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MATERIALS TESTING

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AUGUST 1987

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HEADQUARTERS
DEPARTMENT OF THE ARMY
DEPARTMENT OF THE NAVY
DEPARTMENT OF THE AIR FORCE
Washington, DC, 17 August 1987

MATERIALS TESTING

PREFACE

Purpose. This manual provides the technical information necessary for military personnel to obtain samples and perform engineering tests and calculations on soils, bituminous paving mixtures, and concrete. These tests and calculations are required to achieve proper design with these materials and adequate control over their use in military construction.

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

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

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*This publication supersedes TM 5-530/NAVFAC MO-330/AFM 89-3, 6 February 1971 and rescinds DD Form 1214, Aug 57 and DD Form 1795, Feb 71.

Non-Military Methods. 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 Association of State Highway and Transportation Officials (AASHTO), the American Concrete Institute (ACI), and the Portland Cement Association (PCA).

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

This manual prescribes the following Department of Defense (DD) Forms that can be found in the back of this manual for local reproduction purposes. Forms will be reproduced on 8½ by 11 inch paper.

DD Form 1205	Soil Moisture Content
DD Form 1206	Sieve Analysis Data
DD Form 1207	Grain Size Distribution Graph - Aggregate Grading Chart
DD Form 1208	Specific Gravity Tests
DD Form 1209	Atterberg Limits Determination
DD Form 1210	Soil Compaction Test Data
DD Form 1211	Soil Compaction Test Graph/Swell Data
DD Form 1212	California Bearing Ratio Test Data
DD Form 1213	Unconfined Compression Test
DD Form 1215	Unit Weight Determination "Volume of Hole" Methods
DD Form 1216	Specific Gravity of Bituminous Mix Components
DD Form 1217	Bituminous Mix Design - Aggregate Blending
DD Form 1218	Marshall Method - Computation of Properties of Asphalt Mixtures
DD Form 1219	Bituminous Mix Curves
DD Form 1220	Concrete Mixture Design Data
DD Form 1793	Determination of Asphalt Content (Dulin-Rotarex Extractor)
DD Form 1794	Hydrometer Analysis
DD Form 2463	California Bearing Ratio (CBR) Analysis
DD Form 2464	Report of Foundation and Borrow Investigation

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Chapter 1

Materials Testing Overview

FUNDAMENTALS

The military engineer is responsible for developing and maintaining transportation facilities, housing and special structures, sanitary facilities, and military defenses. Transportation facilities include roads, railways, airports and landing strips, pipelines, and harbor structures. These structures and facilities all are built on and sometimes use the local soil. To design these, the engineer must know the type and characteristics of the soil at the site. For example, in designing a road or airfield, the engineer must determine whether the soil can withstand the loads to be transported, including vehicle weights, or whether soil stabilization or paving will be needed. If the road or airfield requires a pavement, then the suitability of available aggregate materials must be established since speed and efficiency of construction dictate the use of nearby sources. Bituminous paving mixtures and portland cement concrete are made at or near the work site under the control of the local engineering officers who must design the mix and test the performance of the finished product. The engineer must have basic data concerning the properties of these materials in order to use them effectively in construction. Such data are obtained from the tests described in this manual.

MATERIALS TESTS

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

SOIL PROPERTIES

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

CLASSIFICATIONS

Tests and evaluation of results are more easily made by using some common reference or system which has a universal interpretation. Thus, no matter who made the tests or the evaluation, the results would be understood by anyone familiar with the system. One reference for soils used by the military engineer is called the Unified Soil Classification System (USCS). All soils are divided into three major categories, two of which are based on grain size. Further subdivision distinguishes between gravel, sand, silt, clay, and organic content and between well-graded and poorly-graded soils. Each of these types is symbolized by a combination of two or four letters. A more detailed explanation of the USCS will be found in Chapter 2, Section I and in TM 5-330/AFM 86-3, Vol II.

DESIGN REQUIREMENTS

The engineer, having the completed soil classification and all the other information required for a proper design of an airfield or road, can start the design. The design requirements are covered in TM 5-330/AFM 86-3, Vol. II.

BITUMENS

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

CONCRETE

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

STABILIZATION

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

EQUIPMENT

The equipment for all materials testing given in this manual consists of three sets: a soil test set, an asphalt test set, and a concrete test set. The soil test set is considered the basic set. The other two sets are used with it. In selected operations, for example control testing at a concrete batch plant, the concrete test set can be used independent of the soil test set. Certain items listed as part of the sets are not issued with the sets, but must be requisitioned separately. Some of the test methods, such as the specific gravity of solids, the hydrometer analysis, and the shrinkage limit, describe items of equipment which are not issued with the test sets, but would be available locally or from commercial sources.

Test Set, Soil (SC 6635-98-CL-E02)

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

Test Set, Asphalt (SC 6635-98-CL-E03)

This is issued in three chests: a laboratory centrifuge chest, a miscellaneous equipment chest, and a 1,1,1-trichloroethane chest. This set is not complete within itself and therefore must be used with the soil test set. Items such as forms, brushes, cloth, and chemicals (alcohol, sodium hydroxide, and 1,1,1-trichloroethane) are not issued with the set, but must be requisitioned separately.

Test Set, Concrete (SC 6635-98-CL-E04)

This is issued in three chests: a beam testing-machine test, a collapsible steel forms chest, and a miscellaneous equipment chest. This set normally is used with the soil test set. Chemicals, DA forms, and some other items are not issued with the set, but must be requisitioned separately.

Nuclear Moisture Density Gage (NSN 6635-01-030-6896)

This will perform moisture and density tests. The gage is issued in its case with all necessary equipment except test forms. Forms should be locally reproduced from the Engineer School or manufacturers' samples. This equipment requires special storage considerations and licensing of operators.

Chapter 2

Soils

Section I. CLASSIFICATION, EXPLORATION, AND FIELD IDENTIFICATION

DISCUSSION

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

Soil

Soil is a heterogeneous aggregation of uncemented or weakly cemented mineral grains enclosing voids of varying sizes. These voids may contain air, water, organic matter, or different combinations of these materials. The engineer must therefore be concerned not only with the sizes of the particles, but also with the voids between them and particularly what these voids enclose (water, air, or organic materials).

Rocks

Geologists classify rocks into three basic groups: igneous, formed by cooling from a molten state; sedimentary, formed by accumulation and cementation of existing particles and remains of plants and animals; and metamorphic, formed from existing rocks subjected to heat and pressure. Exposed to the atmosphere, these rocks disintegrate and decompose either by mechanical action (wind, water, ice, and vegetation) or 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. This resulting material is called soil.

Soil Formation

This 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, the soils are evaluated by their basic physical properties: gradation of sizes of the different particles, bearing capacity as reflected by soil density, and particle shapes. The engineer extends the evaluation by considering the effect of water action on the soils. With a complete evaluation, the engineer can determine whether the soil is adequate for the project.

Strata

At a particular location there usually will be several layers or 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 top soil or agricultural soil since it supports plant growth. Adequate soil evaluation for engineering uses requires that all strata be identified to whatever depth may be affected by the construction. A vertical cross section through the earth, with the depths and types of soil indicated, is called a soil profile.

PHYSICAL PROPERTIES

The physical properties of a soil aid in determining the engineering characteristics. These properties are the basis for the system of soil classification used in engineering identification of soil types. Physical characteristics of soil particles are size and shape. The proportions of particles of different sizes determine the gradation of the aggregate. Compactness refers to the closeness of packing of the soil particles; the closer the packing, the greater the compactness, and the larger the weight of soil per unit of volume. Plasticity characteristics of fine-grained soil components include liquid and plastic limits, shrinkage ratios, dry strength, and unconfined compressive strength. Specific gravity of soil particles aids in their identification. The presence of organic matter is important to the engineering use of soils. Color, texture, odor, structure, and consistency are readily observed factors that aid in soil description.

Grain-Size Groups

Soils are divided into groups based on the size of the particle grains in the soil mass. Common practice distinguishes the sizes through the use of sieves. A sieve is a screen attached across the end of a shallow cylindrical frame. The screen permits smaller particles to fall through and retains the larger particles on the sieve. Sieves with screen openings of different sizes (the largest on the top and the smallest at the bottom) separate the soil into particle groups based on size. The amount remaining on each sieve is measured and described as a percentage by weight of the entire sample. Size groups in the USCS are shown in Table 2-1. Particles passing the No. 200 sieve, but larger than 0.002 millimeter to 0.005 millimeter are called silt. Those finer are called clays.

Table 2-1. Grain size groups

Size groups	Sieve size	
	Passing	Retained on
Cobbles	No maximum size*	3 in
Gravels	3 in	No. 4
Sands	No. 4	No. 200
Fines	No. 200	No minimum size

*In military engineering, maximum size of cobbles is accepted as 40 inch based upon maximum jaw opening of the crushing unit.

Particle Shape

The shape of the particles influences the strength and stability of a soil. Two general shapes are normally recognized: bulky and platy (Figure 2-1). The bulky shapes include particles which are relatively equal in all three dimensions. In platy shapes, one dimension is very small compared to the other two. For example, a thick book would be considered bulky, but a page of the book would be platy.

Bulky shapes are subdivided depending on the amount of weathering that has acted on them. They may be angular, subangular, subrounded, or rounded (Figure 2-1). The angular shape shows flat surfaces, jagged projections, and sharp ridges. The rounded shape has smooth curved surfaces and almost approaches a sphere. Cobbles, gravel, sand, and silt usually fall into the bulky shape group.

Platy shapes have one dimension relatively small compared to the other two. The faces at approximately right angles to this dimension will have relatively large areas, whereas the other surface will have edges and present small surfaces. Particles of clay soil exhibit this shape although they are too small to be seen with the naked eye. Coarse-grained soil particles are individually discernible to the naked eye. Fine-grained particles with platy or bulky shapes are not.

Soil Gradation

The size and shape of the soil particles discussed above deal with properties of the individual grains in a soil mass. Gradation describes the distribution of the different size groups within a soil sample. The soil may be well graded or poorly graded.

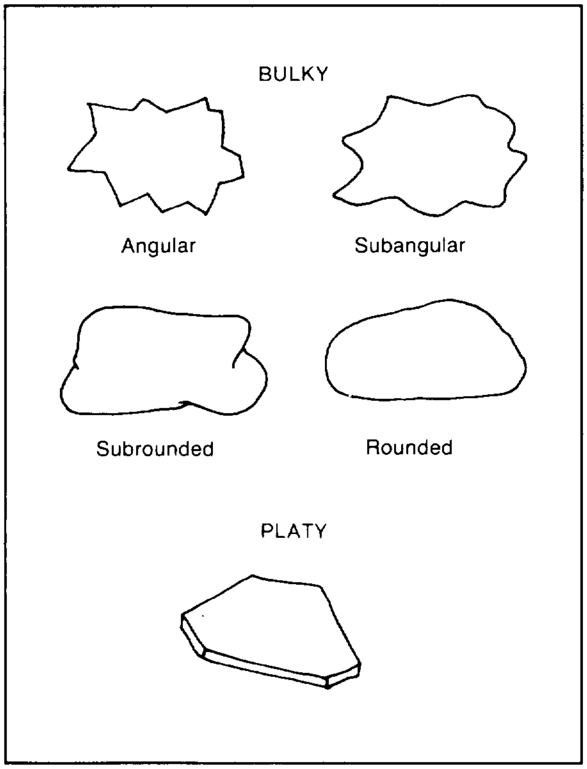


Figure 2-1. Shapes of soil grains

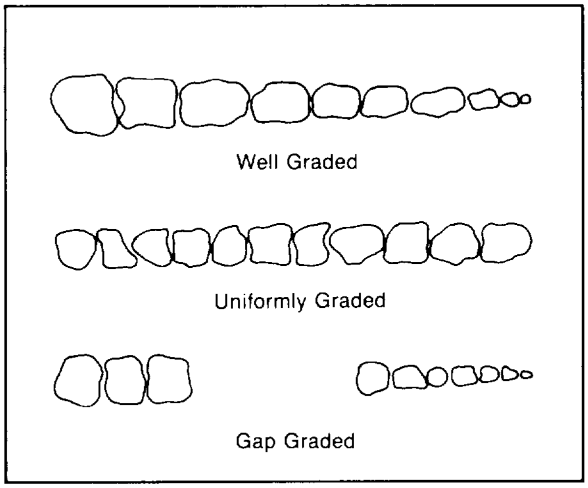


Figure 2-2. Types of soil gradations

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

Poorly-graded soils are either those containing a narrow range of particle sizes or those lacking some intermediate sizes (Figure 2-2). Soils with a limited range of particle sizes are called uniformly graded. Soils which have some intermediate size or sizes not well represented or missing are called gap graded, step graded, uniformly graded, or skip graded.

Compactness

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

Specific Gravity

The specific gravity is the ratio between the weight per unit volume of the material and the weight per unit volume of water at a stated temperature, usually 20°C. There are three ways of determining and expressing specific gravity; specific gravity of the solids, the apparent specific gravity, or the bulk specific gravity. Each kind of specific gravity is explained further, and test procedures are given for their determination in Section IV of this chapter. The specific gravity of the solid substance of most inorganic soils varies between 2.60 and 2.80. Tropical iron-rich laterite as well as some lateritic soils will generally have a specific gravity of 3.0 or more. Sand particles composed of quartz have a specific gravity around 2.65. Clays generally range from 2.68 to 2.75, but can have values as high as 3.50. The solids of soil particles are composed of minerals. Generally these minerals will have a specific gravity greater than 2.60. Therefore, values of specific gravity smaller than 2.60 indicate the possible presence of organic matter.

Moisture

The moisture content of a soil mass is often the most important factor affecting the engineering behavior of the soil. The water may enter from the surface or may move through the subsurface layers either by gravitational pull, by capillary action, or by hygroscopic action. This moisture influences various soils differently and usually has its greatest effect on the behavior of fine-grained soils such as silts and clays.

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

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 a bedrock or an impervious layer of soil whose openings or voids are so small they prevent water passage.

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

- Absorption by the atmosphere is another force acting on soil water. As the moisture evaporates from the soil's surface, it draws more moisture from the soil below. This process will continue until the soil moisture reaches an equilibrium with the water vapor in the air. The amount of water held by the soil is called the hygroscopic moisture.

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

$$\%w = \frac{\text{wt of water}}{\text{wt of dry soil}} \times 100$$

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

Cohesive soils

A cohesive soil has considerable strength when air dried, but has low strength when its moisture content is high. Such clay soils are composed of fine grained (less than 0.002 millimeter) particles of clay minerals. Clay particles usually have flat, platy shapes and are capable of holding a film of adsorbed water on their surfaces. Adsorbed water is held by physicochemical forces and has properties substantially different from ordinary or chemically combined water. The attraction exerted by clay particles for water molecules gives these materials plasticity. Plasticity is a property of the fine-grained portion of a soil that allows it to be deformed beyond the point of recovery without cracking or appreciable volume change. This property permits a clay to be rolled into thin threads at some moisture content without crumbling. Note that only clay minerals possess this property. Some minerals, such as quartz powder, cannot be made plastic no matter how fine the particles nor how much water is added. Thus, the degree of plasticity is a general index to the clay content of a soil. The terms fat and lean are sometimes used to distinguish between highly plastic and moderately plastic soils.

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

- The liquid limit (LL) is the moisture content at an arbitrary limit between the liquid and plastic states of consistency of a soil. Above this value, the 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 the soil exhibits a plastic state.

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

- The shrinkage limit is the boundary moisture content when the 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. The limit tests are described in Section VIII of this chapter.

Organic Soils

Soils having a high content of organic material are described as organic soils. They usually are very compressible and have poor load-maintaining properties.

Effects of Soil Characteristics

The soil characteristics are a measure of the soil's suitability to serve some intended purpose. Generally, a dense soil will withstand greater applied loads (have 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, and 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 the consistency limits aid in describing the soil's suitability. A coarse-grained sandy or gravelly soil generally has good drainage characteristics and may be used in its natural state. A fine-grained clayey soil with a high plasticity index may require considerable treatment, especially if used in a moist location.

SOIL CLASSIFICATION

Soils seldom exist separately as sand, gravel, or any other single component in nature. They are usually mixtures with varying proportions of different sized particles. Each component contributes to the characteristics of the mixture. The USCS is based on the characteristics which indicate how a soil will behave as a construction material.

The physical properties determined by appropriate tests and calculations are used to classify the soil. The criteria for identifying the different soil types are given in Appendix A and in the following paragraphs. The chart of Figure A-1 (Appendix A) shows the procedures to be followed. Categories shown in the chart include soil groups, group symbols, and typical soil names; laboratory classification criteria; field identification procedures; and information required for describing soils. These charts provide a simple, systematic means of soil classification. Once an accurate classification is attained, the potential behavior of the soil under traffic or foundation loads can be judged.

Categories

In the system, all soils are divided into three major categories: coarse grained, fine grained, and peat. The first two are differentiated by grain size, whereas the third is identified by the presence of large amounts of organic material.

Groups

Each of the major categories is further subdivided into groups and a letter symbol is assigned to each group. For example, the symbol assigned to gravel is G, to sand is S, to silt is M, and to clay is C. Refinements in the system also distinguish between well-graded (W) and poorly-graded (P) soils, and between high-compressibility (H) and low-compressibility (L) soils. Highly organic soils, such as peat, are symbolized Pt, whereas organic silts and clays use the letter O as a prefix. The following symbols are combined to describe soil mixture.

Soil Groups	Symbol
Gravel	G
Sand	S
Silt	M
Clay	C

Soil Characteristics	Symbol
Well graded	W
Poorly graded	P
High compressibility	H
Low compressibility	L
Organic (peat)	Pt
Organic (silts and clays)	O
Liquid limits under 50	L
Liquid limits over 50	H

NOTE

Soil mixtures are identified by combining the symbols for each component.

COARSE-GRAINED SOILS

Coarse-grained soils are defined as those in which at least half the material by weight is larger than (retained on) a No. 200 sieve. They are divided into two major divisions: gravels and sands. A coarse-grained soil is classed 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 practice 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) that 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) meaning that such soils will be classed with soils having from 5 to 12 percent fines.

Coarse-Grained Soils with Less Than Five Percent Nonplastic Fines

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

- GW and SW groups. The GW groups include well-graded gravels and gravel-sand mixtures which contain little or no nonplastic fines. 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. The SW groups contain well-graded sands and gravel-sands with little or no plastic fines.

- GP and SP groups. The GP group includes poorly-graded gravels and gravel-sand mixtures containing little or no nonplastic fines. The SP group contains poorly-graded sands and gravel-sands with little or no nonplastic fines. These soils will not meet the gradation requirements established for the GW and SW groups.

Coarse-Grained Soils Containing More Than 12 Percent Fines

These soils may fall into the groups designated GM, GC, SM, and SC. The use of the symbols M and C is based upon the plasticity characteristics of the material passing the No. 40 sieve. The liquid limit and plasticity index are used in specifying the laboratory criteria for these groups. Reference also is made to the plasticity chart shown in Table A-1 which is based upon established relationships between the liquid limit and plasticity index for many different fine-grained soils. The symbol M is used to indicate that the material passing the No. 40 sieve is silty in character. An M usually designates a fine-grained soil of little or no plasticity. The symbol C is used to indicate that the binder soil is predominantly clayey in character.

- GM and SM groups. Typical of the soils included in the GM group are silty gravels and gravel-sand-silt mixtures. Similarly, the SM group contains silty sands and sand-silt mixtures. For both these groups, the Atterberg limits must plot below the A-line of the plasticity chart (Table A-1, Page A-2), or the plasticity index must be less than 4.

- GC and SC groups. The GC group includes clayey gravels and gravel-sand-clay mixtures. Similarly, SC includes clayey sands and sand-clay mixtures. For both these groups, the Atterberg limits must plot above the A-line with a plasticity index of more than 7. See Section VIII of this chapter.

Borderline Soil Groups

Coarse-grained soils which contain between 5 and 12 percent of material passing the No. 200 sieve are classed as borderline and are given a dual symbol (for example, GW-GM). Similarly, coarse-grained soils which contain more than 12 percent of material passing the No. 200 sieve, and for which the limits plot in the shaded portion of the plasticity chart (Table A-1), are classed as borderline and require dual symbols (for example, SM-SC). It is possible, in rare instances, for a soil to fall into more than one borderline zone. If appropriate symbols were used for each possible classification, the result would be a multiple designation consisting of three or more symbols. This approach is unnecessarily complicated. It is considered 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 LL 28 and PI 9, would be designated as SW-SC. If the Atterberg limits of this soil were plotted in the shaded portion of the plasticity chart (for example, LL 20 and PI 5), the soil would be designated either SW-SC or SW-SM, depending on the judgment of the engineer, from the standpoint of the climatic region.

FINE-GRAINED SOILS

Fine-grained soils are those in which more than half the material is smaller than (passes) a No. 200 sieve.

Classification Criteria

The fine-grained soils are not classified on the basis of grain-size distribution, but according to plasticity and compressibility. Laboratory classification criteria are based on the relationship between the liquid limit and plasticity index designated in the plasticity chart in Table A-1. This chart was established by the determination of limits for many soils, with an analysis of the effect of limits upon physical characteristics. The L groups, which have liquid limits less than 50, and the H groups, which have liquid limits greater than 50, are the two major groupings of fine-grained soils. The symbols L and H have general meanings of low and high compressibility, respectively.

Grouping of Fine-Grained Soils

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

- ML and MH groups. Typical soils of the ML and MH groups are inorganic silts. Those of low compressibility 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 may extend into the ML group when their liquid limits are less than 50. This is true of certain types of kaolin clays which have low plasticity. Plastic silts will fall into the MH group.

- CL and CH groups. In these groups, the symbol C stands for clay, while L and H denote low or high liquid limits. These soils plot above the A-line and are principally inorganic clays. The CL group includes gravelly clays, sandy clays, silty clays, and lean clays. The CH group contains inorganic clays of high plasticity including fat clays, the gumbo clays of the southern United States, volcanic clays, and bentonite. The glacial clays of the northern United States cover a wide area in the CL and CH groups.

- OL and OH groups. The soils in these two groups are characterized by the presence of organic matter, hence the symbol, O. All of these soils generally plot below the A-line. Organic silts and organic silt-clays of low plasticity fall into the OL group. Organic clays of high plasticity plot in the OH zone of the plasticity chart. Many of the organic silts, silt-clays, and clays deposited by the rivers along the lower reaches of the Atlantic seaboard have liquid limits above 40 and plot below the A-line. Peaty soils may have liquid limits of several hundred percent and will plot well below the A-line due to their high percentage of decomposed vegetation. A liquid limit test, however, is not a true indicator where a considerable portion consists of other than soil matter.

- Borderline soils. Fine-grained soils with limits which plot in the shaded portion of the plasticity chart are borderline cases and are given dual symbols (for example, CL-ML). Several soil types, exhibiting low plasticity, plot in this general region on the chart where no definite boundary between silty and clayey soils exists.

HIGHLY ORGANIC SOILS

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

FIELD IDENTIFICATION

Lack of time and facilities often makes laboratory soil testing impossible in military construction. Even where laboratory tests are to follow, field identification tests can reduce the number of laboratory test samples to a minimum.

Preparation

Several simple tests are used in field identification. The number of tests depends on the type of soil 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 method of acquiring experience. Lacking such assistance, experience is gained by getting the feel of soils during laboratory testing. An approximate identification can be made by examining a dry sample spread on a flat surface. All lumps should be pulverized until individual grains are exposed, but not broken, since this will change the grain size and the character of the soil. A rubber-faced or wooden pestle and a mixing bowl are recommended, but mashing the sample underfoot on a smooth surface will suffice for an approximate identification. Examination of the characteristics of the particles in the sample makes it possible to assign the soil to one of the three principle groups; coarse-grained, fine-grained, or organic soils. Classification derived from these tests should be recognized as approximations.

An approximate identification of a coarse-grained soil is made by observing grain size, gradation, grain shape, and hardness of the particles.

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

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

Equipment Required

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

Sieves. At least the No. 40, and possibly the No. 4 and the No. 200.

Digging rock. A hand earth auger or posthole digger is useful to secure samples from depths a few feet or more below the surface.

Stirrer.

Knife.

Mixing bowl and pestle.

Paper.

Pan and heating element.

Scales or balances.

Factors Considered

The USCS considers three soil properties: the percentage of gravel, sand, or fines; the shape of the grain-size distribution curve; and the plasticity. These are the primary factors to be considered, but other observed properties, whether made in the field or in the laboratory, should also be included in the soil description. The following information can be invaluable in the design phases:

Color.

Grain size; estimated maximum grain size and estimated percent by weight of fines (material passing No. 200 sieve).

Gradation.

Grain shape.

Plasticity.

Predominant soil type.

Secondary components.

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).

Example of Soil Description

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

Dark brown to white.

Coarse grained, maximum particle size 3 inches, estimated 60 percent gravel, 36 percent sand, and 4 percent fines passing through No. 200 sieve.

Poorly graded (gap-graded, insufficient fine gravel).

Gravel particle subrounded to rounded.

Nonplastic.

Predominantly gravel.

With considerable sand and a small amount of nonplastic fines (silt).

GP (classification symbol).

Slightly calcareous, no dry strength, dense in the undisturbed state.

TESTS FOR FIELD CLASSIFICATION

The following tests produce observations that pertain to the Unified Soil Classification System (Table A-1, Appendix A), and permit field identification as well as classification. Tests appropriate to the given soil sample should be made. Some tests appear to yield duplicate results. The purpose of these tests is to get the best possible identification in the field. Thus, if a simple visual examination will define the soil type, only one or two of the other tests have to be made to verify the identification. When the results from a test are inconclusive, some of the similar tests should be tried to establish the best identification.

Visual Examination

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

- Color. Color helps in distinguishing between soil types, and with experience, aids in identifying the particular soil type. Color may also indicate the presence of certain chemicals or impurities. Color often varies with the moisture content of a soil. 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, 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 result from the presence of iron oxides. White to pink may indicate considerable silica, calcium carbonate, or aluminum compounds.

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

- Grain shape. While the sample is examined for grain sizes, the shapes of the visible particles can be determined. Sharp edges and flat surfaces indicate angular shape, while smooth, curved surfaces indicate rounded shape. Particles may not be completely angular nor completely rounded. These particles are called subangular or subrounded depending on which shape predominates.

- Distribution of grain sizes. Laboratory analysis must be performed when accurate distribution is to be determined. However, an approximation can be made during the visual examination.

Separate the larger grains (gravel and some sand particles) from the remainder of the soil by picking them out individually.

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

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).

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

Estimate the percentage of the fine-grained portion of the coarse-grained soil. If less than 5 percent (nonplastic fines) of the total, the soil may be classified either as a GW, GP, SW, or SP type, depending on the other information noted above.

If the fine-grained portion exceeds 12 percent, the soil will be either M or C and requires further testing to identify.

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

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

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

Breaking or Dry Strength Test

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

Separate the selected sample of the soil on the No. 40 sieve and prepare a pat of soil about 2 inches in diameter and ½ inch thick by molding it in a wet, plastic state. Natural samples may be found in pats which are of the proper size, but may yield incorrect results. This is due to the variations in the natural drying and compaction processes. If natural samples are used, the results must be treated as approximations and verified later.

Allow the pat to dry completely, then grasp the pat between the thumbs and forefingers of both hands and attempt to break it. See Figure 2-3 for the proper way to hold the pat. If the pat breaks, try to powder it by rubbing it between the thumb and forefinger of one hand.

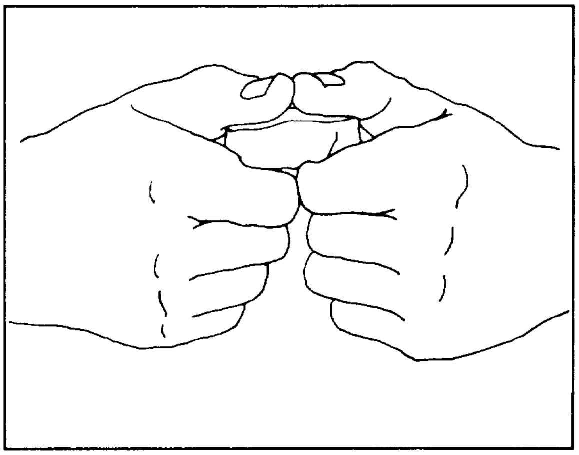


Figure 2-3. Breaking or dry strength test

The results of this test indicate the following:

- Pat cannot be broken nor powdered by finger pressure—Very highly plastic soil (CH).

- Pat can be broken with great effort, but cannot be powdered—Highly plastic soil (CL).

- Pat can be broken and powdered, but with some effort—Medium plastic soil (CL).
- Pat breaks quite easily and powders readily—Slightly plastic soil (ML, MH, or CL).
- Pat has little or no dry strength and crumbles or powders when picked up—Nonplastic soil (ML or MH) or (OL or OH).

NOTE

Dry pats of highly plastic clays quite often display shrinkage cracks. To break the sample along such a crack will give 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.

Roll or Thread Test

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

Prepare a nonabsorbent rolling surface by placing a sheet of glass or heavy wax paper on a flat or level support, then shape the sample into an elongated cylinder and roll the prepared soil cylinder on the surface rapidly into a thread approximately ⅛ inch in diameter. The technique is shown in Figure 2-4. If the moist soil rolls into a thread, it is said to have 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 which cannot be rolled in this manner are nonplastic or have a very low plasticity.

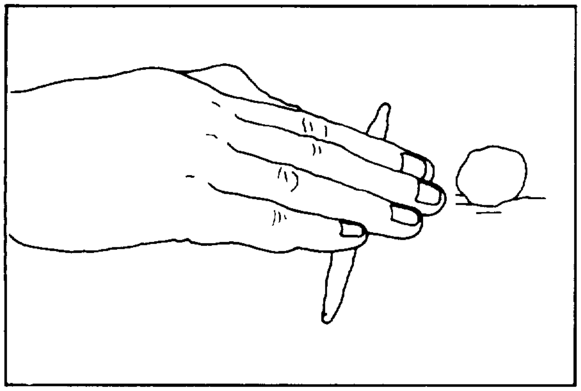


Figure 2-4. Roll or thread test

The results of this test indicate the following:

- Soil may be molded into a ball or cylinder and deformed under very firm finger pressure without crumbling or cracking—High plasticity (CH).
- Soil may be molded, but it cracks or crumbles under finger pressure—Medium plasticity (CL).
- Soil cannot be lumped into a ball or cylinder without breaking up—Low plasticity (CL, ML, or MH).
- Soil forms a soft, spongy ball or thread when molded—Organic material (OL or OH), also peat.
- Soil cannot be rolled into a thread at any moisture content—Nonplastic soil (ML or MH).

NOTE

Micaceous silts and sands can be rolled due to the flaky nature of the mica. The wet shaking test is the only way to distinguish this property.

From the thread test, the cohesiveness of the material near the plastic limit may also be described as weak, firm, or tough. The higher the soil is on the plasticity chart (Table A-1, Appendix A), the stiffer the threads are as they dry out and the tougher the lumps are if the soil is remolded after rolling.

Ribbon Test

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

Form a roll of soil about ½ to ¾ inch in diameter and 3 to 5 inches long. Lay the roll across the palm of one hand (palm up), and 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 ⅛ to ¼ inch thick. Allow the ribbon as formed to hang free and unsupported (Figure 2-5). 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.

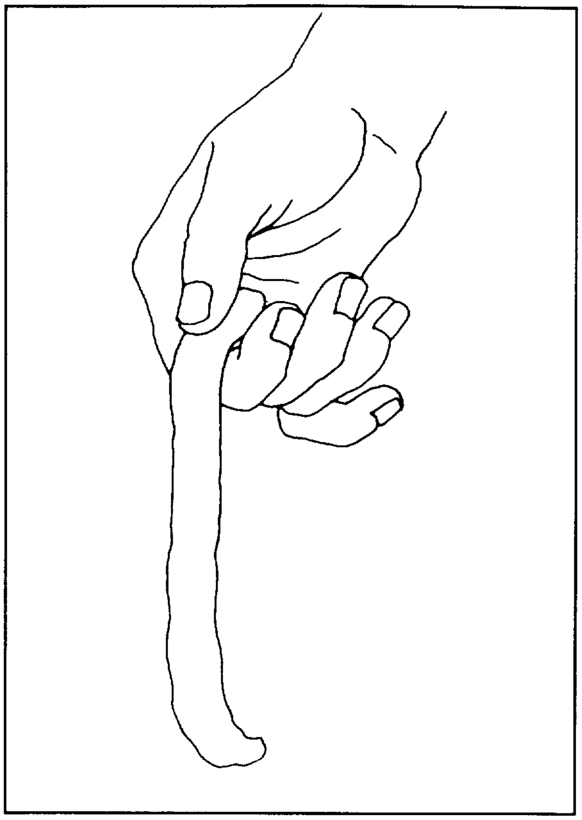


Figure 2-5. Ribbon test

The results of this test indicate the following:

- Sample holds together for a length of 8 to 10 inches without breaking—Highly plastic and highly compressive (CH).
- Soil can be ribboned only with difficulty into 3-inch to 8-inch lengths—Low plasticity (CL).

Wet Shaking Test

The wet shaking test is performed only on the material passing the No. 40 sieve. For this test, enough material to form a ball of material about ¾ inch in diameter is moistened with water. This sample should be just wet enough so that the soil will not stick to the fingers upon remolding or just below the sticky limit.

Smooth the soil pat in the palm of the hand with the blade of a knife or 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 (Figure 2-6, 1 and 2). When the sample is squeezed between the thumb and forefinger of the other hand, the surface water will quickly disappear. The surface will become dull (Figure 2-6, 3), and the material will become firm, resisting deformation. Cracks will occur as pressure is continued, with the sample finally crumbling like a brittle material (Figure 2-6, 4). The vibration caused by the shaking of the soil sample tends to reorient the soil grains, decrease the voids, and force water which had been within these voids 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.

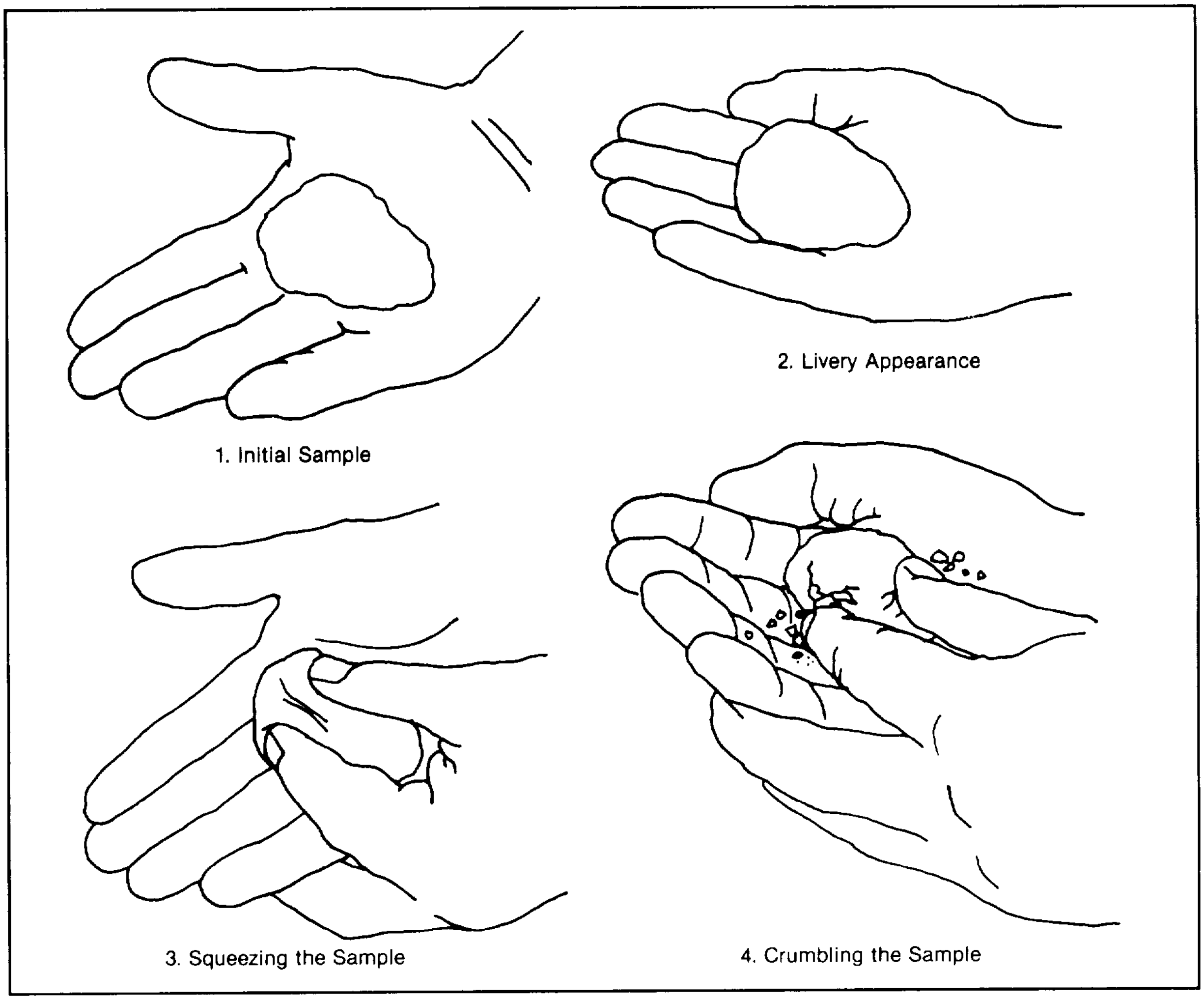


Figure 2-6. Wet shaking test

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 will tend to greatly retard this reaction.

The results of this test indicate the following:

- A rapid reaction to the shaking test is typical of nonplastic, fine sands and silts (Figure 2-6, 2).
- A sluggish reaction indicates slight plasticity (such as might be found from a test of some organic silts) or silts containing a small amount of clay.
- No reaction at all to this test does not indicate a complete absence of silt or fine sand.

Odor Test

Organic soils (OL and OH) usually have a distinctive, musty, slightly offensive odor. The odor can be used as an aid in identifying 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.

Bite or Grit Test

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

The results of this test indicate the following:

- Sandy soils. The sharp hard particles of sand will grate very harshly between the teeth and will be 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 will stick when lightly touched with the tongue.

Shine Test

This test is another means of determining plasticity of a soil. A slightly moist or dry piece of highly plastic clay will produce a definite shine when rubbed with a fingernail or a smooth metal surface such as a knife blade. Lean clay will remain dull after this treatment.

Feel Test

This is a general purpose test and requires 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 which can be determined.

- Consistency. The natural moisture content is an indicator of the soil drainage which may affect this characteristic.

For the consistency test, squeeze a piece of undisturbed soil between the thumb and forefinger to determine its consistency. Consistency is described by such terms as hard, stiff, brittle, friable, sticky, plastic, and soft.

Remold the soil by working it between the hands and observe the results. This can indicate the natural water content. Clays which become fluid on remolding are probably near their liquid limit. If they remain stiff and crumble when reworked, they are probably below the plastic limit.

- Texture. 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 and 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 will feel gritty. Silts, if dry, will dust readily and feel soft and silky to the touch. Clays powder only with difficulty, but feel smooth and gritless like flour.

Sedimentation Test

From the visual observation test, 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 is more difficult although just as important. Smaller particles will settle through water at a slower rate than large particles. Placing a small amount of the fine fraction of soil (such as a heaping tablespoon) in a transparent cup or jar, covering it with about 5 inches of water, and agitating it by stirring or shaking will completely suspend the soil in water. With most cohesive soils, it will be necessary to break up the lumps of soil before adding the water. This can be done by grinding the soil in a canteen cup with an improvised wood pestle. After the soil particles have been thoroughly dispersed in the water and then left, they will start to settle out, beginning with the larger sized particles, in time periods indicated in Table 2-2.

Table 2-2. Sedimentation test

Approximate time of settlement through 5 inches of water	Grain diameter (mm*)	Differentiates
2 seconds	0.4	Coarse sand-fine sand
30 seconds	0.072	Sand-fines
10 minutes	0.03	Coarse silt-fine silt
1 hour	0.01	Silt-clay

* Millimeter

The most important use of the sedimentation test is to differentiate the coarse (0.072 millimeter) fraction from the fine fraction of a soil. Since all of the particles of soil larger than 0.072 millimeter will have settled to the bottom of the cup or jar 30 seconds after the mixture has been agitated, it follows that the particles still remaining in suspension are fines. If the water containing the suspended fines is carefully poured into another container 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, and the pan 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.

Most field identification tests are performed on the No. 40 sieve soil fraction (fines and fine sand portion). This fraction can be separated by using a procedure similar to that already outlined except that the water is poured off within 1 or 2 seconds after completion of agitation. The suspended portion will then include the particles of fine sand range.

A difficulty is encountered with many clay soils because the clay particles will 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 help break up these lumps.

Identification Sequence

Figure 2-7 gives the suggested sequence of tests for identifying and classifying a soil sample using the rapid field procedures just described.

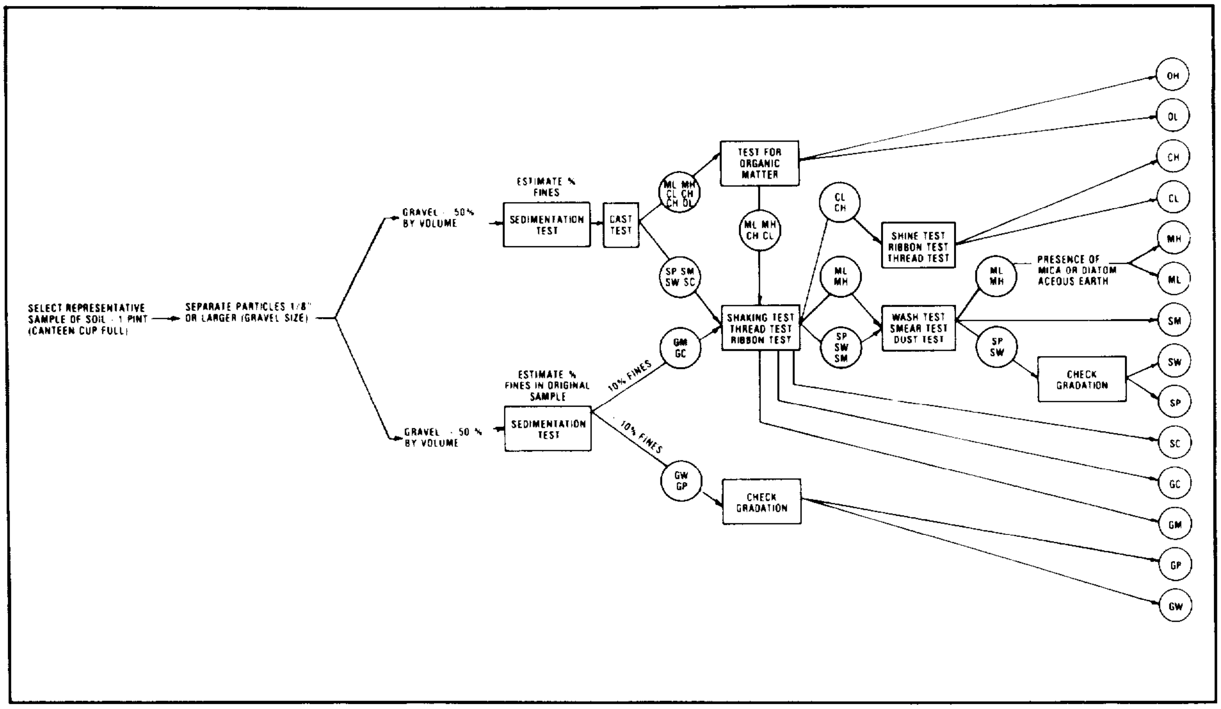

SELECT REPRESENTATIVE SAMPLE OF SOIL - 1 PINT (CANTEEN CUP FULL)
SEPARATE PARTICLES 1/8" OR LARGER (GRAVEL SIZE)
GRAVEL - 50% BY VOLUME
ESTIMATE % FINES
SEDIMENTATION TEST
CAST TEST
ML MH CL CH CH OL
SP SM SW SC
TEST FOR ORGANIC MATTER
ML MH CH CL
SHAKING TEST THREAD TEST RIBBON TEST
CL CH
ML MH
SP SW SM
SHINE TEST RIBBON TEST THREAD TEST
WASH TEST SMEAR TEST DUST TEST
ML MH
PRESENCE OF MICA OR DIATOM ACEOUS EARTH
SP SW
CHECK GRADATION
GRAVEL - 50 % BY VOLUME
ESTIMATE % FINES IN ORIGINAL SAMPLE
SEDIMENTATION TEST
10% FINES
10% FINES
GM GC
GW GP
CHECK GRADATION
OH
OL
CH
CL
MH
ML
SM
SW
SP
SC
GC
GM
GP
GW

Figure 2-7. Suggested procedures for hasty field identification

Section II. SOIL SURVEYS AND SAMPLING

DISCUSSION

The survey of soil conditions at the site of proposed military construction provides information about the nature, extent, and condition of soil layers; the position of the water table and drainage characteristics; and sources of possible construction materials. The soil survey is vital to both the planning and execution of military construction operations.

TYPES OF SOIL SURVEYS

A soil survey consists of gathering soil samples for examining, testing, and classifying soils and developing a soil profile. Two types of soil surveys are the hasty survey, which is made either under expedient conditions or when time is very limited, and the deliberate survey, which is made when adequate equipment and time are available.

Hasty Survey

The hasty survey should be preceded by as careful a study of all available sources of information as conditions permit. If possible, from the air a trained person may observe soil conditions in the proposed construction area. Careful aerial observation gives an overall picture, which is often difficult to secure at ground level because important features may be obscurred in rough or wooded terrain. Rapid ground observation along the proposed road location or at the proposed airfield site will also yield useful information. Observation of the soil profile may be made 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 modification in location, design, and construction.

Deliberate Survey

The deliberate survey does not dismiss the fact that the time factor may be important. Therefore, the scope of a deliberate survey may necessarily 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 methods of exploration used in soil surveys for roads, airfields, and borrow areas are soil samples obtained either by using hand augers or by digging a test pit. Other methods that may be used are power-driven earth augers, sounding rods, or earth-moving equipment under expedient conditions to permit a hasty approach to the underlying soil.

OBJECTIVES OF A SOIL SURVEY

The objective of a soil survey is to gather (explore) as much information of engineering significance as possible about the subsurface conditions for a specified area. The explorations are conducted to determine the following information:

Location, Nature, and Classification of Soil Layers

These are required for adequate and economical earthwork and foundation design of a structure. By classification of the soils encountered, a prediction can be made as to the extent of problems concerning drainage, frost action, settlement, stability, and similar factors. While an estimate of the soil characteristics may be obtained by field observations, samples of the major soil types as well as less extensive deposits, which may influence design, should be obtained for laboratory testing.

Condition of Soils in Place

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

Drainage

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

Groundwater and Bedrock

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

SOURCES USEFUL IN PLANNING SOIL SURVEYS

There are many sources of information available to the soils engineers. These should all be used to the fullest extent to eliminate as much detailed investigation as possible. These sources will be used to locate small areas, within a large general area, which are suitable for further investigation. For the final site selection actual field investigations must be made. A field party must secure reliable data rapidly, since final decisions are based on field observations.

Intelligence Reports and Local Inhabitants

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

Local inhabitants may provide information to supplement intelligence reports or provide information about areas for which intelligence reports are unavailable. Data obtained from this source would include possible location of borrow material, sand and gravel deposits, and peat or highly organic soils, as well as information on the climate the topography of the area.

Maps

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

- Geological maps and brief descriptions of regions and quadrangles are available from the US Geological Survey (1200 South Eads Street, Arlington, VA 22202). Generally, the smallest rock unit mapped is a formation, and geological maps indicate the extent of these formations by means of symbolic letters, color, or patterns. Letter symbols on the map also indicate the location of sand and gravel pits, and the rear of the map sheet sometimes has a brief discussion entitled "Mineral Resources" describing the location of construction materials.

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

- Agricultural soil maps and reports are available for many of the developed agricultural areas of the world. These studies are concerned primarily with surface soils generally to a depth of 6 feet. Their value as aids in the engineering study of surface soils is apparent. 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 sharply reduced in the second area. Factors considered in the field surveys upon which the maps are based 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. Soils usually are classified according to their texture, color, structure, chemical and physical composition, and morphology.

Air Photographs

The use of air photographs in delineating and identifying soils is based upon the recognition of typical patterns formed under similar conditions of soil profile and weathering. Principal elements which can be identified on a photograph and which provide clues to the identification of soils to a trained observer are landform, slopes, drainage patterns, erosional characteristics, soil color or tone vegetation, and land use. Examples of the information which may be derived from the examination of air photographs follow:

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

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

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

- Erosional 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.

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

- Agricultural land use facilitates identification of soils. 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 usually is found in poorly-draining soils underlain by impervious soils, such as clay. Tea grows in well-draining soils.

FIELD INVESTIGATIONS

The field investigation consists of the sampling operation in the field. The extent and methods used will depend upon the time available. The three principal methods of sampling available to the military engineer are the taking of samples from the surface, from excavations already in existence, and from test pits or test holes. In the hasty survey, the number of test pits and test holes are kept to a minimum by using existing excavations for sampling operations. In the deliberate survey, where a more thorough sampling operation is conducted, test holes are used extensively and are augmented by test pits, governed by the judgment of the engineer.

Methods

- Test pits. A test pit is an open excavation which is large enough for a man 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 usually is dug by hand. Power excavation by dragline, clamshell, bulldozer, backhoe, or a power-driven earth auger can expedite the digging if the equipment is available. Excavations below the groundwater table require the use of 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.

- Test holes. Test hole exploration, using the hand auger, is the most common method of digging test holes. It is best suited to cohesive soils, but can be used on cohesionless soils above the water table, provided the diameter of the individual aggregate particles is smaller than the bit clearance of the auger. The auger borings are principally used for work 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 properties of the soil.

Table 2-3 shows methods of underground exploration and sampling in a condensed form.

Table 2-3. Methods of underground exploration and sampling

Common name of method	Materials in which used	Method of advancing the hole	Method of sampling	Value for foundation purposes
Auger boring	Cohesive soils and cohesionless soils above ground water elevation	Augers rotated until filled with soil and then removed to surface	Samples recovered from material brought up on augers	Satisfactory for highway exploration at shallow depths
Well drilling	All soils, rock, and boulders	Churn drilling with power machine	Bailed sample of churned material or clay socket	Clay socket samples are dry samples Bailed samples are valueless
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 ground water 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

Preparation

The location of auger holes or test pits will depend upon the particular situation. In any case, the method described locates the minimum number of holes. The completeness of the exploration will depend upon the time available. A procedure for road, airfield, and borrow area investigations will be described. Soil tests should be made on samples representing the major soil types in the area.

- First develop a general picture of the subgrade conditions. Field reconnaissance should be made to study landforms and soil conditions in ditches and cuts. Techniques have been developed using aerial photographs for delineating areas of similar soil conditions. Full use should also be made of existing data in agricultural soil maps for learning subsurface conditions.

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

Soil samples should be obtained in these preliminary borings. After these samples are classified, soil profiles should be developed, and representative soils should be selected for detailed testing. Test pits, or large-diameter borings, should then be made to obtain the samples needed for testing, or to permit in-place tests. The types and number of samples required will depend on the characteristics of the subgrade soils. Subsoil investigations in the areas of proposed pavement must include measurements of the in-place water content, density, and strength. These are used to determine the depth of compaction and the presence of any soft layers in the subsoil.

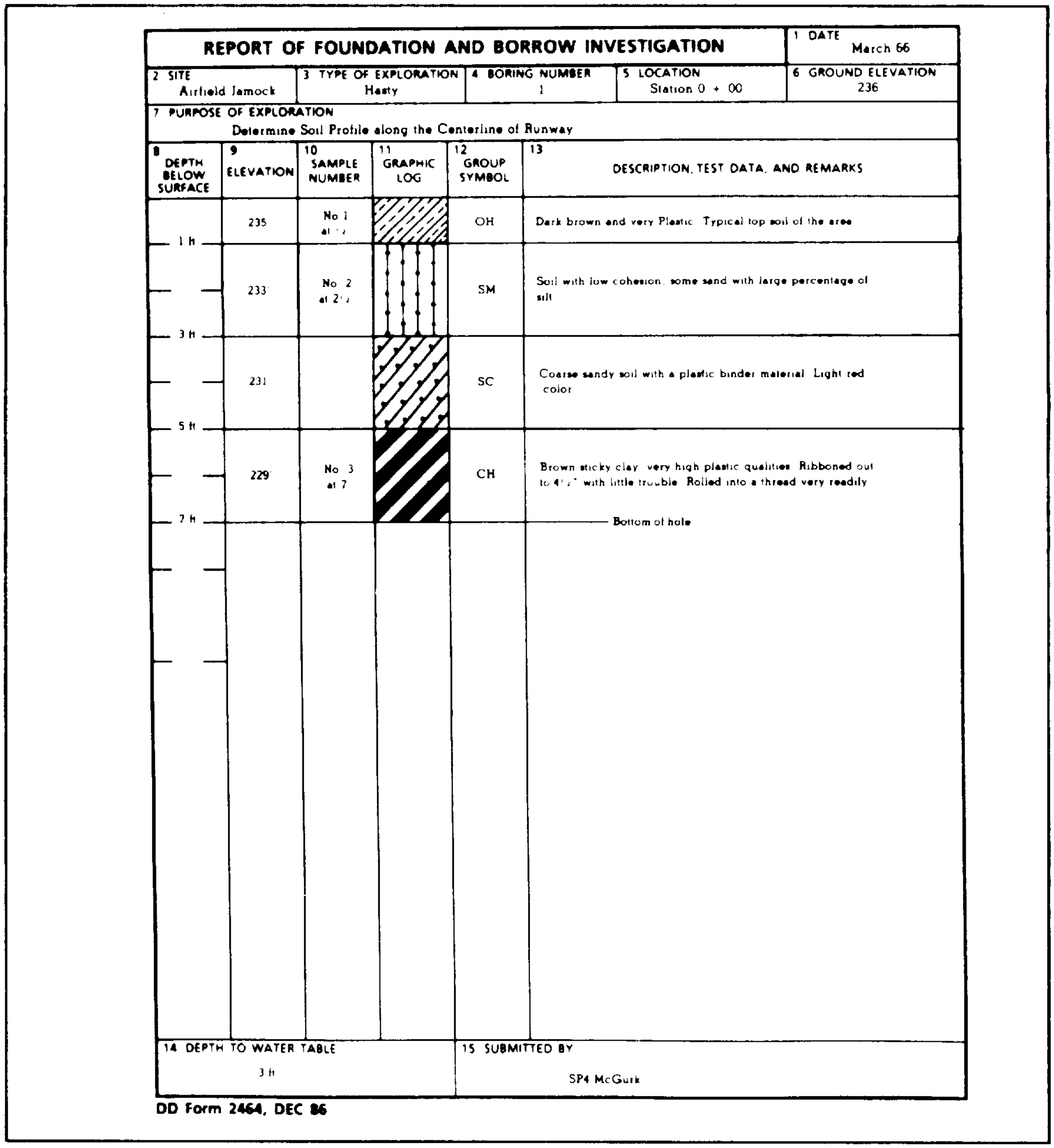
In borrow areas, where material is to be borrowed from adjacent areas, holes should be made and carried 2 to 4 feet below the anticipated depth of borrow. Samples should also be classified and tested for water content, density, and strength.

- Select material and subbase from areas within the airfield site and within a reasonable haul 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 generally are natural materials (unprocessed). Test pits or large borings put down with power augers are needed in gravelly materials.

- Base and pavement aggregates are materials which are generally crushed and processed. A survey should be made of existing producers plus other possible sources in the general area. Significant savings have been 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

The engineer in charge of the soil survey is responsible for properly surveying, numbering, and recording each auger boring, test pit, or other investigations. A log is kept of each test hole which shows 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. Other information which should be included in the log relates to density of each soil, changes in moisture content, depth to groundwater, and depth to rock. A typical boring log is shown as Figure 2-8.



REPORT OF FOUNDATION AND BORROW INVESTIGATION

1 DATE March 66

2 SITE	3 TYPE OF EXPLORATION	4 BORING NUMBER	5 LOCATION	6 GROUND ELEVATION
Airfield Jamock	Hasty	1	Station 0 + 00	236

7 PURPOSE OF EXPLORATION
Determine Soil Profile along the Centerline of Runway

8 DEPTH BELOW SURFACE	9 ELEVATION	10 SAMPLE NUMBER	11 GRAPHIC LOG	12 GROUP SYMBOL	13 DESCRIPTION, TEST DATA, AND REMARKS
1 ft	235	No 1 at ½		OH	Dark brown and very Plastic Typical top soil of the area
3 ft	233	No 2 at 2½		SM	Soil with low cohesion, some sand with large percentage of silt
5 ft	231			SC	Coarse sandy soil with a plastic binder material Light red color
7 ft	229	No 3 at 7		CH	Brown sticky clay very high plastic qualities Ribboned out to 4½" with little trouble Rolled into a thread very readily
					Bottom of hole

14 DEPTH TO WATER TABLE	15 SUBMITTED BY
3 ft	SP4 McGuirk

DD Form 2464, DEC 86

Figure 2-8. Typical boring log

THE SOIL PROFILE

Development

A detailed field log is kept of each auger boring or test pit made during the soil survey. When the survey has been completed, the information contained in the separate logs is consolidated. In addition to the classification and depth of soil layers recorded in each log, it is desirable to show the natural water contents of fine-grained soils along the side of each log. Also, the elevation of the groundwater table should be noted. The elevation is determined during the soil survey by observing the level at which free water stands in the test holes. To get an accurate determination, holes should be covered and inspected 24 hours after being dug to allow the water to reach its maximum level. The soil profile (Figure 2-9) is a graph of a vertical cross section of the soil layers from the surface of the earth downward. It shows the location of test holes, profile of the natural ground to scale, location of any ledge rock encountered, field identification of each soil type, thickness of each soil stratum, and profile of the water table.

Uses

The soil profile has many practical uses in the location, design, and construction of 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 will show whether soils to be excavated are suitable for use in embankments, or if borrow soils will be

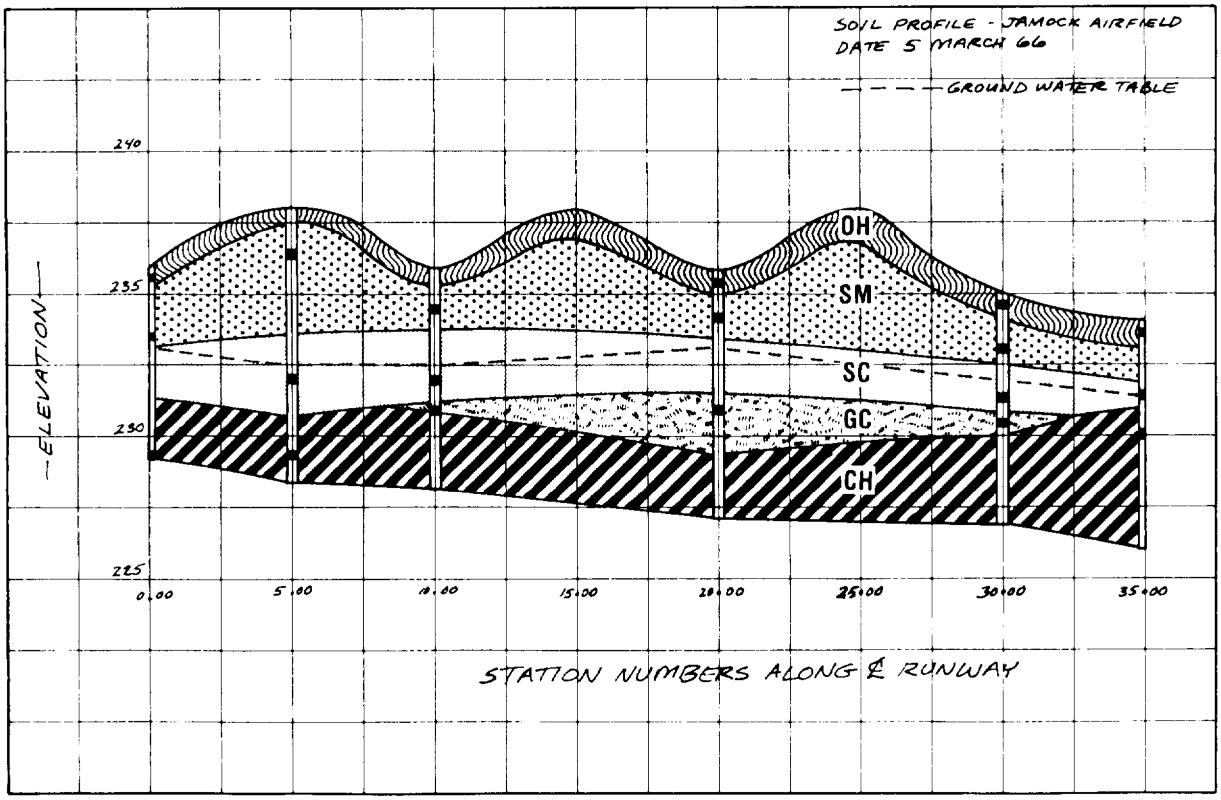


Figure 2-9. Typical soil profile

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 will aid in the planning of drainage facilities to take advantage of the presence of well-draining soils. It may indicate that special drainage installations will be needed with soils which 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.

RECOMMENDED PROCEDURES FOR SOIL SURVEYS

The following guide and step-by-step procedures will help the military engineer when conducting soil surveys.

- Considerations include soil types, securing of samples, density and moisture content of soil in place, drainage characteristics, and depth to groundwater and bedrock.

- Published information includes geological and topographic reports with maps, and agricultural soil bulletins with maps, these require careful interpretation and knowledge of local terms. Aerial photographs used to predict subsurface conditions and previous explorations for nearby construction projects are also useful.

- Field information requires general observation of road cuts, stream banks, eroded slopes, earth cellars, mine shafts, and existing pits and quarries. Test holes may be made with hand auger or power auger if necessary and available. Test pits are necessary where hand auger cannot penetrate or where large samples are required.

- Local inhabitants, preferably trained observers such as contractors, engineers, and quarry workers, can provide valuable information.

Preparation

Planning of the general layout will determine 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.

- Airfield exploration. Place borings at high and low spots, wherever 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 will be less than in a terrain where the soil conditions change frequently.

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

- Depth exploration. Take a cut section 4 feet below subgrade, if possible, and a fill section 4 feet below original ground level, if possible. Effort should be made to locate the groundwater table.

Procedures

- Log the exploration holes or pits.
- Locate and number the samples.
- Determine the elevation and exact location of each hole and tie into the site layout.

PREPARATION OF SOIL PROFILE

This is used to locate grade line, for excavation and grading plan, and for drainage plan. Show the boundaries of soil strata, location and depth of test holes and samples, elevation of bedrock, and water table. Laboratory test results should be included when available. The officer in charge is responsible for seeing that each test pit, boring, or other exploratory excavation is properly surveyed, numbered, and recorded, a log is kept of each test hole or test pit, and each sample is properly numbered, tagged, or otherwise labeled, and prepared for shipment to the laboratory.

Locating Samples

Information obtained from each test pit, boring, or other exploratory excavation must be correlated with data obtained from other sources. In exploration for roads and airfields, locations are established by measurements along the centerline or at right angle offsets to the centerline, as required. Measurements may be made by pacing or taping. The soil test set contains a 100-foot steel tape for this purpose. The location, type, and identification of the exploratory excavation are plotted on a photograph, map, or sketch made in the notebook as illustrated in Figure 2-10. A uniform system should be used for plotting exploratory excavations. The symbols below are in common use.

Type	Abbreviation	Symbol
Pit	P	Square
Boring	B	Circle
Trench	T	Rectangle

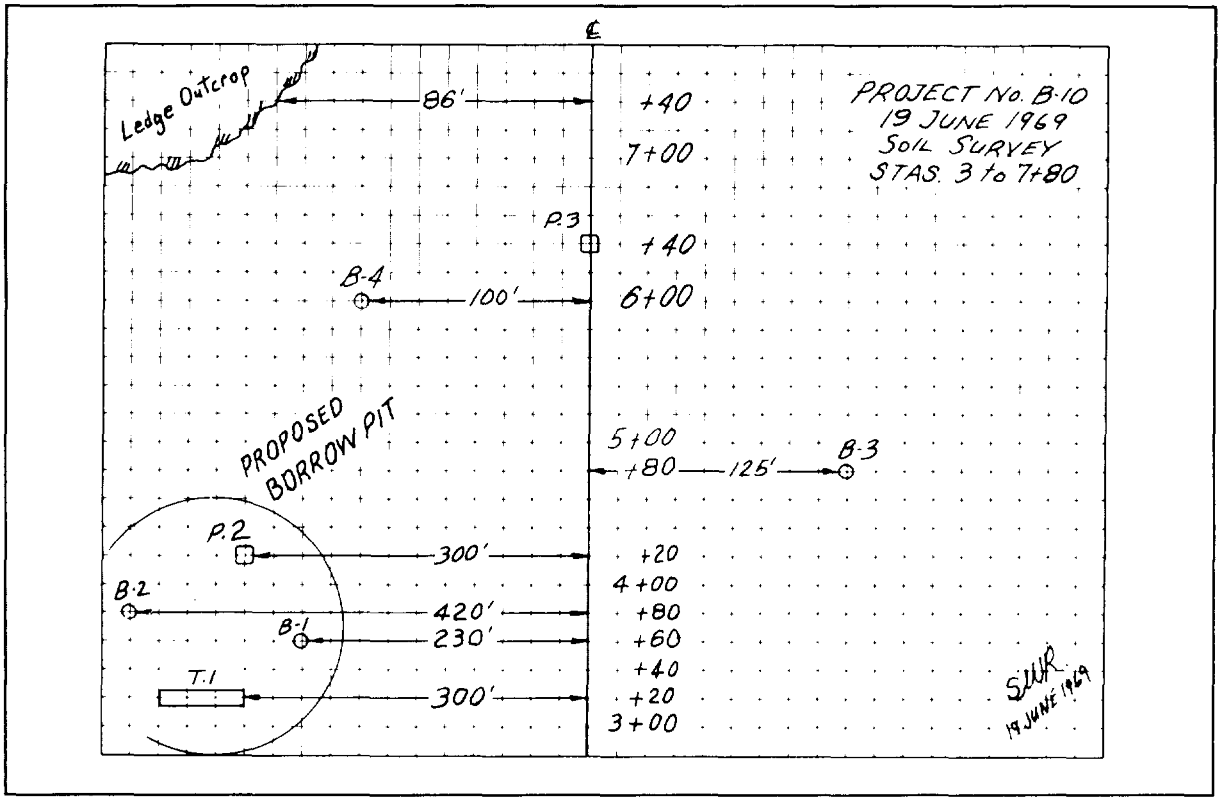
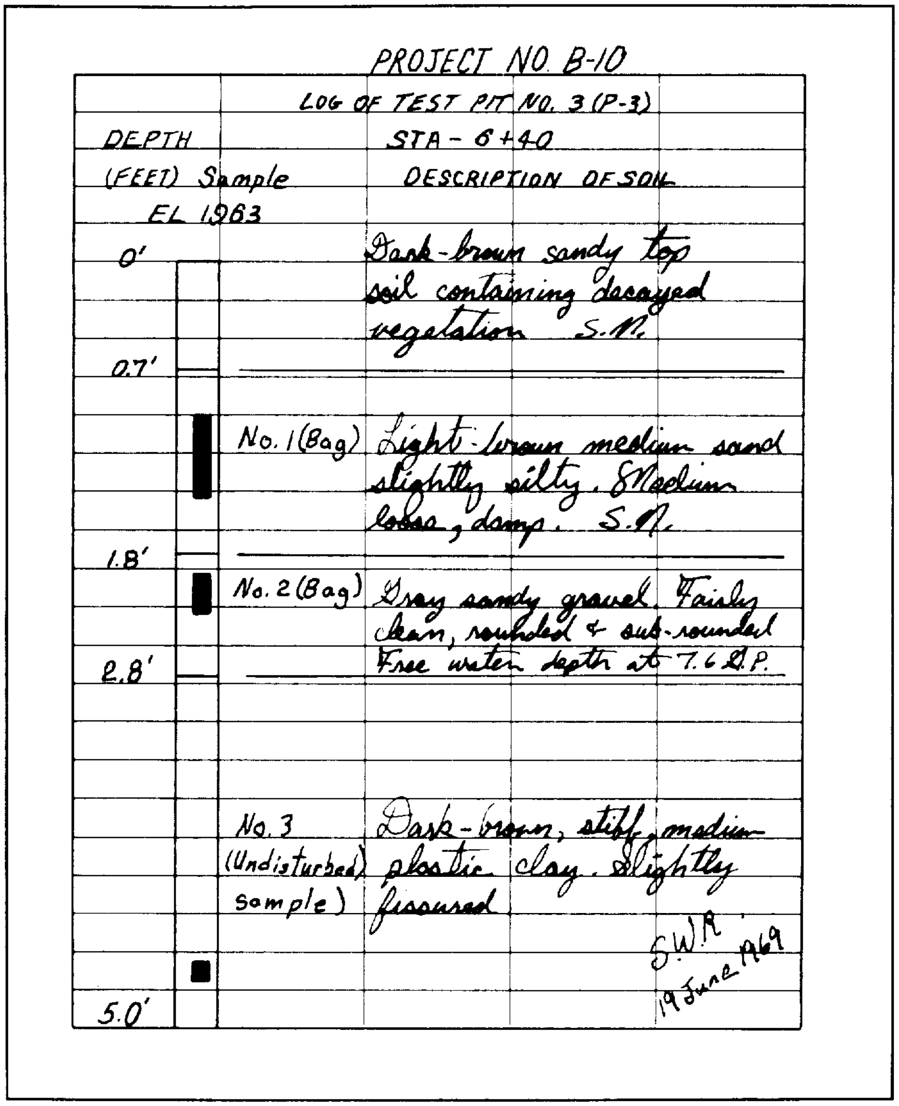

℄
Ledge Outcrop
86'
+40
7+00
PROJECT No. B-10
19 JUNE 1969
SOIL SURVEY
STAS. 3 to 7+80
P.3
+40
B-4
100'
6+00
PROPOSED BORROW PIT
5+00
B-3
+80
125'
P.2
300'
+20
4+00
B-2
420'
+80
B-1
230'
+60
T-1
+40
300'
+20
3+00
SJUR
19 JUNE 1969

Figure 2-10. Typical fieldbook page, locations of soil explorations

Recording Samples

A detailed record or log of each exploratory excavation (test pit) is kept in a field book as shown in Figure 2-11. Logs must be complete and contain all information which may be of value in interpreting the test results and in plotting the soil profiles. Interpretation of the log will be made easier if each of the soils encountered is identified in the field and classified under the USCS. Soil descriptions must be accurate and complete.



PROJECT NO. B-10

LOG OF TEST PIT NO. 3 (P-3)

DEPTH (FEET) EL 1963

Sample

STA - 6+40

DESCRIPTION OF SOIL

0'

Dark-brown sandy top soil containing decayed vegetation S.M.

0.7'

No. 1 (Bag) Light-brown medium sand slightly silty. Medium loose, damp. S.M.

1.8'

No. 2 (Bag) Gray sandy gravel. Fairly clean, rounded & sub-rounded Free water depth at 7.6 D.P.

2.8'

No. 3 (Undisturbed sample) Dark-brown, stiff, medium plastic clay. Slightly fissured

5.0'

S.W.R.
19 June 1969

Figure 2-11. Typical field book page, log of test pits

Numbering Samples

Each sample is identified by the following information:

- A project number which consists of a number, an abbreviation, or a symbol. For example, BF might be used for Benton Field.

- An excavation number which corresponds to the number of the exploratory excavation. For example, P3 would mean that the sample was obtained from test pit number 3.

- A sample number, which is the number of the order in which the sample was obtained from each location. If more than one bag sample is taken from one location, the total number of bags comprising the sample also must be recorded. An example of a complete designation number is as follows:

B10-P3-2
(Bag 1 of 2)

This designation means that the sample was taken on project B10, pit number 3, that it is the second sample taken in this pit, and that the bag is the first of a total of two bags. The sample number is printed with a glazed surface-marking pencil on two tags, one which is placed inside the bag, and the other which is tied on the outside, as shown in Figure 2-12. Gummed labels may be similarly used to mark samples which are contained in moisture content boxes, cylinders, or jars.

