or curing compound) can be applied at about the time of initial setting without damaging the surface of the concrete.

WATER-CEMENT RATIO

18-32. The water-cement ratio is the ratio of the mass of water to the combined mass of cement and SCMs. The water-cement ratio is a critical parameter affecting workability, setting time, and strength development of concrete. Workability and setting time decrease with the decreasing water-cement ratio; strengths increase with the decreasing water-cement ratio. Unless water-reducing admixtures are used, a lower practical limit on the water-cement ratio is 0.45 to 0.50 for most concrete mixtures.

WORKABILITY

18-33. The workability of concrete is a vague term that refers to the ease with which concrete is distributed during placement and consolidated. The property is loosely related to the results of the slump test, but concretes of equal slump will often show different workability properties. The serious loss of workability typically occurs when the concrete is more than an hour or so old or when some unusual event happens with the hydration of the cement. This is normally indicated by a rapid loss in slump.

UNITS OF MEASURE AND CONVERSIONS

18-34. The units of measure used collecting data and for reporting results can be somewhat confusing, but it is important to follow a consistent practice. Two systems of measure are primarily used. One is the System Internationale (SI), often called metric in common practice. The other is the English Inch-Pound System, sometimes called non-SI.

18-35. Much of the world uses only SI-based unit systems for the measurement and reporting of test results. For much of its history, the United States has non-SI Systems. Currently there is a transition in process from the English Inch-Pound System to the SI System, and both systems of units will likely be encountered by the operator for some years to come. A common problem occurs when specifications are in SI units and equipment readout is in non-SI units. Following are procedures for working with units of measure in data collection and in reporting.

DATA COLLECTION

18-36. As a general practice, data should be recorded in the units of measure of the instrument from which the data are obtained. The test methods will instruct which SI units or non-SI units to use. Data will then be converted to units of measure that are consistent with the units of measure required for the report.

CONVERSIONS AND REPORTING

18-37. When data to be reported are simply the data recorded from measurement and the units are appropriate, then no conversions are required. If the data are not in the correct system of units, a conversion table or conversion factor is used to convert them into the required reporting format. For example, slump might be measured in inches, but required to be reported in millimeters. In this example, the data are simply converted from inches to millimeters by multiplying the inches by 25.4 to obtain the millimeters per inch.

18-38. Some reported properties are determined by a calculation using several measured properties. If all of the measurements are recorded in the same system of measurement, calculate the property in those units as instructed in the test method and then convert the result to the required reporting system if it is different from the units used during data collection. For example, concrete strength is calculated by dividing the load required to crush the specimen by the surface area of the end of the specimen. If these are both in the English system (pound and inches), calculate the strength as pounds per square inch. If the reporting requirement is SI, convert the measurements to SI units of megapascals using an conversion factor of 0.00689 megapascals per pounds per square inch.

18-39. A more complicated example occurs when individual measurements used in a calculation are in different (mixed set) units. In this case, before doing any calculations, convert the measurements that are not in the proper system of units for reporting into those units and then calculate the property and report. For example, if the load is in pounds, but the surface area of the specimen is in square millimeters and the reporting convention is the English system (pounds per square inch), convert square millimeters to square inches, and then execute the calculation and report.

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Chapter 19

Field Aggregate Testing

Aggregates used in making concrete are a blend of fine and coarse material, usually sand with natural gravel or crushed rock. The aggregates serve as an inert filler to provide the bulk material used in concrete. Well-graded aggregates contain particles of all sizes, from the largest permitted by the dimension of the structural member to be formed to the finer sand particles. The smaller sand particles fill the spaces between the larger particles, thus providing a dense material that requires a minimum amount of cement paste as a binder. The aggregate materials must be clean and hard, resist weathering, and have no unfavorable reaction with the cement. Aggregates must provide maximum strength and durability in a concrete mixture. Fineness, coarseness, and aggregate gradation are factors considered when deriving the correct concrete mixture for a specific construction purpose. Relative density (specific gravity), absorption, and moisture content also affect the aggregate's ability to bind well with cement and water in a concrete mixture. The components of the final mixture (cement, water, and aggregate) must bond adequately for structural strength and must resist weathering and applied loads. Correct aggregate selection also minimizes the project's overall cost. An engineering analysis determines the aggregate best suited for a particular purpose. Laboratory testing supports the best selection analysis. Field testing includes executing a few of the laboratory tests used to initially qualify the aggregate source. The purpose of this testing is to verify that these critical properties do not vary excessively during construction.

STOCKPILE SAMPLING

19-1. It is difficult to ensure that unbiased samples are obtained from stockpiles. This is due to the segregation that often occurs when materials are stockpiled, allowing coarser particles to roll down to the outside base of the stockpile. 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 stockpile composed of materials drawn from several increments.

19-2. When power equipment is not available, take samples from at least three increments, from the top third, the middle and the bottom third of the stockpile. Shove a board vertically into the stockpile just above the sampling point to prevent further segregation. When sampling fine-aggregate stockpiles, remove the outer surface before taking the sample. The best method of sampling fine-aggregate stockpiles is to use a tube or pipe (1 1/2 to 3 inches in diameter) inserted into the stockpile at five random locations to form the sample.

19-3. Pit deposits are sources of natural sand and gravel. Sample them by channeling exposed faces or drilling test holes in the pits if exposed faces are not available. Ensure that the samples include only materials that are below the overburden or disturbed material.

GRADATION DETERMINATION

19-4. The gradation of aggregate refers to the distribution of particles of aggregate among various sizes. A set of sieves is used in a mechanical shaker that shakes the sample of aggregate for a prescribed period of time allowing each particle an opportunity to pass through any number of sieves until that particle is retained on the sieve slightly smaller than the particle itself. The mass of each material retained on each

sieve is determined and calculated as a percentage of the initial sample mass. Aggregates having a smooth grading curve, no deficiency, and excess of any one particle size usually produce mixtures with fewer voids between particles. A higher proportion of coarse aggregate leaves voids that require more cement paste to fill. This affects the economy of the mixture. Too much fine aggregate increases the amount of surface area that must be coated with cement paste. This may weaken the concrete and is also uneconomical. Proper gradation results in—

- A dense mass of concrete with a minimum of voids.
- An economical mixture.
- A strong structure.

19-5. Optimum strength, water tightness, and durability in the hardened concrete require careful control of aggregate gradation.

19-6. A gradation or sieve analysis conducted according to ASTM C136 indicates whether an aggregate particle size distribution meets the project's requirements. Dense volumes of aggregate can result in a concrete that is denser, stronger, and more economical, watertight, and resistant.

MATERIAL FINER THAN 75-MICROMETER SIEVE

19-7. Within the gradation, very fine material becomes an important factor in the development of a concrete mixture. Extremely fine material (clay, silt, dust, loam) occurring in most aggregates requires relatively large increases in the amount of mixing water because of the increased surface area associated with finer materials. Fine particles also tend to work to the surface of concrete and cause cracking upon drying, referred to as drying shrinkage cracks. If fine particles adhere to larger aggregate particles, they 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 C117 gives the standard test method for fine materials. Some fine material, not exceeding 3 percent of the total aggregate mass, is generally not harmful to concrete; and a small amount of such fines may improve the workability of the mixture.

FINENESS MODULUS

19-8. After conducting the gradation, the particle size distribution of an aggregate can be monitored with a single value—the fineness modulus. The fineness modulus is an empirical factor that gives a relative measure of the proportional particle size distribution of an aggregate, fine, coarse, or a combination of fine and coarse particles. The fineness modulus represents the gradation of a particular sample by converting the cumulative percentage retained from the gradation into this empirical factor. Following the calculations for a gradation on an aggregate sample, the fineness modulus is calculated based on a series of sieves, beginning with the Nos. 100, 50, 30, 16, 8, 4, 3/8 inch, 3/4-inch, 1 1/2 inch, 3 inch, and ending as high as the 6-inch sieve size. The series of sieves used in the calculations depends on the nominal maximum size aggregate in the sample, fine aggregate generally includes the 3/8-inch sieve sizes, and therefore the calculations would only those percentages retained up to the 3/8-inch particles. An aggregate for a large mass concrete project using 6-inch nominal maximum size aggregate would include each of the sieves up to the 6-inch sieve. The fineness modulus values range from 2.3 for fine aggregate to 7.5 for very large, coarse aggregate.

SURFACE MOISTURE

19-9. The specific gravity or relative density refers to the ratio of the mass of the aggregate to the mass of equal volume of distilled water. The absorption refers to the percentage of water absorbed into the pores of the aggregate, but does not involve the water on the outer surface of the aggregate. The outer surface water is referred to as the surface moisture. Tests for relative density, absorption, and surface moisture and make the necessary calculations to design the concrete mixture. For aggregates used in portland cement concrete, determine the relative density 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, the moisture condition of an aggregate can be defined as "not absorbing water from" or "contributing water to" the concrete mixture. Relative density 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. Adjustments to the mixing water are made based on the percentage of surface water calculated from the percentage of the total moisture content of an aggregate, minus the percentage associated with the absorption for that aggregate.

ORGANIC IMPURITIES, LOS ANGELES ABRASION, AND SULFATE SOUNDNESS

19-10. Natural fine aggregates may occasionally possess some unwanted materials that affect the strength of the mortar to bond the aggregate particles together. Organic impurities found in natural fine aggregate must be held to a minimum to satisfactorily provide sufficient strength in the mortar. Natural fine aggregate recovered from pits and from dredging operations often contain organic matter whose presence and levels can be determined by using ASTM C40. Sodium hydroxide reacts with organic matter and changes color with certain levels of organic materials present. This test uses a sodium hydroxide solution applied to a sample of the natural fine aggregate and allowed to stand for 24 hours as the reaction, if any, takes place. The color of the liquid above the settled sample is compared to a standard color solution or with a color chart containing five standard colors. Dark liquids indicate the presence of organic matter; a darker liquid indicates higher levels of organic matter. Samples of natural fine aggregates may be further tested for compressive strength loss in the mortar due to the amount of organic matter in the aggregate.

19-11. Coarse aggregate must be hard and able to resist fracturing, wear, and abrasion during normal handling operations and in the hardened concrete. The Los Angeles abrasion resistance test evaluates the relative strength and abrasion resistance of coarse aggregate for use in concrete that may be subjected to high abrasion (high traffic areas, areas of erosion). ASTM C131 and ASTM C535 are similar tests with exceptions to sample size, number of steel ball bearings, and number of drum rotations. Both tests uses a Los Angeles abrasion test machine, which is a rotating steel drum with a fixed, steel shelf that picks up the aggregate sample and steel ball bearings and drops them onto the rolling sample below. The abrading of the rotating steel drum and impacting and crushing from the falling sample and steel ball bearings determines the aggregate's resistance to degradation.

19-12. The durability of the aggregate to cycles of freezing and thawing when wet affects the durability of the concrete. The sulfate soundness test, which evaluates aggregates for this susceptibility, uses a saturated sulfate salt solution that is absorbed into the pores of the aggregate and then heated to recrystallize the salt, which expands internally, seeking out the weak and less durable aggregate. ASTM C88 describes the procedures, sample sizes, sodium or magnesium sulfate salt solutions, heating stages, number of cycles, washing process, and calculations.

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Chapter 20

Field Concrete Testing

A laboratory provides an ideal setting for generating test results used in the field. However, when tests are conducted partially or entirely in the field, conditions may not be ideal and can have significant effects on test results. The vibration of samples from nearby heavy equipment and extremes in temperature are two examples of such conditions. Proactive measure must be taken to protect samples from the natural environmental effects placed on them.

FRESH CONCRETE TESTS

20-1. After concrete is mixed, a slump test and an air content test are performed and compared to project requirements. These tests are normally used for quality control.

SAMPLING

20-2. 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 in the pan. For ready-mixed concrete, take samples from the transporting vehicle while it is discharging the concrete (figure 20-1).

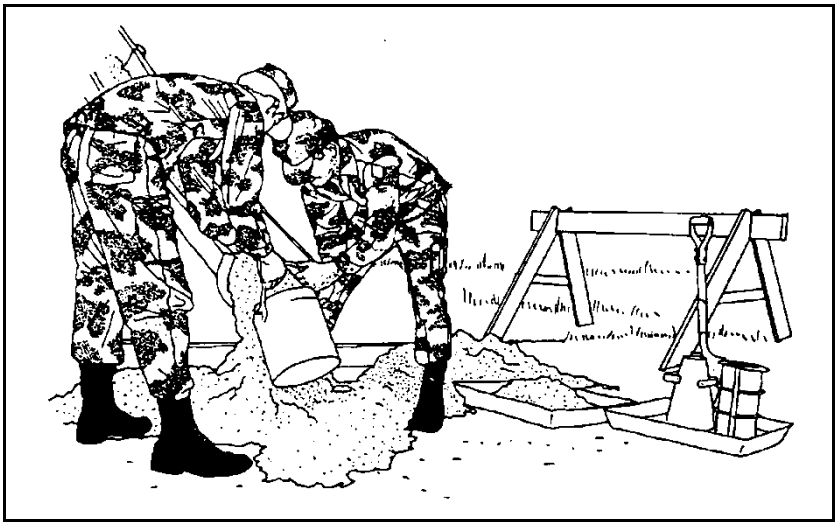


Figure 20-1. Sampling concrete from a truck mixer

20-3. 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 type of truck, the time, and the location of the concrete placement for future reference. For paving concrete, take samples from the batch immediately after it is deposited on to the subgrade. Take at least five samples from different portions of the pile, and mix these samples thoroughly to form the test specimens.

TEMPERATURE

20-4. The temperature of fresh concrete is measured according to ASTM C1064. This property is usually determined to verify that concrete meets the delivery temperature requirements of the project specification.

It may also be used to determine whether unexpected chemical reactions in the cement are occurring that affect early workability properties. These reactions result in temperature increases of 5°C or more during the time between completions of mixing and time of setting. The concrete sample should be protected from high ambient temperatures during such testing.

20-5. This test requires a thermometer capable of reading temperatures of 0°C through 50°C. The thermometer may be of a liquid in glass type, dial type, or digital type. The thermometer should be calibrated at least annually, or when readings are suspect. Calibration is accomplished by measuring the temperature of an ice and water slurry, containing mostly ice. Field measurements of concrete temperature are then corrected by the amount of error noted in this calibration. Liquid in glass thermometers tend to develop errors when the column of liquid develops a gap somewhere in the length of the column. Dial thermometers are particularly noted for developing errors in readings. Deficient dial-gage thermometers will not usually appear to be defective on simple visual inspection.

20-6. The temperature may be determine on a sample of concrete, taken as described above, or directly in the mixer or transport vehicle. Submerge the tip of the thermometer at least 75 millimeters into the concrete and compact the concrete into any void around the thermometer to prevent air from affecting the reading. Allow at least 2 minutes for the temperature to stabilize. Record to the nearest 0.5.

SLUMP TEST

20-7. When the mixture appears to have reached the desired consistency or when directed by project specifications, perform a slump test (ASTM C183). The method of testing covers the procedures to be used in the laboratory and in the field to determine concrete consistency, which is a characteristic of workability. It is not an exact method, but it gives sufficiently accurate results.

20-8. Use the slump test to determine the consistency of a concrete mix by measuring the vertical distance that the concrete settles to the nearest 5 millimeters. Slump testing is a quick way to determine whether the concrete has been batched accurately or whether the concrete may be too old to be used in construction.

20-9. The test is applicable to concretes with a slump over 13 millimeters, but under 225 millimeters. When slump is less than 13 millimeters, the material is no longer plastic and the test is invalidated, except to verify that a specified value has not been met. Slumps over 225 millimeters are more fluid than plastic, and again, the test does not accurately represent the concrete properties. However, most concretes designed for production work are designed to have a slump between 25 and 200 millimeters.

20-10. Contractors usually push the upper end of slump limits because it makes the concrete easier to place. Owners usually prefer slumps in the lower range because they indicate higher strength. Specifications usually cite a slump value as a specific value with a tolerance, for example 100 millimeters ± 25 millimeters.

Equipment

20-11. Use the following items to perform this a slump in a field or simulated field environment:

- A ruler.
- A scoop.
- A trowel.
- A water source.
- A level, flat, smooth surface made of nonabsorptive material.
- A slump cone with a tamping rod.
- Concrete mix datasheet as required in project specifications.

Steps

20-12. Perform the following steps to determine the slump:

- **Step 1.** Moisten the inside of the slump cone; and place it on a rigid, flat, moist, and nonabsorbent surface. Hold it in place during filling by standing on the two foot pieces.

Note. From steps 2 to 10, a total time of no more than 2-1/2 minutes should elapse.

- **Step 2.** Fill the slump cone to one-third of its volume (about 70 millimeters high).
- **Step 3.** Rod the concrete by applying 25 evenly distributed strokes, penetrating the full depth of the first layer in the slump cone.
- **Step 4.** Add a second layer of concrete to the slump cone until two-thirds of its volume is filled (about 150 millimeters).
- **Step 5.** Rod the second layer in the same manner as the first, with the rod just penetrating the underlying layer.
- **Step 6.** Add the third and last layer of concrete, overfilling if possible.
- **Step 7.** 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 8.** Strike off the excess concrete with a screening and rolling motion of the tamping rod so that the cone is completely filled.
- **Step 9.** Remove the slump cone from the concrete.
  - ***Step 9a.*** Place hands on the handles, and press downward.
  - ***Step 9b.*** Step off the footholds.
  - ***Step 9c.*** Raise the cone carefully and quickly in a vertical direction. Raise the cone a distance of 12 inches within 3 to 7 seconds (5 ± 2), using a steady, upward lift with no lateral or twisting motion.
  - ***Step 9d.*** 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 (figure 20-2).
  - ***Step 10a.*** Place the tamping rod along the top of the cone so that it projects over the concrete.
  - ***Step 10b.*** Measure the slump with a ruler, from the bottom of the rod to the top center of the concrete.
  - ***Step 10c.*** Record the slump to the nearest 5 millimeters.

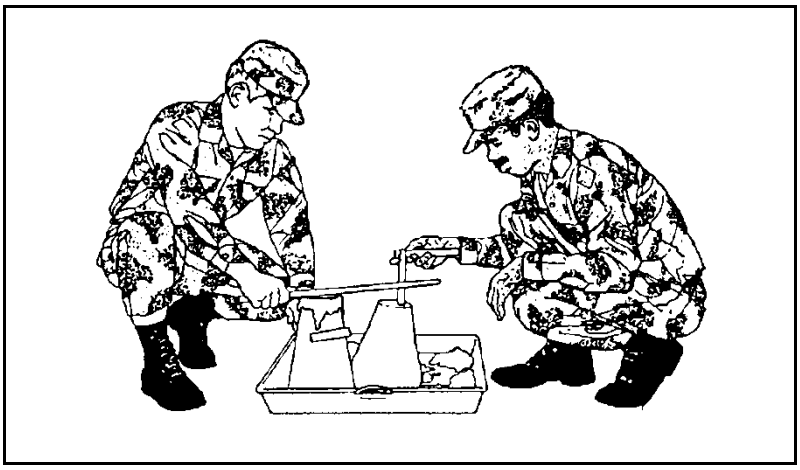


Figure 20-2. Measuring the slump of fresh concrete

Supplementary Test Procedure

20-13. After completing the slump test, 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. Concretes may be designed to have slumps that range from 1 to about 8 inches, depending on the application. When it is not specified, an approximate value can be selected from the list in table 20-1.

Table 20-1. Recommended slumps for various types of construction

Types of Construction	*Slump (in inches)*	
	Maximum[1]	*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
Pumped concrete	8	5

[1] May be increased 1 inch for consolidation by hand methods (rodding and spading).

AIR CONTENT TEST

20-14. ASTM C231 measures total air and is affected by any air voids in the aggregate, which is not useful air for solving the problems for which air entraining is intended. If aggregates have significant air voids, a correction factor must be determined so that the air content measured only reflects the air in the cement paste fraction of the concrete.

20-15. The air measured in fresh concrete is not necessarily the same air content that will exist in the hardened concrete. Air can be lost during transportation and consolidation. It is often specified that air content be measured for acceptance testing purposes at the site of placement.

20-16. ASTM C231 describes two types of air meters. The two meter types are differentiated by the type of measuring hardware on top. Type A meters have both a pressure gage and a transparent standpipe with calibrated markings that allow the user to measure the level of water in the instrument under different pressure loads. Type A meters are rare in the industry today. Type B meters have only a single dial-type pressure meter, marked in units of percent air. The meter may be on a tall standpipe, but it will not have a mechanism for viewing water levels (no window and measuring scale).

20-17. There are many different brands and versions of Type B air meters currently fielded, and replacements of old equipment may not be the same in details of operation. For this reason, it is recommended that the steps outlined in the manufacturer's user manual be followed. They are general steps of a typical determination using a Type B meter, including calibration and determination of the aggregate correction factor.

Equipment

20-18. Use the following items to perform the air content test in a field or simulated field environment:

- An air entrainment meter with 5 percent calibration vessel and instructions for calibration.
- A trowel.
- A tamping rod that is 15 millimeters in diameter and about 600 millimeters long, with a rounded ends or a concrete vibrator.
- A sample of plastic (fresh) concrete.
- Water.

- Oil.
- Rags.
- A pail.
- A kitchen scoop.
- A rubber mallet.

Steps

20-19. Measure the air content according to the following steps.

Step 1. Prepare the Sample

20-20. Secure a sample of fresh concrete that is sufficient to fill the bowl of the apparatus. If the concrete contains aggregate larger than 50 millimeters, sieve the material using a 37.5-millimeter sieve to remove the large aggregate. Do not try to remove the adhering paste from the aggregate in the sieve and add it to the sieved material.