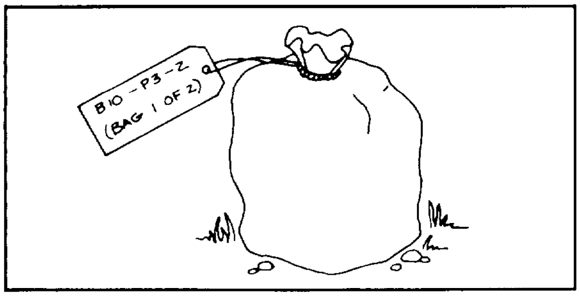


Figure 2-12. Method of labeling bag samples

BAG SAMPLES

Disturbed soil samples taken by means of a shovel, auger, or any other convenient hand tool and placed in bags are known as bag samples. They are used for mechanical analysis, plasticity, specific gravity, frost susceptibility, compaction, and laboratory compacted CBR tests. The size of the sample taken will depend upon the tests to be conducted. Two sizes of bags are issued in the soil test set.

Individual Samples

When exploring foundation conditions, take bag samples of each individual type of soil encountered.

- To take individual samples from a pit, trench, or exposed face, shave off loose and dried soil to obtain a fresh surface and to expose clearly any variations in the soil. Then take a typical sample of each type of soil, or of those types which require further investigation (Figure 2-13).

- When sampling in auger holes, place typical portions of soil obtained along a row in correct order as indicated in Figure 2-14, page 2-30.

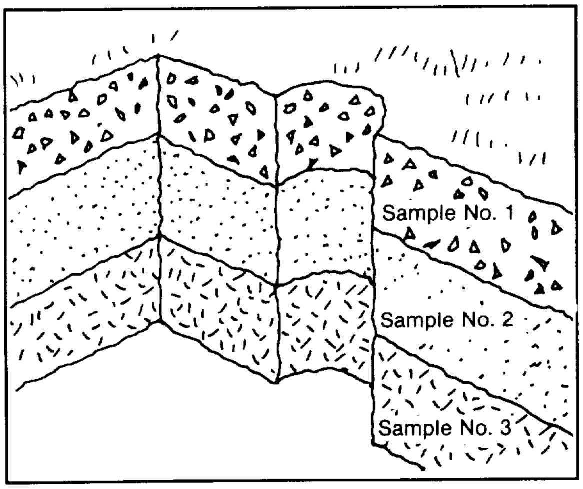


Figure 2-13. Obtaining individual bag samples from an exposed face

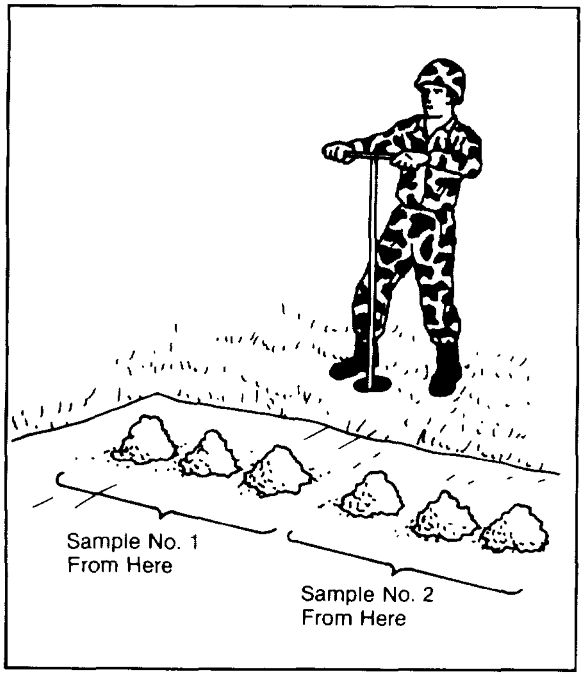


Figure 2-14. Obtaining individual bag samples

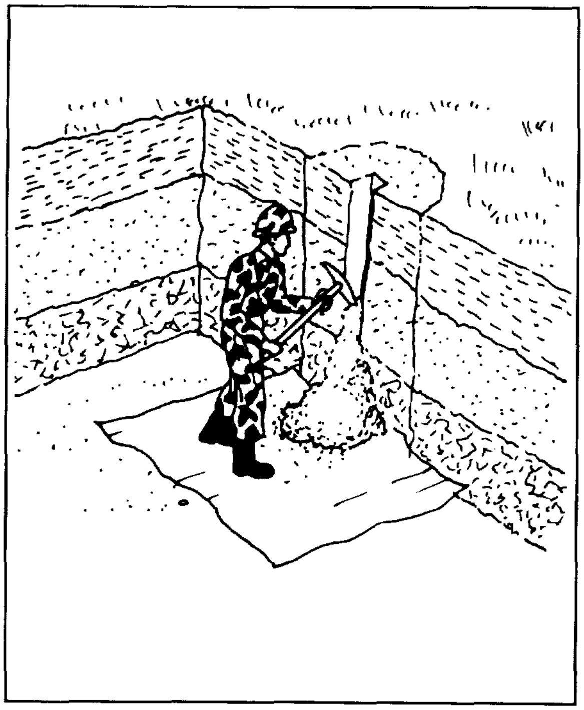


Figure 2-15. Taking a composite sample with an exposed face

Composite Samples

A composite sample is a representative mixture of all soil within a profile or soil mass to be investigated, or of the material contained in a stockpile or windrow of soil excavated from a trench. A test sample is obtained from a composite sample by quartering in the laboratory.

- To take composite samples from test pits, trenches, or power shovel cuts, the following steps are required:

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

Shave off loose and dried soil to obtain a fresh surface for taking the sample.

Spread a quartering cloth or tarpaulin at the top of the bank.

Excavate a channel of uniform cross section from top to bottom (Figure 2-15), and deposit the soil on the canvas.

Collect and bag all material removed to ensure that the sample contains the correct proportions.

- To take composite samples from auger holes, collect all material excavated from the hole after first removing overburden.

- To take representative composite samples from stockpiles or large windrows, take particular care. When material is dumped on large piles the coarse material tends to roll to the bottom, leaving the finer material in the upper portion. The sample is taken from a full height strip after clearing the surface.

- To sample a small windrow, excavate and bag material from a short section, as illustrated in Figure 2-16.

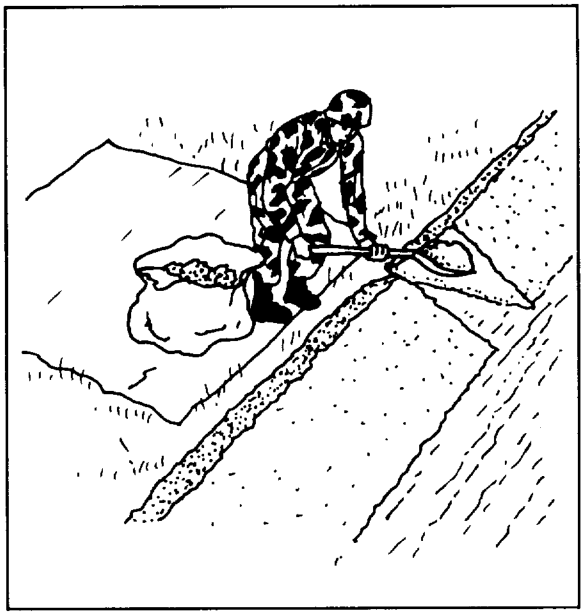


Figure 2-16. Taking a composite sample from a small windrow

MOISTURE CONTENT SAMPLES

The natural moisture content of soil is determined from samples taken in the field and placed in a container which is sealed to prevent loss of moisture 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 is enough to determine the moisture content of fine-grained soils.

Larger samples are required for soils which contain gravel. Three sizes of metal dishes are provided in the soil test set. These have tight fitting covers and do not require sealing if the test is made within one day after the sample is taken. If a longer time interval will elapse between sampling and testing, the boxes may be sealed by the method illustrated in Figure 2-17. Any other clean containers which can be adequately sealed may be used for moisture content samples.

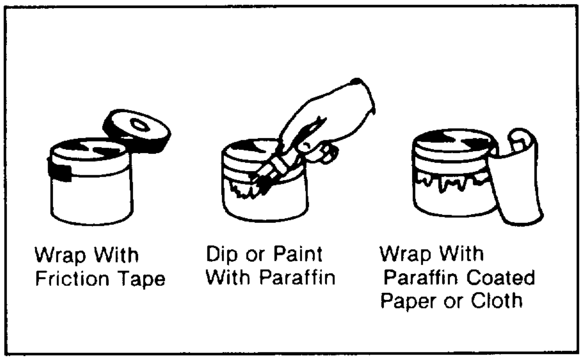

Wrap With Friction Tape
Dip or Paint With Paraffin
Wrap With Paraffin Coated Paper or Cloth

Figure 2-17. Sealing container to retain moisture content of a sample

UNDISTURBED SAMPLES

An undisturbed soil sample is one that is cut, removed, and packed with the least possible disturbance. They are samples in which the natural structures, void ratio, and moisture content are preserved as carefully as possible. Samples of this type are used for determining the density (unit weight) of soil in the laboratory, and 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.

Types of undisturbed samples are chunk samples, cut by hand with shovel and knife, and cylinder samples, obtained by use of a cylindrical sampler or the CBR mold equipped with sampling cutter. Expedient methods of obtaining cylinder samples are also used.

The method of sampling chosen depends upon the equipment available, the tests required, and type of soil. All undisturbed samples must be handled with care. Cohesionless soil samples must be kept in the container until ready for testing, and the container should be handled 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 of 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, the sample must be taken and packed in the container for shipment without allowing its structure to change. Protection against change in moisture content during sampling and shipment is also required.

CHUNK SAMPLES

The simplest type of undisturbed sample is obtained by cutting out a chunk of soil of the desired size and covering it to prevent loss of moisture and breakage. This method can be used only with soils that will not deform, break, or crumble while being removed.

The process of obtaining a chunk sample from a subgrade or other level surface, such as the bottom of a test pit, is illustrated in Figures 2-18, 2-19, and 2-20. The first step is to smooth the ground surface and mark the outline of the chunk. A trench is excavated around the chunk (Figure 2-18). Then the excavation is deepened, and the sides of the chunk trimmed with a knife (Figure 2-19). The chunk is then cut off at the bottom with a knife, trowel, or hacksaw blade, and carefully removed from the hole (Figure 2-20).

To obtain a chunk sample from the vertical face of a test pit or trench (Figures 2-21, 2-22, and 2-23.), the first step is to carefully smooth the surface of the face and mark the outline of the chunk. Soil is excavated from the sides and back of the chunk (Figure 2-21). The chunk is shaped with the knife (Figure 2-22). Then the chunk is cut off and carefully removed from the hole (Figure 2-23).

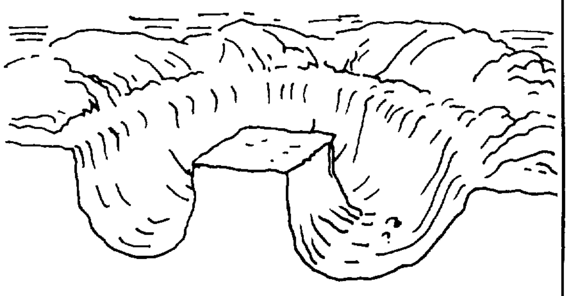


Figure 2-18. Taking a chunk sample from a level surface

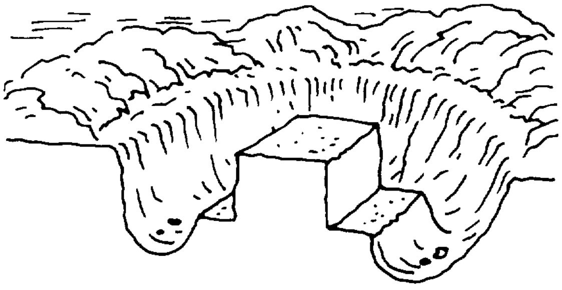


Figure 2-19. Taking a chunk sample from a level surface

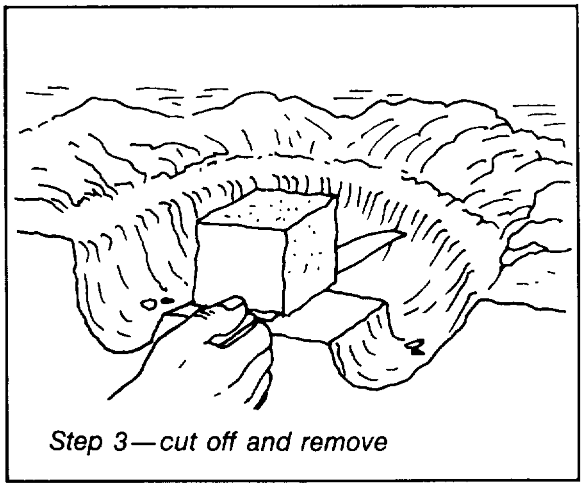

Step 3—cut off and remove

Figure 2-20. Taking a chunk sample from a level surface

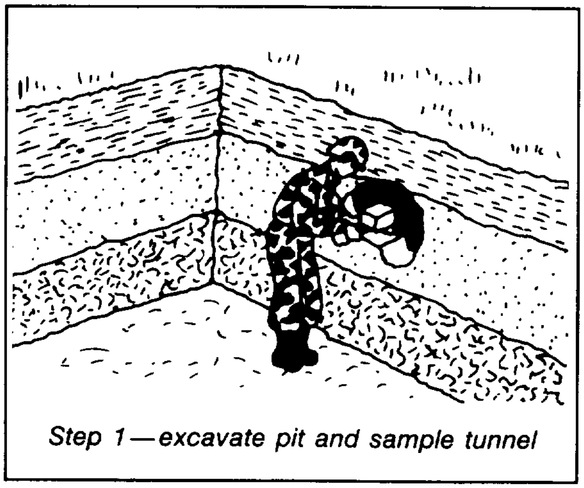

Step 1—excavate pit and sample tunnel

Figure 2-21. Taking a chunk sample from a vertical face

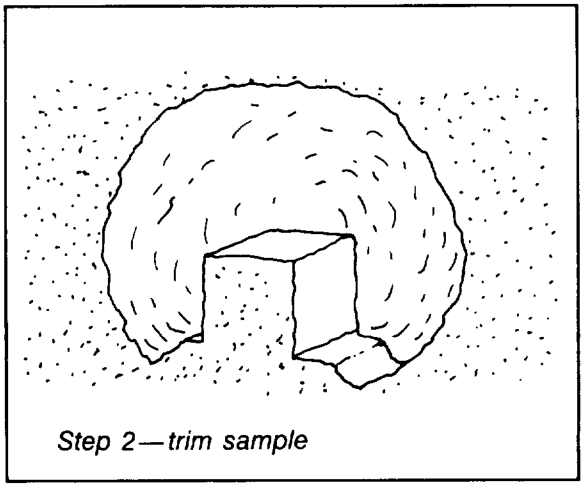

Step 2—trim sample

Figure 2-22. Taking a chunk sample from a vertical face

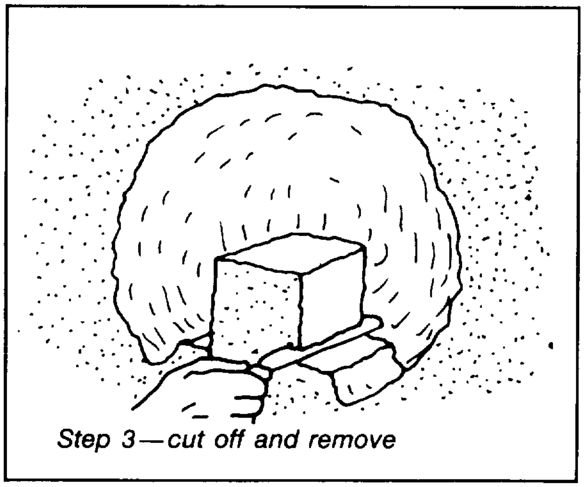

Step 3—cut off and remove

Figure 2-23. Taking a chunk sample from a vertical face

The chunk sample must be sealed after removing it from the hole. One method is to apply three coats of melted paraffin, as illustrated in Figure 2-24. Each coat is allowed 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. They should be wrapped with cheesecloth or other soft cloth (Figure 2-25) and sealed in paraffin. If cloth is not available, the sample can be reinforced with several loops of friction tape or twine. Three more coats of paraffin then are applied. Extreme care is required to prevent damage to the sample while performing these operations.

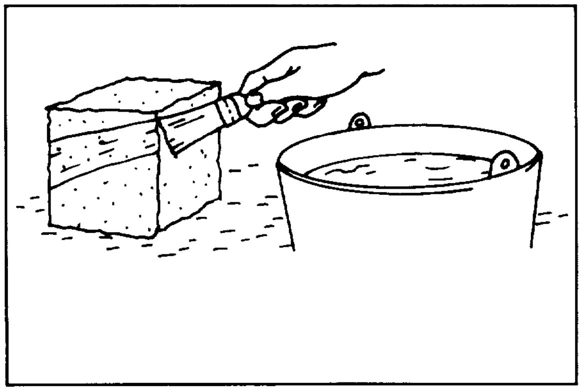


Figure 2-24. Applying paraffin to seal chunk sample

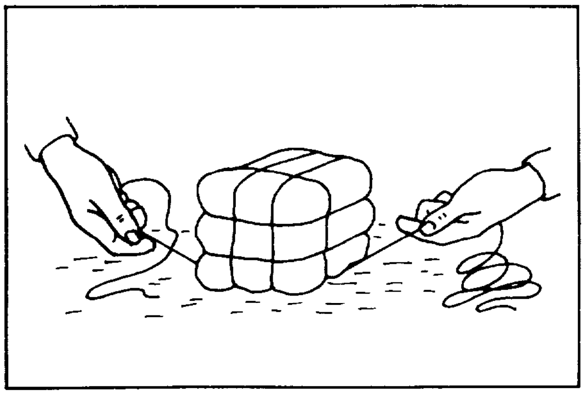


Figure 2-25. Wrapping a weak chunk sample prior to final sealing

An alternate method of sealing a chunk sample is to dip the entire sample in melted paraffin (Figure 2-26) after the first brush coat has been applied and the sample wrapped. This requires a large container and more paraffin, but gives a more uniform coating. By repeated dipping, the layer of paraffin can be built up to a minimum ⅛-inch thickness.

Samples that are to be shipped require additional protection. This may be accomplished by placing the chunk in a small box and packing, as illustrated in Figure 2-27, or by applying many coats of cloth and paraffin.

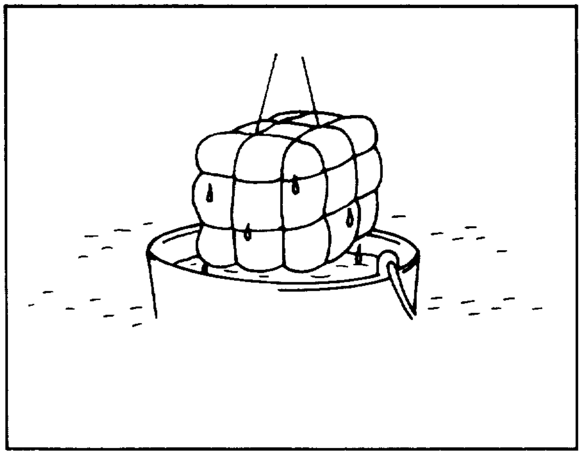


Figure 2-26. Dipping a chunk sample into melted paraffin

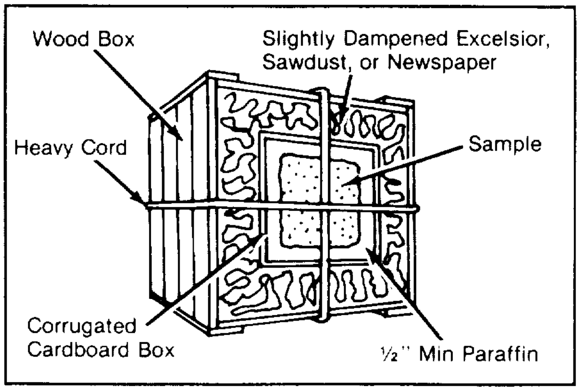


Figure 2-27. Packing a chunk sample for transportation or shipment to laboratory

CYLINDER SAMPLES BY SOIL TRAFFICABILITY SAMPLER

Apparatus

The soil trafficability sampler (Figure 2-28) consists of a sample tube and an assembly to force the tube into the soil. It is forced by hand pressure, not by blows from a hammer. A movable piston is fitted within the cylinder and is attached to a rod which extends through the center of the drive tube and terminates in a flat dish or base plate at the upper end. The outer drive tube is attached to the sample cylinder at the bottom and has two handles at the top end. One of the handles is knurled and can be turned to lock the inner rod when the piston is in position. A long and a short spacer bar are bolted to the outer tube and are used to establish the size of the sample core. It is recommended that the sampler not be used for other than extremely soft and yielding soils. The walls of the cylinder are very thin and can be deformed if they come in contact with a hard object. Even hard or dry soil can damage the sampler. Its primary use is for samples to test the remolding characteristics in soils having initially low or very low supporting value. Additional information on the soil trafficability sampler and soil trafficability test set can be found in TM 5-330.

Procedure

The piston is adjusted so that it is flush with the cutting edge of the sampler cylinder. The knurled handle is locked.

- The sampler is placed firmly in contact with the soil to be sampled.

- The disk at the top is held to prevent vertical movement, and the knurled handle is unlocked. Then the sampler cylinder is forced into the soil.

- When the cylinder is fully in the soil, the knurled handle is locked to clamp the piston and keep the soil sample from slipping out due to the vacuum created.

- The entire sampler is now rotated a half turn to shear the soil at the base of the cylinder, then carefully withdrawn from the hole and inverted so that the dish becomes a base plate. 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.

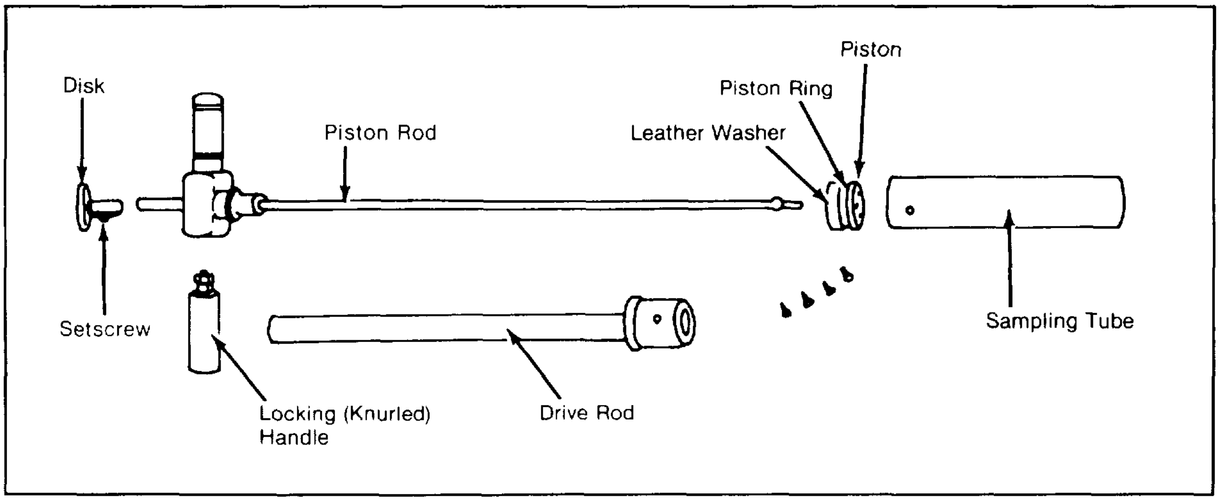


Figure 2-28. Soil trafficability sampler

- The longer spacer bar is swung into position to act as a stop while the piston ejects the sample.

- The knurled handle is released and the drive tube is pushed until the spacer bar hits the base plate and a portion of the sample is pushed up out of the cylinder.

The portion of the sample flush with the cutting edge of the sampler is cut and discarded. This amount of soil discard offsets any uneven shearing at the bottom of the hole and gives the soil sample a true cylindrical shape.

- The short spacer bar is swung into position, and the long bar is moved out of the way.

The soil sample is ejected until the short bar stops the action. Some of the soil sample will still be in the cylinder.

- The soil sample is cut off flush with the sampler 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.

- The remaining soil in the sampler is discarded.

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

Maintenance

The soil trafficability sampler requires proper maintenance and adjustment to produce consistent results. It is essential to keep the inside of the sampling tube, the piston tube, the piston ring, and the leather washer reasonably clean. After 5 to 25 samplings, depending on the type of soil, the tube should be immersed, first in water and then in fuel oil, and the piston worked up and down five or six times in each liquid. After the excess fuel oil has been wiped off, light machine oil should be squirted into the tube. If the instrument becomes stiff and hard to work, the tube should be removed, the piston disassembled and thoroughly cleaned, and the leather washer oiled. Care should be taken 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.

The effective piston rod length should be adjusted to keep the face of the piston flush with the cutting edge of the tube when the piston rod handle (disk) is fully depressed. This is done by loosening the setscrew on the handle, screwing the handle up or down to the correct position, and retightening the setscrew.

CYLINDER SAMPLES BY CBR MOLD

In soft, fine-grained soils, cylinder samples for undisturbed CBR or density tests may be taken directly in the CBR compaction cylinder by using the sampling collar (cutter) as illustrated in Figure 2-29. First, the surface of the ground is smoothed and the sampling collar and mold pressed into the soil with moderate pressure. A trench is excavated around the cylinder (Figure 2-30). The mold is again pressed firmly down over the soil, using the hand driver or loading bar if necessary (Figure 2-31). A loading bar may be improvised from any piece of timber of suitable size. The soil is carefully trimmed 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 field CBR jack, although this jack has only approximately 2 inches of travel. It is better to use a truck jack, if available. In either case, the sampler should not be forced down ahead of the trimming on the outside of the cylinder. The trench is then excavated deeper and the process repeated until the soil penetrates well into the extension collar

(Figure 2-29 - 2-32). If stones interfere, they are picked out carefully and the space filled with soil. This fact is recorded in the log of the sample where pertinent.

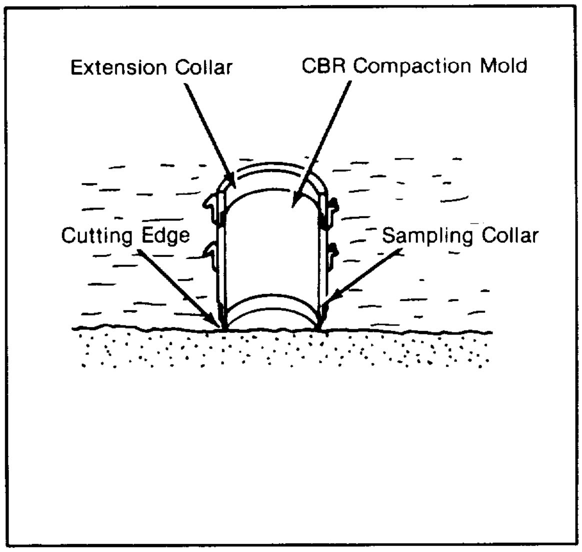

Extension Collar
CBR Compaction Mold
Cutting Edge
Sampling Collar

Figure 2-29. Section through CBR mold

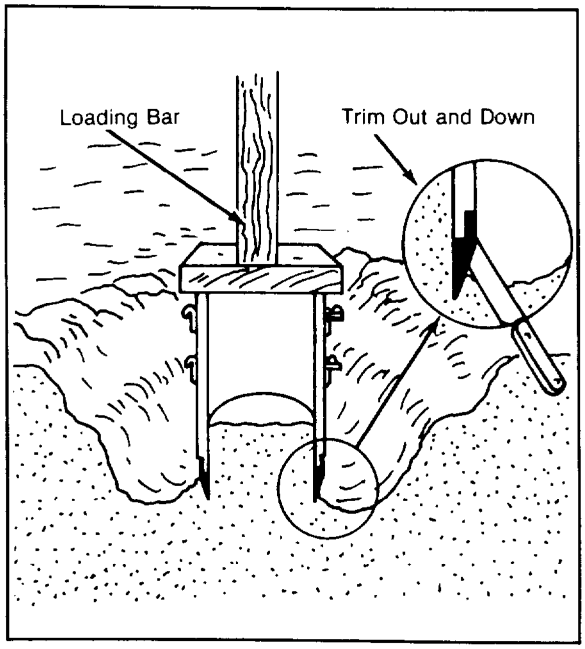

Loading Bar
Trim Out and Down

Figure 2-31. Using loading bar to drive cylinder

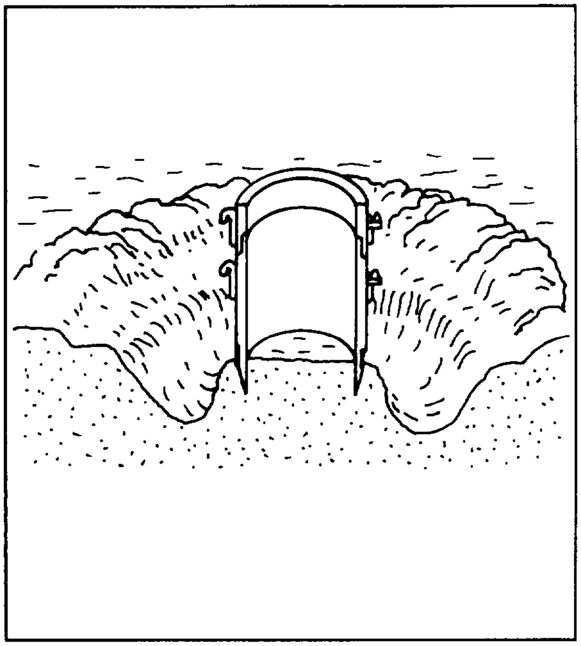


Figure 2-30. Trench cut around cylinder

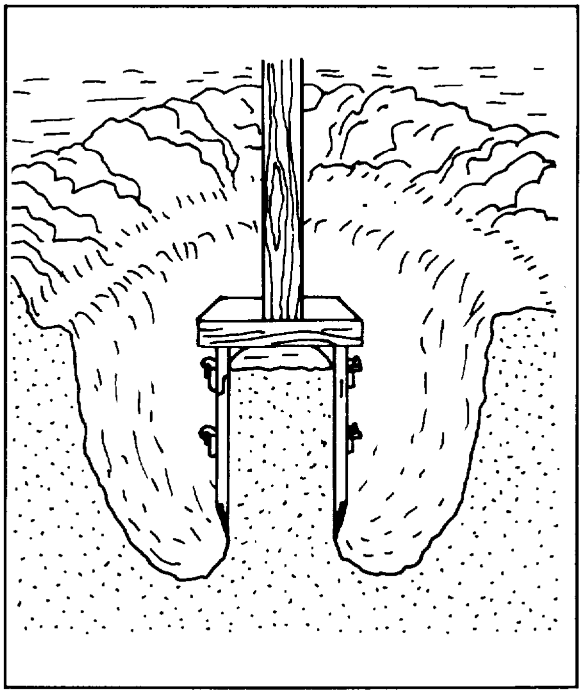


Figure 2-32. Cylinder in position prior to cutting sample

The sample is cut off at the bottom of the mold, using a shovel, knife, or wire saw (Figure 2-33). The mold and sample are then removed from the hole.

The upper collar is removed and the top surface of the sample trimmed down into the mold a distance of about ½ inch. This recess is filled with paraffin to seal the end of the sample (Figure 2-34). The mold is turned over and the cutting edge removed. This end is trimmed down into the mold about ½ inch, as before, and the resulting space is filled with paraffin. If the sample is to be handled very much prior to testing, the ends should be overfilled with paraffin and then trimmed exactly flush with a straightedge. Boards are placed over each end and clamped in place, using bolts, wire, or string (Figure 2-35). Samples which must be transported some distance or which may have to be handled quite a bit before testing should be wrapped in cloth and soaked in paraffin layers.

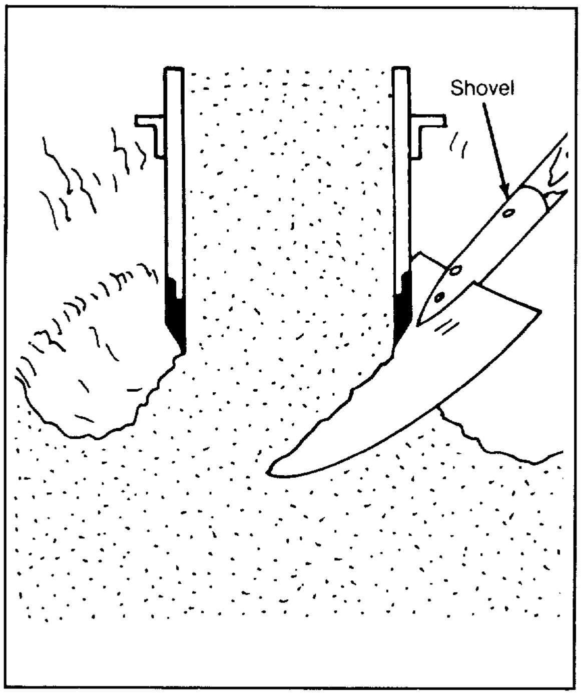


Figure 2-33. Cutting off cylindrical sample

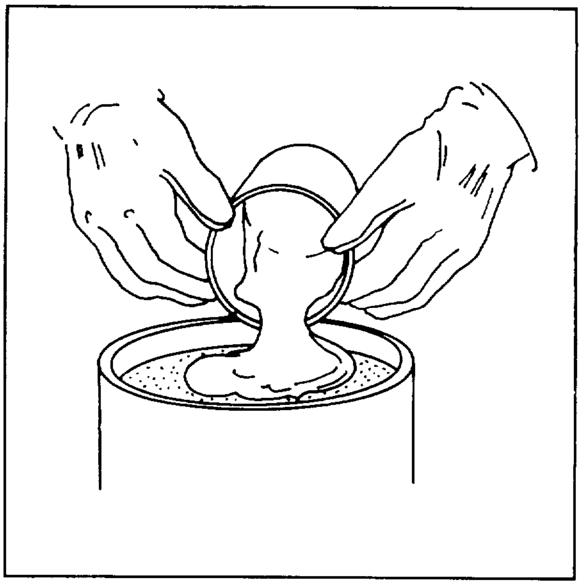


Figure 2-34. Sealing sample in CBR mold

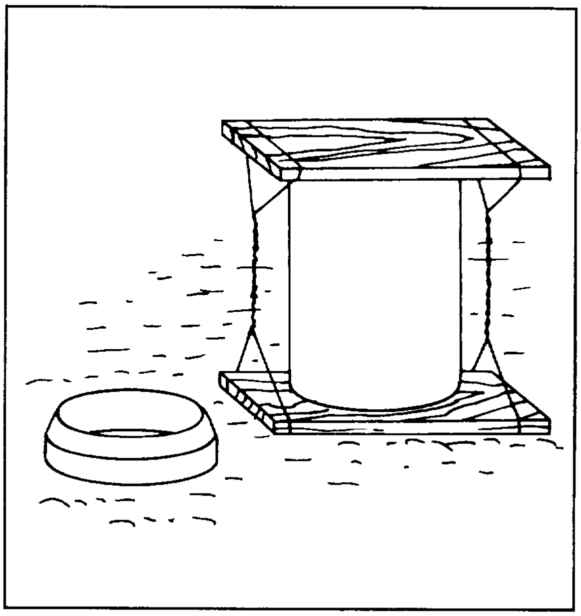


Figure 2-35. Protecting sample in CBR mold

QUARTERING SAMPLES

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

Samples Weighing Over 100 Pounds

Quartering of a sample of this size is shown in Figures 2-36, 2-37, 2-38, and 2-39. The sample is first mixed and piled on the quartering canvas (Figure 2-36). Each shovelful is placed in the center of the cone so that the soil runs down evenly in all directions in order to mix the sample. The cone is then flattened with the shovel, spreading the material to a circular layer of approximately uniform thickness (Figure 2-37). A stick or pipe is inserted under the center of the pile (under the canvas) and both ends of the stick are lifted, thus dividing the sample into two parts (Figure 2-38). The stick is removed, leaving a fold in the canvas. The stick is again inserted under the pile, this time at right angles to the first division, and again lifted, dividing the sample into four parts (Figure 2-39, page 2-40). Two diagonally opposite quarters are discarded and care is taken to clean the fines from the canvas. The remaining material is remixed by taking alternate shovelfuls from each quarter. The quartering process is repeated as necessary to reduce the sample to the desired size.

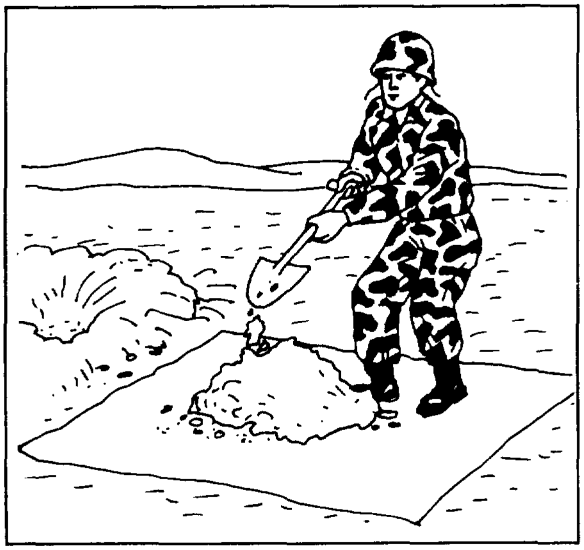


Figure 2-36. Mixing and piling a sample weighing more than 100 pounds

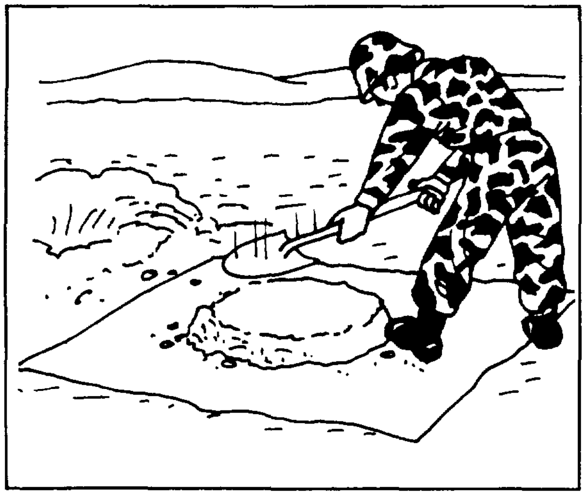


Figure 2-37. Flattening sample to uniform thickness

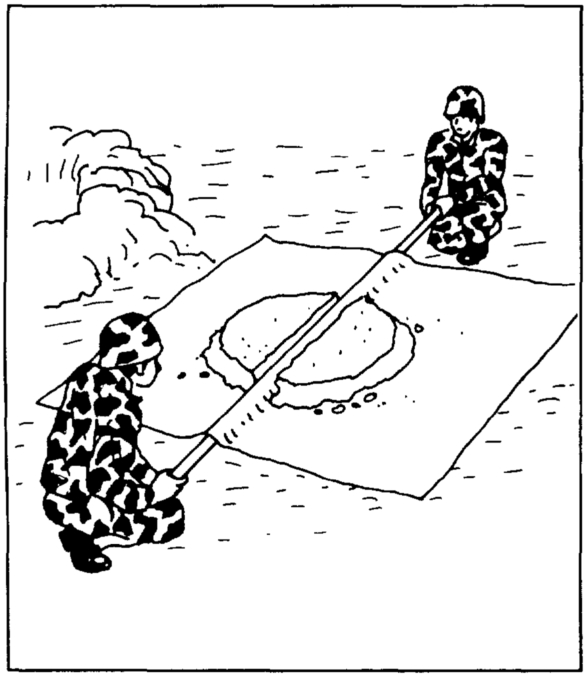


Figure 2-38. Halving the sample

Samples Weighing 25 to 100 Pounds

In quartering this size of sample, the soil is piled on the canvas and mixed by alternately lifting the corners of the canvas and pulling over the samples as if preparing to fold the canvas diagonally, as illustrated in Figure 2-40. The sample is then flattened and quartered.

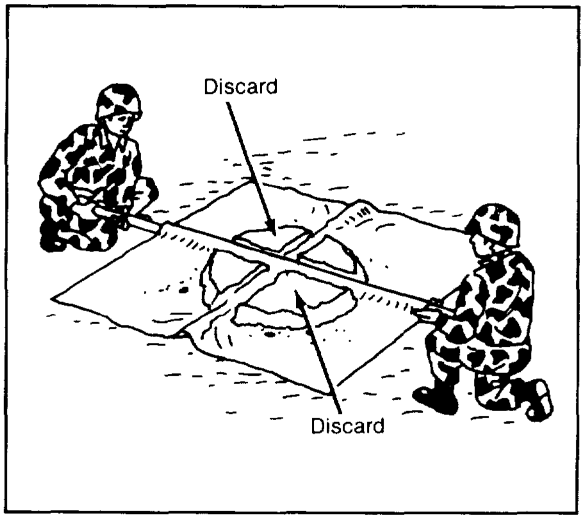


Figure 2-39. Quartering the sample

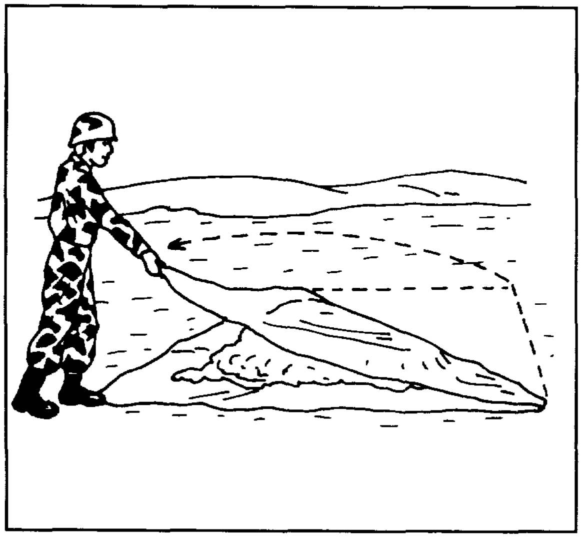


Figure 2-40. Mixing a sample weighing 25 to 100 pounds

Samples Weighing Less Than 25 Pounds

The process of quartering samples less than 25 pounds is similar to the process for 100 pounds and more. The sample is placed on the canvas or a clean sheet of paper. It is mixed thoroughly with a trowel and formed into a conical pile (Figure 2-41). The cone is flattened by pressing downward with the trowel (Figure 2-42). The trowel is used to divide the sample into quarters. Diagonally opposite quarters are discarded (Figure 2-43). The process is repeated as necessary to reduce the size of the sample for testing.

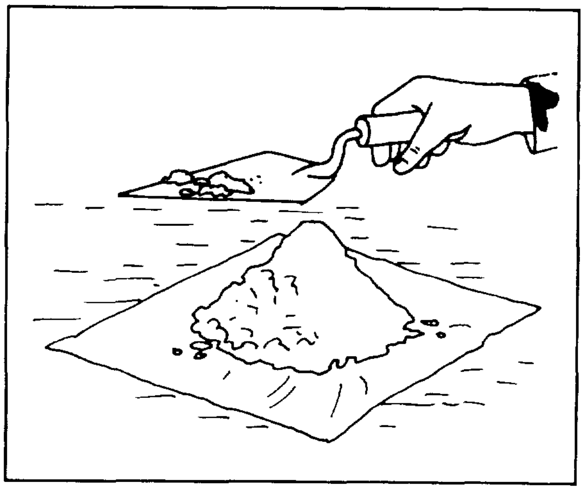


Figure 2-41. Preparing a sample weighing less than 25 pounds

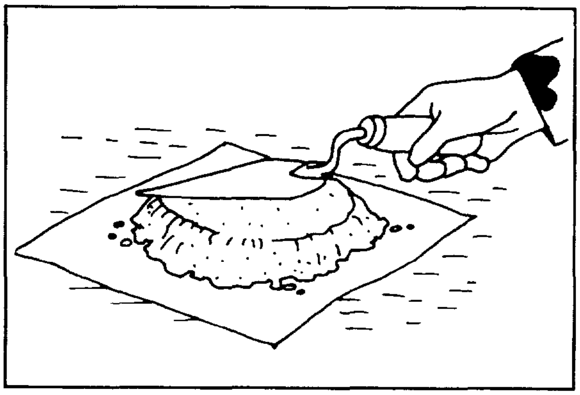


Figure 2-42. Flattening a small sample

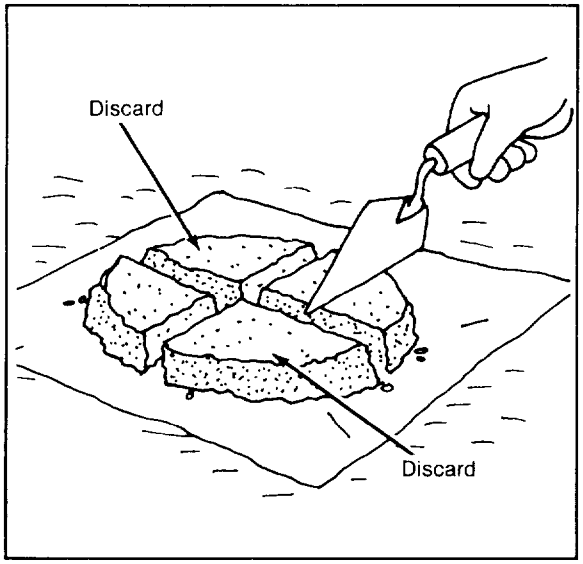


Figure 2-43. Quartering a small sample

Section III. SIEVE ANALYSIS

DISCUSSION

Soil grains and gradation are discussed in Section I of this chapter, with some consideration of the effects of grain characteristics on the physical properties of soils. The use of grain size and grain-size distribution in soil classification and visual-manual tests and their use for field identification are also covered in Section I. Although estimates of grain size of coarser materials may be made in this way, the accurate determination of the grain-size distribution or gradation of coarse soil fractions requires a sieve analysis, or grain-size analysis. The proportion of material of each grain present in a given soil is determined by this test. Sieve separation of coarse and fine fractions of mixed soil is required to provide samples for further testing.

GRAIN-SIZE DISTRIBUTION

This refers to the portion of soil particles of different sizes which are contained in a given soil. The determination of the grain-size distribution of a soil, also called mechanical analysis, may be accomplished by a screening process (sieve analysis) for coarse-grained or cohesionless soils, by a settling process in water (wet mechanical analysis or hydrometer analysis) for fine-grained or cohesive soils, or by both the screening and settling processes on the same sample (combined mechanical analysis). Any grain-size distribution is valid only when the soil particles are completely separated one from the other, so that the particles can pass through the sieve opening or settle through the water individually. Sieve analysis is applicable to soil containing small amounts of material passing the No. 200 sieve, provided the grain size distribution of that portion passing the No. 200 sieve is not of interest. The sieve analysis is conducted either on the entire sample or on the sample after the fines are removed by prewashing.

Preparation

- Prewashing. The Corps of Engineers specifies a sieve analysis with prewashing unless otherwise allowed by the specifications. Even the most coarse grained and apparently the cleanest soil will at times produce a higher than expected percentage of fine particles. Where time is critical and lower accuracy is permissible, cumulative prewashing may be eliminated. If there is any doubt as to the need for prewashing, a moist pat of the soil may be quickly dried on the stove or in the oven and its dry strength examined by crushing it between the fingers. If the soil can be crushed and powdered easily, sieve analysis can be made without prewashing (dry sieve analysis). In preparing a sample for dry sieve analysis, however, it may still be necessary to pulverize the finer portion of the sample to ensure separation of the particles. Caution should be used so the natural particle shape is not crushed. The minimum sample weight required for a sieve analysis depends upon the maximum particle size in the sample as follows:

Maximum particle size (sieve opening)	Minimum dry weight of test specimen
3 in	6,000 G
1½ in	3,000 G
¾ in	1,500 G
⅜ in	600 G
No. 4	200 G

- Drying. A majority of the large samples must be dried in preparation for tests such as mechanical analysis, compaction, and CBR.

For air drying, the soil is placed in pans or spread on a canvas and exposed to the direct rays of the sun or to a draft of dry air. Frequent agitation or stirring speeds up the drying. A fan blowing air over the sample will greatly increase the rate of drying. Care must be taken not to blow away the fine material.

For oven drying, samples are dried in an oven to constant weight. This is done most frequently in connection with the determination of the moisture (water) content of the soil.

Some samples, particularly coarse-grained soils which do not contain appreciable amounts of organic matter, can be dried quickly in a frying pan over a hot-plate or gasoline burner or by using the alcohol burn-off method. The sample must be stirred frequently and every precaution taken to prevent the sample from being heated beyond the temperature needed to evaporate the free water present, 110° ± 5° C.

Apparatus

The following equipment (Figure 2-44) is provided in the soil test set for sieve analysis:

Balance, specific gravity determination, 200-gram capacity (not shown)

Brush, varnish, 1 inch

Cover, test sieve, 8-inch diameter

Cloth, cotton duck, 60 × 60 inches

Mortar, porcelain

Oven, laboratory (not shown)

Pan, test sieve, 8-inch diameter

Pans or evaporating dishes, as required

Pestle, must be rubber tipped as shown in Figure 2-44

Scale, bench, three weigh beams, 2,610-gram capacity

Scale, bench, twin weigh beams, 21,100-gram capacity

Scoop, kitchen

Shaker, testing sieve, laboratory, hand operated (Figure 2-45, page 2-44).

Sieves test, 8-inch diameter:

Full Height (2 inches deep): 2, 1½, 1, ¾, and ¼ inch

Half Height (1 inch deep): Numbers 4, 10, 40, 60, 100, and 200 (Figure 2-46, page 2-45).

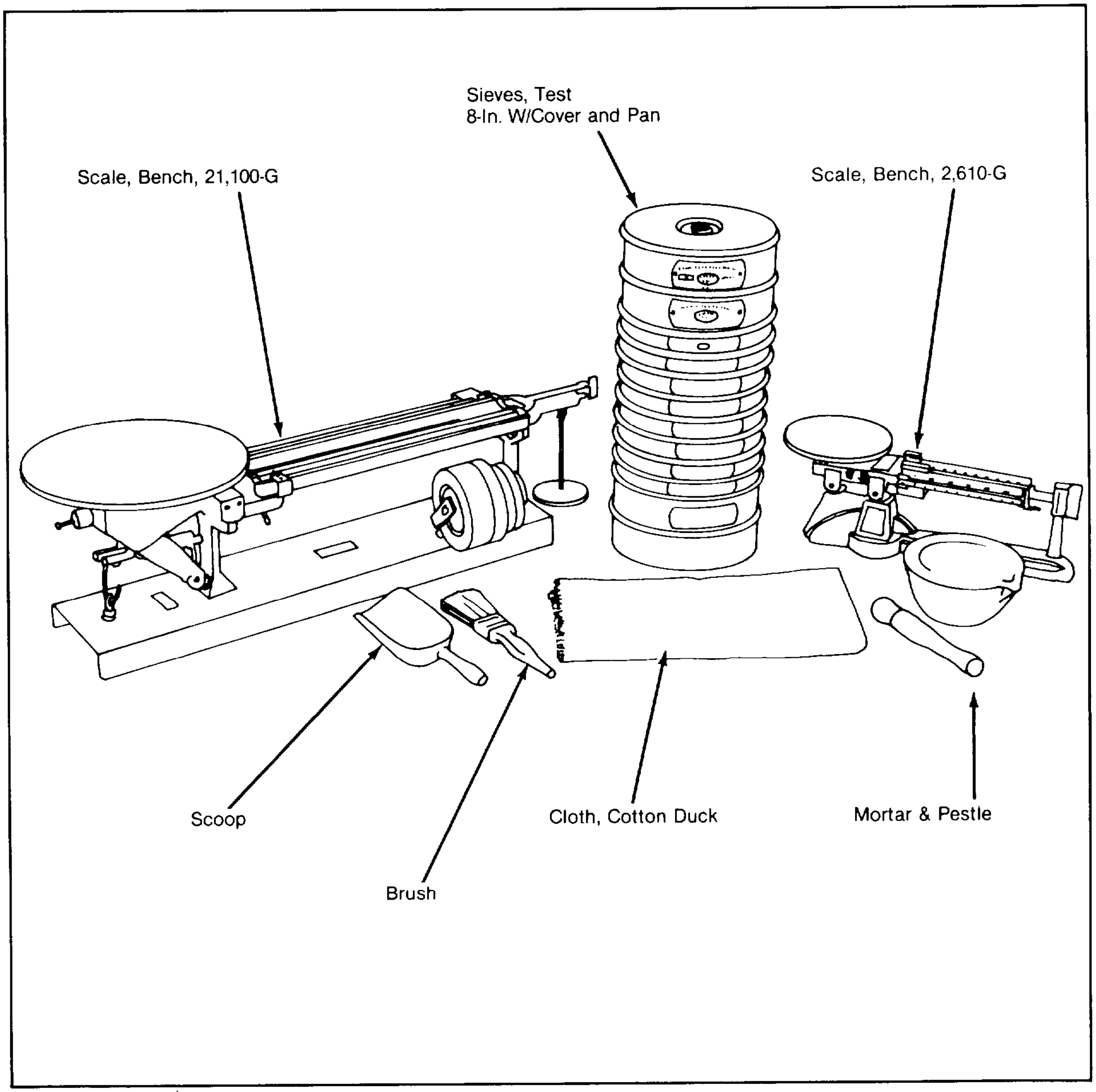

Sieves, Test
8-In. W/Cover and Pan
Scale, Bench, 21,100-G
Scale, Bench, 2,610-G
Scoop
Cloth, Cotton Duck
Mortar & Pestle
Brush

Figure 2-44. Equipment for sieve analysis

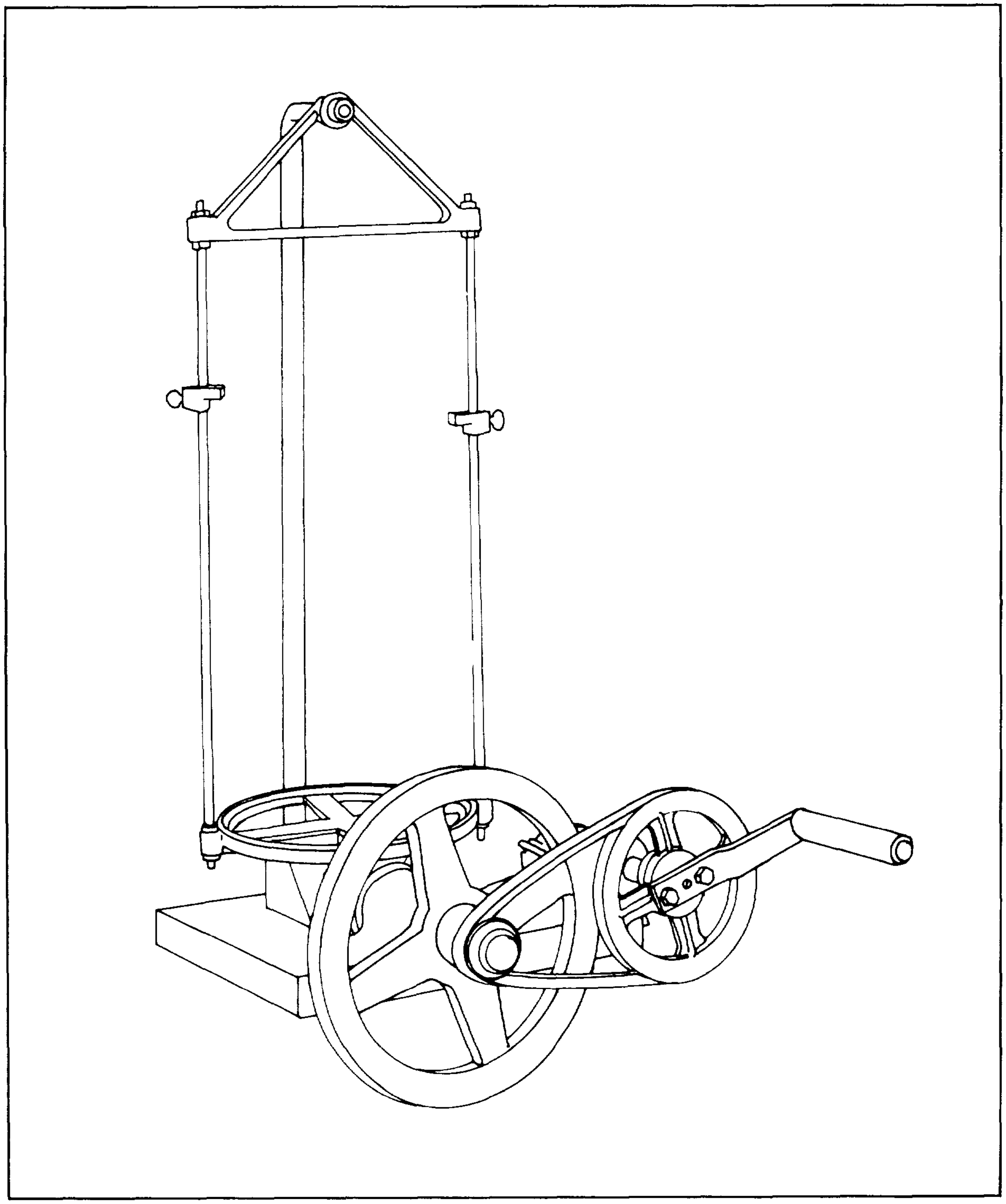


Figure 2-45. Hand-operated sieve shaker

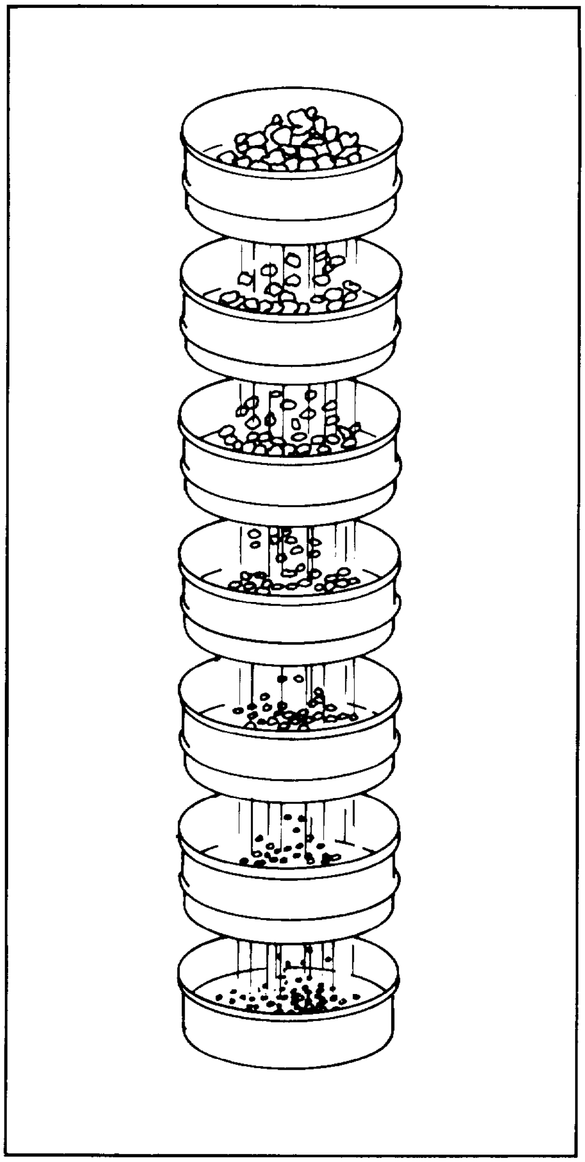


Figure 2-46. Testing sieves for dry sieve analysis

NOTE

A full set of sieves includes 3, 2, 1/2, 3/4 inch and US Numbers 3, 4, 8, 10, 16, 20, 30, 40, 50, 60, 80, 100, and 200 in addition to the above. A set consisting of 3, 1 1/2, 3/4, and 3/8 inch and US Numbers 4, 8, 16, 30, 50, 100, and 200 is recommended by The American Society for Testing and Materials (ASTM) to give uniform spacing of points on the grain-size distribution graph, since each opening is twice the size of the one below.

SIEVE METHOD

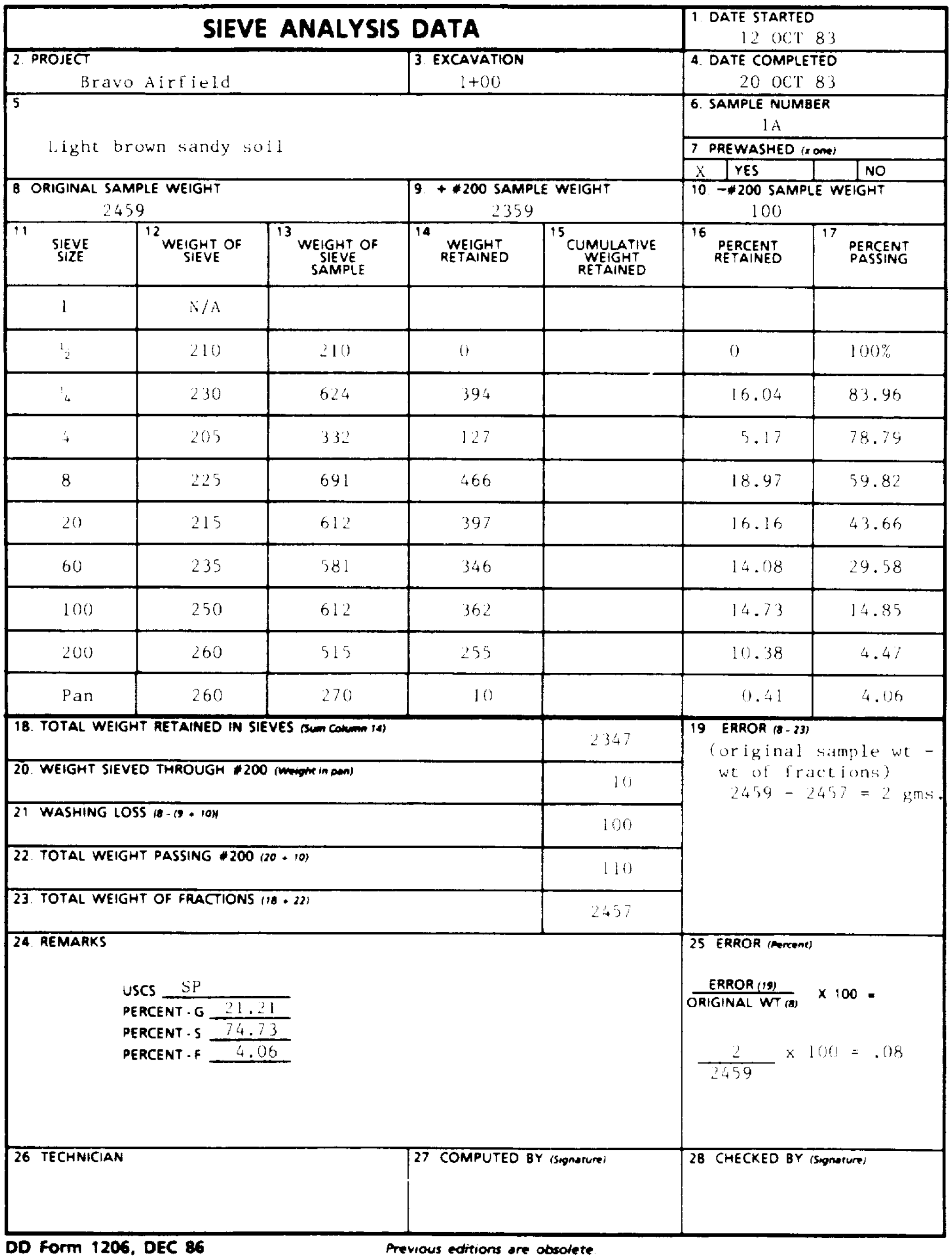
Preparation

Material to be analyzed using the sieve method is first air dried, after which any clusters of soil are thoroughly broken up using the fingers or a mortar and pestle. Care must be taken to separate only the individual particles and not to crush any of them. If the sample is larger than the recommended amount, it should be reduced in size by quartering until a representative sample about the correct size is obtained. Too large a sample can overload a sieve, and the result will be incomplete separation and errors in the test.

Procedure

The prepared sample is now ready for the actual test which is conducted according to the following directions:

- Record all identifying information of the sample, such as project, excavation number, sample number, description of sample, and date on DD Form 1206 (Sieve Analysis Data) (**Figure 2-47, page 2-46**).



SIEVE ANALYSIS DATA		1. DATE STARTED 12 OCT 83
2. PROJECT Bravo Airfield	3. EXCAVATION 1+00	4. DATE COMPLETED 20 OCT 83
5 Light brown sandy soil		6. SAMPLE NUMBER 1A
		7. PREWASHED (x one) X YES ☐ NO
8. ORIGINAL SAMPLE WEIGHT 2459	9. + #200 SAMPLE WEIGHT 2359	10. −#200 SAMPLE WEIGHT 100

11 SIEVE SIZE	12 WEIGHT OF SIEVE	13 WEIGHT OF SIEVE SAMPLE	14 WEIGHT RETAINED	15 CUMULATIVE WEIGHT RETAINED	16 PERCENT RETAINED	17 PERCENT PASSING
1	N/A					
½	210	210	0		0	100%
¼	230	624	394		16.04	83.96
4	205	332	127		5.17	78.79
8	225	691	466		18.97	59.82
20	215	612	397		16.16	43.66
60	235	581	346		14.08	29.58
100	250	612	362		14.73	14.85
200	260	515	255		10.38	4.47
Pan	260	270	10		0.41	4.06

18. TOTAL WEIGHT RETAINED IN SIEVES (Sum Column 14)	2347	19 ERROR (8 - 23) (original sample wt - wt of fractions) 2459 - 2457 = 2 gms.
20. WEIGHT SIEVED THROUGH #200 (Weight in pan)	10	
21. WASHING LOSS (8 - (9 + 10))	100	
22. TOTAL WEIGHT PASSING #200 (20 + 10)	110	
23. TOTAL WEIGHT OF FRACTIONS (18 + 22)	2457	

24. REMARKS

USCS SP
PERCENT - G 21.21
PERCENT - S 74.73
PERCENT - F 4.06

25 ERROR (Percent)

$$\frac{\text{ERROR (19)}}{\text{ORIGINAL WT (8)}} \times 100 =$$

$$\frac{2}{2459} \times 100 = .08$$

26 TECHNICIAN

27 COMPUTED BY (Signature)

28 CHECKED BY (Signature)

DD Form 1206, DEC 86 *Previous editions are obsolete*

Figure 2-47. Data sheet, example of dry sieve analysis

- Oven dry the sample at 110° ± 5°C, and allow it to cool to a constant weight.

- Weigh the dried and cooled sample and record the weight as the weight of the original sample.

- Select a nest of sieves suitable to the soil being tested in accordance with specifications. Weigh each selected sieve and record its weight. The top sieve, if possible, should be able to pass the largest particles in the sample. Particles larger than the largest sieve must be measured. The remaining sieves are selected based on the knowledge of the use to which the grain-size distribution curve will be put. If this information is lacking, all the sieves below the largest one should be used. The sieves issued in the test set, in order of decreasing size openings, shown in parentheses (in millimeters), are as follows:

  2 inch (50.8 millimeters)
  1½ inch (38.1 millimeters)
  1 inch (25.4 millimeters)
  ¾ inch (19.0 millimeters)
  ¼ inch (6.35 millimeters)
  No. 4 (4.76 millimeters)
  No. 10 (2.00 millimeters)
  No. 40 (0.420 millimeters)
  No. 60 (0.250 millimeters)
  No. 100 (0.149 millimeters)
  No. 200 (0.074 millimeters)

- Arrange the sieves in a nest with the largest opening sieve on top and each progressively smaller opening sieve in order below it. Attach the bottom pan to the smallest sieve used.

- Place the sample on the top sieve of the nest and put on the cover plate.

- Place the nest of sieves in the shaker and shake the nest for about 10 to 15 minutes at moderate speed. The length of time depends on the amount of fine material in the sample. Shaking should continue until the amount of material on any sieve does not show appreciable change.

- Shake the nest by hand if a shaker is not available. Use a lateral and vertical motion accompanied by a slight jarring to keep the sample moving continuously over the screens. Jarring is accomplished by dropping the nest lightly on a stack of papers or magazines. Use extreme caution not to break the nest, and do not open it to rearrange or manipulate the particles.

- Complete the shaking, set the nest down and remove the top cover.

- Start with the top sieve. Weigh it and the material retained on it.

- Record this weight (sieve and sample) on the line opposite the sieve size.

- Set the weighed sieve and material aside until the weighing of all sieves is completed. Do not discard any of the material.

- Remove the next sieve and material, weigh, record, and set the sieve and material aside.

- Continue this process for each sieve and the bottom pan in the nest.

Calculations

Subtract the weight of the empty sieve or pan from the sieve plus material weight, for weight retained. Total the individual material weights (total weight of fractions) and compare the total to the original sample weight as determined above. The difference is the amount of error. The amount of error divided by the original sample weight will produce the percentage of error. If the error exceeds (±) 1 percent, check each sieve fraction for possible error. If no error has been made in the weighing, but the weight error still exceeds 1 percent, rerun the test.

SIEVE ANALYSIS WITH PREWASHING

Samples containing cohesive soil must be washed prior to sieve analysis to break up the lumps and to remove the coating of cohesive particles.

Preparation

The sample is oven dried and weighed, then it is placed in a clean container, and clean water is added to cover the sample completely. Allow the sample to soak until the adhering and lumpy particles are completely disintegrated. This process may take from 2 to 24 hours depending on the amount and plasticity of the fine material. Stirring and breaking up lumps will hasten the process. Use caution not to lose the material.

Procedure

When the sample has disintegrated and no lumps exist, it is ready for prewashing according to the following procedures:

- Transfer the sample and water from the container to a No. 200 sieve (no more than 200 grams at a time). If the sample contains an appreciable amount of coarse particles, use a nest of a No. 4 or a No. 40 sieve and a No. 200 sieve to prevent overloading the smaller opening sieve. Large particles may be washed individually over the No. 200 sieve and set aside for future use.

- Wash the sample thoroughly until the material on the sieve and the wash water is clear (Figure 2-48). Rub or stir the sample carefully using the fingers. Rinse the fingers over the sieve so no material is lost.

- The wash water and the material which has passed through the No. 200 sieve is decanted very carefully, then oven dried, weighed, and recorded as the original minus No. 200 sample before sieving. The entire minus No. 200 sample is set aside for future use.

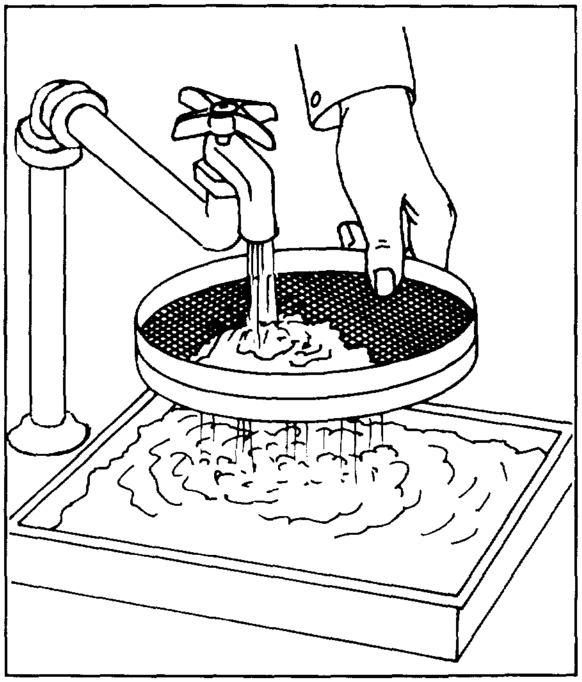


Figure 2-48. Prewashing a sample

- The washed sample and any separately washed large particles are oven dried and reweighed, then recorded as the total plus No. 200 sample before sieving.

- The washed and dried sample is dry sieve tested.

- The amount passing the No. 200 sieve (weight retained in pan) is added to the original minus No. 200 weight to obtain the total weight of minus No. 200 material. This total is added to the other fraction weights to compute the total weight of fractions after testing the error in weights.

Calculations

The figures recorded during the test are used to compute the percentages and then to compile a grain-size distribution curve.