Step 2. Prepare and Fill the Bowl

20-21. Dampen the interior of the measuring bowl; and place the bowl on a flat, level, rigid surface. Place concrete in the bowl and consolidate according to the procedures for rodding or vibrating consolidation. Only use rodding for concretes with a slump greater than 75 millimeters. Rod or vibrate concretes with a slump of 25 to 75 millimeters. Use only vibration for concretes with a slump less than 25 millimeters.

Step 3. Consolidation

20-22. Consolidate by rodding or vibrating.

- **Step 3a. Rodding procedure.** Place concrete in the measuring bowl in three approximately equal layers. The final layer should slightly overfill the container. Consolidate each layer by rodding 25 times over the surface. When rodding the first layer, avoid forcibly striking the bottom of the bowl. When rodding the second and third layers, penetrate the underlying layer by about 25 millimeters. After rodding each layer, tap the sides of the bowl sharply with a rubber mallet to close any voids left by the rod and to remove any air entrapped by the placing process.
- **Step 3b. Vibrating procedure.** Place the concrete in the measuring bowl in two approximately equal layers. The second layer should slightly overfill the container. Consolidate each layer by inserting a vibrator at three locations over the surface. Cease vibrating when the surface of the layer is smooth and large air bubbles cease to come out. Do not vibrate to the point of frothing. Do not let the vibrator rest on the bottom or sides of the bowl during vibration.

Step 4. Strike Off

20-23. Strike off the excess concrete with a rigid straight edge (strike-off bar), using a sawing motion. Ideally, there will be about 3 millimeters of overfill material to strike off.

Step 5. Prepare for Test

20-24. Clean the rim of the bowl and lid so that an airtight seal results when the lid is put on. Put the lid on and fasten it down. Close the valve between the air chamber and the bowl, and open the two petcocks on the top of the lid. Using a rubber syringe or bulb, inject water through one petcock until water flows out of the other petcock. Jar the apparatus, and continue injecting water until no air comes out of the second petcock.

Step 6. Conduct the Air Content Test

20-25. Close the air bleeder valve on the air chamber, and pump air into the chamber until the needle on the pressure meter reaches the initial-pressure line. Allow a few seconds for the air to cool, and thump the gauge to ensure that the needle is working freely. Adjust the air with the pump or bleeder valve to get the

needle back on the initial reading line if it has drifted off. Check the petcocks for evidence of air leakage. Air leakage at this point would indicate a problem with the valve between the air chamber and the bowl, which would cause faulty readings.

20-26. The initial-pressure line is constructed differently among brands of meters. Some have an adjustable line whose position is established during calibration. Others have a numbered scale called test mark below the zero reading. The number on this scale for the initial-pressure setting is determined during calibration.

20-27. Close both petcocks on the lid, and open the valve between the air chamber and the bowl. Tap the sides of the bowl gently with a rubber mallet. Read the percent air on the pressure gauge. Record this number. Subtract the aggregate correction factor, and report this as the air content.

Calibration

20-28. The exact steps in calibration vary among brands, so the manufacturer's user manual should be consulted for this step. Meters should be calibrated at least every 3 months.

Aggregate Correction Factor

20-29. Aggregate correction factors range from a few tenths of a percent for dense and low absorptive aggregates to as much as 1 percent for low-density and highly absorptive aggregates. Correction factors should be established for all aggregates used in concrete, but these do not need to be repeated with great frequency. Correction factors are normally relatively stable for a given source, but changes in density and absorption indicate the possible need for a check on the correction factor.

20-30. Determine the aggregate correction factor (percent) according to the following steps.

Step 1. Make a Sample

20-31. Assemble a sample of coarse and fine aggregate that represents the amounts in a sample of concrete used for an air content test. These are calculated from the following formulas:

$$F_s = (S/B) \times F_b$$
$$C_s = (S/B) \times C_b$$

where—
F_s = weight of fine aggregate in concrete sample under test (in kilograms or pounds)
S = volume of concrete sample (same as volume of bowl) (in cubic millimeters or cubic feet)
B = volume of concrete in a batch
F_b = total weight of fine aggregate in the moisture condition used in a batch (in kilograms or pounds)
C_s = weight of coarse aggregate in concrete sample under test (in kilograms or pounds)
C_b = total weight of coarse aggregate in the moisture condition used in batch (in kilograms or pounds)

Note. Use mass units of kilograms or pounds, but be consistent throughout the calculation.

Step 2. Place Aggregate in the Bowl

20-32. Mix samples of fine aggregate and coarse aggregate, and place in the measuring bowl filled about one-third full of water. Prevent the entrapment of air by putting in small amounts of aggregate at a time. Add additional water to completely cover the aggregate if needed. Stir and rod the aggregate sample, and tap the sides of the bowl with a rubber mallet to remove any entrapped air. Remove foam, and let the aggregate stand in water for approximately the same time that would elapse in the making and testing of a normal batch of concrete.

Step 3. Conduct the Test Procedure

20-33. Completely fill the bowl with water, and execute an air determination as described above for a complete concrete sample. The measured air content is the aggregate correction factor, and the number is subtracted from the air content determination made on a sample of concrete.

FLEXURAL-STRENGTH TEST (MODULUS OF RUPTURE)

20-34. ASTM C78 covers the determination of flexural strength. 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 megapascals (pounds per square inch). Perform this test to determine the flexural strength (modulus of rupture) of the test specimen to the nearest 0.1 megapascals or 10 pounds per square inch. Record the specimen identification, modulus of rupture, any defects noted, and specimen's age. Measurements may be in metric or English units. If it is not practical to be consistent, appropriate conversions to one system or the other will be required before final calculations are made.

TEST BEAMS

20-35. Test beams may be cast in suitable molds or sawed from hardened concrete. ASTM C192 covers the fabrication and handling of beams in a laboratory setting, ASTM C31 covers the fabrication and handling of beams in field conditions, and ASTM C42 covers the sawing beams from hardened concrete. Beam molds for casting test beams from fresh concrete are available in many sizes. The most commonly used size is 150 by 150 by 530 millimeters. Limits on specimen size are—

- The smallest beam dimension is at least three times the maximum aggregate size.
- The length of the beam is at least 50 millimeters longer than three times the depth.
- The ratio of beam width to depth is less than or equal to 1.5.

Samples

20-36. Make test specimens at least once for each 100 cubic yards (or fraction thereof) for each class of concrete placed in any 1 day or as directed in the project specifications. Make at least three specimens for each test age and mixture design being evaluated in the laboratory. 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.

Specimens

20-37. The steps for making cast test specimens are discussed the following paragraphs.

Step 1. Location of Specimen Molding

20-38. Mold specimens on a flat, rigid surface that is free of vibration and other disturbances at a place as near as practicable to the place that they will be stored during the first 24 hours. If specimens must be moved or if it is desired that specimens be put into the curing room the same day as they are molded, move them to the place of storage as soon as possible after molding. Avoid vibrations, jarring, and other physical disturbances as much as possible.

Step 2. Placing and Consolidating Concrete in Molds

20-39. Assemble the 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. Consolidation is accomplished by rodding or by internal vibration. Internal vibration is required when the slump is less than 25 millimeters. Rodding or internal vibration is allowed when slump is greater than 25 millimeters.

Step 3. Rodding Option

20-40. 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.

Step 4. Internal Vibration Option

20-41. Vibrate each of the two layers at four equally spaced locations along the specimen centerline. Insert the vibrator slowly, and do not allow the tip to touch the mold. Vibrate at each location for 5 to 10 seconds. The shorter time is suitable for concrete with a slump over 3 inches, while lower slump mixes should be vibrated for longer periods, but not usually more than 10 seconds. Normally, enough vibration has been applied when the surface becomes relatively smooth and large air bubbles cease to break through the surface.

Step 5. Strike Off and Finishing

20-42. After consolidation, strike off excess material and finish the test according to any special instructions in the project specifications. If there are no special finishing instructions, finish with a wood or magnesium float.

Curing Beams

20-43. Leave the specimens in the molds for the first 16 to 24 hours after molding. Do not move the specimens during this period. Cover the exposed surface with wet burlap or plastic to prevent drying; or if specimens are in a curing room, do not allow water to drip on the specimen surface. After the initial curing period, specimens may be moved and demolded. Keep the specimens wet in a curing tank or fog room after demolding. Standard curing temperature is 23 ± 2°C. Curing tanks contain saturated lime water, and specimens are totally submerged. Specimens protruding from the tank may suffer evaporation atmosphere and consequent drying and cooling.

Equipment

20-44. Use the following items to perform this test in a laboratory environment:

- A calibrated load testing device.
- A concrete beam.
- A measuring tape.
- A stopwatch (if using a testing machine that does not control load rate)
- Safety goggles.
- A calculator.
- A 16-millimeter diameter tamping rod, if using the tamping form of consolidation.

Steps

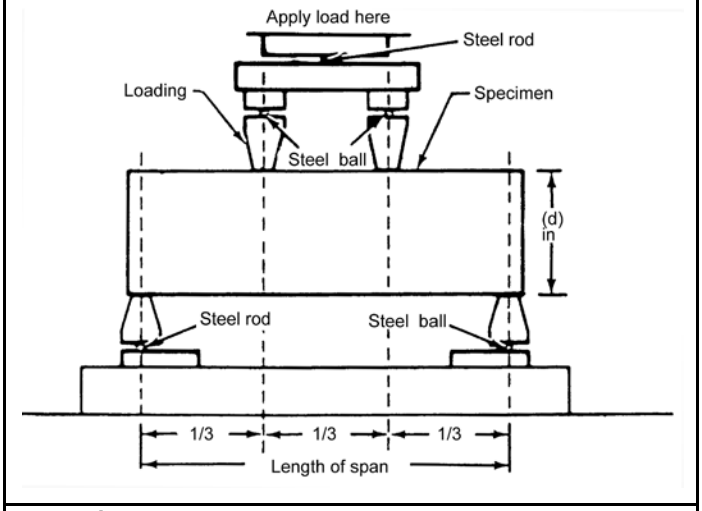
20-45. The steps for performing the flexural-strength tests are discussed in the following paragraphs.

WARNING

Wear safety goggles throughout this test.

Step 1. Assemble the Test Appartus

20-46. Assemble the test apparatus, and check for functional operation (figure 20-3).



Legend:

d	distance
in	inch

Figure 20-3. Apparatus for flexural-strength test

Step 2. Measure and Record the Length of Span

20-47. Measure and record the length of span, in millimeters or inches, depending on the measuring device. 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. The normal length of specimens in the United States is 530 millimeters and the normal length of span is 450 millimeters.

Step 3. Place the Specimen in the Load Tester

20-48. Place the specimen in the load tester and bring the loading surface into contact with the test specimen.

Step 4. Zero the Gauge

20-49. Zero the gauge. Some apparatuses are equipped with an automated hydraulic pump and corresponding gauge, while others are equipped with a loading jack and proving ring.

Step 5. Apply a Load

20-50. Apply a load at a minimum continuous rate of 6,500 to 9,000 newtons or 1,500 to 2,100 pounds/minute. The choice of units will depend on the instrument setup configuration.

Step 6. Record the Total Load at Failure

20-51. Record the total load at failure in newtons or pounds, depending on the readout of the instrument.

Step 7. Determine and Record the Specimen Width and Depth

20-52. Determine and record the width and depth of the specimen (in millimeters or inches) at the point of failure. Standard U.S. specimens are 150 by 150 millimeters.

Step 8. Determine the Point of Failure

20-53. 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, the specimen is considered unusable and should be discarded (not more than 23 millimeters for 450-millimeter span).

20-54. 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 pounds per square inch or megapascals)
P = applied load (in pounds or newtons)
L = length of span (in millimeters or inches)
b = width of specimen at failure point (in millimeters or inches)
d = depth of specimen at failure point (in millimeters or inches)

20-55. 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 megapascals or pounds per square inch)
P = applied load (in newtons or pounds)
b = width of specimen at failure point (in millimeters or inches)
d = depth of specimen at failure point (in millimeters or inches)
a = distance between the failure point and the nearest support, measured along the centerline of the bottom of the specimen (in millimeters or inches)

Step 9. Record Information

20-56. Record the following information about the test (some information may be unavailable at the time of the test):

- Specimen identification number.
- Average width to the nearest millimeter or 0.05 inch.
- Average depth to the nearest millimeter or 0.05 inch.
- Span length (in millimeters or inches).
- Maximum applied load (in newtons or pounds).
- Modulus of rupture (to the nearest 0.1 megapascals or 10 pounds per square inch).
- Curing history (how the specimen was cured) and apparent moisture condition of the specimen at the time of the test (surface wet or surface dry).
- Defects noted in the specimen.
- The age of the specimen (in days).

COMPRESSIVE-STRENGTH TEST

20-57. The compressive strength of hardened concrete, as measured by compression tests on standard forms of cylindrical specimens, is used in the design of structures. ASTM C39 is the basic test method for determining strength of concrete. ASTM C42 covers preparation of test specimens from in place concrete. ASTM C192 covers preparation of specimens from laboratory concrete. 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 also used 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. Compressive strength is measured in units of megapascals or pounds per square inch.

CASTING A CONCRETE CYLINDER

20-58. The standard test specimen is a cylinder that is 150 millimeters in diameter by 300 millimeters long. Smaller cylinders (100 by 200 millimeters) are also often used. The governing rule on cylinder size is that the diameter needs to be at least three times the maximum aggregate size. Length-to-diameter ratios should be 2:1. For testing, cylinders are 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.

20-59. Fill the cylinders as near as possible to the place where they will be stored for the first 16 to 48 hours. Sufficient concrete, about 70 kilograms 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 it may be contaminated with excess water. Use appropriate sampling procedures for procuring the sample as described for fresh concrete testing.

Equipment

20-60. Use the following items and information to perform the compressive-strength test in a field or simulated field environment (figure 20-4):

- A tamping rod, about 15 millimeters in diameter and 600 millimeters long, with a rounded end.
- Oil for releasing the forms (if using reusable molds)
- Rags.
- Cylinder molds (reusable or single use).
- A sheet of plastic or burlap.
- A kitchen scoop or trowel.
- A grease pencil or other permanent marking device (note that many permanent markers fade in prolonged exposure to direct sunlight).
- Water for cleanup.
- A curing tank or room.
- Calipers that are capable of measuring at least 150 millimeters, to within 1 millimeter.
- A carpenter’s square that is at least 250 millimeters on one side and 150 millimeters on the other side.

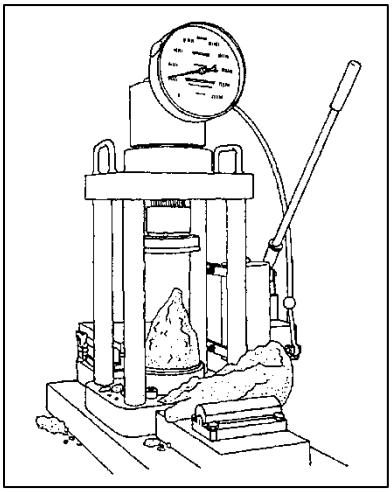


Figure 20-4. Hand-operated compressions tester (shown testing a concrete cylinder)

Steps

20-61. Perform the following steps to produce and label a concrete cylinder for testing:

- **Step 1.** Prepare the mold if using reusable molds. (Single-use molds (usually plastic) require no preparation, but they should be inspected to ensure that they are not physically distorted. Common distortions are of round at the top edge or convex bottom surfaces.)
  - ***Step 1a.*** Clean and dry the mold.
  - ***Step 1b.*** Oil reusable molds lightly.
  - ***Step 1c.*** Assemble the mold.
- **Step 2.** Make the cylinder.
  - ***Step 2a.*** Fill the mold one-third full with fresh concrete.
  - ***Step 2b.*** 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.
  - ***Step 2c.*** Tap the side of the mold 8 to 10 times with the tamping rod.
  - ***Step 2c.*** Add concrete to the mold to fill it two-thirds full.
  - ***Step 2e.*** 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.
  - ***Step 2f.*** Tap the side of the mold 8 to 10 times with the tamping rod.
  - ***Step 2g.*** Add concrete to the mold to overfill it slightly.
  - ***Step 2h.*** Repeat step 2e. The tamping rod must pass entirely through the top layer and 1 inch into the preceding layer.
  - ***Step 2i.*** Tap the side of the mold 8 to 10 times with the tamping rod.
  - ***Step 2j.*** 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, the following information:
  - ***Step 3a.*** The specimen number.
  - ***Step 3b.*** The date the cylinder was made.
  - ***Step 3c.*** The project or placement where the concrete was obtained.

Note. The system of labeling is optional, but the specimen is labeled on the mold with a permanent label. Do not rely on a specimen lid label (if used).

- **Step 4.** Cover the cylinder with plastic or wet burlap to maintain moisture in the sample if a tight-fitting lid is not used. 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 (plus 8) hours.
- **Step 7.** Label the concrete cylinder. The label itself may be transferred, or the information may be recorded directly on the cylinder with a grease pencil or permanent marker.
- **Step 8.** Cure the cylinder as directed by the project requirements.

Specimens

20-62. The number of specimens tested depends on the job specifications. If no requirement is listed in the specifications, a minimum of two will be molded for each test age for each 100 cubic yards, or fraction thereof, of each class of concrete placed in any 1 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

20-63. After an initial curing period of 16 to 48 hours, remove specimens (from the jobsite) 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 23 ± 2°C. 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. ASTM C511 covers moist rooms and curing tanks.

20-64. Occasionally, test specimens are made in the field to determine when forms may be removed. Make these in addition to the specimens made for quality control or specification compliance testing. 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.

20-65. 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, wet sand, or a closed plastic bag containing a small amount of water (a water-soaked rag is effective). Provide protection from freezing and physical damage during storage or shipment. Moist curing is continued when the specimens are received in the laboratory.

PREPARING TEST SPECIMENS FOR TESTING

20-66. The preparation of test specimens is discussed in the following paragraphs.

Specimen Condition

20-67. The physical dimensions and features of test specimens have important effects on strength measurements (ASTM C39). Important details are given in the following paragraphs.

20-68. Measure (to the nearest 0.25 millimeter) the diameter at several points on the circumference, at midheight of the specimen. If any diameter is more than 2 percent different from any other diameter, do not test the specimen. This sometimes occurs when single-use molds become distorted into an oval shape during storage or handling. The average of this diameter, if within tolerance, is used to calculate the cross-sectional area of the specimen that is required input for final calculations.

20-69. The ends of the specimens are required to be plane (no depressions and projections) to within 0.050 millimeter. The ends of the test specimen must also be perpendicular to the axis of the cylinder within 0.5°. This angle is approximately equivalent to a 1.5-millimeter gap across a 150-millimeter diameter test specimen. This can be determined holding a carpenter’s square against the side of the specimen and measuring the maximum gap under the side of the carpenter’s square that crosses the end of the specimen.

20-70. If specimens fail to meet the above requirements, they may be ground or sawed to achieve these tolerances or capped if the defects can be overcome with caps no thicker than 8 millimeters for concrete strengths less than or equal to 35 megapascals or 5 millimeters for concrete strengths between 35 and 50 megapascals. Specimens for concretes stronger than 50 megapascals must be ground to the above tolerances. (See figure 20-5, page 20-14.)

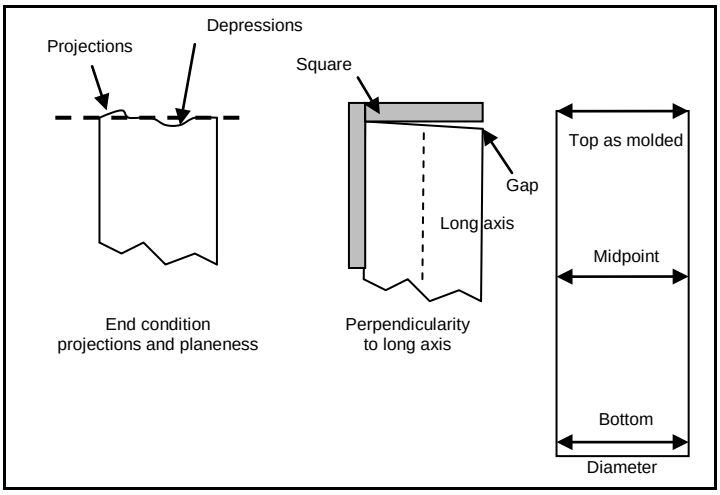


Figure 20-5. Important specimen-condition features

Cylinder Capping

20-71. If capping is required, the following methods are acceptable: capping with neat cement, high-strength gypsum, or sulfur (ASTM C617), or using unbonded caps (ASTM C1231). The basic requirement for capping materials is that they be stronger than the concrete to which they are applied. Ordinary plaster of paris, compressible rubber, or fibrous materials are not suitable for caps. Strength must be verified by fabricating and testing 50- by 50-millimeter cubes made of the capping material. During capping, protect moist-cured specimens against drying by covering them with wet burlap.

Neat Cement and High-Strength Gypsum

20-72. Neat cement caps and high-strength gypsum caps are somewhat impractical options because of the difficulties in forming the material onto the ends of the test specimens within the tolerances of thickness, planeness, and perpendicularity to the axis of the cylinder. An advantage of neat cement caps is that portland cement is normally an easily obtained material, so no specialty products need to be procured. The strength of both materials must be higher than the concrete for which they are testing. This must be determined by qualification tests on the strength of the capping material using 50- by 50-millimeter cubes or 50- by 100-millimeter cylindrical specimens. Neat cement caps must also be cured for several days to gain sufficient strength to qualify as a capping material. A water-cement ratio of about 0.35 is usually sufficient to give adequate strength. High-strength gypsum will gain strength to levels of approximately 5,000 pounds per square inch within 1 to 2 hours if water-gypsum ratios are kept in the range of 0.26 to 0.30. Gypsum caps cannot be stored more than 4 hours in a moist room.

20-73. Capping with sulfur is a commonly used technique in testing laboratories. Hardware (capping jig) is available to hold test specimens in rigid vertical positions while forming the cap, and sulfur sets very rapidly. Sulfur caps require at least 2 hours of curing time before testing. Molten sulfur must be heated to at least 129°C to ensure adequate strength development. Since the melting point of sulfur is only 113°C, the entire mass of capping material must be melted before it can be heated beyond the melting point to the required temperature. 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 discarded.

Unbonded Caps

20-74. Unbonded caps are covered by ASTM C1231. The use of unbonded caps is a very practical option compared to the other capping procedures described above if equipment is locally available. When using unbonded caps, there are no capping materials that must be mixed, formed, and cured prior to determining strength. The following are potential disadvantages:

- The rubber pads must be replaced periodically, so a supply of these must be available.
- The caps must be qualified by comparing results with another capping method for strengths over 7,000 pounds per square inch.

20-75. Unbonded caps are applicable for determining strengths greater than 1500 pounds per square inch and less than 12,000 pounds per square inch. Test-specimen condition requirements include the following:

- Diameter less than 2.0 percent variation.
- Planeness of ends less than or equal to 5 millimeters (gaps under straight edge).
- Perpendicular to axis of cylinder 0.5° (3 millimeters in 300 millimeters).
- Rings and pads must be certified for C1231/C1231M use (table 20-2). Rings and pads are sized for 4- and 6-millimeter test specimens. Pad hardness must conform to the requirements in table 20-2.

Table 20-2. Requirements for unbonded caps

Strength Range in Megapascals (Pounds per Square Inch)	*Hardness (Shore Durometer)*	*Number of Uses Before Replacement*	*Qualification Tests Required*
10–40 (1500–6,000)	50	100	No
20–50 (2,500–7,000)	60	100	No
30–50 (4,000–7,000)	70	100	No
50–80 (7,000–12,000)	70	50	Yes (ASTM C1231)
Legend: ASTM American Society for Testing and Materials			

DETERMINING COMPRESSIVE STRENGTH OF A CYLINDRICAL SPECIMEN

20-76. Perform the compressive strength test to determine the compressive strength of the concrete cylinder to within 10 pounds per square inch. The form of the fracture pattern at failure gives information about possible errors in test conditions.

Equipment

20-77. Use the following items to perform the compressive strength test in a well-ventilated laboratory:

- A cylindrical concrete test specimen.
- A load-testing machine, capable of measuring the largest load anticipated for the concrete under test. Normally, a machine of at least 1,000 kilonewtons (about 200,000 pounds) is sufficient for most applications using 150 by 300 millimeter test cylinders. A lower loading capacity is suitable for smaller specimens.
- Heat-resistant gloves (if using sulfur caps).
- Capping compound materials and capping jigs (sulfur or cement caps) or unbonded cap devices of the proper size for the test specimens.
- A ruler, readable to 0.25 millimeter.

Note. Conventional, commercially available rulers are usually readable to only 1 millimeter. Errors in strength determinations associated with measuring the diameter of a 150-millimeter test specimen to the nearest 0.25 millimeter is about ± 0.3 percent. Errors associated with measurements taken to the nearest 1 millimeter result in an error of about ± 1.3 percent.

- Calipers with an opening larger than the diameter of the largest test specimen.
- Safety goggles.
- A protective apron.
- A face shield.
- Oil (if using capping compound).
- Rags.
- A calculator.

Steps

20-78. The following paragraphs discuss the steps to determine the concrete cylinder's compressive strength.

Step 1. Determine and Record the Average Diameter

20-79. 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.

Step 2. Prepare the Concrete Cylinder

20-80. Follow the procedures described below for each capping type used.

- **Sulfur capping material.**
  - **Step 1.** Completely melt the capping compound in the melting pot to a temperature of 130 to 145°C. Ensure that enough compound is melted to make several caps.
  - **Step 2.** Clean and lightly oil the baseplate of the capping apparatus.
  - **Step 3.** Set the baseplate into the stand of the capping-apparatus.
  - **Step 4.** Pour a small amount of the heated (liquid) capping compound into the baseplate.
  - **Step 5.** 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.
  - **Step 6.** Remove the cylinder from the capping apparatus when the capping compound has solidified.
  - **Step 7**. Inspect the cap for uniformity and defects. If any defects are seen, remove the cap, recap the cylinder, and return to step 1.
  - **Step 8.** Repeat steps 1 through 7 for the uncapped end of the cylinder.
- **Neat portland or hydraulic cement caps and high-strength gypsum caps.**
  - ***Step 1.*** Mix the paste at the water-cement ratio used for the qualification of the material.

Note. Forming the caps can be tricky (ASTM C617).

  - ***Step 3.*** Cure for the length of time required to get adequate strength, as determined in the material qualification.
- **Unbonded caps.**
  - ***Step 1.*** Clean and examine the bearing surface of the steel cap for cracks or splits exceeding 10 millimeters in length. 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.*** Center caps on the cylinder, and place the test specimen on the lower bearing block of the testing machine. Carefully align the axis of the cylinder with the center of thrust of the testing machine by centering the upper retaining ring on the spherically seated bearing block.
- ***Step 3.*** Rotate the moveable portion gently by hand, as the spherically seated block is brought to bear on the upper retaining ring, so that uniform seating is obtained. After application of less than 10 percent of the anticipated strength, ensure that the test specimen is vertical and centered in the retaining rings. If the specimen is misaligned, release the load, recenter, and reload.
- ***Step 4.*** Complete the loading of the specimen as described in step 5.

Step 3. Prepare the Compression Tester

- **Step 3a.** Clean the tester's bearing plates (loading surfaces).
- **Step 3b.** Check the tester for proper operation.
- **Step 3c.** Set the gauge to zero.

Step 4. Place the Capped Cylinder in the Compression Tester

20-81. Place the capped cylinder into the compression tester, and center it on the bearing plates. Secure the protective cage around the cylinder.

Step 5. Apply the Test Load

20-82. Apply the test load at a rate of 0.15 to 0.35 megapascals/second (20 to 50 pounds per square inch/second).

Step 6. Read the Gauge and Record the Load Applied

20-83. Read the gauge, and record the load applied at the time of failure.

Step 7. Inspect the Broken Cylinder and Record Information

20-84. Inspect the broken cylinder, and record the following information:

- Identification number.
- Diameter (in units of the measuring device [millimeters or inches])
- Cross-sectional area, in square inches, $A = \pi(\mathrm{d}/2)^2$
  where—
  A = cross-sectional area
- Record the maximum load applied, in the units reported for the machine (in newtons or pounds).
- Compressive strength (calculated to the nearest 0.1 megapascal).
- Type of break (figure 20-6, page 20-18). The types of breaks are as follows:
  - ***Type A.*** Type A is expected under normal operating conditions.
  - ***Types B through E.*** Types B through E may indicate problems with adherence to test procedures.
- Defects in specimen or caps.
- Age of specimen.

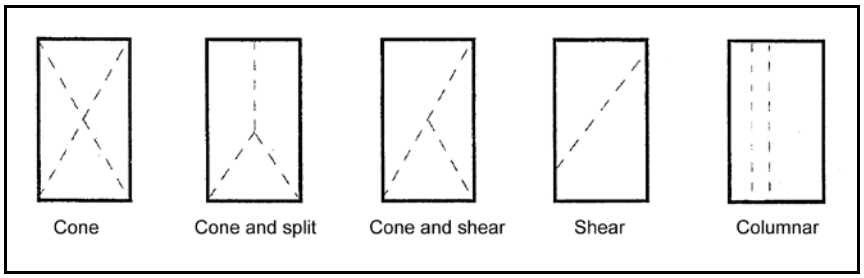


Figure 20-6. Types of breaks

Step 8. Calculate and Record the Compressive Strength

20-85. Calculate and record the compressive strength of the cylinder in units of megapascals or pounds per square inch using the following formula. Convert units as necessary.

$$compressive\ strength = \frac{P}{A}$$

where—
P = load at time of failure (in newtons or pounds)
A = area of the end of the test specimen (in square millimeters or square inches)

NONDESTRUCTIVE TESTING

20-86. The portable seismic pavement analyzer (PSPA) is a portable, nondestructive device that rapidly measures the physical integrity of hydraulic-cement concrete and asphalt concrete pavements. All measurements are made on the pavement surface, but information on properties at depth can be derived from the output. Figure 20-7 shows two PSPA units and the portable computers that control the instruments and log the data.

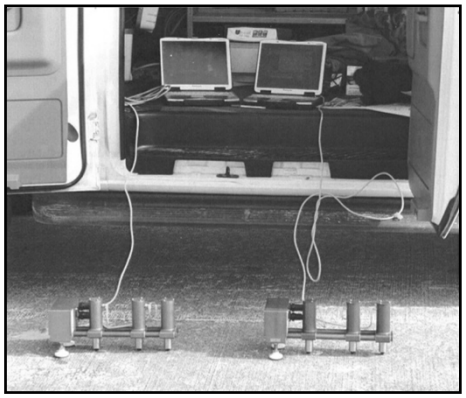


Figure 20-7. PSPA instruments

20-87. The principle of the PSPA operation depends on ultrasound. Ultrasound waves are transmitted into the concrete through the rubber feet under the electronics box on the left side of the instrument. The waves radiate down and out through the concrete from this point. The three cylindrical components mounted on the extension arm to the right of the electronics box are the receivers that capture the ultrasound as reflected and/or transmitted waves after they have moved through the concrete. The spacing of the three receivers allows focusing data on various depths of the structure.

20-88. The output data gives information on the modulus of elasticity, which can be related to strength; and it also detects irregularities in the pavement (layers of delamination, pavement thickness).

20-89. Training in the operation of the PSPA takes about 1 day. Details of the operation are outside the scope of this document, but are included with the manual that accompanies the instrument. If the manual is missing, operational instructions can be found in ERDC/GSL SR-06-9.

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Chapter 21

Contingency Field Testing and Troubleshooting

Under some conditions, operations may require troubleshooting fresh and early hardened properties of concrete without the benefit of normal laboratory facilities. The purpose of this chapter is to present some approaches to isolating the causes of these problems, and to present some nonstandard procedures that can help gain some insight into problems with concrete in the event standard testing facilities are not available. These ad hoc test methods are not suitable for official acceptance testing purposes, but can help determine the source of some problems. The most common field problems that sometimes cause interruptions in operations include low strength, early loss in workability, and large irregularities in the time of setting, the loss of air, and early cracking. Sometimes, a combination of these will exist.

NONSTANDARD TESTS

21-1. Nonstandard tests are as follows:

- **Strength.** It is difficult to measure strength numerically without the benefit of a load tester. However, approximate conditions can be estimated by simply impacting concrete with a hammer. Concrete with serious strength problems will crumble fairly easily under the impact of a hammer, with paste and aggregate easily separating out. The concrete actually feels somewhat soft under the impact. Of course, this is the condition that is expected in concrete in the first few hours or first 24 hours after setting, but should not exist after several days of curing. Concrete that has attained normal strength will normally require significantly more impact energy (unless the impact is very close to an edge or corner, it is fairly easy the chip the edge off of concrete). The concrete will feel brittle when fractured, and fractures will often break through the coarse aggregate particles.
- **Loss of workability.** The loss of workability is normally evaluated in the field by measuring changes in slump (ASTM C143) at intervals of about 15 minutes. In the absence of a slump cone, relative behavior can be assessed with homemade slump cones (such as a small bucket with the bottom cut out). A reasonable facsimile of a slump cone can be fabricated from sheet metal. Data from these ad hoc devices will not correctly reflect executed slump tests, but changes in properties can be reasonably assessed using such devices. In extreme cases, the loss of workability can be assessed simply by trying to move the concrete around with a shovel.
- **Time of setting.** Standard laboratory equipment for measuring the time of setting according to ASTM C403 is relatively complicated. However, this behavior can be assessed in the field with simple ad hoc techniques. The time of initial setting normally corresponds to the time when the paste in the concrete shows some perceptible resistance to penetration with a pointed stick (pencil, ballpoint pen). It is also closely indicated by the loss of sheen on the surface of the concrete. To see this behavior, cover the surface of the concrete to prevent evaporation of the bleed water. As the concrete starts to set, it also starts to consume some or all of the bleed water on the surface. Unless bleeding is excessive, all of the bleed water will be absorbed, leaving a dull appearance on the surface. The time of final setting can be approximated as the point at which there is significant resistance to penetration of a pointed stick with moderate force (for example, the concrete can be walked on).

- **Bleeding.** Bleeding is seldom routinely measured on concrete. However, sometimes questions arise about the concrete mixture when conspicuous bleeding is seen in the field. The standard test method is ASTM C232, which basically directs the user to fill a standard container, pour off the bleed water as if forms, and report the total. A reasonable alternative is to use a 150- by 600-millimeter strength mold or similar-size container. Fill it to the rim with concrete, screed off the excess and cover to prevent evaporation. Check the sample over a period of about an hour. If bleeding is more than 1 to 2 millimeters, bleeding may be too high. This simple approach is useful for comparing among mixtures. Excessive bleeding is also often evidenced by the mounds of paste deposited around small holes in the surface of the concrete. These are bleed channels that form when concrete bleeds heavily, carrying cement fines to the surface.
- **Air content.** The two most commonly used standard methods for measuring air content are the ASTM C231 (pressure method) and ASTM C173 (volumetric method). These both require specific hardware to execute. However, air can be estimated in the field without these, but a number of calculations are involved that require some materials property data, a container of known volume, and a device for determining weight. Knowing the density and amount of each of the main components of a concrete batch (cement, SCMs, aggregate, and water), a theoretical weight of a given volume of concrete can be calculated. Then, when the concrete is actually made, the amount to which the weight falls below the theoretical weight indicates the amount of air in the concrete.

TROUBLESHOOTING

21-2. Common field problems and troubleshooting approaches are discussed in the following paragraphs.

LOW STRENGTH

21-3. Low strength in concrete can derive from one thing or a combination of several things: insufficient cement content, too much water, poor quality cement, weak aggregate, mixing-water chemistry, temperature, poor curing, and unexpected admixture effects.

21-4. Low cement content sometimes occurs in concrete due to batching errors or fraud (cement is the most expensive component of concrete). Prebagged materials are not immune to this problem. Cement content is difficult to measure without laboratory facilities; however, where the cement content is very low, the mortar fraction of the concrete will tend to look like wet sand rather than smooth cement paste that tends to dominate the appearance. The mortar is also soft and easily dug out of the concrete with a knife or other metal object even after several days of curing.

21-5. The addition of mixing water beyond what is allowed in the project specifications is probably the major cause of low strengths. This error is difficult to determine without experienced laboratory capability. This problem is particularly acute with concrete batched and hauled a significant distance to the placement site. Concrete more than about 1.5 hours old (less in very hot weather) tends to start setting up. Workability can be restored by the addition of more water. The addition of more than about 10 percent more water than allowed in the batch water will have significant effects on strength. The only field activity that can be used to detect this is to observe practices surrounding the delivery of concrete. Sometimes, adding water is allowed because some of the water was intentionally left out of the mix with the intention of adding it later. However, many delivery operators are barely aware that adding water may be forbidden or the practice is so common that they tend to ignore the rules on it. As a result, objectionable additions of water are often done quite openly. If batch tickets are available, these may be useful in determining if extra water should be allowed.

21-6. Cement or SCMs are sometimes of inferior quality. In theory, these have both been tested at a competent laboratory before delivery and the test results included on a mill certificate. Unfortunately, not all mill certificates accurately represent the cement or SCM being delivered. Some insight into the contribution of cement to a strength problem can be gained if a sample of the cement can be obtained. Make a mortar of two parts sand and one part cement (by volume), and add just enough water to make a moderately stiff mortar (no immediate bleeding). If standard molds are not available, paper cups or other small containers can be used. Store the container in a moist condition as close to 23°C as practical.

Significant strength should be evident by 3 days. Most cements reach at least 10 megapascals by 3 days, and most reach about 25 megapascals (3500 pounds per square inch) by this age, although there will be some variation if ad hoc methods are used. Reasonable strength specimens should break with a hammer, but require at least moderate intensity being applied. The specimens may not be sharply brittle at this strength (paragraph 21-1). This brittleness might not be perceptible until about 28 days. Low strength materials will be soft, breading with low energy hammering.

21-7. Weak aggregate can contribute to low strength, although low strengths from this cause are not usually extremely low. Weak aggregate is evidenced by many particles that are easily broken by hand or light mechanical pressure.

21-8. Mixing water from industrial wastewater contaminated sources and from wash water around concrete mixing facilities can cause setting and strength-gain problems. Water from normally acceptable drinking water sources is fine. Most natural waters are also fine, but sometimes industrial contaminations aren't known. Mixing water can be easily tested with ad hoc methods. Make mortars as described above for estimating cement strength. Make one batch with test water and another with drinking water. If strengths at 3 days and setting times are reasonably close, then the mixing water probably is not a problem.

21-9. The temperature has strong effects on the rate of strength gain. Below about 5°C strength gain is very slow, about half as fast as at ideal temperatures of around 25°C. Freezing concrete before it is about 3 days old can cause damage that results in low strengths. High temperatures accelerate strength gain even more and are not a problem, except that hot concretes tend to lose significant amounts of water due to evaporation, particularly under windy conditions. Many agencies limit concrete placing to temperatures between a low of 5°C and rising about 30°C due to early stiffening and drying concerns.

21-10. Failure to provide adequate curing water can cause low strengths, particularly of the near surface concrete. In extreme cases, the concrete surface is easily abraded. Concrete rarely dries to a depth of more that about 50 millimeters due to poor curing. Surface cracking is another symptom of this kind of poor curing. Curing usually needs to be continued for at least 3 days unless the weather is wet enough that significant evaporation does not occur.

21-11. Occasionally admixtures interact with specific cement sources and result in extremely slow strength gain. These problems are more common with WRAs than other chemical admixtures. It is a more common problem when admixtures are used at, or higher than, the maximum recommended dosages. This can be investigated using testing similar to the investigation of mixing water, paragraph 21-8, except make one batch with and one batch without admixture (or at a lower dose). The strength problem normally accompanies an extremely long setting time (many hours or even days).

EARLY LOSS OF WORKABILITY

21-12. Early loss of workability is evidenced by problems with spreading and consolidating concrete. Slump measurements are useful in documenting this behavior. Slump that drops from the upper range of specified slump to below the minimum range in less than an hour is usually a problem. If concrete can be placed and consolidated before this loss of workability sets in, there will be no further problem with the concrete. Early loss in workability is relatively common in otherwise-acceptable concrete when temperatures of the concrete get much above 32°C. Other causes include cement, mixing-water chemistry, and admixture effects.

21-13. Some cements exhibit a tendency to show a weak setting behavior within a few minutes after adding mixing water (called false setting). Although relatively weak, this stiffening is sufficient to interfere with emptying the truck, or spreading and consolidating concrete. This rarely happens if the concrete has been mixed for more than 2 to 3 minutes, so it is not a problem with transit-truck delivered concrete (as long as it is an agitating or revolving drum truck). The problem usually occurs when concrete is mixed on site. Times for on-site mixing are typically about 1 minute. Increasing the mixing time by a minute or two will work through this problem, and it will not cause further problems in the concrete.

21-14. Admixture effects similar to those described for low-strength concrete problems can also cause early loss in workability. These can be investigated using the same concept as investigating strength problems (compare slump losses for concrete with different dosages of admixture and with no admixture).

21-15. Mixing-water chemistry problems sometimes cause cement to hydrate more rapidly than normal. This can be tested by comparing slump loss between concrete or mortar made with drinking water and with water.

SETTING-TIME PROBLEMS

21-16. Setting time and early stiffening are often assumed to be different extremes of the same phenomenon. However, early stiffening is usually caused by a different chemical reaction than setting. Sometimes, concrete will have early loss of workability (as indicated by slump loss), but delayed time of setting. True setting does not usually occur until after 2 hours.

21-17. Even though setting-time problems are caused by different chemical reactions than like workability loss problems, are caused by temperature, cement, mixing water, and admixtures. Isolating these problems follows the same logic as the workability loss problems, but the time of setting rather than slump loss is used as the metric.

EARLY AGE CRACKING

21-18. Early-age cracking, for purposes of this discussion, is cracking that occurs during the first 24 hours after placing. One form of early age cracking occurs, or is at least initiated, before the concrete reaches the initial setting. This is called plastic shrinkage cracking. Another form of cracking occurs after time of setting, but usually about 24 hours, although this depends on the weather. This cracking is caused by drying of hardened concrete and by thermal effects, or a combination of the two.

21-19. Plastic shrinkage cracking occurs because the rate of evaporation of bleed water exceeds the rate of formation of bleed water. This is difficult to anticipate. Concretes made with water-cement ratios below about 0.45 are particularly susceptible because of the relatively low bleed rate. This includes much paving concrete. This problem is particularly acute when conditions are windy and concrete temperatures are high. Air temperature and relative humidity also contribute, but to a lesser degree. The solution to this problem is to keep concrete as cool as possible and protect fresh concrete from too much wind or drying, which can be difficult in many construction environments. Much concrete is placed at night in strongly drying environments.

21-20. After the time of setting, concrete continues to dry, although usually at a slower rate than before setting. This drying can contribute to cracking as early as a few hours after setting when curing is particularly poor, but also at much later ages with moderately good curing. Thick placements of concrete (thick pavements, large structures) tend to generate a significant amount of heat internally due to the hydration of the cement. This, along with any environmental heating, can cause the concrete to reach temperatures above 65°C during the first 24 hours. The concrete then cools after it has become rigid due to setting, so it tends to shrink and crack. This is normally dealt with by sawing joints into the concrete that direct the cracking to locations that can then be sealed and managed. Sawing too late or at intervals that are too wide is the most common cause of random cracks during or soon after setting.