- Weight retained. Subtract the weight of the empty sieve or pan from the weight of the sieve plus sample. This weight is the weight of material retained.

- Error. Total the individual weights retained on each sieve, excluding material retained in the pan. Total retained on sieves plus total passing the No. 200 sieve (washed and sieved) equals total weight of fractions. Comparing the original weight to the total weight of fractions will determine the amount of error.

- Percent of error. The amount of error divided by the original sample weight multiplied by 100 will equal the percentage of error.

- Cumulative weight retained. Cumulative weights are determined when only a portion of the original sample is used. This procedure should only be used as an expedient method when the washing of a large amount of minus No. 200 material would be difficult. This may be determined by field identification. Expedient method results should not be used for design purposes.

Computations

Example. At Bravo Airfield, a 2,459-gram sample has a large amount of fines.

- The total sample is split over a No. 4 sieve. The plus No. 4 is sieved and weighed as previously described. The minus No. 4 in the pan is then added to minus No. 4 from split.

- The minus No. 4 material is weighed and recorded in remarks as original minus No. 4 sample weight, then split down to a workable size, 522 grams, which is also recorded. The 522-gram sample is washed, dried, sieved, weighed, and recorded.

- Compare the 522-gram fraction to the original minus No. 4 sample weight.

$$\frac{1{,}836}{522} = 3.52$$

- The cumulative weight retained column is computed by multiplying the weights retained on the sieves smaller than No. 4 by 3.52 to bring the weight retained in sieves back into proportion with the original 1,836-gram sample of minus No. 4. The percent retained is then determined using the cumulative weights. The remaining portions of the data sheet will be determined as follows.

- Percent retained. Individual weights retained on each sieve divided by the total weight of fractions will determine the percent retained on each sieve.

- Percent finer. Percent finer is determined by subtracting the percent retained on each sieve from the total sample percentage (100 percent).

  Percent finer – percent retained = percent finer than the next consecutive sieve.

- Proportions of gravels, sands, and fines expressed in percent.

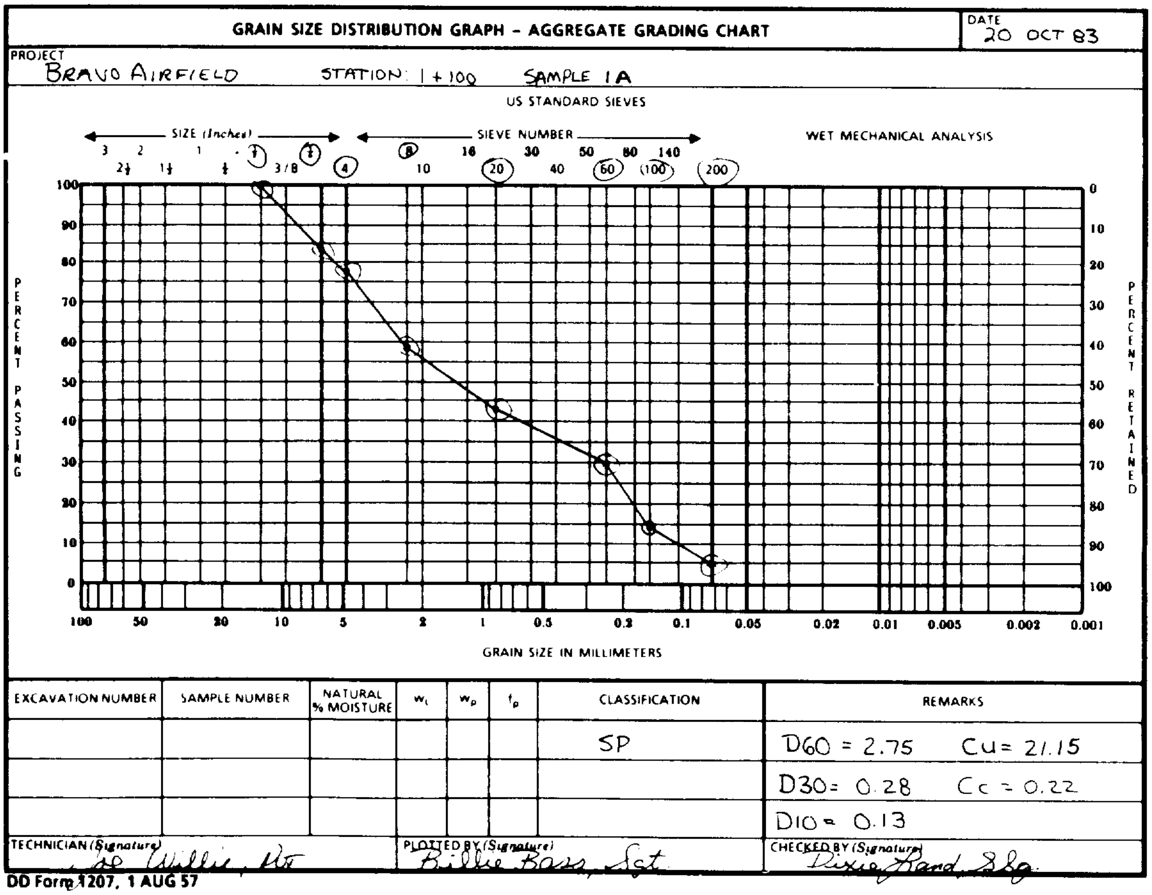
  100 percent – percent finer than No. 4 = percent gravel.

  Percent finer than No. 4 – percent finer than No. 200 = percent sand.

  Percent finer than No. 200 = percent fines.

Plotting the Curve

The DD Form 1207 (Grain Size Distribution Graph-Aggregate Grading Chart) (Figure 2-49) is used to show that the sieve analysis results in a graphic form. The graph uses semilogarithmic coordinates. The grain sizes are plotted on a logarithmic scale while the percentages are plotted on an arithmetical scale. The grain sizes are plotted using the sieve number along the top edge. The percentage values from Figure 2-47 are plotted, and a smooth curve is drawn connecting the plotted points. The curve for the sample FT-PI-I (Figure 2-47, page 2-46) is shown as one of the curves on Figure 2-49. This is the final product of a sieve analysis.



GRAIN SIZE DISTRIBUTION GRAPH - AGGREGATE GRADING CHART

DATE 20 OCT 83

PROJECT BRAVO AIRFIELD STATION: 1 + 100 SAMPLE 1A

EXCAVATION NUMBER	SAMPLE NUMBER	NATURAL % MOISTURE	w_l	w_p	I_p	CLASSIFICATION	REMARKS
						SP	D60 = 2.75 Cu = 21.15
							D30 = 0.28 Cc = 0.22
							D10 = 0.13

TECHNICIAN (Signature) Joe Willie, Pvt

PLOTTED BY (Signature) Billie Bass, Sgt

CHECKED BY (Signature) Dixie Band, 2Lt

DD Form 1207, 1 AUG 57

Figure 2-49. Data sheet, grain-size distribution graph

This curve clearly shows the distribution and range of particle sizes, and is particularly helpful in determining the soil classification and usability of the soil as a foundation or construction material. Coefficients of uniformity and curvature, used in the USCS to classify sands and gravels by their gradation characteristics, are determined from their grain-size distribution curve as follows:

- The grain size, in millimeters, which corresponds to 10 percent passing on the grain-size distribution curve, is called Hazen's effective size. It is designated by the symbol D_{10}. The uniformity coefficient (C_u) is the ratio between the grain diameter, in millimeters, corresponding to 60 percent passing on the curve (that is D_{60} and the diameter of the 10 percent passing (D_{10}). Hence, $C_u = D_{60}/D_{10}$.

- The coefficient of curvature, designated by the symbol $C_c = (D_{30})^2/(D_{60} \times D_{10})$. D_{10} and D_{60} have meanings previously assigned while D_{30} is the grain diameter, in millimeters, corresponding to 30 percent passing on the grain size distribution curve.

- For example, the uniformity coefficient and coefficient of curvature of the well-graded sand, SW, in Figure 2-49 (DD Form 1207) is determined from the grain-size distribution curve as follows:

D_{60} reads 0.72 millimeter.
D_{30} reads 0.36 millimeter.
D_{10} reads 0.09 millimeter.

$$C_u = \frac{0.72}{0.09} = 8.0$$

$$C_c = \frac{0.36 \times 0.36}{0.72 \times 0.09} = 2.0$$

Both C_u and C_c meet the criteria shown in Table A-1 (Appendix A) for well-graded sand, SW (C_u greater than 6 and C_c between 1 and 3). To be classified as GH, gravel must have a C_u greater than 4 and C_c between 1 and 3.

Section IV. SPECIFIC GRAVITY TEST

DISCUSSION

The specific gravity of a solid substance is the ratio of the weight of the solid to the weight of an equal volume of water. For exact analysis, the specifications require distilled or demineralized water and all measurements of water and solids to be made at stated temperatures. In dealing with soils, the specific gravity is necessary for certain soil tests, such as the wet mechanical analysis. It is also necessary for computations involving volume or weight relationships, such as computing the void ratio, percent saturation, and similar values. Specific gravity of a soil is largely dependent upon the density of the minerals making up the individual soil particles. However, as a general guide, some typical values can be given for soils based on their organic content.

For most inorganic soils, specific gravity ranges from 2.60 to 2.80. Many within this group fall between 2.65 and 2.75. Occasional soils derived from unusually heavy material may have a specific gravity as high as 3.00.

Soils with large amounts of organic matter or porous particles (such as diatomaceous earths) have specific gravities below 2.60. Some range as low as 2.00.

SPECIFIC GRAVITY OF SOIL

Soil particles generally contain pores or voids that may be large enough to be visible or too small to be seen even at high magnification. Thus, the specific gravity of the soil minerals can only be measured when particles are of such small size that there are no voids in the individual particles. The test procedure must ensure that no air trapped in surface pores is included in the measured particle volume. However, larger soil particles will contain voids that are impermeable. Hence, they are included in the measured volume and cause the apparent specific gravity to be less than the specific gravity of mineral solids. When the sample volume for determining bulk specific gravity is measured, all surface pores that could hold capillary water are taken as part of the partial volume. The specific gravity of a soil mass can be expressed in one of three different forms—the specific gravity of the solids, the apparent specific gravity, and the bulk specific gravity.

Specific Gravity of Solids

This is designated G_a and is the ratio of the weight in air of a given volume of soil particles to the weight of an equal volume of distilled water, both at a stated temperature. The specific gravity of solids is only applied to that fraction of a soil that passes a No. 4 sieve.

Apparent Specific Gravity

This is designated G_a and is the ratio of the weight in air of a given volume of the impermeable portion of soil particles 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.

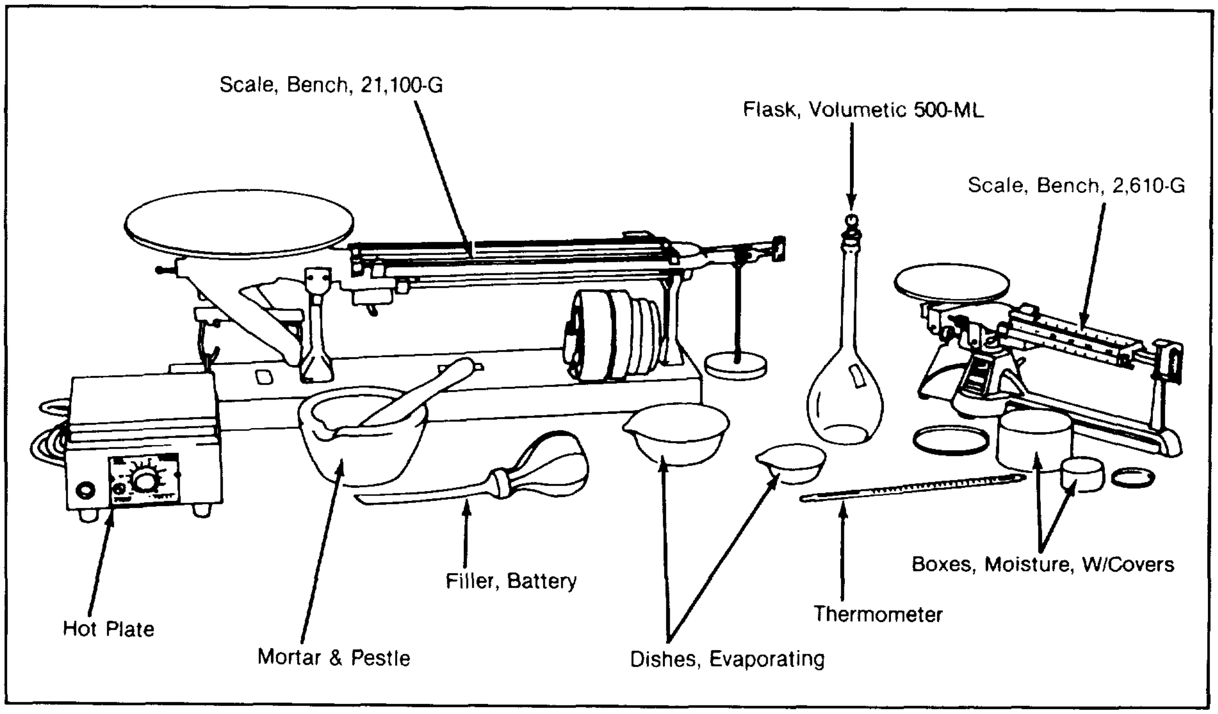


Figure 2-50. Apparatus for determining specific gravity of soils

Bulk Specific Gravity

This is designated G_m and is the ratio of the weight in air of a given volume of a permeable material (including permeable and impermeable voids) to the weight of an equal volume of distilled water at a stated temperature.

APPARATUS AND SAMPLE SELECTION

Apparatus

Apparatus used to perform specific gravity tests (Figure 2-50) follows:

Scale, bench, 21,000-gram capacity

Tares, moisture

Dishes, Evaporating

Filler, battery

Flask, volumetric, 500-milliliter capacity (not in test set)

Funnel, metal (not shown)

Hot plate, electric

Mortar and pestle

Scale, bench, 3 weigh beams, 2,610-gram capacity

Centigrade thermometer, general laboratory

The test also requires some equipment that is not in the test set, but must be procured from commercial sources. Otherwise, use expedient substitutes from local sources.

Sample Selection

Particular care must be taken 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. These should be tested at their natural water content and the oven-dried weight determined at the end of the test.

- When the sample contains large and small particles, it should be separated on the No. 4 sieve. The specific gravity of the fine fraction and the apparent specific gravity of the coarse fraction are determined separately. A composite specific gravity for the entire sample is computed on the basis of the percent, by weight, of the fine and coarse material.

- If the specific gravity is to be used in conjunction with a hydrometer analysis (Section V), it must be determined only on the fraction of the sample which passes the No. 200 sieve.

PROCEDURES

The specific gravity test is made in one of two ways, depending on whether the requirements call for soil at natural water content or oven-dried soil. The weight of the volumetric flask and water must be calibrated over the range of temperatures likely to be encountered in the laboratory. Selection of a representative soil sample and its handling during transfers from container to container are critical in this testing procedure.

CALIBRATION OF THE VOLUMETRIC FLASK

Calibration consists of direct weighing of the flask and water over a selected temperature range likely to be encountered during testing, using the following steps.

1. Determine the dry weight of the flask (Wb), weight of bottle.

2. Fill the flask with distilled water to slightly below the calibration mark.

3. Place the filled flask into a water bath of about 30° to 35°C and allow it to remain until the water and flask reach the bath temperature.

4. Remove the flask from the bath and add or remove a slight amount of water until the bottom of the meniscus is even with the calibration

mark. Dry the outside of the flask and remove any excess moisture which may be adhering to the inside of the neck above the water surface.

5. Weigh the flask and water to the nearest 0.01 gram.

6. Shake the flask gently immediately after the weighing, and immerse a thermometer to about mid-depth in the water. Read and estimate the temperature of the water to the nearest 0.1°C.

7. Repeat the preceding six steps at approximately the same temperature (30° to 35°C).

8. Make two more determinations (preceding seven steps), one at room temperature and another at approximately 5°C cooler than room temperature.

9. Plot the weight of flask and water (W_{bw}) against the temperature (T) for each determination and draw a smooth curve through the plotted points (Figure 2-51). This is the calibration curve for the particular flask, which should be labeled and maintained for future reference.

TESTING

Specific gravity tests are usually easier to make if they begin with an oven dried sample. However, they may begin with the natural water content still present in the soil, and the oven-dried weights obtained after the test has been performed.

Soils with Natural Water Content.

Conduct the test using the following steps:

1. Record all identifying information for the sample on DD Form 1208 (Specific Gravity Test) (Figure 2-52, page 2-56).

2. Select a representative sample of approximately 50 to 80 grams and place it in a dish. The sample may be weighed directly in a preweighed flask to avoid possible loss of soil during transfer.

3. Add sufficient distilled water to form a slurry while mixing the sample with a spatula or stirrer.

4. Transfer the slurry to the volumetric flask and add distilled water until the flask is half full. Use a battery filler to ensure that all of the sample is washed into the flask. Allow the suspension to soak 4 to 6 hours. Soaking is not required on clean sands.

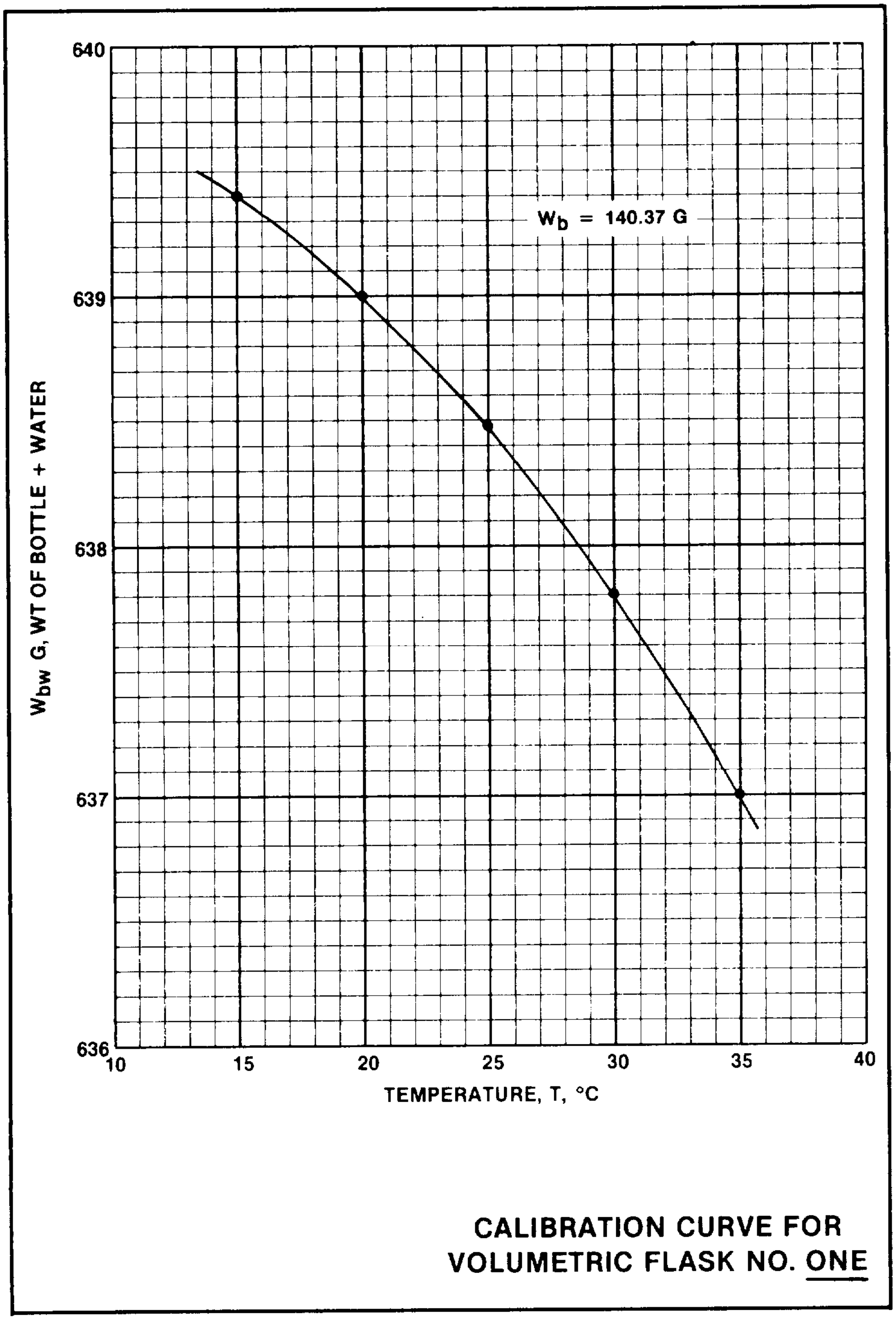
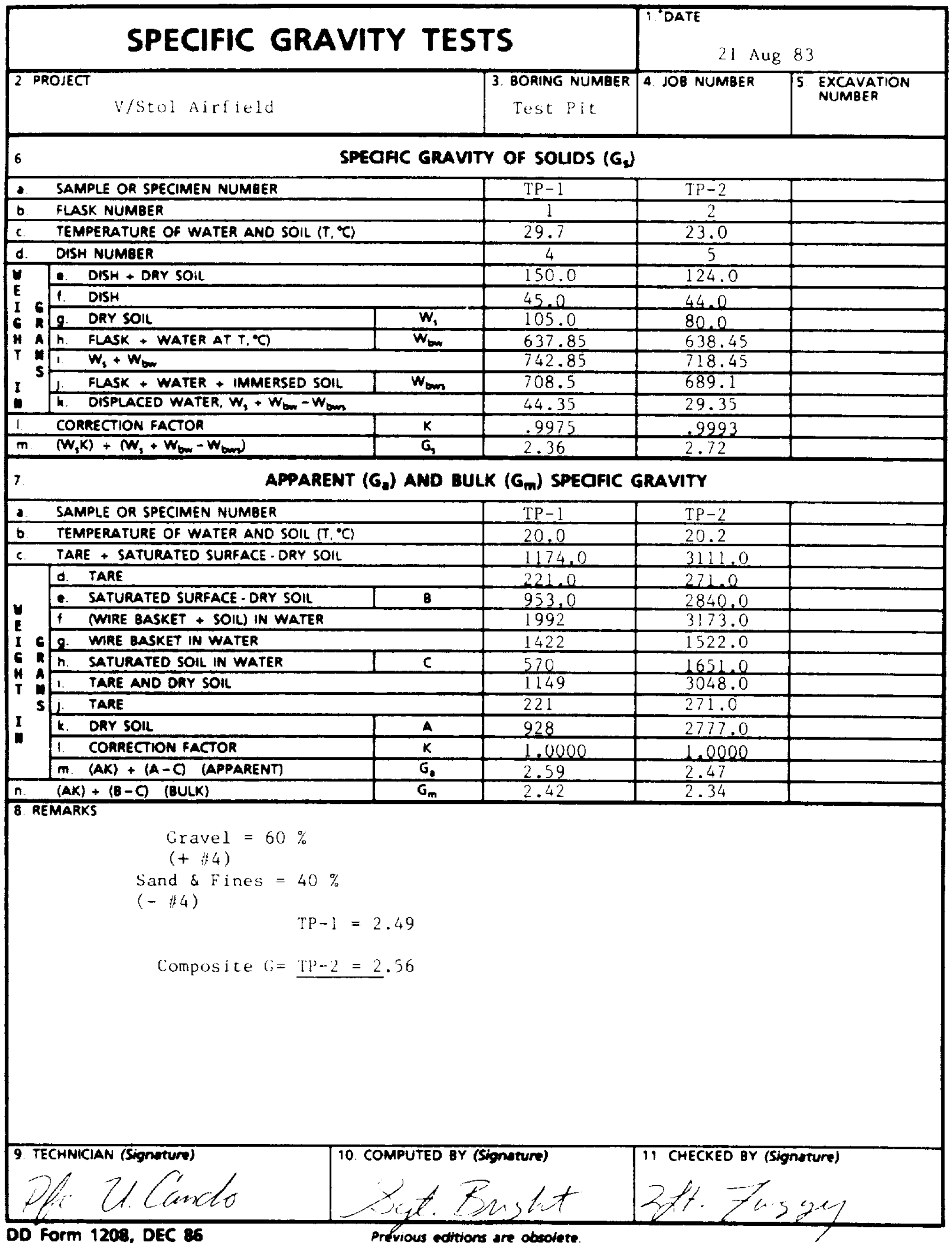


Figure 2-51. Calibration curve for a volumetric flask



SPECIFIC GRAVITY TESTS

1. DATE 21 Aug 83			
2. PROJECT V/Stol Airfield	3. BORING NUMBER Test Pit	4. JOB NUMBER	5. EXCAVATION NUMBER

6 SPECIFIC GRAVITY OF SOLIDS (G_s)

a. SAMPLE OR SPECIMEN NUMBER		TP-1	TP-2	
b. FLASK NUMBER		1	2	
c. TEMPERATURE OF WATER AND SOIL (T, °C)		29.7	23.0	
d. DISH NUMBER		4	5	
WEIGHT IN GRAMS e. DISH + DRY SOIL		150.0	124.0	
f. DISH		45.0	44.0	
g. DRY SOIL	W_s	105.0	80.0	
h. FLASK + WATER AT T, °C)	W_{bw}	637.85	638.45	
i. $W_s + W_{bw}$		742.85	718.45	
j. FLASK + WATER + IMMERSED SOIL	W_{bws}	708.5	689.1	
k. DISPLACED WATER, $W_s + W_{bw} - W_{bws}$		44.35	29.35	
l. CORRECTION FACTOR	K	.9975	.9993	
m. $(W_sK) \div (W_s + W_{bw} - W_{bws})$	G_s	2.36	2.72	

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

a. SAMPLE OR SPECIMEN NUMBER		TP-1	TP-2	
b. TEMPERATURE OF WATER AND SOIL (T, °C)		20.0	20.2	
c. TARE + SATURATED SURFACE - DRY SOIL		1174.0	3111.0	
WEIGHT IN GRAMS d. TARE		221.0	271.0	
e. SATURATED SURFACE - DRY SOIL	B	953.0	2840.0	
f. (WIRE BASKET + SOIL) IN WATER		1992	3173.0	
g. WIRE BASKET IN WATER		1422	1522.0	
h. SATURATED SOIL IN WATER	C	570	1651.0	
i. TARE AND DRY SOIL		1149	3048.0	
j. TARE		221	271.0	
k. DRY SOIL	A	928	2777.0	
l. CORRECTION FACTOR	K	1.0000	1.0000	
m. (AK) ÷ (A - C) (APPARENT)	G_a	2.59	2.47	
n. (AK) ÷ (B - C) (BULK)	G_m	2.42	2.34	

8. REMARKS

Gravel = 60 %
(+ #4)
Sand & Fines = 40 %
(- #4)
TP-1 = 2.49
Composite G= TP-2 = 2.56

9. TECHNICIAN *(Signature)* Pfc U. Cando	10. COMPUTED BY *(Signature)* Sgt. Bright	11. CHECKED BY *(Signature)* 2Lt. Foggy

DD Form 1208, DEC 86 *Previous editions are obsolete.*

Figure 2-52. Data sheet, specific gravity test

5. Connect the flask to a vacuum line and apply a vacuum of approximately 29.0 inches of mercury. Agitate the flask gently at intervals during the evacuation process. Soils of high plasticity will require about 6 to 8 hours to remove the air; soils of low plasticity require about 4 to 6 hours. A vacuum pump is not available in the test set.

6. Remove entrapped air by boiling the suspension gently for at least 10 minutes while occasionally rolling the flask to aid in the removal of entrapped air. A slow boil should be used as fast boiling may cause material to be boiled out of the flask. The use of indirect heat such as a sand bath is recommended. This consists of placing a pudding pan of clean ottawa sand directly onto a heat source. Allow the flask and contents to cool before proceeding.

7. Fill the flask with deaired, distilled water to about ¾ inch below the 500-milliliter graduation.

8. Test the deairing by applying a vacuum of 29.0 inches of mercury and observe the water surface in the flask, then carefully remove the vacuum. When a vacuum is not available, boil the suspension a second time. The suspension can be considered deaired sufficiently when the water surface in the flask is lowered less than ⅛ inch during this process.

9. Use a pipette to add more deaired, distilled water to the flask until the bottom of the meniscus is even with the calibration line on the neck of the flask.

10. Dry the outside of the flask and any moisture above the water surface inside the flask.

11. Weigh the flask and contents to the nearest 0.01 gram and record the information. This is (W_{bws}).

12. Stir the suspension immediately after weighing to assure even temperature distribution. Immerse a thermometer to the mid-depth of the flask. Read and record the temperature to the nearest 0.1°C.

13. Carefully transfer the contents of the flask to an evaporating dish. Rinse the flask with distilled water until the sample is entirely transferred to the evaporating dish.

14. Oven dry the sample at a temperature of 110° ± 5°C until it reaches a constant weight. Allow the soil to cool in a tightly covered container and weigh the soil and container.

15. Determine the dry weight of the test specimen, (W_s), to the nearest 0.01 gram and record the weight on the data sheet.

Oven-Dried Soils

When the soil can be dried before the test, use the following steps:

1. Record the information identifying the soil sample.

2. Oven dry a representative fraction approximately 50 to 80 grams of the sample to a constant weight at 110° ± 5°C, and cool the sample in the desiccator or airtight container to room temperature.

3. Weigh the sample to the nearest 0.01 gram and record the weight.

4. Transfer the soil to a volumetric flask taking care not to lose any material during the operation. As a check, reweigh the soil and flask after the transfer or weigh the sample in a preweighed tare initially.

5. Fill the flask about half full with distilled water.

6. Connect the flask to a vacuum of approximately 29.0 inches of mercury for 2 to 4 hours. Entrapped air may be removed using the

primarily stated procedure and to include a second boil.

7. Continue with the remainder of the test as described in steps 6 to 12. Procedures 13 to 15 can be used as a check on the initial weights.

CALCULATIONS

The test furnishes the following information (first column in Figure 2-52, page 2-56):

Weight of flask (bottle) plus water, plus solids at the test temperature (W_{bws} in grams).

Weight of dish/flask (container) plus dry soil (in grams).

Weight of dry soil, (W_s), obtained by subtracting the weight of the container from above.

Weight of flask plus water at test temperature (W_{bw} in grams) obtained from the calibration curve (figure 2-51, page 2-55).

To compute the specific gravity, (G_s), the following formula is used:

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

where K is a correction factor based on the density of water at 20°C (Table 2-4, page 2-62).

APPARENT AND BULK SPECIFIC GRAVITY

Apparent specific gravity and bulk specific gravity were defined earlier. The test is described below.

Preparation

Bulk specific gravity is usually determined for the coarser materials (retained on the No. 4 sieve) of a soil sample. Large stones may be determined individually.

- Separate the soil sample on a No. 4 sieve and use the material retained on the sieve for the test. A representative sample of approximately 2 kilograms is required.

- Do not oven dry the sample if at all possible. Air drying is permissible.

- Wash the sample to remove dust or coatings from the surface and immerse the sample in water for 24 hours.

- Just before making the test, remove the sample from the water and roll it in a large absorbent cloth to remove the excess surface moisture. Large particles may be wiped individually. The surfaces may still appear damp.

- Take care to avoid excessive evaporation during the surface drying.

Apparatus

The apparatus must be set up in a special manner to perform this test. The apparatus required for a bulk specific gravity test consists of the following:

Balance (capacity of 5 kilograms or more; sensitive to 0.1 gram)

Mesh, wire basket (approximately 8 inches in diameter and 8 inches high)

Container (large enough to permit immersing the wire basket)

Suitable equipment for suspending the wire basket from the center of the balance scale pan

Thermometer (range 0° to 50°C, graduated to 0.1°C)

Procedures

Perform the first step in the test as quickly as possible after the surface drying to avoid excessive evaporation.

- Weigh the saturated surface dry sample and record the value to the nearest 1.0 gram.

- Place the sample into the wire mesh basket and immerse the basket and sample into the water container.

NOTE

The weight of the wire basket in water must be determined and subtracted from the overall weight.

- Hang the basket from the balance and support the container so that the basket hangs freely in the water.

- Determine the weight of the sample in water and record the value.

- Measure and record the water temperature.

- Oven dry the sample at 110° ± 5°C to a constant weight.

- Cool the sample to room temperature, weigh the sample and record the weight.

Calculations

The test furnishes the dry soil weight (oven dried), the saturated surface dry weight, and the weight of the sample in water. Table 2-4, page 2-62, lists the bulk correction factor (K) based on water temperature.

- To determine the bulk specific gravity (G_m):

$$G_m = \frac{\text{weight of dry soil in air} \times K}{\text{weight of saturated sample in air} - \text{weight of sample in water}} = \frac{AK}{B\text{-}C}$$

- To determine the apparent specific gravity (G_a):

$$G_a = \frac{\text{weight of dry soil in air} \times K}{\text{weight of dry soil in air} - \text{weight of sample in water}} = \frac{AK}{A\text{-}C}$$

- To determine the specific gravity (G) of an entire soil sample (larger and smaller than the No. 4 sieve) from the component determinations:

$$G = \cfrac{100}{\cfrac{\text{\% passing No. 4 sieve}}{G_s} + \cfrac{\text{\% retained on No. 4 sieve}}{G_a}}$$

Record the composite G in the remarks column of the data sheet. The percentages of plus No. 4 and minus No. 4 can be determined for the original specific gravity. If all samples are representative, use the percentages from the sieve analysis.

EXPEDIENT METHODS

The method described above are the laboratory tests performed to determine the specific gravity of a soil sample. On occasion, the accuracy of these methods is not required or a quick determination is requested.

For instance, the water density correction factor (K) and the associated water temperature measurement, may be ignored in bulk specific gravity determination.

At times, there are stones present in the soil sample. An expedient method to determine the specific gravity of these particles is to tie a string or thread around the circumference and suspend the particle from the hook of a beam balance and weigh it to the nearest 0.1 gram (Figure 2-53). Note that this limits the minimum size of a particle that can be tested by this method since it must be large enough to be tied with a string. The particle may come from an oven-dried or saturated sample. A can, glass beaker, or container of water is set on a platform below the balance and the particle is immersed in the water (Figure 2-54). The suspended particle is not permitted to rest on the bottom nor to touch the sides of the container. The sample is weighed again to the nearest 0.1 gram. The weight of the thread or string is considered negligible and

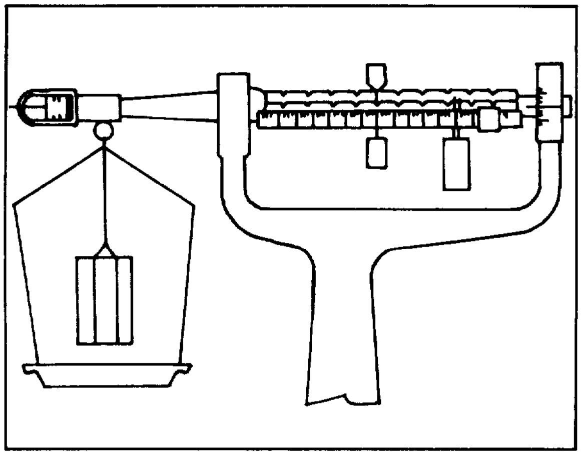


Figure 2-53. Expedient method of determining specific gravity: weighing in air

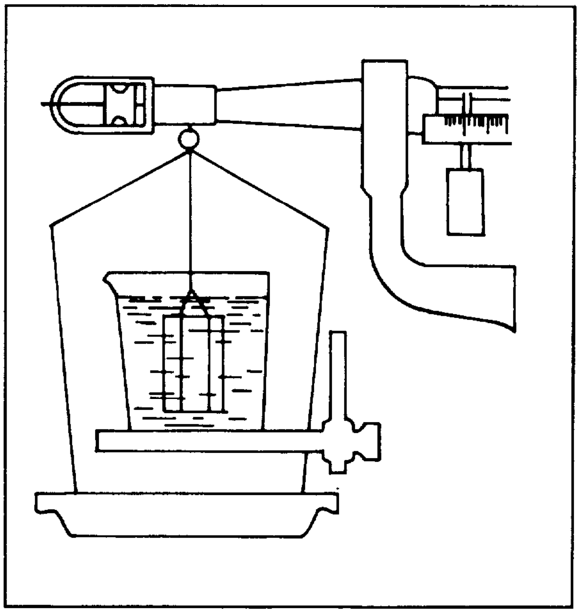


Figure 2-54. Expedient method of determining specific gravity: weighing in water

is disregarded in the calculations. The specific gravity (G_a) is calculated as follows:

$$G_a = \frac{\text{weight of specimen in air}}{\text{weight of specimen in air} - \text{weight of sample in water}}$$

A simple, quick method uses only a graduated cylinder. The air-dried sample is weighed to the nearest 0.1 gram. The graduate is filled with water to a predetermined volume. The sample is submerged in the water, and the change in the volume of water is noted. The change in water volume reading (in cubic centimeters) is equal to the volume of the sample. The same number (in grams) is the weight of an equal volume of water. The average specific gravity (Gavg) is computed as follows:

$$\text{Gavg} = \frac{\text{air dry weight of sample}}{\text{weight of equal volume of water}}$$

When a sample is too large to be submerged in a graduated cylinder another method can be used. A can or container large enough to take the sample is placed in a pan that will catch any overflow. The can is filled with water to the top. The air-dried and weighed sample is submerged in the can, and the overflow volume of water caught in the pan is measured. This value converted to grams is the weight of a sample. The formula for computing Gavg is:

$$G_m = \frac{\text{weight of air dried sample (in grams)}}{\text{weight of an equal volume of water (in grams)}}$$

Table 2-4. Relative density of water and correction factor (K) at various temperatures

Temp °C	Relative density	Correction factor, K	Temp °C	Relative density	Correction factor, K	Temp °C	Relative density	Correction factor, K
18.0	0.99862	1.0004	23.0	0.99756	0.9993	28.0	0.99626	0.9980
.1	60	4	.1	54	3	.1	23	0
.2	58	4	.2	51	3	.2	20	0
.3	56	3	.3	49	3	.3	17	0
.4	54	3	.4	46	2	.4	14	0.9979
.5	52	3	.5	44	2	.5	11	9
.6	50	3	.6	42	2	.6	08	9
.7	49	3	.7	39	2	.7	06	8
.8	47	2	.8	37	1	.8	03	8
.9	45	2	.9	34	1	.9	00	8
19.0	0.99843	1.0002	24.0	0.99732	0.9991	29.0	0.99597	0.9977
.1	41	2	.1	29	1	.1	94	7
.2	39	2	.2	27	0	.2	91	7
.3	37	1	.3	24	0	.3	88	6
.4	35	1	.4	22	0	.4	85	6
.5	33	1	.5	20	0	.5	82	6
.6	31	1	.6	17	0.9989	.6	79	6
.7	29	1	.7	14	9	.7	76	5
.8	27	0	.8	12	9	.8	73	5
.9	25	0	.9	09	9	.9	07	5
20.0	0.99823	1.0000	25.0	0.99707	0.9988	30.0	0.99567	0.9974
.1	21	0	.1	04	8	.1	64	4
.2	19	0	.2	02	8	.2	61	4
.3	17	0.999	.3	0.99699	8	.3	58	3
.4	15	9	.4	97	7	.4	55	3
.5	13	9	.5	94	7	.5	52	3
.6	10	9	.6	91	7	.6	49	3
.7	08	8	.7	89	7	.7	46	2
.8	06	8	.8	87	6	.8	43	2
.9	04	8	.9	84	6	.9	40	2
21.0	0.99802	0.9998	26.0	0.99681	0.9986	31.0	0.99537	0.9971
.1	00	8	.1	78	6	.1	33	1
.2	0.99798	8	.2	76	5	.2	30	1
.3	96	7	.3	73	5	.3	27	0
.4	93	7	.4	70	5	.4	24	0
.5	91	7	.5	68	4	.5	21	0
.6	89	7	.6	65	4	.6	18	0.9969
.7	87	6	.7	63	4	.7	15	9
.8	85	6	.8	60	4	.8	12	9
.9	83	6	.9	57	3	.9	09	9
22.0	0.99780	0.9996	27.0	0.99654	0.9983	32.0	0.99505	0.9968
.1	78	6	.1	51	3	.1	02	8
.2	75	5	.2	48	2	.2	0.99499	8
.3	73	5	.3	46	2	.3	96	7
.4	70	5	.4	43	2	.4	93	7
.5	68	5	.5	40	2	.5	90	7
.6	65	4	.6	37	1	.6	86	6
.7	63	4	.7	34	1	.7	83	6
.8	61	4	.8	32	1	.8	80	6
.9	58	4	.9	29	1	.9	77	5

Note: Relative density of water based on density of water at 4°C equal to unity. Data obtained from Smithsonian Tables, compiled by various authors. Correction factor, K, is found by dividing the relative density of water at the test temperature by the relative density of water at 20°C.

Section V. HYDROMETER ANALYSIS

DISCUSSION

Hydrometer analysis is based on Stokes' law which relates the terminal velocity of a free-falling sphere in a liquid to its diameter. The relation is expressed by the following equation.

$$V = \frac{G_s - G_w}{18\,n} D^2$$

where:

V = terminal velocity

G_s = specific gravity of solids

G_w = specific gravity of the liquid in which the sphere is falling

n = viscosity of the liquid

D = diameter of the sphere.

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 the smaller ones. The hydrometer analysis is an application of Stokes' law that permits the calculation of the grain-size distribution in silts and clays, where the soil particles are given the sizes of equivalent spherical particles.

The density of a soil-water suspension depends upon the concentration and specific gravity of the soil particles. If the suspension is allowed to stand, the particles will gradually settle out of the suspension, and the density will be decreased. The hydrometer is the instrument used to measure the density of the suspension at a known depth below the surface. The density measurement, together with knowledge of specific gravity of the soil particles, determines the percentage of dispersed soil particles in suspension at the time and depth of measurement. The depth at which the measurement is made is found by calibrating the hydrometer. Stokes' law is used to calculate the maximum equivalent particle diameter for the material in suspension at this depth and for the elapsed time of settlement. A series of density measurements at known depth of suspension and at known times of settlement gives the percentages of particles finer than the diameters given by Stokes' law. Thus, the series of readings will reflect the amount of different sizes of particles in the fine grained soils. The particle diameter (D) is calculated from Stokes' equation using the corrected hydrometer reading. Use the nomographic chart for the solution of Stokes' equation.

PREPARATION

Prior to making the test, the hydrometer must be calibrated and a meniscus correction determined. A dispersing agent should be acquired.

Hydrometer Calibration

The hydrometer must be calibrated to determine its true depth in terms of the hydrometer reading (Figure 2-55A, page 2-64).

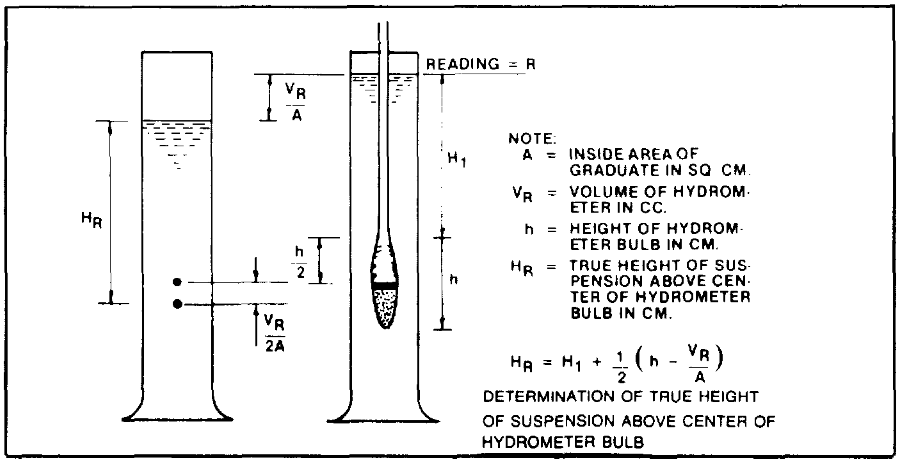

READING = R
$\frac{V_R}{A}$
H_R
H_1
$\frac{h}{2}$
h
$\frac{V_R}{2A}$
NOTE:
A = INSIDE AREA OF GRADUATE IN SQ CM.
V_R = VOLUME OF HYDROMETER IN CC.
h = HEIGHT OF HYDROMETER BULB IN CM.
H_R = TRUE HEIGHT OF SUSPENSION ABOVE CENTER OF HYDROMETER BULB IN CM.
$H_R = H_1 + \frac{1}{2}\left(h - \frac{V_R}{A}\right)$
DETERMINATION OF TRUE HEIGHT OF SUSPENSION ABOVE CENTER OF HYDROMETER BULB

Figure 2-55A. Hydrometer calibration

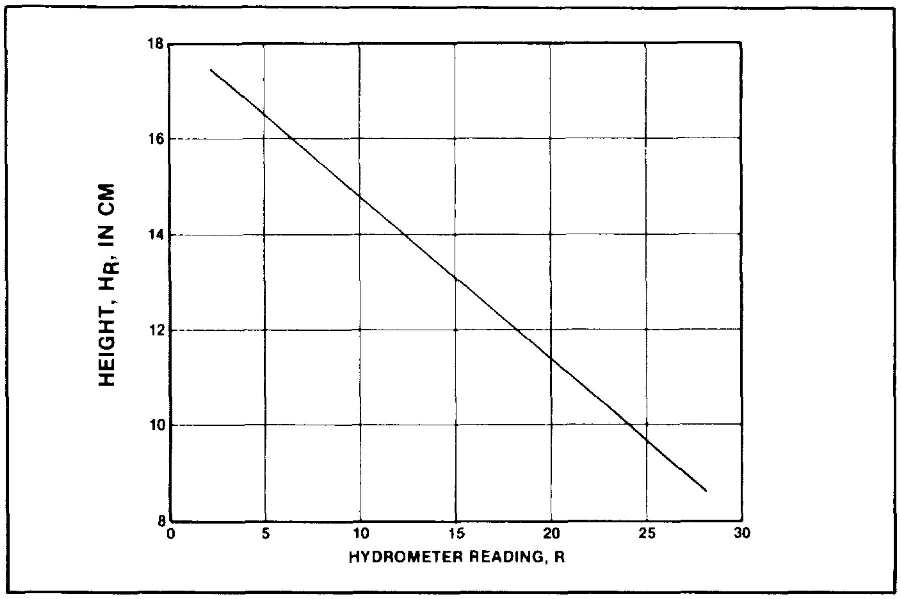


Figure 2-55B. Typical hydrometer calibration curve

- Determine the volume of the bulb, either by measuring the volume of water displaced or by weighing the hydrometer. Since the specific gravity of the hydrometer is about 1, the weight in grams may be recorded as the volume in cubic centimeters.

- Compute the area of the graduate (A). Measure the distance between two graduations. The volume between two measured marks is divided by the measured distance to obtain the area.

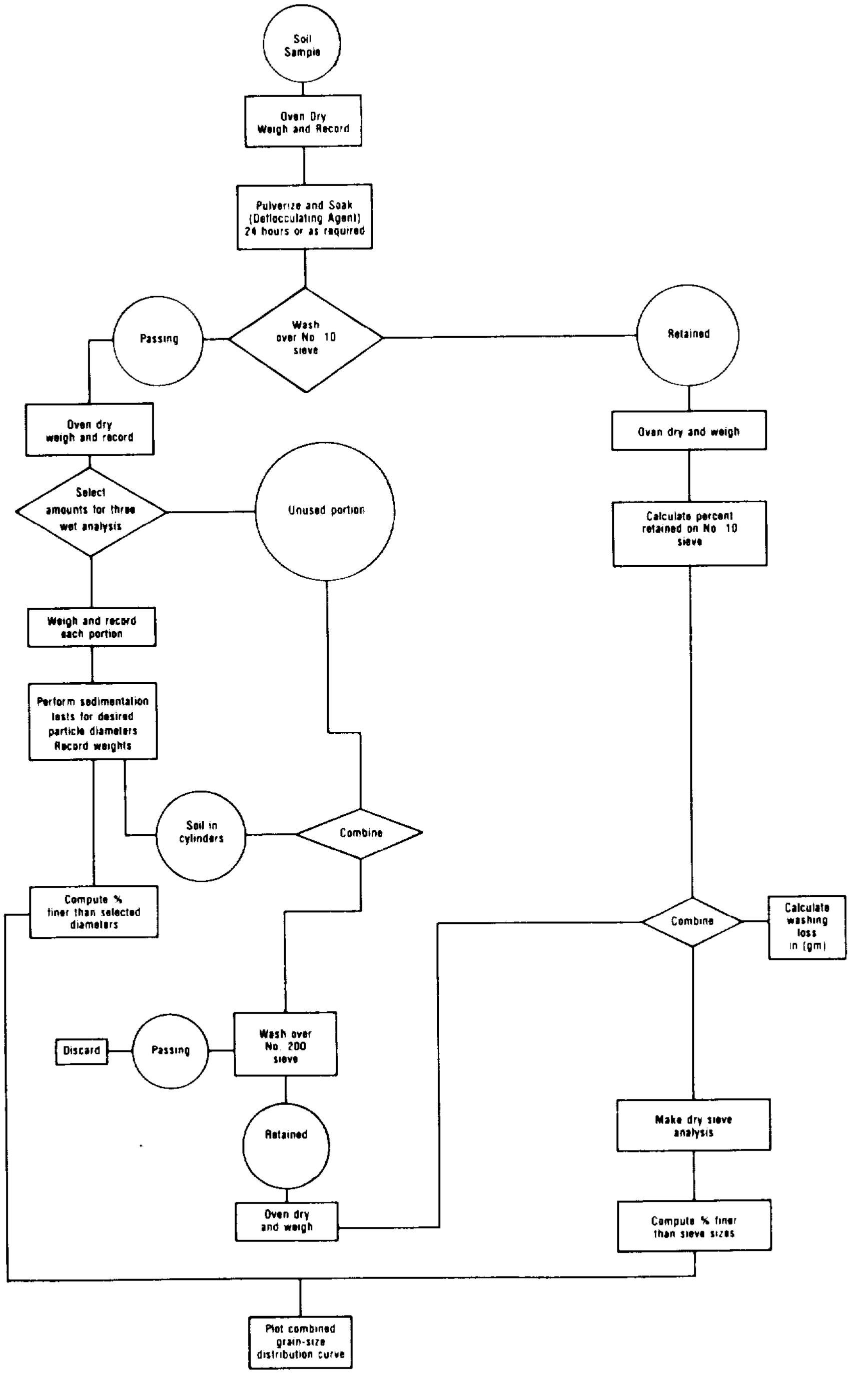
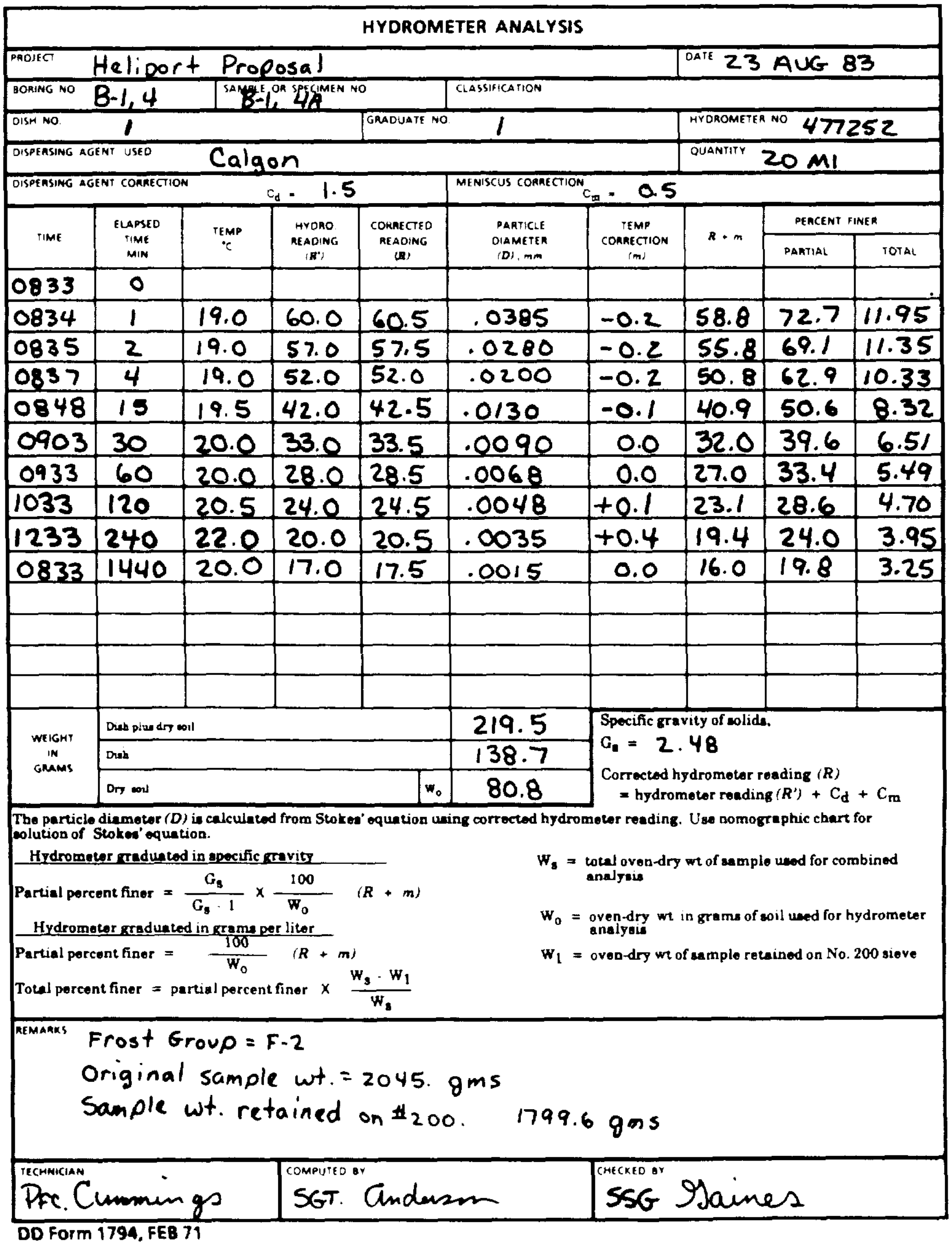

Soil Sample
Oven Dry Weigh and Record
Pulverize and Soak (Deflocculating Agent) 24 hours or as required
Passing
Wash over No 10 sieve
Retained
Oven dry weigh and record
Select amounts for three wet analysis
Unused portion
Oven dry and weigh
Calculate percent retained on No 10 sieve
Weigh and record each portion
Perform sedimentation tests for desired particle diameters Record weights
Soil in cylinders
Combine
Compute % finer than selected diameters
Combine
Calculate washing loss in (gm)
Discard
Passing
Wash over No. 200 sieve
Retained
Oven dry and weigh
Make dry sieve analysis
Compute % finer than sieve sizes
Plot combined grain-size distribution curve

Figure 2-56. Combined mechanical analysis, sedimentation method



HYDROMETER ANALYSIS

PROJECT Heliport Proposal
DATE 23 AUG 83
BORING NO B-1, 4
SAMPLE OR SPECIMEN NO B-1, 4A
CLASSIFICATION
DISH NO. 1
GRADUATE NO. 1
HYDROMETER NO 477252
DISPERSING AGENT USED Calgon
QUANTITY 20 MI
DISPERSING AGENT CORRECTION C_d = 1.5
MENISCUS CORRECTION C_m = 0.5

TIME	ELAPSED TIME MIN	TEMP °C	HYDRO. READING (R')	CORRECTED READING (R)	PARTICLE DIAMETER (D), mm	TEMP CORRECTION (m)	R + m	PERCENT FINER	
								PARTIAL	TOTAL
0833	0								
0834	1	19.0	60.0	60.5	.0385	-0.2	58.8	72.7	11.95
0835	2	19.0	57.0	57.5	.0280	-0.2	55.8	69.1	11.35
0837	4	19.0	52.0	52.0	.0200	-0.2	50.8	62.9	10.33
0848	15	19.5	42.0	42.5	.0130	-0.1	40.9	50.6	8.32
0903	30	20.0	33.0	33.5	.0090	0.0	32.0	39.6	6.51
0933	60	20.0	28.0	28.5	.0068	0.0	27.0	33.4	5.49
1033	120	20.5	24.0	24.5	.0048	+0.1	23.1	28.6	4.70
1233	240	22.0	20.0	20.5	.0035	+0.4	19.4	24.0	3.95
0833	1440	20.0	17.0	17.5	.0015	0.0	16.0	19.8	3.25

WEIGHT IN GRAMS	Dish plus dry soil		219.5
	Dish		138.7
	Dry soil	W_o	80.8

Specific gravity of solids,
G_s = 2.48

Corrected hydrometer reading (R)
= hydrometer reading (R') + C_d + C_m

The particle diameter (D) is calculated from Stokes' equation using corrected hydrometer reading. Use nomographic chart for solution of Stokes' equation.

Hydrometer graduated in specific gravity

$$\text{Partial percent finer} = \frac{G_s}{G_s - 1} \times \frac{100}{W_0} (R + m)$$

Hydrometer graduated in grams per liter

$$\text{Partial percent finer} = \frac{100}{W_0} (R + m)$$

$$\text{Total percent finer} = \text{partial percent finer} \times \frac{W_s - W_1}{W_s}$$

W_s = total oven-dry wt of sample used for combined analysis

W_0 = oven-dry wt in grams of soil used for hydrometer analysis

W_1 = oven-dry wt of sample retained on No. 200 sieve

REMARKS
Frost Group = F-2
Original sample wt. = 2045. gms
Sample wt. retained on #200. 1799.6 gms

TECHNICIAN Pfc. Cummings
COMPUTED BY SGT. Anderson
CHECKED BY SSG Gaines

DD Form 1794, FEB 71

Figure 2-57. Data sheet, hydrometer analysis

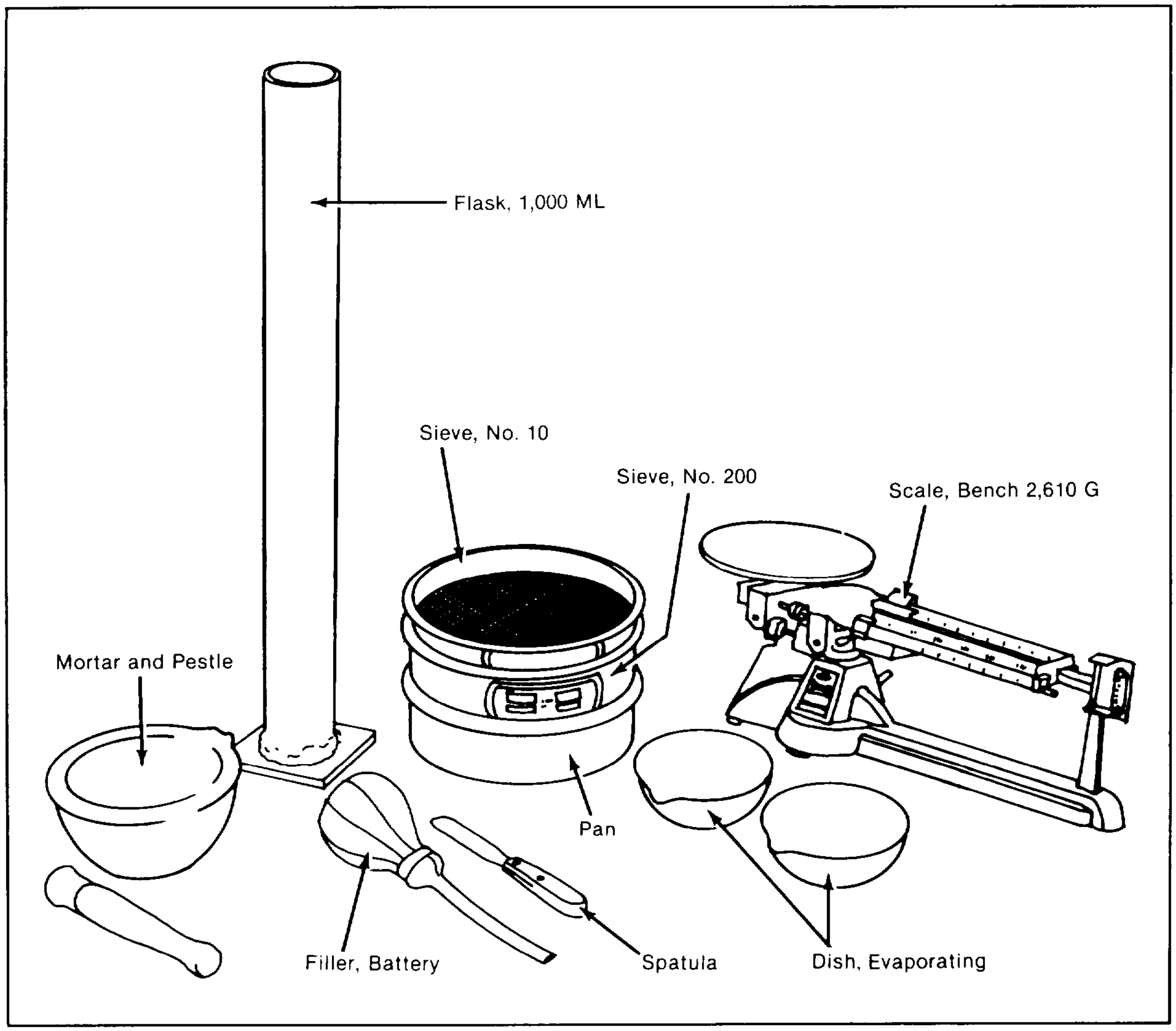

Flask, 1,000 ML
Sieve, No. 10
Sieve, No. 200
Scale, Bench 2,610 G
Mortar and Pestle
Pan
Filler, Battery
Spatula
Dish, Evaporating

Figure 2-58. Apparatus for grain size distribution by hydrometer analysis

Measure the distance from the lowest graduation to the other major graduation (R).

- Measure the distance from the neck of the bulb to the lowest graduation. The distance H1, corresponding to a reading, R, equals the sum of the two distances measured in the following steps.

- Measure the distance from the neck of the bulb to the tip of the bulb. This value is h, the height of the bulb, and h/2 locates the center of volume of a symmetrical bulb. If a nonsymmetrical bulb is used, the center of volume can be determined with sufficient accuracy by projecting the shape of the bulb onto a sheet of paper and locating the center of gravity of the projected area.

- The measured values are substituted in the following formula:

$$H_R = H_1 + \frac{1}{2}\frac{(h - V_R)}{A}$$

where:

H_R = true height of suspension above the center of the hydrometer bulb in centimeters)

H_1 = distance corresponding to the reading on the hydrometer (in centimeters)

h = height of hydrometer bulb (in centimeters)

V_R = volume of hydrometer (in cubic centimeters)

A = inside area of graduate (in square centimeters)

- By plotting the true height (H_R) against the reading (R) for the major graduations, a calibration curve can be drawn. This relation is essentially a straight line for a hydrometer having a streamlined shape, Figure 2-55B, page 2-64 shows a typical calibration curve.

Meniscus Correction

Hydrometers are calibrated to read correctly at the surface of the liquid. Soil suspensions are not transparent, and the reading is made not at the surface, but where the upper rim of the meniscus touches the glass. A correction for this meniscus (C_M) must be determined using clear distilled or demineralized water. This correction will be constant for a given hydrometer. The hydrometer is immersed in the water, and the difference in height between the surface and the meniscus is carefully determined. This value (C_M) is approximately 0.5, which may be used for routine work where greater accuracy is not required.

Preparation of Sample

The soil is separated on the No. 10 or No. 200 sieve depending on the soil of interest, and the percentage finer than this sieve is determined. Samples for hydrometer analysis are taken from representative material finer than the No. 10 sieve as illustrated in the flow chart (Figure 2-56, page 2-65).

- The approximate size of a sample for hydrometer analysis varies according to the type of soil being tested.

Soil type	Dry weight in grams
Fat clays	30
Lean clays and silty soils	50
Sandy soils	75

- The exact dry weight of the sample in suspension must be determined either before or after the test. For economy of time and labor, it is preferable to oven dry and weigh the sample before the test. In this case, any loss of material during the test will affect the results. Also, for some clays, drying may cause permanent changes in the grain sizes. Samples of such soils should be preserved at natural water content, if possible, and tested before oven drying. The dry weight is determined at the conclusion of the hydrometer analysis.

Dispersing Agent

Very fine soil grains in suspension normally tend to flocculate (adhere to each other in clusters) and to settle through the solution as a larger unit. This leads to errors in the analysis.

- To prevent flocculation, a dispersing agent is added to the solution which keeps the individual particles separated. In most instances, 15 milliliters of a dispersing agent solution is adequate. If flocculation does occur, another sample must be prepared using an increased amount of dispersing agent (20 milliliters). The agents listed in Table 2-5 are satisfactory for most types of soil. They are listed in the approximate order of effectiveness.

- Sodium silicate should not be used. Phosphate solutions are somewhat unstable and should not be stored for extended periods of time.

Table 2-5. Dispersing agents

Dispersing agent	Stock solution	
	Concentration	G / L
Sodium tripolyphosphate	0.4N	29
Sodium polyphosphate	0.4N	36
Sodium tetraphosphate Quadrofos*	0.4N	31
Sodium hexametaphosphate (Calgon)**	0.4N	41

*Trade names
**Calgon is buffered with Na_2CO_3

- A dispersing agent increases the liquid density, and a correction must be made to the observed reading. The correction (C_d) is determined by first adding the amount of dispersing agent to be used to a graduate partially filled with distilled or demineralized water. The graduate is then filled to the 1,000 milliliter mark with distilled or demineralized water and stirred or shaken to mix the solution. A hydrometer reading is taken (Figure 2-56, page 2-65). The C_d is the difference between this reading and another taken in pure distilled or demineralized water.

Other Considerations

- Temperature changes can affect a hydrometer test. The test should be conducted where temperature variations are at a minimum. The solution should be kept away from heat sources such as radiators and sunlight and from places where temperatures can change, such as near open windows.

- If oven-dry weight is required before the test, the sample should be dried in the oven, allowed to cool to room temperature, and weighed to the nearest 0.1 gram. This dry weight should be recorded.

HYDROMETER TEST

Apparatus

Equipment for performing this test, SC6635-98-CL-EO2-HR, 19 APR 82, is not in the soil test set (Figure 2-58, page 2-67). The apparatus required includes the following:

- Soil hydrometer (calibrated at 20°C – graduated in specific gravity; range 0.995 to 1.040, accuracy ± 0.001; or graduated in grams per liter, range 0 to 50, accuracy ± 1)

- Dispersion cup and stirrer to keep the soil particles dispersed, (Figure 2-59, page 2-70).

- Sedimentation cylinder of glass (essentially 18 inches high, $2\frac{1}{2}$ inches in diameter, and marked for a volume of 1,000 milliliters).

Centigrade thermometer range 0° to 50°C, and accurate to 0.5°C

Timing device (watch or clock) with second hand

Balance, sensitive to 0.1 gram

Oven

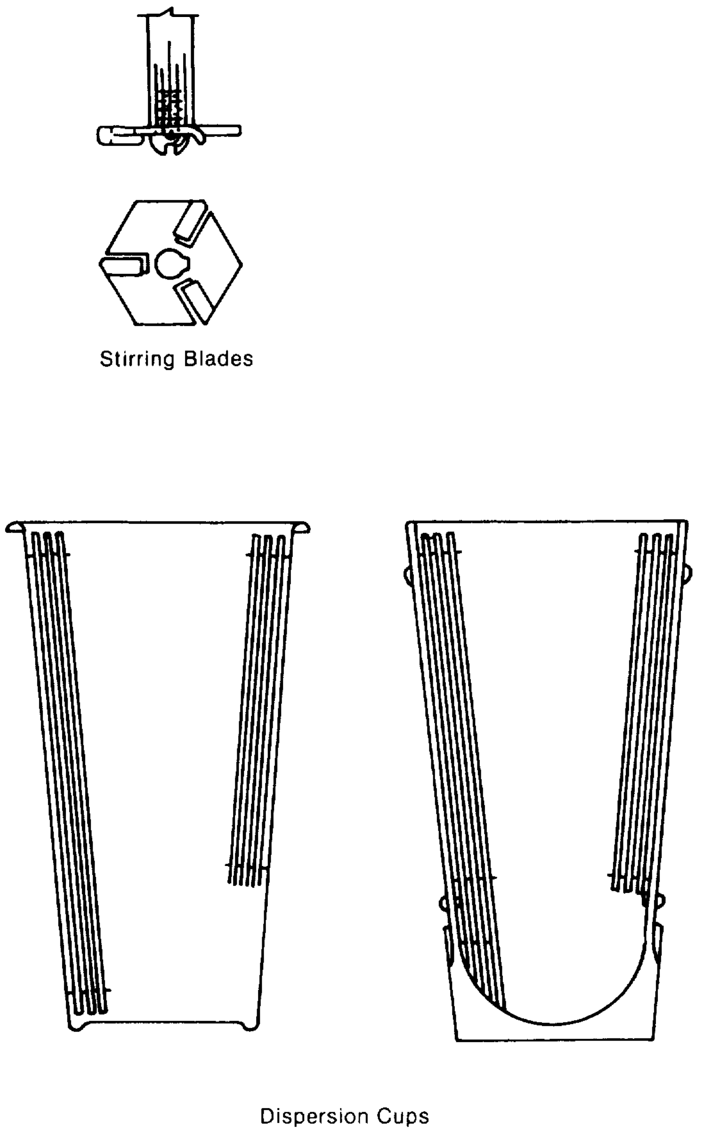
A sheltered location for taking readings is required. The sedimentation cylinders must not be disturbed during the test. A steady bench or a level floor in a location having adequate light for taking readings should be provided.

Preparation

- Record all identifying information for the sample on DD Form 1794, Hydrometer Analysis (Figure 2-57, page 2-67).

- Record the C_m and the (C_d).

- Determine or estimate the specific gravity of the solids and record this value.



Stirring Blades

Dispersion Cups

Figure 2-59. Detail of stirring paddle and dispersion cups

Procedure

Place the sample in a numbered dish and add sufficient distilled or demineralized water until the sample is submerged. Add the dispersing agent. Allow the sample to soak overnight or until all soil lumps have disintegrated.

- Transfer the soil-water slurry from the dish to a dispersing cup, washing (with distilled water) any residue in the dish into the cup. Add distilled water to the cup until the surface is 2 to 3 inches below the top, or too much water will splash out during the mixing and carry out some of the soil sample. Place the cup into the dispersing machine and mix for 1 to 10 minutes depending on the plasticity of the soil; the lower the plasticity, the shorter the mixing time.

- Transfer the mixed suspension into a 1,000 milliliter sedimentation cylinder, rinse any material clinging to the sides of the dispersing cup, and add distilled or demineralized water until the volume equals 1,000 milliliters. At this time, bring the suspension to the temperature expected to prevail during the test.

- One minute before starting the test, hold the graduate in one hand and place the other hand or a suitable rubber cap over the open end to act as a stopper. Shake the suspension vigorously for a few seconds to raise any sediment from the bottom of the graduate into suspension. Continue the agitation for the remainder of the minute by turning the cylinder upside down and back. In some cases, the sediment at the bottom may have to be loosened with a glass rod before shaking. Figure 2-60 illustrates a type of hand agitator which may be used instead of shaking. This agitator prevents the accumulation of sediment at the base and on the sides of the graduate. It is moved up and down through the suspension for the minute.