21-21. The practice of curing with wet burlap (or other absorbent material) in dry windy climates (particularly deserts) can cause cracking. The rate of evaporation from the wet absorbent materials can be high enough to cause significant evaporative cooling, resulting in a significant temperature drop in the concrete surface. If the rest of the concrete is hot, a sufficient temperature gradient can develop over the thickness of the structure to result in cracking. This can be prevented by covering the absorbent material with plastic.

PART FOUR

Soils Stabilization

Part Four 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.

Chapter 22

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

22-1. Stabilization is the process of blending and mixing materials with a soil to improve the strength and durability of the soil. 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 (TM 3-34.64/MCRP 3-17.7G and TM 5-822-14/AFJMAN 32-1019).

STABILIZATION USES

22-2. 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.

22-3. 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 Soil Quality

22-4. Stabilization is commonly used for better soil gradation, the reduction of the PI or swelling potential, and increased durability and strength. Stabilized pavement surfaces can be used for many common applications (roads, airfield facilities, construction platforms). Stabilized pavements generally do not provide the durability of asphalt or concrete surfacing.

Reduce Surface Pavement Thickness

22-5. 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/AFM 88-7. 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.

STABILIZATION METHODS

22-6. 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 (scarifiers, plows, disks, graders, rotary mixers, reclaimers/stabilizers) have been satisfactory.

22-7. 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 procedures. TM 5-822-14/AFJMAN 32-1019 lists the most suitable treatments for various soil types to stabilize these soils for different objectives and contains detailed descriptions of proper construction techniques.

Mechanical

22-8. 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, stabilization with an additive may be necessary. Compaction is also a form of mechanical stabilization.

Additive

22-9. 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. Additives may stabilize the soil by chemically, physically or a combination of the two (physicochemically). An example of pure chemical stabilizers are lime, acids, and enzymes that cause a chemical change in the soil (ion exchange, covalent reactions). An example of pure physical stabilizers are bitumens that bind the soil by adhesion and fibers that bind by mechanical friction and interpenetration. Physicochemical stabilizers (lignosulfonates, lime-cement-fly ash [LCF]) may bind soil particles together, but also induce chemical changes in the soil (pozzolonic reactions). 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 (gradation, workability, plasticity) than to improve the strength and durability sufficiently to permit a thickness reduction design.

MODIFICATION

22-10. 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 strength and durability of the soil.

22-11. 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 cement in small amounts.
- Adding chemical modifiers.

22-12. 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, control dust, hold fine material for better compaction, and reduce frost heave by lowering the freezing point of water in the soil. Bituminous or polymer emulsion may be used to waterproof the surface of the soil and to control dust.

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Chapter 23
Stabilizing Agents

This chapter 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, bituminous materials, and nontraditional stabilizers.

STABILIZER TYPES

23-1. To select the proper stabilizer for a particular soil, perform a sieve analysis test to determine gradation and fines content and an Atterberg limits test according to the procedures given in this manual.

TRADITIONAL STABILIZERS

23-2. Traditional stabilizers are discussed in the following paragraphs.

Cement

23-3. Portland cement can be used to modify and improve the quality of the soil or to transform the soil into a cemented mass with increased strength and durability.

23-4. 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

23-5. 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, GWGC, 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 if more than 25 percent of the soil passes the No. 200 sieve.

Fly Ash

23-6. 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. Some sources of fly ash may contain high amounts of sulfates. In some soils, this may lead to the formation of ettringite, which may swell excessively.

Bituminous

23-7. Most bituminous soil stabilization has been performed with AC, 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 if gradation and plasticity requirements are met.

Combination

23-8. 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 workability and mixing characteristics and to reduce the plasticity of the soil. 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.

NONTRADITIONAL STABILIZERS

23-9. Nontraditional stabilizers are discussed in the following paragraphs.

Polymers

23-10. Polymer emulsions (sometimes referred to as latex) used for soil stabilization are generally vinyl acetate or acrylic-based polymers suspended in water by surfactants. They typically consist of 45 to 50 percent polymer content by weight. Analogous to latex paint, the emulsion cures by water loss. The polymers used for soil stabilization typically have excellent tensile and flexural strength, adhesion to soil particles, and resistance to water. These materials are often limited by a short shelf life (less than 2 years) due to bacterial growth and settling. Polymer emulsions should not be mixed with gray or salt water for dilution.

Fibers

23-11. There are numerous fiber types available. For soil stabilization, a combination of availability, fiber properties, and cost per performance, pares the fiber type to polypropylene. These fibers are low-cost, are available in different lengths and diameters, and have been extensively tested. Monofilament, fibrillated, and tape fibers are available. The testing has shown that monofilament and tape fibers are more efficient for rapid stabilization. Fibrillated fibers may require extensive mixing to effectively defibrillate the fiber for maximum efficiency.

23-12. The most important fiber property for soil stabilization is fiber length. Longer fibers generally result in improved performance ,but hamper construction. The longer fibers are more difficult to mix, and considerable balling may occur, diminishing efficiency. Testing of different fiber lengths has shown that fiber lengths from 3/4 to 1 1/2 inches result in good soil improvements with minimal balling.

23-13. The soil type has a secondary role in fiber selection. Polypropylene fibers mixed into soil without adhesives (cement) rely exclusive on mechanical interlock, friction, and fiber entanglement between soil particles to anchor the fiber in place. Thus, a high surface area fiber (such as a tape fiber) may be more efficient in a fine-grained clay soil than in a coarse-grained soil (such as gravel) where fewer soil grains are in contact with the fiber.

Other

23-14. There are a variety of other nontraditional soil stabilization and modification additives available (acids, lignin derivatives, enzymes, tree resin emulsions, silicates). These additives vary widely in their chemical composition and mechanism of action, may be in liquid or solid form, and are often touted to be applicable for most soils. Research studies in this area have demonstrated that many nontraditional soil additives have little to no benefit for granular soil types. Sandy and gravelly soils are often problematic for stabilization, requiring a binder (cement, asphalt emulsion) to provide cohesion between soil grains. Medium- and fine-grained soils are likely to benefit most from the other nontraditional stabilizers. However, these must be considered on a case-by-case basis.

Combinations

23-15. Blends of soil with polymer emulsions and hydraulic cements and with fibers and hydraulic cements have shown promise in laboratory testing and field demonstrations. A synergism exists between

emulsions and hydraulic cement as the water in the emulsion hydrates the cement and the polymer adds flexibility, durability, and strain resistance to the soil cement, increasing the cracking resistance. The use of fibers is also very attractive due to their ability to improve the resistance of soil to deformation and resist the tensile stresses that lead to cracking. Fibers have been found to be more efficient than polymer emulsion on a weight-to-weight basis of comparison. However, fibers may be difficult to use since they tend to ball up when graded and they may be difficult to place in windy situations.

TIME REQUIREMENTS FOR TESTING

23-16. 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 (table 23-1). 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.

Table 23-1. Estimated time required for test procedures

Construction Type	*Stabilizing Agent*	*Time Required[1]*
Expedient	Any	None
Nonexpedient	Lime	30 days
	LCF	30 days
	Cement	6 to 9 days
	Bitumen	1 day
	Nontraditional	1 to 9 days
[1]These criteria do not include time required for gradation.		
Legend: LCF lime-cement-fly ash		

STABLIZER SELECTION AND CONCENTRATIONS

23-17. When selecting a stabilizer additive, many factors must be considered. These factors, the design criteria, and the selection and mixing of stabilizers can be found in TM 5-822-14/AFJMAN 32-1019 and FM 5-430-00-2/AFJPAM 32-8013/MCRP 3-17.7B, Volume II. If lime is used as a preliminary additive to reduce the PI or to alter the soil gradation before adding the primary stabilizing agent (bitumen, cement), 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 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 potential hydrogen (pH) test.

23-18. Several factors need to be considered in the selection of a stabilizer. The environmental conditions, the availability and cost of stabilizers, logistical considerations, the equipment available, the soil type, and the strength and durability needed for the desired application. The selection of the stabilizer will be based on modified charts and tables from TM 5-822-14/AFJMAN 32-1019. In general, the higher the amount of fines in the soil, the harder it is to stabilize. Thus, clays can be difficult to stabilize. Lime may be needed to reduce the plasticity and increase friability of high-plasticity clay before cement is added. For sandy and gravelly soils, some type of binding agent (cement or emulsion) is necessary to adhere the particles together. The selection of a stabilizer begins with a classification of the soil type (figure 23-1, page 23-4). Once the soil is classified according to figure 23-1; table 23-2, page 23-4, is used to make an initial selection of appropriate stabilizers for the soil type and table 23-3, page 23-5, is utilized to choose an appropriate stabilizer dosage level.

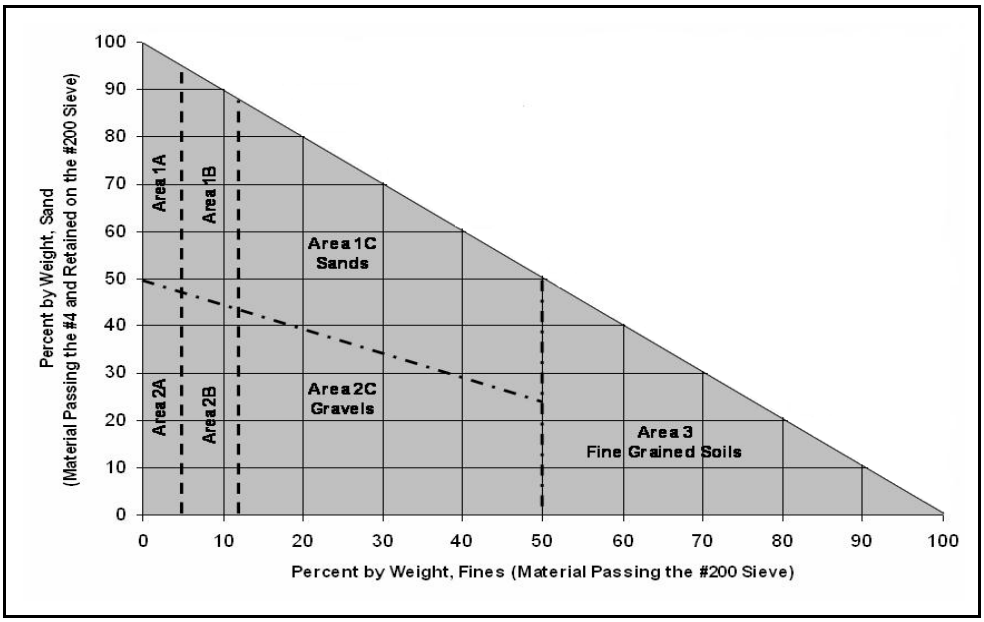


Figure 23-1. Gradation triangle for soil classification

Table 23-2. Soil stabilizers for different soil types

Area	*Class*	*Type of Stabilizer*[1]	*Restriction (LL and PI)*	*Restriction (% Pass No. 200)*[2]	*Remarks*
1A	• SW • SP	• Polymer/bituminous emulsion and cement • Portland cement and fiber • Lime-cement-fly ash and fiber	• PI not to exceed 25		• Soils near or above their optimum moisture content may require drying prior to emulsion stabilization.

1B	• SW-SM
	• SP-SW
	• SW-SC
	• SP-SC
		• Polymer/bituminous emulsion and cement
		• Portland cement and fiber
		  • Lime and fiber
		  • Lime-cement-fly ash and fiber
			• PI not to exceed 10
			• PI not to exceed 30
			• PI not less than 12
			• PI not to exceed 25
					• Soils near or above their optimum moisture content may require drying prior to emulsion stabilization

1C	• SM • SC • SM-SC	• Polymer/bituminous emulsion and cement • Portland cement and fiber • Lime and fiber • Lime-cement-fly ash and fiber	• PI not to exceed 10 • Restriction on LL and PL • PI not less than 12 • PI not to exceed 25	• Not to exceed 30 percent by weight	• Soils near or above their optimum moisture content may require drying prior to emulsion stabilization

Table 23-2. Soil stabilizers for different soil types (continued)

Area	*Class*	*Type of Stabilizer*[1]	*Restriction (LL and PI)*	*Restriction (% Pass No. 200)*[2]	*Remarks*
2A	• GW • GP	• Polymer/bituminous emulsion and cement • Portland cement and fiber • Lime-cement-fly ash and fiber	• PI not to exceed 25		• Well-graded material only • Material should contain at least 45% by weight of material passing No. 4 sieve • Soils near or above their optimum moisture content may require drying prior to emulsion stabilization
2B	• GW-GM • GP-GM • GW-GC • GP-GC	• Polymer/bituminous emulsion and cement • Portland cement and fiber • Lime and fiber • Lime-cement-fly ash and fiber	• PI not to exceed 10 • PI not to exceed 30 • PI not less than 12 • PI not to exceed 25		• Well-graded material only • Material should contain at least 45% by weight of material passing No. 4 sieve • Soils near or above their optimum moisture content may require drying prior to emulsion stabilization
2C	• GM • GC • GM-GC	• Polymer/bituminous emulsion and cement • Portland cement and fiber • Lime and fiber • Lime-cement-fly ash and fiber	• PI not to exceed 10 • PI less than or equal to 20, plus 50% passing No. 200 sieve • PI not less than 12 • PI not to exceed 25	• Not to exceed 30% by weight	• Well-graded material only • Material should contain at least 45% by weight of material passing No. 4 sieve
3	• CH • CL • MH • ML • OH • OL • ML-CL	• Portland cement and fiber • Lime and fiber	• LL less than 40 and PI less than 20 • PI not less than 12		• Organic and strongly acid soils falling within this area are not susceptible to stabilization by conventional means.

[1]Monofilament polypropylene fiber (length and denier will vary depending on soil type).

Legend:

LL	liquid limit
No.	number
PI	plasticity index

Table 23-3. Suggested stabilizer amounts for expedient stabilization

Soil Type	*Fines Content*	*Plasticity Index*	*Stabilizer Type*[1]	*Percent Additives*
Sand 1A and 1B	• Low	• All	• Portland cement emulsion polymer fiber[2]	• 3-5 • 3-5 • 0.2-0.5

Table 23-3. Suggested stabilizer amounts for expedient stabilization (continued)

Soil Type	*Fines Content*	*Plasticity Index*	*Stabilizer Type*[1]	*Percent Additives*
Sand 1C	• High	• High • Low • Intermediate	• Lime and portland cement • Portland cement • Fiber[2]	• 3-5 and 3-5 • 5 • As above • 0.2-0.5
Gravel 2A and 2B	• Low	• All	• Portland cement • Emulsion polymer • Fiber[2]	• 3-5 • 3-5 • 0.2-0.5
Gravel 2C	• High	• High • Low • Intermediate	• Lime and portland cement • Portland cement • Fiber[2]	• 3-5 and 3-5 • 5 • As above • 0.2-0.5
Clays, Silts 3	• High • High • High	• High (greater than 20) • Low (less than 12) • Intermediate (12 to 20)	• Lime and portland cement • Portland cement • Lime or portland cement • Fiber[2]	• 5-7 and 5-7[3] • 7[3] • As above • 0.2-0.5

[1]All of these stabilizers require a polymer emulsion surface coating of 0.25-1 gallon per square yard (1:2 dilution with water) for weatherproofing. The reapplication of surface coating may be necessary, depending on the traffic and conditions.
[2]Monofilament polypropylene fiber used in combination with stated stabilizers—length and denier will vary depending on soil type. In general, the finer the soil, the more fiber surface area (length and shape) is needed to interact with the soil-stabilizer matrix.
[3]In some cases, intermediate and high-plasticity clays may require stabilizer amounts that are significantly higher than suggested.

23-19. The environmental conditions present at the stabilization site may need special consideration. In particular, the temperature will be the most critical factor that influences the curing time. When temperatures are below 45°F, curing times will be significantly decreased. In general, for every 20°F temperature decrease, curing time doubles. Thus, for 3 percent Type III cement, 1 day of cure time is needed at 75°F, 2 days would be needed at 55°F. The change in curing time may be offset by increasing the stabilizer concentration from 3 to 6 percent for a 1 day curing time at 55°F (13°C).

23-20. The amounts of stabilizers used vary with the particular soil type and the expected durability needed for the mission. In TM 5-822-14/AFJMAN 32-1019, stabilizer amounts are determined by specific tests and criteria. Given the constraints of a rapid deployment scenario, there is not enough time to perform mix design tests with various stabilizers and amounts to determine if those specific criteria are met. If time allows, it is always best to perform a strict mix design according to TM 5-822-14/AFJMAN 32-1019. However, the suggested stabilizer amounts given in table 23-3 should provide acceptable performance.

EQUIPMENT

23-21. 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

23-22. 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 oven-dried soil passing the No. 40 sieve equal to 20.0 grams. Air-dried soil may be used if an oven is not available. In either case, the soil should be dried until the weight is constant, indicating that additional water is not evaporating.
- **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 and lime thoroughly.
- **Step 6.** Add 100 milliliters of the distilled water to the soil-lime mixtures.
- **Step 7.** Shake the soil-lime-water mixture for at least 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.** Wait 1 hour, transfer part of the slurry, 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.

EXPEDIENT STABILIZATION

23-23. In many cases, the military engineer needs soil improvement quickly in the theater of operations with minimal logistics and stabilization material. In this case, the best stabilizer of choice is cement. However, common portland cement (Type I) is not a rapid-cure material and may take up to 7 days to provide enough strength to supporting large loads (such as cargo aircraft). It also provides only limited durability and generally fails by cracking and surface raveling, which may pose a danger to aircraft due to foreign object debris (FOD). Cement is also extremely heavy due to its high specific gravity, placing a further logistical burden on the military planner. Using lower amounts of cement than typically called for (TM 5-822-14/AFJMAN 32-1019) reduces the logistical burden at the expense of durability. For expedient and temporary structures, suggested amounts of cement for common soils are given in table 23-3. Many of these concerns about cement can be reduced by incorporating monofilament polypropylene fibers into the soil cement mix and protecting the soil cement surface by waterproofing. It has been found that using monofilament fibers between 19 and 51 millimeters can—

- Increase the load-bearing capacity so that large loads can be handled in 3 days (sometimes less, depending on temperature).
- Reduce FOD potential by bonding to the soil cement and creating a mat-like composite.
- Be used with high early strength cement (Type III) so that large loads (C-130 and C-17 aircraft) can be placed on the structure within 24 hours of construction.

23-24. Construction with fibers is simple and is conducted in similar fashion to cement stabilization. Simply calculate the correct spacing of the fiber bags to achieve the desired concentration of fiber in the same fashion as with cement. The use of a reclaimer/stabilizer for soil mixing is mandatory. Fiber concentrations of 0.1 to 0.5 percent based on the weight of soil have shown to be successful. For most soils, fiber concentrations above 0.5 percent have demonstrated decreased density compared to lower fiber contents, which may decrease durability. Stabilization depths of 150 millimeters are recommended and can be used for soils above CBR 8 to 10 percent. The stabilization of more than one lift is typically not practical in expedient situations. A balance between fiber length and constructability must be maintained. Longer fibers improve soil engineering characteristics to a larger degree, but are harder to construct with due to balling and tearing out of the soil during grading. It has been found that the grading of soil-fiber-cement is extremely difficult and should only be attempted if absolutely necessary and within an hour or two after compaction. If an area is recognized to need smoothing during compaction, stop compaction, grade it, and recompact it. Typically, the fiber balls in front of the grader blade and rips out of the soil, leaving large

scars and tears in the surface. A rule of thumb is that the surface started with is the surface ended with. In practice, this is best accomplished by preparing, compacting, and grading the surface to final elevation immediately prior to stabilization.

23-25. The measurement of density for fiber-stabilized materials can be accomplished in the laboratory but is not possible in the field. Nuclear density devices tend to give false readings due to the fiber, and sand cone measurements give large errors due the problems excavating the fiber-stabilized soil. If laboratory measurements of density can be determined, the highest fiber content that produces equal density to a non-fiber-stabilized specimen should be chosen.

23-26. If no laboratory data is available for optimum water content, the rules of thumb for cement stabilization are:

- For cement amounts less than 5 percent, use a water content 1 percent above the optimum for the unstabilized soil.
- For cement contents above 5 percent, use a water content 2 percent above the optimum.

23-27. Generally, it is best to compact cement-stabilized soil slightly on the wet-side of optimum to provide extra water for hydration of the cement.

23-28. For the low-stabilizer amounts, it is absolutely necessary to cap the surface of the soil with a waterproofing agent (such as polymer emulsion) designed for soil stabilization. This prevents water from entering the stabilized soil matrix and reducing the strength. It also helps prevent fiber pull out from the surface and provides dust control. The minimum rate of application is suggested at 0.25 gallons per square yard at a 1:3 dilution ratio of polymer emulsion: water, and it must be reapplied, as necessary, depending on traffic. For airfield structures, application rates as high as 1 gallon per square yard may be used if the soil will accept that application rate. Bituminous emulsions can also be used, but polymer emulsions are much more durable under traffic. Synthetic fluids used for dust control should not be applied to cement-stabilized surfaces because they have been found to reduce the strength and durability.

23-29. If logistics allow, stabilization according to TM 5-822-14/AFJMAN 32-1019 is recommended. However, if testing facilities are not available, the high dosage rates for the suggested amounts of stabilizers can be used if logistics allow and the structure may become a longer term use facility. Fibers can be used with any of the stabilizers. For any stabilized soil to be used as a pavement surface, it should be treated, as necessary, with polymer or bituminous emulsion to extend durability.