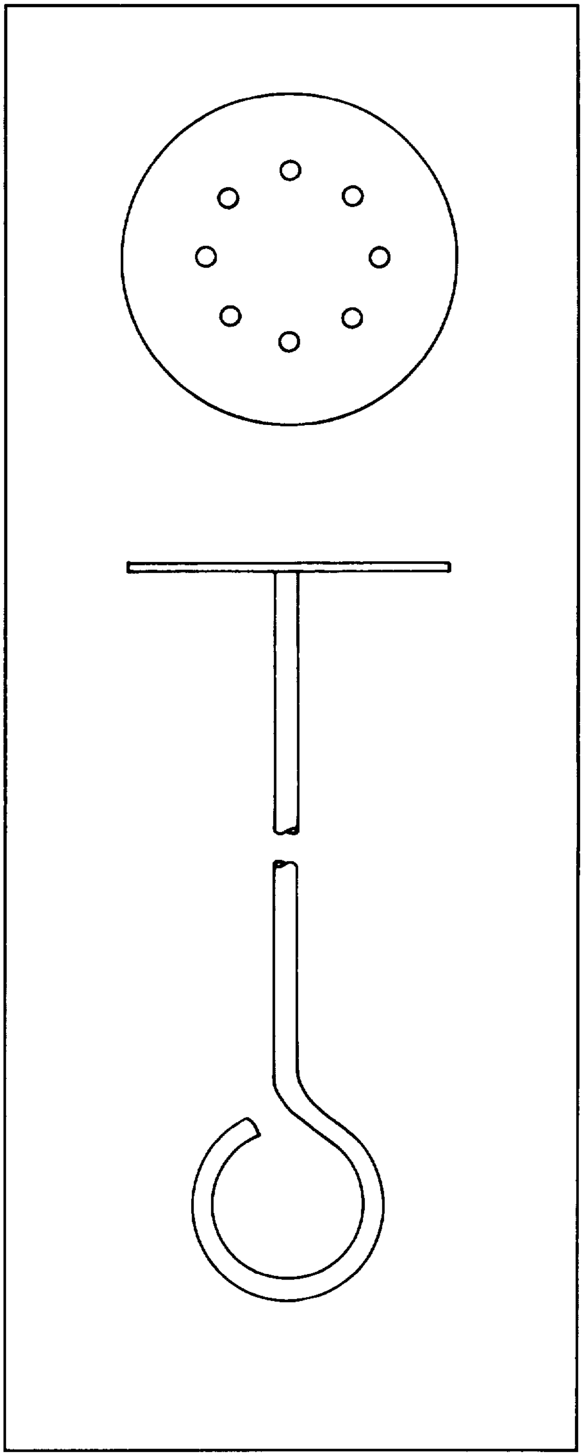


Figure 2-60. Hand agitator

- After 1 minute of agitation, set the cylinder on a table. If foam has developed on the surface of the suspension, remove it by touching it lightly with a piece of soap. Slowly immerse the hydrometer in the liquid, wait 20 to 25 seconds, and take a reading. Take care in inserting and removing the hydrometer to prevent disturbing the suspension.

- Observe and record hydrometer readings after 1 and 2 minutes have elapsed from the time the cylinder was set down. Make all readings at the top of the meniscus. As each reading is completed, remove the hydrometer carefully and place it in a container of clean water. The soil material will settle on or adhere to the hydrometer if it is left in the suspension and cause an error in the readings.

- Repeat the readings and record them after elapsed times of 4, 15, and 30 minutes and 1, 2, 4, and 24 hours. Remove the hydrometer from the suspension after each reading and place it in a graduate of clean water. When the hydrometer analysis is used for soil classification purposes, the 4 hour and 24 hour readings may not be required.

- With hydrometers graduated to read grams per liter, record the actual reading. With hydrometers graduated to read specific gravity, read only the last two figures and estimate the third. Record the specific gravity indicated reading, minus 1,000, multiplied by 1,000. For example, a reading of 1.0225 is recorded as 22.5.

$$1.0225 - 1.000 = .0225$$
$$.0225 \times 1.000 = 22.5$$

- At the end of the 2-minute hydrometer reading and after each subsequent reading, place a thermometer in the suspension and record the temperature to ± 0.5°C. Temperature changes will affect the test results.

- If the dry weight had not been determined, carefully wash all of the suspension into an evaporating dish. Oven dry the material, allow it to cool, and determine the sample weight.

Calculations

The recorded readings must be corrected and converted into size distribution of the particles.

- Corrected hydrometer readings (R). Add the C_m to the actual hydrometer readings, (R'). The result is the corrected hydrometer readings, (R).

- Particle diameter. The particle diameter is calculated for each reading on the basis of Stokes' equation using the nomograph shown in Figure 2-61. The hydrometer calibration furnishes the true height (H_R) on the hydrometer. The R-scale is designed for the particular hydrometer used in the test. The nomograph contains the key which shows the steps to follow in computing the diameter (D) at each reading. Place a straightedge (engineer's triangle or transparent scale) on the specific gravity of solids (point one) and the temperature at the time of reading (point two). Mark the intersection of this line on the A-scale (point three). Refer to the hydrometer calibration for the H_R and locate it on the scale (point four). Using the straightedge, connect point four and the time of reading (point five) to locate point six, the velocity of settlement. Place the straightedge on points three and six and read the particle diameter (D) (point seven) in millimeters at the intersection of this line and the scale of diameters. Record the diameter (D) found on the data sheet.

- Temperature correction. The temperature at each reading is used to determine the correction factor (m) from Table 2-6, page 2-74.

- Percent passing. Before determining the percent passing, the dispersing agent correction and the temperature correction must be added to the corrected hydrometer reading, $R - C_d + m$.

- Partial percent passing. The grain size distribution is computed as a percentage.

Hydrometer calibrated in specific gravity.

$$\text{Percent finer (by weight)} = \frac{G_s}{G_{s1}} \times \frac{100}{W_o} \times (R + m)$$

Hydrometer calibrated in grams per liter.

$$\text{Percent finer (by weight)} = \frac{100}{W_o} \times (R - C_d + m).$$

where:

G = specific gravity of solids

W_o = oven-dry weight of soil (in grams) used for hydrometer analysis

$R - C_d$ = corrected hydrometer reading plus dispersing agent correction and temperature correction.

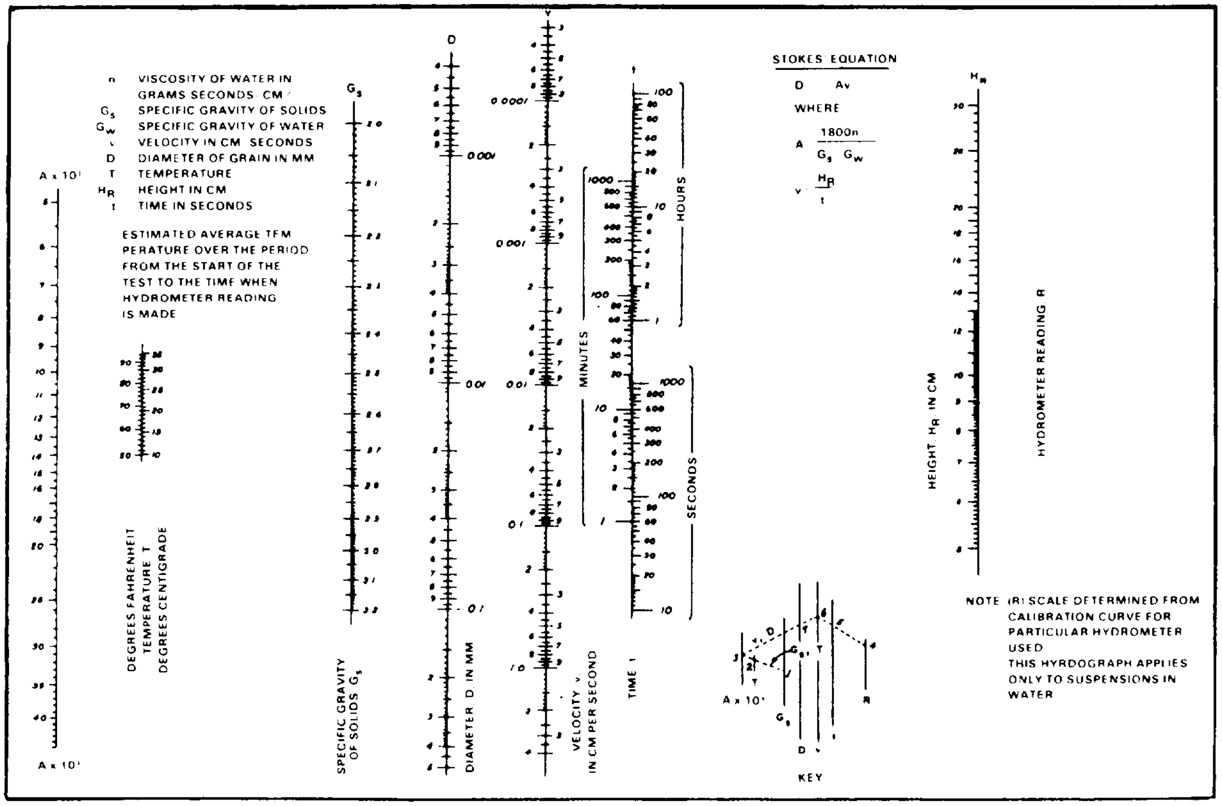

n VISCOSITY OF WATER IN GRAMS SECONDS CM
G_s SPECIFIC GRAVITY OF SOLIDS
G_w SPECIFIC GRAVITY OF WATER
v VELOCITY IN CM SECONDS
D DIAMETER OF GRAIN IN MM
T TEMPERATURE
H_R HEIGHT IN CM
t TIME IN SECONDS
ESTIMATED AVERAGE TEMPERATURE OVER THE PERIOD FROM THE START OF THE TEST TO THE TIME WHEN HYDROMETER READING IS MADE
A x 10'
DEGREES FAHRENHEIT
TEMPERATURE T
DEGREES CENTIGRADE
G_s
SPECIFIC GRAVITY OF SOLIDS G_s
D
DIAMETER D IN MM
0.0001
0.001
0.01
0.1
v
VELOCITY v IN CM PER SECOND
TIME t
HOURS
MINUTES
SECONDS
STOKES EQUATION
D Av
WHERE
A $\frac{1800n}{G_s \; G_w}$
v $\frac{H_R}{t}$
H_R
HEIGHT H_R IN CM
HYDROMETER READING R
KEY
A x 10'
G_s
D
v
T
t
R
NOTE (R) SCALE DETERMINED FROM CALIBRATION CURVE FOR PARTICULAR HYDROMETER USED
THIS HYRDOGRAPH APPLIES ONLY TO SUSPENSIONS IN WATER

Figure 2-61. Stokes' law nomograph

- Total percent passing. Total percent passing = partial percent passing × $W_s - W_1/W_s$

where:

W_S = total oven dry weight of sample used for combined analysis

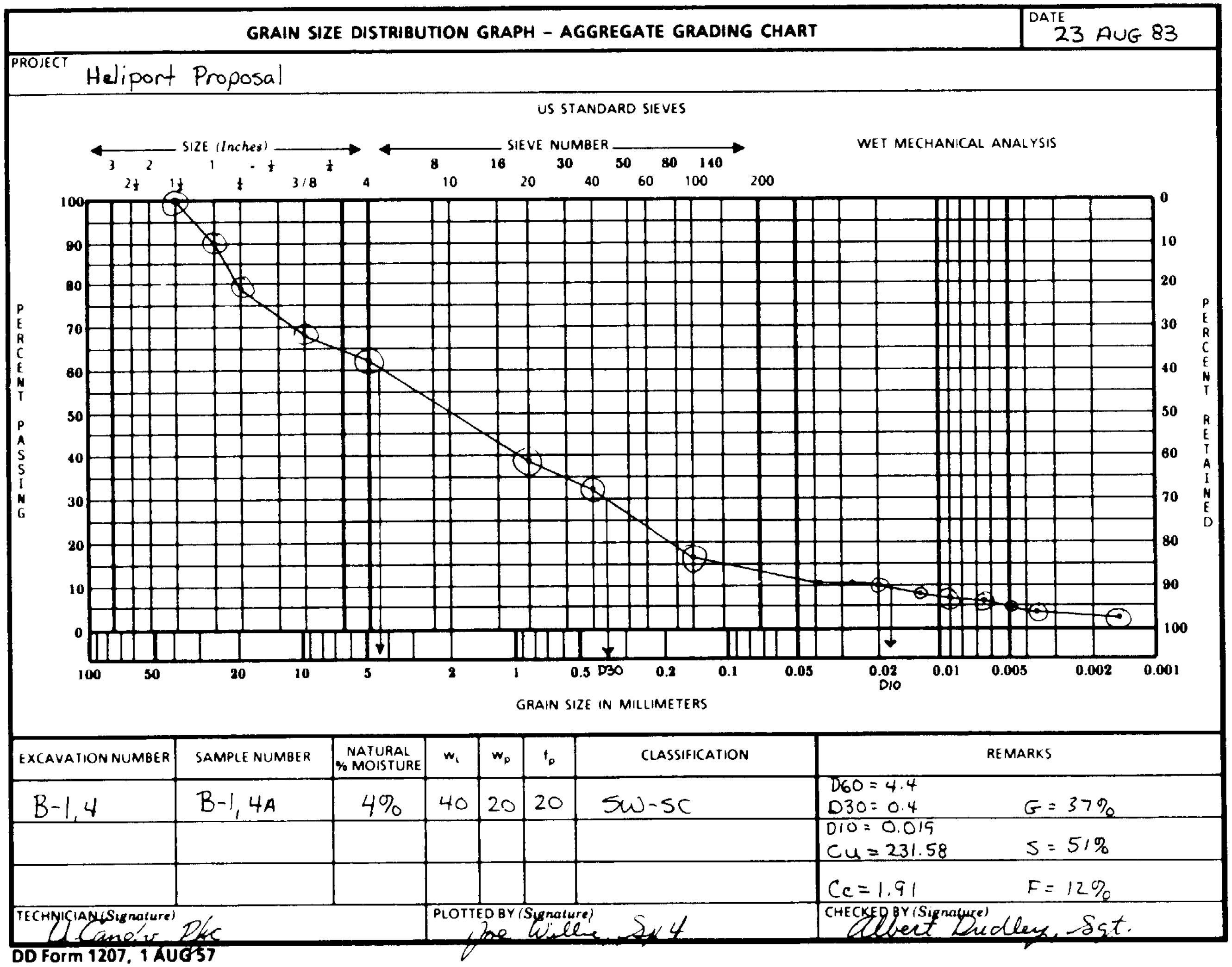
W_1 = oven dry weight of sample retained on plus No. 200 sieve

Presentation of Results

Hydrometer analysis separates the fine particles of a soil into a grain-size distribution. The percentages can be plotted on a grain size distribution graph (DD Form 1207, Figure 2-62) using the particle diameters (D), and the percentages.

Table 2-6. Temperature correction factor (m) for hydrometer analysis

°C	°F	m	°C	°F	m
14.9	57.2	−0.9	24.0	75.2	+0.8
14.5	58.1	−0.8	24.5	76.1	+0.9
15.0	59.0	−0.8	25.0	77.0	+1.0
15.5	59.9	−0.7	25.5	77.9	+1.1
16.0	60.8	−0.6	26.0	78.8	+1.3
16.5	61.7	−0.6	26.5	79.7	+1.4
17.0	62.6	−0.5	27.0	80.6	+1.5
17.5	63.5	−0.4	27.5	81.5	+1.6
18.0	64.4	−0.4	28.0	82.4	+1.8
18.5	65.3	−0.3	28.5	83.3	+1.9
19.0	66.2	−0.2	29.0	84.2	+2.1
19.5	67.1	−0.1	29.5	85.1	+2.2
20.0	68.0	0.0	30.0	86.0	+2.3
20.5	68.9	+0.1	30.5	86.9	+2.5
21.0	69.8	+0.2	31.0	87.8	+2.6
21.5	70.7	+0.3	31.5	88.7	+2.8
22.0	71.6	+0.4	32.0	89.6	+2.9
22.5	72.5	+0.5	32.5	90.5	+3.0
23.0	73.4	+0.6	33.0	91.4	+3.2
23.5	74.3	+0.7	33.5	92.3	+3.3
			34.0	93.2	+3.5



GRAIN SIZE DISTRIBUTION GRAPH - AGGREGATE GRADING CHART

DATE 23 AUG 83

PROJECT Heliport Proposal

EXCAVATION NUMBER	SAMPLE NUMBER	NATURAL % MOISTURE	w_L	w_P	I_P	CLASSIFICATION	REMARKS
B-1, 4	B-1, 4A	4%	40	20	20	SW-SC	D60 = 4.4 D30 = 0.4 G = 37%
							D10 = 0.019 Cu = 231.58 S = 51%
							Cc = 1.91 F = 12%

TECHNICIAN (Signature) U. Canelo Pfc

PLOTTED BY (Signature) Joe Willie, Sp4

CHECKED BY (Signature) Albert Dudley, Sgt.

DD Form 1207, 1 AUG 57

Figure 2-62. Data sheet, grain-size distribution graph — aggregate trading chart

Section VI. COMBINED MECHANICAL ANALYSIS

DISCUSSION

Sieve analysis and hydrometer analysis (Sections III and V of this chapter) are performed individually when the requirement for some individual characteristic, such as frost susceptibility or grain-size distribution, is specified. Many natural soils are mixed, that is, they contain both fine and coarse-grained materials. Soil stabilization is accomplished in some cases by mixing finer graded soils with coarse sands and gravels. A complete grain-size analysis is needed to classify such mixed materials. To obtain a complete grain-size distribution curve of coarse and fine particles when there is a particular interest for both, a combined mechanical analysis is conducted. The analysis is performed in one of two ways, depending on the method of determining the fraction finer than the No. 200 sieve; by hydrometer analysis or by sedimentation.

COMBINED SIEVE AND HYDROMETER ANALYSIS

Apparatus

The apparatus for each of the two analyses is the same as used for the individual test. The equipment should be assembled and checked prior to starting the analysis.

Sample Preparation

A representative sample is selected and prepared as for sieve analysis. The size of the sample should be large enough to yield sufficient amounts of materials for both sieve and hydrometer analyses.

- A visual inspection of the sample will usually indicate the need for any intermediate steps such as large screen separation of sizes larger than the No. 4 sieve or prewashing to separate the sizes.

- Samples with fines of little or no plasticity are oven dried, weighed, and separated on the No. 200 sieve. The plus and minus No. 200 sieve fractions are preserved for sieve and hydrometer analyses respectively.

- Soils containing plastic fines may be oven dried initially. However, if the fines tend to adhere to the large particles and to form large lumps, the sample must be soaked in a pan with enough water to cover the material. Soaking must continue until all lumps and coatings have disintegrated. This will vary from 2 to 24 hours depending on the amount and plasticity of the fines.

- The water-soil mixture is washed over a No. 200 sieve with a No. 10 sieve placed over the No. 200 sieve to retain excess coarse material, if necessary. The coarse fractions are preserved for sieve analysis, and the fines and water passing the No. 200 sieve are set aside for hydrometer analysis.

- Excess water with the fines is removed by evaporation, filtration, or wicking.

- If there is a possibility of altering grain size of the fines by oven drying, the dry weight measurement should be made after the hydrometer test.

Procedure

The combined analysis is performed as follows:

- Record all identifying information for the sample on both sieve analysis (Figure 2-49) and the hydrometer analysis (Figure 2-57) data sheets.

- Perform a sieve analysis on the portion of the sample which was retained on the No. 200 sieve.

- Perform a hydrometer analysis on the portion passing the No. 200 sieve.

- Record all the test information on the appropriate data sheets.

Calculations

The computations consist of the following steps.

- Compute the percentage of the total sample retained on the No. 200 sieve.

$$\text{Percent retained on No. 200 sieve} = \frac{W_1}{W_s} \times 100$$

where:

W_1 = dry weight of sample retained on the No. 200 sieve

W_s = total dry weight of sample used for combined analysis.

Compute the sieve analysis data as previously described except that the percent retained on each sieve is based only on that portion of the total material used for the sieve analysis. The portion may be less than the amount retained on the No. 200 sieve, and a partial percent retained must be computed as follows:

$$\text{partial percent retained} = \frac{\text{weight retained on a sieve}}{\text{total weight used for sieve } (W_2)} \times 100$$

The total percent retained is computed as follows:

$$\text{total percent retained} = \text{partial percent retained} \times \frac{W_1}{W_s}$$

$$\text{total percent finer} = 100 - \text{total percent retained.}$$

- Compute the hydrometer analysis data as previously described except that the results are shown as a partial percent finer. As in the sieve analysis above, the amount of material used for hydrometer analysis may be less than the amount which passed the No. 200 sieve or less than $(W_s) - (W_1)$. A partial percent finer is computed as follows:

with hydrometer calibrated in specific gravity

$$\text{partial percent finer} = \frac{(G_s)}{(G_{s-1})}\frac{100}{W_o}(R - C_d + m)$$

with hydrometer calibrated in grams per liter

$$\text{partial percent finer} = \frac{100}{W_o}(R - C_d + m)$$

where:

W_o = oven-dry weight in grams of soil used for hydrometer analysis

G_s = specific gravity of solids

R = corrected hydrometer reading

m = temperature correction

C_d = dispersing agent correction.

- The total percent finer is computed as follows:

$$\text{total percent finer} = \text{partial percent finer}\frac{(W_s - W_1)}{W_s}$$

Presentation of Results

The combined mechanical analysis results in the percent fine by weight in terms of the different sized particles in the soil sample. The tabulated results are useful for comparison to a set of specifications. An easily interpretable form is the grain-size distribution chart (Figure 2-49). The values of percent finer per grain size are plotted on a semilogarithmic chart. The plotted points are joined by a smooth curve. This curve can be used to determine the uniformity coefficient (C_u) and the coefficient of curvature (C_c).

The data in the example shown on the data sheet of Figure 2-61 are plotted on DD Form 1207 (Figure 2-62), to give an example of such a curve for a mixed soil. For this soil, the diameter corresponding to 60 percent passing (D_{60}) is 1.2 millimeters. That for 10 percent passing (D_{10}) is 0.11 millimeter. Hence the uniformity coefficient is—

$$C_u \frac{D_{60}}{D_{10}} = \frac{1.2}{0.11} = 10.9$$

The diameter for 30 percent passing (D_{30}) is 0.4 millimeter. Thus, the coefficient of curvature is—

$$C_c \frac{(D_{30})^2}{(D_{60})\,(D_{10})} = \frac{(0.4)^2}{(1.2)\,(0.11)} = 1.21$$

Section VII. MOISTURE CONTENT TESTING

DISCUSSION

Moisture content (w) is the ratio, expressed as a percentage, of the weight of water in a soil mass to the weight of the soil matter (dry weight):

$$w = \frac{\text{weight of water}}{\text{weight of dry soil}} \times 100$$

With many soils, close control of moisture content during field compaction by rolling is necessary to develop a required density and strength in the soil mass. The amount of compaction effort that must be exerted to obtain a specified density depends upon 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. This is a necessary field procedure in the construction of embankments and compaction of highway subgrades. Adjustment of moisture during construction can be done only if the moisture content is known promptly. The time required for oven drying often cannot be afforded.

There are several methods of determining moisture content of soils. The most accurate method uses an oven with temperature control. For expedient determinations, soils are sometimes dried in a frying pan or container heated by an external source, either stove or exhaust manifold. However, heating most soils to excessive temperatures results in chemical changes that lead to errors in moisture content results. Hence, drying soils by an uncontrolled heat source is generally less accurate than drying in a thermostatically controlled oven. Other expedient methods for determining moisture content are the calcium carbide gas pressure method and the nuclear moisture-density gage.

OVEN DRYING METHOD

Apparatus

The apparatus needed for the oven drying method is as follows:

Oven, preferably with fan forced circulation, thermostatically controlled to 110° ± 5°C

Balance, sensitive to 0.1 gram

Desiccator (unnecessary if samples are weighed immediately after cooling)

Specimen containers, with snug-fitting lids, aluminum or stainless steel

Procedure

The determination is accomplished by the oven method in the following manner:

- Record identifying information of the sample on the data sheet, DD Form 1205 (Soil Moisture Content) (Figure 2-63).

- Weigh and record the weights of the containers to be used in the test.

- Place the sample in a container, weigh it, and record the total weight of the container (tare) and wet soil. If the weight of the sample is less than 100 grams, weigh it to the nearest 0.1 gram. Otherwise, the weight should be to the nearest 1.0 gram.

- Place the open container and sample in an oven heated to 110° ± 5°C and dry it until the sample weight becomes constant. The time required will vary with the amount and kind of soil being dried and the characteristics of the oven. Most samples can be dried overnight. However, large samples of fat clay may require longer drying times.

- Remove the container from the oven, cover it, and place it in the desiccator to cool. If the container is too large for the desiccator, allow the sample to cool to nearly room temperature in the open air.

- Weigh the cooled sample and the container and record the weight (tare and dry soil) on the data sheet.

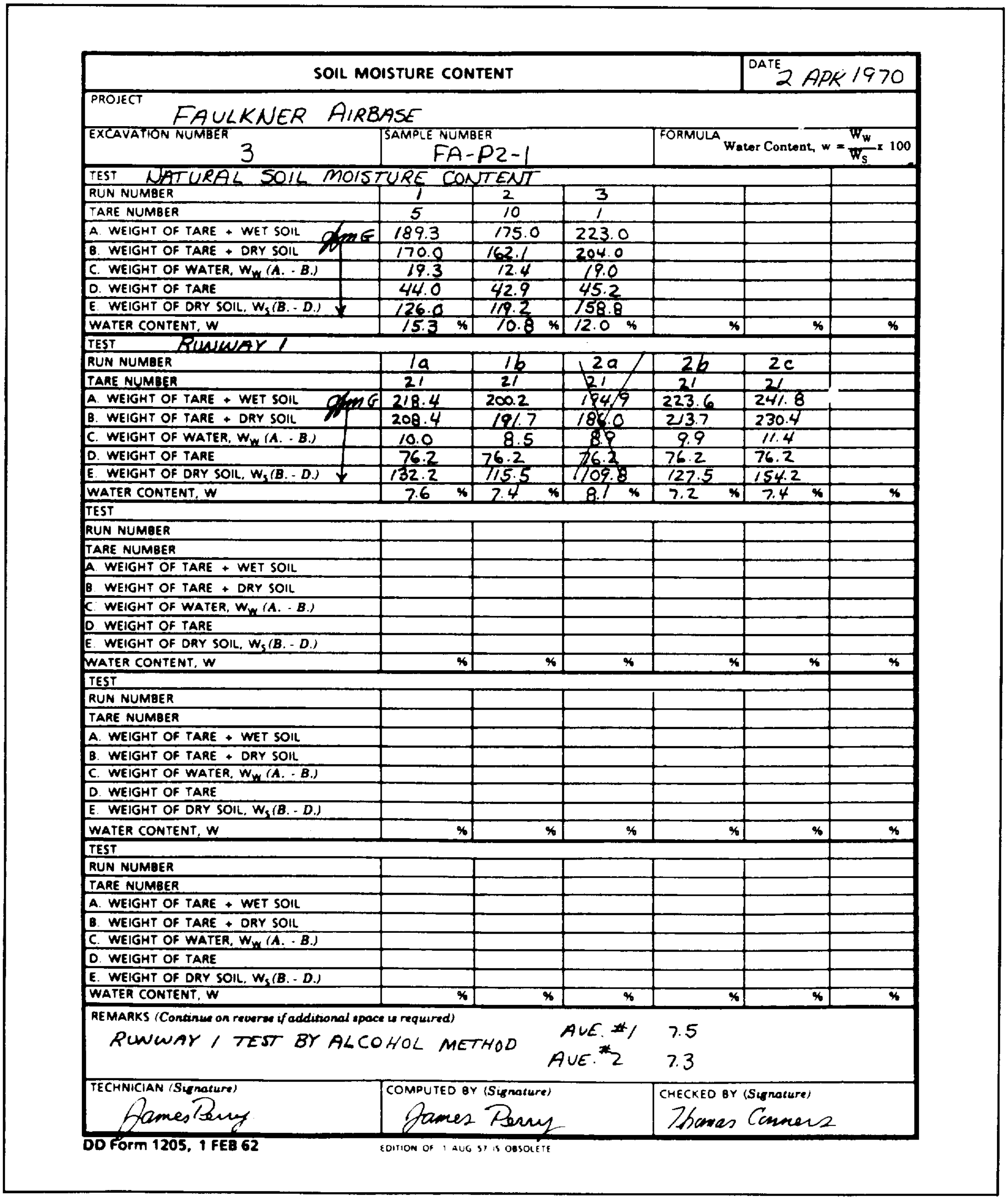
Calculations

The moisture content (in percent) is equal to:

$$\text{percent water content} = \frac{\text{weight of water}}{\text{weight of dry soil}} \times 100$$

$$= \frac{\text{weight of wet soil} - \text{weight of dry soil}}{\text{weight of dry soil}} \times 100$$

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



SOIL MOISTURE CONTENT

DATE 2 APR 1970

PROJECT FAULKNER AIRBASE

EXCAVATION NUMBER 3

SAMPLE NUMBER FA-P2-1

FORMULA Water Content, $w = \frac{W_W}{W_S} \times 100$

TEST NATURAL SOIL MOISTURE CONTENT

RUN NUMBER	1	2	3			
TARE NUMBER	5	10	1			
A. WEIGHT OF TARE + WET SOIL	189.3	175.0	223.0			
B. WEIGHT OF TARE + DRY SOIL	170.0	162.1	204.0			
C. WEIGHT OF WATER, W_W (A. - B.)	19.3	12.4	19.0			
D. WEIGHT OF TARE	44.0	42.9	45.2			
E. WEIGHT OF DRY SOIL, W_S (B. - D.)	126.0	119.2	158.8			
WATER CONTENT, W	15.3 %	10.8 %	12.0 %	%	%	%

TEST RUNWAY 1

RUN NUMBER	1a	1b	~~2a~~	2b	2c	
TARE NUMBER	21	21	~~21~~	21	21	
A. WEIGHT OF TARE + WET SOIL	218.4	200.2	~~194.9~~	223.6	241.8	
B. WEIGHT OF TARE + DRY SOIL	208.4	191.7	~~186.0~~	213.7	230.4	
C. WEIGHT OF WATER, W_W (A. - B.)	10.0	8.5	~~8.9~~	9.9	11.4	
D. WEIGHT OF TARE	76.2	76.2	~~76.2~~	76.2	76.2	
E. WEIGHT OF DRY SOIL, W_S (B. - D.)	132.2	115.5	~~109.8~~	127.5	154.2	
WATER CONTENT, W	7.6 %	7.4 %	8.1 %	7.2 %	7.4 %	%

TEST

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	%	%	%	%	%	%

TEST

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	%	%	%	%	%	%

TEST

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	%	%	%	%	%	%

REMARKS *(Continue on reverse if additional space is required)*

RUNWAY 1 TEST BY ALCOHOL METHOD

AVE. #1 7.5

AVE. #2 7.3

TECHNICIAN *(Signature)* James Perry

COMPUTED BY *(Signature)* James Perry

CHECKED BY *(Signature)* Thomas Conners

DD Form 1205, 1 FEB 62

EDITION OF 1 AUG 57 IS OBSOLETE

Figure 2-63. Data sheet, soil moisture content

The computations are made on the data sheet (Figure 2-63, page 2-81). Item A minus item B equals the weight of water (W_w). Item B minus item D equals the weight of dry soils (W_S). The division is made, converted to a percentage, and recorded as water content (W).

Example: A pan weighing 44.0 grams was used to contain a sample of soil to determine its moisture content. The total weight, including the pan, was 189.3 grams which after drying was reduced to 170.0 grams. The resulting percent of moisture in the soil is:

Weight of water = (weight of wet sample plus tare) – (weight of dry sample plus tare) = 189.3 – 170.0 = 19.3 grams

Weight of dry soil = (weight of dry sample plus tare) – (weight of tare) = 170.0 – 44.0 = 126.0 grams

Water content =

$$100 \times \frac{\text{weight of water}}{\text{weight of dry soil}} = 100 \times \frac{W_w}{W_s}$$

$$w = 100 \times \frac{19.3}{126.0} = 15.3 \text{ percent}$$

CALCIUM CARBIDE GAS PRESSURE METHOD

Apparatus

Calcium carbide pressure (speedy) moisture tester to hold a 26 gram soil specimen

Balance, readable to 0.1 gram

Two 1¼ inch (31.75 millimeter) steel balls

Cleaning brush and cloth

Scoop

Calcium carbide reagent.

Procedure

- When using the 26 gram tester, place approximately 24 grams of calcium carbide and two 1¼ inch (31.75 millimeter) steel balls in the larger chamber of the moisture tester.

- Weigh a 26 gram specimen of the soil to be tested. Place the soil sample in the cap of the tester and, with the pressure vessel in an approximately horizontal position, insert the cap in 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.

- Raise the moisture tester to a vertical position so that the soil in the cap will fall into the pressure vessel.

- Shake the instrument vigorously for 10 seconds and rest for 20 seconds. Then repeat shaking so that all lumps will be broken up to permit the calcium carbide to react with all available free moisture. When steel balls are being used in the tester, shake the instrument with 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 for at least 1 minute with granular soils and for 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.

- When the needle stops moving, read the dial while holding the instrument horizontally at eye level. Record the dial reading as the percent of moisture by wet mass.

- Record the specimen mass and the dial reading.

- With the cap of the instrument pointed away from the operator, slowly release the gas pressure. Empty the pressure vessel and examine the material for lumps. If the sample is not completely pulverized, repeat the test using a new sample.

Calculation

- Determine the percentage of moisture by dry mass of the soil from the calibration curve (Figure 2-64, page 2-84) and the conversion chart (Figure 2-65, page 2-85).

The calibration curves, moisture content determination by the calcium carbide gas pressure method (Figure 2-64), are used for materials that need the pulverizing steel balls. Using the highest reading obtained during the test (direct reading), read over to the curve and then down to oven dry moisture percentage. The curve to be used will depend on the test time.

The conversion chart (Figure 2-65) is used to determine oven-dry moisture contents of materials that do not need the steel balls. If direct readings are not on the conversion chart, interpolate the difference between the two known direct readings.

Example: A speedy test is performed on a sand. The highest reading obtained was 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.

The following values are known:

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

To find the value of X, place the differences in the following steps into a ratio.

Step 1	3.0%	Step 3	2.2%
Step 2	3.5%		x
Differences	4.0%	Step 4	4.3%

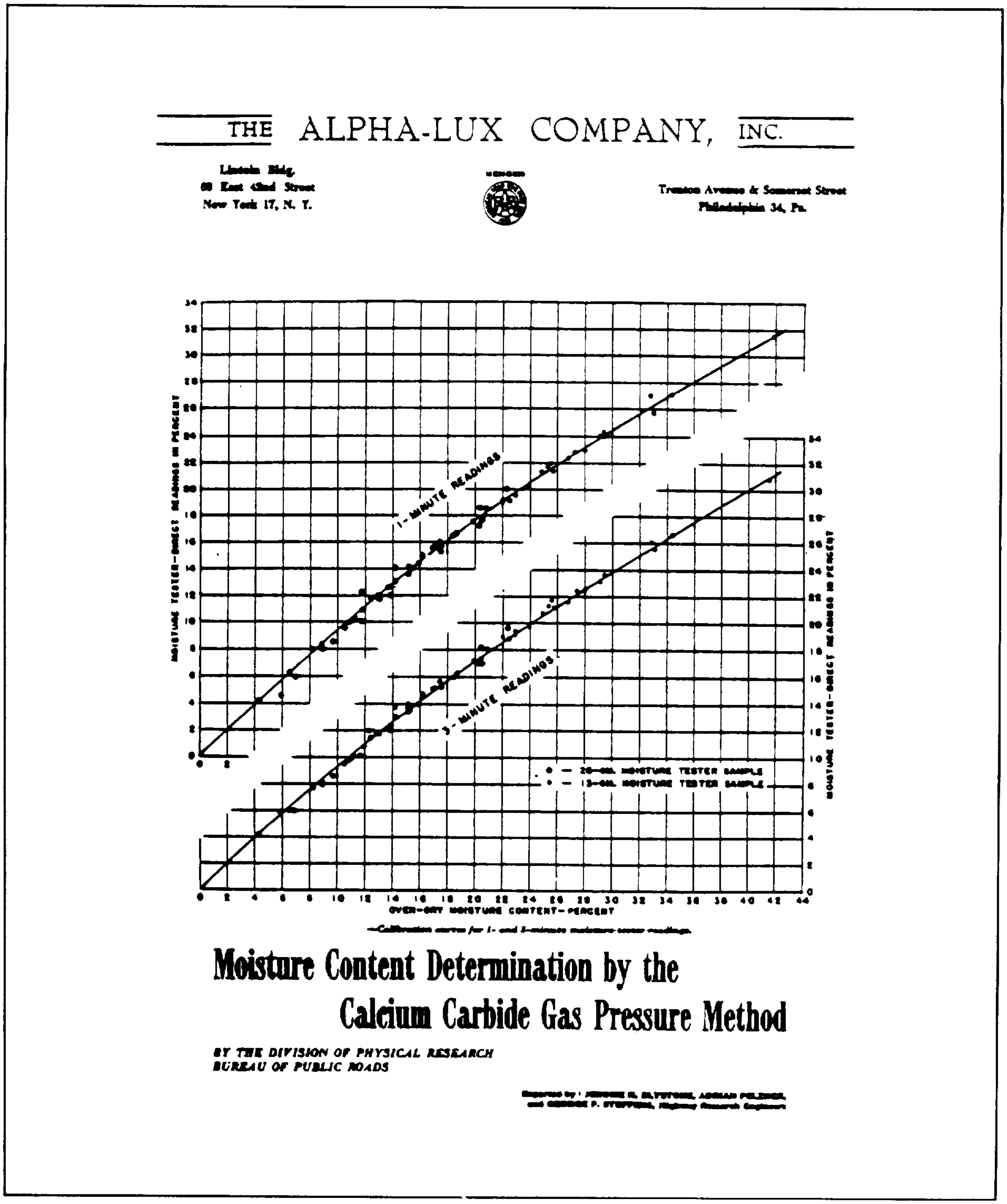
$$\frac{\text{step 1} - 0.5}{\text{step 2} - 1.0} : \frac{\text{step 3} - X}{\text{step 4} - 1.1}$$

$$\frac{0.5}{1.0} : \frac{X}{1{:}1} \quad \text{Cross multiplied } X = 0.55$$

3.5% wet weight = 3.2% + 0.53% = 3.75% comparable oven dry

- The curves and charts are normally supplied with the moisture tester. However, each moisture tester should be checked for the accuracy of its gage and the accuracy of the conversion curve. Check the accuracy of the tester gage by using a calibration kit (obtained from the tester manufacturer) equipped with a standard gage. In case of discrepancy, adjust the gage on the tester to conform with the standard gage. 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 (Figure 2-65) beyond 44 percent moisture content.

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



THE ALPHA-LUX COMPANY, INC.

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60 East 42nd Street
New York 17, N. Y.

Trenton Avenue & Somerset Street
Philadelphia 34, Pa.

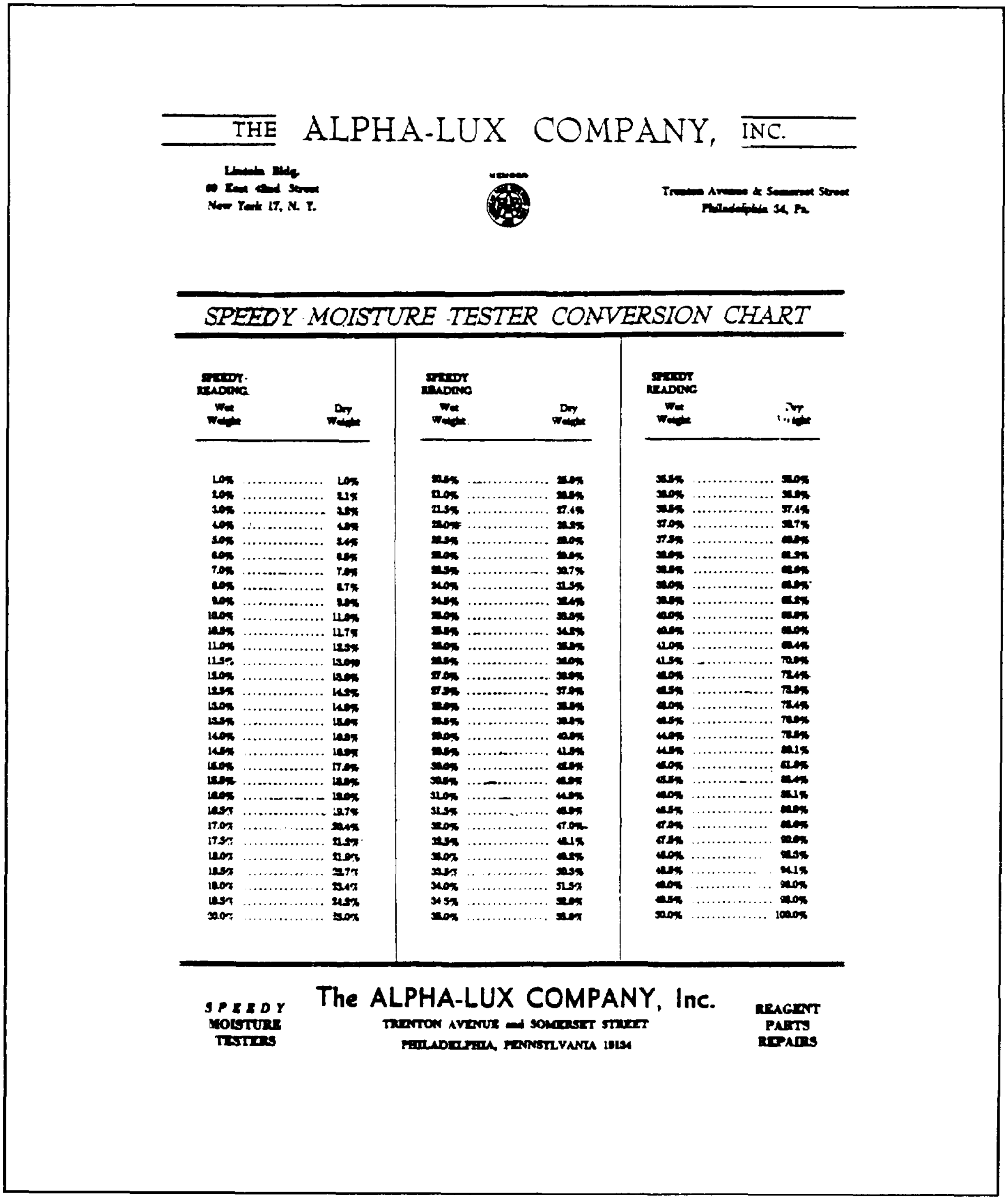
MOISTURE TESTER—DIRECT READINGS IN PERCENT
0 2 4 6 8 10 12 14 16 18 20 22 24 26 28 30 32 34
1-MINUTE READINGS
3-MINUTE READINGS
● — 20-GM. MOISTURE TESTER SAMPLE
● — 13-GM. MOISTURE TESTER SAMPLE
0 2 4 6 8 10 12 14 16 18 20 22 24 26 28 30 32 34 36 38 40 42 44
OVEN-DRY MOISTURE CONTENT—PERCENT

—Calibration curves for 1- and 3-minute moisture-tester readings.

Moisture Content Determination by the Calcium Carbide Gas Pressure Method

BY THE DIVISION OF PHYSICAL RESEARCH
BUREAU OF PUBLIC ROADS

Figure 2-64. Calibration curve



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Trenton Avenue & Somerset Street
Philadelphia 34, Pa.

SPEEDY MOISTURE TESTER CONVERSION CHART

SPEEDY READING Wet Weight	Dry Weight	SPEEDY READING Wet Weight	Dry Weight	SPEEDY READING Wet Weight	Dry Weight
1.0%	1.0%	20.5%	25.8%	35.5%	55.0%
2.0%	2.1%	21.0%	26.6%	36.0%	56.2%
3.0%	3.1%	21.5%	27.4%	36.5%	57.4%
4.0%	4.2%	22.0%	28.2%	37.0%	58.7%
5.0%	5.2%	22.5%	29.0%	37.5%	60.0%
6.0%	6.3%	23.0%	29.9%	38.0%	61.2%
7.0%	7.5%	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%	11.1%	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%	35.1%	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%	73.9%
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%	90.4%
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%

SPEEDY
MOISTURE
TESTERS

The ALPHA-LUX COMPANY, Inc.
TRENTON AVENUE and SOMERSET STREET
PHILADELPHIA, PENNSYLVANIA 19134

REAGENT
PARTS
REPAIRS

Figure 2-65. Speedy moisture tester conversion chart

Section VIII. LIQUID AND PLASTIC LIMITS

DISCUSSION

Clays and some other fine-grained soils exhibit the physical property of 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 liquid limit (LL) is the greatest water content which the material may contain and still remain plastic. More water causes it to become a thick liquid. The plastic limit (PL) is the lowest water content which the material may contain for plastic behavior. With less water, the soil becomes brittle and breaks into fragments if remolding is attempted. The plasticity index (PI) is the numerical difference between the liquid and plastic limits:

$$(PI) = (LL) - (PL)$$

A large PI indicates a very plastic soil; a small one denotes a soil having little plasticity. As water content decreases below the plastic limit, the soil mass shrinks and becomes stiffer. The shrinkage limit is the water content where, with further drying, shrinkage ceases. Since there is no sharp distinction between the liquid, plastic, and brittle solid states of consistency, standardized procedures have been established for determining the liquid and the plastic limits.

Research with large numbers of clay soils was used to establish the soil plasticity chart for laboratory classification of fine-grained soils (Appendix A, Table A-1) that is part of the USCS. Values of LL and PI 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 upon the behavioral characteristics of the particular soil.

Additional information on the liquid limit and plastic limit tests are available in MIL-STD-621A.

LIQUID LIMIT TEST PREPARATION

The liquid limit 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 as the water content at which two halves of a soil cake placed in a standard cup and divided into two sections by a standard grooving tool will flow together for a distance of 0.5 inch along the bottom of the groove when the cup is dropped 25 times from a distance of one centimeter (0.394 inch) at the rate of two drops per second.

Preparation of Samples

Soil samples should be at natural water content when prepared for the test. Although the effects of drying may be negligible for many soils, it is significant for some. Any drying which occurs before testing may change the limit values. Certain soil colloids undergo an irreversible change on air or oven drying. Liquid and plastic limit tests performed on these soils after drying will yield improper results. Factors which must be considered before testing are:

- The liquid limit is performed on material finer than the No. 40 sieve.
- Samples should be large enough to produce 150 to 200 grams of material for testing.
- The selected sample must not be subjected either to air drying or oven drying before testing. Occasionally additional moisture may be required to produce the liquid limit. Allow the sample to set 4 to 6 hours.

- If the sample contains no material coarser than the No. 40 sieve, it should be thoroughly mixed. It will then be ready for testing.

- If the sample contains material coarser than the No. 40 sieve, it is soaked in clean water for 24 hours and washed over a No. 40 sieve using a minimum of wash water. The fines and wash water are caught in a large dish or collecting pan and saved.

- Material retained on the No. 40 sieve is oven dried at 110° ± 5°C, and then dry sieved through the No. 40 sieve.

- The portion that passed through the No. 40 sieve during the dry sieving is combined with the wash-through portion for testing.

- The combined (passing No. 40) sample is dried to a thick, syrupy consistency by decantation or evaporation. Care is taken to prevent the sample from caking or lumping during the process.

- Neither chemical substances nor dry soil are added to hasten settlement or to speed drying.

- The soil sample should be thoroughly mixed just before starting the test.

Preparation of Liquid Limit Devices

The liquid limit testing machine (Figure 2-66, page 2-88) must be inspected, checked, and adjusted prior to the test to determine that it is in good working condition.

- Check the pin connecting the cup for wear. It must not be worn to the point where it permits side play of the cup.

- Check the screws connecting the cup to the hanger arm. They should be tight.

- Check the cup for wear. Long use will develop a groove through the cup. This can be felt by running the fingers over the inside surface. If the cup is worn to this extent it should be replaced.

- Check the grooving tool for wear. The dimensions which control the size of the groove are shown in Figure 2-66. The tool should be discarded when the point width exceeds 0.086 inch.

- Each time the test is run, or at proper intervals, verify the height of the drop of the cup. The gage for this check is on the handle of the grooving tool (Figure 2-66). Using the gage, adjust the height by means of adjusting screw (A) until the point on the cup that strikes the base (B) is exactly one centimeter (0.394 inch) above the base. Tighten the screws (C) to secure the adjustment and check it by turning the crank at approximately two revolutions per second. If the adjustment is correct, a slight click will be heard. If the cup is lifted off the gage or no sound is heard, further adjustment is necessary.

LIQUID LIMIT TEST

Apparatus

The apparatus for a liquid limit test consists of the following as shown in Figure 2-67, page 2-89).

Balance, readable to 0.01 gram. Specimen containers, stainless steel or aluminum, approximately 2 inch diameter by ⅛ inch high, with snug fitting lids.

Dishes, evaporating

Filler, battery

Liquid limit machine, complete with brass cup

Grooving tool

Mortar, porcelain

Oven, thermostatically controlled to 110° ± 5°C

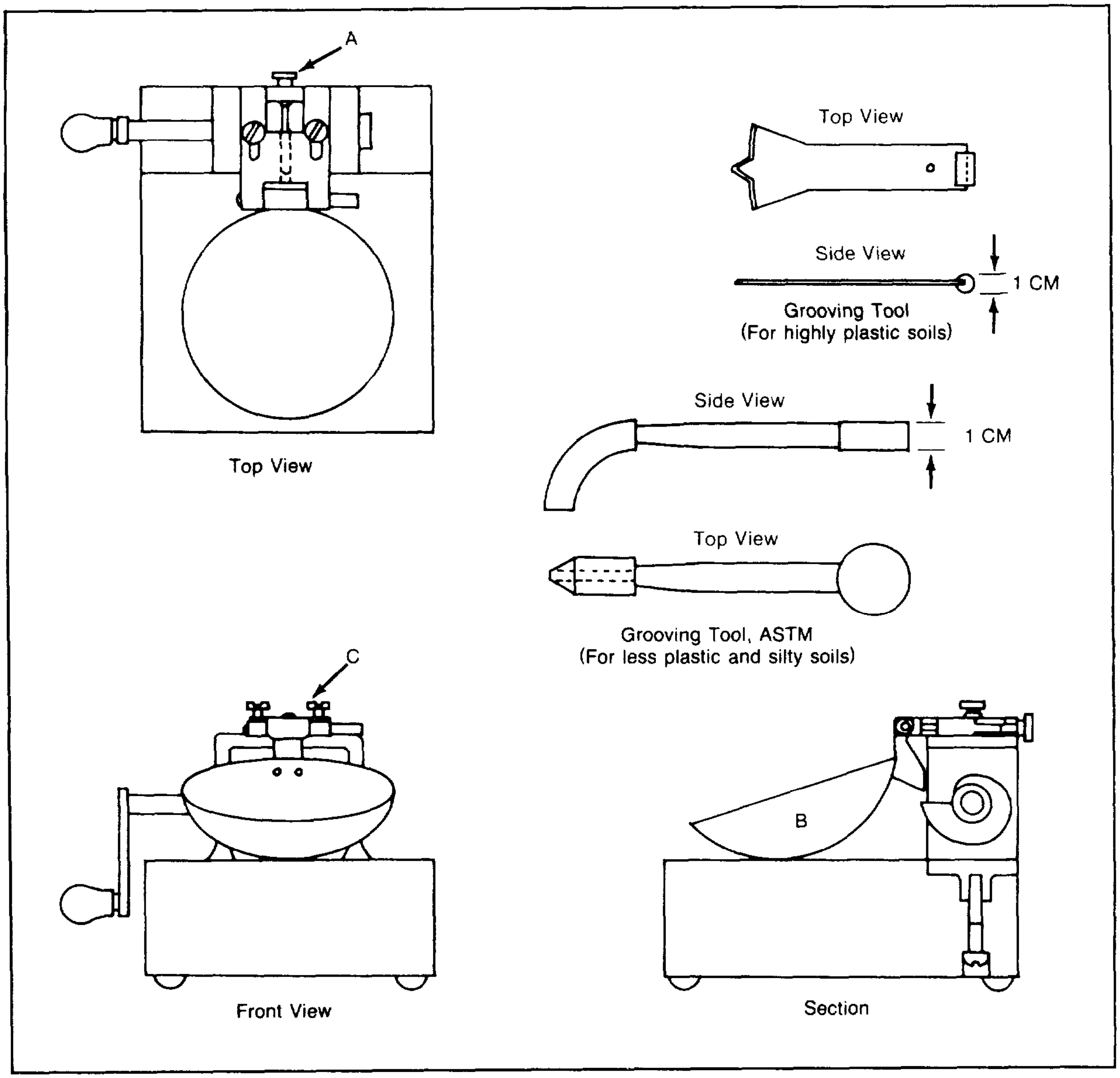

A
Top View
Top View
Side View
1 CM
Grooving Tool
(For highly plastic soils)
Side View
1 CM
Top View
Grooving Tool, ASTM
(For less plastic and silty soils)
C
B
Front View
Section

Figure 2-66. Apparatus for determining Attenberg limits

Pan, sieve, 8-inch frame

Pestle, rubber-tipped

Screwdriver

Sieve, testing, 8-inch frame full height, US No. 40

Spatula

Tongs, crucible

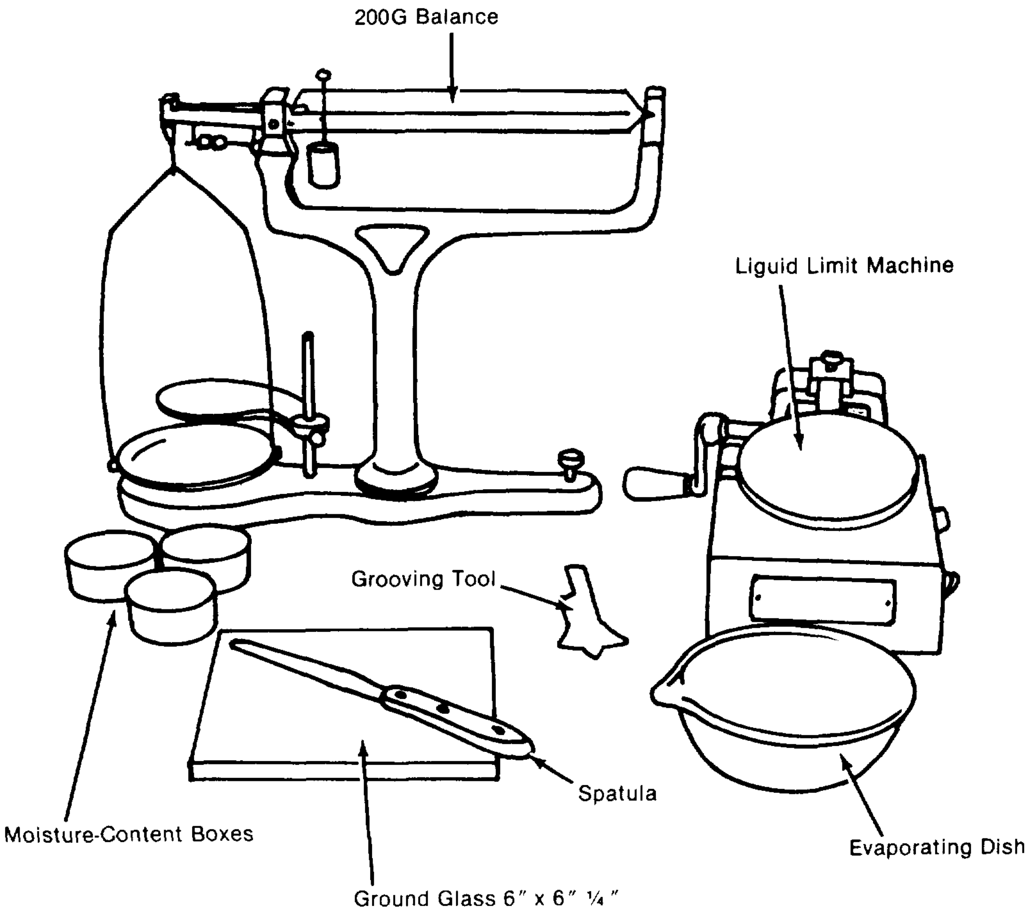
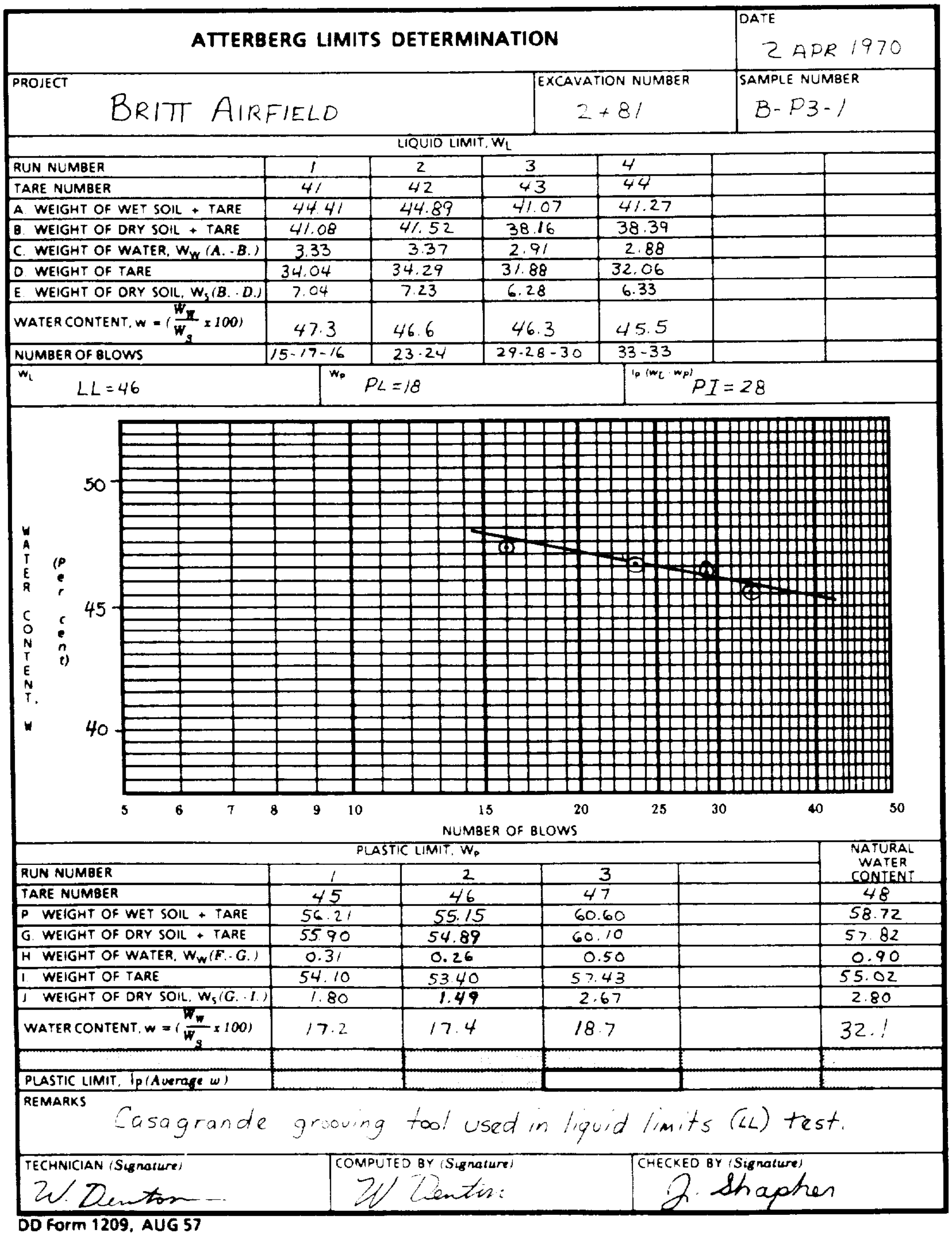

200G Balance
Liguid Limit Machine
Grooving Tool
Spatula
Moisture-Content Boxes
Evaporating Dish
Ground Glass 6" x 6" ¼"

Figure 2-67. Mechanical liquid limit device



ATTERBERG LIMITS DETERMINATION		DATE 2 APR 1970
PROJECT BRITT AIRFIELD	EXCAVATION NUMBER 2+81	SAMPLE NUMBER B-P3-1

LIQUID LIMIT, W_L

RUN NUMBER	1	2	3	4		
TARE NUMBER	41	42	43	44		
A. WEIGHT OF WET SOIL + TARE	44.41	44.89	41.07	41.27		
B. WEIGHT OF DRY SOIL + TARE	41.08	41.52	38.16	38.39		
C. WEIGHT OF WATER, W_W (A. - B.)	3.33	3.37	2.91	2.88		
D. WEIGHT OF TARE	34.04	34.29	31.88	32.06		
E. WEIGHT OF DRY SOIL, W_S (B. - D.)	7.04	7.23	6.28	6.33		
WATER CONTENT, $w = (\frac{W_W}{W_S} \times 100)$	47.3	46.6	46.3	45.5		
NUMBER OF BLOWS	15-17-16	23-24	29-28-30	33-33		

W_L LL=46	W_P PL=18	I_P (W_L - W_P) PI=28

PLASTIC LIMIT, W_P

					NATURAL WATER CONTENT
RUN NUMBER	1	2	3		
TARE NUMBER	45	46	47		48
P. WEIGHT OF WET SOIL + TARE	56.21	55.15	60.60		58.72
G. WEIGHT OF DRY SOIL + TARE	55.90	54.89	60.10		57.82
H. WEIGHT OF WATER, W_W (F. - G.)	0.31	0.26	0.50		0.90
I. WEIGHT OF TARE	54.10	53.40	57.43		55.02
J. WEIGHT OF DRY SOIL, W_S (G. - I.)	1.80	1.49	2.67		2.80
WATER CONTENT, $w = (\frac{W_W}{W_S} \times 100)$	17.2	17.4	18.7		32.1
PLASTIC LIMIT, I_P (Average w)					

REMARKS

Casagrande grooving tool used in liquid limits (LL) test.

TECHNICIAN (Signature)	COMPUTED BY (Signature)	CHECKED BY (Signature)
W. Denton	W. Denton	J. Shapher

DD Form 1209, AUG 57

Figure 2-68. Data sheet, Atterberg limits determination

Procedure

- Record all identifying information for the sample on the top section of the data sheet, DD Form 1209 (Atterberg Limits Determination) (Figure 2-68).
- Place 50 to 80 grams of the prepared sample in the cup and level it off to a depth of approximately 1 centimeter at the point of greatest thickness (Figure 2-69). When leveling the soil, squeeze it downward and spread it using as few strokes of the spatula as possible. Be careful to prevent entrapping air bubbles in the mass. Return the unused soil to the sample stock.

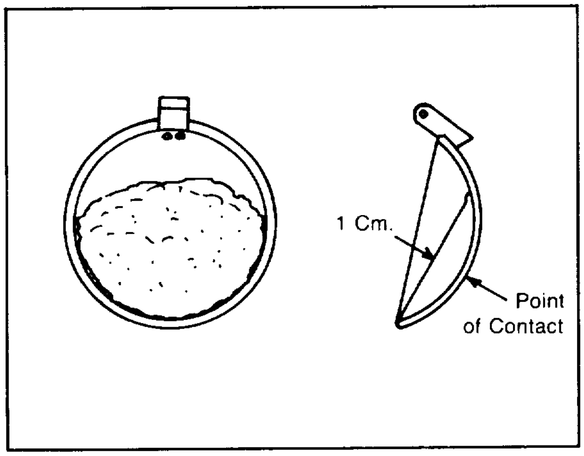


Figure 2-69. Leveling sample in the cup

- Hold the grooving tool and the cup as shown in Figure 2-70 and make the groove by drawing the tool down through the sample along the diameter through the centerline of the cam follower so that a clean, sharp groove of the proper dimensions will be formed.

More than one stroke may be necessary to make the groove. Each stroke should be a little deeper than the preceding one until the last stroke scrapes the bottom of the cup clean. However, do not exceed six strokes and be careful not to tear the sides of the groove or to allow the soil cakes to slip in the cup.

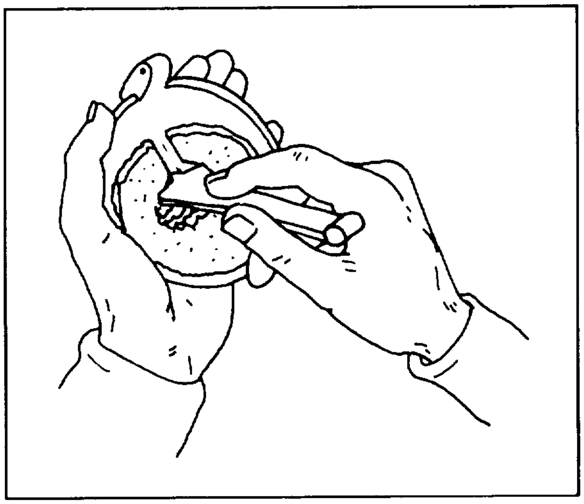


Figure 2-70. Holding cup and grooving tool

Note that with some sandy and highly organic soils, it is impossible to draw the tool through the sample without tearing the sides of the groove. In such cases, the groove is made with the spatula (Figure 2-71) and shaped with the grooving tool.

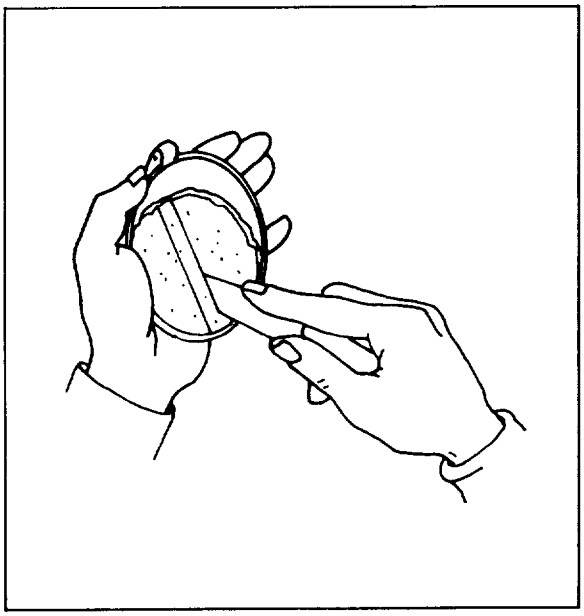


Figure 2-71. Cutting groove with spatula in sandy soil

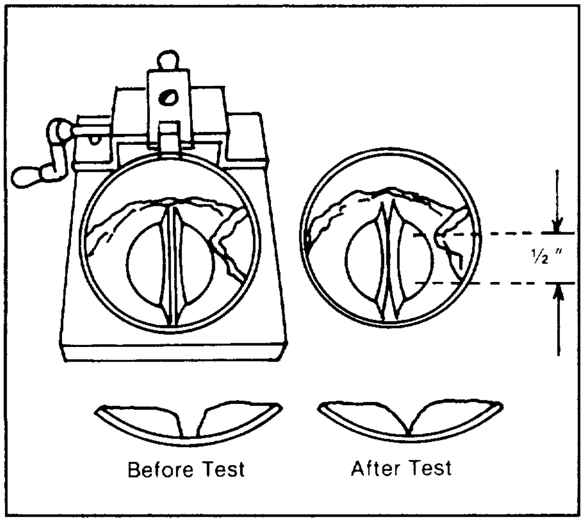


Figure 2-72. Liquid limit test

Before laying the grooving tool aside, clean the cutting edge by running the thumb over the tool or by washing it. This prevents the soil from drying on the tool and lost time trying to scrape or wash it clean.

- Attach the cup to the carriage of the device and turn the crank at the rate of two revolutions per second. Count the number of blows until the two halves of the soil cake come into contact at the bottom of the groove for a distance of ½ inch (Figure 2-72). Record the number of blows.

- Remove about 10 grams from the cup by drawing the spatula from edge to edge perpendicular to the groove and through the portion of the cake that closed (Figure 2-73). This sample is used to determine the water content. All weighing should be accurate to ± 0.01 gram and the water content computed to one decimal place.

Transfer the remaining soil in the cup to the mixing dish. Wash and dry the cup and grooving tool. Repeat steps two through five for three additional samples, each of which has had the water content reduced by drying. Drying is accomplished by continued mixing with the spatula and aided by a small electric fan, if desired. The water content adjustment must be sufficient to produce a noticeable change in the number of blows to close the groove, and results in a series with some more and some less than 25 blows which close the groove. Preferably, the range should be from 15 to 35 with two tests between 15 and 25 blows and two tests between 25 and 35 blows.

- Material remaining in the mixing dish should be reserved for the plastic limit test.

Calculations

Computing the liquid limit is done graphically by plotting a flow curve with the number of blows and the water content as coordinates (Figure 2-68, page 2-90). The water content (on the left edge) is plotted on an arithmetic scale and the number of blows (along the bottom) is plotted on a logarithmic scale. If the points are relatively close to lying on a straight line, a straight line is drawn as nearly as possible through the plotted points, thus establishing an average between the points. However, if the plotted points do not

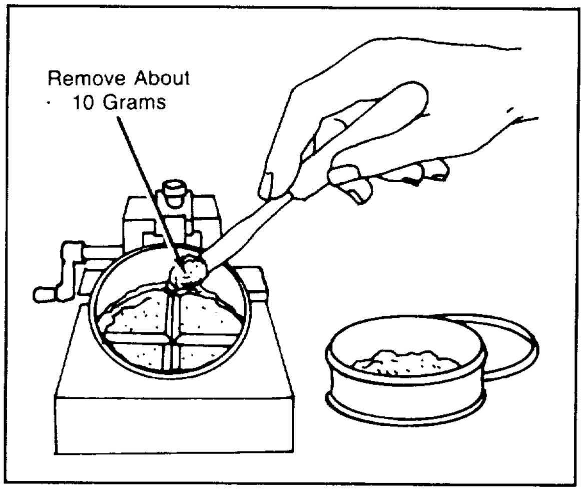


Figure 2-73. Removing moisture content portion

define a straight line, additional tests should be made and new points plotted. The liquid limit is the water content corresponding to the point where the straight line crosses the 25 blow coordinate. The liquid limit is read to the nearest 0.1 percent, but is reported to the nearest whole number (Figure 2-68). In a given test, the determinations resulted in the following values:

	Number of Blows	Water Content
(1)	16	47.3
(2)	24	46.6
(3)	29	46.3
(4)	33	45.5

After plotting these four points and drawing a straight line as nearly as possible through them, the water content corresponding to the 25 blow coordinate is 46.3. Thus, the liquid limit for this sample would be reported as 46.

ONE POINT LIQUID LIMIT TEST

The one point test is based on the premise that the slope of the flow curve for soils within a given geologic environment is essentially constant. Thus, the liquid limit flow curve could be drawn using only one test point provided that a test slope has been established by experience with other soils in the area. The simplified test should only be used in geologically similar areas and where adequate correlations have been made to define the slope of the flow curve.

Apparatus

See liquid limit test. The sample is prepared as for a standard test except that its consistency is controlled to result in 20 to 30 blows to cause closure.

Procedure

See liquid limit test. The amount removed from the cup for the water content determination is replaced from the prepared sample. Remix the entire amount in the cup without adding water. Regroove the specimen and operate the device once again. The number of blows to close the groove should be within two blows of the first measurement. If the difference is greater than two blows the mixing was insufficient and the entire test must be repeated.

Calculations

Determine the liquid limit by using the following equation:

$$LL = W_n \frac{(N)}{(25)} \tan B$$

where:

W_n = water content at N blows

N = number of blows to close the groove

tan B = slope of the flow line

Values of $(N/25)^{0.12}$ are given in Table 2-7. The exponent 0.12 is an average value to be used in this simplified method if the slope of the flow curve is not known for the particular soil. If the correct slope is known, use the exponent corresponding to this slope in the calculation.

Table 2-7. Values of $(N/25)^{0.12}$ for liquid limit test

N	$(N/25)^{0.12}$
20	0.974
21	0.979
22	0.985
23	0.990
24	0.995
25	1.000
26	1.005
27	1.009
28	1.014
29	1.018
30	1.022

PLASTIC LIMIT TEST

The plastic limit of a soil is the water content, expressed as a percentage of weight of oven dried soil, at which the soil begins to crumble when rolled into a thread ⅛ inch in diameter. Approximately 20 grams of material is required for the plastic limit test. The sample is prepared as described for a liquid limit test, or is the material remaining after the liquid limit is tested.

Apparatus

The apparatus for the plastic limit test consists of the following:

Surface for rolling, such as a ground glass plate, a piece of linoleum, or a tabletop of close grained wood. Paper may be used as a rolling surface provided that it does not give off lint which can be picked up by the sample during rolling. The use of the lint free paper towel as a rolling surface will enable faster testing.

Spatula

Containers (boxes, stainless steel, such as those used in the liquid limit test)

Balance, sensitive to 0.01 gram
Oven, thermostatically controlled to 110° ± 5°C

Evaporating dish.

Procedure

- Record all identifying information for the sample on the data sheet, bottom half (Figure 2-68).