SOIL STABILIZATION IN FROST AREAS

23-30. While bitumens, 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, D.C. 20314 (for Army projects).
- Headquarters, Air Force Engineering and Services Center, Tyndall Air Force Base, FL 32401 (for Air Force projects).
- Headquarters, Naval Facilities Engineering Command, Alexandria, VA 22332 (for Navy projects).

LIMITATIONS

23-31. 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 with caution and only 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 should also 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

23-32. 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 1 month or for cement-stabilized soils when the soil temperature is less than 40°F and is not expected to increase for 1 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

23-33. Hot, dry weather is preferred for all types of bituminous stabilization. When ACs 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 compaction should be commenced immediately after lay down operations. Adequate compaction may not 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.

DRYING OF WET SOILS

23-34. The military engineer who must execute earthwork under wet conditions faces one of the perennial challenges of engineering. The proper compaction and engineering properties of soils highly depend on moisture content, yet nature seldom provides natural materials at the ideal moisture content. Experience soon teaches the field military engineer that attempting to subvert the fundamental laws that describe the effect of moisture content on construction and engineering properties of soils and aggregates leads only to frustration and failure. These wet soils may be due to temporary weather conditions (thawing of frozen soils, rainfall) to permanent situations (high water table). One may tackle the drying of such soils by conventional methods of spreading, discing, and the like, but the results are likely to be slower and more weather-dependent than military operations can tolerate.

23-35. When faced with the necessity of construction with soils that are far wetter than desired, the military engineer may opt for chemical drying to deal with isolated wet spots or, if necessary, more extensive areas of wet soils.

23-36. The chemicals of choice for drying are oxides of calcium and magnesium, generally referred to as quicklime. Depending on the source, they may be further defined as high-calcium quicklime, magnesium quicklime, or dolomitic quicklime. All are adequate for the drying task at hand. The oxides (CaO) hydrate with water and chemically bind water in the soil into a new form of hydrated lime ($CaO{\cdot}[OH]_2$). This reaction releases considerable heat (is exothermic) which, in turn, evaporates more water and further reduces the moisture content of the soil. Because this reaction is very avid and exothermic, it is effective for drying soils down to freezing temperatures.

23-37. Lime is a generic term used to cover a variety of materials. Only quicklime is effective for drying soils. Hydrated lime is already combined with water and will not draw moisture into new chemical forms. It will have certain chemical reactions (cation exchange) with some clay minerals that will make them less plastic and easier to dry by conventional methods, but this is much different from the avid drying of quicklime that is not related to soil type. Calcium carbonate ($CaCO_3$) is sometimes marketed as agricultural lime and has no value for engineering stabilization or drying uses.

23-38. Any product that contains free quicklime (CaO) can be used as a drying agent. The more CaO it contains, the more effective it will be as a drying agent. Portland cement, fly ash (particularly high-calcium, Class C ashes), and kiln dust are examples of common industrial materials with significant CaO content that can be used for soil drying. Their effectiveness for drying directly depends on their free CaO content.

PICK-AND-CLICK TESTS

23-39. Specimens covering a wide range of cement contents are molded at optimum moisture and maximum density. After 36 hours or more of hardening while kept moist and after a 3-hour soaking period, the specimens are inspected by picking with a pointed instrument (a dull ice pick, bayonet) and by sharply clicking each specimen against a hard object (concrete, a 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.

23-40. 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

23-41. After determining the maximum density and OMC, mold the specimens for the wet-dry and freeze-thaw tests. The test procedures are also given in TM 5-822-14/AFJMAN 32-1019.

23-42. 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 each 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 (or in an atmosphere of high humidity) for 7 days to permit cement hydration before testing.

WET-DRY TEST PROCEDURES

23-43. After the 7-day curing period, submerge the specimens in tap water at room temperature for 5 hours and 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 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-dried weights.

FREEZE-THAW TEST PROCEDURES

23-44. 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 no 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 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 brushing 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 the oven-dried weights.

CALCULATIONS AND CRITERIA

23-45. 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

23-46. The final oven-dried weight of the specimen includes some water used for cement hydration that cannot be at 230°F. The average amount of water retained in the specimen is based on the type of soil—gravels, plus 1 1/2; sand, plus 2 1/2 percent; silt, plus 3 percent; and clays, plus 3 1/2 percent. This correction is computed by using the following formula:

$$corrected\ oven-dried\ weight = \left(\frac{measured\ oven-dried\ specimen\ weight}{percent\ of\ water\ retained}, 100\right) 100$$

Example. A sample composed mostly of sand weighs 3.77 pounds at the end of the test. The water-of-hydration correction is 2.5 percent.

$$corrected\ oven-dried\ weight = \left(\frac{3.77}{2.5}, 100\right) 100$$

Soil-Cement Loss

23-47. The soil-cement loss can now be calculated as a percentage of the original dry weight or by using the following formula:

$$soil\ cement\ loss = \left(\frac{original\ oven\text{-}dried\ weight - final\ corrected\ oven\text{-}dried\ weight}{original\ oven\text{-}dried\ weight}\right) 100$$

Example. A sample of soil has an original weight of 3.99 pounds. This is found by using the following formula:

$$soil\ cement\ loss = \left(\frac{3.99 - 3.68}{original\ oven\text{-}dried\ weight}\right) 100$$

This value would be reported to the nearest whole number or as 8 percent.

Weight Loss Criteria

23-48. 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

23-49. The strength of soil-cement specimens tested in compression at various ages should increase with age and with increases in cement. The range of cement contents should produce results meeting the above requirements. A sample that has an unconfined compression strength of about 300 pounds per square inch after curing 7 days and shows increasing strength with age can be considered adequately stabilized. Fibers tend to reduce strength slightly, compared to a specimens with no fibers.

Cement Weight-to-Volume Conversion

23-50. The required cement content by weight can be converted to the equivalent cement content by volume for control during construction since this may be the easier quantity to use in the field. The following formula illustrates the calculation:

$$volume\ of\ cement\ (percent) = \frac{D - \left(\frac{D}{C}\right)}{94}100$$

where—

D = oven-dried density of soil-cement (pounds per cubic foot)

$C = \frac{100 + \%\ cement\ by\ weight}{100}$

94 = weight of 1 cubic foot of cement

23-51. The nomograph in figure 23-1, page 23-4, 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

23-52. 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 it is economical and the results will be close enough to be on the safe side. If time permits, the procedures for the freeze-thaw test are followed for greater design economy.

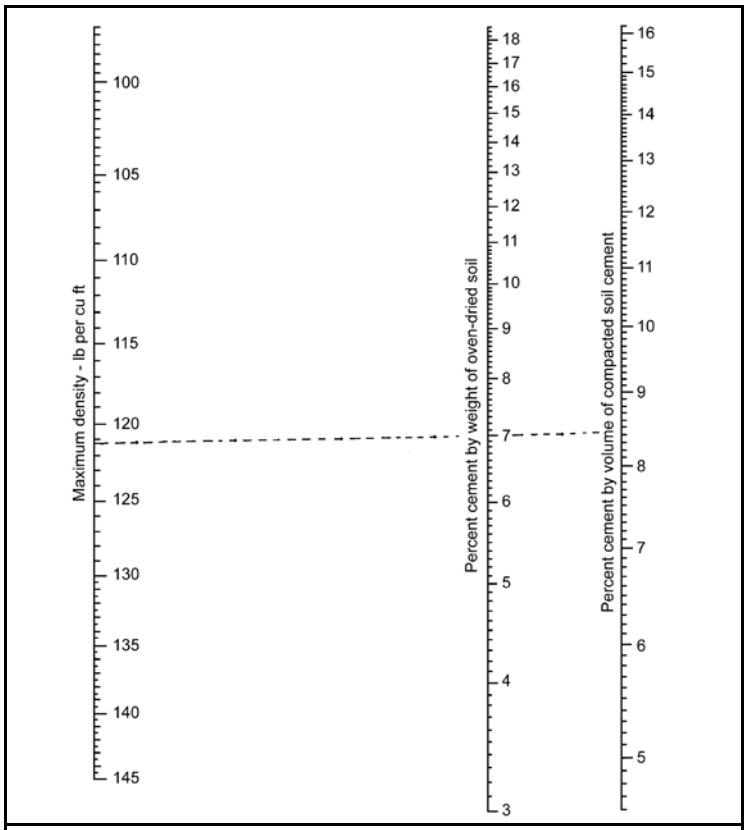
23-53. The two procedures used are for—

- Soils with no material retained on the No. 4 sieve.
- Soils with material retained on the No. 4 sieve.

23-54. These 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 contain appreciable organic impurities together with miscellaneous granular materials (cinders, caliche, chat, chart, marl, red dog, scoria, shale, slag) should be tested using the full procedures for sandy soils versus the modified methods. When coarse-grained or sandy soils (usually GW, GP, GM, SW, or SM) are encountered, they may be classified for testing purposes using 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.

23-55. Perform the following steps to modify 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 (figure 23-2).



Legend:

lb	pound
cu ft	cubic foot

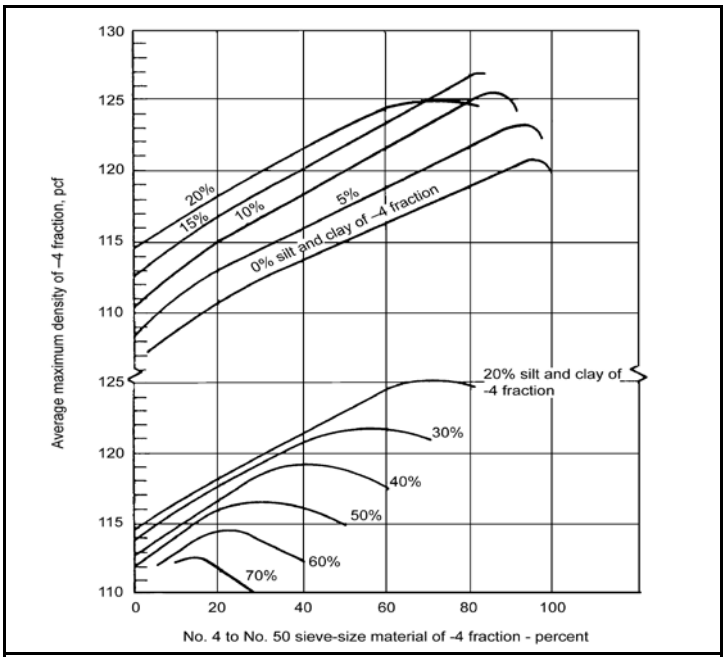
Figure 23-2. Relation of cement content by weight to cement content by volume

- **Step 5.** Perform compressive-strength tests to verify the cement requirements.

SOILS WITH NO MATERIAL RETAINED ON THE NUMBER 4 SIEVE

23-56. Perform the following steps for soils with no material retained on the No. 4 sieve:

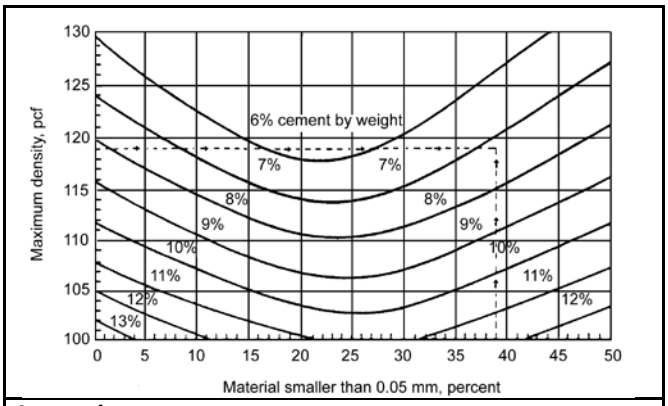
- **Step 1.** Determine the maximum density and OMC for a mixture of soil and cement. (Figure 23-2 will give an estimated density. This value and the percentage of material smaller than 0.05 millimeter are used with figure 23-3, page 23-14, to determine indicated cement content.)



Legend:
No. number
pcf pound per cubic foot

Figure 23-3. Average maximum densities of the -4 fraction of soil-cement mixtures

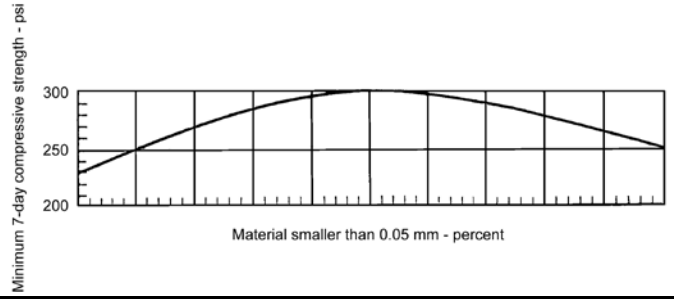
- **Step 2.** Use the maximum density value and figure 23-4 to determine the cement required.



Legend:
mm millimeter
pcf pound per cubic foot

Figure 23-4. Indicated cement contents of soil-cement mixtures not containing material retained on the No. 4 sieve

- **Step 3.** Mold three compressive-strength specimens at the maximum density and OMC.
- **Step 4.** Moist-cure the specimens for 7 days, and test them for strength.
- **Step 5.** Plot the value of the average compressive strength on figure 23-5. If this plot is above the curve, the CF is probably too low and needs adjustment. Prepare two new test specimens: one at the cement content as computed above and the second with 2 percent higher cement content. Perform the full freeze-thaw test on these two specimens.



Legend:

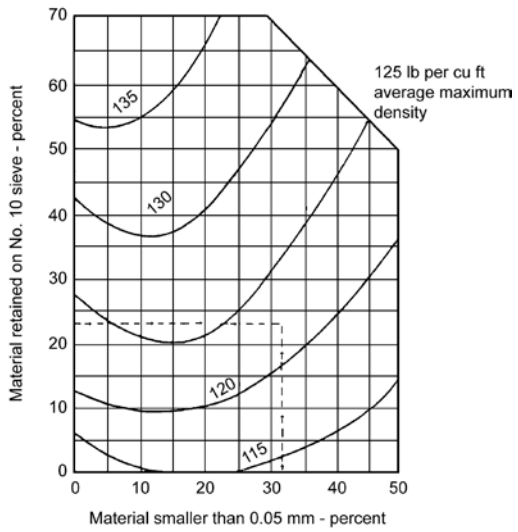
mm	millimeter
psi	pound per square inch

Figure 23-5. Minimum 7-day compressive strengths required for soil-cement not containing material retained on the No. 4 sieve

SOILS WITH MATERIAL RETAINED ON THE NUMBER 4 SIEVE

23-57. 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 23-5 to estimate the 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 (figure 23-6). The 45 percent maximum retained on the No. 4 sieve still applies. Also, replace material larger than 1/4 inch with an equivalent weight of material passing the 1/4-inch sieve and retained on the No. 4 sieve.



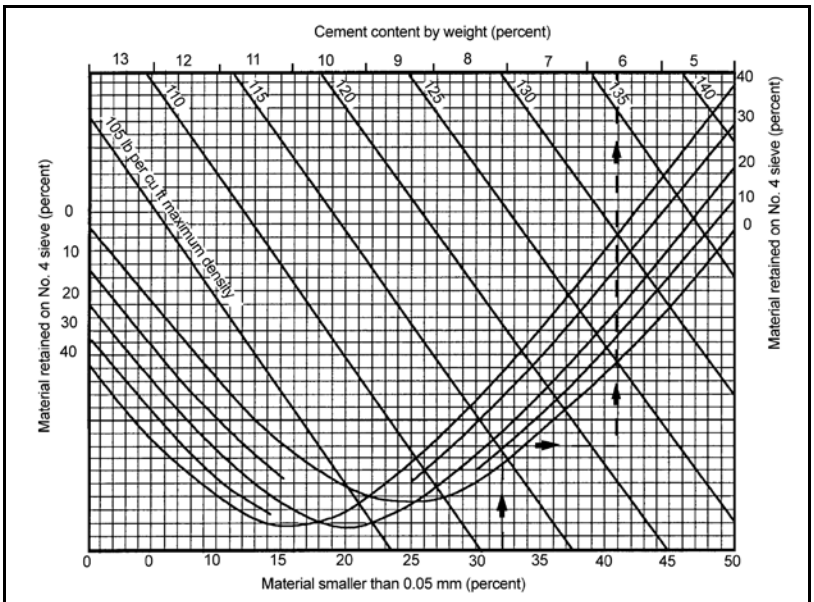
Legend:

cu ft	cubic foot
lb	pound
mm	milimeter
No.	number

Figure 23-6. 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 23-6.

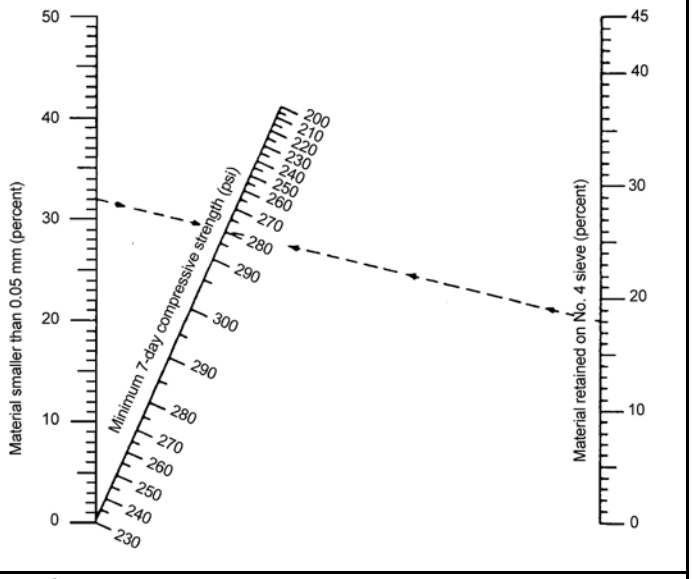
- **Step 3.** Mold-test specimens at the maximum density and OMC.
- **Step 4.** Moist-cure the specimen for 7 days and test them for compressive strength and average.
- **Step 5.** Use figure 23-7 to determine the allowable compressive strength for the soil-cement mixture. Connect the points on the right- 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. (See figure 23-8.)



Legend:

cu ft	cubic foot
lb	pound
mm	millimeter
No.	number

Figure 23-7. Indicated cement contents of soil-cement mixtures containing material retained on the No. 4 sieve



Legend:	
mm	millimeter
No.	number
psi	pound per square inch

Figure 23-8. Minimum 7-day compressive strengths required for soil-cement mixtures containing material retained on the No. 4 sieve

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Appendix A

Metric Conversion Chart

Table A-1 is a metric conversion chart. A complete listing of preferred metric units for general use is contained in Federal Standard 376B.

Table A-1. Metric conversion chart

Metric to United States Unit			*United States to Metric Unit*		
Multiply	***By***	***To Obtain***	***Multiply***	***By***	***To Obtain***
Length					
Centimeters	0.0394	Inches	Inches	2.5400	Centimeters
Kilometers	0.6210	Miles (statute)	Miles (statute)	1.5609	Kilometers
Kilometers	0.5400	Miles (nautical)	Miles (nautical)	1.8530	Kilometers
Meters	3.2800	Feet	Feet	0.0305	Meters
Meters	1.0940	Yards	Yards	0.9144	Meters
Millimeters	0.0390	Inches	Inches	25.4000	Millimeters
Area					
Square centimeters	0.1550	Square inches	Square inches	6.4500	Square centimeters
Square meters	10.7600	Square feet	Square feet	0.0929	Square meters
Square meters	1.1960	Square yards	Square yards	0.8360	Square meters
Volume					
Cubic centimeters	0.6100	Cubic inches	Cubic inches	16.3900	Cubic centimeters
Cubic meters	35.300	Cubic feet	Cubic feet	0.0283	Cubic meters
Cubic meters	1.3080	Cubic yards	Cubic yards	0.7650	Cubic meters
Milliliters	0.0338	Ounces	Ounces	29.6000	Milliliters
Liters	1.0507	Quarts	Quarts	0.9460	Liters
Liters	0.2640	Gallons	Gallons	3.7900	Liters
Weight					
Grams	0.0353	Ounces	Ounces	28.4000	Grams
Kilograms	2.2000	Pounds	Pounds	0.4540	Kilograms
Metric tons	1.1020	Short tons	Short tons	0.9070	Metric tons
Metric tons	0.9840	Long tons	Long tons	1.0160	Metric tons
Pressure					
Kilonewton	224.8089	Pounds	Pounds	0.0044	Kilonewton
Megapascals	145.0377	Pounds per square inch	Pounds per square inch	0.0068	Megapascal
Newtons	0.2248	Pounds per minute	Pounds per minute	4.4482	Newton
Temperature					
Celsius	Add 17.8 and multiply by 1.8	Fahrenheit	Fahrenheit	Subtract 32 and multiply by 0.5556	Celsius

Legend:
U.S. United States

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Appendix B

Unified Soil Classification System

The adoption of the principles of soil mechanics has inspired numerous attempts to devise a simple classification system that identifies the properties of specific soils. Many classifications have been established based on texture, plasticity, strength, and other soil characteristics. Some classification systems have gained acceptance, but seldom has a system provided the entire amount of soil information that is needed by engineers. Almost every engineer who practices soil mechanics is required to modify the classification system that is being used by incorporating individual judgment and personal experience. When designs and plans are reviewed by persons who are uninvolved with the task, a common system of soil classification is necessary to convey the behavior and characteristics of the materials that are related to the project. The USCS is based on identifying soils according to their textural and plasticity qualities and on their grouping with respect to behavior. Soils rarely exist separately as sand, gravel, or any other single component. They are usually mixtures that contain varying proportions of particles of different sizes. Each component contributes its characteristics to the soil mixture. The USCS is based on the soil characteristics that indicate how the soil will perform as an engineering construction material. The properties that form the basis of soil identification include—

- The percentage of gravel, sand, and fines that pass the No. 200 sieve.
- The shape of the grain size distribution curve.
- The plasticity and compressibility characteristics. (In the USCS, the soil is given a descriptive name and a letter symbol that indicates the principal characteristics.)

This appendix describes various soil groups and discusses the methods of identification so that a uniform classification procedure can be followed. The placement of soils into their respective groups is accomplished by visual examinations and laboratory tests. The USCS may not be entirely adequate in all cases. It is a flexible system that should not be blindly followed, nor should it be regarded as being completely rigid.

SOIL COMPONENT DEFINITIONS

B-1. The limiting boundaries between the various size ranges have been arbitrarily set at the U.S. standard sieve sizes that are listed. In the finest soil component (below the No. 200 sieve), silt and clay are used respectively to distinguish materials that exhibit lower plasticity from those with higher plasticity. The -200 sieve material is silt if the LL and PI plot below the A line on the plasticity chart. If the LL and PI plot above the A line on the chart, the material is clay. (LL and PL tests are based on -40 sieve fraction of a soil.) The preceding definition is applicable for inorganic silts and clays and for organic silts, but it is not valid for organic clays (organic clays plot below the A line). The names of basic soil components can be used as nouns or adjectives when they are used to describe or classify soils.

B-2. Before soils can be properly classified in a system, it is necessary to establish a basic terminology for the various soil components and to define the terms that are used. In the USCS, cobbles, gravel, sand, and fines (silt or clay) are used to designate the size ranges of soil particles. Gravel and sand ranges are further subdivided into groups (table B-1, page B-2).

Table B-1. Soil particle size ranges

Component	*Size Range*
Cobbles	Above 3 in
Gravel • Coarse • Fine	3 in to No. 4 sieve 3 in to ¾-inch sieve 3/4 in to No. 4 sieve
Sand • Coarse • Medium • Fine	No. 4 to No. 200 sieve No. 4 to No. 10 sieve No. 10 to No. 40 sieve No. 40 to No. 200 sieve
Fines (clay or silt)	Below No. 200 sieve (no minimum size)
Legend: in inch No. number	

PROCEDURES

B-3. Classification system procedures apply to the identification of soils, regardless of the intended engineering uses. Table B-2 assists in identifying the symbols and soil descriptions within this system. Columns 1 through 5 identify the three major divisions of the classification system and the group symbols that distinguish the individual soil types. The names of the typical and representative soil types that are found in each group are shown in Column 6.

B-4. Figure B-1, page B-5, demonstrates the schematic method of classifying soils from the results of laboratory tests.

SOIL GROUPS AND SYMBOLS

B-5. Soils are primarily identified as coarse-grained, fine-grained, and highly organic. On a textural basis, coarse-grained soils 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 usually easily identified by visual examination due to the presence of fibrous matter (wood, roots, leaves.) Coarse-grained soils are subdivided into gravel and gravelly soils and sand and sandy soils. Fine-grained soils are subdivided on their LL and plasticity properties. The symbol L is used for soils with LLs of 50 and less, and the symbol H is used for soils with LLs in excess of 50. Peat and other highly organic soils are designated by the symbol Pt and are not subdivided. There is typically 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.

Table B-2. Characteristics of soil groups pertaining to embankments and foundations

Major Divisions (1)	(2)	Letter (3)	Symbols: Hatching (4)	Symbols: 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 > 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	

Table B-2. Characteristics of soil groups pertaining to embankments and foundations (continued)

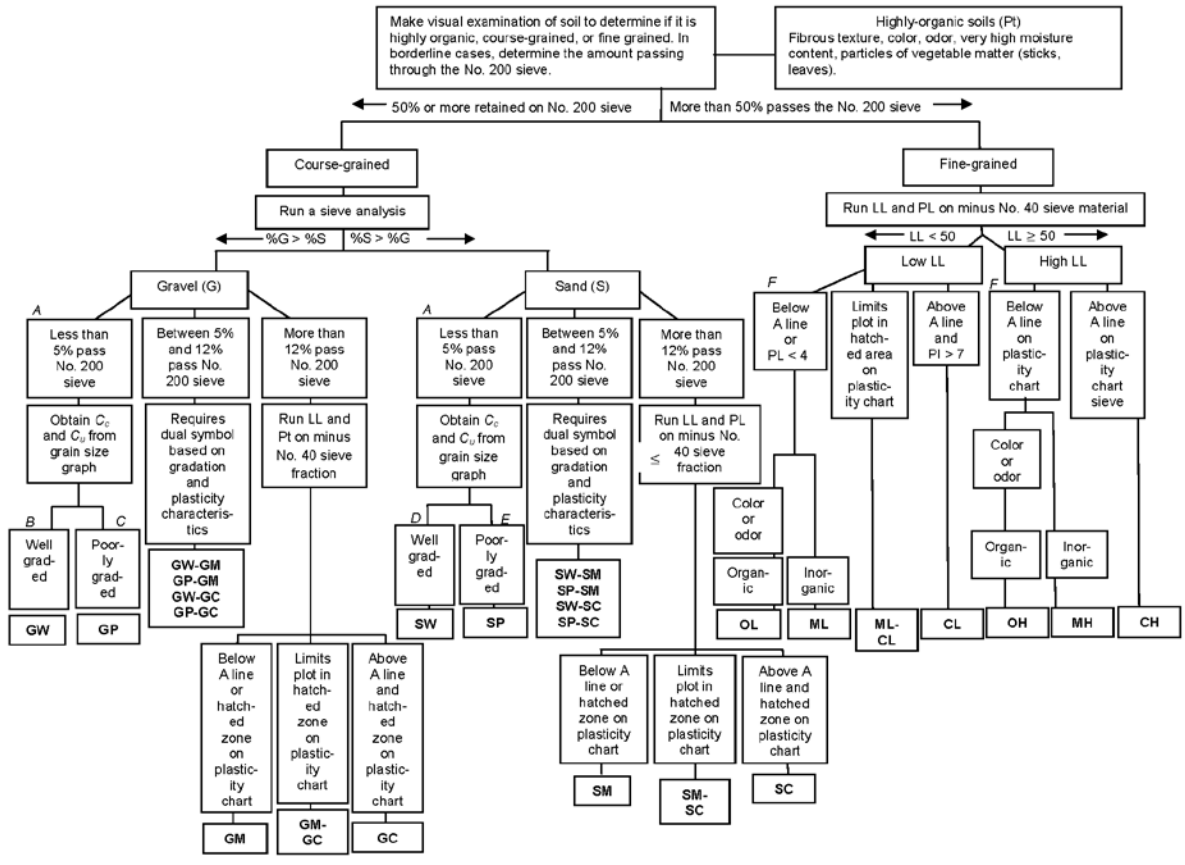
Letter (3) (continued)	*Compaction Characteristics (9)*	*Maximum Dry Unit Weight Standard Proctor* (pounds per cubic foot) (10)*	*Value for Foundations (11)*	*Requirements for Seepage Control (12)*
GW	Good; tractor, rubber-tired, or steel-wheeled roller	125 -135	Good bearing value	Positive cutoff
GP	Good; tractor, rubber-tired, or steel-wheeled roller	115 -125	Good bearing value	Positive cutoff
GM	Good, with close control; rubber-tired or sheepsfoot roller	120 -135	Good bearing value	Toe trench to none
GC	Fair; rubber-tired or sheepsfoot roller	115 -130	Good bearing value	None
SW	Good; tractor	110 -130	Good bearing value	Upstream blanket and toe drainage or wells
SP	Good; tractor	100 -120	Good to poor bearing value, depending on density	Upstream blanket and toe drainage or wells
SM	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
SC	Fair; sheepsfoot or rubber-tired roller	105 -125	Good to poor bearing value	None
ML	Good to poor, but close control is essential; rubber-tired or sheepsfoot roller	95 -120	Very poor, susceptible to liquefaction	Toe trench to none
CL	Fair to poor; sheepsfoot or rubber-tired roller	95 -120	Good to poor bearing value	None
OL	Fair to poor. Sheepsfoot roller	80 -100	Fair to poor bearing value; may have excessive settlements	None
MH	Poor to very poor; sheepsfoot roller	70 - 95	Poor bearing value	None
CH	Fair to poor sheepsfoot roller	75 -105	Fair to poor bearing value	None
OH	Poor to very poor; sheepsfoot roller	65 - 100	Very poor bearing value	None
Pt	Compaction not practical		Remove from foundations	

Table B-2. Characteristics of soil groups pertaining to embankments and foundations (continued)

Notes:
1. The 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 product 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 compact soil at OMC when using the standard proctor test (ASTM D1557-91).

Legend:

ASTM	American Society of Testing and Materials
cm	centimeter
LL	liquid limit
OMC	optimum moisture content
sec	second



A	If lines 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 cannot be determined by color or odor, an 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 over-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

Legend:	
LL	liquid limit
No.	number
PL	plastic limit

Figure B-1. USCS procedures (continued)

COARSE-GRAINED SOILS

B-6. 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. In the following paragraphs, GW, GP, SW, and SP soils are defined as having less than 5 percent passing the No. 200 sieve. Soils that have between 5 and 12 percent passing the No. 200 sieve are considered borderline. These soils are subdivided as—

- **GW and SW.** GW and SW groups comprise well-graded gravelly and sandy soils that have little or no nonplastic fines (less than 5 percent passing the No. 200 sieve). If the fines exhibit plasticity, this information should be evaluated and the soil classified and discussed in laboratory identification. In areas subject to frost action, the material should not contain more than 3 percent of soil grains smaller than 0.02 millimeters.
- **GP and SP.** Poorly graded gravels and sands that contain 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 under laboratory identification.
- **GM and SM.** GM and SM groups usually 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 materials (including well-graded and poorly graded materials) is not considered significant. Some of the sands and gravels in this group contain a binder that is composed of natural cementing agents which are proportioned so that the mixture shows negligible swelling or shrinkage. The dry strength of the 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 that have little or no plasticity, and the mixture will exhibit no dry strength.
- **GC and SC.** GC and SC groups comprise gravelly or sandy soils with fines (more than 12 percent passing the No. 200 sieve) which have low or high plasticity. The PI and LL of soils should plot above the A line on the plasticity chart. The gradation of the materials (including well-graded and poorly graded materials) is not considered significant. The plasticity of the binder fraction has more influence on the soil behavior than the variation in gradation. The fine fraction is typically composed of clays.
- **Borderline soils.** Borderline soils may be classified as belonging to G and S groups. They are divided into the following secondary groups:
  - 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).

Fine-Grained Soils

B-7. Fine-grained soils are subdivided into groups based on whether they have a low or high LL. These two groups are subdivided as—

- **ML and MH.** The symbol *M* is used in ML and MH groups 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. Soils in the ML and MH groups are sandy silts, clayey silts, inorganic silts with relatively low plasticity, loess soils, and rock flours. Micaceous and diatomaceous soils usually fall within the MH group, but they may extend into the ML group when their LL is less than 50. The same is true for certain types of kaolin clays and for some elite clays that have relatively low plasticity.
- **CL and CH.** The symbol *C* in the CL and CH groups stands for clay. *L* and *H* denote 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. Medium- and high-plasticity clays are classified as CH. CH clays include fat clays, gumbo clays, certain volcanic clays, and bentonite. Glacial clays of the northern United States cover a wide portion of the CL and CH groups.
- **OL and OH.** Soils in the OL and OH groups are characterized by the presence of organic matter. 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.** Highly organic soils are usually very compressible and have undesirable construction characteristics. They are classified into one group and are designated by the symbol *Pt*. Peat, humus, and swamp soils with a highly organic texture are typical soils of this group. Particles of leaves, grass, branches, or other fibrous vegetable matter are common components of the soils.

SOIL GROUP IDENTIFICATION

B-8. The USCS is arranged so that most soils can be classified into at least the three primary groups of coarse-grained, fine-grained, and highly organic by 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 can be made through laboratory testing. However, in many instances, a tentative classification determined in the field is of great benefit and may be the only identification necessary, depending on the purposes for which the soils are to be used. The field identification and individual laboratory test methods are explained in great detail in chapter 2. 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 using laboratory tests. Conversely, field methods are practical for preliminary laboratory identification and may be used to group soils so that only a minimum number of laboratory tests are required.

LABORATORY IDENTIFICATION

B-9. Soils identification in the laboratory is performed by determining the gradation and plasticity characteristics of the materials. The gradation is determined by a 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 LL and PL tests on the soil fraction that is finer than the No. 40 sieve. Laboratory test procedures for the LL and PL determination can be found in chapter 14.

MAJOR SOIL GROUPS

B-10. In the laboratory identification procedures that are shown in figure B-1, 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; however, in some borderline cases (those 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 being coarse-grained, and more than 50 percent identifies the soil as being fine-grained. The percentage limit of 50 has

been arbitrarily selected for convenience in identification, because a numerical difference of 1 or 2 percent will produce no significant change in the soil behavior. After the major group is established, the identification procedure is continued according to figure B-1, page B-5.

COARSE-GRAINED SOILS

B-11. A complete sieve analysis must be executed on coarse-grained soils, and a gradation curve must be plotted on a grain size chart. For some soils that contain 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 that are smaller than the No. 200 sieve size. Preliminary identification is made by determining the percentage of material in the gravel and sand 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.* 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 that is passing the No. 200 sieve. The subgroups are the same for gravels and sands; therefore, are discussed jointly as follows:

- **GW, SW, GP, and SP.** These groups comprise nonplastic soils that contain less than 5 percent passing the No. 200 sieve and in which the fine fraction does not interfere with the free-draining properties of the soil. If the above criteria are met, an examination is made of the shape of the grain size curve. Well-graded materials are classified as GW or SW; poorly graded materials are classified as GP or SP. The gradation curve and curve data of a soil should meet the following qualifications to be classed and graded:
  - Grain size distributions of well-graded materials plot as smooth and regular concave curves, with no sizes lacking and no excess of material in any size range.
  - The coefficient of the uniformity of well-graded gravels is greater than 4, and the coefficient of the uniformity of well-graded sands is greater than 6. The coefficient of the uniformity is determined by dividing the grain size diameter passing at 60 percent (D_{60}) by the grain size diameter passing at 10 percent (D_{10}).
  - The coefficient of curvature must be between 1 and 3. The coefficient of curvature is determined by the following formula:

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

Where—

C_c = *coefficient of curvature*

D_{30} = *grain diameter at 30 percent passing by weight*

D_{60} = *grain diameter at 60 percent passing by weight*

D_{10} = *grain diameter at 10 percent passing by weight*

Note. The coefficient of curvature ensures that the grading curve will have a concave curvature that is within relatively narrow limits for a given grain diameter at 60 percent passing by weight and a grain diameter at 10 percent passing by weight combination. Gradations that do not meet this criteria are classed as poorly graded. Poorly graded soils (GP and SP) are those that have nearly straightline gradations, convex gradations, nearly vertical gradations, and hump gradations which are typical of skip-graded materials.

- **GM, SM, GC and SC.** Soils in the GM, SM, GC, and SC groups are composed of materials that have more than a 12 percent fraction passing the No. 200 sieve. They may or may not exhibit plasticity. For identification, LL and PL tests are required on fractions that are finer than the No. 40 sieve. These tests should be run on representative samples of moist material. Drying affects the limits values and is explained in the discussion of fine-grained soils below. Materials in which the LL and PI plot below the A line on the plasticity chart 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. 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 that contain between 5 and 12 percent material passing the No. 200 sieve are classed as borderline and carry a dual symbol (such as GW-GM). Similarly, coarse-grained soils that have 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

B-12. When the identity of a fine-grained soil has been established, further identification is accomplished by the LL and PL tests in conjunction with the plasticity chart. The plasticity chart is a plot of LL versus PI on which a diagonal line (A line) and a vertical line are imposed at a LL of 50. The A line is defined by the following formula:

$$PI = 0.73\ (LL - 20)$$

B-13. An A line above an LL of about 29 represents an important empirical boundary between typical inorganic clays that are generally located above the line and plastic soils that contain organic colloids or inorganic silty soils. The vertical line at an LL of 50 separates silts and clays of low LL from those of high LL. 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 soils. The separation between CL and OL or ML soils 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 have a PI above 7; OL or ML soils have a PI below 4. Soils that plot 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 proven that compressibility is about proportional to the LL and that soils which have the same LL possess about equal compressibility, assuming that other factors are essentially the same. When comparing the physical characteristics of soils that have the same LL, increasing the PI will increase the cohesive characteristics and decrease the permeability. Plots of the results of limits tests on various samples from the same fine-grained deposit show that the points lie on a straight line or in a narrow band that is almost parallel to the A line. The following bullets contain more information on soil tests:

- **ML, CL, and OL.** A soil that has an LL of less than 50 falls into the low LL group. A plot of the LL and PI on the plasticity chart shows whether the soil falls above or below the A line and cross-hatched zone. Soils that plot above the A line and cross-hatched zone are classed as CL and are usually typical, inorganic clays. Soils that plot below the A line or cross-hatched zone are inorganic silts, very fine sandy silts, organic silts, or organic silt-clays of low plasticity. Because 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 that is usually identified by color and odor. A comparison may be made between the LL and PL of a moist sample and of one that has been oven-dried. An organic soil will show a radical drop in plasticity after being oven- or air-dried. An inorganic soil typically shows a change in the limits values of only 1 or 2 percent. This may be an increase or a decrease; therefore, the classification should be based on the plot of limits values that are determined before drying. Soils that contain organic matter usually have lower specific gravities and may have higher water contents than inorganic soils. These properties may assist in identifying organic soils. In special cases, the organic content may be determined by chemical methods.
- **MH, CH, and OH.** Soils with an LL that is greater than 50 are classed with the symbol *H*. To identify these 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. Inorganic materials are classed as MH, and organic materials are classed as OH.
- **Highly organic soils.** Highly organic soils are usually identified according to color, texture, and odor. Moisture determinations usually reveal a natural water content of several hundred percent, which is far in excess of that found most soils. Specific gravities of solids in these soils may be quite low. Some peaty soils can be remolded and tested for the LLs and PLs. These soils usually have an LL of several hundred percent and fall well below the A line on the plasticity chart.

Borderline Classifications

B-14. Soils will be encountered that fall close to the boundaries which are 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 part of the system. The accepted rule in classifying borderline soils is to use a double symbol. It is possible for a soil to fall into more than one borderline zone. If the appropriate symbols are used for each classification, the result should be a multiple designation that consists of three or more symbols. This approach is unnecessarily complicated; therefore, it is best to use a double symbol that represents the probable behavior of the soil. The symbols that represent the poorer of the possible groupings should be used.

Coarse-Grained Soils

B-15. In previous discussions, coarse-grained soils were classified as GW, GP, SW, and SP if they contained less than 5 percent of material passing the No. 200 sieve. Similarly, soils were classified as GM, GC, SM, and SC 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; the soils that fall within it are assigned a double symbol, depending on the gradation characteristics of the coarse fraction and the plasticity characteristics of the -40 sieve fraction. For example, a well-graded sandy soil with 8 percent passing the No. 200 sieve, an LL of 28, and a PI of 9 would be designated as SM-SC. Another type of borderline classification occurs for soils that contain appreciable amounts of fines (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 that plot in the same region as discussed below.

Fine-Grained Soils

B-16. Discussion has been presented of a zone on the plasticity chart below an LL of about 29 and a PI of 4 to 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 -40 sieve fraction of a coarse-grained soil (GM, GC, SM, and SC) plots within the cross-hatched zone on the plasticity chart, a double symbol (such as ML-CL) is used.

B-17. In the descriptive name of the soil type as indicated in table B-2, page B-3, 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

B-18. 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 that 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 properly understand the subdivision.

Descriptive Soil Classification

B-19. 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 a name rather than an impersonal classification symbol. This results, primarily, from the fact that people are accustomed to talking in terms of gravels, sands, silts, and clays and find 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 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

B-20. Column 6, table B-2, page B-3, lists typical names given to the soil types that are usually found within the various classification groups. By following 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 (marls, calyces, coral, 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

B-21. Records of field explorations in the form of boring logs can be of great benefit to engineer, if adequate information. In addition to the group symbol and the name of the soil, the general characteristics of the soils (plasticity, strength, moisture) provide information that is 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, table B-2, page B-3. 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-3).

Table B-3. Terzaghi classification

Unconfined Compressive Strength (Tons/Sq Ft)	*Consistency*
Less than 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
Greater than 4.00	Hard
Legend: sq ft square foot	

B-22. The following are examples of descriptive classifications:

- Uniform, fine, clean sand with rounded grains.
- Well-graded, gravelly, silty sand; angular chert gravel, 1/2-inch maximum size; silty binder with low plasticity, well-compacted and moist.
- Light brown, fine, sandy silt; very low plasticity; saturated and soft in the undisturbed state.

- Dark gray, fat clay; stiff in the undisturbed state; soft and sticky when remolded.

EMBANKMENTS AND FOUNDATIONS

B-23. The soil properties that are of concern for embankments or foundations are 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.