- Take approximately 5 grams of the prepared material. The sample should be taken at any stage of drying when the mass is just plastic enough to be shaped into a ball that will not stick to the fingers when squeezed.

- Shape the test sample into an ellipsoid and roll it between the fingers and rolling surface (Figure 2-74) using enough pressure to form the soil mass into a thread ⅛ inch in diameter. The rate of rolling should be between 80 and 90 strokes per minute. One stroke is considered a complete forward and backward motion returning to the starting position.

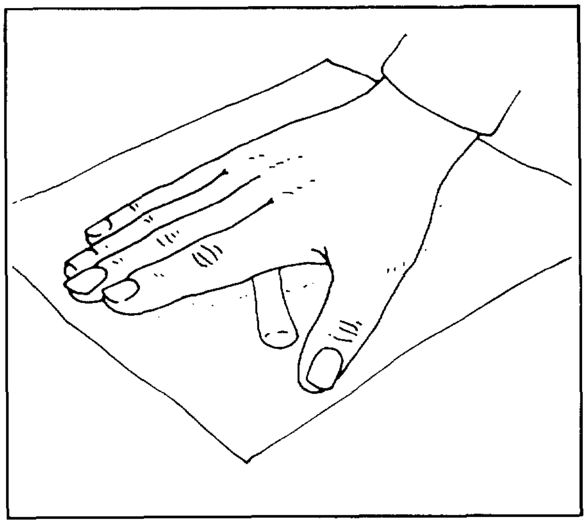


Figure 2-74. Rolling the soil, plastic limit test

- When the thread diameter reaches ⅛ inch without crumbling, remold the sample into a ball and repeat the rolling process. Continue remolding and rolling until the ball has dried sufficiently for the rolled thread to crumble (Figure 2-75). The plastic limit has been reached when the thread crumbles (regardless of diameter) if the original ⅛-inch diameter has been reached at least once. Do not precipitate crumbling at exactly the ⅛-inch diameter by reducing the rate of rolling, the hand pressure, or both; and do not roll beyond the ⅛-inch diameter. It is permissible, however, to reduce the total amount of deformation for poorly plastic soils by making the initial diameter of the ellipsoidal mass nearer the required ⅛-inch final diameter. Continue to roll the sample until all the soil has crumbled. It does not crumble all at the same time.

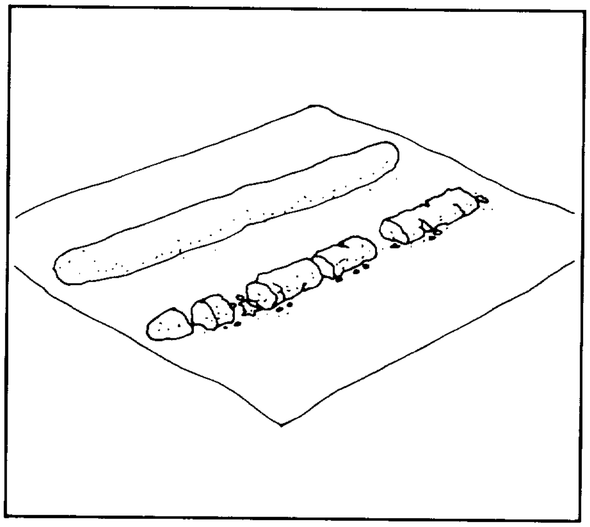


Figure 2-75. Rolled threads, uncrumbled and crumbled

- Collect the crumbled portions and determine the water content.

- Repeat the process with an additional portion of the prepared sample to obtain an average value for the plastic limit. The two tests should agree within ± 1 percent moisture content, or the test must be repeated.

- Record the sample weight and computations on the data sheet. Weighing should be accurate to 0.01 gram and water content computed to one decimal place. Report the limit to the nearest whole number.

Calculations

Compute the plastic limit as follows:

$$\text{water content for each run} = \frac{\text{weight of water}}{\text{weight of oven dried soil}} \times 100$$

plastic limit = average of water content from two or more runs (Figure 2-68)

plasticity index = liquid limit − plastic limit; or PI = LL − PL

Example (Figure 2-68): In this example, the following data was obtained. Determine the plasticity index.

liquid limit = LL = 46

plastic limit = PL = 18

plasticity index = PI = LL − PL; PI = 46 − 18 = 28

Results: The PI and LL values are then plotted on the plasticity chart (Figure 2-76, page 2-96) to determine the types of fines within the sample.

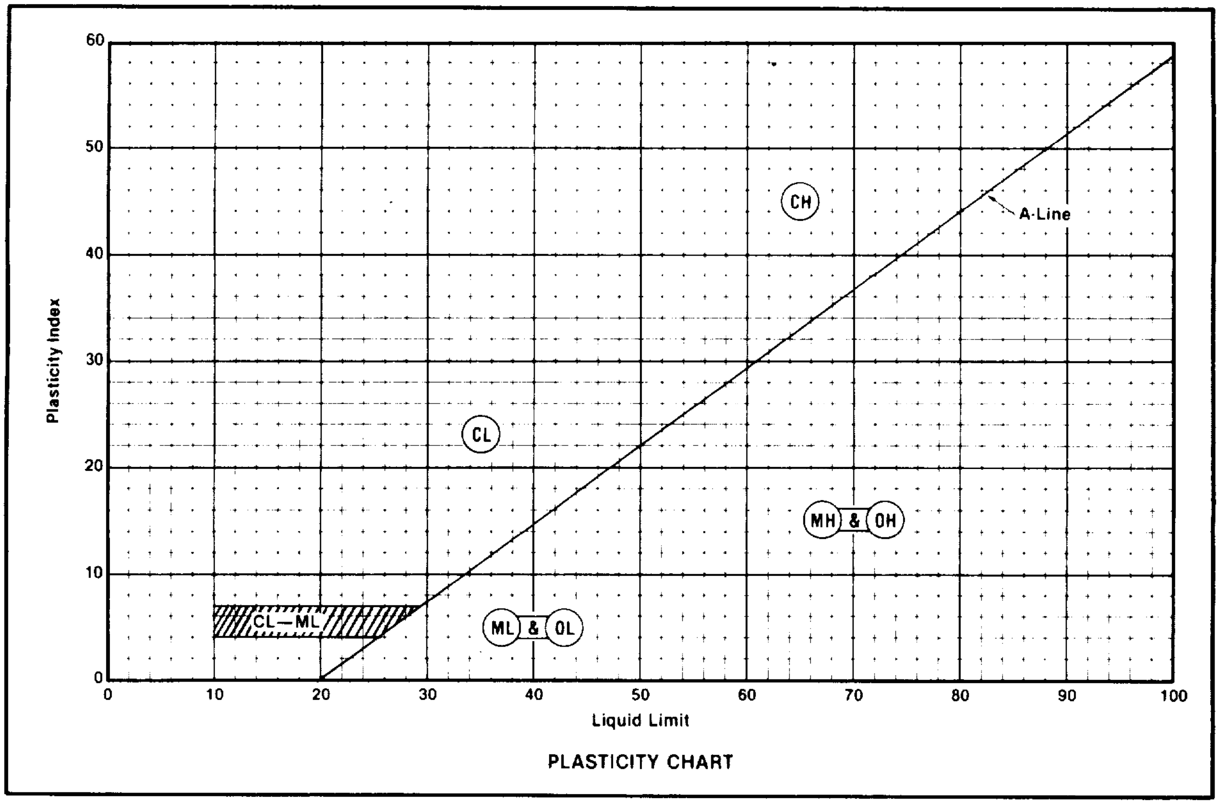


Figure 2-76. Plasticity chart

Section IX. COMPACTION TEST

DISCUSSION

Compaction is one of the basic construction procedures involved in building subgrades and bases for roads and airport pavements, embankments, earthfill dams, and similar structures. Compaction is the process of increasing the amounts of solids per unit volume of soil by mechanical means. This increase in density has an important effect in improving such soil properties as strength, permeability, and compressibility. In the field, compaction is accomplished by rolling or tamping with special equipment, or by the passing of construction equipment. Laboratory compaction may be accomplished by the impact of hammer blows, vibration, static loading, or any other method which does not alter the water content of the soil. In general, efficient and adequate compaction in the field is attained by compacting the soil at a moisture content and to a density established in the laboratory.

As related to compaction, the term dry density means dry unit weight, that is, pounds per cubic foot of soil computed on an oven-dry basis.

Laboratory compaction generally is accomplished by placing the soil in a cylinder of known volume and dropping a tamper of known weight onto the soil from a known height for a given number of blows. The amount of work done to the soil per unit volume of soil in this dynamic compaction procedure is called compactive effort. For most soils and for a given compactive effort, the density of the soil will increase to a certain point as the moisture content is increased and then start to decrease with any further increase in moisture content. The moisture content at which maximum density occurs is called the optimum moisture content (OMC) for that compactive effort. Each compactive effort for a given soil has its own OMC. As the compactive effort is increased, the maximum density generally increases and the OMC decreases.

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

PREPARATION

The ASTM compaction procedure may be performed either in the standard Proctor mold or the larger laboratory CBR mold. The mold to be used is determined by the maximum particle size of the soil sample. Soils which do not contain gravel are compacted in the Proctor mold, whereas soils continuing up to ¾-inch material should be compacted in the CBR mold.

Apparatus

The apparatus used in the compaction test consists of the following:

Compaction cylinder (Proctor mold). A cylindrical metal mold 4 inches in diameter and 4.584 inches high having a volume of 1/30 cubic foot. The mold is fitted with a detachable base plate and a removable extension collar 2½ inches high.

Compaction cylinder (CBR mold). This is a cylindrical metal mold 6 inches in diameter and 7 inches high. The compacted sample is controlled to 4.5 inches by means of a spacer disk. The spacer disk is 1/16 inch smaller in diameter than the mold and 2½ inches high. The CBR mold is used for samples containing material retained on the No. 4 sieve and for any type of sample needed for the CBR test. The volume of the mold is 0.0736 cubic foot using the 2½-inch spacer mold. Details of the mold used for CBR testing can be found in MIL-STD-621A.

Compaction tamper. The tamper has a drop weight which weighs 10.0 pounds and has a striking face 2.00 inches in diameter. The tamper is held in a guide sleeve or on a shaft that regulates the height of drop to 18.0 inches. The tamper is illustrated in Figure 2-77.

Balances, 20 kilogram capacity (readable to 1 gram) and 1 kilogram capacity (readable to 0.1 gram).

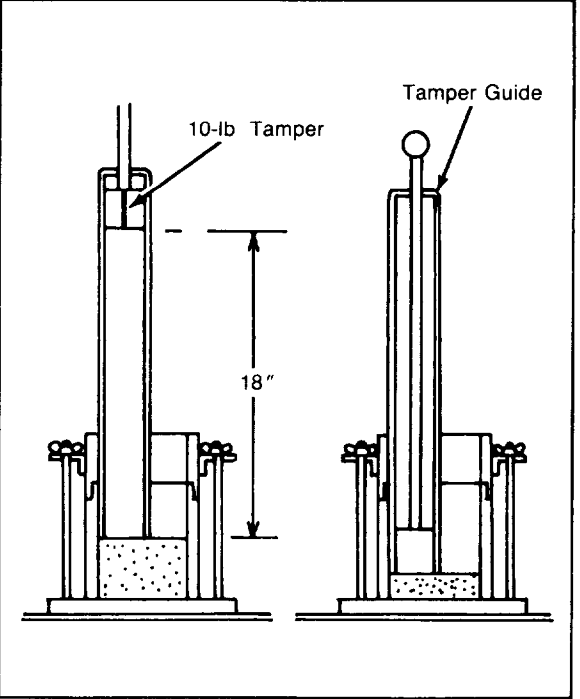


Figure 2-77. Soil compaction in a mold

Three sieves to process the soil (2 inch, ¾-inch, and No. 4)

Straightedge, steel, beveled along one edge

Mixing tools, such as pans, spoon, trowel, spatula, or electric mixer

Other general laboratory equipment including that necessary to perform moisture content determinations.

Preparation of Sample

The amount of soil required for the standard compaction test varies with the kind and gradation of the soil to be tested. For soils passing the No. 4 sieve that are to be tested in the 4-inch mold, 30 pounds of soil is normally sufficient. For samples containing gravel that are to be tested in the CBR mold, approximately 75 pounds of processed material is required for five molds. The soil to be tested will be air dried. Break the lumps present in the sample, but take care that the natural size of the individual particles is not reduced. Then screen the material through a ¾ inch and a No. 4 sieve.

- If all material passes the No. 4 sieve, mix the sample thoroughly to prepare for compaction in the 4-inch mold. If the entire sample passes the ¾-inch sieve and contains 10 percent or less material larger than the No. 4 sieve, discard the plus No. 4 fraction and perform the test using the 4-inch compaction mold. If the entire sample passes the ¾-sieve, but contains more than 10 percent material retained on the No. 4 sieve, test it in the CBR mold. Mix the sample thoroughly, then determine its initial water content. Store the sample in an airtight container until it is ready for processing at different water contents for compaction. If some of the sample is retained on the ¾-inch sieve, but the amount is 10 percent or less, remove and discard the plus ¾-inch fraction and perform the test in the CBR mold.

- If the sample contains more than 10 percent retained on the ¾-inch sieve, remove the material retained on the ¾-inch sieve and replace it with an equal weight of material passing the ¾-inch sieve, but retained on the No. 4 sieve. One method of doing this is to sieve an additional sample through the nest of sieves (¾ inch and No. 4), saving only the material retained in between the two sieves (minus ¾ inch and plus No. 4) to form a stockpile. As each 15 pound sample is sieved over the ¾-inch sieve, the plus ¾-inch material is weighed and replaced by an equal amount of weight of the minus ¾-inch and plus No. 4 material from the stockpile. Recombine the portion passing the ¾-inch sieve and retained on the No. 4 sieve with the initial minus ¾-inch sample in the desired proportions. Mix the material well and store it in an airtight container until ready for use. Reconstituting the soil restricts the maximum size of particles and avoids the problem of trying to compact material into a layer thinner than the largest sized particles.

- Example (size correction for gravelly soils). Total weight of an air dried natural sample is 110 pounds. The results of the computations and weight adjustment are given in Table 2-8. This computation permits using all of the material retained on the No. 4 sieve and making the correction by reducing some of the material which has passed the No. 4 sieve. If the weight of the material passing the sieve was held constant, more natural material would have to be sieved while the original 44 pounds retained on the No. 4 sieve would have to be transferred and held during this extra step.

PROCEDURE

The compaction test procedures for non-gravelly soils and gravelly soils are the same with two exceptions. First, the 4-inch mold is used for the fine soil and CBR mold is used for the gravelly soil. Second, 25 tamper blows per layer are used in the 4-inch mold, and 55 blows per layer are used in the CBR mold. This results

Table 2-8. Results of soil compaction test

Screen size	Natural sample		Processed sample	
	Retained on screen lb	Retained on screen %	Desired percentage %	Computed weight lb
¾ in	16.0	14.6	0	0
No. 4	44.0	40.0	54.6	44.0
Passing No. 4	50.0	45.4	45.4	36.5
Total	110.0	100.0	100.0	80.5

in equal compactive efforts for the two mold sizes and soil volumes. Use the following steps for the test:

1. From the prepared material, weigh out 12 to 15 pounds (CBR mold) or 5 to 7 pounds (Proctor mold) dry weight of soil. This is an estimation of OMC.

2. Determine the quantity of water which, when mixed with the soil, will cause it to adhere or ball together slightly when squeezed firmly in the palm of the hand. Add the water to the sample and mix it thoroughly. This is an estimation of OMC.

3. Store the water-mixed material in an airtight container for approximately 24 hours to permit the moisture content to become uniform.

4. Weigh out at least four additional specimens as above. Two specimens will be below OMC at 2 percent intervals, and two specimens will be above OMC at 2 percent intervals. Mix and store each specimen in a separate airtight container. Note that sandy soils absorb moisture readily, and 24 hour storage may not be required. In this case, separate the five or more specimens and add the water before testing. Do not reuse a tested specimen.

5. Cover the proper mold with a light coat of oil and weigh it to the nearest gram. Do not include the spacer disk and extension collar. Record the weight on the data sheet, DD Form 1210 (Figure 2-79, page 2-100).

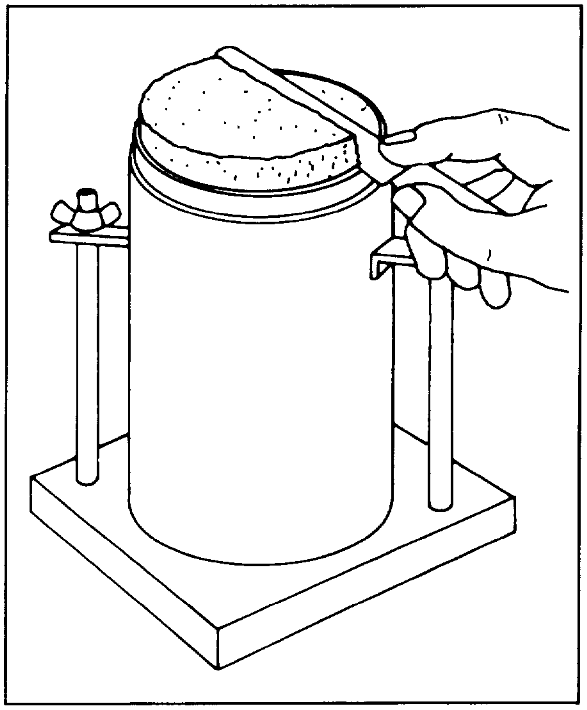


Figure 2-78. Trimming the compacted soil sample

6. Prepare the mold by attaching the collar and setting it on a concrete floor or other rigid support, such as a pedestal. If using the CBR mold with the spacer disk, place filter paper on the top of the disk.

SOIL COMPACTION TEST DATA

DATE 2 APR 1970

PROJECT JAMOCK AIRFIELD

EXCAVATION NUMBER VI-ECHO

SAMPLE NUMBER 1

NUMBER OF LAYERS 5

CONVERSION FACTORS
1728 cu. in. per cu. ft.
453.6 gm. per lb.

NUMBER OF BLOWS PER LAYER 25

WEIGHT OF TAMPER 10 lb.

HEIGHT OF DROP 18-in

MAXIMUM PARTICLE SIZE 0.187" (No. 4)

SPECIFIC GRAVITY 2.72

DIAMETER OF MOLD (in.) 4.0

HEIGHT OF SOIL SAMPLE (in.) 4.584

VOLUME OF SOIL SAMPLE (cu. ft.) 1/30

	UNITS										
RUN NUMBER		1		2		3		4		5	
WEIGHT OF WET SOIL + MOLD	lb.	12.8		13.1		13.3		13.4		13.2	
WEIGHT OF MOLD	lb.	8.7		8.7		8.7		8.7		8.7	
WEIGHT OF WET SOIL	lb.	4.1		4.4		4.6		4.7		4.5	
WET UNIT WEIGHT, $\gamma = \left(\frac{\text{Weight of wet soil (lb.)}}{\text{Vol of soil sample (cu. ft)}}\right)$	LB PER CU FT	123.0		132.0		138.0		141.0		135.0	
TARE NUMBER		2-1	2-2	2-3	2-4	2-5	2-6	2-7	2-8	2-9	2-10
A WEIGHT OF WET SOIL + TARE	gm	47.7	44.7	44.1	42.8	44.1	45.6	45.1	46.7	45.2	45.6
B WEIGHT OF DRY SOIL + TARE	gm	46.5	43.6	42.5	41.2	42.2	43.7	42.5	44.0	42.3	42.4
C WEIGHT OF WATER, W_w (A - B.)	gm	1.2	1.1	1.6	1.6	1.9	1.9	2.6	2.7	2.9	3.2
D WEIGHT OF TARE	gm	22.6	22.2	21.5	20.8	22.2	23.2	21.4	22.5	22.2	21.4
E WEIGHT OF DRY SOIL, W_s (B. - D.)	gm	23.9	21.4	21.0	20.4	20.0	20.4	21.1	21.5	20.1	20.5
WATER CONTENT, $w = \left(\frac{W_w}{W_s} \times 100\right)$	PERCENT	5.0	5.1	7.6	7.8	9.5	9.3	12.3	12.6	14.4	15.6
AVERAGE WATER CONTENT	PERCENT	5.1		7.7		9.4		12.5		15.0	
DRY UNIT WEIGHT, $\gamma_d = \left(\frac{\gamma}{1 + w/100}\right)$	LB PER CU FT	117.0		122.6		126.1		125.3		117.4	

TECHNICIAN (Signature) Sp. 6 Farrels

COMPUTED BY (Signature) McMann

CHECKED BY (Signature) 1Lt. Blake

DD Form 1210, AUG 57

Figure 2-79. Data sheet, soil compaction test data

7. Place enough of the first prepared specimen into the mold to produce a compacted layer slightly less than 1 inch thick. This is usually 1½ inch uncompacted thickness.

8. Compact the soil with the tamper, using 25 blows when compacting in the 4-inch mold and 55 blows when compacting in the CBR mold. Hold the tamper vertically, place its face on top of the soil, raise the handle until it reaches the top, and let the weight fall freely onto the soil. Change the position of the guide and tamper and repeat the process until the soil layer has received the prescribed number of blows. Be sure that the tamping is evenly distributed over the soil in the mold.

Each compacted layer should be 1 inch thick. (If more than 1 inch, add less soil for the next layer. If less than 1 inch, add more soil for the next layer.) Fill the mold by compacting the soil in five layers so that the last layer completely fills the mold and extends slightly into the collar. The final layer should extend no more than ⅜ inch into the collar.

Use a sharp pointed instrument and scarify the surface of the compacted layer to a depth of about ¼ inch. A grid of scratch lines ¾ inch to 1 inch apart provides the proper degree of scarifying. Add the next layer of the specimen and repeat the tamping process. Continue adding and tamping until five layers have been compacted in the mold. The fifth layer should extend slightly above the top of the mold and into the collar.

9. Cut around the inside edge of the collar to prevent shearing the compacted soil when removing the collar. Remove the collar and use the straightedge or spatula to trim the excess portion above the exact level of the top of the mold (Figure 2-78, page 2-99). Use a sawing motion of 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.

10. Remove the mold with the compacted specimen from the base plate, invert and remove the spacer disk, and fasten the mold to the base plate. Weigh the mold and base plate plus the specimen. Record the weight of the mold plus the soil on DD Form 1210 (Soil Compaction Test Data) (Figure 2-79).

11. Take two tares of the compacted soil (two samples from each layer), for a moisture content determination. Weigh these samples to 0.1 gram and save them for the remaining moisture content procedures.

12. Repeat steps above for the remaining specimens.

13. After completing the compaction portion of the test, complete the moisture content determinations.

Calculations

Compute the following quantities for each compacted specimen:

Wet unit weight. The weight of the compacted specimen and the mold, step (11) above, minus the weight of the mold alone, step (5), is the weight of the compacted specimen. The wet unit weight is equal to the weight of the compacted specimen divided by the volume of the mold. The volume of the 4-inch mold is 1/30 cubic foot; the volume of the CBR mold is 0.0736 cubic foot.

Moisture content (w).

$$w = \frac{\text{weight of water}}{\text{weight of dry soil}} \times 100$$

Dry unit weight.

$$\text{dry unit weight} = \frac{\text{wet unit weight}}{1 + \frac{(w)}{(100)}}$$

Compaction Curve

The compaction curve on DD Form 1211 (Soil Compaction Test Graph) (Figure 2-80) is the curve obtained by plotting moisture content versus dry density for each test. For a typical cohesive soil, dry density increases as the moisture in the soil increases. 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 maximum dry density can be obtained. After plotting the compaction curve for any given soil, it is possible to determine the moisture content which will give the maximum dry density for the particular soil directly from the plotting curve.

The compaction curve shows the OMC. This is the moisture content at which the maximum dry density is obtained for a given compaction procedure.

By determining the highest point on the compaction curve (apex) and dropping vertically down to the horizontal moisture scale or line, it is found that OMC for this particular soil, Figure 2-80, is 10.9 percent.

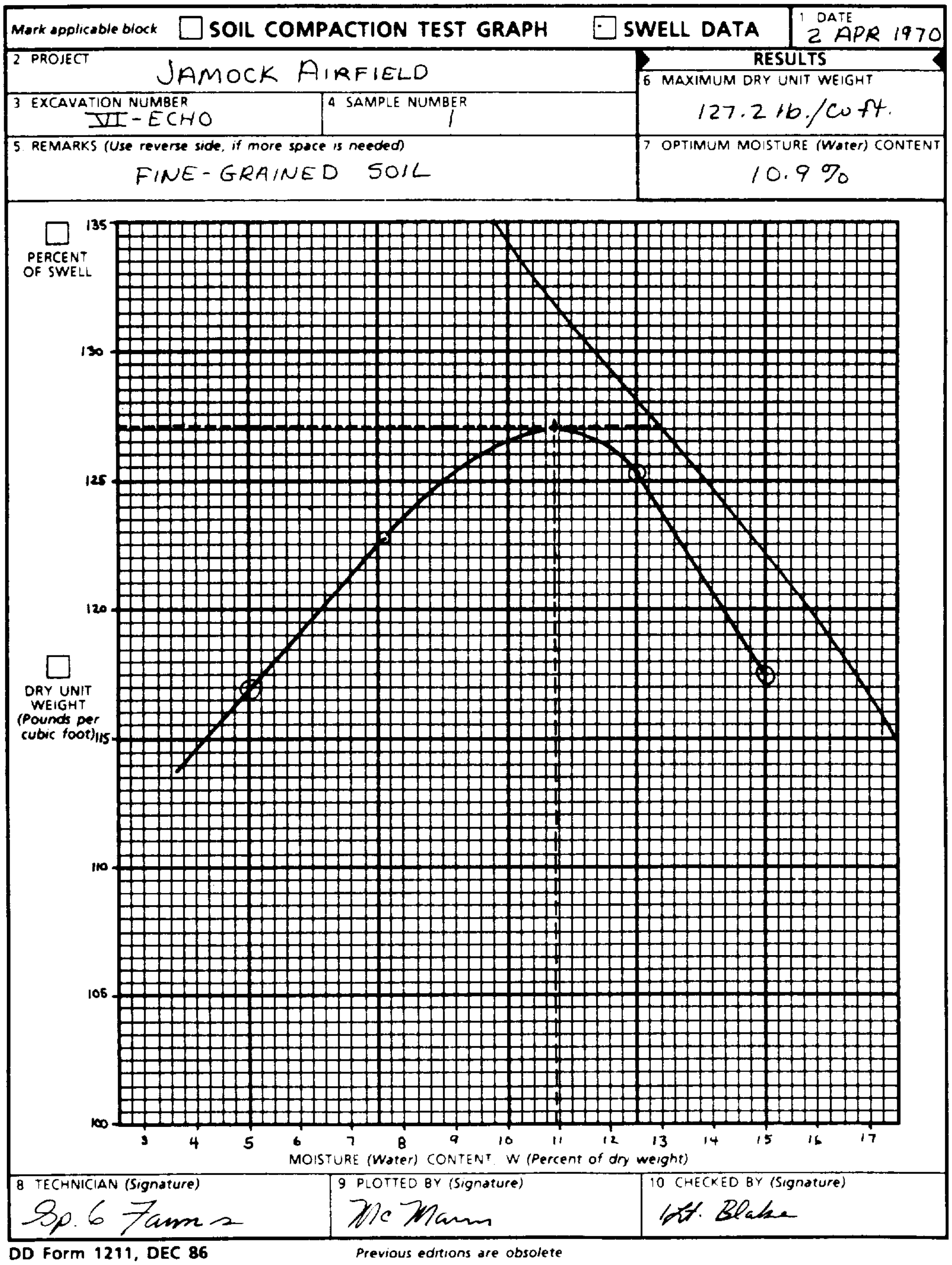
The compaction curve also shows maximum dry density (100 percent). The maximum dry density of 100 percent effort may be obtained by running a tangent from the highest point on the compaction curve for the particular soil to the vertical dry density scale (see Figure 2-80), in this case 127.2 pounds per cubic foot.

Compaction Specifications

To determine if a soil has been compacted sufficiently in the field, some specifications for roads and airfields must exist. In a theater of operations the recommended specifications for airfields, at least for the more permanent fields in rear and support areas, state that at least 100 percent compaction (or 95 percent of maximum dry density) should be obtained for all the base course construction plus at least 6 inches of the subgrade. The standard compaction range is 5 percent (100 to 105 percent). All other soil construction fill and embankment construction should be compacted to not less than 95 percent. For theater of operations roads, all of the base course construction, plus at least the upper six inches of the natural subgrade soil, should be compacted to not less than 95 percent of maximum dry density. In the zone of interior, and for special airfields, the specifications are essentially the same. The only exception is that the upper base course material is compacted to 100 percent of maximum CE55 dry unit weight, and the depth of compaction varies according to the design wheel loads.

- Percent moisture. To obtain the maximum dry unit weight or density in the field, it is necessary to control the moisture content of the construction 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 maximum dry density because more passes of the rollers will be required to compact the soil properly. In the specifications for each job the limits for moisture contents should be specified. If not specified, the limits should be assumed as ± 2 percent (± 2°) of OMC. Using Figure 2-80, where OMC is 10.9 percent, the moisture limits would be from 8.9 to 12.9 percent. This gives a limit for a workable and practical specification block.

- Percent compaction. Some soils will not or cannot be compacted to 100 percent at a reasonable equipment effort regardless of combination. In those cases it is not mandatory to compact to 100 percent. For each job the specifications will state the percent compaction required for the particular loadings. Assume that the specifications require 90 percent of maximum dry density. To find the dry density required, multiply the maximum dry density (100 percent), regardless of its value, by .90. This will give the density limit. If the specifications state



Mark applicable block ☐ **SOIL COMPACTION TEST GRAPH** ☐ **SWELL DATA**

1 DATE 2 APR 1970

2 PROJECT JAMOCK AIRFIELD

3 EXCAVATION NUMBER VII - ECHO

4 SAMPLE NUMBER 1

5 REMARKS *(Use reverse side, if more space is needed)* FINE-GRAINED SOIL

RESULTS

6 MAXIMUM DRY UNIT WEIGHT 127.2 lb./cu ft.

7 OPTIMUM MOISTURE *(Water)* CONTENT 10.9 %

8 TECHNICIAN *(Signature)* Sp. 6 James

9 PLOTTED BY *(Signature)* McMann

10 CHECKED BY *(Signature)* Lt. Blake

DD Form 1211, DEC 86 *Previous editions are obsolete*

Figure 2-80. Data sheet, soil compaction test graph

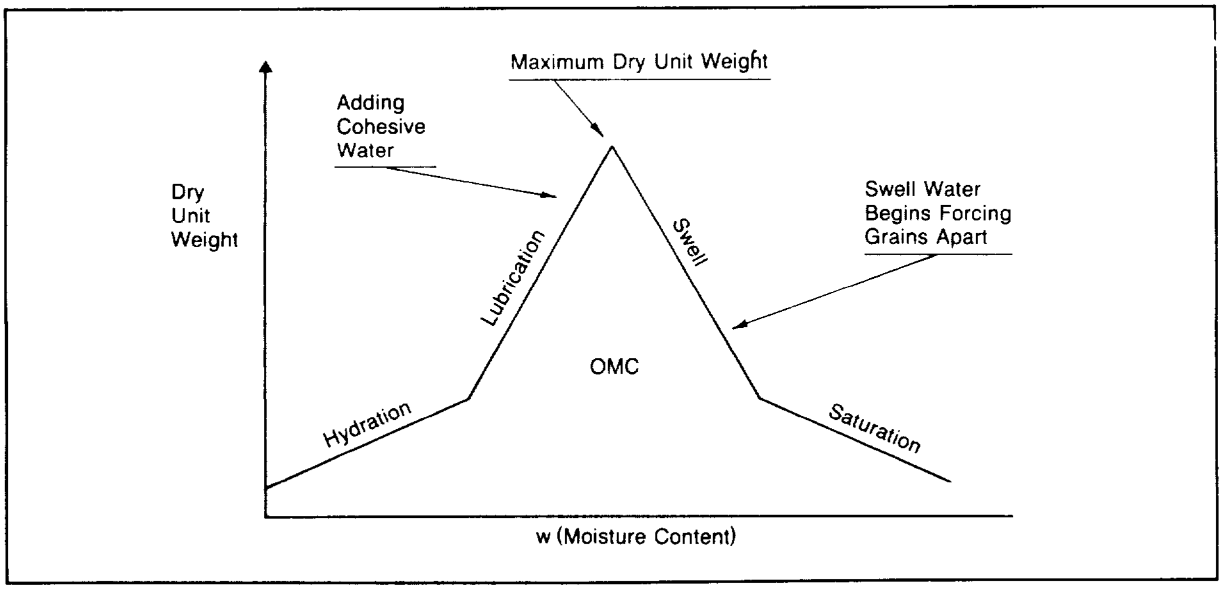


Figure 2-81. Effect of water on density

between 90 and 95 percent, the 90 percent density will constitute the lower limit with 95 percent as the upper limit. A specification block (Figure 2-80) can now be constructed.

- Effect of water on density. Figure 2-81 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 optimum moisture content is reached. The dry density then begins to decrease using a constant compactive effort.

Upon the addition of 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 adsorbed 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 and it characterizes this phase.

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 find the void spaces. This is known as the saturation phase. In the swell and saturation phases the water begins to take the place of the solids, thus decreasing the dry density.

- Effect of different compactive efforts on density. The mass per unit volume of a soil varies directly with the amount of energy expended to compact that soil. Therefore, the greater the compactive effort, the greater the amount of solids per unit volume. This results in a stronger and more stable soil.

As the compactive effort is increased, the dry density of the soil increases. This means that if more energy is used to compact a soil, the increased energy will cause the particles to be rearranged to a greater extent, thus increasing the mass of soil particles per unit volume. If the compactive effort is decreased, the particles will not be rearranged to as great an extent, thus decreasing the dry density. See Figure 2-82, which shows how the dry density will vary with compactive effort.

The OMC varies inversely with respect to compactive effort. If the compactive effort is increased, the soil does not have to be as wet to obtain the maximum dry density. In other words, the OMC will be decreased with increasing compactive effort (Figure 2-82).

- Effect of different types of soils on density. Different soils will 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 2-83, page 2-106). Note from the sharpness of the curves that moisture content is much more critical in optaining maximum density for coarse-grained soils than for the fine grained soils.

COMPACTION EQUIPMENT

Equipment normally available to the military engineer for the compaction of soils includes rollers of the sheepsfoot, pneumatic tire, and steel wheel types. Other construction equipment and loaded hauling units may also be used. Crawler-type tractor units are efficient in compacting free-draining sands and gravels which should be kept wet during the compaction process. This equipment is not efficient for the compaction of cohesive soils. Compaction equipment utilization is covered in TM 5-331A.

For this 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

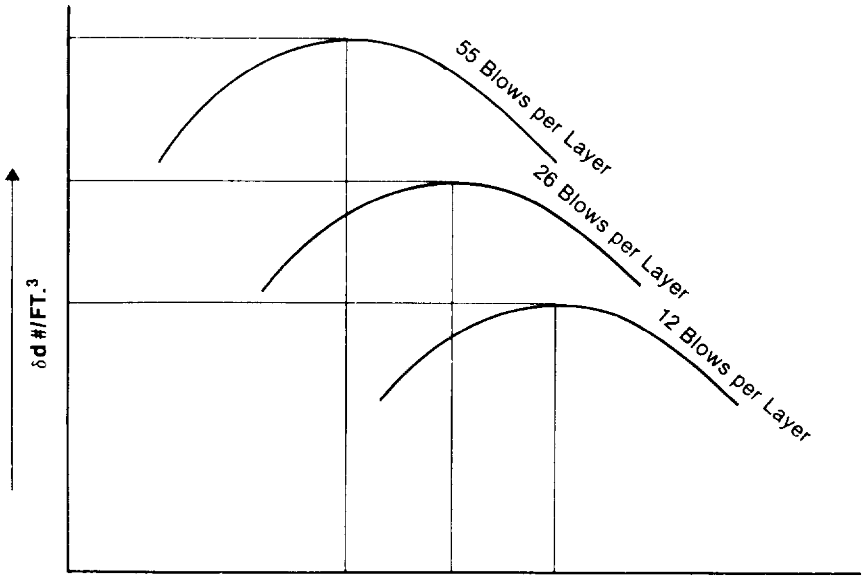


Figure 2-82. Effect of different cornpactive efforts on density

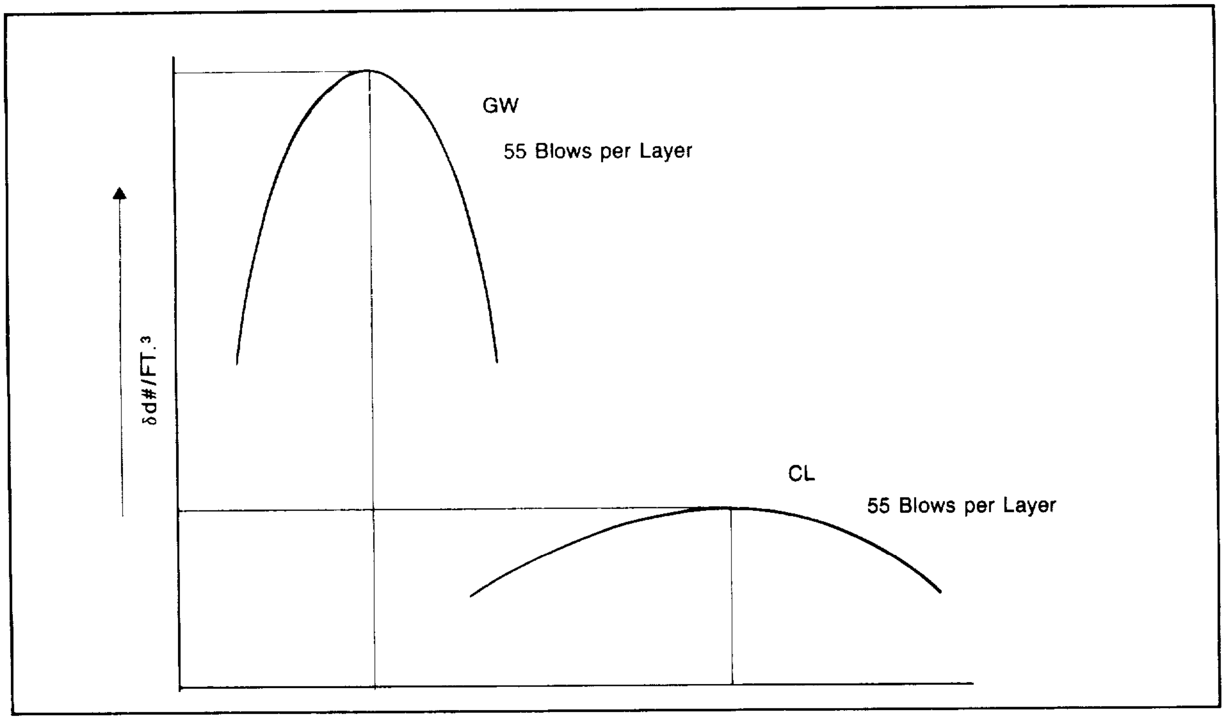


Figure 2-83. Effects of different soil types on density

are required. This data may be 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.

COMPACTION THEORY

A knowledge of the theory of compaction is essential for a job to fulfill its intended purposes. Compaction is the process of increasing the mass of soil particles per unit volume, thus increasing the strength and stability of the soil.

The moisture content of the soil affects the dry density. This is the basis for the compaction test. By varying the moisture content and compacting the soil, it is possible to obtain various dry densities. The dry density is plotted versus the moisture content. The various points are joined by a smooth curve. The peak of this curve gives the maximum dry density and the optimum moisture content. The optimum moisture content is the moisture content at which it is easiest to obtain the maximum dry density with that particular compactive effort.

After the compaction curve is obtained, it is possible to determine the specification block. This block will vary with the specification for a particular project. If the field results fall within the specification block, the job is meeting specification.

ADJUSTMENT OF MOISTURE CONTENT

The compaction curve is drawn with dry density and moisture content as coordinates. To construct an acceptable curve, at least two of the plotted points should fall on one side and two on the other 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 density, the moisture content must be determined. This can take up to 24 hours.

Compaction Curve Construction

An acceptable curve can be obtained by starting at a water content to produce a sample just wet enough to ball together when it is squeezed and released. Additional points can be obtained by increasing the water content by ± 2 to 4 percent for each of four subsequent samples.

One method to determine when the OMC has been reached while testing is to compare the wet weights of the various samples after compaction. The OMC and maximum density have been passed when the wetter compacted sample shows a decline in weight over the previous drier sample. If this occurs with the third or fourth sample, the curve will have at least two plotted points on either side of the OMC.

To aid in the selection of increments (2 percent to 4 percent increases), hints can be taken from typical curves. Generally, the more plastic the soil, the flatter the curve, and the higher the OMC.

Calculations

Compute the amount of water required to raise the moisture content of a soil to a certain percentage using the following data:

Weight of soil sample

Present moisture content

Desired moisture content.

An example of the computation is given below. Assume a 12 pound sample with a 3 percent moisture content must be raised to 10 percent.

$$\text{Dry weight} = \frac{\text{wet unit weight}}{1 + \frac{w}{100}} = \frac{12}{1 + \frac{(3)}{(100)}} = \frac{12}{1.03} = 11.65 \text{ pounds}$$

where moisture content (w) is—

$$\%w = \frac{\text{weight of water}}{\text{weight of dry soil}} \times 100$$

$$\text{thus: weight of water} = \frac{\text{moisture content} \times \text{dry weight of soil}}{100} =$$

$$\frac{7 \times 11.65}{100} = 0.816 \text{ pounds}$$

The 7 percent moisture content is derived from the required value (10 percent) minus the content in the soil (3 percent).

To convert to grams:

$0.816 \times 454 = 370$ grams

$$\text{or: } = \frac{\text{weight of soil}}{1 + \frac{\text{actual weight}}{100}} \times \frac{\text{desired actual}}{100} =$$

$$\frac{12}{3} \times \frac{7}{100} = \frac{12}{1.03} \times .07 = 0.8155 \text{ pounds}$$

DEGREE OF SATURATION

At complete saturation the voids (space between particles) in the solid mass are completely filled with water. That is, no air is present, and the degree of saturation (S) is equal to 100 percent. The zero air voids curve (100 percent saturation) for the soil used (specific gravity, G, equal to 2.72) is shown on the plot of Figure 2-80, page 2-103. This curve is obtained by plotting dry densities corresponding to complete saturation at different moisture contents.

Complete Saturation

When $S = 100$ the moisture content (w_{100}) required to fill all voids depends upon only the specific gravity of the soil particles (G) and the dry unit weight (w_d).

$$w_{(100)} = \frac{62.43}{w_d} - \frac{1}{G}$$

(The unit weight of 1 cubic foot of water is 62.4 pounds.) This expression may be used to compute points for plotting the zero air voids curve. Table 2-9 gives values of water content and densities at zero air voids (complete saturation) for the usual range of specific gravity and dry density. The specific gravity of the soil is determined by the described methods.

Zero Air Voids

This curve represents theoretical values which are practically unattainable because it is not possible to remove all the air contained in the voids of a soil by compaction alone. Typically, at moisture contents beyond optimum, the actual compaction curve closely parallels the theoretically perfect compaction curve. Any values of dry density which 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.

Degree of Saturation and Actual Compaction Curve

With the plot of Figure 2-80 available, the degree of saturation corresponding to any point on the moisture-density curve may be calculated by dividing the actual water content of the specimen by water content which corresponds to 100 percent saturation (at the same density).

For example: at maximum density for the soil represented by the plot of Figure 2-80, the actual water content is 10.9 percent. At 100 percent saturation and the same density, the corresponding moisture is 12.3 percent. Therefore, the degree of saturation and maximum density is as follows:

$$10.9\%/12.3\% = 0.89 \text{ or } 89\%.$$

OTHER COMPACTIVE EFFORTS

Under some circumstances it may be necessary to use a compactive effort in the laboratory. Usually this is done to study the effect of variation in density upon 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 MIL-STD-621A. In unusual circumstances the laboratory compaction procedure may be changed to produce a compactive effort which 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 12, 26, 55, or 72 blows per layer. This is discussed under the CBR (Section XI).

Table 2-9. Data for zero air voids curve

Specific Gravity of Soil	Water Content, w_{100}, in Percent of Dry Weight for Dry Unit Weight, w_d, in Pounds per Cubic Foot of																			
	50	55	60	65	70	75	80	85	90	95	100	105	110	115	120	125	130	135	140	145
2.40	83.2	71.8	62.4	54.4	47.5	41.6	36.4	31.8	27.7	24.1	20.8	17.8	15.1	12.6	10.4	8.3	6.4	4.6	2.9	1.9
2.45	84.0	72.7	63.2	55.2	48.4	42.4	37.2	32.6	28.6	24.9	21.6	18.6	15.9	13.5	11.2	9.1	7.2	5.4	3.8	2.2
2.50	84.9	73.5	64.1	56.1	49.2	43.2	38.0	33.5	29.4	25.7	22.4	19.5	16.8	14.3	12.0	9.9	8.0	6.2	4.6	3.1
2.51	85.0	73.7	64.2	56.2	49.4	43.4	38.2	33.6	29.5	25.9	22.6	19.6	16.9	14.5	12.2	10.1	8.2	6.4	4.8	3.2
2.52	85.2	73.8	64.4	56.4	49.5	43.6	38.4	33.8	29.7	26.0	22.8	19.8	17.1	14.6	12.4	10.3	8.3	6.6	4.9	3.4
2.53	85.3	74.0	64.5	56.5	49.7	43.7	38.5	33.9	29.8	26.2	22.9	19.9	17.2	14.8	12.5	10.4	8.5	6.7	5.1	3.5
2.54	85.5	74.1	64.7	56.7	49.8	43.9	38.7	34.1	30.0	26.4	23.1	20.1	17.4	14.9	12.7	10.6	8.7	6.9	5.2	3.7
2.55	85.6	74.3	64.8	56.8	50.0	44.0	38.8	34.2	30.2	26.5	23.2	20.2	17.5	15.1	12.8	10.7	8.8	7.0	5.4	3.8
2.56	85.8	74.5	65.0	57.0	50.1	44.2	39.0	34.4	30.3	26.7	23.4	20.4	17.7	15.2	13.0	10.9	9.0	7.2	5.5	4.0
2.57	86.0	74.6	65.1	57.1	50.3	44.3	39.1	34.5	30.5	26.8	23.5	20.6	17.8	15.4	13.1	11.0	9.1	7.3	5.7	4.2
2.58	86.1	74.8	65.3	57.3	50.4	44.5	39.3	34.7	30.6	27.0	23.7	20.7	18.0	15.5	13.3	11.2	9.3	7.5	5.8	4.3
2.59	86.3	74.9	65.4	57.4	50.6	44.6	39.4	34.8	30.8	27.1	23.8	20.9	18.1	15.7	13.4	11.3	9.4	7.6	6.0	4.5
2.60	86.4	75.1	65.6	57.6	50.7	44.8	39.6	35.0	30.9	27.3	24.0	21.0	18.3	15.8	13.6	11.5	9.6	7.8	6.1	4.6
2.61	86.6	75.2	65.7	57.7	50.9	44.9	39.7	35.1	31.1	27.4	24.1	21.2	18.4	16.0	13.7	11.6	9.7	7.9	6.3	4.8
2.62	86.7	75.3	65.9	57.9	51.0	45.1	39.9	35.3	31.2	27.6	24.3	21.3	18.6	16.1	13.9	11.8	9.9	8.1	6.4	4.9
2.63	86.8	75.5	66.0	58.0	51.2	45.2	40.0	35.4	31.4	27.7	24.4	21.4	18.7	16.3	14.0	11.9	10.0	8.2	6.6	5.0
2.64	87.0	75.6	66.2	58.2	51.3	45.4	40.2	35.6	31.5	27.8	24.6	21.6	18.9	16.4	14.2	12.1	10.1	8.4	6.7	5.2
2.65	87.1	75.8	66.3	58.3	51.5	45.5	40.3	35.7	31.6	28.0	24.7	21.7	19.0	16.6	14.3	12.2	10.3	8.5	6.9	5.3
2.66	87.3	75.9	66.5	58.5	51.6	45.7	40.5	35.9	31.8	28.1	24.8	21.9	19.2	16.7	14.4	12.4	10.4	8.7	7.0	5.5
2.67	87.4	76.1	66.6	58.6	51.7	45.8	40.6	36.0	31.9	28.3	25.0	22.0	19.3	16.8	14.6	12.5	10.6	8.8	7.1	5.6
2.68	87.6	76.2	66.7	58.7	51.9	45.9	40.7	36.1	32.1	28.4	25.1	22.2	19.4	17.0	14.7	12.6	10.7	8.9	7.3	5.8
2.69	87.7	76.3	66.9	58.9	52.0	46.1	40.9	36.3	32.2	28.6	25.3	22.3	19.6	17.1	14.9	12.8	10.9	9.1	7.4	5.9
2.70	87.8	76.5	67.0	59.0	52.2	46.2	41.0	36.4	32.3	28.7	25.4	22.4	19.7	17.3	15.0	12.9	11.0	9.2	7.6	6.0
2.71	88.0	76.6	67.2	59.2	52.3	46.3	41.1	36.6	32.5	28.8	25.5	22.6	19.9	17.4	15.1	13.0	11.1	9.3	7.7	6.2
2.72	88.1	76.8	67.3	59.3	52.4	46.5	41.3	36.7	32.6	29.0	25.7	22.7	20.0	17.5	15.3	13.2	11.3	9.5	7.8	6.3
2.73	88.2	76.9	67.4	59.4	52.6	46.6	41.4	36.8	32.7	29.1	25.8	22.8	20.1	17.7	15.4	13.3	11.4	9.6	8.0	6.4
2.74	88.4	77.0	67.6	59.6	52.7	46.7	41.5	37.0	32.9	29.2	25.9	23.0	20.3	17.8	15.5	13.4	11.5	9.7	8.1	6.6
2.75	88.5	77.2	67.7	59.7	52.8	46.9	41.7	37.1	33.0	29.4	26.1	23.1	20.4	17.9	15.7	13.6	11.7	9.9	8.2	6.7
2.76	88.6	77.3	67.8	59.8	53.0	47.0	41.8	37.2	33.1	29.5	26.2	23.2	20.5	18.1	15.8	13.7	11.8	10.0	8.4	6.8
2.77	88.8	77.4	68.0	60.0	53.1	47.1	41.9	37.4	33.3	29.6	26.3	23.4	20.7	18.2	15.9	13.8	11.9	10.1	8.5	7.0
2.78	88.9	77.5	68.1	60.1	53.2	47.3	42.1	37.5	33.4	29.8	26.5	23.5	20.8	18.3	16.1	14.0	12.1	10.3	8.6	7.1
2.79	89.0	77.7	68.2	60.2	53.4	47.4	42.2	37.6	33.5	29.9	26.6	23.6	20.9	18.5	16.2	14.1	12.2	10.4	8.8	7.2
2.80	89.2	77.8	68.3	60.3	53.5	47.5	42.3	37.7	33.7	30.0	26.7	23.8	21.0	18.6	16.3	14.2	12.3	10.5	8.9	7.4
2.81	89.3	77.9	68.5	60.5	53.6	47.7	42.5	37.9	33.8	30.1	26.8	23.9	21.2	18.7	16.4	14.4	12.4	10.7	9.0	7.5
2.82	89.4	78.1	68.6	60.6	53.7	47.8	42.6	38.0	33.9	30.3	27.0	24.0	21.3	18.8	16.6	14.5	12.6	10.8	9.1	7.6
2.83	89.5	78.2	68.7	60.7	53.9	47.9	42.7	38.1	34.0	30.4	27.1	24.1	21.4	19.0	16.7	14.6	12.7	10.9	9.3	7.7
2.84	89.7	78.3	68.8	60.8	54.0	48.0	42.8	38.2	34.2	30.5	27.2	24.3	21.5	19.1	16.8	14.7	12.8	11.0	9.4	7.9
2.85	89.8	78.4	69.0	61.0	54.1	48.2	43.0	38.4	34.3	30.6	27.3	24.4	21.7	19.2	16.9	14.9	12.9	11.2	9.5	8.0
2.86	89.9	78.5	69.1	61.1	54.2	48.3	43.1	38.5	34.4	30.8	27.5	24.5	21.8	19.3	17.1	15.0	13.1	11.3	9.6	8.1
2.87	90.0	78.6	69.2	61.2	54.4	48.4	43.2	38.6	34.5	30.9	27.6	24.6	21.9	19.5	17.2	15.1	13.2	11.4	9.8	8.2
2.88	90.1	78.8	69.3	61.3	54.5	48.5	43.3	38.7	34.7	31.0	27.7	24.7	22.0	19.6	17.3	15.2	13.3	11.5	9.9	8.3
2.89	90.3	78.9	69.5	61.5	54.6	48.6	43.4	38.9	34.8	31.1	27.8	24.9	22.2	19.7	17.4	15.3	13.4	11.6	10.0	8.5
2.90	90.4	79.0	69.6	61.6	54.7	48.8	43.6	39.0	34.9	31.2	28.0	25.0	22.3	19.8	17.6	15.5	13.5	11.8	10.1	8.6
2.95	91.0	79.6	70.2	62.2	55.3	49.3	44.1	39.6	35.5	31.8	28.5	25.6	22.9	20.4	18.1	16.0	14.1	12.3	10.7	9.2
3.00	91.5	80.2	70.7	62.7	55.9	49.9	44.7	40.1	36.0	32.4	29.1	26.1	23.4	21.0	18.7	16.6	14.7	12.9	11.3	9.7
3.05	92.1	80.7	71.3	63.3	56.4	50.5	45.3	40.7	36.6	32.9	29.6	26.7	24.0	21.5	19.2	17.2	15.2	13.5	11.8	10.3
3.15	93.1	81.8	72.3	64.3	57.4	51.5	46.3	41.7	37.6	34.0	30.7	27.7	25.0	22.5	20.3	18.2	16.3	14.5	12.8	11.3

Note: Zero air voids curve equivalent to a degree of saturation, S, equal to 100 percent. Data for zero aid voids curve computed from the Degree of Saturation formula in this section.

Section X. DENSITY DETERMINATION

DISCUSSION

Proper field control is essential in earthwork construction. The control tests are conducted on the soil at the job site as construction proceeds. If at any time a test indicates that operations are not producing a soil condition specified by the design tests, immediate action should be taken to remedy the situation.

The stress deformation characteristics of a 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. Water contents are compared with the optimum moisture 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 the 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 (such as sand, oil, or water) and computing the volume of the hole, which is equal to the volume of soil removed. The soil's moisture content and density are then determined.

The method for in-place density depends upon 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 soils 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. Water displacement 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 below 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 which is being compacted in the field is not the same one which was 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, the job of inspection may be determining and verifying the number of passes and the combination of the 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 DISPLACEMENT METHOD

The sand-displacement method may be used in either fine-grained or coarse-grained materials. Calibrated sand is used to determine the volume of the hole from which a sample has been taken. This test method is described in MIL-STD-621A and is abbreviated in the following paragraphs:

Preparation

The test consists of digging out a sample of the material to be tested, determining the volume of the hole, and determining the dry weight of the sample. There are three requirements that must be met for this test.

- The volume of the sample should be close to the same volume at which the sand was calibrated. Samples of 0.05 cubic foot volume are satisfactory for materials with a maximum aggregate size of ½ inch. For 1-inch particles, a minimum of 0.075 cubic foot is required; for 2-inch particles, 0.100 cubic foot is required.

- 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. Uniformly-graded and well-rounded sand, preferably having a gradation where all the sand passes a No. 20 sieve and is retained on a No. 40 sieve, is most suitable for this test. There should be practically no material finer than the No. 200 sieve in the sand.

Apparatus

The essential equipment, some of which is shown in Figure 2-84, page 2-112, follows:

Can, 1 gallon capacity, with close fitting lid

Cylinder, soil compaction

Chisel

Hammer

Knife

Nails, twentypenny (20d) common

Scale, bench, twin weigh beams, 21,100 gram capacity

Spoon, cooking

Straightedge, steel

Tester, soil density, sand method, double cone assembly, with 1 gallon sand jug and sand

Tray, template, soil density, 4 inch hole or 6 inch hole

Bake pan and moisture content cans

Paint brush with moderately long bristles

Modeling clay.

Calibration

The volume of the jar and connecting cone up through the valve and the empty weight of the apparatus must be known before the sand can be calibrated. A data sheet (Figure 2-85, page 2-113) should be prepared for recording the information. The sand calibration is repeated prior to each test day. Surface calibration must be made to account for surface irregularities between the surface of the cone base and the surface to be tested.

- Apparatus calibration. To determine the weight and volume of the apparatus, the procedure is as follows:

Weigh the empty dry cone and jar and record the weight (in grams).

Screw the cone tightly on the jar, place the apparatus upright, and open the valve.

Fill the apparatus with water at room temperature until the water level stands some distance up in the top cone. Make sure no air bubbles are trapped in the apparatus.

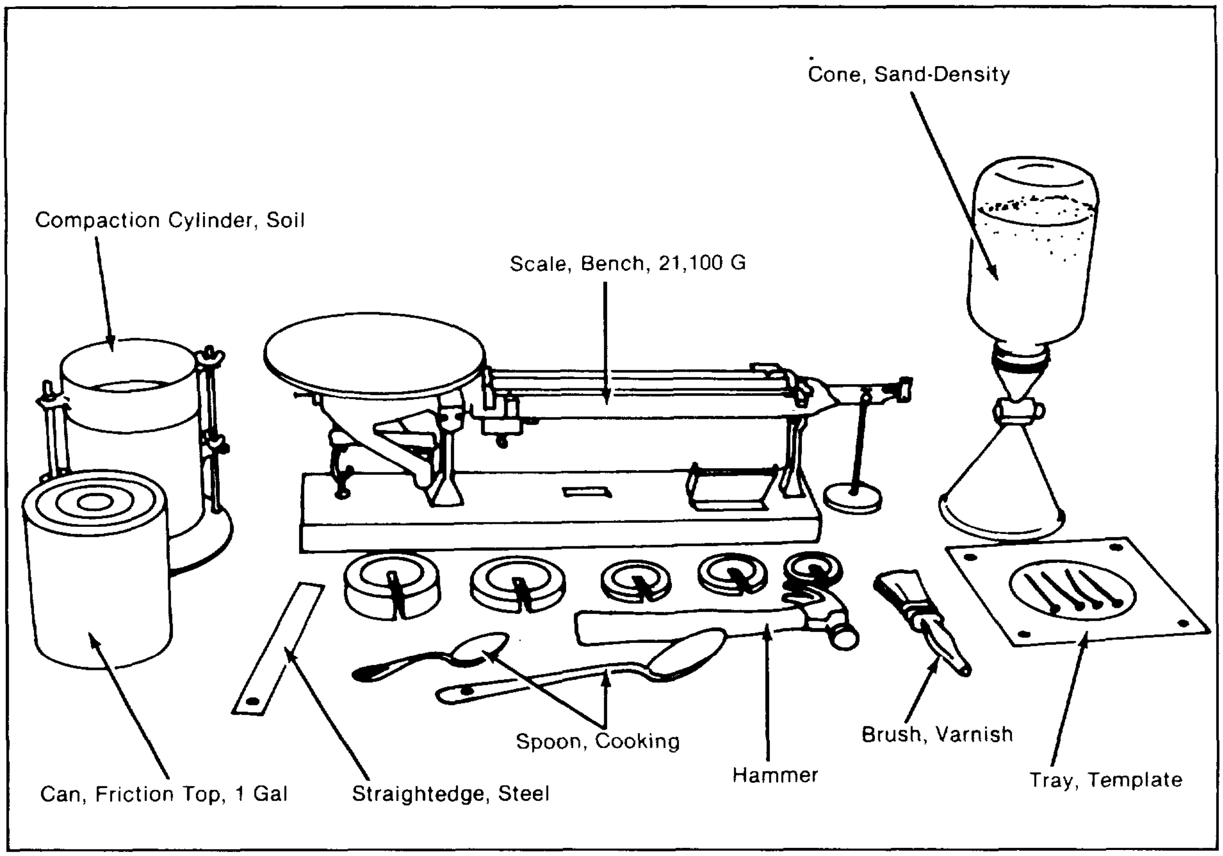

Cone, Sand-Density
Compaction Cylinder, Soil
Scale, Bench, 21,100 G
Spoon, Cooking
Brush, Varnish
Hammer
Tray, Template
Can, Friction Top, 1 Gal
Straightedge, Steel

Figure 2-84. Sand displacement method apparatus

Close the cone valve, pour off the excess water, shake the apparatus to check for entrapped bubbles, and dry the cone and outside of the apparatus.

Weigh the water-filled apparatus and record the weight (in grams).

Empty the water from the apparatus.

Repeat all steps above two or more times.

Compute the volume of the apparatus by averaging the measured weights, converting the average to pounds (divide by 453.6 grams per pound), and converting the average weight in pounds to volume in cubic feet. Note the example of the computation in Figure 2-85.

- Sand calibration. The bulk density of sand is affected by changes in temperature and humidity. Sand calibration must be checked prior to each test day. The information is recorded on DD Form 1215 (Unit Weight Determination) (Figure 2-86, page 2-114). The procedure is as follows:

Weigh the empty cone and jar and record the weight (figure 2-86, line 2).

Check the cone to see that it is screwed tightly on the jar and place the apparatus on a firm, level surface.

Close the cone valve and fill the upper cone with air dried sand.

CALIBRATION OF SAND-CONE APPARATUS

Sane No. 1 **Batch No.** A **Date** ____

	1	2	3	Avg
Weight of apparatus filled with water, grams	11,295	11,299	11,303	
Weight of apparatus empty and dry, grams	4248	4249	4250	
Weight of water, grams	7047	7050	7053	7050

$$\textbf{Weight of water in pounds} = \frac{\textbf{Weight in grams}}{453.6} = \frac{7050}{453.6} = 15.542 \ \textbf{pounds}$$

$$\textbf{Volume of apparatus} = \frac{\textbf{Weight water in pounds}}{62.3} = \frac{15.542}{62.3} = 0.249 \ \textbf{cubic feet}$$

Figure 2-85. Data sheet, calibration of sand-cone apparatus

Open the cone valve. As the sand runs into the jar keep adding more sand to the upper cone to maintain a steady, full stream of sand through the valve. Be careful not to jar the apparatus while the sand is being poured.

Close the valve when the jar and lower cone are filled and the sand stops flowing.

Dump the excess sand from the upper cone.

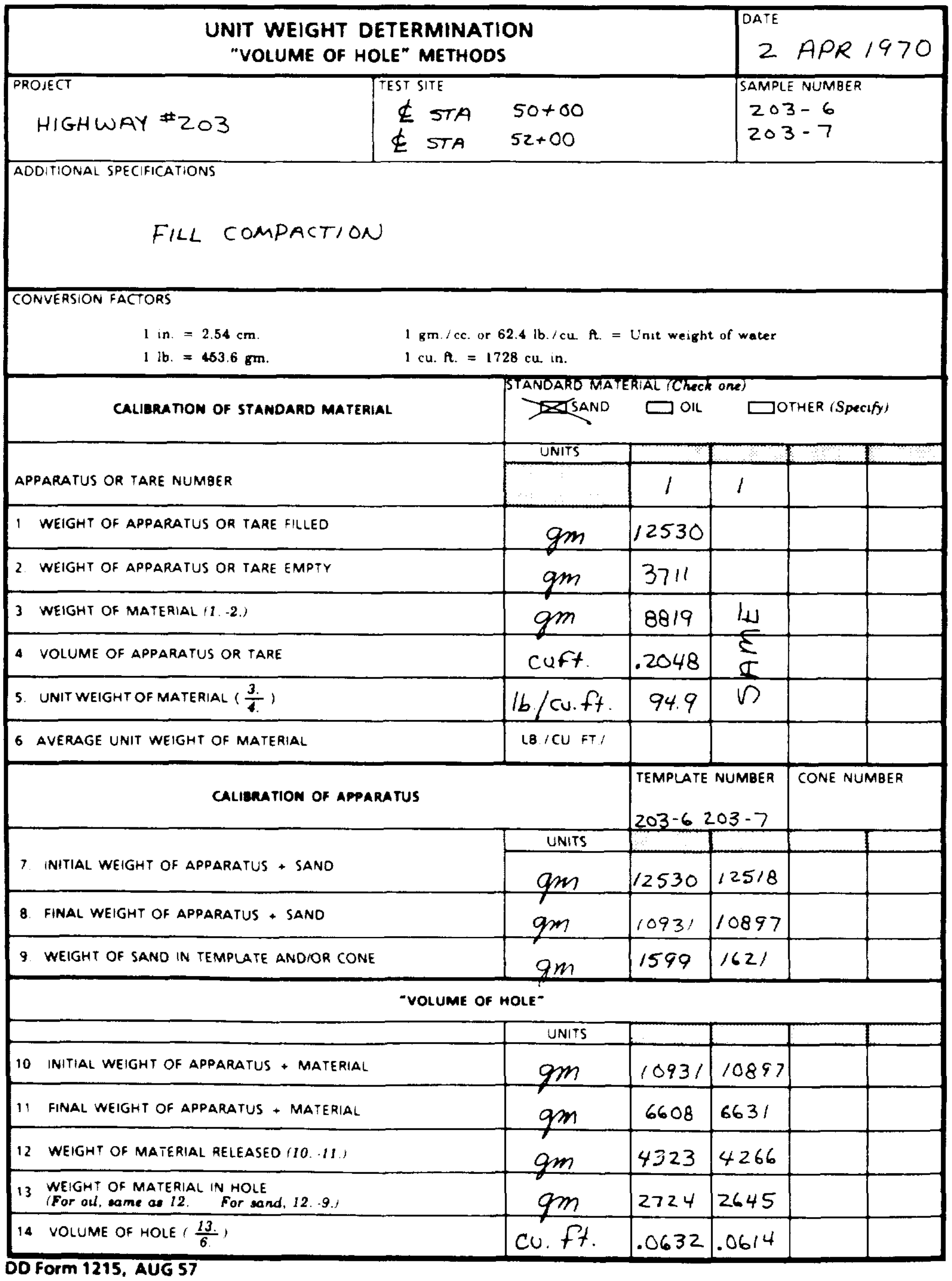
Weigh the sand filled apparatus and record the weight (Figure 2-86, line 1).

Compute the calibrated density (in pounds per cubic foot) by subtracting the empty apparatus from the weight of the filled apparatus (Figure 2-86, line 1 minus line 1 equals line 3) and dividing first by 453.6 to convert grams to pounds and then by the volume of the apparatus (Figure 2-86, line 4).

An alternate and faster method of sand calibration is to use a container of known volume, such as the Proctor or CBR mold:

Weigh the empty mold with the base plate attached.

Attach the collar and place the sand cone with the valve open on top.



UNIT WEIGHT DETERMINATION
"VOLUME OF HOLE" METHODS

DATE: 2 APR 1970

PROJECT	TEST SITE	SAMPLE NUMBER
HIGHWAY #203	₵ STA 50+00 ₵ STA 52+00	203-6 203-7

ADDITIONAL SPECIFICATIONS: FILL COMPACTION

CONVERSION FACTORS

1 in. = 2.54 cm.
1 lb. = 453.6 gm.
1 gm./cc. or 62.4 lb./cu. ft. = Unit weight of water
1 cu. ft. = 1728 cu. in.

CALIBRATION OF STANDARD MATERIAL	STANDARD MATERIAL *(Check one)* ☒ SAND ☐ OIL ☐ OTHER *(Specify)*				
	UNITS				
APPARATUS OR TARE NUMBER		1	1		
1 WEIGHT OF APPARATUS OR TARE FILLED	gm	12530			
2 WEIGHT OF APPARATUS OR TARE EMPTY	gm	3711			
3 WEIGHT OF MATERIAL *(1.-2.)*	gm	8819	SAME		
4 VOLUME OF APPARATUS OR TARE	cuft.	.2048			
5 UNIT WEIGHT OF MATERIAL ($\frac{3.}{4.}$)	lb/cu.ft.	94.9			
6 AVERAGE UNIT WEIGHT OF MATERIAL	LB/CU FT/				

CALIBRATION OF APPARATUS		TEMPLATE NUMBER		CONE NUMBER	
		203-6	203-7		
	UNITS				
7 INITIAL WEIGHT OF APPARATUS + SAND	gm	12530	12518		
8 FINAL WEIGHT OF APPARATUS + SAND	gm	10931	10897		
9 WEIGHT OF SAND IN TEMPLATE AND/OR CONE	gm	1599	1621		

"VOLUME OF HOLE"

	UNITS				
10 INITIAL WEIGHT OF APPARATUS + MATERIAL	gm	10931	10897		
11 FINAL WEIGHT OF APPARATUS + MATERIAL	gm	6608	6631		
12 WEIGHT OF MATERIAL RELEASED *(10.-11.)*	gm	4323	4266		
13 WEIGHT OF MATERIAL IN HOLE *(For oil, same as 12. For sand, 12.-9.)*	gm	2724	2645		
14 VOLUME OF HOLE ($\frac{13.}{6.}$)	cu. ft.	.0632	.0614		

DD Form 1215, AUG 57

Figure 2-86. Data sheet, unit weight determinations

Close the valve on the cone when the sand stops running into the mold.

Carefully remove the sand cone and the mold collar in turn.

Strike off the excess sand remaining on top of the mold.

Brush off any sand adhering to the outside of the mold.

Weigh the full mold.

Repeat the entire process and average the two weights (full mold).

Subtract the weight of the empty mold from the average of the full weights. Divide the difference in weights by the known volume of the container to determine the density of the sand.

- Surface calibration. This must be made prior to starting the test. It is performed after the surface is prepared and the template tray is in place, and it is part of the test procedure.

Procedure

The preparation and calibration up to this point have dealt solely with the equipment. The next procedures are performed on the soil at the site where the test is required.

- Surface calibration. The surface irregularities inside of the metal tray must be considered. To do this the volume of space between the inner surface of the upper cone and the test surface must be checked.

Prepare the ground surface where density is to be determined by clearing and leveling. Do not attempt to pack or smooth the surface with a spatula, trowel, or other tool.

Seat the template tray (Figure 2-84, page 2-112) flush on the surface, especially around the center hole.

Seal any spaces on the inside edge under the plate with modeling clay and force nails through the holes in the template to hold it firmly in position.

Close the valve, turn the apparatus over, large cone down, and place it over the hole in the plate. Do not jar or shake the container and keep the heavier equipment at a distance. Jarring will increase the amount of sand that falls into the hole and will pack the sand.

Open the valve carefully and permit the sand to fill the space under the cone, taking care not to shake or vibrate the container. Avoid performing the test while heavy equipment is operating in the area because vibrations may affect test results.

Close the valve and weigh the apparatus and remaining sand. Record this weight (Figure 2-86, line 8). The difference in weight between the starting and final valves (lines 7 and 8) is the weight of sand in the template and cone (line 9).

Make the calibration and recover as much of the sand from the plate as feasible without disturbing the template or the soil on the hole. Brush the remaining sand particles lightly from within the template. Place the sand removed into a can and save.

- Digging the sample. The next procedure is to dig into the soil through the center hole in the plate. This requires a hole which is about 4- or 6-inches deep and approximately the same diameter as the hole in the plate. The digging of samples in nonplastic coarse-grained soils is difficult, especially in keeping the walls of the hole undisturbed. Do not try driving a chisel or other tool with a hammer unless no other method works and then only near the center of the hole. Keep the inside of the hole as free of pockets and sharp projections as possible. Coarse-grained material with plastic fines or fine-grained material with plastic fine-grained soils are easier to dig. Take care to remove all loose particles from the hole and to see that all the removed particles are included in a container whose weight has been previously determined and recorded (Figure 2-87, line 23). Keep the lid on the container as much as possible to prevent excessive moisture loss until it can be weighed. Weigh the sample and container immediately and record the weight (line 15). Mark the container for later identification when the soil moisture content is determined.

Sand Density Procedure

After the soil sample has been removed from the hole, the sand density procedure is as follows:

Place the sand cone apparatus on the metal plate over the hole. Do not jar or shake the container. This causes compaction of the sand.

Open the valve and allow the sand to fill the hole in the ground and the space under the cone.

When the sand stops flowing close the valve, remove the apparatus, and weigh it and the remaining sand. Record this weight (Figure 2-86, line 11, page 2-114).