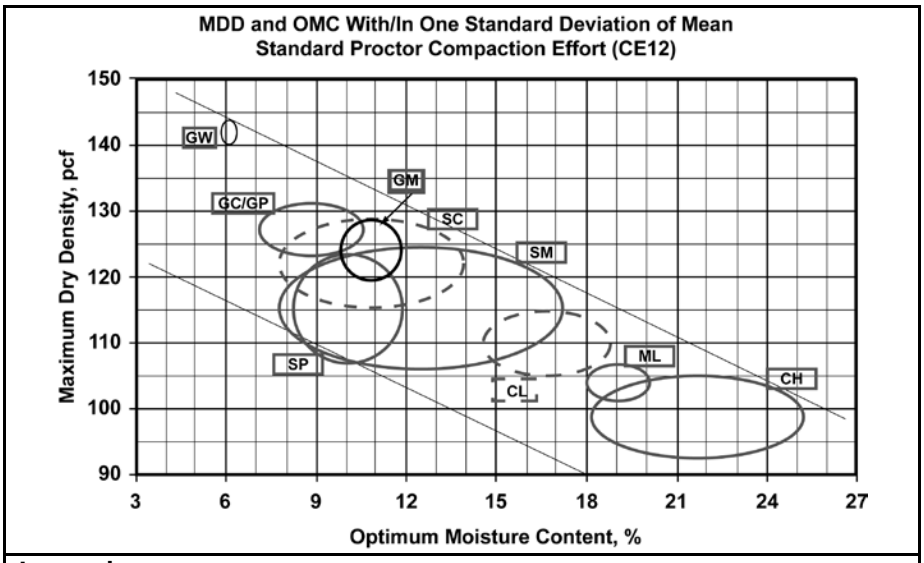
B-24. Keep in mind that only generalized characteristics of the soil groups are included in this appendix, and they should be used primarily as a guide and not as the complete answer to a problem. For example, it is possible to design and construct an earth embankment of almost any type of soil and on practically any foundation. However, when a choice of materials is possible, certain 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 and 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.

SOIL CLASSIFICATION SHEET FEATURES

B-25. General characteristics of the soil groups that are pertinent to embankments and foundations are presented in table B-2, page B-5.

B-26. Figure B-2 provides a statistical breakdown of MDD and OMC for the principal soil classifications for standard proctor effort. Each bubble represents one standard deviation variation of MDD and OMC from a worldwide database of soils. This can assist in estimating construction properties for soils beyond the ranges given in table B-2. This is a lower compaction effort that is typically used in embankment and foundation construction and not in highway and airfield construction. For critical structures that are subject to high loading, the modified proctor effort (ASTM D1557) is recommended (figure B-3).

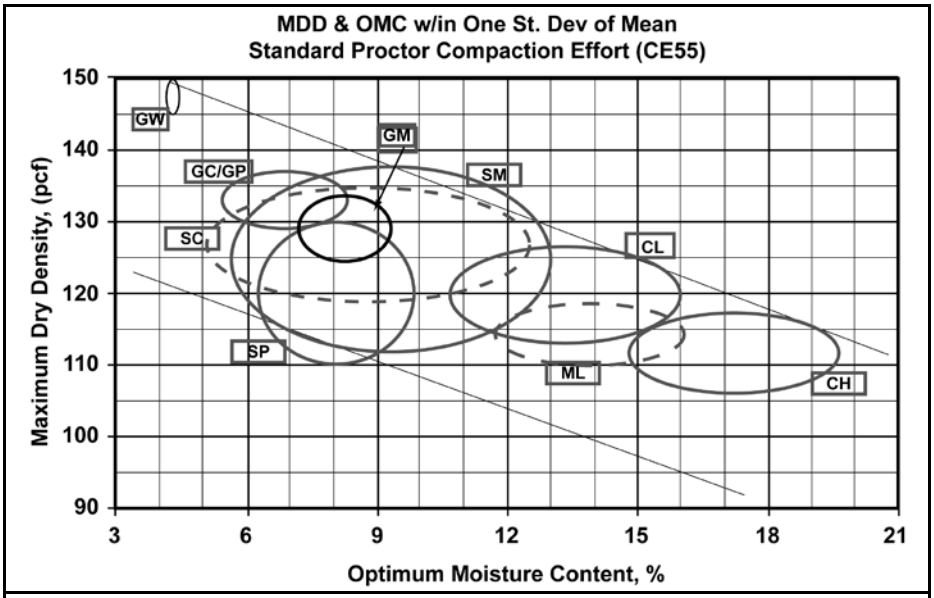
Note. Standard proctor is a reduced compaction effort from the modified effort that was discussed in chapter 17. Design values are intended for long-term structures. For contingency design, unsoaked CBR design may be appropriate, depending on the structure design life and loading restrictions in inclement weather.



Legend:

CE	compaction effort
MDD	maximum dry density
OMC	optimum moisture content
pcf	pound per cubic foot

Figure B-2. MDD and OMC estimates for standard proctor compaction effort



Legend:

CE	compaction effort
dev	deviation
MDD	maximum dry density
OMC	optimum moisture content
pcf	pound per cubic foot
st	standard
w/	with

Figure B-3. MDD and OMC estimates for modified proctor compaction effort

Suitability of Soils for Embankments

B-27. There are three major factors that influence the suitability of soils for use in embankments: permeability, strength, and ease of compaction. Gravelly and sandy soils with little or no fines (GW, GP, SW, and SP) are stable, pervious, and able to attain good compaction with crawler tractors and rubber-tired rollers. Poorly graded materials may not be as desirable as those that 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 should have flatter embankment slopes than SW soils. Gravels and sands with fines (GM, GC, SM, and SC) have variable characteristics that depend on the nature of the fine fraction and the gradation of the sample. These materials are often sufficiently impervious and stable and can be used for impervious sections of embankments. Soils in these groups should be carefully examined to ensure that they are properly zoned in relation to other materials in an embankment.

B-28. Of the fine-grained soils, the CL group is best adapted for embankment construction. The soils are impervious, reasonably stable, and provide fair to good compaction with sheepsfoot or rubber-tired rollers. Although they are not as desirable for rolled-fill construction, MH soils may be used in the cores of hydraulic-fill structures. Soils in the ML group may or may not have good compaction characteristics and must be closely controlled in the field to secure the desired strength. CH soils have fair stability when they are used on flat slopes, but they have detrimental shrinkage characteristics that may necessitate blanketing or incorporating them in thin interior cores of embankments. Soils containing organic matter (OL, OH, and Pt) are not commonly used for embankment construction because of the detrimental effects of the organic matter present. These materials may often be used to advantage in blankets and stability berms where strength is not important.

Permeability and Seepage Control

B-29. Permeability and the requirements for seepage control are essentially functions of the same property of a soil, so they will be discussed jointly. The subject of seepage in relation to embankments and foundations may be divided into three categories: seepage through embankments, seepage through foundations, and control of uplift procedures.

Seepage Through Embankments

B-30. In the control of seepage through embankments, it is the relative permeability of adjacent materials that governs their use in a given location. An earth embankment is not watertight; therefore, the allowable quantity of seepage through it is largely governed by the use of the structure. For example, considerable seepage may be allowed in a flood-control project, but the structure will still fulfill the storage requirements. Much less seepage is allowed in an irrigation project 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 homogeneous embankments to slow the flow of water. More pervious materials are usually placed on the outer slopes of areas where it is important that seepage not emerge on the downstream slope or where the possibility of drawdown exists on upstream slopes. Coarse-grained, free-draining soils (GW, GP, SW, SP) are best-suited for this purpose. When 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 successfully constructed of sand (SW, SP, and SM) or 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 largely depend on the material that they drain. Free-draining sands (SW and SP) or gravels (GW and GP) are usually preferred, but silty sand (SM) can effectively drain clay (CL and CH) and may be satisfactory.

Seepage Through Foundations

B-31. As with embankments, the use of the structures involved often determines the amount of seepage control in foundations. There have been cases in which the flow of water through a pervious foundation did not constitute an excessive water loss; therefore, seepage control measures were not necessary when adequate provisions were made against piping in critical areas. If seepage control is desired, 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. A line of relief well and a drainage trench at the downstream toe will not reduce the amount of seepage; however, both will control seepage and route the flow into collector systems where it can be led away harmlessly. Slightly less pervious material (silty gravels, silty sands, silts) may require a minor amount of seepage control (such as that afforded by a toe trench). If they are sufficiently impervious, no control may be necessary. The relatively impervious soils (GC, SC, CL, OL, MH, CH, OH) usually pass such a small volume of water that seepage control measures are not necessary.

Uplift Pressures Control

B-32. The 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, a positive cutoff may be preferred. Blanket, well, or trench installations can be expensive. Free-draining sands (SW and SP) are usually less permeable than gravels, and the volume of water that must be controlled for pressure relief is usually less. A positive cutoff may not be required with free-draining sands, so an upstream blanket, well, or toe trench installation may be effective. In some cases, a combination of a blanket and a trench or well may be preferred.

B-33. Silty soils (GM, SM, ML) do not usually 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. The more impervious soils (GC, SC, CL, OL, MH, CH, OH) do not usually require treatment for the control of uplift pressures; however, they assume importance when they occur as a relatively thin top stratum over more pervious materials. Uplift pressures in the lower layers that act on the base of the impervious top stratum can cause heaving and the formation of boils. Treatment of the lower layers by the methods mentioned above is usually indicated. The control of uplift pressures should not be applied indiscriminately merely because certain types of soils are encountered. 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

B-34. Table B-2, page B-3, shows the general compaction characteristics of the various soil groups. The evaluations that are given and the equipment that is listed are based on average field conditions in which proper moisture control and lift thickness 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. An advantage of the sheepsfoot roller is that it creates a rough surface that generates a better bond between lifts. It also kneads the soil, affording better moisture distribution. The rubber-tired equipment that is referenced in the table includes heavily loaded compactors and earthmoving equipment that possess a minimum wheel load of 15,000 pounds. When standard, wobble-wheeled rollers are used for compaction, the thickness of the compacted lift is usually reduced to about 2 inches. Typical compaction characteristics are as follows:

- Steel-wheeled rollers and vibration produce good compaction on granular soils that retain little or no fines. Well-graded materials (GW and SW) usually exhibit better results than poorly graded soils (GP and SP).
- Sandy soils are best compacted by crawler tractors. Rubber-tired equipment and steel-wheeled rollers are also effective on gravelly materials.
- Coarse-grained soils with fines of low plasticity (groups GM and SM) show good compaction characteristics with steel-wheeled rollers and vibratory or rubber-tired equipment. However, the range of moisture contents for effective compaction is typically very narrow, and close attention must be paid to the compaction moisture content.
- Soils of the ML group may be compacted with rubber-tired equipment or with sheepsfoot rollers.

- Gravels and sands with plastic fines (GC and SC) show fair compaction characteristics, although this quality may vary with character and the amount of fines. Rubber-tired or sheepsfoot rollers are ideal for these materials, especially sheepsfoot with vibration.

B-35. Sheepsfoot rollers are typically used for compacting fine-grained soils. When larger grain sizes are present, vibratory sheepsfoot rollers are preferable. Compaction characteristics are variable. Lean clays and sandy clays (CL) are best, fat clays and lean organic clays or silts (OL and CH) are fair to poor, and organic or micaceous soils (MH and OH) are poor. There is little change in the overall compaction density of fine-grained soils over a wide range of moisture contents. Maintaining a proper moisture range is essential for the performance of these soils.

B-36. For most construction projects, it is necessary to investigate the soil compaction characteristics by using a field test section. Table B-2 shows the ranges of unit dry weight for soils that are compacted according to the compaction test method that is discussed in chapter 2. Lower dry densities that have resulted from lesser compaction efforts in embankments and foundations are also shown in table B-2. These values are for guidance only. Design or construction control should be based on laboratory test results.

Suitability of Soils for Foundations

B-37. The suitability of soils for foundations of embankments or structures depends primarily on the strength and consolidation characteristics of the subsoil. 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 compensated for by overbuilding. 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 satisfactory for one type of construction but require special treatment for another.

B-38. Strength and settlement soil characteristics depend on variables (structure, in-place density, moisture content, cycles of loading in their geologic history) that are not readily evaluated by a classification system. Only general statements can be made concerning the suitability of various soil types (especially fine-grained soils) as foundations:

- Gravels and gravelly soils (GW, GP, GM, and GC) have good bearing capacities and experience little consolidation under load.
- Well-graded sands (SW) usually have good bearing values.
- Poorly graded sands and silty sands (SP and SM) may exhibit variable bearing capacities, depending on their density. This is true to some extent for all coarse-grained soils, but it is especially critical for uniformly graded soils of the SP and SM groups. When saturated, these soils may present an additional construction problem.
- Soils in the ML group may be subject to liquefaction and may have poor bearing capacities, particularly where heavy structure loads are involved.
- Soils in the CL group are probably the best from a foundation standpoint. But in some cases, the soils may be soft and wet and exhibit poor bearing capacities and fairly large settlements under loads.
- Consolidated CH soils and soils in the MH groups may normally show poor bearing capacities and large settlements.
- Organic soils (OL and OH) have poor bearing capacities and usually exhibit large settlements under loads.

B-39. For most fine-grained soils, the structure foundation is governed by factors (the bearing capacity of the soil and the magnitude of the load. It is possible that simple spread footings might adequately carry the load without excessive settlement. If the soils are poor and the structure loads are relatively heavy, alternate methods are indicated. Pile foundations may be necessary in some cases. In special instances (particularly in cases that involve CH and OH soils), it may be desirable and economically feasible to remove certain soils from the foundation. Highly organic soils are usually very poor foundation materials. They are capable of carrying light loads, but they are unsuited for most construction purposes. If highly organic soils occur in the foundation, they may be removed (if limited in extent) or displaced (by dumping firmer soils

on top) or piling may be driven through them to form a stronger layer. Proper treatment will depend on the structure involved.

GRAPHICAL PRESENTATION OF SOILS DATA

B-40. It is customary to use various symbols to present the results of soils explorations as schematic representations of borings and test pits. Commonly used hatching symbols include small, irregular round symbols for gravel; dots for sand; vertical lines for silt; and diagonal lines for clay. 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. The appropriate symbols are shown in table B-2, page B-3. Hatching symbols 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. 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 table B-2.

ROADS AND AIRFIELDS

B-41. The properties desired in soils for foundations under roads and airfields and for base courses under flexible pavements are adequate strength, good compaction, sufficient drainage, frost resistance, and acceptable compression and expansion characteristics. Some of these properties may be supplied by proper construction methods. For instance, 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 by increasing the thickness of overlying concrete pavement or of base materials in flexible pavement construction. The proper design of roads and airfield pavements may require a more detailed evaluation of soil properties 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.

SOILS CLASSIFICATION SHEET FEATURES

B-42. General characteristics of the soil groups that are pertinent to roads and airfields are presented in table B-4, page B-18.

B-43. Table B-4 provides a statistical breakdown of maximum dry density (MDD) and optimum moisture content (OMC) for the principal soil classifications for modified proctor effort (ASTM D1557). Each bubble represents one standard deviation variation of MDD and OMC from a worldwide database of soils. This can assist in estimating construction properties for soils that are beyond the ranges given in table B-4.

Note. The dry densities that are listed are for the modified effort that is discussed in chapter 17. Design values are intended for long-term structures. For the contingency design, the unsoaked CBR design may be appropriate, depending on the structure design life and on the loading restrictions during inclement weather.

Coarse-Grained Soil Group Subdivision

B-44. In table B-4, page B-18, the basic soil groups (GM and SM) have been subdivided into two groups that are designated by the suffixes d and u. These suffixes 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 observations and laboratory tests of the behavior of the soils in these groups. The basis for the subdivision is the LL and PI of the fraction of the soil passing the Number 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

B-45. The descriptions in Columns 7, 8, and 9 of table B-4 show the suitability of the soil groups for use as subgrade, subbase, or base materials, provided that they are not subject to frost. 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. Proper design procedures should be used as follows:

- Coarse-grained soils are usually the best subgrade, subbase, and base materials. The GW group has excellent qualities as subgrades and subbases and is good as a base material. “Excellent” is not used for any of the soils for base courses and should only be used in reference to high-quality, processed crushed stones.
- Poorly graded gravels and some silty gravels (GP and GM) are usually only slightly less desirable as subgrade or subbase materials and may be used as base materials under favorable conditions. However, poor gradation and other factors sometimes reduce the value of soils to the extent that they offer only moderate strength, and their value as a base material is less.
- GM, GC, and SW groups are reasonably good subgrade materials, but they are usually not suitable as bases.
- SP and SMd soils are usually considered fair to good subgrade and subbase materials, but they are unsuitable for base materials.
- SMu and SC soils are fair to poor subgrade and subbase materials and are not suitable for base materials.
- Fine-grained soils range from fair to very poor subgrade materials:
  - 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.

Table B-4. Characteristics of soil groups pertaining to roads and airfields

Major Divisions (1)	(2)	Letter (3)	Symbols: Hatching (4)	Symbols: Color (5)	Name (6)	Value As Subgrade When not Subject to Frost Action (7)	Value As Subbase When not Subject to Frost Action (8)
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
		GM 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
		SM 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

Table B-4. 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 practi-cally impervious	Rubber-tired roller, sheepsfoot roller	115 -135	20 - 30	200 - 500
Poor to not suitable	Slight to medium	Slight	Poor to practi-cally 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 practi-cally impervious	Rubber-tired roller, sheepsfoot roller	100 -130	10 - 20	100 - 300
Not suitable	Slight to high	Slight to medium	Poor to practi-cally 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	-	-	-

Table B-4. Characteristics of soil groups pertaining to roads and airfields (continued)

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. Suffix u will be used otherwise.
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 material and other angular material.** Steel-wheeled and rubber-tired rollers are recommended for hard, angular material with limited fines or screenings. Rubber-tired equipment is recommended for softer materials subject to degradation.
   b. **Finishing.** Rubber-tried 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:
      (1) Crawler-type tractor with total weight in excess of 30,000 pounds.
      (2) Rubber-tired equipment with 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 approximantely 65 to 150 psi.
      (3) Sheepsfoot roller with unit pressue (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 be 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 (ASTMD 1557).
4. The maximum CBR value (Column 15) that can be used in the design of airfields is, in some cases, limited by gradation and plasticity requirments.

Legend:

ASTM	American Society of Testing and Materials
CBR	California Bearing Ratio
cu in	cubic inch
lb	pound
LL	liquid limit
pcf	pound per cubit foot
PL	plastic limit
psi	pound per square inch

B-46. The above qualities are compensated for in flexible pavement design by increasing the thickness of overlying base materials. They are compensated for in rigid pavement design by increasing the pavement thickness or by adding a base course layer. Fine-grained soils are unsuitable as subbase and 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 table B-4 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 are relative indications of strengths from plate-bearing tests that are used in rigid pavement design. These tests are used for the design of pavements; therefore, actual test values should be used instead of the approximate values that are shown in the table.

B-47. For wearing surfaces on unsurfaced roads, sand-clay-gravel mixtures (GC) are generally considered the most satisfactory. However, the percentage of fines should not be too large of a percentage of fines, and the PI should be in the range of 5 to 15.

Potential Frost Action

B-48. The relative effects of frost action on the various soil groups is shown in table B-4. 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 that is necessary for the formation of ice lenses may become available from a high groundwater table, from a capillary supply, from within the soil voids, or through infiltration. The degree of ice formation that will occur is influenced by environmental factors

(topographic positions, stratifications of the parent soil, transitions into cut sections, the lateral flow of water from side cuts, localized pockets of perched groundwater, drainage conditions). Silts, fines, silty sands are the worst offenders. Coarse-grained materials with little or no fines are only slightly affected. Clays (CL and CH) are subject to frost action, but the loss of strength may not be as great for silty soils. Inorganic soils that contain less than 3 percent of grains finer than 0.02 millimeter in diameter by weight are not typically susceptible to frost. When frost-susceptible soils are encountered in subgrades and frost is a definite problem, two acceptable methods of design of pavements are available—a sufficient depth of acceptable granular material is placed over the soils to prevent freezing in the subgrade (preventing the detrimental effects of frost action) or a reduced depth of granular material is used (allowing freezing in the subgrade). 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 diminish ice segregation in the subgrade and subbase.

Compressibility and Expansion

B-49. Two types of soil characteristics are applicable to road and runway design—

- The relatively long-term compression or consolidation under the dead weight of the structure.
- The short-term compression and rebound under moving wheel loads.

B-50. The long-term consolidation of soils becomes a factor in design when heavy fills are made on compressible soils. If adequate provisions are made for this type of settlement during construction, it will have little influence on the load-carrying capacity of the pavement. However, when elastic soils that are subject to compression and rebound under wheel loads are encountered, adequate protection must be provided. Even small movements of soil may be detrimental to the base and wearing course of pavements.

B-51. It is fortunate that the free-draining, coarse-grained soils (GW, GP, SW, and SP) that make the best subgrade and subbase materials exhibit almost no tendency toward high compressibility or expansion. The compressibility of soils increases with an increasing LL. 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

B-52. 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 table B-4, page B-18. The presence of moisture in base, subbase, and subgrade materials (except for free-draining, coarse-grained soils) may cause pour water pressures and loss of strength. The moisture may stem from the infiltration of rainwater or by the capillary rise from an underlying water table. While free-draining materials permit the rapid draining of water, they also permit the rapid ingress of water. If they are adjacent to less pervious materials and have free access to water, they may serve as reservoirs to saturate the less pervious materials. In most instances adequate drainage systems should be provided. Gravelly and sandy soils with little or no fines (groups GW, GP, SW, and SP) have excellent drainage characteristics; GMd and SMd soils have fair to poor drainage characteristics; and GMu, GC, SMu, and SC soils may be practically impervious. ML, MH, and Pt soils have fair to poor drainage characteristics. The other groups have poor drainage characteristics or are practically impervious.

Compaction Equipment

B-53. The compaction of soils for roads and runways 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 when saturation or near saturation takes place. Processed materials (such as crushed rock) are often used as base course and require special treatment in compaction. Compaction equipment that usually produces the desired densities is shown in table B-4, page B-18. Several types of equipment are listed for some of the soil groups 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.

B-54. Steel-wheeled rollers are recommended for angular materials that possess limited amounts of fines, crawler tractors and rubber-tired rollers are recommended for gravels and sands, and sheepsfoot rollers are recommended for coarse-grained or fine-grained soils that have some cohesive qualities. Rubber-tired rollers are also recommended for final compaction operations for most soils, except for those with a high LL. The suggested minimum weights for the various types of equipment are shown in table B-4, page B-18. It also shows ranges of unit dry weight for soils that are compacted according to the compaction test method that is described in chapter 2. These values are included primarily for guidance, the design or control of construction should be based on actual test results.