The weight of sand required to fill the hole equals the weight of the apparatus and sand (line 8 transferred to line 10) minus the final weight from step (3) above and minus weight of sand in the template and cone (line 9).

Compute the volume of the hole by dividing the weight of sand in the hole by the calibrated density or unit weight (line 6).

Density Determination

With the volume of the hole computed, the remaining requirements are the moisture content and the dry density of the soil sample.

- The container with the sample is oven dried and weighed. Record this weight (Figure 2-87, line 16). The difference (line 15 – line 16 = line 17) is the weight of water in the sample. Subtracting the tare weight (line 18) from the dry soil and tare weight (line 16) results in the weight of dry soil (line 19). Moisture content (line 20) is the weight of water (line 17) divided by the dry weights (line 19) and expressed as a percentage.

- If drying the entire sample with container is not possible, an alternate but less desirable method is to select a specimen from the container and determine the water content for it by using a speedy moisture tester conversion chart (Figure 2-65, page 2-85).

- The wet density or unit weight (line 25) is computed by dividing the wet soil weight (line 24) by 453.6 to convert the grams to pounds and then by the volume of the hole (line 14). The dry density or unit weight is computed as follows:

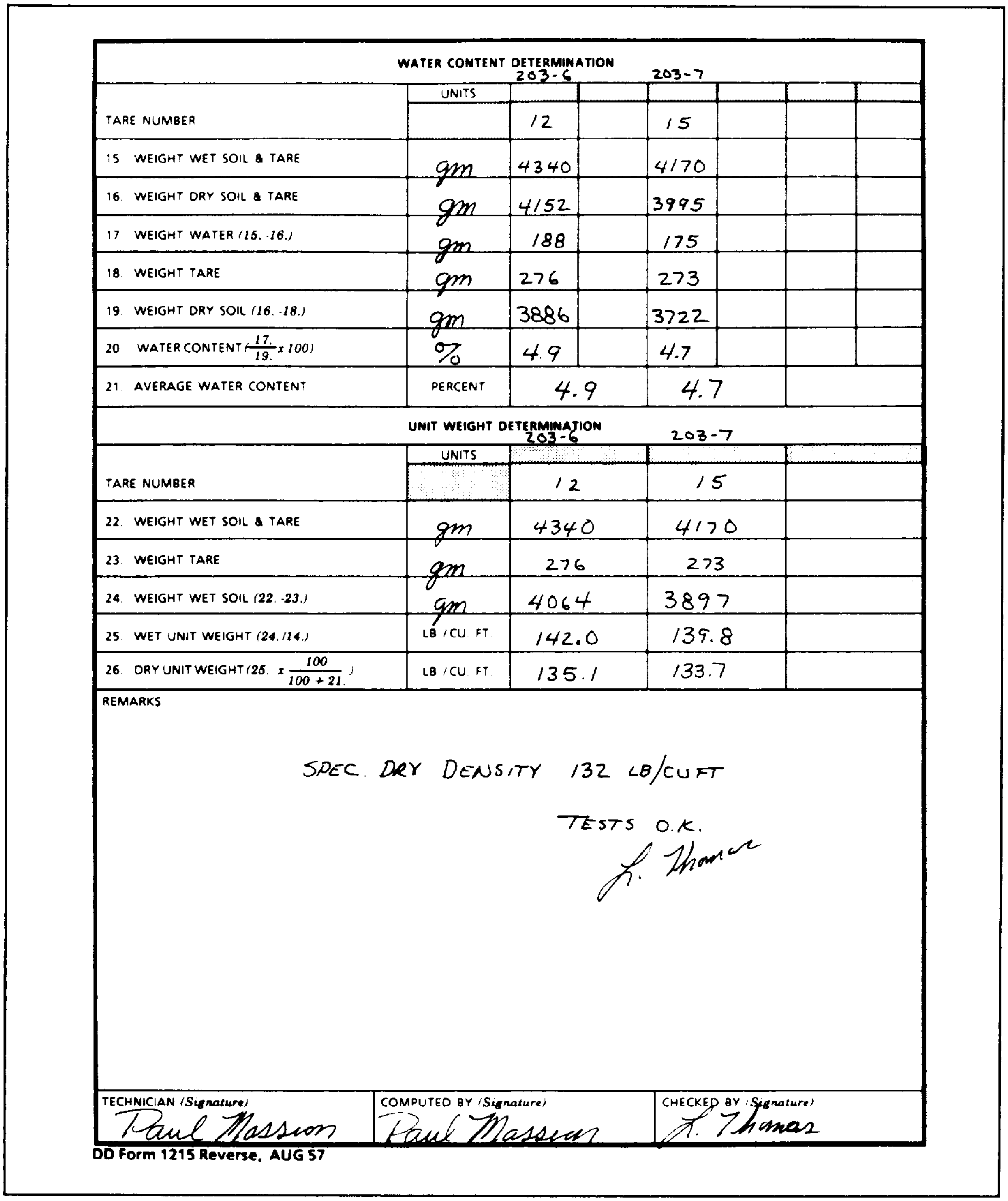
$$\text{dry density (line 26)} = \frac{\text{wet density}}{1 + \frac{(w)}{(100)}}$$

The calcium carbide gas pressure method may be used when time is critical.

NUCLEAR MOISTURE DENSITY GAGE

This method is used to determine the dry density and moisture content of soils. Although individual models of equipment will vary in procedure, the general principles in performing this test are similar for all gages. One model is shown in Figure 2-88, page 2-118.

Each gage contains two sources of radioactive material, typically cesium and a combination of americium mixed with beryllium powder. The cesium emits gamma radiation which the detector in the gage can count when it is passed through the soil. This count can be translated into density. The americium/beryllium emits neutrons following collisions with hydrogen which are moderated and detected by the gage. The moisture content can be determined by counting the hydrogen in the soil.



WATER CONTENT DETERMINATION

	UNITS	203-6		203-7			
TARE NUMBER		12		15			
15 WEIGHT WET SOIL & TARE	gm	4340		4170			
16 WEIGHT DRY SOIL & TARE	gm	4152		3995			
17 WEIGHT WATER *(15. -16.)*	gm	188		175			
18 WEIGHT TARE	gm	276		273			
19 WEIGHT DRY SOIL *(16. -18.)*	gm	3886		3722			
20 WATER CONTENT *($\frac{17.}{19.}$ x 100)*	%	4.9		4.7			
21 AVERAGE WATER CONTENT	PERCENT	4.9		4.7			

UNIT WEIGHT DETERMINATION

	UNITS	203-6	203-7	
TARE NUMBER		12	15	
22. WEIGHT WET SOIL & TARE	gm	4340	4170	
23. WEIGHT TARE	gm	276	273	
24. WEIGHT WET SOIL *(22. -23.)*	gm	4064	3897	
25. WET UNIT WEIGHT *(24./14.)*	LB /CU FT	142.0	139.8	
26. DRY UNIT WEIGHT *(25. x $\frac{100}{100 + 21.}$)*	LB /CU FT	135.1	133.7	

REMARKS

SPEC. DRY DENSITY 132 LB/CU FT

TESTS O.K.

L. Thomas

TECHNICIAN *(Signature)*	COMPUTED BY *(Signature)*	CHECKED BY *(Signature)*
Paul Massion	Paul Massion	L. Thomas

DD Form 1215 Reverse, AUG 57

Figure 2-87. Reverse of data sheet, unit weight determination

Preparation

Prior to testing, a standard is obtained to indicate the radioactivity level of the source on that day. The gage is placed on a reference block, usually made of Teflon, and a 4-minute count procedure is started. The moisture and density counts obtained will serve as standard counts for testing that day.

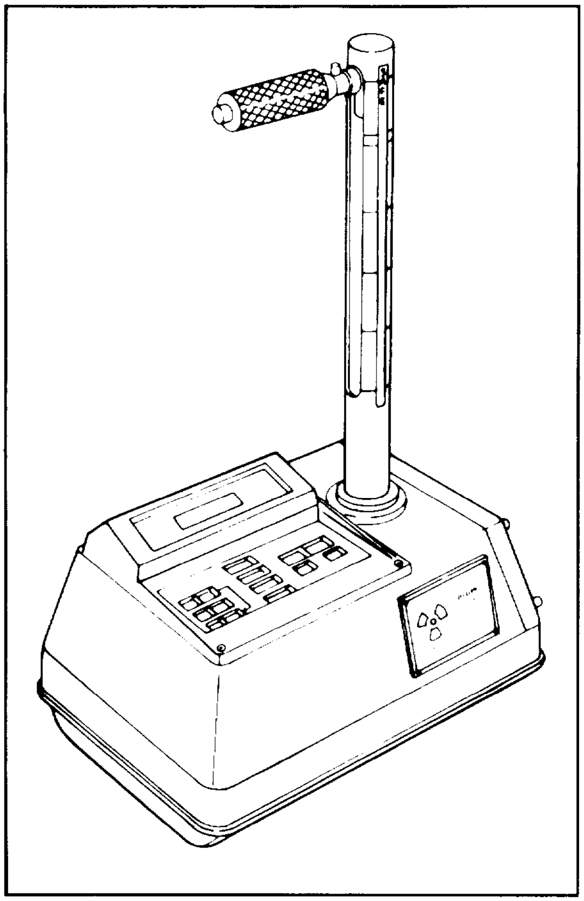


Figure 2-88. Nuclear method density tester

Procedure

To get a reading, the test site is smoothed with a guide plate. The gage may be placed directly on the soil for a backscatter reading to measure the top 2 to 3 inches, or a hole is made by driving a drill pin through a hole in the guide plate. The gage's source rod can be extended into this hole to obtain readings of depths up to 8 inches.

Calculations

To determine the density and moisture content, the counts obtained by the gage are divided by the standard counts, giving two ratios.

A calibration chart is entered with the ratios, and wet density and moisture content are read from the charts. Wet density divided by one plus the moisture content (expressed as a decimal) gives dry density. Newer gages may perform these calculations and report the moisture content and dry density directly.

NOTE

Nuclear moisture-density gages require proper storage and transportation. Operators must be personnel in the Materials Quality Specialist (SIG) career management field, who have received training and licensing on the gage. This training is conducted by the Engineer School, Fort Belvoir, VA, during the Materials Quality Specialist course.

Further information is available in manufacturer's operating instructions and in TM 5-6635-386-12&P.

WATER DISPLACEMENT METHOD

The volume of the hole from which a soil density sample is taken can be measured 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 water tight 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. The cylinder in the device is filled with water and the apparatus is placed over the area where the sample is to be removed. The balloon is allowed to fill with water and an initial reading is taken. After the sample is removed from the ground, the device is replaced over the dug hole in the original position. The water is allowed to flow by gravity into the balloon in the hole. Blowing through the hose attached to the device increases the air pressure on the water surface and forces the water filled balloon in the hole to conform to all the contours of the hole. Observing the scale attached to the float results in 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.

DRIVE CYLINDER METHOD

The drive-cylinder method of determining in-place density can be used satisfactorily in moist, cohesive, fine-grained soil and in many sands which exhibit tendencies toward cohesiveness. The drive samplers are thin-walled cylinders driven into the soil to obtain an undisturbed sample. These may be driven samplers or an expedient form of a sampler.

Preparation

To obtain an equal sample at all times the sample must be cut flush with the ends of the cylinder. This is done with a steel straightedge sharpened on one edge, with a knife, or with a special cutter used with the soil trafficability sampler. The samples are dug out to produce an undisturbed soil sample. A balance, capable of weighing to 0.1 gram, is needed to weigh the soil sample in its different states, and an oven is needed to dry the sample. A complete description of equipment and procedures for this method is given in MIL-STD-621A. An abbreviated procedure is given below.

Procedures

- Near surface. The weight of the cylinder (in grams) and the volume (in cubic inches) are determined and recorded prior to starting the test.

The soil at the point of testing is prepared by clearing all loose material and exposing a soil surface. The drive cylinder is placed in position on the surface and driven into the soil so that the top of the cylinder is at least ½ inch below the original surface. The cylinder is dug from the ground by cutting soil away from around the cylinder for several inches and undercutting the cylinder several inches below its lower edge. The sample is trimmed flush with the ends of the cylinder. If the sample is disturbed during trimming or a large particle is encountered during driving, the sample is discarded and another one is dug.

The wet weight of the sample is determined as quickly as possible after the trimming is completed. A moisture content specimen (of about 100 grams) is removed from the center of the sample. Compute the moisture content.

- Below surface. Samples from below the surface require digging a hole to reach the desired level. Shallow holes may be dug with a shovel until the desired depth is reached. For deeper holes the auger is used to dig the hole to the required level. For the loose material a drive sampler is used that will reach the level, and a sample is taken by driving the top of the cylinder about ½ inch below the surface. The cylinder is removed and the sample is trimmed. Determine wet weight, dry weight, and moisture content.

Calculations

The in-place dry density is expressed as—

$$\frac{\text{dry weight of soil (pounds per cubic foot)}}{\text{volume of soil (pounds per cubic foot)}}$$

Applying the moisture content to the cylinder specimen, the dry weight is calculated as follows:

$$\text{dry weight} = \frac{(\text{weight of cylinder} + \text{wet specimen} - \text{weight of cylinder})}{1 + \dfrac{(\text{moisture content in percent})}{100}}$$

The soil volume is computed from measurements of the drive cylinder. The dry weight and soil volume are entered into the formula to obtain the dry density. Note that the drive cylinder is usually constructed to standard specification and, once the volume is computed, it can be used as a standard for the tests. By including necessary conversions, it is possible to multiply the dry weight of the sample in grams by a precomputed factor to obtain the dry density. For example: a drive cylinder having an inner diameter of 2.58 inches and a length of 14.1 inches is used to secure a sample. The empty cylinder weighs 2.65 pounds, and the cylinder containing the wet sample weighs 6.89 pounds. Moisture content of the sample is determined to be 6.4 percent. Then the dry density is computed as follows:

$$\text{dry weight} = \frac{6.89 - 2.65}{1.064} = 3.98 \text{ pounds}$$

$$\text{volume of cylinder} = \frac{4\,(2.58)^2 \times 14.1}{1728} = 0.0426 \text{ cubic foot}$$

$$\text{in-place dry density} = \frac{3.98}{0.0426} = 93.4 \text{ pounds per cubic foot}$$

Section XI. CALIFORNIA BEARING RATIO TEST

DISCUSSION

The California Bearing Ratio (CBR) method of design for flexible pavements started as an essentially empirical method. Developments and thorough testing led to the establishment of a distinct pattern for design relations. From this pattern, standard procedures evolved for constructing any desired set of CBR curves.

The Corps of Engineers adopted the method and made it applicable to the design of flexible pavements for airfields in locations where frost action is not the controlling factor.

The test procedure determines the CBR to be used in the design of a specific airfield and consists of two principal steps. First, the soil test specimens are prepared; second, the penetration test is performed upon the prepared soil samples. Although one standardized procedure has been established for the penetration portion of the test, it is not possible to establish one procedure for the preparation of test specimens since soil conditions and construction methods vary widely. The soil test specimens are prepared to duplicate the soil conditions existing (or expected to occur later) in the field.

The method of preparing the test specimens and the number of specimens depend upon the type of airfield, the soils encountered at the site, and other factors. The soil sample should be tested in the laboratory at a density comparable to the density required on the construction site. There are situations where moisture conditions are favorable and the subgrade will not accumulate moisture approaching a saturated condition. Samples should be tested at a moisture content approximating actual moisture conditions expected during the time the road or airfield is used. In all other conditions, samples are laboratory tested in a saturated condition.

Although penetration tests are most frequently performed upon laboratory specimens, they also may be performed upon undisturbed soil samples or in the field upon the soil in place.

THE CBR TEST

The basic operations for conducting the CBR test are the same regardless of variations in soil conditions and types of construction.

The test essentially measures the soil resistance to penetration prior to reaching its ultimate shearing modulus since the confining effects of the molds do exert some influence. The CBR is designated as a ratio in percent from 0 to 100 with a crushed well-graded (passing through a ¾-inch sieve) limestone serving as the 100 percent material.

Minor variations in the CBR test will cause wide variations in the results. For this reason, the step-by-step procedures are detailed. Difficulties may still arise. Material with gravel or stones does not yield entirely satisfactory results. A number of tests must be conducted to establish a reasonable average value.

The CBR values range from as low as three to as high as 80 depending on the type of soils. The fine-grained soils vary from three for organic clays to 15 for micaceous or diatomaceous silts and sands. The sand-silt-clay coarse-grained combinations range from 10 for the clayey mixtures to 40 for the gravelly and silty sands. Gravelly soils range from 20 for the clayey group to 80 for the well-graded gravels and gravel-sand mixtures. Table A-1 (Appendix A) lists the typical range for soils classified under the USCS.

APPARATUS

The equipment used for preparing and testing remolded specimens is listed below.

Some of it is illustrated in Figure 2-89. A more detailed description of the equipment can be found in MIL-STD-621A:

CBR mold

Spacer disk, 2.5 inches high

Compaction tamper, sleeve type, having a 10 pound drop weight and an 18 inch height of drop

Adjustable stem perforated base plate with tripod support and dial gage (read to 0.001 inch) attachments for measuring soil expansion. One base plate is required for each mold

Set of slotted annular surcharge weights for loading the cylinder during the test

Penetration piston with an end area of 3 square inches (1.95 inches in diameter) and long enough to pass through the surcharge weights and to penetrate the soil

CBR screw jack or testing machine capable of forcing the piston into the specimen at a rate of 0.05 inch per minute

An extensometer for measuring the rate of penetration

Other general equipment such as mixing bowls, spatulas, straightedges scales, soaking tanks or buckets, oven, and moisture content cans.

NOTE

If a 2-inch spacer is used in CBR testing the resulting change in volume and compactive effort may not meet ASTM, American Association State Highway and Transportation Officials (AASHTO) or recognized standards for the CBR test.

Preparation of Specimens

CBR values are most useful when the specimens tested duplicate the density and moisture content expected in the field. The most critical condition for most materials will exist when the maximum amount of water has been absorbed. For this reason, the CBR test is made on specimens which have soaked for four days while confined in molds under a surcharge equal to the weight of the planned base and pavement.

- Size of materials. The soil to be used is air dried and mixed thoroughly. Artificial drying may be used provided that the sample temperature does not exceed 100° ± 5 °F. Clods of soil should be thoroughly broken up, but the individual particles must not be broken. A representative amount is screened first over the ¾-inch sieve and then the No. 4 sieve. For soils that contain particles over ¾ inch in size, all of the material retained on the ¾-inch sieve is removed and replaced with an equal percentage by weight of material between the No. 4 sieve and the ¾-inch sieve. The sample is recombined and thoroughly mixed.

- Compaction of samples. Compaction tests are carried out and the data is entered on DD Form 1212, California Bearing Ratio Test Data (Figures 2-90 and 2-91). The compactive effort used and the number of compacted samples required will depend upon the soil type, weight and type of field compaction equipment, and other job conditions. Normally, compactive efforts of 12, 26, and 55 blows per layer (for five layers) are used in each of three successive compaction tests. This permits plotting the moisture, density, and CBR relationship curves. Compaction will always be accomplished in the 6-inch diameter CBR mold with a 2½-inch spacer disk in the mold beneath the sample. Since the mold is 7 inches high, the sample will be 4½ inches high. The compaction tamper is the sleeve type tamper which weighs 10 pounds and falls 18 inches.

- Soaking the sample. After the sample has been compacted and excess material has been

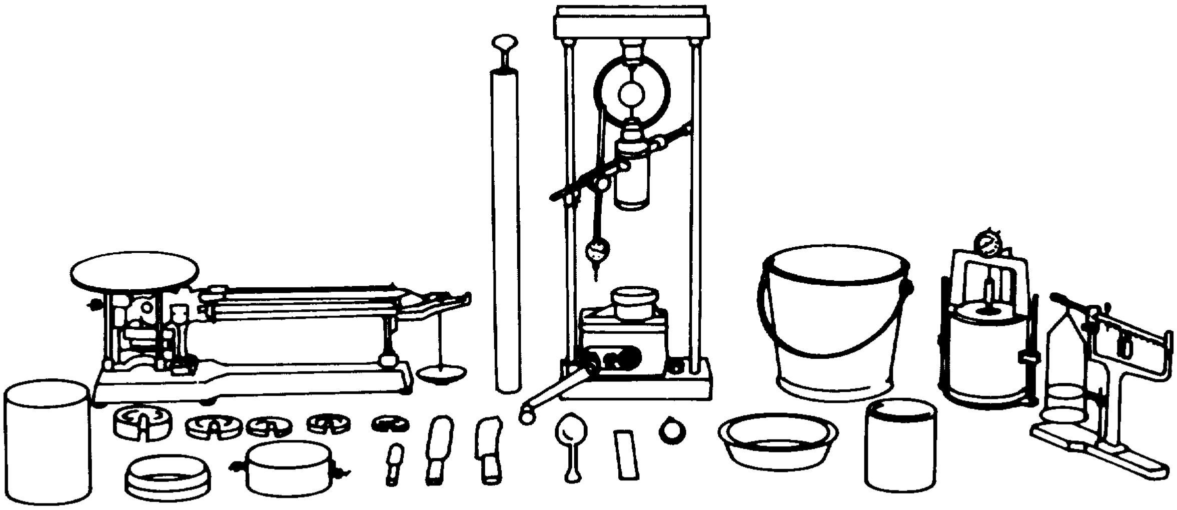


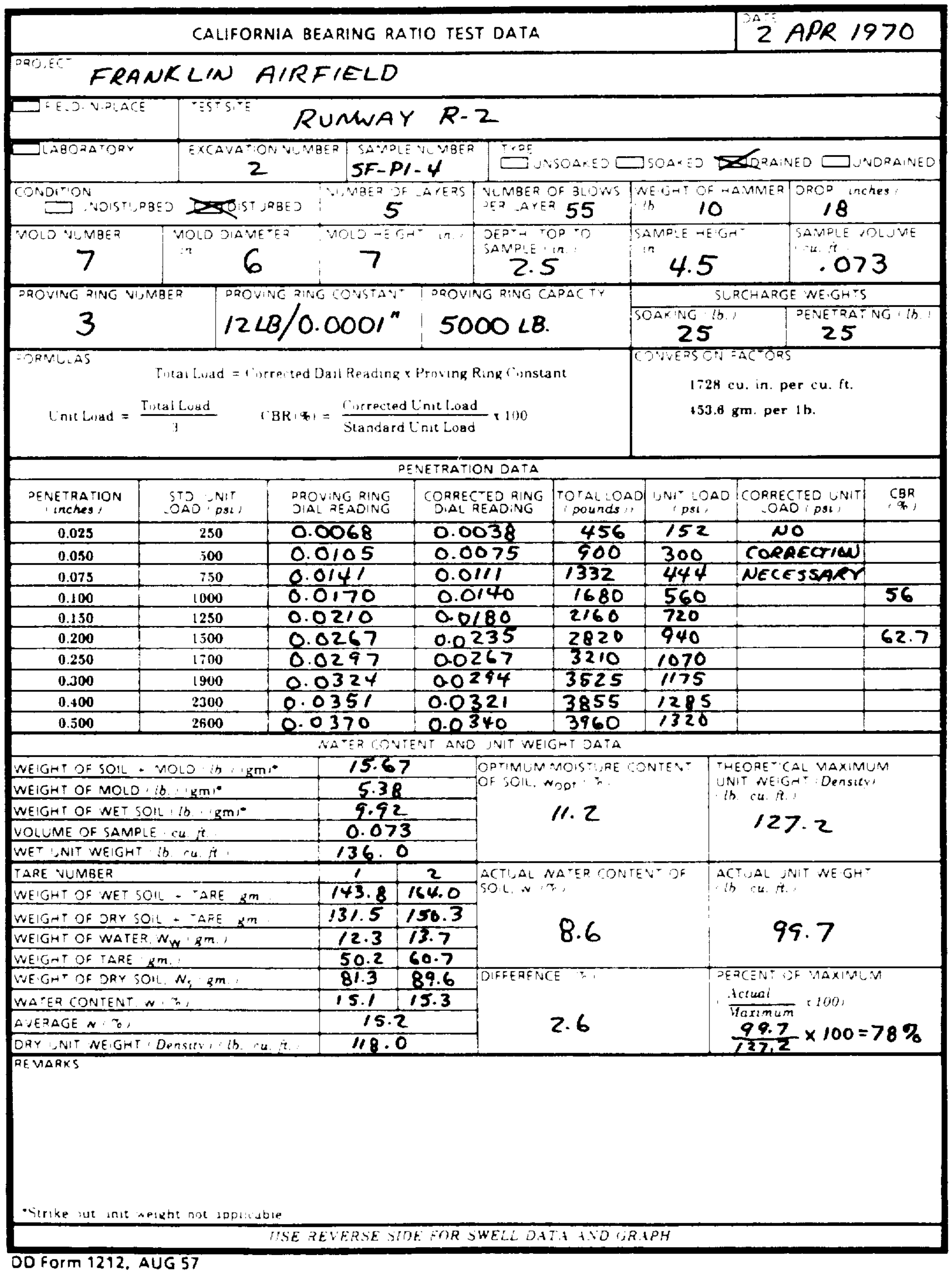
Figure 2-89. Laboratory CBR test apparatus

trimmed away so that the sample is flush with the top of a mold, the perforated base plate and the spacer block are removed. A piece of filter paper is placed over the trimmed end (top during compaction) and the perforated base is placed on this end. The top of the compacted sample will become the bottom of the soaked sample.

- Invert the mold so that the portion next to the spacer block during the compaction becomes the top surface.

- Place the adjustable stem with the perforated plate on the top surface with a filter paper underneath.

- Apply surcharge weights equal to the expected base and pavement weight (within ± 5 pounds, but in no case less than 10 pounds). One 5 pound surcharge weight is equivalent to 3 inches of overlying material.



CALIFORNIA BEARING RATIO TEST DATA

DATE 2 APR 1970

PROJECT FRANKLIN AIRFIELD

☐ FIELD-IN-PLACE

TEST SITE RUNWAY R-2

☐ LABORATORY

EXCAVATION NUMBER 2

SAMPLE NUMBER SF-P1-4

TYPE ☐ UNSOAKED ☐ SOAKED ☒ DRAINED ☐ UNDRAINED

CONDITION ☐ UNDISTURBED ☒ DISTURBED

NUMBER OF LAYERS 5

NUMBER OF BLOWS PER LAYER 55

WEIGHT OF HAMMER (lb) 10

DROP (inches) 18

MOLD NUMBER 7

MOLD DIAMETER (in) 6

MOLD HEIGHT (in.) 7

DEPTH TOP TO SAMPLE (in.) 2.5

SAMPLE HEIGHT (in) 4.5

SAMPLE VOLUME (cu. ft.) .073

PROVING RING NUMBER 3

PROVING RING CONSTANT 12LB/0.0001"

PROVING RING CAPACITY 5000 LB.

SURCHARGE WEIGHTS

SOAKING (lb.) 25

PENETRATING (lb.) 25

FORMULAS

Total Load = Corrected Dail Reading x Proving Ring Constant

$$\text{Unit Load} = \frac{\text{Total Load}}{3} \qquad \text{CBR}(\%) = \frac{\text{Corrected Unit Load}}{\text{Standard Unit Load}} \times 100$$

CONVERSION FACTORS

1728 cu. in. per cu. ft.

453.6 gm. per lb.

PENETRATION DATA

PENETRATION (inches)	STD. UNIT LOAD (psi)	PROVING RING DIAL READING	CORRECTED RING DIAL READING	TOTAL LOAD (pounds)	UNIT LOAD (psi)	CORRECTED UNIT LOAD (psi)	CBR (%)
0.025	250	0.0068	0.0038	456	152	NO	
0.050	500	0.0105	0.0075	900	300	CORRECTION	
0.075	750	0.0141	0.0111	1332	444	NECESSARY	
0.100	1000	0.0170	0.0140	1680	560		56
0.150	1250	0.0210	0.0180	2160	720		
0.200	1500	0.0267	0.0235	2820	940		62.7
0.250	1700	0.0297	0.0267	3210	1070		
0.300	1900	0.0324	0.0294	3525	1175		
0.400	2300	0.0351	0.0321	3855	1285		
0.500	2600	0.0370	0.0340	3960	1320		

WATER CONTENT AND UNIT WEIGHT DATA

WEIGHT OF SOIL + MOLD (lb.) (gm)*	15.67	
WEIGHT OF MOLD (lb.) (gm)*	5.38	
WEIGHT OF WET SOIL (lb.) (gm)*	9.92	
VOLUME OF SAMPLE (cu. ft.)	0.073	
WET UNIT WEIGHT (lb. cu. ft.)	136.0	
TARE NUMBER	1	2
WEIGHT OF WET SOIL + TARE (gm.)	143.8	164.0
WEIGHT OF DRY SOIL + TARE (gm.)	131.5	150.3
WEIGHT OF WATER, W_W (gm.)	12.3	13.7
WEIGHT OF TARE (gm.)	50.2	60.7
WEIGHT OF DRY SOIL, W_s (gm.)	81.3	89.6
WATER CONTENT, w (%)	15.1	15.3
AVERAGE w (%)	15.2	
DRY UNIT WEIGHT (Density) (lb. cu. ft.)	118.0	

OPTIMUM MOISTURE CONTENT OF SOIL, w_{opt} (%) 11.2

THEORETICAL MAXIMUM UNIT WEIGHT (Density) (lb. cu. ft.) 127.2

ACTUAL WATER CONTENT OF SOIL, w (%) 8.6

ACTUAL UNIT WEIGHT (lb. cu. ft.) 99.7

DIFFERENCE (%) 2.6

PERCENT OF MAXIMUM $\left(\frac{\text{Actual}}{\text{Maximum}} \times 100\right)$

$\frac{99.7}{127.2} \times 100 = 78\%$

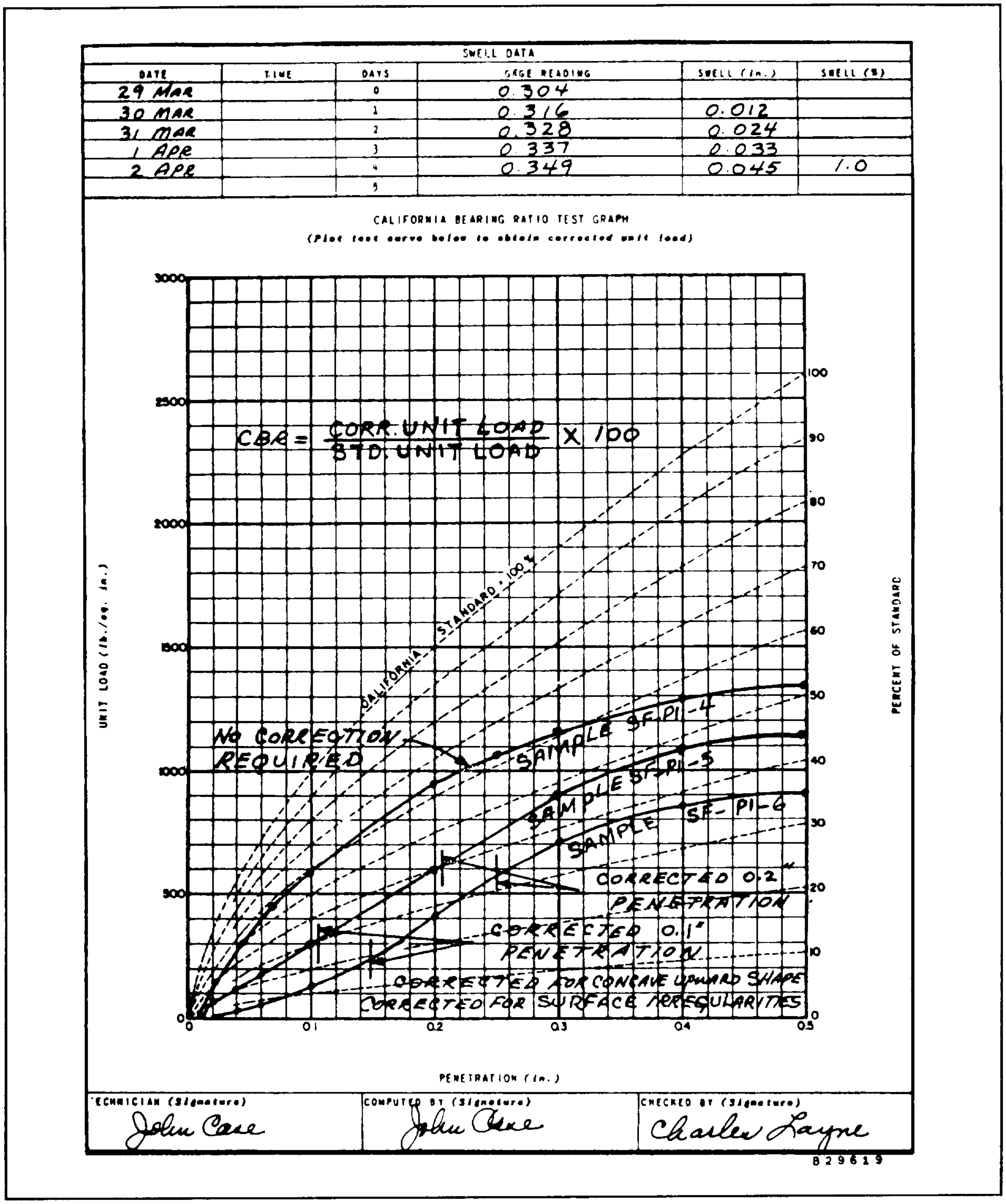
REMARKS

*Strike out unit weight not applicable

USE REVERSE SIDE FOR SWELL DATA AND GRAPH

DD Form 1212, AUG 57

Figure 2-90. Data sheet, CBR test data



SWELL DATA

DATE	TIME	DAYS	GAGE READING	SWELL (in.)	SWELL (%)
29 MAR		0	0.304		
30 MAR		1	0.316	0.012	
31 MAR		2	0.328	0.024	
1 APR		3	0.337	0.033	
2 APR		4	0.349	0.045	1.0
		5			

CALIFORNIA BEARING RATIO TEST GRAPH

(Plot test curve below to obtain corrected unit load)

UNIT LOAD (lb./sq. in.)
3000
2500
2000
1500
1000
500
0
PENETRATION (in.)
0
0.1
0.2
0.3
0.4
0.5
PERCENT OF STANDARD
100
90
80
70
60
50
40
30
20
10
0
CBR = CORR. UNIT LOAD / STD. UNIT LOAD X 100
CALIFORNIA STANDARD = 100 %
NO CORRECTION REQUIRED
SAMPLE SF-PI-4
SAMPLE SF-PI-5
SAMPLE SF-PI-6
CORRECTED 0.2" PENETRATION
CORRECTED 0.1" PENETRATION
CORRECTED FOR CONCAVE UPWARD SHAPE
CORRECTED FOR SURFACE IRREGULARITIES

TECHNICIAN (Signature)	COMPUTED BY (Signature)	CHECKED BY (Signature)
John Case	John Case	Charles Layne

829619

Figure 2-91. Reverse of data sheet, CBR test data

- Enter the amount of the surcharge used in the appropriate space on DD Form 1212 (Figure 2-90, page 2-124).

- Immerse the mold in water in a bucket or other suitable container to allow a free access of water to the top and bottom of the specimen, as illustrated in Figure 2-92. The adjustable stem extending from the perforated swell plate is set so that the tripod dial can travel in either direction (the sample may consolidate, or it may swell). In order to register consolidate, the swell gage must be turned initially one complete revolution so that the 0 day reading is + 0.100. Therefore, one can determine if the sample is consolidating.

- Use the tripod and extensometer to obtain an initial reading for swell measurements and record it on the top line on the reverse of DD Form 1212.

- Normally, the required soaking period is 4 days. A shorter immersion period for pervious soils is permissible, if it is apparent that a maximum degree of saturation has been obtained. Take dial readings at 1 day intervals during the soaking period.

- When the soaking period is ended, read the extensometer to determine the amount of consolidation or swell. The amount of swell or consolidation is computed as a percent of the initial height of the specimen. More than 3 percent swell is considered excessive.

- After soaking the sample, free surface water is removed and the specimen is allowed to drain for about 15 minutes prior to conducting the penetration portion of the test. Care must be taken not to disturb the surface of the specimen during the removal of free water. It may be necessary to tilt the sample. The surcharge weight and the perforated swell plate are removed, and the specimen is weighed. This weight is entered in the lower portion of the data sheet. For computing percent saturation, the tare weight of the mold must be determined before the start or at the conclusion of the test. The weight of the dry soil is calculated by using moisture content samples from the top and bottom of the mold, and the penetrating portion of the test is completed.

PENETRATION TEST

The penetration test is always the same, regardless of variations in specimen preparation. The following procedure also is applicable to undisturbed mold samples and in-place tests.

Apparatus

The parts of the CBR apparatus, including proving ring, penetration piston, and penetration dial, are assembled as shown in Figure 2-93.

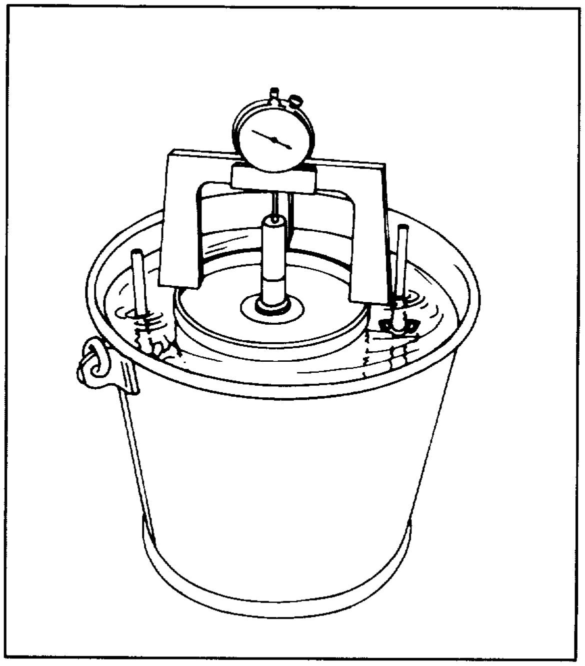


Figure 2-92. Soaking CBR test sample

Three proving rings having capacities of 2,000, 5,000, and 7,000 pounds are included in the equipment for this test. The total load on the penetration piston should be estimated on the basis of the expected CBR. The proper proving ring should be selected so that its capacity will not be exceeded, since overloading can do permanent damage to a proving ring. Standard unit loads on the piston at 0.3 inch, 0.4 inch, and 0.5 inch penetration are 1,900, 2,300, and 2,600 pounds per square inch, respectively.

Procedure

- Surcharge. A penetration surcharge is applied and must be sufficient to produce an intensity of loading equal to the weight of base and pavement. To prevent the soil from being squeezed into the hole of the surcharge weights, one 5 pound annular disk, if available, is placed on the sample; the penetration piston is seated; and finally, the remainder of the weights are applied. It should be noted that for laboratory molded samples the penetration test is performed on the bottom of the sample as originally molded, since the sample is inverted before penetration.

- Seating piston. The penetration piston is seated by bringing it into contact with the sample with sufficient pressure to cause the load dial to register a load of one pound. This ensures a satisfactory seating of the piston and is considered as the zero load in further tests.

- Application of load. Load is applied to the penetration piston by cranking the jack at such a rate that the piston travels at 0.05 inch per minute. The extensometer is set on the pistons and mold as a guide. The mechanical jack must be calibrated for the rate of 0.05 inch per minute, or the rate of penetration may be determined by comparing the progress of the hand on the penetration dial with the second hand of the clock or watch. Readings of the proving ring dial are made at the following depths of penetration: 0.025, 0.050, 0.075, 0.1, 0.150, 0.2, 0.250, 0.3, 0.4, and 0.5 inch. Proving ring dial readings are recorded at each reading and later converted to load in pounds using the calibration sheet provided with each proving ring or the calibration constant engraved on the proving ring block.

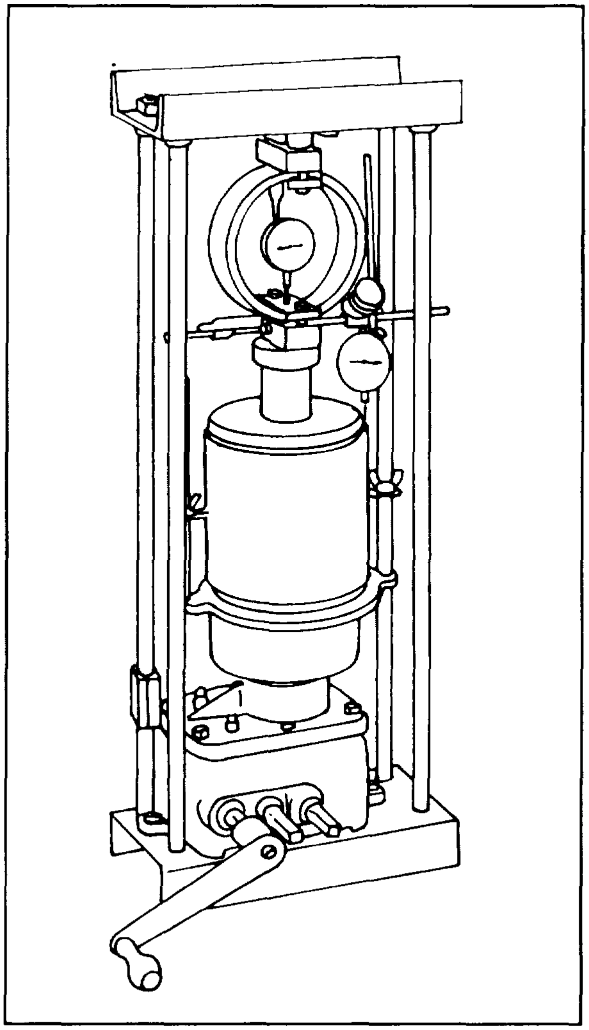


Figure 2-93. Assembled apparatus, CBR penetration test

- Moisture content. The sample is removed from the loading device and a moisture content sample is taken from the upper inch beneath the penetration piston. Another sample is taken from the bottom inch of the sample.

Calculations

- Plotting tests data. The unit load obtained in pounds per square inch (psi) by the computation indicated in Figure 2-90, page 2-124. Using the reverse side of DD Form 1212 (Figure 2-91, page 2-125), the unit load in psi is plotted against depth of penetration in inches. To obtain true penetration loads, it is sometimes necessary to adjust the zero point of the curve to correct for surface irregularities, and for an initial concave-upward shape which develops on some soils under certain conditions. The correction is made by extending downward a line tangent to the steepest portion of the curve which extends for a distance of not less than 0.1 inch in the penetration range. The point at which this line intersects the zero load line then becomes the zero penetration point of the curve. The 0.1 inch and 0.2 inch points are moved to the right on the curve and the same distance that the zero point has been advanced to the right. In Figure 2-91, necessary corrections have been made to two of the curves shown. Corrected unit loads are entered in the appropriate places on DD Form 1212. If no correction is needed, the corrected unit load will remain the same as the unit load.

- Calculation of CBR. The correct load values are determined at 0.1 inch and 0.2 inch penetrations. CBR values are obtained by dividing the loads at 0.1 inch and 0.2 inch by the standard loads of 1,000 and 1,500 psi, respectively. The standard loads of 1,000 and 1,500 psi are always constant and represent the loads required to penetrate a well graded, minus ¾ inch, crushed limestone sample compacted using CE55 compactive effort. Each ratio is multiplied by 100 to obtain the CBR in percent. The CBR is usually selected at 0.1 inch. If the CBR at 0.2 inch is greater, the test should be rerun. If check tests give similar results at 0.2 inch penetration, the CBR at 0.2 inch should be used.

- Presentation of test results. When the CBR test is completed, the results should contain the following information:

Compaction procedure (Modified AASHTO)

Compaction effort (12, 26, or 55 blows/layer)

Molding water content (moisture content prior to soaking)

Density (prior to soaking)

Soaking and penetration surcharges

Expansion of sample (percent swell)

Moisture content after soaking (saturated condition)

Density after soaking (saturated condition)

Stress-penetration curve and corrections, if necessary

Calculations and selection of corrected CBR value.

REMOLDED SAMPLE TEST

In determining CBR, remolding of the test soil produces a change in the shearing resistance of some types of soils. Natural clays, for instance, show a decrease in shearing resistance even when the moisture content remains unchanged. This loss in strength is attributed to a breakdown of the adsorbed water films and the soil's natural structure. Some soils exhibit a swelling tendency upon remolding, and this must be considered. The soils are grouped into three classes with respect to behavior during saturation: cohesionless sands and gravels, cohesive soils, and highly swelling soils. Separate procedures are given for each of the groups.

Cohesionless Sands and Gravels

This group includes the GW, GP, SW, and SP classifications (Table A-1). Cohesionless soils compact readily, and specimens should be prepared at high densities and at the range of moisture content anticipated in the field. If soaking does not lower the CBR, it may be omitted from the further tests of the same material. In the construction phase, the material should be placed in an almost saturated condition.

Cohesive Soils

This group of soils usually falls into the GM, GC, SM, SC, ML, CL, and OL classifications (Table A-1). They are tested in a manner to show their behavior over the entire range of anticipated moisture content. Compaction curves are developed for 55, 25, and 12 blows per layer. Each specimen is soaked and penetrated to develop a complete family of curves showing the relationship between density, molding water content, and corrected CBR.

Swelling Soils

This group of soils usually comprises the MH, CH, and OH classifications. The test procedures are the same as for cohesive soils. However, the objectives of the testing program are different. Tests on swelling soils are made to determine a moisture content and unit weight which will minimize expansion. The proper moisture content and density are not necessarily the optimum values as computed from the Modified AASHTO compaction tests. Since the minimum swell and the highest soaked CBR occur at a moisture content slightly wetter than the optimum value, it may be necessary to prepare high swelling samples for a wider range of moisture and density than normal. The test results can be evaluated by an experienced engineer who will select the proper values required by the design.

UNDISTURBED SAMPLE TEST

Tests on undisturbed samples are used when the base design calls for uncompacted soil, such as highly compressible clay which loses strength upon remolding, or when correlating field in-place tests to the design moisture condition. For this latter condition, duplicate samples should be tested to determine the correction necessary for the in-place tests. The reduction that occurs from 4 days of soaking is applied as a correction to the field in-place test.

Care and patience are necessary to maintain the relatively undisturbed samples in this condition. If proper lateral support is not given on the sides of the samples, erroneous CBR values will result. In fine-grained materials, molds or metal jackets are satisfactory. With samples cut or trimmed from a pedestal, paraffin of a mixture of 10 percent resin and paraffin is used to fill the annular space and offer support. For gravelly soils, the box method is desirable. Wax paper or paraffin is used to cover the sample and prevent moisture loss during transportation to the laboratory.

Soaking and penetration tests are performed after removal of the paper or paraffin from the end of the specimen and after the surface is leveled (with a thin layer of sand, if necessary).

FIELD IN-PLACE TEST

Under certain conditions, the field in-place test is used to determine the load-carrying capacity of a soil. The penetration phase of the test is the same as described previously.

Conditions for Use

The field in-place test is used under any one of the following conditions:

When the in-place density and moisture content are such that the degree of saturation (percentage of voids filled with water) is 80 percent or more.

When the material is coarse-grained and cohesionless so that the changes in moisture content will not greatly affect it.

When the material has been in place and undisturbed for several years. The moisture content in such material does not become constant but fluctuates within rather narrow ranges. The time required for the moisture content to stablize cannot be stated definitely, but the minimum time is approximately three years. The field in-place test is considered a satisfactory indicator of the load-carrying capacity in this case.

Apparatus

- The following equipment is needed to conduct the field in-place CBR tests.

- Soil test apparatus, field in-place CBR. This includes a jack to apply the load, calibrated proving rings, penetration piston with extensions, dial gages and support, surcharge weights, and surcharge plate. This apparatus is furnished in a field CBR chest as part of the soils test set. The in-place test is performed in conjunction with a loaded truck to provide resistance for the CBR screw jack.

- Beam kit, field CBR. This kit includes a loading beam, a penetration beam, and a beam clamp packed in a beam and weight chest as part of the soil test set.

- Other equipment. This includes the moisture content cans, truck jack, carpenter's level and plumb, stopwatch, and wrenches. These items are all included in the regular soil test set.

Preparation

- The apparatus is assembled on the rear bumper of a standard Army 2½ ton truck, which is positioned directly over the spot where the test is to be performed.

- The truck is jacked up so that no weight rests on the tires. A short section of 6 inch I-beam to bridge the bumpers and a clamp for securing the jack to the beam are provided.

- The ground surface is leveled and prepared by removing loose material. The apparatus is assembled and positioned under the bumper (Figure 2-94). The swivel head must be adjusted so that the penetration piston is plumb, and the device is locked in position. The dial foot must rest upon a firm, hard object which has a solid foundation far enough from the area affected by the test so it is not influenced by the reaction of the test through the soil. Except in the case of an extremely high CBR material which is thoroughly interlocked or cemented, this influence will seldom make any difference beyond a 1-foot radius.

- The steel plate (as an initial 10 pound surcharge weight) should be seated in a layer of fine sand to distribute the surcharge reaction

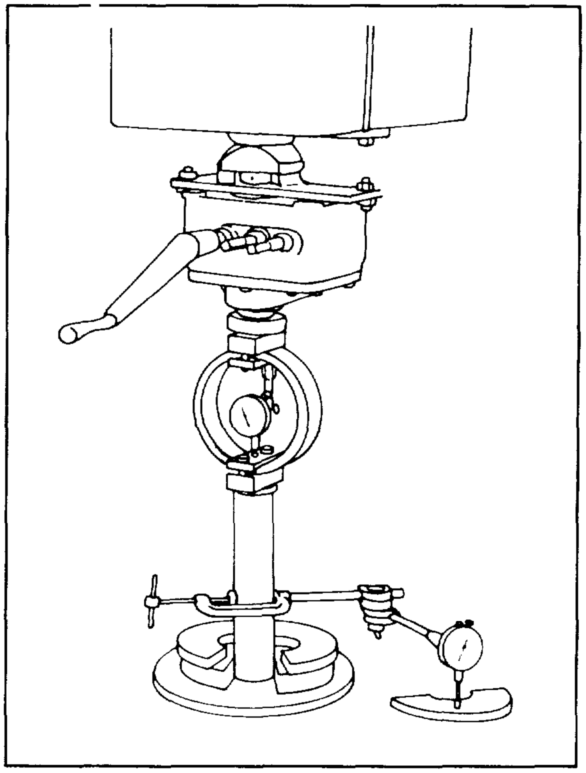


Figure 2-94. Assembled apparatus, field in-place CBR test

uniformly below all points of the plate. The penetration piston is seated, and an initial load of 1 pound is applied to establish a firm bearing. The deflection gages and the load gages are zeroed before starting the test. Surcharge weights are applied to the steel plate to establish the equivalent load intensity of the expected load (Table 2-10). The in-place also has an extensometer and expansion I-beam for registering the depths.

Procedure

With the penetration piston set, the surcharge weights added, and the load and penetration dials set to zero, the test is performed using the following steps:

1. Jack the penetration piston at a rate of 0.05 inch per minute.
2. Read the proving ring (load) readings at the following depths of penetration: 0.025, 0.050, 0.075, 0.100, 0.150, 0.200, 0.250, 0.300, 0.400, and 0.500 inch, the same as for the laboratory CBR.
3. Use the proving ring calibration and determine the bearing value in psi.
4. Plot the penetration versus load deformation curve with corrections if necessary (Figure 2-91).
5. Compute the corrected CBR in percent.
6. Obtain a sample of the soil at the point of penetration for moisture content determination.

Table 2-10. Surcharge weights, depth versus density

Depth inches	Wet Density, lb/cu ft														
	80	85	90	95	100	105	110	115	120	125	130	135	140	145	150
4	15	15	16	17	18	19	20	21	22	23	24	25	25	26	27
6	22	23	25	26	28	29	30	31	33	34	35	37	38	40	41
8	29	31	33	35	36	38	40	42	44	45	47	49	51	53	55
10	36	39	41	43	45	48	50	52	55	57	59	61	64	66	68
12	44	46	49	52	55	57	60	63	65	68	71	74	76	79	82
14	51	54	57	60	64	67	70	73	76	80	83	86	89	92	
16	58	62	65	69	73	76	80	84	87	91	95	98	102	105	
18	65	70	74	78	81	86	90	94	98	102	106	110	115	119	
20	73	77	82	86	91	96	100	105	109	114	118	123	127	132	
22	80	85	90	95	100	105	110	115	120	125	130	135	139	145	
24	87	93	98	104	109	115	120	126	131	136	142	147	152	158	
26	95	100	106	112	118	124	130	136	142	148	154	159	165	171	
28	102	108	14	121	127	134	140	146	153	159	165	172	178	184	
30	109	116	123	129	136	143	150	157	164	170	177	184	191	198	

*30 lb surcharge on surface and above line.

**90 lb surcharge is maximum surcharge that will be used for ordinary test. In special cases additional weight may be needed, therefore surcharge weight up to 198 lbs has been computed.

7. Repeat the procedure in steps 1 through 6 two or more times (for total of three tests) in the same type of soil.

If the results do not show reasonable agreement, three additional tests should be made. The numerical average of the six tests is used as the CBR at that location. Reasonable agreement is designated as the following:

CBR Range	Permissible Tolerance
Less than 10	3
10-30	5
30-60	10
Above 60	unimportant

For example: Test results of 6, 8, and 9 are reasonable and average 8. Test results of 23, 18, and 20 are reasonable and average 20. Values below 20 are rounded off to the nearest unit. Values above 20 are rounded off to the nearest 5 units.

PRESENTATION AND ANALYSIS OF CBR DATA

The CBR value, molding water content, and dry density for each speciment can be presented in several ways that facilitate analysis. These are illustrated in DD Form 2463 (California Bearing Ratio (CBR) Analysis) as shown in Figure 2-95, pages 2-137 - 2-141.

Test Program for Nonswelling Soils

This test program applies to the majority of the soils used in construction. As Table 2-11 indicates, soils which fall into this group might be used as compacted subgrade, select, or subbase materials depending upon their strengths and location regarding the construction site. To illustrate the methods of evaluating the design CBR, the data given on the CBR analysis sheet for Airfield Delta 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 9.2 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. Note that for this soil and the limits used, the greatest assured strength does not occur for the 4 percent moisture content range centered on OMC. This occurs for many soils having an appreciable clay content. Also note that the analysis is based upon an initial selection of density limits. The engineer may find it better to evaluate other density limits that meet the minimums given in Table 2-12, page 2-134 to see if an adjustment to these limits yields greater strengths. To develop the data, the soils analyst must do the following steps:

1. Establish the OMC of the soil at 55 blows per layer. This is done using the standard CE55 compaction test. For this example, OMC equals 14 percent.

2. Establish a moisture range for CBR investigation. The moisture range for CBR is OMC ± 4 percent. This is a time saving guide as experience shows that the maximum CBR will normally occur at compaction moisture contents within this range and that testing soils beyond these limits is wasted effort. For this example, the moisture content range of investigation is 10 to 18 percent.

3. Compact the samples within the moisture content range of investigation at different levels of compactive effort. This allows for evaluation of soil strength when field placement is other than 100 percent CE55. The levels of compactive effort selected in the laboratory are based on the compaction requirements (Table 2-12). If the soil is to be used as compacted subgrade or select material, it will be placed at less than 100 percent CE55. Laboratory compactive efforts of

Table 2-11. Summary of CBR test programs

Test Program	Type Soil Normally Tested*	Compaction Blows/Layer	Probable Use of Test Results
Remolded Laboratory Tests			
Nonswelling soils	All except CH, MH, OH, GW, GP, SW & SP	For CBR ≥ 20 26, 55, 72 For CBR > 20 12, 26, 55	For compacted subgrade, select, and subbase material
Swelling soils	CH, MH & OH	12, 26, 55	Low quality compacted subgrades
Free draining soils	GW, GP, SW & SP	26, 55, 72	Compacted subgrade, select, and subbase material
Tests for Insitu Strengths			
Field CBR	All	N/A	Uncompacted subgrade
Undisturbed	CH, MH & OH	N/A	Uncompacted subgrade - allows evaluation of swell

*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.

12, 26, and 55 blows per layer are usually selected. If the soil is a very high quality subgrade (CBR 20) or a subbase, the laboratory tests should include samples compacted in excess of 55 blows per layer. Normally 26, 55, and 72 blows per layer are adequate. The only stipulations placed on the levels of compactive effort are that a 55 blow per layer compaction curve be obtained and that data be developed at two other levels of compactive effort encompassing the specified placement densities. In this example the soil is a CL (cohesive) subgrade, so samples were compacted at 12, 26, and 55 blows per layer.

4. Soak the samples and measure the swell. The first assumption from Table 2-11 was that this is a nonswelling soil. Note that this is based upon USCS classification and is only an approximation. The swell must be monitored to verify that the soil being tested is in fact nonswelling.

5. Perform CBR penetration tests and determine the corrected CBR for each sample. This is done according to the technique discussed in Section XI. Note that accumulating the required data involves a considerable amount of work. At a minimum, 15 molds (five per level of compactive effort) must be made. In this example 21 molds were compacted, soaked, and then penetrated. The results are compiled on the data summary sheet.

6. Plot the data on graphs of dry density versus molding moisture content and corrected CBR versus molding moisture content.

Table 2-12. Summary of compaction requirements

Material	Percentage Compaction of Materials with Design CBR Values of 20 and Above
Base course	No less than 100% of CE55 maximum density
Subbase and subgrade	No less than 100% of CE55 maximum density
	Percentage 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 CE55 maximum density. Cohesive fill will not be placed at less than 90% of CE55 maximum density.

Note: A cohesive soil is one with a PI above 5. A cohesionless soil is one with a PI ≤ 5.

7. Reformat the data on the CBR family of curves data sheet. 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 this data may be transferred directly from the data summary table. The remainder must be interpolated from the CBR/dry density versus molding moisture content graphs.

8. Plot the CBR family of curves. This step places the laboratory data in a form which lends itself to analysis. The trends of strength variation is determined as the moisture content and dry density change. The blank places between actual data points are filled in. Drawing a CBR family of curves involves considerable practice, numerous attempts, and subsequent adjustments. The example shows that the available data has been plotted on a graph of corrected CBR versus molded dry density for constant moisture contents. For low molding water contents (from 10 to 12 percent) there was an increase in strength with dry density, and at high moisture contents (16, 17, and 18 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.

9. Proceed with engineering analysis. The work done to this point is the job of the laboratory technician. However, it is the responsibility of the engineer to insure that the CBR data is properly obtained and presented. The engineer must analyze the data and understand how the results affect the design and economic factors. Before considering the details of the analysis, it is possible to observe two points of interest about the soil being used for the example. First, the maximum strength of this soil does not occur at OMC but at 1.4 percent dry (see corrected CBR versus molding water content). Second, the CBR family of curves shows that, for some moisture contents, the soil loses strength as the dry density or level of compactive effort increases. The impact of these two factors is important because, if maximum strength from this soil is desired, that strength might be achieved at a moisture content dry of OMC and at some level of compactive effort less than CE55. The reasons for these two phenomena are highly speculative. It is essential that the engineer realize that soil placement at OMC and the most compactive effort possible are not always the answers to good construction.

10. Establish a density range at which soil will be placed in the field. Minimum limits for field compaction requirements are based on settlement criteria (Table 2-12). To facilitate construction, it is common to specify a reasonable range of densities which can be economically obtained and then examine the strength values which 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. Experience shows that a 5 percent range (90 to 95 percent of maximum CE55 dry density) is reasonable. Another factor is the actual field compaction experience obtained from either a test strip or prior construction. Such data can be accumulated by measuring the in-place dry densities for different numbers of passes with the available compaction equipment and determining the point where additional passes give little increase in density. For this example the soil must be placed at least 90 percent of maximum CE55 dry density (Table 2-12). Assuming that experience with the soil shows that 93 to 98 percent of maximum CE55 dry density can be achieved with reasonable effort, the limits of densities in pounds per cubic feet (pcf) will be the following:

$$0.93 \times 130 = 120.9 \text{ pcf}$$
$$0.98 \times 130 = 127.4 \text{ pcf}$$

These limits can be imposed on the CBR family of curves by drawing two vertical lines at 120.9 and 127.4 pcf.

11. Determine the assured CBR values between the specified density limits. If the builder is allowed to place the soil between 120.9 and 127.4 pcf, this step involves determining the CBR values obtained for each moisture content. The change in CBR for any specific moisture content line between the two density limits shows that a range of strengths is possible. Since the builder will be allowed to place the soil anywhere between the established density limits, the CBR value selected as a potential design strength should represent the worst case. Using 10 percent moisture as an example, the CBR at 120.9 pcf is 8.5. At 127.4 pcf the CBR is a maximum of 11.3.

Of the two, the 8.5 is the minimum strength for the specified density range. This procedure was followed for the remaining moisture contents, and the results are tabulated as follows.

w(moisture)	CBR
10	8.5
11	9.2
12	10.0
12	10.3
13	11.3
14	13.6
15	10.3
16	7.4
17	5.4
18	4.2

12. Determine the CBR values for potential moisture content specification ranges. Like the density range, a moisture content range which 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. The engineer may specify a smaller range 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 example problem uses a 4 percent specification range. One possible range is 10 to 14 percent. If the engineer specified that the soil be placed within these limits, the worst possible strength would be a CBR of 8.5 (see tabulation under step 10). Such an analysis can be done for the other possible 4 percent ranges, and the result is as follows:

w(range)	CBR
10-14	8.5
11-15	9.2
12-16	7.4
13-17	5.4
14-18	4.2

13. Select the moisture content range which gives the greatest design CBR. In steps 10 and 11 CBR values were selected, assuming that the engineer is going to allow the builder to place the soil anywhere between potential moisture content and density specification limits. Now the engineer selects the set of limits he desires. The tabulation in step 11 shows that the soil, if placed between 11 and 15 percent moisture, will give the largest of the possible CBR values. Thus, 9.2 becomes the design CBR, and the field quality control measures to be used to ensure that a CBR of 9.2 is achieved are; compaction from 93 to 98 percent of maximum CE55 dry density or; 120.9 to 127.4 pcf.

Test Program for Swelling or Expansive Soils

There is a small group of soils which expand objectionably after being compacted and saturated. This presents a problem in pavement design as this expansion or swell can damage the structure through reverse-settlement. The measure of swell used by the Corps of Engineers is the free swell index which is expressed as a percentage of the initial sample height. Objectionable swell is defined as that in excess of 3 percent. The expansive nature of a soil is mainly due to the type of clay minerals present. Montmorillonitic clay, for example, 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 ability to expand or shrink as water is taken in or removed from the soil.

Experience shows that the PI is an excellent indicator of expansive soils. The following guidelines are suggested.

Swell Potential

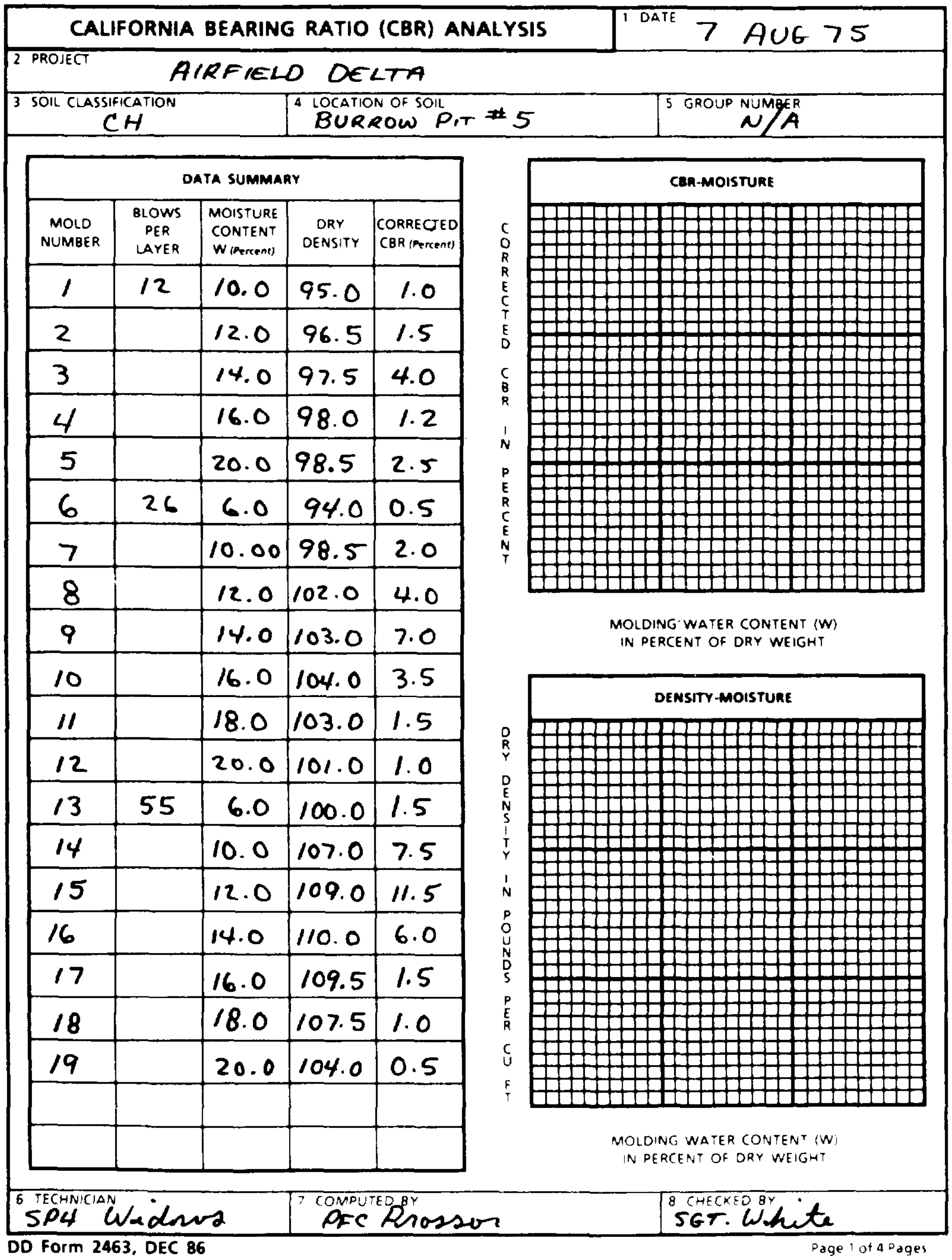
PI	0-14	not expansive
PI	14-25	marginal
PI	25-40	critical
PI	40	highly critical

Although a high PI does not guarantee that the soil is expansive, critical soils should be checked more closely for swell tendencies.

Table 2-11, page 2-133) indicates that the potentially expansive soils by USCS classification are CH, MH, and OH. The test procedure to determine a design CBR for an expansive soil is similar to that discussed for the nonswelling soil, but the objective is different. For nonswelling soils the object was to find the greatest assured CBR value for some range of densities and moisture contents. The object of the test program for the expansive soils is to find the moisture content ranges which will prevent objectionable swell and provide the highest soaked CBR. Generally, the minimum swell and the highest soaked CBR will occur at a molding moisture content wet of OMC. The following is an example of such a test for a CH soil taken from the subgrade at the proposed Airfield Delta (Figure 2-95, page 2-141). As most of the steps are similar to those developed for the case of nonswelling soils, only differences will be discussed.

Establish the OMC of the soil at 55 blows per layer, then establish a moisture range for CBR investigation. The OMC ± 4 percent range of investigations may not apply. It may be necessary to prepare samples over a wider range of moistures with most of the work being done on samples wet of OMC. In the example, samples were prepared over a range of OMC ± 8 percent. It will be illustrated later that much of the laboratory work done on samples dry of OMC is not essential. Continue the test using the following steps:

1. Compact the samples within the moisture content range of investigation at different levels of compactive effort. As the type of soil being tested will be cohesive and have a CBR less than 20, Table 2-12, page 2-134, indicates compaction must be at least 90 percent of maximum CE55 dry density. An upper limit can be established as expansive soils are very difficult to compact at levels greater than 100 percent of maximum CE55 dry density. Laboratory compactive efforts of 12, 26, and 55 blows per layer are adequate for nearly all cases.