GRAPHICAL PRESENTATION OF SOILS DATA

B-55. 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 table B-2, page B-3, or table B-4 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 table B-2.

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Appendix C

Rapid Soils Analysis Kit

C-1. The RSAK addresses the need for rapid onsite soil characterization and provides soil classification and construction design parameters within 1 hour. The RSAK includes a small field laboratory that consists of a microwave, electric balance, sieve shaker, grinder, PL tool, sieves, bowls, spatulas, and scoops (figure C-1.) The kit is packaged into two large pelican cases that weigh approximately 70 to 75 pounds each (figure C-2.) Soil moisture, grain size distribution, and PL are determined from a small soil sample. The numerical data that are generated from soils tests are entered into regression software to determine the soil classification. The software uses multiple-linear regression that is based on soil property databases to estimate the OMC and MDD. Built-in, higher order regression equations compute complete design curves for Proctor density, as-built CBR, and soaked CBR for the constructed condition of the soil of interest (figure C-3, page C-2).

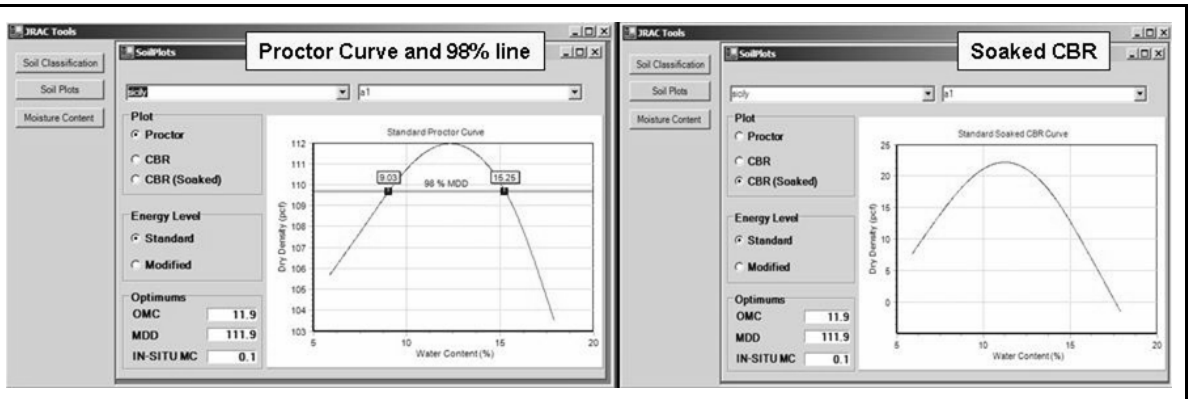


Figure C-1. RSAK contents



Legend:
ID identification

Figure C-2. RSAK packaged contents



Legend:

CBR	California Bearing Ratio
JRAC	joint rapid airfield construction
MDD	maximum dry density
OMC	optimum moisture content
pcf	pound per cubic foot

Figure C-3. RSAK software output

PROBLEM

C-2. An accurate and expedient method of determining soil classification is essential to establish the soil design criteria for contingency airfield and lines of communication construction using in situ soil. During a contingency design and construction scenario, only a few hours are available to accumulate the ground truth data. Currently, only subjective field analysis techniques are available. The results fail to provide tangible numerical data that can be used to establish construction criteria.

BENEFIT AND SAVINGS

C-3. The RSAK dramatically reduces the logistics of performing in-theater soils analyses. The RSAK is easily transported as a two-man carry and can be used on site in forward operations with a setup time of only a few minutes. A traditional soil laboratory must be stationed at a rear operating base. Collected field samples must be sent for an analysis that requires days of testing, but an RSAK analysis is complete in 1 hour. The RSAK employs microwave drying that enables rapid in situ moisture content determination without the need for nuclear test devices.

DISTRIBUTION SOURCES

C-4. RSAK software is available from ERDC. Hardware is commercially available from worldwide soil equipment distributors. Current hardware costs for a single kit to the government from commercial vendors is approximately $5,000. The hardware and the RSAK, as a whole, is currently not an NSN product and must be assembled by the users or by ERDC.

TRAINING

C-5. Current training requires a 4-hour time block with a technical engineer.

CONTENTS

C-6. The contents of the commercially-available RSAK are inserted into two large 1660 pelican cases. Some of the equipment is standard and is included in supplied military soils kits. Table C-1 lists the equipment packed into each case. Specific model numbers vary depending on the vendor and can be requested from the ERDC-WES at the contact given.

Table C-1. RSAK contents

Case 1	*Qty*	*Case 2*	*Qty*
Work gloves	1	3-in-diameter sieve shaker	1
Microwave	1	Porcelain bowls	4
Microwave-safe plates	4	Square scoop	1
Timer	1	Long spatula	1
Microwave brick	1	Short spatula	1
PL roller	1	Sample splitter	1
PL rolling board	1	Sample cups	9
Extra paper	2	Sieve brush, fine	1
Long spoon	1	Sieve brush, coarse	1
Mortar and pestle	1	Permanent marker	1
Porcelain pestle	1	3-in, 3/8-in brass	1
8-in No. 40 stainless sieve	1	3-in, No. 4 sieves, stainless (1-in)	1
Sartorius GE812 balance	1	3-in, No. 10 sieves, stainless (1-in)	1
500-g calibration weight	1	3-in, No. 20 sieves, stainless (1-in)	1
Towels	6	3-in, No. 40 sieves, stainless (1-in)	1
Sample pans	2	3-in, No. 100 sieves, stainless (1-in)	1
Water bottle (squeeze)	1	3-in, No. 200, stainless (1-in)	1
Metal power strip	1	3-in pan, stainless	1
Large pelican case	1	3-in lid, stainless	1
		Coffee grinders	2
		Geotechnical gauges	2
		Sand gauges	2
		Large pelican case	1

Legend:
g gram
in inch
No. number
PL plastic limit
qty quantity

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Appendix D

United States Marine Corps Soil Test Set

The soil test set is a portable system that consists of several major testing components packed inside a series of three deployable cases. It also includes various sieves, containers, tarps, tools, and items that are designed to aid in testing soils. Some components of the set require power and can be operated by batteries or optional alternating current power adapters. The soil test set is an organic table of equipment item in the engineer support battalions, Marine logistics groups and wing support squadrons, Marine aircraft wings. It allows Marines to separate soils, assess the in situ bearing capacity, determine the CBR values, determine the soil moisture content, and determine the field identification classification.

CASE 1

D-1. Case 1 (figure D-1) contains laboratory equipment. It is used to determine soil classification (grain size, clay content, and soil weight), electrical soil moisture content, and soil trafficability. Table D-1, page D-2, shows the contents of Case 1.



Figure D-1. Case 1—laboratory equipment

Table D-1. Case 1 contents

Nomenclature	*Qty*	*Nomenclature*	*Qty*
Shipping and storage container	1	Sieve test, Number 040	3
Bottle, screw cap, 32-ounce, transparent	4	Sieve test, Number 004	3
Auger, earth-boring, hand	1	Mortar and pestle	2
Scale, weighing, 4,000-gram	1	Bag, textile	10
Lead test, EC-5	1	Cloth, duck	5
Indicator, digital display	1	Printer, automatic data processing	1
Spatula, laboratory	2	Tape, paper, computing machine	1
Trowel	2	Wrench, box and open end combination, 11-millimeter	1
Spoon, food service	2	Wrench, box and open end combination, 13-millimeter	2
Penetrometer	1	Wrench, box and open end combination, 18-millimeter	1
Jug, water, collapsible, 5-gallon	1	Bowl, food, mixing, 3-quart	6
Brush, paint	2	Battery, nonrechargeable, CR2	AR
Pestle, laboratory	2	Battery, nonrechargeable, AA	AR
Sieve test, Number 200	1	Towel, hand	6
Legend: AR as required qty quantity			

CASE 2

D-2. Case 2 (figure D-2) contains a DCP. It is used to estimate soil strength, measure the penetration resistance of in situ soil, assess soil density, and measure field or in-place CBR. Table D-2 shows the contents of Case 2.

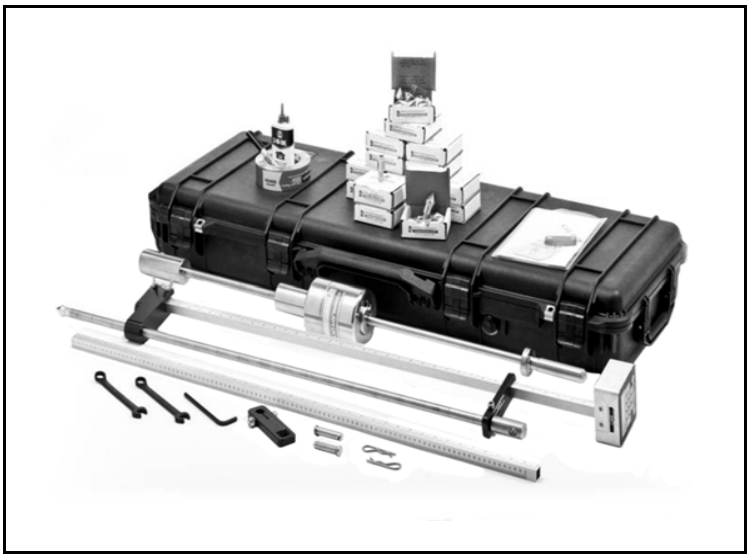


Figure D-2. Case 2—DCP

Table D-2. Case 2 contents

Nomenclature	*Qty*	*Nomenclature*	*Qty*
Case, photographic equipment, with foam insert	1	Cone, disposable	12
Ruler, magnetic	1	Adapter, soil testing	2
Shaft, assembly, shouldered	1	Wrench, box and open end combination, 9/16-inch	2
Connecting link, rigid, 37.75-inch	1	Lubricating, oil, general-purpose	AR
Indicator, height, 40-inch	1	Tape, pressure-sensitive, adhesive	AR
Battery, nonrechargeable, AAA	AR	Pen, plastic tip	AR
Upper attachment, vertical scale	1	Key, socket head screw, 1/4-inch	1
Pin, straight, headed	1	Foot, vertical, scale	1
Drive point, hardened	1		
Legend: AR as required qty quantity			

CASE 3

D-3. Case 3 (figure D-3) contains a speedy moisture tester. It is used to measure the moisture content of soils by combining chemicals with soil samples. The dial gauge identifies the moisture content of a given soil sample as a percentage. Table D-3, page D-4, shows the contents of Case 3.



Figure D-3. Case 3—speedy moisture tester

Table D-3. Case 3 contents

Nomenclature	*Qty*	*Nomenclature*	*Qty*
Detector, moisture content	1	Gasket	AR
Case, electrical test and measuring equipment	1	Cup, soil moisture test	1
Scale, weighing, 200-gram	1	Insulating compound, electrical	AR
Pencil, mechanical, china marker	AR	Bottle, brush	1
Calcium carbide, technical	AR	Spoon, measuring, 50-millimeter	1
Weight, balance, 200-gram	1	Ballbearing, 1 1/8-inch-diameter	2
Legend: AR as required qty quantity			

Glossary

The glossary lists acronyms and terms with Army or joint definitions. Where Army and joint definitions differ, (Army) precedes the definition. Terms for which TM 3-34.43/MCRP 3-17.7H/NAVFAC MO 330/AFH 32-1034 is the proponent are marked with an asterisk (*). The proponent publication for other terms is listed in parentheses after the definition.

SECTION I – ACRONYMS AND ABBREVIATIONS

AAR	alkali aggregate reaction
AASHTO	American Association of State Highway and Transportation Officials
AC	asphalt cement
ADRP	Army doctrine reference publication
AEA	air-entraining admixtures
AFH	Air Force handbook
AFJMAN	Air Force joint manual
AFJPAM	Air Force joint pamphlet
AFM	Air Force manual
AFMAN	Air Force manual
AISI	American Iron and Steel Institute
ASR	alkali silica reaction
ASTM	American Society for Testing and Materials
attn	attention
BBR	bending beam rheometer
C	Celsius
$CaCO_3$	calcium carbonate
CaO	oxides
$CaO(OH)_2$	hydrated lime
CBR	California Bearing Ratio
CE	compaction effort
CF	cement factor
CRREL	Cold Regions Research and Engineering Laboratory
DA	Department of the Army
DC	District of Columbia
DCP	dynamic cone penetrometer
DCS	deputy chief of staff
DD	Department of Defense
DT	direct tension
ed	edition
EM	engineering manual

ERDC	Engineer Research and Development Center
F	Fahrenheit
FASST	fast all-season soil strength
FL	Florida
FM	field manual
ft	foot
GPS	global positioning system
GSL	geotechnical and structures laboratory
HMA	hot-mix asphalt
HN	host nation
JR	Junior
JP	joint publication
kN	kilonewton
lbf/ft^3	foot pound per cubic foot
LCF	lime-cement-fly ash
LL	liquid limit
LWD	light weight deflectometer
MC	medium-curing
MCRP	Marine Corps reference publication
MDD	maximum dry density
m/m^3	meters per cubic meter
MMC	minimum moisture content
MO	maintenance operating
MS	medium-setting
MSCoE	Maneuver Support Center of Excellence
NAVFAC	Naval Facilities Engineering Command
NCOIC	noncommissioned officer in charge
No.	number
NSN	national stock number
OAC	optimum asphalt content
OMC	optimum moisture content
PAV	pressurized aging vessel
PCA	Portland Cement Association
PCN	publication control number
PFWD	portable falling weight deflectometer
PG	performance graded
pH	potential hydrogen
PI	plasticity index
PIN	publication identification number
PL	plastic limit
PSPA	portable seismic pavement analyzer

RAP	reclaimed asphalt pavement
RC	rapid curing
RI	Rhode Island
RS	rapid-setting
RSAK	rapid soils analysis kit
SC	slow-curing
SI	System Internationale
SR	special report
SS	slow setting
SSD	saturated, surface-dry
TM	technical manual
TMD	theoretical maximum density
TR	technical report
UFC	United Facilities Criteria
U.S.	United States
USA	United States Army
USACE	United States Army Corps of Engineers
USAES	United States Army Engineer School
USAF	United States Air Force
USCS	Unified Soil Classification System
USMC	United States Marine Corps
USN	United States Navy
VA	Virginia
Vol.	volume
WRA	water-reducing admixtures

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None.

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Index

Entries are by page number.

A

accelerators, 18-4
additive, 22-1, 22-2, 23-2, 23-3, 23-5, 23-6, 23-8
admixtures, 18-1, 18-4, 18-6, 21-2, 21-3, 21-4
adsorbed water, 1-5
aerial photographs, 2-3, 2-4, 2-6, 10-3
aggregate, 1-4, 2-6, 18-3
  base and pavement, 2-7
  coarse, 1-5
aggregate correction factor, 20-6
air, 18-1
air content, 21-2
air content test, 20-4
air entraining, 18-4
air-entraining admixtures (AEAs), 18-4
alkali aggregate reaction (AAR), 18-5
angular, 1-3
asphalt, 11-2
asphalt emulsion test, 13-3
asphalts, 13-1

B

ball test, 1-21
batch plant, 17-2
bite or grit test, 1-18
bitumen field-identification tests, 13-1
bituminous binder, 13-5
bituminous material, 11-1, 12-1
bituminous materials, 11-8
bleeding, 18-5, 21-2
blended cements, 18-2, 18-3
breaking test, 1-20
bulk specific gravity, 4-7
bulky, 1-3

C

calcium carbide gas pressure test, 3-6
calibration, 20-6
California Bearing Ratio (CBR) analysis, 9-11
California Bearing Ratio (CBR) data presentation, 9-11
California Bearing Ratio (CBR) ratio mold, 2-16
California Bearing Ratio (CBR) test, 9-1, 10-2
California Bearing Ratio (CBR) test penetration appartus, 9-8
capillary action, 1-5
capillary fringe, 1-5
capillary tube vicsometer, 14-3
cement, 18-2
classification, 2-2
clay soils, 9-32
cleanliness, 13-5
coarse aggregate, 13-6, 15-1
coarse-grained soil, 1-11
coarse-grained soils, 1-7, 1-8, B-6
cohesionless soils, 1-6
cohesiveness, 1-6
commercial aggregates, 12-3
compaction equipment, 7-13
compaction specification block, 7-10
compaction test, 7-1, 10-2
compactive effort, 7-12, 7-13
compressive strength, 20-10
concrete, 18-2
cone penetration refusal, 9-33
conversions, 18-7
curing, 18-5
curing beams, 20-8
curing compound, 18-5
cylinder capping, 20-14
cylinders, 20-13

D

data collection, 18-7
deliberate survey, 2-1
density, 1-4
density tester, 8-14
dispersing agents, 5-10
drive cylinder method, 8-11
drum mix plant, 17-2
drying shrinkage cracking, 18-5
dry-strength test, 1-20
dual-mass dynamic cone penetrometer (DCP), 9-26
dual-mass dynamic cone penetrometer (DCP) kit, 9-25
ductility test, 14-3
durability, 18-6
dynamic cone penetrometer, 9-24
dynamic cone penetrometer (DCP), 9-30

E

early age cracking, 21-4
entrained air, 18-1
entrapped air, 18-1
expansive soils, 9-20
exploration, 10-2

F

fall cone, 6-8
feel test, 1-18
field tests, 1-11
fine aggregate, 13-6, 15-1
fine-grained soils, 1-7, 1-9, B-7
fineness modulus, 19-2
frost-susceptibility groups, 5-16

G

grain shape, 1-3, 1-15
grain size, 1-15
grain size analysis, 10-2
grain size distribution, 1-15
graph presentation, 7-8
gravel, 12-2
gravitational pull, 1-5
grout, 18-2

H

hasty field identification, 1-13
hasty survey, 2-1
highly organic soils, 1-7, 1-10

hydraulic cement, 18-2
hydrometer analysis, 5-15
hygroscopic moisture, 1-5

I

information sources, 2-3
inplace field California Bearing Ratio (CBR) test, 9-24
intelligence reports, 2-3

L

ledge stone, 12-2
light weight deflectometer (LWD), 9-38
liquid limit (LL), 1-6, 6-1, 10-2
liquid materials, 12-1
Los Angeles abrasion test, 15-5
loss of workability, 21-1

M

maps, 2-3
Marshall test, 16-2, 16-8, 17-3
microwave oven-dry test, 3-4
mineral aggregates, 11-1
mineral dust, 13-6
mineral filler, 13-6
modification, 22-2
modulus of soil reaction, 9-33
moisture content, 1-5, 1-18, 3-1
moisture susceptibility, 13-5
mortar, 18-2

N

nonnuclear density devices, 8-15
nuclear moisture, 8-14

O

odor test, 1-16
oven-dry test, 3-1

P

particle analysis, 10-2
paste, 18-2
pat test, 1-20
penetration measurement, 9-29
penetration test, 14-3
percent compaction, 7-10
percent moisture, 7-10
plastic limit (PL), 1-6, 6-12, 10-2
plastic shrinkage cracking, 18-5
plasticity, 1-6
plasticity index (PI), 1-6, 6-15
plate bearing test, 9-33
platic limit (PL)
  device, 6-13
platy, 1-3
poorly graded soils, 1-4
portable falling weight deflectometer (PFWD), 9-38, 9-39
pour test, 13-3

Q

quarry stone, 12-2
quartering, 2-18

R

rapid soils analysis, 2-1
rapid soils analysis kit (RSAK), C-1
reporting, 18-7
retarders, 18-4
ribbon test, 1-21
rocks, 1-1
roll test, 1-18
rounded, 1-3

S

safety precautions, 11-5
sample test schedule, 10-2
samples, 2-11, 20-7
  chunk, 2-11
  cylinder, 2-14
sampling, 2-5, 2-7, 2-11, 12-1, 19-1, 20-1
sand, 12-2
sand cone method, 8-2
sand displacement method, 8-2
sands, 9-32
Saybolt-Furol test, 14-3
sedimentation test, 1-16
semisolid materials, 12-1
setting time, 21-4
shine test, 1-22
shrinkage, 18-6
shrinkage limit, 1-6
sieve analysis, 15-1, 17-2
sieve-analysis, 5-1
slump test, 20-2
smear test, 13-3
softening-point test, 14-3
soil, 1-1
soil components, 1-2
soil description, 1-12
soil formation, 1-1
soil groups, 1-2, 1-7
  symbols, 1-7
soil layers, 2-2
soil particles, 1-4
soil profile, 1-2, 2-20
soil properties, 1-11
soil survey, 2-1, 2-2
soil technical report, 10-3
soil types, 1-2
solid materials, 12-1
solubility test, 13-4
special considerations, 9-32
specific gravity, 1-5, 4-7, 15-1, 15-5
specific-gravity test, 4-1
specimens, 20-7, 20-12
stabilization, 22-1
stabilizers, 23-1, 23-4
steel shot volume replacement method, 8-6
stockpiles, 12-2
strength, 18-6, 21-1
subangular, 1-3
subrounded, 1-3
subsurface water, 1-5
surface water, 1-5
swelling, 9-20

T

technical soils report, 10-3
temperature, 18-6
  concrete, 20-1
test beams, 20-7
thin-film oven test, 14-4
thread test, 1-18
time of setting, 18-6, 21-1
troubleshooting, 21-2

U

unbounded caps, 20-15
undisturbed sample testing, 9-10

V

visual test, 1-12

W

water density effect, 7-11
water placement method, 8-15

water-cement ratio, 18-6
water-reducing admixtures (WRAs), 18-4
weather, 9-32
well-graded soils, 1-4
wet-shaking test, 1-19
workability, 18-7

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