CALIFORNIA BEARING RATIO (CBR) ANALYSIS

1 DATE: 7 AUG 75

2 PROJECT: AIRFIELD DELTA

3 SOIL CLASSIFICATION: CH

4 LOCATION OF SOIL: BURROW PIT # 5

5 GROUP NUMBER: N/A

DATA SUMMARY

MOLD NUMBER	BLOWS PER LAYER	MOISTURE CONTENT W (Percent)	DRY DENSITY	CORRECTED CBR (Percent)
1	12	10.0	95.0	1.0
2		12.0	96.5	1.5
3		14.0	97.5	4.0
4		16.0	98.0	1.2
5		20.0	98.5	2.5
6	26	6.0	94.0	0.5
7		10.00	98.5	2.0
8		12.0	102.0	4.0
9		14.0	103.0	7.0
10		16.0	104.0	3.5
11		18.0	103.0	1.5
12		20.0	101.0	1.0
13	55	6.0	100.0	1.5
14		10.0	107.0	7.5
15		12.0	109.0	11.5
16		14.0	110.0	6.0
17		16.0	109.5	1.5
18		18.0	107.5	1.0
19		20.0	104.0	0.5

CBR-MOISTURE

MOLDING WATER CONTENT (W) IN PERCENT OF DRY WEIGHT

DENSITY-MOISTURE

MOLDING WATER CONTENT (W) IN PERCENT OF DRY WEIGHT

6 TECHNICIAN: SP4 Widows

7 COMPUTED BY: PFC Rosson

8 CHECKED BY: SGT. White

DD Form 2463, DEC 86

Page 1 of 4 Pages

Figure 2-95. Sample test results Airfield Delta

CALIFORNIA BEARING RATIO (CBR) ANALYSIS

1 DATE

2 PROJECT

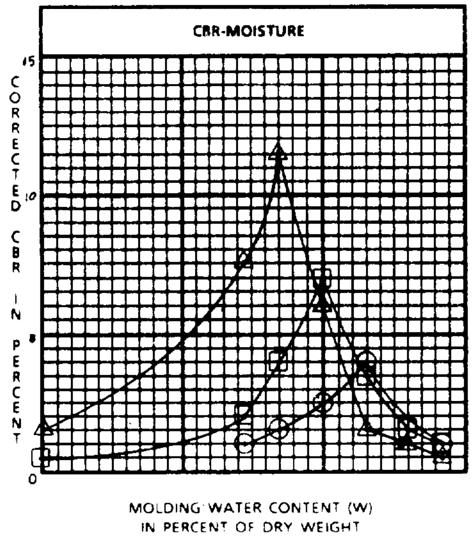
3 SOIL CLASSIFICATION

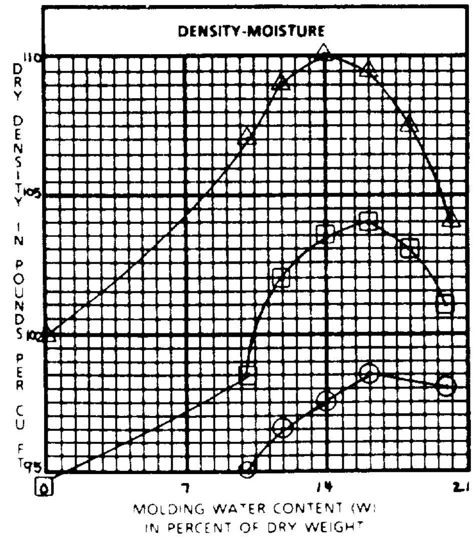
4 LOCATION OF SOIL

5 GROUP NUMBER

DATA SUMMARY

MOLD NUMBER	BLOWS PER LAYER	MOISTURE CONTENT W (Percent)	DRY DENSITY	CORRECTED CBR (Percent)





6 TECHNICIAN

7 COMPUTED BY

8 CHECKED BY

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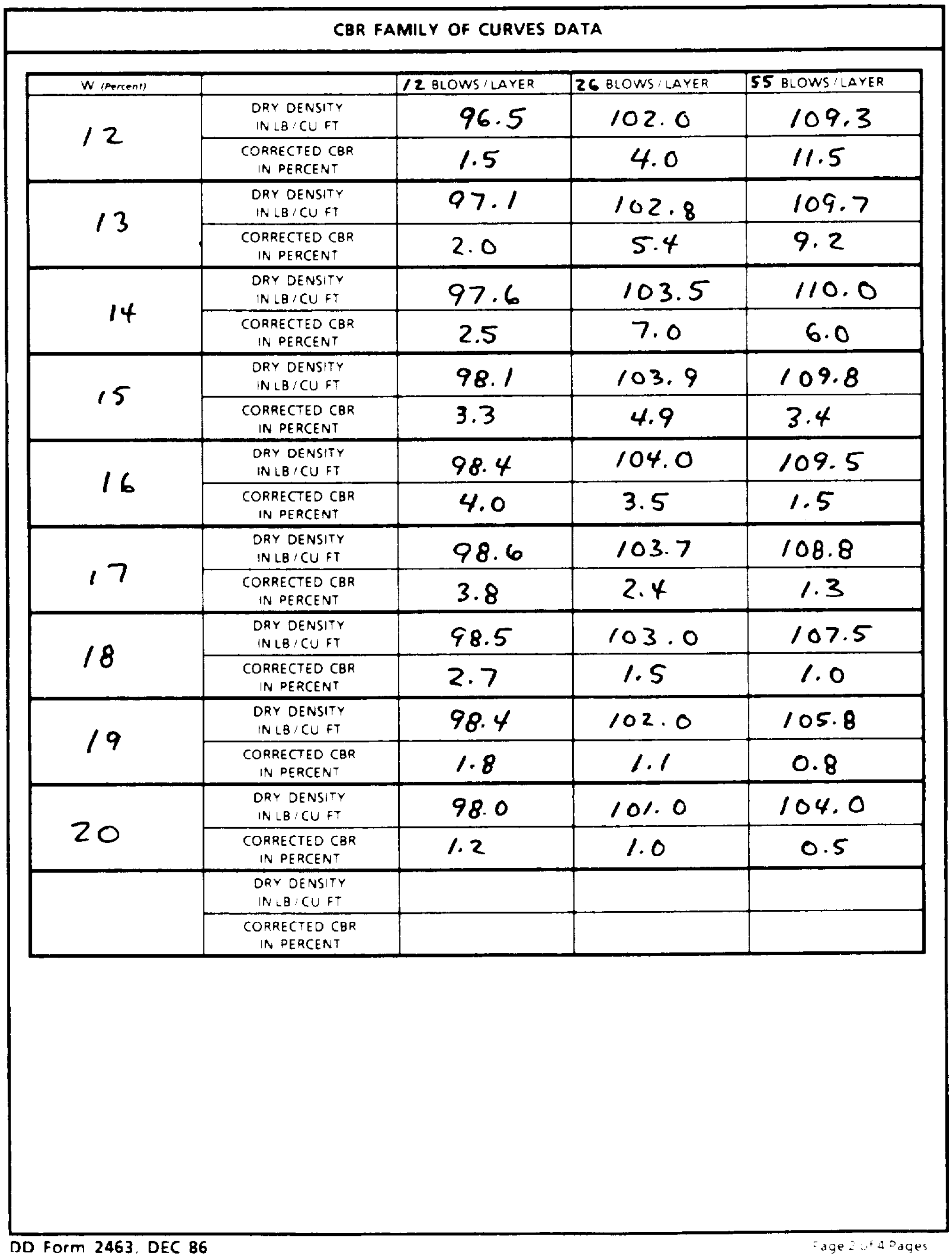
Page ' of 4 Pages

Figure 2-95. Sample test results, Airfield Delta (continued)



CBR FAMILY OF CURVES DATA

W (Percent)		12 BLOWS / LAYER	26 BLOWS / LAYER	55 BLOWS / LAYER
12	DRY DENSITY IN LB / CU FT	96.5	102.0	109.3
	CORRECTED CBR IN PERCENT	1.5	4.0	11.5
13	DRY DENSITY IN LB / CU FT	97.1	102.8	109.7
	CORRECTED CBR IN PERCENT	2.0	5.4	9.2
14	DRY DENSITY IN LB / CU FT	97.6	103.5	110.0
	CORRECTED CBR IN PERCENT	2.5	7.0	6.0
15	DRY DENSITY IN LB / CU FT	98.1	103.9	109.8
	CORRECTED CBR IN PERCENT	3.3	4.9	3.4
16	DRY DENSITY IN LB / CU FT	98.4	104.0	109.5
	CORRECTED CBR IN PERCENT	4.0	3.5	1.5
17	DRY DENSITY IN LB / CU FT	98.6	103.7	108.8
	CORRECTED CBR IN PERCENT	3.8	2.4	1.3
18	DRY DENSITY IN LB / CU FT	98.5	103.0	107.5
	CORRECTED CBR IN PERCENT	2.7	1.5	1.0
19	DRY DENSITY IN LB / CU FT	98.4	102.0	105.8
	CORRECTED CBR IN PERCENT	1.8	1.1	0.8
20	DRY DENSITY IN LB / CU FT	98.0	101.0	104.0
	CORRECTED CBR IN PERCENT	1.2	1.0	0.5
	DRY DENSITY IN LB / CU FT			
	CORRECTED CBR IN PERCENT			

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Page 2 of 4 Pages

Figure 2-95. Sample test results, Airfield Delta (continued)

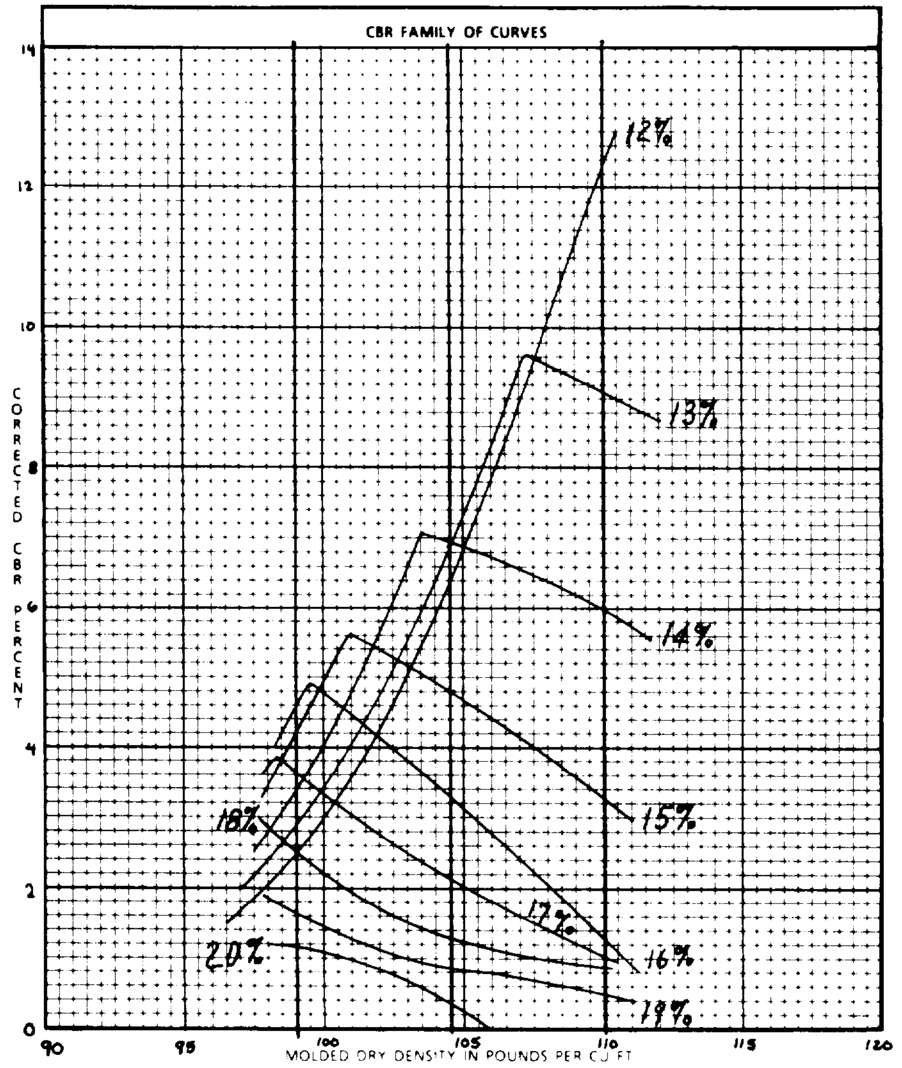

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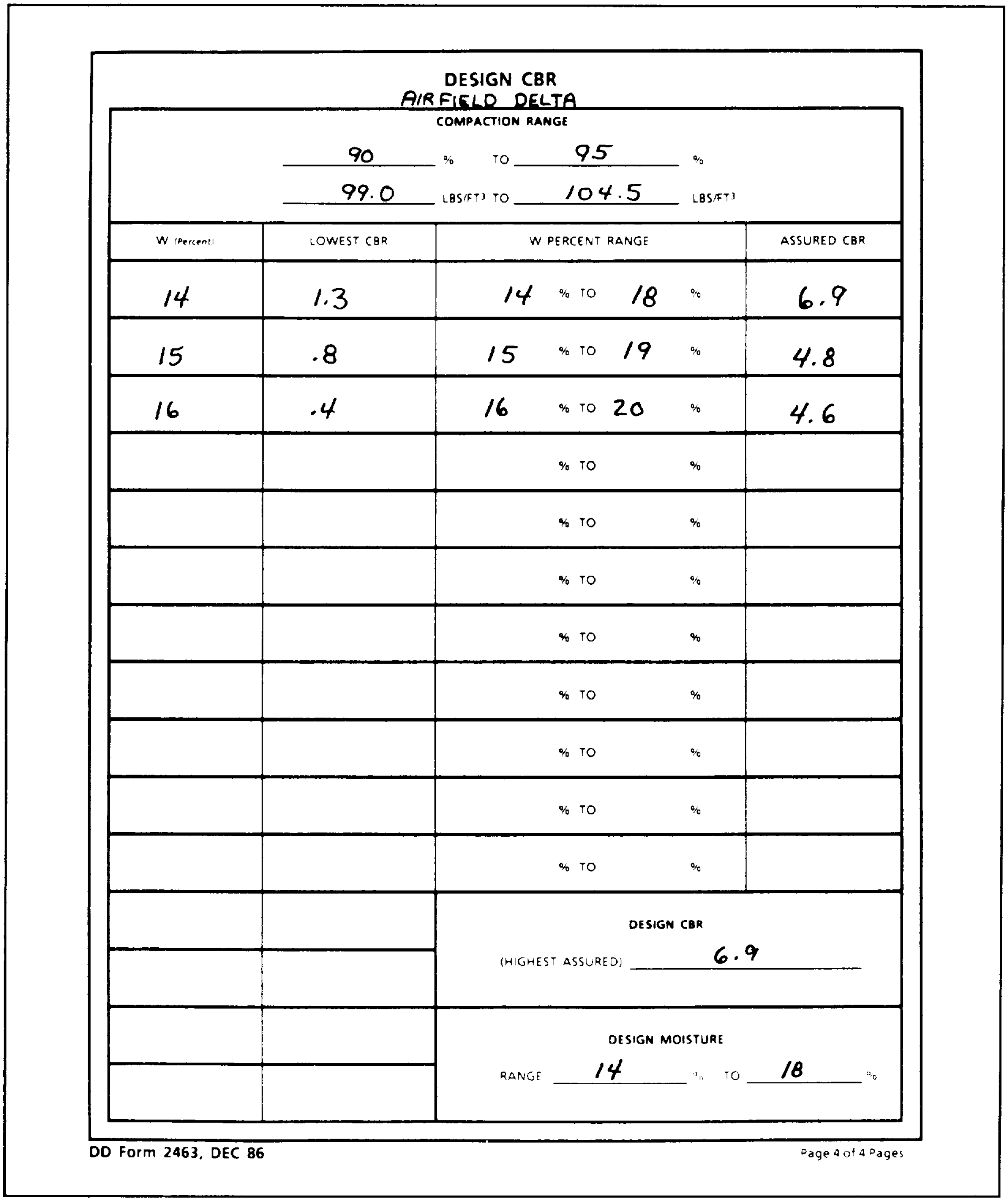
Figure 2-95. Sample test results Airfield Delta (continued)



DESIGN CBR

AIRFIELD DELTA

COMPACTION RANGE

90 % TO 95 %

99.0 LBS/FT³ TO 104.5 LBS/FT³

W (Percent)	LOWEST CBR	W PERCENT RANGE	ASSURED CBR
14	1.3	14 % TO 18 %	6.9
15	.8	15 % TO 19 %	4.8
16	.4	16 % TO 20 %	4.6
		% TO %	
		% TO %	
		% TO %	
		% TO %	
		% TO %	
		% TO %	
		% TO %	
		% TO %	

DESIGN CBR

(HIGHEST ASSURED) 6.9

DESIGN MOISTURE

RANGE 14 % TO 18 %

DD Form 2463, DEC 86

Page 4 of 4 Pages

Figure 2-95. Sample test results, Airfield Delta (continued)

2. Soak the samples and measure the swell. This step deviates from the same step for the nonswelling soil. For each sample the expansion is measured and the percent of swell is computed and plotted against the molding water content. As an example, the triangular data point at 10 percent moisture on the swell data curve (Figure 2-96) was obtained as follows:

initial sample height = 4.50 inches

height after soaking = 4.72 inches

molding water content = 10.0 percent

level of compactive effort = blows per layer

$$\begin{array}{c}\text{free swell index}\\ \text{or}\\ \text{percentage of swell}\end{array} = \frac{4.72 - 4.50}{453} \times 100 = 4.9 \text{ percent}$$

After obtaining the swell index for each mold to be penetrated, plot the points on a swell versus molding moisture content graph (Figure 2.96). A curve is then usually fit to only the CE55 data points. This particular curve shows that only soil placed and compacted at molding moisture contents of 14 percent or greater will swell 3 percent or less.

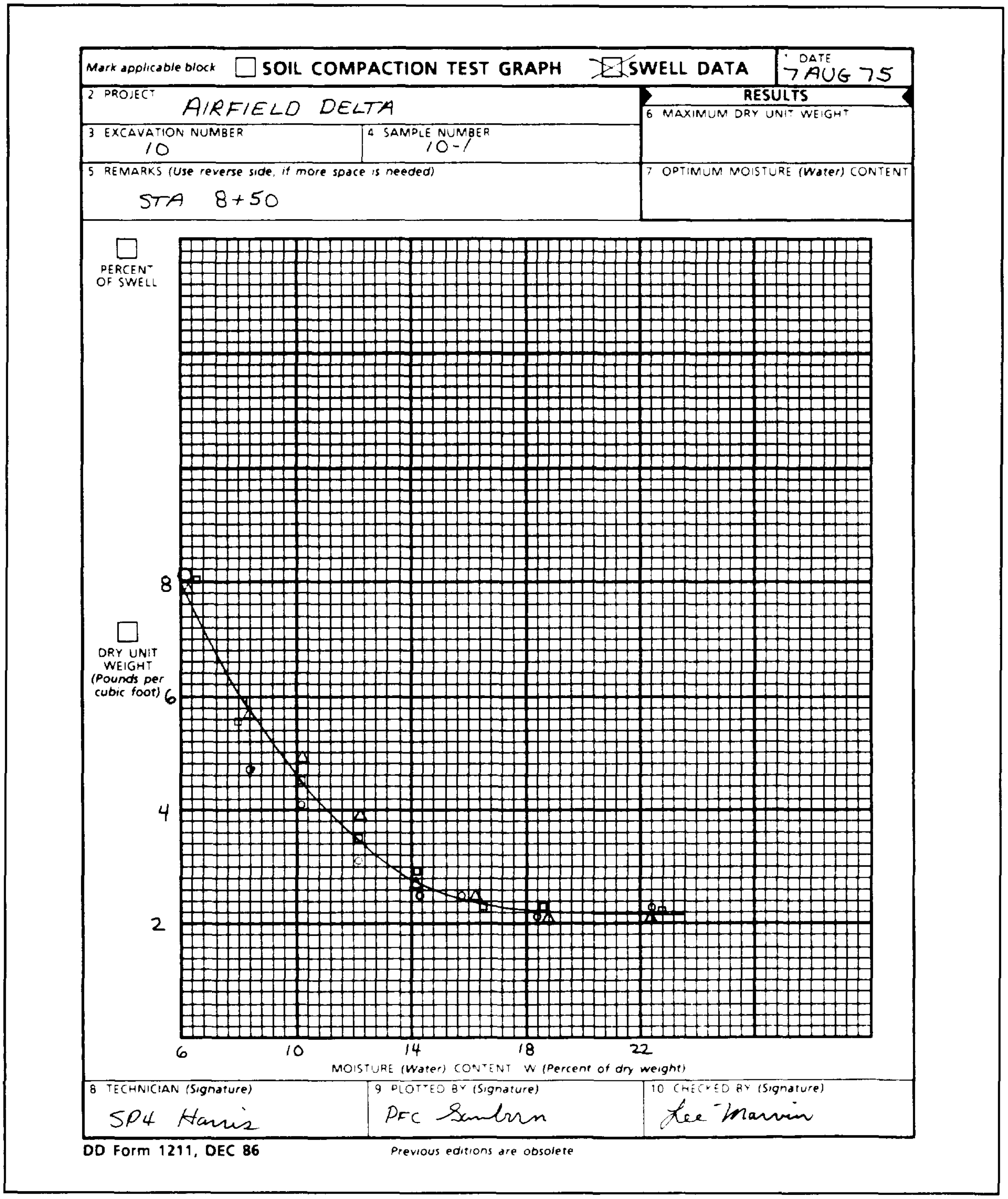
3. Perform CBR penetration tests and determine the corrected CBR for each sample.

4. Plot the data on graphs of dry density versus molding moisture content and corrected CBR versus molding moisture content.

5. Reformat the data on the CBR family of curves data sheet.

6. Plot the CBR family of curves.

7. Establish a density range at which soils will be placed in the field. This example assumes that prior experience or benefit of a test strip is not available. Hence, only the guidelines from



Mark applicable block ☐ **SOIL COMPACTION TEST GRAPH** ☒ **SWELL DATA**

1 DATE 7 AUG 75

2 PROJECT AIRFIELD DELTA

RESULTS

6 MAXIMUM DRY UNIT WEIGHT

7 OPTIMUM MOISTURE (Water) CONTENT

3 EXCAVATION NUMBER 10

4 SAMPLE NUMBER 10-1

5 REMARKS *(Use reverse side, if more space is needed)* STA 8+50

8 TECHNICIAN *(Signature)* SP4 Harris

9 PLOTTED BY *(Signature)* PFC Sanborn

10 CHECKED BY *(Signature)* Lee Marvin

DD Form 1211, DEC 86 *Previous editions are obsolete*

☐ PERCENT OF SWELL
☐ DRY UNIT WEIGHT (Pounds per cubic foot)
8
6
4
2
6
10
14
18
22
MOISTURE (Water) CONTENT W (Percent of dry weight)

Figure 2-96. Swell data curve, Airfield Delta

Table 2-12 (page 2-134) will be used. The minimum level of compaction is 90 percent of maximum CE55 dry density. Assuming a reasonable specification range, the upper limit will be set at 95 percent of maximum CE55 dry density. The actual densities are as follows:

$$(110)(.90) = 99 \text{ pcf}$$

$$(110)(.95) = 104.5 \text{ pcf}$$

As in the previous example, the above limits are shown by two vertical lines on the CBR family of curves.

8. Determine the assured CBR values between the specified density limits. As more than 3 percent swell is not acceptable, evaluation of the CBR values at moisture contents less than 14 percent is needless. The CBR values of the applicable moisture contents are as follows:

w	CBR
14	3.4
15	4.2
16	3.2
17	2.2
18	1.3
19	.9
20	.4

9. Determine the CBR values for potential moisture content specification ranges.

w (range)	CBR
14-18	1.3
15-19	.9
16-20	.4

10. Select the moisture content range which gives the greatest design CBR. Step 9 shows that the 14 to 18 percent range gives the largest CBR. Thus the design CBR is 1.3, and this value is obtained when the soil is placed at:

w = 14-18 percent

d = 99.0 - 104.5 pcf

11. Analyze the results. Note that obtaining this design value was done at the expense of strength. This technique does not provide for drying the soil to a moisture content less than the amount of placement. Should such extreme drying take place, excessive shrinkage and pavement failure might be expected. However, it is an advantage that it takes considerable effort to remove water from expansive soils and such soils will normally be protected from drying by the overlying pavement. The first thing the engineer must consider when encountering an expansive soil is to test another location. However, this is not always feasible, and this technique does not allow for determination of a design CBR at which swell is not excessive.

Expansive soils can be chemically stabilized, allowing building. The addition of small amounts of lime considerably reduces the potential for shrinkage and swell. Soils stabilization is in TM 5-330/AFM 86-3, Vol II.

Test Program for Free-Draining Soils

Determining a design CBR for this group of soils requires the least testing of the three remolded laboratory test programs. Table 2-11 (page 2-133) gives the USCS classification and the uses of the soils in this group.

The ease in testing is due to the free draining characteristics or lack of fines in the soil. The CBR analysis sheet for Borrow Pit A (Figure 2-97) shows that the density versus moisture content curves have a concaved upward shape and show maximum densities between 7 and 9 percent moisture depending upon the level of compactive effort. For each curve, there is a limiting moisture content (shown by the triangle, diamond, or circle) at which moisture above that required to fill the voids after compaction is squeezed or drained from the soil. The dashed lines of the right of the limiting moisture content 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 for such soils field placement is relatively easy. To ensure maximum dry density for any level of compactive effort, the only control measure necessary is to have more water available than that required for the maximum dry density 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 wet of the limiting 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. To arrive at a design CBR, the steps outlined in the previous examples will again be followed.

- Establish the OMC of the soil at 55 blows per layer. The OMC for this example is the limiting moisture content of 8 percent. To ensure that a free-draining soil is being tested, this curve should display a maximum dry density at a limiting moisture content.

- Compact the samples at different levels of compactive effort. Compaction curves must be made for three levels of compactive effort up to the limiting moisture contents. As these soils frequently have CBR values greater than 20, the placement compactive effort is in excess of 100 percent of maximum CE55 dry density. Therefore, 26, 55, and 72 blows per layer are usually used.

- 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.

- Perform CBR penetration tests. Only the samples at the limiting moisture contents for each level of compactive effort need 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.

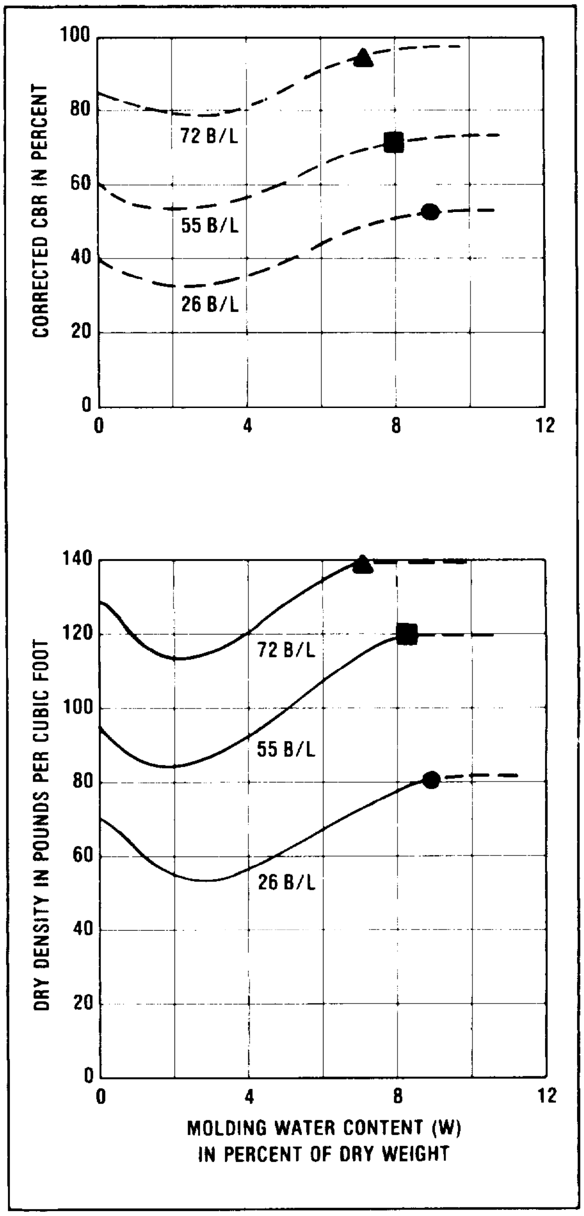


Figure 2-97. CBR analysis sheet, borrow pit "A"

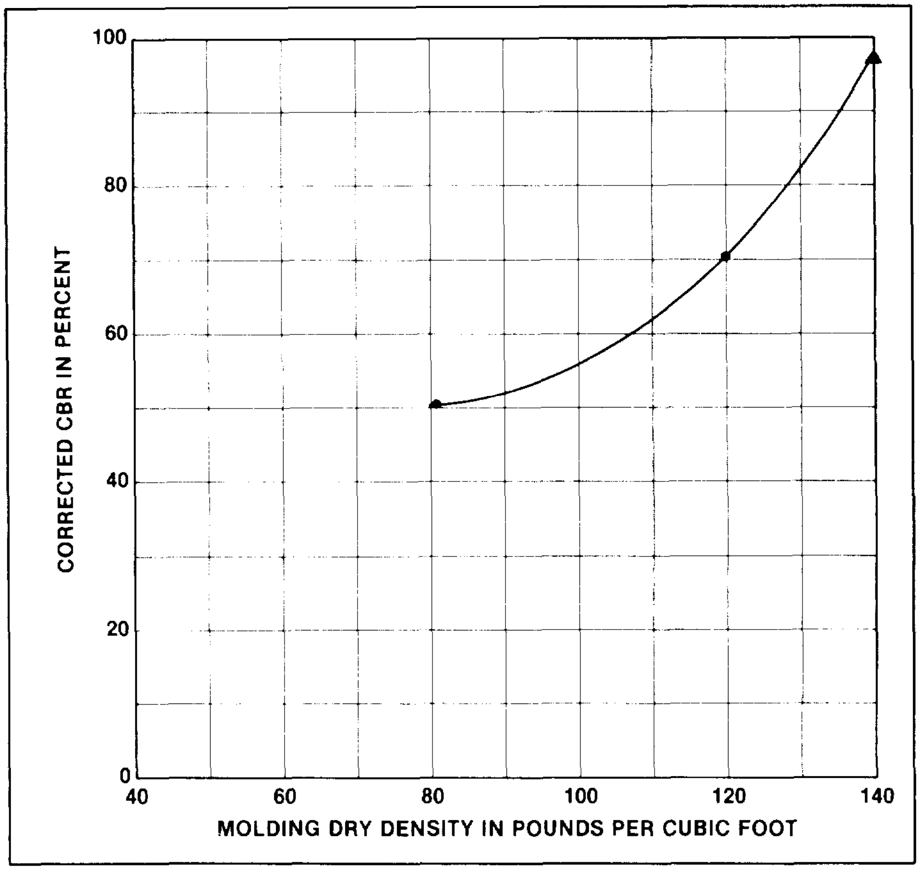


Figure 2-97. CBR analysis sheet borrow pit "A" (continued)

- Plot the data on graphs of dry density versus molding moisture content and corrected CBR versus molding moisture content. Only the plot of dry density versus molding water content is required. The corrected CBR versus molding water content graph is presented only for discussion.

- 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.

- Establish a density range at which the soil will be placed in the field. Using the criteria from Table 2-12 (page 2-134) and noting that the CBR for this soil is always greater than 20, the minimum level of compaction allowed is 100 percent of maximum CE55 dry density. Because additional information is lacking, specify 100 to 105 percent of maximum CE55 dry density. This means compaction as follows:

(1.00) (120) = 120 pcf

(1.05) (120) = 126 pcf

- Determine the design CBR and placement moisture content. The CBR family of curves show that the minimum CBR is 70 at 120 pcf. The placement moisture content necessary to ensure that this strength is obtained is 8 percent or greater.

Considerably greater CBR values can be achieved if more field compaction is applied to the soil. If this is not too costly, it may be advantageous to specify greater densities. Note that the short cuts used in this procedure depend upon the fact that the soil is actually free draining. This can be determined from the shapes of the compaction curves.

Section XII. PLATE BEARING TEST

DISCUSSION

In designing concrete pavements for roads or airfields, certain features must be considered. These include the anticipated loads, the type and amount of planned traffic, the supporting value of the subgrade, and the characteristics of the concrete. The anticipated traffic and loading are design factors and are beyond the scope of this manual. The materials testing technician may be called upon, however, to perform the plate bearing test to determine the modulus of subgrade reaction (k) of the subgrade for a concrete pavement. The modulus of subgrade reaction is the load per unit area required to produce a soil deformation of the subgrade. It is expressed in pounds per square inch per inch of deformation. A detailed discussion of the plate bearing test can also be found in MIL-STD-621A.

PREPARATION AND TEST PROCEDURES

The test is performed on representative areas, and corrections or adjustments can be made by the designer. These are based on experience or knowledge of the local conditions which can alter the load supporting value of the subgrade. Moist soils exhibit a marked reduction in modulus with an increase in moisture content. However, since the design is not too sensitive to small changes in k, a bracket or range of values is sufficient.

Equipment

The following equipment is needed to conduct the plate bearing test:

Hydraulic jack, 25,000 pound minimum capacity

Proving ring or hydraulic gage

Bearing plates, 18-inch, 24-inch, and-30 inch diameter steel plates, each 1 inch or 1½ inch thick aluminum alloy plate

Dial indicators, 0.001 inch, three required (with extensometers)

Dial support beam and clamps (A frame for three-sided support)

Load reaction beam (steel I or H beam)

Load reaction equipment having a dead load of at least 25,000 pounds.

Preparation of Test Area

A small area of subgrade should be stripped free of overburden to the elevation of the subgrade surface which will be used. It should be uncovered for a sufficient area to eliminate any surcharge or confining effects. If the subgrade is to be fill material, a test embankment about 30 inches high should be constructed after the necessary stripping. The subgrade should be compacted to the specified densities at OMC. The test should be made with the soil as close to OMC as practical. It is advisable to conduct comparative tests on the subgrade representing conditions after the pavement has been in place for some time. Methods of correction are available when it is impractical to simulate actual conditions. On granular subgrades, the natural condition of the soil should not be disturbed while the test area is being prepared. Prior to placement of the 30-inch diameter plate, all loose material and boulders which protrude more than 1 inch above the lowest point should be removed and replaced by a layer of sand or plaster of paris mixture to prevent the plate from resting on these points. The thickness of this layer should be held to the absolute minimum needed to prevent point bearing.

Loading System

Loads are applied by a hydraulic jack working against a jacking frame and through a steel bearing plate 30 inches in diameter and 1 inch thick (Figure 2-98). The load must be applied in the exact center of the plate to reduce the possibility of tilting the plate during the test. A nest of plates (30-inch, 24-inch, and 18-inch diameter) is used to assist in the centering. If cribbing is needed, it is best to place such filling members between the top plate and the jack. If a proving ring is used to measure the load, it should be placed on top of the jack. A ball joint should be used between it and the load reaction device. The load reaction device must be long enough so the its supports will be at least 8 feet from the bearing plate. Aluminum alloy plates which have the same rigidity as the 1-inch steel plates may be used if weight reduction is necessary.

Load Reaction Devices

The load reaction from the jack can be taken by bearing against a truck trailer, anchored frame, or any other device big enough for the test. Granular subgrades require a reaction of at least 25,000 pounds. For most cohesive subgrades, 10,000 pounds is sufficient.

Deformation Measuring

The movement of the plate due to the applied loads should be measured in three places spaced 120 degrees apart around the circumference of the 30-inch plate and $\frac{1}{4}$ inch from the rim. Dial gages, accurate to 0.0001 inch, are needed for granular subgrades. However, gages accurate to 0.001 inch, may be used if time readings are made.

Loading Procedure

The loading system and bearing plate should be seated by applying a load of 707 pounds (one psi) on the 30-inch diameter plate for light load pavements less than 15 inches thick or 1,414 pounds (two psi) for heavy load pavements more than 15 inches. The seating load must be allowed to remain until complete deformation of the subgrade occurs. The dial gages are read, and the values are recorded as a zero correction.

- An additional load of 7,070 pounds (10 psi) is applied and held until complete deformation has taken place. Before releasing the load, a value of k'_u (preliminary subgrade modulus) is computed from the average deflection on the rim:

$$k'_u = \frac{10 \text{ psi}}{\text{average deflection in inches}}$$

where the deflection readings on the dials at 10 psi are averaged and the units are psi/inch.

- For cohesive subgrades, where k'_u is less than 200 psi/inch, the test is considered complete, and the load may be released. Some clay soils may require a time settlement curve to determine when complete deformation has been reached. The load is held until the rate of deflection is less than 0.0002 inch per minute (0.002 inch in 10 minutes) which indicates that most of the consolidation has taken place. A field plot of average dial reading versus time on

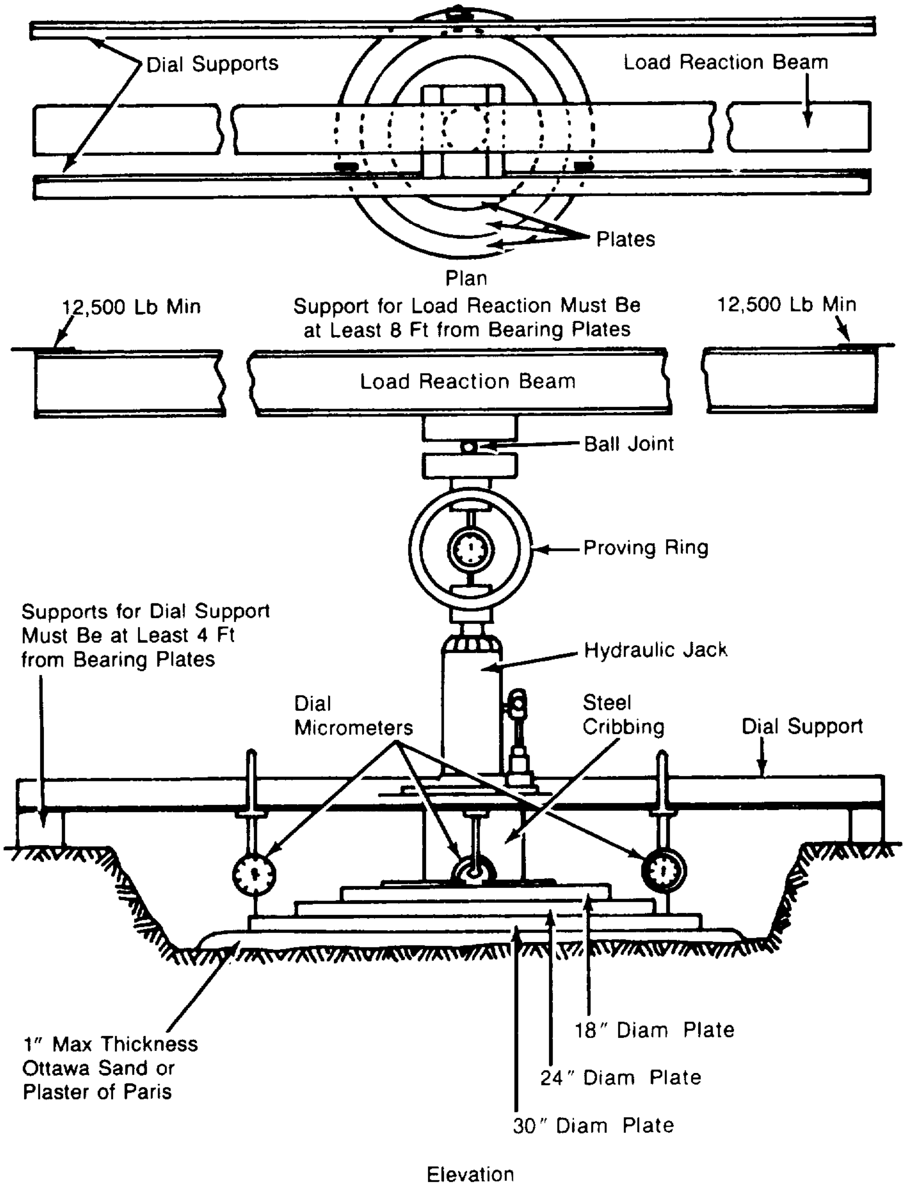

Dial Supports
Load Reaction Beam
Plates
Plan
12,500 Lb Min
Support for Load Reaction Must Be at Least 8 Ft from Bearing Plates
12,500 Lb Min
Load Reaction Beam
Ball Joint
Proving Ring
Supports for Dial Support Must Be at Least 4 Ft from Bearing Plates
Hydraulic Jack
Dial Micrometers
Steel Cribbing
Dial Support
1" Max Thickness Ottawa Sand or Plaster of Paris
18" Diam Plate
24" Diam Plate
30" Diam Plate
Elevation

Figure 2-98. Plate bearing test loading

arithmetical coordinates may be made to show settlement rates.

- For granular subgrades or some cohesive subgrades with a k'_u of 200 psi/inch or more, the load is applied in successive increments of 3,535 pounds (5 psi) to a maximum load of 21,210 pounds (30 psi). The load is not released between increments. Each increment loading is held for at least 16 minutes and until less than 0.0002 inch per minute deformation is read. The final load (30 psi) is held until almost complete deformation is reached, but not less than 16 minutes. In general, this will be when the rate of deformation is less than 0.0001 inch per minute (0.001 inch per 10 minutes) and most of the consolidation has occurred.

Unit Load Deformation Curve

A curve of unit load versus deformation should be plotted for material with a k'_u of 200 psi/inch or more. The plotted points are joined with a smooth curve (Figure 2-99). If the curve approximates a straight line between 10 psi and 30 psi loadings and goes through the "0" point, no correction is needed. If it does not approximate a straight line or the straight line does not extend through the "0" point, a straight line is drawn through the origin parallel to the straight line or the straight line portion of the curve (Figure 2-100). If no portion approximates a straight line, the correction line is drawn parallel to the average slope as determined by at least three points in the area of least curvature (Figure 2-100). A new value of deformation is then read at the 10 psi point on the corrected curve.

EVALUATION OF RESULTS

Deflection Correction

There is some bending in the bearing plate when load is applied, even when a nest of plates is used, and bending results in a greater deflection at the center than on the rim. Since the deflection is measured at the rim, the measurement is smaller than if taken at the center, and the k'_u is higher. For k'_u of less than 100, the bending correction is negligible and can be omitted. For k'_u between 100 and 200, the correction

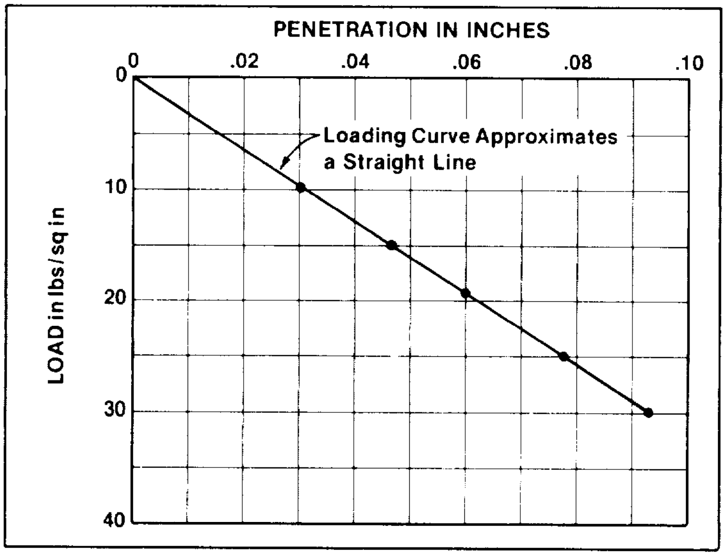
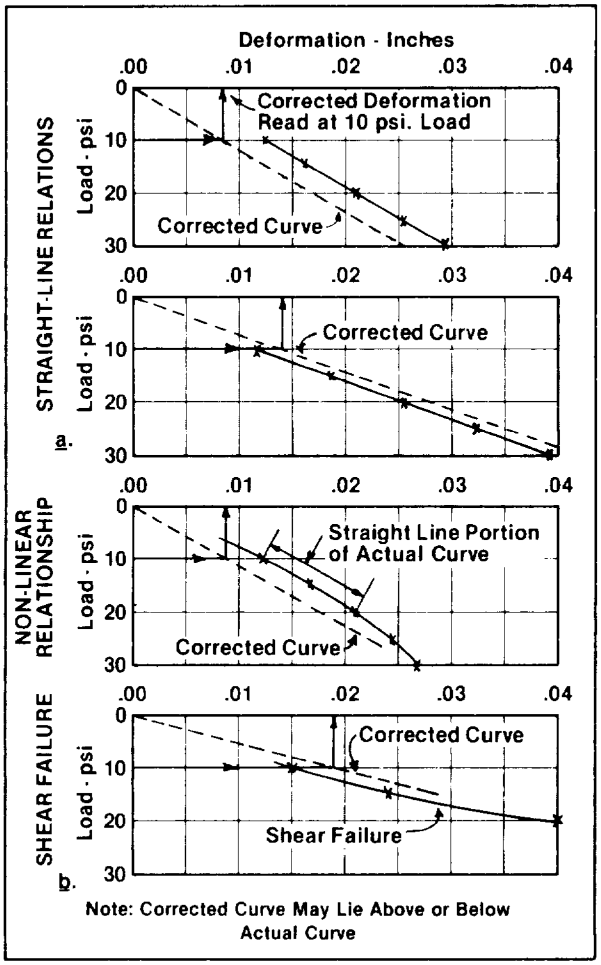


Figure 2-99. Load-deformation curve from plate bearing test



Note: Corrected Curve May Lie Above or Below Actual Curve

Figure 2-100. Zero corrections to load-deformation curves

graph (Figure 2-101, page 2-152) is used. A k'$_u$ greater than 200 requires a load deformation curve to be drawn and a new value of k'$_u$ determined.

This value is used to enter the curve and to read the corrected value k'$_u$, the subgrade modulus corrected for plate deflection.

Saturation Correction

For a better evaluation, a correction for subgrade saturation should be considered. The most applicable method is an adaptation of the consolidation test. The samples are selected from the location where the tests are to be performed. The correction is determined by loading two samples of the undisturbed subgrade material, one with the moisture content in place (sample A) and the other (sample B) with the undisturbed subgrade in a saturated condition. The same seating load used in the plate bearing test is applied to both samples. When the vertical dial becomes stable, a zero reading is taken and an additional 10 psi load is applied (total of 11 or 12 psi). After the dial becomes stable again, the

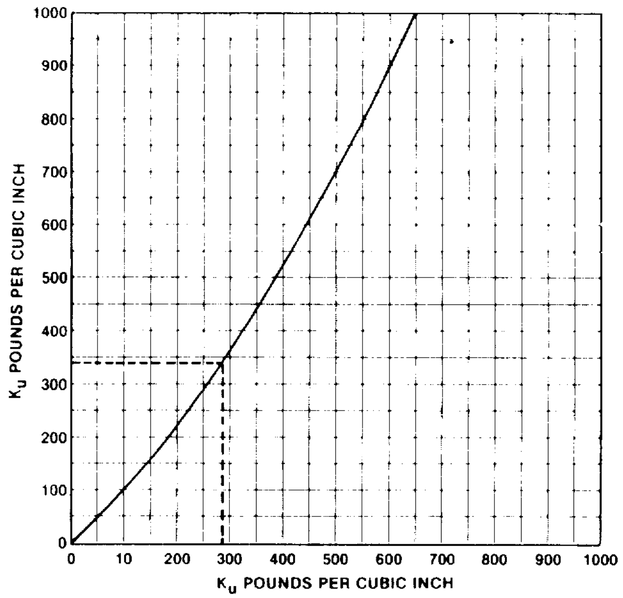


Figure 2-101. Plate bending correction for Ku

reading reflects the deformation under a 10 psi load. Certain soils may swell under saturation and cause the material to squeeze up over the restraining ring when pressure is applied. The top of both A and B samples should be trimmed about $\frac{1}{16}$ inch to allow for swelling so that the height of both samples is the same at the start of the test. The correction for saturation is applied in proportion to the deformation of both samples under a load of 10 psi. The value of the corrected subgrade modulus (k) is:

$$K = \frac{d}{d_s} \times k_u$$

where:

k = subgrade modulus corrected for saturated and bending of the plate

k_u = subgrade modulus, uncorrected for saturation, but corrected for bending of the plate when necessary

d_s = deformation of a standard specimen under a unit load of 10 psi in inches

d = deformation of a specimen at field conditions under a load of 10 psi in inches.

NOTE

The value of $\frac{d}{d_s}$ is limited to less than one.

Interpretation of Corrections

The k_u is rarely used in design without a correction. For coarse granular soils, k_u may be used without a correction if compaction requirements are met. Cohesive soils in arid regions with a low water table may have smaller saturation corrections than those determined by the test. If examination of other pavements in the area indicates no loss in support at the joints due to infiltration or leakage and the moisture content is lower than optimum for compaction, the uncorrected (for saturation) value of k_u may be used. In some instances, the correction does not fulfill the intended purpose and is not satisfactory. Two examples of these soils are the cohesionless or nearly cohesionless silts and fine sands which pump and the uniform, poorly-graded and poorly-compacted sand that tends to shift at pavement joints. For these conditions, a k value is assumed for the design.

Correlation of Subgrade Modulus and CBR

If time and equipment are not available for a full scale loading test, an approximate k value may be determined from the CBR value. This must be done for each soil type encountered. Table A-2 (Appendix A) shows typical k and CBR design values based on the classification of the subgrade soil.

Section XIII. UNCONFINED COMPRESSION TEST

DISCUSSION

The unconfined compression test is used to estimate the shearing strength of clay soils. It cannot be performed on cohesionless soils or soils which cannot stand unsupported. The test is most applicable to saturated clay soils and is performed upon relatively undisturbed samples at their natural moisture contents. In some cases the test is also run on remolded samples of the same soils at the same moisture content. This manual does not describe this procedure.

The test is usually performed on a small cylindrical sample. It resembles the compression test on a concrete cylinder except that the sample and loads are much smaller. The principal information obtained is the value of the compressive unit stress which corresponds to failure of the cylinder or to 15 percent compressive strain (reduction in height). This value, designated as q_u, is expressed in pounds or tons per square foot. The unconfined compressive strength is assumed to be equal to twice the shearing strength (cohesion) of such soils. This relationship is stated as $q_u = 2c$, in which c is the cohesion and q_u is the unconfined compressive strength. The relationship between stress and strain (unit deformation, or in this case, unit shortening) may also be determined during the test. Since the test is made on a sample which must stand by itself with no support, and is made immediately after the sample is placed in the test machine, the shearing strength is regarded as a minimum. The use of the test value is, therefore, conservative in many instances.

PREPARATION AND TEST PROCEDURE

The testing is done by loading the sample in the frame provided for the CBR test. Samples are secured with the soil trafficability sampler, or samples from soils stabilization may be determined.

Apparatus

Apparatus for conducting the test is assembled from components found under soil test apparatus. A penetrometer plus other items may be necessary. The assembled apparatus for the unconfined compression test is shown in Figure 2-102 (page 2-154). Moisture contents are determined by oven drying the total specimen.

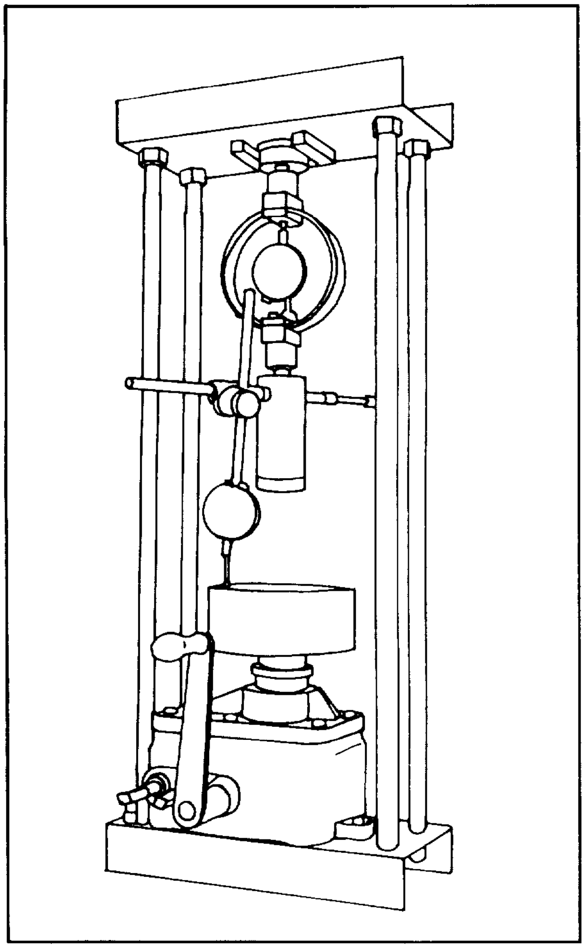


Figure 2-102. Assembled apparatus, unconfined compression test

- The CBR frame with the jack down is placed on a table and positioned so the crank can be turned. Preferably, it should be clamped in place. The sample is placed directly on the jack plunger or on a metal plate which rests firmly on the plunger plate.

- A cone penetrometer proving ring is selected from the soil trafficability apparatus. Note that when the unconfined compression test is completed, care must be taken to return the proper dial to the proving ring, since the penetrometer's dial indicator is calibrated for one particular ring only. If the face of the penetrometer is graduated so that 0 to 300 equals exactly 360 degrees of the dial face, assume that a deflection of 0.1 inch equals 150 pounds. Therefore, a deflection of 0.001 inch equals 0.15 pound. However, the readings taken frequently will be in the lower range of the ring's capacity, and much greater accuracy will be assured by a recalibration of the ring with the 0.0001 inch dial in place. The ring may be calibrated simply for the unconfined compression test (seldom more than 100 pounds total load except for very stiff or hard clays). This is accomplished by attaching the ring vertically in the vise, using a convenient thread on the penetrometer shaft, and by balancing known weights on it or an adapter above it. Calibrated weights may be used or weights may be prepared by weighing them on the solution balance. A series of weights should be used (for example, ½, 1, 2, 4, 6, 8, 10, 20, 30 pounds up to the limit of the anticipated test loads), and the corresponding dial readings should be recorded. Some penetrometer rings have been made whose dials have been calibrated to the ring. Therefore, 360 degrees on the dial will not be equal to precisely 300 cone index, but will be slightly more or less. If such a ring is used with the 0.0001 inch dial, the ring must be calibrated for the dial.

- The recalibrated ring is attached to the CBR ring-to-frame adapter and to the field CBR piston by means of the two penetrometer ring adapters. Piston extensions may be used as needed. The ring dial should be set at zero with the piston to be used hanging free.

- The strain or deformation of the sample is read from a 0.001 inch sensitivity dial attached to the dial clamp on the CBR frame and positioned to read the travel of the jack upward. A zero reading is recorded as the sample is lifted upward on the jack and comes into contact with the CBR piston. There should be no play whatsoever between the ring-to-frame adapter and the frame. If play is found, the adapter should be forced tightly into the clamp using paper shims.

Procedures

The sample used for an unconfined compression test is obtained with the soil trafficability sampler.

- Any protective coating or wrapping used to preserve the sample must be carefully removed before testing. The sample obtained by the soil trafficability sampler is 1.87 inches in diameter and 3.45 inches long. The sample should be tested immediately after being extruded from the sample tube and weighed. The height of the sample and the area of its cross section must be accurately known. The area may be determined by measuring the circumference with a flexible tape and then computing the diameter. If tape is not available, a strip of paper about ½ inch wide is wrapped around the sample, marked for length, and measured. The average of at least three measurements should be used for both the height and area. DD Form 1213 is provided for the unconfined compression test (Figure 2-103, page 2-156).

- Weigh the test specimen and record the weight on DD Form 1213 (Unconfined Compression Test) (Figure 2-103). This permits computation of the unit weight of the specimen.

- The sample is placed in the loading frame, the loading apparatus is adjusted, and both the vertical dial and proving ring dial are set at zero. The sample is loaded by turning the jack handle causing the jack to move upward at a rate of 0.05 inch per minute. This is the same rate required for the CBR penetration test. During the early stages of the test, readings on both dials should be taken corresponding to every 0.01 inch movement of the vertical dial. Later, as the stress-strain curve (Figure 2-104, page 2-157) begins to flatten, readings may be taken at 0.02 inch and, still later, at 0.05 inch intervals in the vertical dial readings. Readings of both dials are recorded in the appropriate spaces on the data sheet. Stop the test when the axial load remains constant or when 20 percent axial strain has been produced. Record the duration of the test, in minutes, to peak strength (time to failure) and type of failure (shear or bulge). Sketch the specimen after failure on the stress-strain plot sheet, Figure 2-104.

- After the sample has failed, place the entire specimen in a preweighed container and determine the moisture content in the usual way by the oven-drying method.

Calculations

Necessary calculations are indicated on the data sheet (Figure 2-103).

- The unit strain, E, is computed by the following equation.

$$E = \frac{\Delta L}{Lo}$$

where:

ΔL = the change in length of the specimen, in inches determined from the vertical dial readings

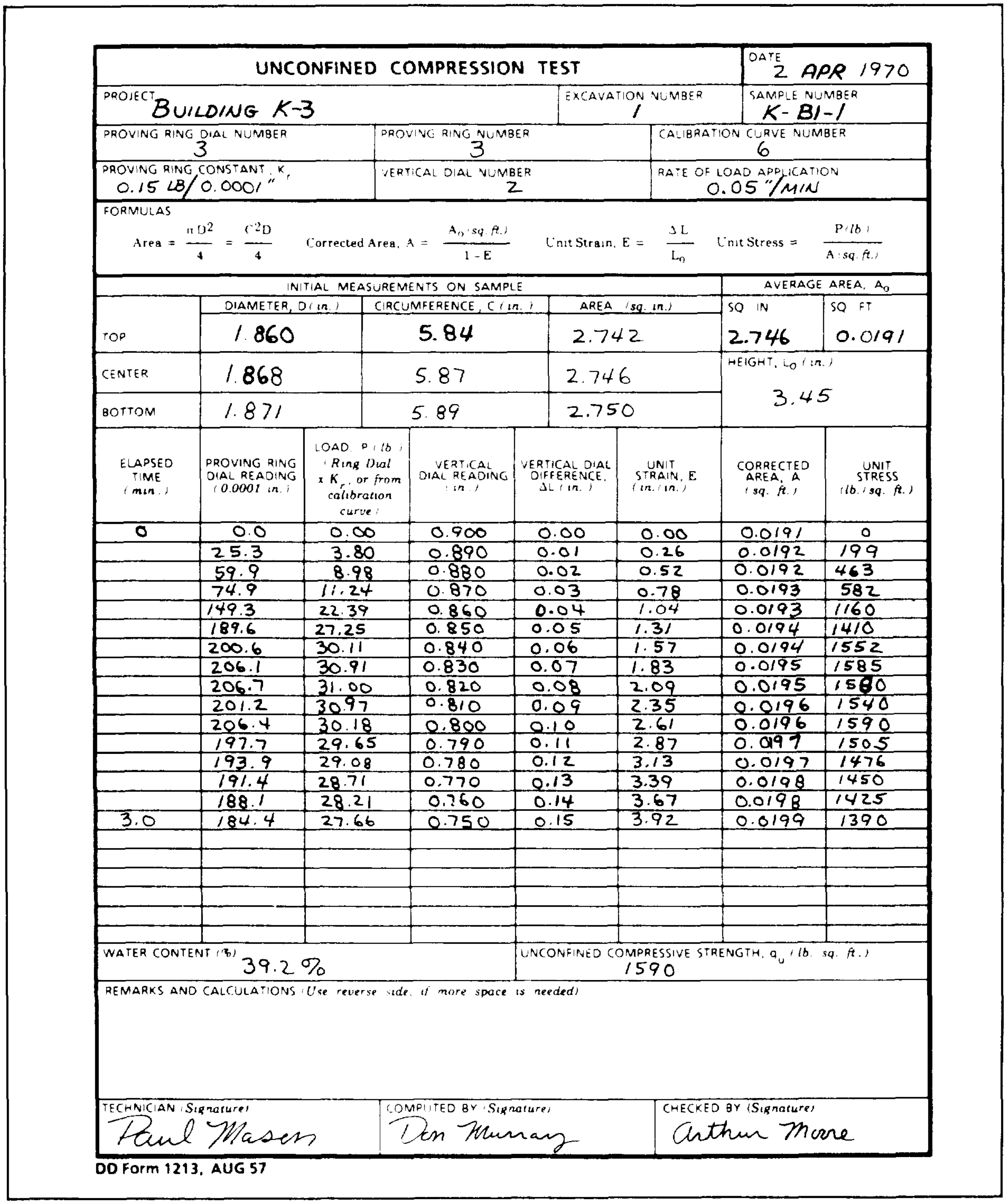
Lo = the initial length of the specimen, in inches.

- The average corrected area, A in square feet, which corresponds to any axial of strain is computed from the following expression:

$$A = \frac{A_o}{1 - E}$$

where:

A_o = the initial area of the test sample, in square feet.



UNCONFINED COMPRESSION TEST			DATE 2 APR 1970
PROJECT Building K-3		EXCAVATION NUMBER 1	SAMPLE NUMBER K-B1-1
PROVING RING DIAL NUMBER 3	PROVING RING NUMBER 3	CALIBRATION CURVE NUMBER 6	
PROVING RING CONSTANT, K_r 0.15 LB/0.0001"	VERTICAL DIAL NUMBER 2	RATE OF LOAD APPLICATION 0.05"/MIN	

FORMULAS

$$\text{Area} = \frac{\pi D^2}{4} = \frac{C^2 D}{4} \quad \text{Corrected Area, } A = \frac{A_0 \text{ (sq. ft.)}}{1 - E} \quad \text{Unit Strain, } E = \frac{\Delta L}{L_0} \quad \text{Unit Stress} = \frac{P \text{ (lb.)}}{A \text{ (sq. ft.)}}$$

INITIAL MEASUREMENTS ON SAMPLE				AVERAGE AREA, A_0	
	DIAMETER, D (in.)	CIRCUMFERENCE, C (in.)	AREA (sq. in.)	SQ IN	SQ FT
TOP	1.860	5.84	2.742	2.746	0.0191
CENTER	1.868	5.87	2.746	HEIGHT, L_0 (in.) 3.45	
BOTTOM	1.871	5.89	2.750		

ELAPSED TIME (min.)	PROVING RING DIAL READING (0.0001 in.)	LOAD, P (lb.) (Ring Dial x K_r, or from calibration curve)	VERTICAL DIAL READING (in.)	VERTICAL DIAL DIFFERENCE, ΔL (in.)	UNIT STRAIN, E (in./in.)	CORRECTED AREA, A (sq. ft.)	UNIT STRESS (lb./sq. ft.)
0	0.0	0.00	0.900	0.00	0.00	0.0191	0
	25.3	3.80	0.890	0.01	0.26	0.0192	199
	59.9	8.98	0.880	0.02	0.52	0.0192	463
	74.9	11.24	0.870	0.03	0.78	0.0193	582
	149.3	22.39	0.860	0.04	1.04	0.0193	1160
	189.6	27.25	0.850	0.05	1.31	0.0194	1410
	200.6	30.11	0.840	0.06	1.57	0.0194	1552
	206.1	30.91	0.830	0.07	1.83	0.0195	1585
	206.7	31.00	0.820	0.08	2.09	0.0195	1580
	201.2	30.97	0.810	0.09	2.35	0.0196	1540
	206.4	30.18	0.800	0.10	2.61	0.0196	1590
	197.7	29.65	0.790	0.11	2.87	0.0197	1505
	193.9	29.08	0.780	0.12	3.13	0.0197	1476
	191.4	28.71	0.770	0.13	3.39	0.0198	1450
	188.1	28.21	0.760	0.14	3.67	0.0198	1425
3.0	184.4	27.66	0.750	0.15	3.92	0.0199	1390

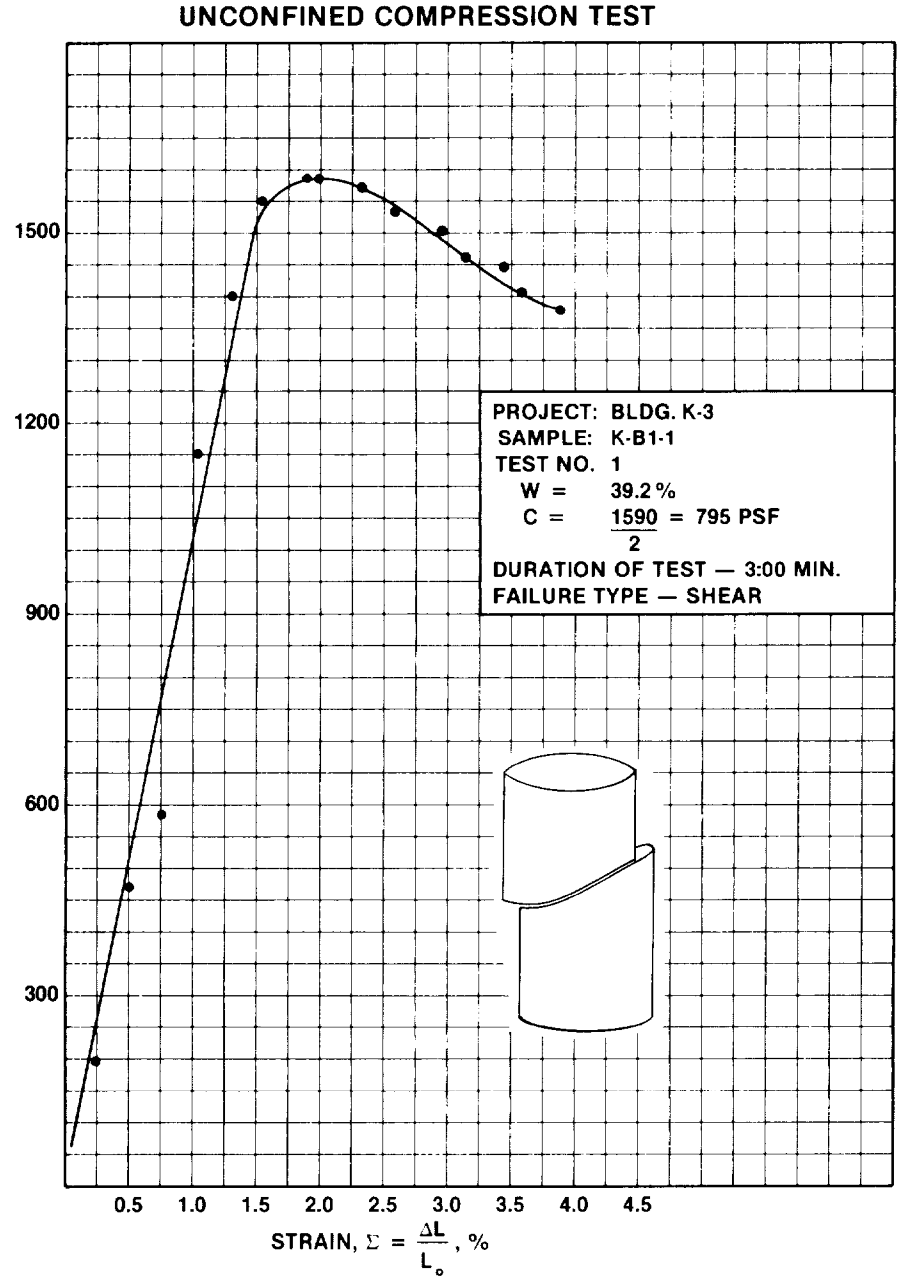
WATER CONTENT (%) 39.2 %	UNCONFINED COMPRESSIVE STRENGTH, q_u (lb. sq. ft.) 1590

REMARKS AND CALCULATIONS (Use reverse side, if more space is needed)

TECHNICIAN (Signature) Paul Mason	COMPUTED BY (Signature) Don Murray	CHECKED BY (Signature) Arthur Moore

DD Form 1213, AUG 57

Figure 2-103. Data sheet, unconfined compression test



PROJECT: BLDG. K-3
SAMPLE: K-B1-1
TEST NO. 1
W = 39.2%
C = $\frac{1590}{2}$ = 795 PSF
DURATION OF TEST — 3:00 MIN.
FAILURE TYPE — SHEAR

Figure 2-104. Stress-strain curve, unconfined compression test

- The total axial (P) load in pounds is determined by multiplying the quantity (proving ring reading minus initial proving ring reading) by the calibration factor for the proving ring or by using a calibration curve.

- The unit stress is determined by dividing pounds by the corresponding corrected area (A) in square feet.

- The stress-strain curve is drawn as indicated in Figure 2-104 (page 2-157) by plotting the strain or deformation, $\frac{4\triangle L}{Lo}$ in percent, on the horizontal scale and the unit stress (P/A) in pounds per square foot on the vertical scale.

- The unconfined compressive strength (q_u) is normally taken to be the peak value (P/A) on the stress-strain curve. As previously noted, the shearing strength is expressed as $c = q_u/2$. In the example shown (Figure 2-104), c equals 795 pounds per square foot.

Section XIV. TECHNICAL SOILS REPORT

DISCUSSION

A good program for soils testing not only requires that careful and complete tests be performed, but also that the tests be completed as quickly as possible and that the data be presented in a clear, logical manner. A delay in presenting a soil report results in a delay in construction, usually when time is critical. Therefore, the Materials Quality Specialist must be familiar with the sequence of testing, with the presentation of results, and with the tests themselves.

OUTLINE OF SOILS TESTS REQUIRED

The tests required by any program depend upon the type of construction being planned. However, there are a number of tests run consistently on road and airfield construction programs. A complete testing program should include:

TEST	REFERENCE TO PROCEDURE	
Soils exploration	Pages	through
Compaction	Pages	through
Plasticity tests		
Liquid limit	Pages	through
Plastic limit	Pages	through
Mechanical analysis		
Sieve analysis	Pages	through
Hydrometer analysis	Pages	through
California Bearing Ratio	Pages	through
Soil trafficability	TM 5-330, Chapter 9	

PURPOSE OF THE TESTS

The soil tests listed above are specific tests used to gain knowledge about general questions that must be answered for the control of soils during construction.

- Suitability of subgrade and borrow materials.
- Degree to which soil can be compacted.
- Bearing value of the subgrade and borrow material under projected future conditions.
- Location of ledge rock and groundwater table.
- Susceptibility to detrimental frost action.

Each test supplies data to the soils engineers which enables them to answer questions about a particular soil based on the engineering evaluation of scientific data rather than a meaningless guess. This makes the soils technicians valuable because they are first required to obtain accurate information and then to evaluate the meaning of the data from an engineer's viewpoint.

ORGANIZATION AND SCOPE OF THE TESTS

Because of the number of tests to be performed for a particular project, careful planning may avert considerable delays in the presentation of the results. The technician should list the tests required and list their sequence in a manner that will permit running the tests continuously, without delays due to time needed for soaking or drying samples. The following discussion of soil tests cites 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, location of ledge rock, depth to the groundwater table, field moisture content, and field identification of the soils sampled.

Compaction Test

The compaction test indicates the maximum dry density 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, the plasticity and mechanical analysis tests can be started.

Plasticity Tests

One use of liquid limit and plastic limit 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, attained through the classification of soils, is to ensure that the sample being tested meets specifications set on the liquid limit and plasticity index for the particular project for which it will be used. These tests should be performed during periods while the technician is waiting for compaction test moisture content samples to dry, CBR samples to soak, or any time between other tests.

Mechanical Analysis

The mechanical analysis is another test which 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.

California Bearing Ratio Test (CBR)

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 tests is available. This information may be obtained from approximated values of optimum moisture content.

Soil Trafficability

Tactical information concerning soil trafficability may be required on a large scale in the future. For this purpose soil test set Number 2 (Trafficability Equipment) has been designed. It includes the cone penetrometer and sampling equipment. This subject is discussed in TM 5-330, chapter 9/AFM 86-3, Vol II.

COMPILATION OF RESULTS

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 to follow should help organize a report.

Frequently portions of the information shown in the outline will be required at different times. For this reason, a preliminary report and several supplementary reports may actually be made before the project is completed. However, if all of the information provided follows the same basic outline, filing the data and assembling the final report will be simplified.

RECOMMENDED OUTLINE FOR A SOILS REPORT

The following outline is recommended to organize a technical soils report:

1. Project
    - 1-01 General description of project
    - 1-02 Extent and authority for proposed construction
    - 1-03 Purpose and scope of report
2. Description of the Site
    - 2-01 Description of location and existing facilities
    - 2-02 Topography, cultivation, and drainage
    - 2-03 Climate
3. Geology
    - 3-01 Description of origin of subsurface material at or near the site
    - 3-02 Description of overburden and bedrock
    - 3-03 Influence of geologic features on design
4. Aggregates
    - 4-01 Field exploration
    - 4-02 Field tests
    - 4-03 Laboratory tests
    - 4-04 Results of field and laboratory investigations
5. Mix Designs
    - 5-01 Asphaltic concrete
    - 5-02 Portland cement concrete
6. Foundation Conditions
    - 6-01 Field investigations
7. Fill and Borrow Materials
    - 7-01 Field explorations
    - 7-02 Field tests
    - 7-03 Laboratory tests
    - 7-04 Results of field and laboratory investigations

8. Design Criteria and Other Conditions
    - 8-01 Design loads and tire pressures
    - 8-02 Factors of safety used
    - 8-03 Allowable settlements
9. Design
    - 9-01 Analysis (Separate paragraphs for each item: flexible pavement, rigid pavement, settlement, stability and seepage)
    - 9-02 Alternate solutions
    - 9-03 Cost estimates for analysis and alternates
10. Conclusions and Recommendations
    - 10-01 Site selection (if possible)
    - 10-02 Economical design
    - 10-03 Minimum specifications
    - 10-04 Solutions for difficult problems

NOTE

Paragraphs 6, 7, 8, 9 and 10 may have to be repeated for each different runway or facility.

Annexes

A. General plan drawings (geology should be indicated)

B. Location plan drawings (existing and proposed features)

C. Profiles

D. Cross section

E. Boring logs

F. Laboratory testing data

G. Field testing data

H. Detailed numerical analysis (settlement and stability).

NOTE

Not every subject in the outline will apply to every report. In many reports some of the items may be covered in a sentence or two, while others may require a page of discussion.

Chapter 3

Bituminous Mixtures

Section I. DESCRIPTION OF BITUMINOUS MATERIALS

DISCUSSION

Bituminous pavements are a mixture of mineral aggregates, mineral filler, and a bituminous material or binder. They are used as the top portion of a flexible pavement structure to provide a resilient, waterproof, load-distributing medium which protects the base course from detrimental effects of water and the abrasive action of traffic. This chapter discusses the materials used in the construction of bituminous surfaces and the methods of testing these materials and the mixes prepared from them. The design considerations, such as bearing capacity and thickness of pavement, are described in TM 5-330/AFM 86-3, Vol II. Mixing and placing operations, as well as the considerations for preparing the mixes, are described in TM 5-337.

The aggregate may consist of crushed stone, crushed or uncrushed gravel, slag, sand and mineral filler, or a combination of some of these materials. Other materials which may be used as aggregate in certain geographical areas include vesicular lava and coral. Aggregates normally constitute 90 percent or more by weight of bituminous mixtures, and their properties greatly effect the finished product.

BITUMINOUS MATERIALS

The bituminous material is the adhesive agent in the bituminous mixture and may be either a tar or an asphaltic material. These are sometimes combined with rubber in efforts to enhance the performances of the pavement, particularly to improve the temperature susceptibility of tar. The asphalts are generally obtained from the distillation of crude petroleum (Figure 3-1, page 3-2). Asphalt may also be obtained from natural deposits. The tars are obtained from the destructive distillation of bituminous coal (Figure 3-2, page 3-3). The functions of a bituminous cement are to seal the surface which then resists the penetration of water. All bituminous materials possess these qualities.

The bituminous cements are available in several forms suitable for different procedures of mixing or application under wide variations in temperature. Some bituminous materials are solid or semisolid at room temperatures. Other grades are relatively viscous (thick) liquid at room temperatures. Mixing bituminous materials with solvents or water produces cutbacks or emulsions that are liquid at atmospheric temperatures. Such liquid asphalts

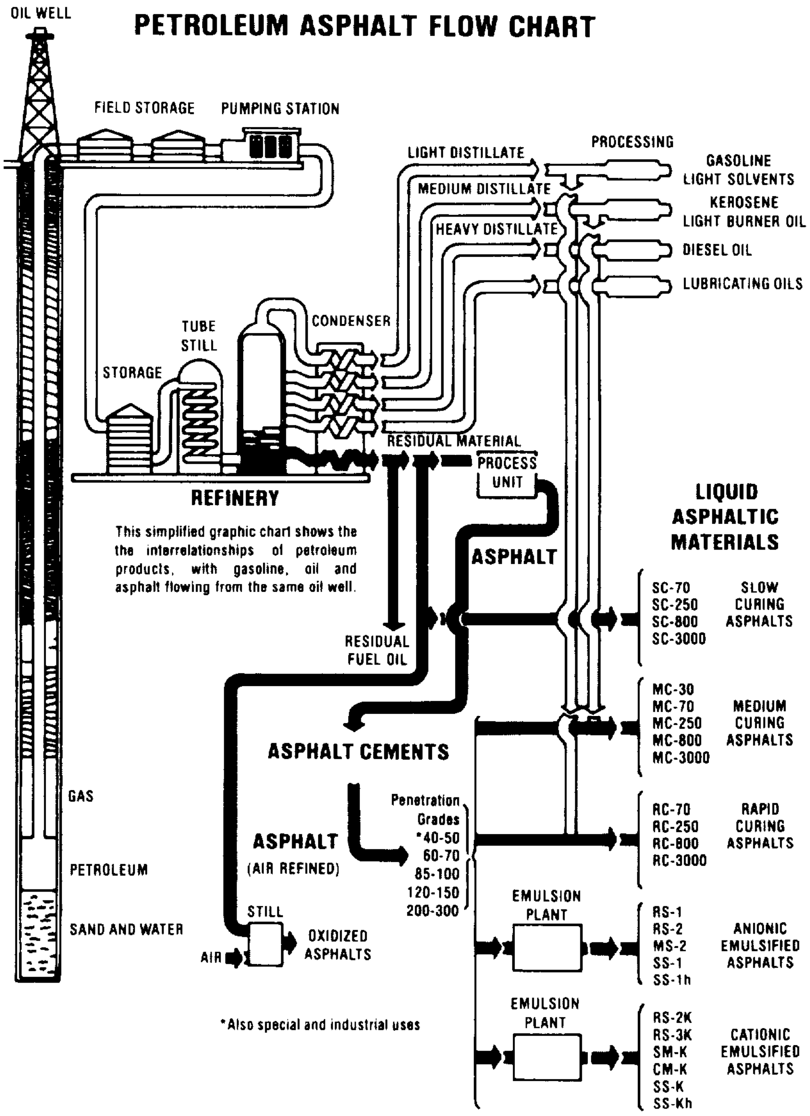

OIL WELL
PETROLEUM ASPHALT FLOW CHART
FIELD STORAGE
PUMPING STATION
LIGHT DISTILLATE
PROCESSING
GASOLINE
LIGHT SOLVENTS
MEDIUM DISTILLATE
KEROSENE
LIGHT BURNER OIL
HEAVY DISTILLATE
DIESEL OIL
LUBRICATING OILS
CONDENSER
TUBE
STILL
STORAGE
RESIDUAL MATERIAL
PROCESS
UNIT
REFINERY
This simplified graphic chart shows the the interrelationships of petroleum products, with gasoline, oil and asphalt flowing from the same oil well.
ASPHALT
LIQUID
ASPHALTIC
MATERIALS
SC-70
SC-250
SC-800
SC-3000
SLOW
CURING
ASPHALTS
RESIDUAL
FUEL OIL
MC-30
MC-70
MC-250
MC-800
MC-3000
MEDIUM
CURING
ASPHALTS
ASPHALT CEMENTS
GAS
Penetration
Grades
*40-50
60-70
85-100
120-150
200-300
ASPHALT
(AIR REFINED)
RC-70
RC-250
RC-800
RC-3000
RAPID
CURING
ASPHALTS
PETROLEUM
SAND AND WATER
STILL
AIR
OXIDIZED
ASPHALTS
EMULSION
PLANT
RS-1
RS-2
MS-2
SS-1
SS-1h
ANIONIC
EMULSIFIED
ASPHALTS
*Also special and industrial uses
EMULSION
PLANT
RS-2K
RS-3K
SM-K
CM-K
SS-K
SS-Kh
CATIONIC
EMULSIFIED
ASPHALTS

Figure 3-1. Simplified flow chart showing recovery and refining of petroleum asphaltic materials

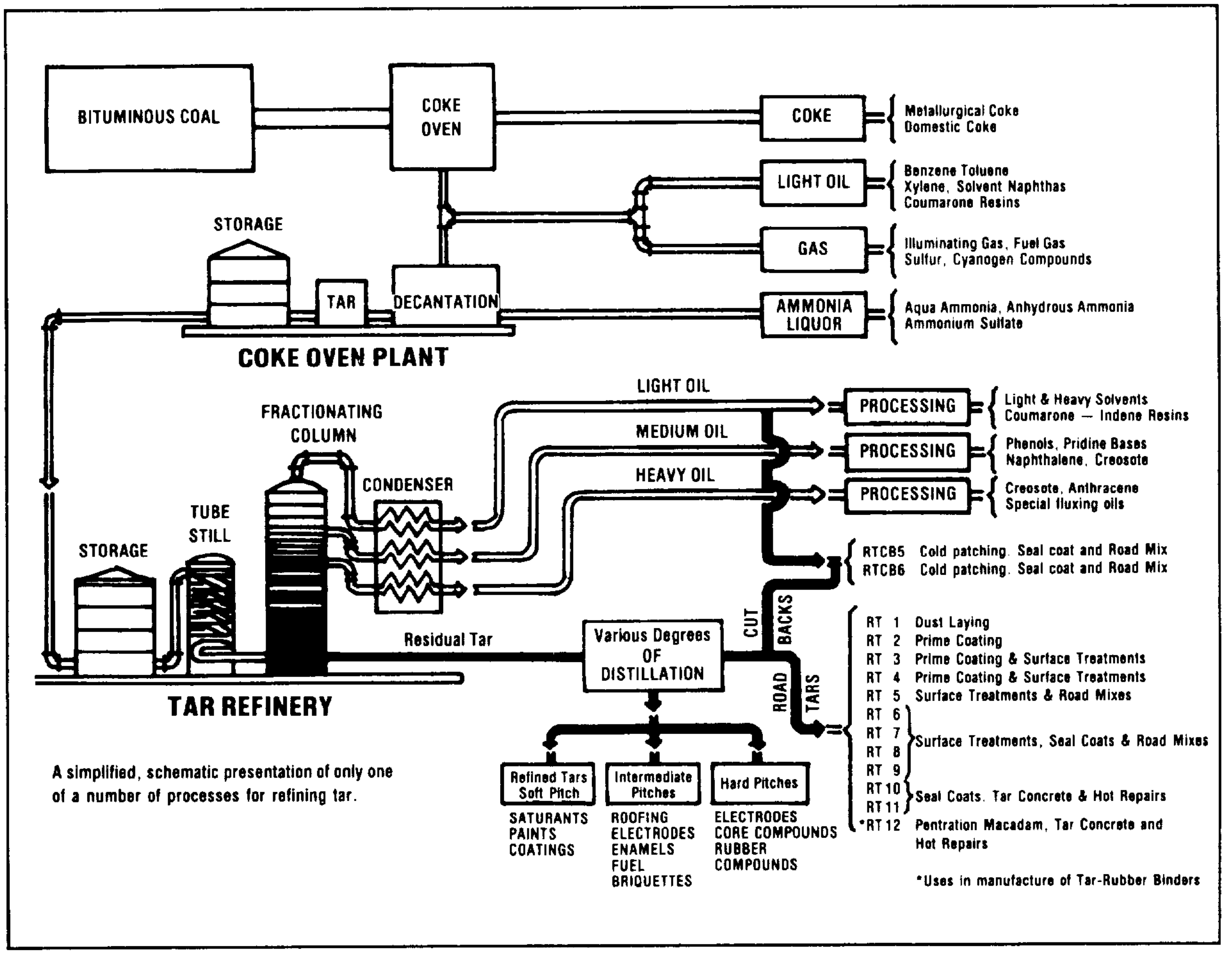

BITUMINOUS COAL
COKE OVEN
COKE
Metallurgical Coke
Domestic Coke
LIGHT OIL
Benzene Toluene
Xylene, Solvent Naphthas
Coumarone Resins
STORAGE
GAS
Illuminating Gas, Fuel Gas
Sulfur, Cyanogen Compounds
TAR
DECANTATION
AMMONIA LIQUOR
Aqua Ammonia, Anhydrous Ammonia
Ammonium Sulfate
COKE OVEN PLANT
LIGHT OIL
FRACTIONATING COLUMN
PROCESSING
Light & Heavy Solvents
Coumarone — Indene Resins
MEDIUM OIL
PROCESSING
Phenols, Pridine Bases
Naphthalene, Creosote
HEAVY OIL
CONDENSER
PROCESSING
Creosote, Anthracene
Special fluxing oils
TUBE STILL
STORAGE
RTCB5 Cold patching. Seal coat and Road Mix
RTCB6 Cold patching. Seal coat and Road Mix
CUT BACKS
Residual Tar
Various Degrees OF DISTILLATION
RT 1 Dust Laying
RT 2 Prime Coating
RT 3 Prime Coating & Surface Treatments
RT 4 Prime Coating & Surface Treatments
RT 5 Surface Treatments & Road Mixes
RT 6
RT 7
RT 8
RT 9
Surface Treatments, Seal Coats & Road Mixes
RT 10
RT 11
Seal Coats. Tar Concrete & Hot Repairs
*RT 12 Pentration Macadam, Tar Concrete and Hot Repairs
ROAD TARS
TAR REFINERY
A simplified, schematic presentation of only one of a number of processes for refining tar.
Refined Tars Soft Pitch
Intermediate Pitches
Hard Pitches
SATURANTS
PAINTS
COATINGS
ROOFING
ELECTRODES
ENAMELS
FUEL
BRIQUETTES
ELECTRODES
CORE COMPOUNDS
RUBBER
COMPOUNDS
*Uses in manufacture of Tar-Rubber Binders

Figure 3-2. Simplified flow chart showing production of road tars from bituminous coals

and tars are used for cold mixes or are applied as sprays in building pavements.

ASPHALTS

Asphalts may be natural or manufactured and may be solid, semisolid, or liquid in consistency. Natural asphalts occur in lakes (as lake asphalt), pits, or rock structures (as rock asphalt). Asphalt cement is one of the components from the refining of crude petroleum.

The military engineer is seldom concerned with the natural asphalts because they are not usually available in those areas of interest. The engineer depends most generally upon the manufactured asphalts which are obtained when crude petroleum is refined for the purpose of separating the various fractions (Figure 3-1). The crude oil vapors are separated into gasoline, kerosene, and fuel oils, while the residue is asphalt cement and lubricating oils. The longer the process and the higher the temperatures, the harder the residue becomes because of the increased loss in volatiles. This distillation may be accomplished either under vacuum or under atmospheric conditions and may or may not include air blowing. The refineries also often use a solvent precipitation process to achieve the above described separations. All asphalt cements are solid or semisolid at room temperature (77°F) and must be converted to a fluid state by heating, emulsifying, or dissolving in a petroleum solvent.

Grading

Asphalt cements are graded in three ways. The penetration grade is determined by the distance a standard needle under a standard loading will penetrate a sample in a given time under a given temperature condition. The asphalt petroleum (AP) number is a number from 00 to 7 assigned to these penetration ranges. The viscosity grade is determined using a standard viscometer under standard conditions. Table 3-1 lists the ranges presently recognized along with the relative consistencies.

Table 3-1. Penetration grades and AP numbers of asphalt cement

Penetration grade	AP No.	Relative consistency
40-50	7	Hard
60-70	5	
85-100	3	Medium
120-150	1	Soft
200-300	00	

Fluidity

When heating equipment is not available or is impractical to use, asphalt cement can be made fluid by adding solvents (called cutter stock or flux oils). Cutter stock may be any one of the more volatile petroleum distillate products. The resulting combination is called asphalt cutback. Exposure to air causes the solvents to evaporate and leave the asphalt cement to perform its functions.

- The classification of the cutback is based on the rate of evaporation of the distillate that is in the mixture. Gasoline or naphtha (high volatility) produces a rapid curing cutback (RC); kerosene (medium volatility) produces a medium curing cutback (MC); and fuel oil (low volatility), produces a slow curing cutback (SC). Road oils, referred to occasionally, are a heavy petroleum oil in the SC grade of liquid asphalt. Table 3-2 shows the percentage of components by grade for the three types of asphalt cutbacks.

- As more cutter stock is mixed with a given amount of asphalt cement, a thinner liquid results. In practice, different amounts of cutter stock are added to a given amount of asphalt cement to obtain various viscosities, or grades, of cutbacks. The Corps of Engineers has approved a set of specifications for cutbacks based on kinematic viscosity. The number assigned to

Table 3-2. Asphalt cutback composition (expressed in percent of total volume)

Type	Components		Grades				
		Solvent	30	70	250	800	3000
Rapid curing RC	Asphalt cement			65	75	83	87
		Gasoline or naphtha		35	25	17	13
Medium curing MC	Asphalt cement		54	64	74	82	86
		Kerosene	46	36	26	18	14
Slow curing SC	Asphalt cement			50	60	70	80
		Fuel oil		50	40	30	20

each grade corresponds to the lower limit of kinematic viscosity as determined by a standard test. The upper limit of each grade is equal to twice the lower limit or grade number. The units used in the test are stokes or centistokes.

$$\frac{(\text{stokes})}{(100)}.$$

Thus a number 70 cutback refers to a viscosity range of 70 to 140 centistokes. The other grades and their limits are 250 (250-500), 800 (800-1,600), and 3,000 (3,000-6,000). In addition, the MC has a 30 grade. Figure 3-3 shows the scale of viscosity grades. The higher the viscosity, the thicker the liquid.

- Asphaltic penetrative soil binder is a special cutback asphalt composed of low penetration grade asphalt and a solvent blend of kerosene and naphtha. It is similar in character to standard low viscosity, medium-curing cutback asphalt, but differs in many specific properties. It is used as a soil binder and dust palliative.

Emulsion

Emulsified asphalt is a mixture of asphalt and water in which soaps, colloidal clays, and numerous other organic agents are used as emulsifiers. It is a heterogeneous system in which water forms the continuous phase of the emulsion and the minute globules of asphalt

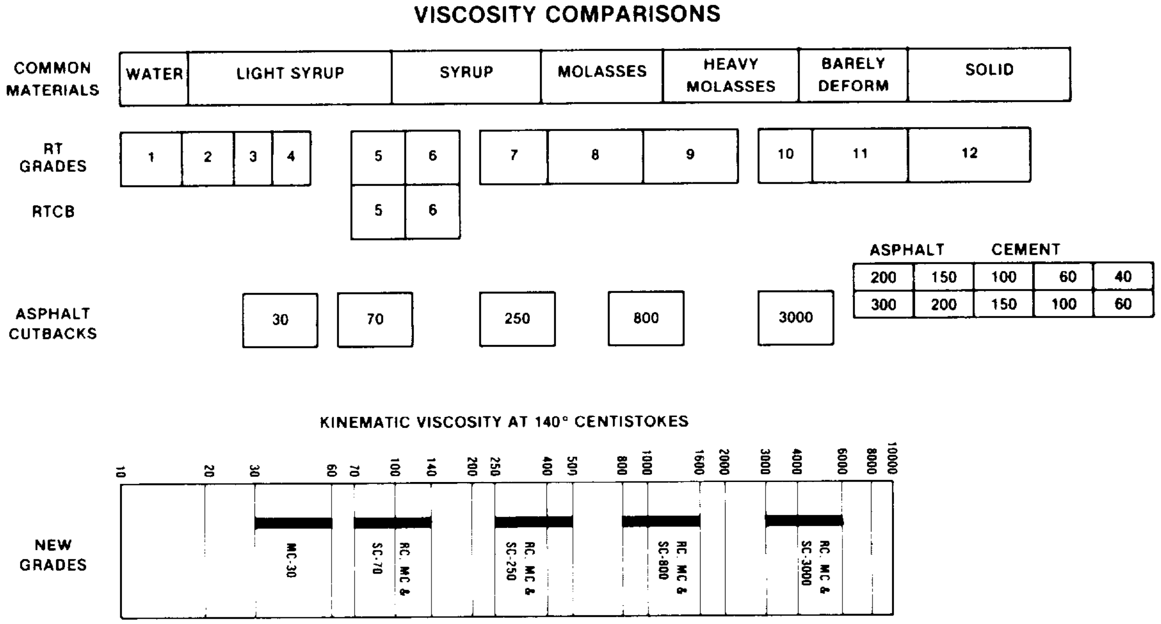


Figure 3-3. Viscosity grades at room temperature

form the discontinuous phase. There is also an inverted asphalt emulsion in which the continuous phase is asphalt (generally liquid asphalt) and the discontinuous phase is minute globules of water in relatively small quantities. Emulsified asphalts may be anionic (negatively charged asphalt globules) or cationic (positively charged asphalt globules) depending upon the emulsifying agent.

- Emulsions are classified according to setting time. Setting or breaking rate is the speed at which the emulsion breaks or the asphalt and water separate. This rate is generally dependent upon 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 (Figure 3-3, page 3-5).

- Emulsions are also grouped according to their ability to mix with damp aggregate. The RS emulsion breaks so fast that it cannot be mixed and is therefore called a nonmixing type. The MS and SS emulsions break slowly enough to permit good mixing until each particle of the aggregate is uniformly coated. Emulsions may also be satisfactorily used as a tack coat for bituminous pavements.

TARS

Tars are a product of the distillation of coal. No natural source of tars exists. Coal tar is a general term applied to all varieties of tar obtained from coal. It is produced by one of several methods depending on the desired end product.

When bituminous coal is destructively distilled, coke and gas are formed. Tar, ammonia, light oils, sulfur, and phenol may be recovered. Coke-oven tar is produced in the greatest amount. Its chemical, physical, and adhesive characteristics make it most suitable for road tar purposes. Water-gas tar is obtained in the manufacture of carbureted water-gas. The nature of the carbureting oil largely determines the character of the water-gas tar produced and may vary widely in specific gravities, viscosities, and other physical and chemical properties.

Road tars are manufactured in 12 grades of viscosity (Figures 3-2 and 3-3). There are also some special grades for use in rubberized-tar binders. Grades 1 through 7 are liquid at room temperature, while grades 8 through 12 are semisolid or solid. The difference is due to the liquid coal distillates in the tar; the more distillate, the more liquid (or less solid) the tar. The road tar cutbacks (RTCB) are products of cutting back the heavier or harder grades with coal tar distillates. RTCBs are manufactured in 2 viscosity grades (5 and 6) only.

Tar, which is insoluble in petroleum distillates, is sometimes mixed with oil resistant, unvulcanized rubber to form a rubberized-tar binder material.

CHARACTERISTICS AND USES OF BITUMENS

Tables 3-3 and 3-4 (pages 3-10 and 3-11), list the bituminous materials, sources, curing, temperatures, and grades associated with bituminous operations.

Safety Precautions

Extreme caution must be observed when handling bituminous materials. Asphalt cement, which is solid at room temperature, is heated to high temperatures to make it workable as a binder material. Heated asphalt can cause severe burns if allowed to come in contact with the skin. Asphalt cutbacks contain highly flammable volatiles. The vapors will ignite at relatively low temperatures, and the lowest temperature at which they will ignite is called the flash point. 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 3,000 and SC-70, about 150°F; and for the remaining SC grades, about 25°F higher per grade up to 225°F for SC-3,000.

Table 3-3. Characteristics of bituminous materials

Material	Form	Grade designation	Temperature of application ranges				Flashpoint (min)		Remarks
			Spraying[1]		Mixing				
			°F.	°C.	°F.	°C.	°F.	°C.	
Penetrative Soil Binder	Liquid	-	130-150	55-65	-	-	80	27	Contains naphtha. Caution: Highly flammable.
Cutback (RC)	Liquids - asphalt residues fluxed with more volatile petroleum distillate.	RC -70	[2]105-175	[2] 41-79	95-135	35-57			Rapid curing cutbacks contain highly volatile naphtha cutterstock. Naphtha evaporates quickly leaving asphalt cement binder permitting early use of surface. Caution: Highly flammable.
		-250	145-220	63-104	135-175	57-79	80	27	
		-800	180-255	[2] 82-124	170-210	77-99	80	27	
		RC -3000	[2]215-290	[2]102-143	200-240	93-116	80	27	
(MC)	Same as above	MC-30	70-140	21-60	55-95	13-35	100	37	Medium curing cutbacks contain less volatile kerosene cutterstock. Kerosene evaporates less rapidly than naphtha. Caution: Flammable.
		-70	105-175	41-79	95-135	35-57	100	37	
		-250	145-220	63-104	135-175	57-79	150	65	
		-800	180-255	82-124	170-210	77-99	150	65	
		-3000	215-290	102-143	200-240	93-116	150	65	
(SC)	Same as above	SC -70	105-175	41-79	95-135	35-57	150	65	Slow curing cutbacks contain slightly volatile diesel fuel cutterstock. Diesel fuel evaporates slowly. Caution: Flammable.
		-250	145-220	63-104	135-175	57-79	175+	79+	
		-800	180-225	82-124	170-210	71-99	200+	93+	
		-3000	215-290	102-143	200-240	93-116	225+	107+	
Asphalt Cements (AC)	Solids	40-50			300-350	149-177			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 (FM 5-530) to determine flashpoint.
		60-70	285-350	141-177	275-325	135-163			
		85-100	285-350	141-177	275-325	135-163			
		120-150	285-350	141-177	275-325	135-163			
		200-300	260-325	127-163	200-275	93-135			

Table 3-3. *Characteristics of bituminous materials (continued)*

Powdered Asphalt (PA).	Hard and solid asphalts ground to powder.								Used with SC to produce extra tough road surfaces.
Asphalt Emulsions (RS).	Liquids - asphalt particles held in an aqueous suspension by an emulsifying agent.	RS-1	50-140	10-60	Nonmixing	10-60			Freezing destroys emulsion. Use for road and plant mixes with coarse aggregates (SS). All emulsions with "K" suffix are cationic.
		RS-2	50-140	10-60	50-140	10-60			
		RS-2K	50-140	10-60	50-140	10-60			
		RS-3K	50-140	10-60	50-140	10-60			
(MS).	Same as above	MS-2	50-140	10-60	50-140	10-60			
		SM-K	50-140	10-60	50-140	10-60			
		CM-K	50-140	10-60	50-140	10-60			
(SS).	Same as above	SS-1	50-140	10-60	50-140	10-60			
		SS-1h	50-140	10-60	50-140	10-60			
		SS-K	50-140	10-60	50-140	10-60			
		SS-Kh	50-140	10-60	50-140	10-60			
Road Tars (RT).	Liquids	RT-1	60-125	15-52					Priming oils. RT-4 through RT-12 not generally used.
		RT-2	60-125	15-52					
		RT-3	80-150	27-66					
Road Tar Cutbacks (RTCB).	Same as above	RTCB-5	60-120	16-49					Patching mixtures. Caution: Flammable.
		RTCB-6	60-120	16-49					
Rock Asphalt.	Solids								Mixed and used locally where found. Cutback may be added if necessary.

---

[1] Low temperature is based on a viscosity of 200 centistokes kinematic viscosity and the higher temperature is based on a 50 centistokes viscosity.

[2] RC Cutbacks are seldom used for spraying.

NOTE

The spraying and mixing temperatures (Table 3-3) in many cases are above the flash point. Use extreme caution when handling these mixtures. Do not expose their vapors to an open flame.

Advantages and Disadvantages

Advantages and disadvantages of the bituminous materials used in construction are as follows.

- Asphalt cement cutbacks are flammable. Asphalt pavements are susceptible to damage by the blast from jet planes, and they can be dissolved by petroleum products which may be spilled on them, such as during refueling at an airfield. Tars, on the other hand, are not affected by petroleum derivatives since they are products of coal. Tars, when used as a prime for base courses, seem to possess better penetration qualities than asphalts and are less susceptible to stripping (loss of bond to aggregate) in the presence of water. Tars are affected by temperatures and have a wide range in viscosity with normal surrounding temperature changes. Tar can become so soft during warmer weather that the pavement will rut under traffic. In colder weather it can become so brittle that the pavement will crack. The temperature susceptibility of tar binders is improved by blending with oil resistant rubber. Asphalt pavements and tar pavements are generally ready for traffic within a very few hours after placement since they can be used as soon as they reach normal temperature.

- Asphalt emulsions are not flammable and are liquid at normal temperatures. Since they are mixed with water, they can be used with more damp aggregate than required for the cutbacks. Additional water may be added to the emulsion up to proportions of 1:3 for use in slurry seal coats. Because emulsions contain water, they have certain disadvantages. During freezing weather, the emulsions can freeze and the components separate. Emulsions are difficult to store for extended periods because they tend to break even in unopened drums. When shipped, the water in the emulsion takes up valuable space which could be used to transport hard-to-obtain materials.

Table 3-4. Typical uses of bituminous materials

Purpose or Use	Grade or Designation[1]		
	CB - Asphalt Cutback[2]		
	RC **Rapid Curing**	**MC** **Medium Curing**	**SC** **Slow Curing**
Dust palliative	DCA-70[3]	MC-30, 70, 250 A.P.S.B.[4]	SC-70, 250
Prime coat:			
Tightly bonded surfaces		MC-30	
Loosely bonded - fine grained surfaces		MC-70	SC-70
Loosely bonded - coarse grained surfaces		MC-250	SC-250
Tack coat	RC-250, 800	MC-250, 800	
Surface treatment and seal coat:			
Coarse sand cover	RC-70, 250	MC-250, 800	
Clean coarse aggregate cover	RC-250, 800, 3000	MC-800	
Graded gravel aggregate cover		MC-250, 800	SC-800
Gravel mulch		MC-250	SC-250
Mixed in-place - Roadmix:			
Open-graded aggregate:			
Sand	RC-70, 250	MC-800	
Maximum diameter 1 in, high percentage passing No. 10		MC-800	
Macadam aggregate	RC-250, 800		
Dense-graded aggregate:			
High percentage passing No. 200		MC-250	SC-250
Maximum diameter 1 in, medium percentage passing No. 200		MC-250, 800	SC-250, 800
Premix or cold patch:			
Open-graded aggregate	RC-250	MC-800	SC-800
Dense-graded aggregate		MC-250	SC-250
Cold-laid plant mix:			
Open-graded aggregate:			
Sand	RC-250, 800		
Maximum diameter 1 in, high percentage passing No. 10	RC-800		SC-800
Macadam aggregate	RC-800, 3,000		
Dense-graded aggregate:			
High percentage passing No. 200		MC-800	SC-800
Maximum diameter 1 in, medium percentage passing No. 200		MC-800	SC-800
Aggregate precoating followed with asphalt		MC-30	SC-70
Hot-laid plant mix	RC-3,000	MC-3,000	SC-3,000
Penetration macadam:			
Cold weather	RC-800, 3,000		SC-3,000
Hot weather			

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[1] Prevailing temperature during construction also affects selection of bitumen and may be the determining factor rather than size and gradation of aggregate.

[2] Caution: Do not overheat aggregate when cutbacks are used to produce hot mixes.

[3] DCA-70 is a water emulsion of a polyvinyl acetate containing chemical modifiers (formerly UCAR-131). Proprietary product of Union Carbide Corporation, New York, N.Y.

[4] Asphaltic penetrative soil binder.

Table 3-4. *Typical uses of bituminous materials (continued)*

AC Asphalt Cement with a Penetration of –	AE Anionic and Cationic Asphalt Emulsion	RT-RTCB Road Tar and Road Tar Cutback
	SS-1, 1h	RT-1
		RT-2
		RT-3
		RT-4
200-300	RS-1, 2	RT-4, 5, 6, 7, 8, 9
	RS-1, 2	
120-150, 200-300		RT-6, 7, 8, 9, 10
		RT-6
	MS-2	
85-100	MS-2	RT-7
	SS-1h	RT-5, 6, 7
	SS-1	RT-6, 7, 8, 9
	MS-2	RT-5, 6, 7, 8 or RTCB-5, 6
	MS-2	
	MS-2	
	SS-1	RT-5, 6, 7, 8, 9
	SS-1	
85-100, 120-150		RT-11, 12
120-150	RS-1	RT-10, 11
85-100	RS-1	RT-12

Section II. SAMPLING MATERIALS

DISCUSSION

The materials used in conducting tests must represent those that will be used in construction. Otherwise, test results will be misleading. Sampling of materials for testing should receive close attention. When large samples must be subdivided into small units for the actual tests, care must be taken to keep the sample representative of the original mass. Aggregate samples should be reduced to proper size for testing by means of quartering. Methods for sampling natural deposits of sands and gravels are discussed in soil surveys and are applicable here.

Samples of bituminous materials may be taken at the place of manufacture or at the point of delivery. This manual assumes that sampling is done at the point of delivery. Samples may be taken for either of two purposes; to obtain an average of the delivered material and to find the maximum variation in the material's characteristics. Samples may be taken for analysis to identify bituminous materials if records are not available.

AGGREGATE SAMPLING

Aggregate varies in size from the larger stones or rocks to the gravels and sands. These materials for paving may still be in their natural deposits or may be in stockpiles previously gathered.

Stone from Ledges and Quarries

Separate samples of unweathered stone weighing at least 50 pounds each should be obtained from all strata that appear to vary in color and structure. A sketch plan with elevation, showing the thickness, length, width, and location of the different layers should be prepared so that the quantity available can be estimated.

Natural Deposits of Sand and Gravel

Samples should be selected which represent the different materials available in the deposit. The area should be sketched and the approximate quantities of different materials indicated. If the deposit is an open-face bank or pit, the sample should be taken by channeling the face so that it will represent material that visual inspection indicates may be used. The face should be cut immediately before sampling, and any material which has fallen from the surface along the face discarded. Any overlying material (overburden) that is not suitable for use as an aggregate is not included in the sample, 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. Representative samples for each change in strata can be obtained from the pit. If the material being sampled is all sand, about 25 pounds is sufficient for the tests. If it consists of sand and gravel, a somewhat larger sample (about 100 pounds) will be required for preliminary tests. The coarser the gravel portion, the larger the sample required.

Stockpiles

If the material has been stockpiled previously, great care must be exercised in obtaining samples. There is a natural tendency toward separation of similar size fractions into groups in the stockpile. The material near the outer edges and near the base of the pile is likely to be coarser than the average. A face should be cut into the stockpile near the base, the center, and the top, on at least two opposite sides. Samples obtained from these different sections of the pile are combined to give a representative sample.

Commercial Aggregates

It is preferable, when possible, to obtain samples of commercial aggregates at the plant during loading from stockpiles or bins. Separate samples should be obtained at different times while the material is being loaded in order to determine variations in the grading of the material. Samples from a bin should be taken from the entire cross section of the flow of material as it is being discharged. Testing separate samples will give 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. Samples from railroad cars should be taken from three or more trenches dug across the car at points that appear on the surface to be representative of the material. When obtaining the sample, remember that segregation of the different sizes has probably taken place and choose samples which are representative.

BITUMINOUS MATERIALS SAMPLING

Clean, dry containers must be used in sampling, and the containers must be tightly closed and properly marked. A sample for routine examination should not be less than 1 quart.

Liquid Materials

When sampling liquid bituminous materials from storage tanks or tank cars, take samples from near the top, middle, and bottom. Semisolid and solid materials shipped in tank cars may be made fluid by the application of heat and sampled in the same way. Samples can be taken from drain cocks on the side of the tank or car. If cocks are not present, the samples can be obtained by lowering weighted bottles or cans into the material, as illustrated in Figure 3-4. The bottle or can should be fitted with a stopper which can be removed by a string or wire after it has been lowered to the proper depth. The samples from the three levels should be tested separately to detect stratification.

Solid and Semisolid Materials

When sampling semisolid or solid materials in drums, barrels, or cakes, take samples at least 3 inches below the surface and 3 inches from the side of the container or cake. A clean hatchet may be used on hard material and a stiff putty knife on soft material. An auger, brace with a ¾-inch or larger bit, or other suitable tools may also be used.

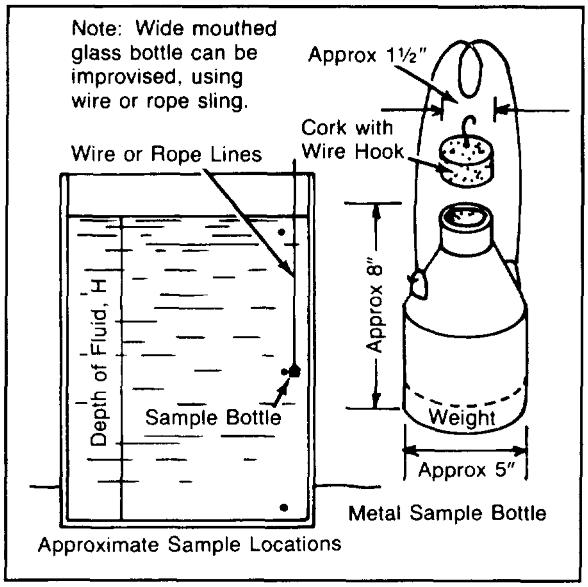


Figure 3-4. Sampling liquid bituminous materials from tank car or storage tank

Section III. FIELD IDENTIFICATION

DISCUSSION

Laboratory tests conducted on bituminous materials to check compliance with specifications are not considered field tests. They are described in this section for information only. The field tests discussed in this section are limited to the bitumen identification procedures, flash point tests, and the penetration tests. They are applicable to both tars and asphalts. They are conducted to determine safe uses for a material.

Field identification enables the military engineer to determine the type of surface that can be constructed with the type and grade of material available. With the type of surface known, the construction procedure may be outlined and scheduled. This procedure will then determine the proper equipment and the necessary safety procedures.

The aggregate materials must also be tested for acceptable bituminous construction use.

BITUMEN IDENTIFICATION

Field tests may be performed to identify the bituminous paving materials as asphalt cement, asphalt cutback, asphalt emulsion, road tar, or RTCB. In addition, the viscosity grade of the bitumen (Figure 3-3, page 3-5) must be identified. In order to distinguish among the several asphaltic and tar products, it is necessary to know something of their origin, physical properties, and the manner in which they are normally used. Some of this information is contained in Tables 3-3 and 3-4 (pages 3-7 — 3-10). The identification procedure outlined in Figure 3-5 is based upon a consideration of the physical properties of these materials.

Asphalts and Tars

The first procedure in the identification of an unknown bituminous material is to determine, by a solubility test, whether it is an asphalt or a tar. Attempt to dissolve an unknown sample (a few drops, if liquid, or enough to cover the head of a nail, if solid) in any petroleum distillate. Kerosene, gasoline, diesel oil, or jet fuel is suitable for this test. Since asphalt is derived from petroleum, it will dissolve in the petroleum distillate. Road tar will not dissolve. If the sample is an asphalt, the sample distillate mix will consist of a dark, uniform liquid. Asphalt cements or cutbacks dissolve completely. Asphalt in emulsions dissolves and forms black beads or globules in the bottom of the container. 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.

Cements and Cutbacks

The various grades of asphalt cement are solid at room temperatures while cutbacks are liquid, and a pour test will distinguish between them. Place a sample of the material in a small container and attempt to pour it. If the material does not pour, it is an asphalt cement. If it pours, it is a cutback or emulsion. Note that, at 77°F, even the softest asphalt cement will not pour or deform if the container is tilted.

Grades of Asphalt Cement

The various grades of asphalt cement are distinguished principally by their hardness, as measured by a field penetration test. The information obtained may be sufficient for planning for or starting emergency construction. The exact penetration grade is not determined, but the field test will distinguish between hard,

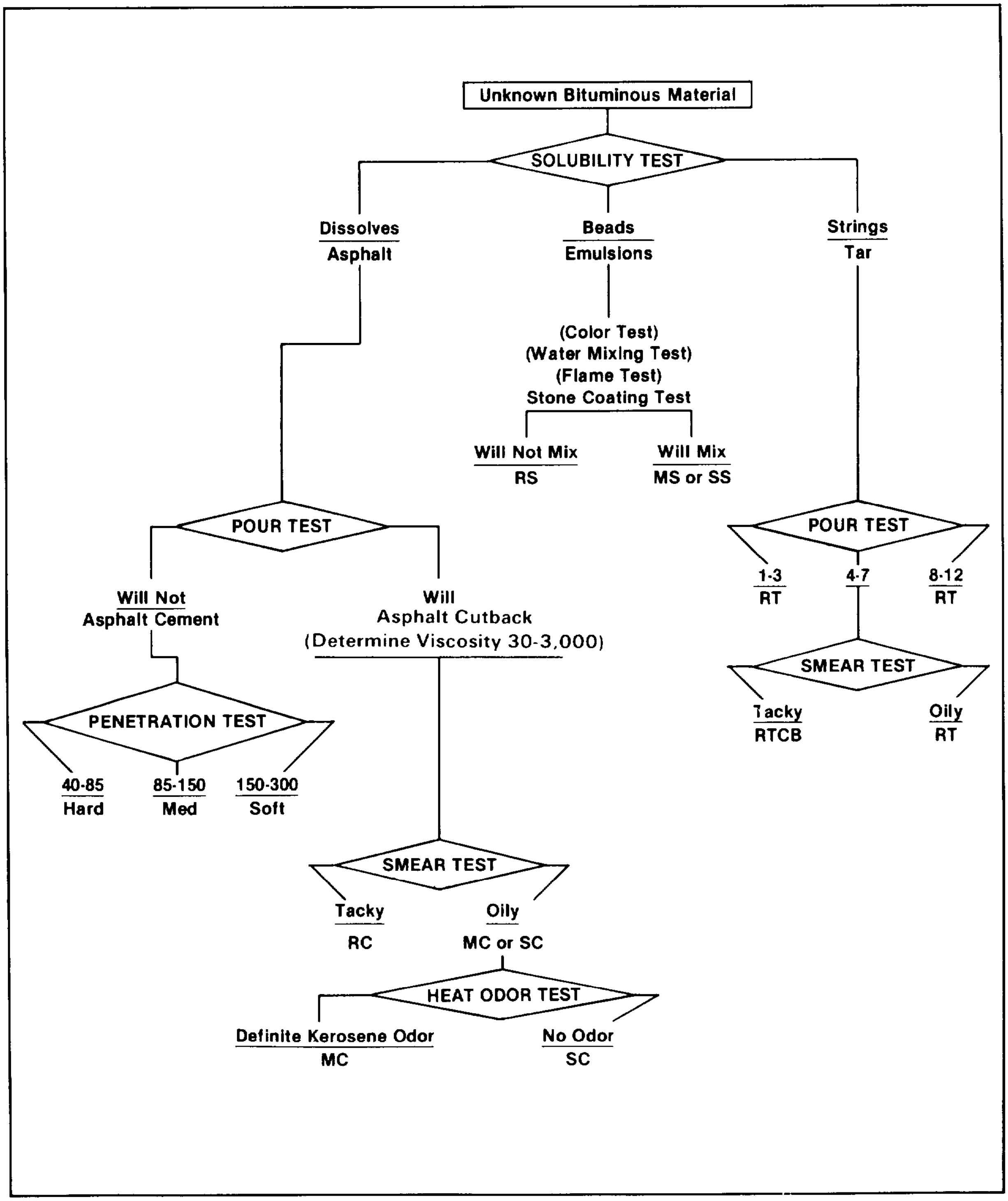

Unknown Bituminous Material
SOLUBILITY TEST
Dissolves
Asphalt
Beads
Emulsions
Strings
Tar
(Color Test)
(Water Mixing Test)
(Flame Test)
Stone Coating Test
Will Not Mix
RS
Will Mix
MS or SS
POUR TEST
Will Not
Asphalt Cement
Will
Asphalt Cutback
(Determine Viscosity 30-3,000)
PENETRATION TEST
40-85
Hard
85-150
Med
150-300
Soft
SMEAR TEST
Tacky
RC
Oily
MC or SC
HEAT ODOR TEST
Definite Kerosene Odor
MC
No Odor
SC
POUR TEST
1-3
RT
4-7
8-12
RT
SMEAR TEST
Tacky
RTCB
Oily
RT

Figure 3-5. Identification of unknown bituminous materials

medium, or soft groups of asphalt cement. The test is performed by attempting to push a sharpened pencil or nail into the container of asphalt (at about 77°F) using a 10-pound force. If only a slight penetration is made with considerable difficulty, a hard asphalt cement is present. If the penetration is made with some difficulty, a medium asphalt cement is present. If the penetration is made with ease, the asphalt cement is a soft asphalt cement in the high penetration scale. Even the highest penetration will not pour or deform at 77°F if its container is tilted.

Grades of Asphaltic Cutback

As stated previously, an asphalt cement would not pour at 77°, but a cutback would. The pour test can be used to determine whether the unknown material is an asphalt cutback. If the material pours, it is an asphalt cutback. The approximate viscosity grade number of the cutback is found by comparing the flow to well known materials such as water, syrup, and others. In this test, if made at a temperature below 77°F, the materials will appear more viscous than at 77°F, and the opposite if pour tested when warmer than 77°F. It has been found that the cutbacks of a given viscosity grade will pour in a manner similar to the following:

30 - Water
70 - Light syrup
250 - Syrup
800 - Molasses
3,000 - Barely deform

After the pour test, the approximate viscosity grade of the cutback is known, but the type (RC, MC, or SC) is not.

- Perform the smear test to determine if a cutback is an RC. This is done by making a uniform smear of the substance on a piece of glazed paper or other nonabsorbent surface. Volatile materials, if present, will evaporate. Since RC materials are cut back with a very volatile substance, most of the volatiles will evaporate within 10 minutes. The surface of the smear will then become extremely tacky. This is not true of the lighter grades (MC and SC) which remain fluid and smooth for some time. An MC will not result in a tacky surface for a matter of hours. SC materials may require several days.

- Perform a prolonged smear test to identify an 800 or 3,000 grade MC or SC cutback. This is necessary because these grades contain such small quantities of cutter stock that they, too, may become tacky in the 10-minute period specified above. A thin smear of the material is made on a nonabsorbent surface and left to cure for at least 2 hours. By the end of that time, if the material being tested is not an MC or SC, the smear will be hard, or just slightly sticky. However, if the material being tested is an MC or SC, the smear will be uncured and still quite sticky. If the material is an RC 3,000, it will cure completely in 3 hours, whereas an RC 800 will take about 6 hours to cure. Even after 24 hours, an MC or SC will still be sticky.

- Cut back MC materials with kerosene and SC materials with oil. This helps to differentiate between them. Heat is used to drive off the kerosenes, if present, and an odor from the vapors results. Heat the unknown sample in a closed container to capture the escaping vapors, using minimal heat. An MC sample will have a strong petroleum or kerosene odor. An SC sample will have no kerosene or petroleum odor but may have a slight hot motor oil odor. The ability to tell an RC from an MC, and an SC from either, is an essential part of field identification.

Asphalt Emulsions

Another asphaltic material used in paving work is asphalt emulsion, which is a mixture of asphalt, water, and an emulsifying agent. It is usually distinguished by its dark brown color, while the other bitumens are black. If mixed with kerosene or some other petroleum distillate, the emulsion can be detected by the appearance of small black globules or beads which fall to the bottom of the container. If mixed with water, an emulsion will accept the extra water and still remain a uniform liquid. The other bitumens will not mix with water. 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. After it has been established that the material is an emulsion, determine whether the emulsion is a mixing grade (slow setting, MS or SS) or a nonmixing grade (fast setting, RS). Try to mix a small amount (6 to 8 percent by weight) with damp sand using a metal spoon. A fast-setting emulsion cannot be mixed. It breaks immediately, gumming up the spoon with the relatively hard original asphalt cement. A slow-setting emulsion mixes easily, coating the sand. Do not add too much emulsion to the sand. This would saturate the sand and give inconclusive results.

Road Tars

If the unknown bituminous material did not dissolve in the solubility test, but formed a stringy mass (Figure 3-5, page 3-15) the material is a tar. The next step is to determine its viscosity grade by the pour test. By comparing the flow to that of the common materials (Figure 3-3, page 3-5), the viscosity of the tar may be closely estimated. The grades vary from RT-1 to RT-12. If the identified tar has a viscosity in the range of an RT-4 to RT-7 material, a smear test must be performed to determine whether it is a road tar or a road tar cutback. The smear test is performed in the manner previously described for cutback asphalt. The material is a road tar if stickiness increases in about 10 minutes. It is not important which grade of cutback is available since both are used under approximately the same conditions.

AGGREGATE IDENTIFICATION

Aggregate is identified by roughness, hardness, cleanliness, impermeability and gradation, and is selected for the best combination of these characteristics. Availability, length of haul and overburden must also be considered.

Angular and Rough

The aggregate in a pavement must transmit the traffic load to the base, usually by the interlocking and surface friction of the different particles. Angular particles with a rough texture are the best for this purpose since they do not tend to slide past each other. However, more binder may be required since the angular shape has a greater surface area per unit volume than a round particle.

Hard and Durable

The aggregate must withstand the loads without cracking or being crushed. Resistance to weathering is also a function of the durability. An aggregate's resistance to wear can be determined by the Los Angeles abrasion test. The Mohs hardness scale may be used to determine the hardness of the aggregate. This scale is fully explained in TM 5-545. It ranges from 1 for talc or mica to 10 for diamond. By trying to scratch the aggregate or the common materials, 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 level of hardness are:

fingernail - about 2

copper coin - between 3 and 4

knife blade - about 5

window glass - about 5.5.

Clean and Dry

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 will prevent the penetration or the adherence of bitumen and result in stripping of the binder. If the aggregate is not clean, it should be washed, either as part of the crushing operation or by spreading it on a hard surface and hosing it with water. When washing is impractical, dry screening may remove a great deal of dust and clay. Hand picking may have to be done if no other method can be used. The aggregate should be made as clean as possible with the equipment and work hours available.

Impermeable

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 weakens and breaks, and stripping occurs. Stripping is the loss of bituminous coating from the aggregate particles due to the action of water, leaving exposed aggregate surfaces. One of the three following tests can be used to determine the detrimental effect of water on a bituminous mix.

- Stripping test. A test sample is prepared by coating a specific amount of aggregate with bituminous material at the right temperature for the grade of bitumen to be used. The mixture is spread in a loose, thin layer and air cured for 24 hours. A representative sample is placed in a jar (no more than one-half full) and covered with water. The jar is closed tightly and allowed to stand 24 hours. At the end of 24 hours the jar, with the sample, is shaken vigorously for 15 minutes. A visual examination is made to determine the percentage of exposed aggregate surface which is then reported as the percent of stripping.

- Swell test. Asphaltic mixtures containing fines of doubtful quality are sometimes measured for swell as a basis for judging the possible effects on a pavement. This test is more frequently used with dense graded mixtures using liquid asphalts. A sample of the mix is compacted in a metal cylinder and cooled to room temperatures. A height measurement is obtained for the specimen. The specimen and mold are placed in a pan of water, and a dial gage is mounted above the sample in contact with the surface. An initial reading is taken. The specimen is allowed to soak for a specified period (usually 24 hours) or until there is no further swelling. Another reading of the dial is taken. The difference in readings divided by the original height and expressed in percent is the swell of the mixture. Experience has shown that bituminous pavement made with clear, sound stone, slag, or gravel aggregate and mineral filler produced from limestone will show test values of swell of less than 1.5 percent of the original specimen thickness.

- Immersion-compression test. This test is intended to measure the loss of Marshall stability resulting from the action of water on compacted bituminous mixtures containing penetration grade asphalt. The result is a numerical index of reduced stability obtained by comparing the Marshall stability with the stability of specimens that have been immersed in water for a prescribed period. Eight standard Marshall specimens (4 inches in diameter and $2\frac{1}{2} \pm \frac{1}{16}$ inches high) are prepared for each test. The specific gravity of each of the specimens is determined. The set of eight are separated into two sets of four so that the average specific gravity of one is essentially the same as the other set. One set is now tested using the Marshall method, and the other set is first immersed in water (at 140° ± 1°F) for 24 hours and then tested. The result is computed as a ratio of soaked stability to unsoaked stability, expressed as a percentage as follows:

$$\text{index or reduced stability} = \frac{S_2}{S_1} X\ 100$$

where:

S_1 = average stability of unsoaked specimens

S_2 = average stability of soaked specimens. Mixes with an index of less than 75 percent are rejected or an approved method of processing aggregate and treating asphalt is required to increase the index to a minimum of 75 percent.

Particle Size

In bituminous construction, it is common practice to designate aggregates according to particle size. There are three types of designations under this system, based on two sieve sizes, No. 4 and No. 200.

- Coarse aggregate is all material too large to pass the No. 4 sieve.

- Fine aggregate passes the No. 4 sieve, but is retained on the No. 200 sieve. In bituminous paving, the fine aggregate is usually a sand, but small pieces of crushed rock may be used.

- Mineral filler or mineral dust refers to all nonplastic materials which pass the No. 200 sieve. Most clays are too plastic and are not used. Generally, crushed rock dust, agricultural mineral filler, lime, or portland cement may be used as the mineral filter.

Gradation

- Uniform gradation. All particles are approximately the same size and normally less than 1 inch.

- Macadam gradation. This group consists of uniformly sized particles except that they are in excess of 1 inch.

- Open gradation. The sizes of particles in this group may have a considerable range from large to small, usually containing little or no mineral filler. The void spaces in the compacted aggregate are relatively large.

- Dense gradation. There is a good representation of all particle sizes, coarse, fine, and mineral filler.

Section IV. TESTS ON BITUMENS

DISCUSSION

The field identification tests on bitumens identify the material during expedient conditions or until more detailed tests can be performed. The identification determines whether the material is an asphalt or a tar and whether it is a cutback or an emulsion.

Bituminous materials are manufactured to meet specifications established by the federal government, AASHTO, and ASTM. These specifications define the extreme limits permitted in the manufacture of the material and assure the user that the material will possess definite characteristics and fulfill the project requirements. Conforming to specifications tests includes determining the specific gravity of the material, its solubility, its analysis by distillation, and its softening point. The equipment for performing these tests is not included in the asphalt test set and is not normally available to the materials technician. However, these tests are described for information and, when the equipment is available, to identify the material beyond field identification, to furnish information for mix design, or to establish safe handling procedures.

SPECIFIC GRAVITY TEST

Specific gravity of a bituminous material is defined as the ratio of the weight of a given volume of the material at 77°F (25°C) to the weight of an equal volume of distilled water at the same temperature. The results of a bitumen specific gravity test are used in the selection of the temperature-volume-weight correction factor to convert volumes to volume at 77°F (25°C). Space is provided on DD Form 1216 (Specific Gravity of Bituminous Mix Components) (Figure 3-6) to make this determination. A pycnometer (specific gravity bottle) is weighed empty and reweighed filled with water. The water is emptied from the bottle, and the bitumen is added. The pycnometer and bitumen are weighed. Water is added to the same level as the start of the test, and the entire combination is weighted. The procedure outlined on the form is followed to compute the weight of water displaced by the bitumen and the bitumen's apparent specific gravity. The specific gravity of asphalt cements will generally be in the range of 1.00 to 1.06 with the higher values being characteristic of the harder materials.

The specific gravity of an asphalt has little bearing on quality or other properties of the asphalt. However, the specific gravity is needed for other tests and computations. It is needed to adjust the specific gravity of the water bath in the ductility test. In acceptance and control testing on a job, it is used as a check on the uniformity of succeeding shipments of asphalt.

FLASH POINT AND FIRE POINT TESTS

These tests are applicable to asphaltic materials and are of some use in the identification of these materials. Their greatest usefulness, however, is in determining safe heating temperatures. Material heated above its flash point presents a very real danger, particularly if it is exposed to an open flame.

FLASH POINT AND FIRE POINT BY CLEVELAND OPEN CUP

This test is used for all petroleum products except fuel oils and those having open cup flash points below 175°F (70°C).

Apparatus

The equipment consists of:

Flash point apparatus. (Figure 3-7, page 3-22)

Knife

Frying pan or copper beaker

Hot plate

Stopwatch.

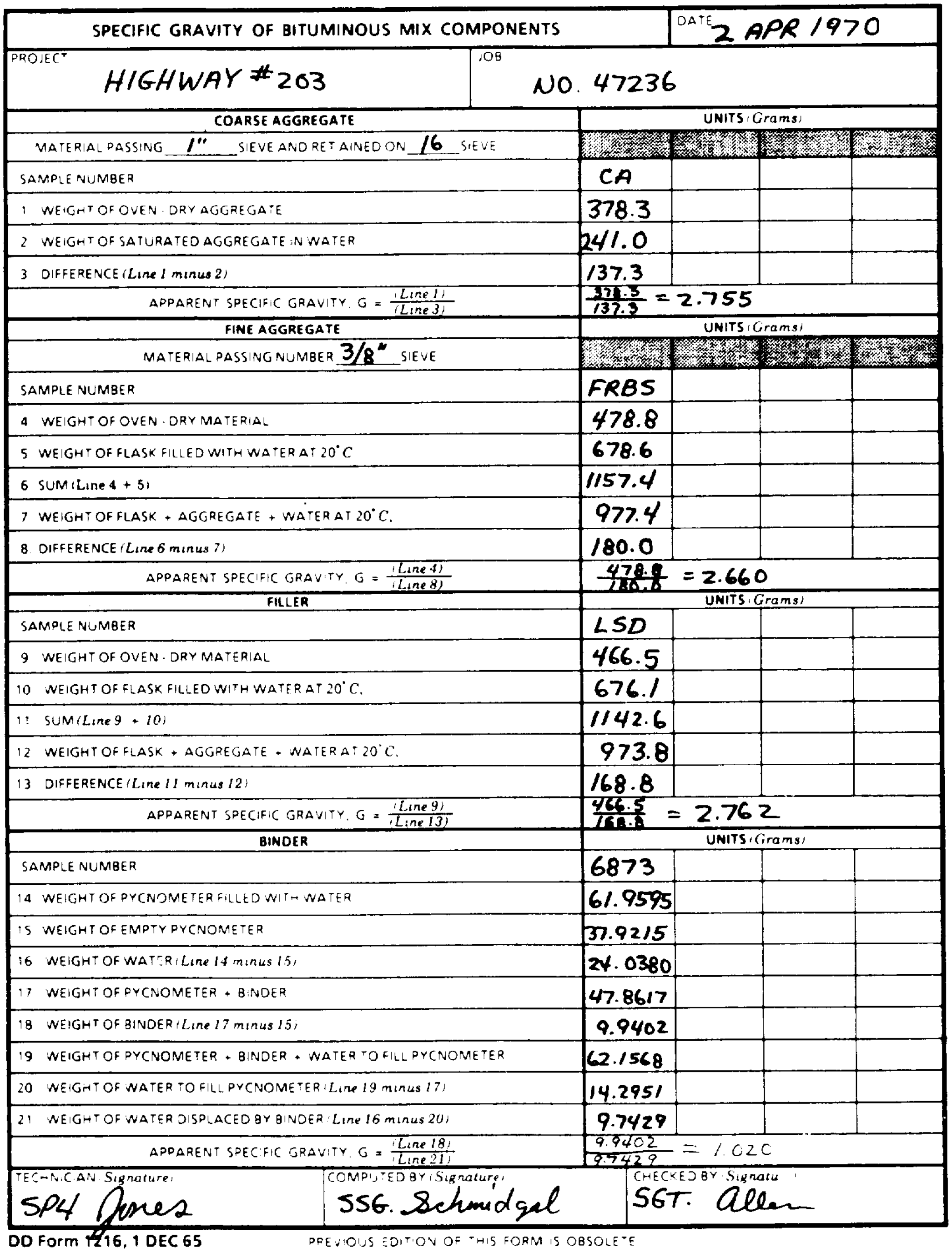
Procedure

It is best to perform this test away from any bright light or to shield the apparatus, if necessary. The cup should be cleaned thoroughly before starting. The bituminous material may be heated until it is fluid enough to pour into the cup. Within the last 30°F, before the expected flash point is reached, the cup and its contents should not be moved or disturbed. Air movement or drafts across the specimen surface should be prevented. The procedure of the test is as follows:

- Set up the open cup apparatus.

- Adjust the thermometer in a vertical position ¼ inch above the bottom of the cup and about midway between the center and back of the cup.

- Fill the cup with the heated material until the top of the meniscus is exactly at the filling line, and let the material cool.



SPECIFIC GRAVITY OF BITUMINOUS MIX COMPONENTS	DATE 2 APR 1970			
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 3/8" 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 PYCNOMETER + 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. Schmidgal	CHECKED BY *(Signature)* SGT. Allen		

DD Form 1216, 1 DEC 65 PREVIOUS EDITION OF THIS FORM IS OBSOLETE

Figure 3-6. Sample data sheet, specific gravity of Bituminous mix components (DD Form 1216)

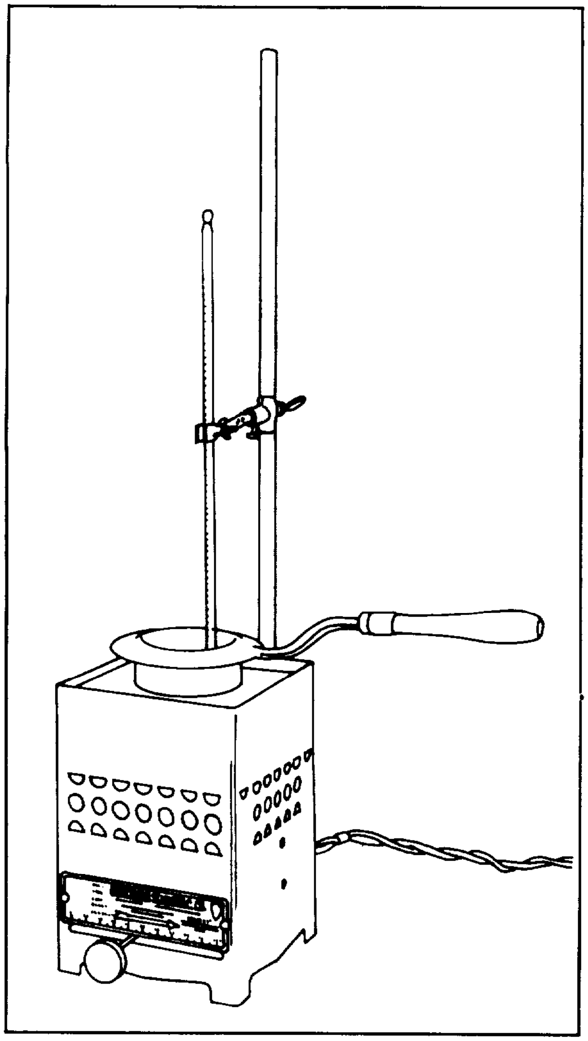


Figure 3-7. Flash point apparatus; Cleveland open cup

- Apply heat to the cup so that the specimen temperature is raised at a rate of 25° to 30°F per minute until a temperature of about 100° below the probable flash point is reached.

- Reduce the heat at this time and adjust it so that for the last 50°F before the expected flash point, the temperature rise will be not less than 9°F and not more than 11°F per minute. Use the stop watch to regulate this rate. Failure to set the rate of rise between these limits will result in inaccurate readings.

- Adjust a test flame to $\frac{1}{8}$ to $\frac{3}{16}$ inch in diameter, the size of the comparison bead if one is mounted on the apparatus.

- Start at least 50°F (28°C) below the expected flash point and pass the test flame in a straight line across the center of the cup at right angles to the thermometer and level to the upper edge of the cup. The time for each pass should be no more than 1 second. Repeat the test flame pass for each successive 5°F (2.8°C). The flash point is reached when a flash (distinct flicker) appears at any point on the surface of the material. Read the thermometer at this time and record the temperature as the flash point.

Continue heating at the same rate and applying the test flame at the same interval until the oil ignites and continues to burn for at least 5 seconds. Record the temperature at this point as the fire point.

Precision

Duplicate tests on the same material by the same operator should not differ by more than 15°F (8.3°C). Results by different laboratories should be considered suspect if the flash points differ by more than 30°F and the fire points differ by more than 25°F.

FLASH POINT BY TAG OPEN CUP

This test is used for the RC and MC asphalt cutbacks having a flash point below 200°F.

Apparatus

The equipment for this test consists of:

Flash point tester

Tag open cup (Figure 3-8)

Hot plate

Thermometer (20°F to 230°F in 1° divisions)

Torch or test flame.

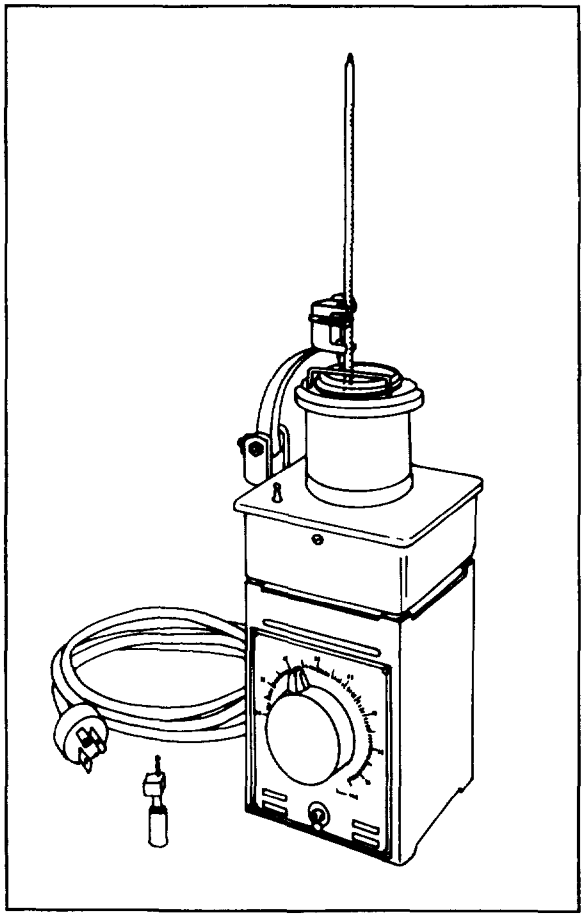


Figure 3-8. Flash point apparatus, Tag open cup

Procedure

The tester should be set up in a draft free and dimly lit location. The copper water bath is to be filled to $\frac{1}{8}$ inch below the top (when the top is in place). The bath may have an overflow to control the water level. The glass cup is cleaned and dried, and the water bath is assembled. The procedure of the test is as follows:

- Place the thermometer vertically midway between the center and the outer edge of the cup, and diametrically across from the guide wire. Set the bottom of the bulb about $\frac{1}{4}$ inch above the bottom of the glass cup.

- Fill the glass cup with the sample to just $\frac{5}{16}$ inch below the edge.

- Place the guide wire in position touching the rim of the glass cup.

- Adjust the heat for the sample temperature to rise at 2° ± $\frac{1}{2}$°F per minute. The thicker material should be stirred occasionally.

- Adjust the test flame to not greater than $\frac{5}{32}$ inch in diameter, the size of the comparison bead if one is mounted on the apparatus. (Some instruments have a $\frac{5}{32}$-inch hole in comparison instead of the bead.)

- Remove any bubbles which may have formed on the surface before starting the flame test.

- Pass the flame at successive 2°F intervals, pass the flame across the sample in a continuous motion making each pass last 1 second.

- Record as the flash point the temperature at the time the test flame application causes a distinct flash in the interior of the cup.

- Repeat the test using a fresh sample and starting at least 20°F below the previously determined flash point.

Calculation

The results of two properly conducted tests by the same operator on the same asphalt should not differ by more than 180°F. The results of two properly conducted tests from two different samples of the same asphalt should not differ by more than 27°F.

PENETRATION TEST

The grade of an asphalt cement is based upon the results of the standard penetration test. Penetration is defined as the distance that a standard needle vertically penetrates a sample of the material under standard conditions of time, temperature, and loading. Standard conditions are 5 seconds, 25°C (77°F) and 100 grams, respectively. The units of penetration are hundredths of a centimeter. Other conditions of temperature, load, and time that are used for special testing are given.

Apparatus

Equipment required for this test is as follows:

Bath, hot water
Copper beaker or frying pan
Stainless steel box
Electric hot plate
Sieve pan, 8-inch diameter
Penetrometer, asphalt testing (Figure 3-9)
Thermometer (66° to 80°F)
Stopwatch.

Procedure

The described test is dependent upon the use of the water bath maintained as closely as possible to the standard temperature of 77°F. Since the penetration of an asphalt cement varies with temperature, the bath is maintained at 77°F. If this is impractical or less accuracy is required, the test may be performed with the sample at room temperature. The procedure of the test is as follows:

- Melt the sample at the lowest possible temperature, using the electric hot plate and frying pan or copper beaker. Stir thoroughly until the sample is homogeneous and free of air bubbles.

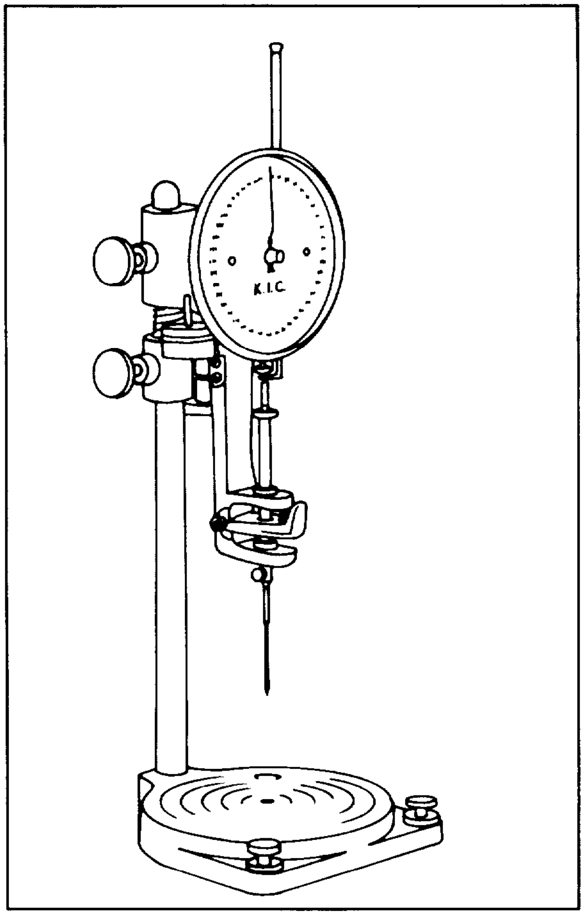


Figure 3-9. Asphalt penetrometer

- Pour the sample into the sample container (moisture content box) to a depth of not less than $\frac{3}{8}$ inch for the harder grades and $\frac{5}{8}$ inch for the softer grades. Protect the sample from dust and allow it to cool in an atmosphere not lower than 65°F for 1 hour.

- Place the sample in its container in the sieve pan (or any other small, flat-bottomed pan or container which is about 2 inches deep) and immerse the pan and sample in the water bath maintained at 77°F (25°C) for 1 hour.

- Keep the sample in the pan which is filled with water from the water bath. The water should completely cover the sample container in order to maintain the temperature during the test. The transfer dish containing the sample in its container and the water should be placed on the stand of the penetrometer.

- Adjust the needle to make contact with the surface of the sample. Place a light in a position so that the actual needle point and its image reflected on the surface of the specimen are clearly defined. Contact may be judged with the point and its image touch on the surface.

- Note the reading of the dial or bring the indicator on the dial to zero.

- Release the needle for a period of 5 seconds. Then read the distance penetrated on the thermometer. The needle and plunger are designed to have a standard weight of 100 grams. The dial is divided into 38 major divisions, each marked in 10 smaller subdivions. Each of the smaller subdivisions represents a penetration by the needle of $\frac{1}{100}$ centimeter. Thus, readings on the dial give penetration values without conversion.

- Test at least three points on the surface not less than $\frac{3}{8}$ inch from the side of the container and not less than $\frac{3}{8}$ inch apart. After each test, return the sample and pan to the water bath, and carefully wipe the needle toward its point with a clean, dry cloth to remove all adhering asphalt.

Results

The reported penetration is the average of at least three tests whose values do not differ by more than the following.

PENETRATION	0 to 49	50 to 149	150 to 249	250
Maximum difference between highest and lowest determination	2	4	6	8

The penetration test, as pointed out previously, is used to classify asphalt cements for purchasing and identifying purposes, but it has other uses as well. It can be used to detect overheating or prolonged heating of asphalts in storage tanks. Also, when an asphalt is extracted from a pavement, the penetration test affords a means of estimating how the asphalt has changed with time and weathering.

DUCTILITY TEST

In this test, dumbbell-shaped specimens of asphalt are molded under standard conditions. The dumbbell-shaped specimens are conditioned in a water bath to standard temperature (usually 77°F) then extended at the rate of 5 centimeters per minute until the threads connecting the two ends break. The difference in centimeters between the final length at break and the original length is the ductility.

The ductility test is helpful in estimating the ability of an asphalt 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 load or severe changes in temperature. Ductility is affected by various factors such as method of refining and consistency. Blown asphalts (asphalts that have been hardened by blowing air through them, which causes oxidation) have low ductility. This is one reason why they are not used as paving asphalts. Within the group of asphalts produced by steam and vacuum distillation, ductility will vary according to the consistency at a given temperature. Note that the ductility test is also sensitive to other factors, such as imperfections in the specimens or impurities such as mineral filler in a sample of asphalt recovered from a pavement.

SOFTENING POINT TEST

In the softening point test, a $\frac{3}{8}$-inch diameter steel ball is positioned on a brass ring filled with asphalt. The assembly is placed in the beaker containing freshly boiled, distilled water and is slowly heated. As the asphalt becomes warmer, it begins to soften, and the weight of the ball forces the asphalt out of the ring. The temperature at which the asphalt touches the bottom of the beaker is recorded as the softening point.

The softening point is another consistency test and varies inversely with the penetration test. Like the penetration test, the softening point test can be used to determine changes in an asphalt due to excessively high or prolonged heating. An abnormal increase in the softening point is an indication of excessive heating. The softening point test is used in studies on asphalts recovered from pavements after extended service to determine effects of aging. If an asphalt shows an unusual increase in softening point, considerable aging and hardening have occurred.

VISCOSITY TESTS

Saybolt-Furor Test

This test measures the time in seconds required to pass 60 cubic centimeters of liquid asphalt material at a given temperature through a tube of standard dimensions. Its purpose is to determine the viscosity of the material from which the spraying temperatures are established for field application of the bitumen.

Kinematic Viscosity Test

This test, like the Saybolt-Furol, measures the time that a given amount of liquid asphalt material will flow through a tube of standard dimensions under rigidly controlled conditions of temperature and pressure (or vacuum). The test establishes the viscosity of the liquid and, when correlated with the specific gravity of the material at the same temperature, results in a numerical designation called kinematic viscosity. The units used for kinematic viscosity are the stokes (square centimeters per second) or centistokes ($^{1}/_{100}$ stokes). The kinematic viscosity test requires special laboratory equipment that is not available in the field.

SOLUBILITY OF BITUMINOUS MATERIALS

The solubility of asphalt cement can be determined using carbon disulfide, carbon tetrachloride, benzene, or trichloroethylene. Trichloroethylene has the advantage of presenting no fire hazard and also has relatively low toxicity. Because of these factors, trichloroethylene is most often used in the solubility test for asphalt cements. Whichever solvent is used, the procedure is similar.

About 2 grams of the sample are dissolved in 100 milliliters of solvent. The mixture is poured into and washed through a tared Gooch crucible. The crucible is then dried and weighed. The increase in the weight of the crucible is the portion of the sample that is insoluble in the solvent.

The solubility test is a quality control test used in specifications to ensure getting an asphalt cement that is not contaminated with mineral matter or improperly refined.

SPOT TEST

The spot test (often called the Oliensis spot test) is a solubility test that takes advantage of the selective solvent action of certain hydrocarbons; usually standard naphtha is specified. The test is applicable only to petroleum asphalts and should not be applied to natural asphalts containing nonbituminous matter insoluble in xylene.

A 2-gram sample of asphalt is dissolved in 10 milliliters of naphtha. The mixture is thoroughly stirred with a stirring rod. A drop of the asphalt and solvent is then placed on a piece of Whatman No. 50 filter paper. After 5 minutes, the filter paper is examined. If the drop forms a yellowish-brown stain with a darker nucleus, the test is said to be positive. If the stain is uniformly brown, the test is negative. In the latter case, the sample is stoppered and set aside for 24 hours, and the test is repeated. If the stain with the darker nucleus again develops, the test is said to be positive and is so reported. A negative result is an indication that the asphalt sample is a homogeneous material. A positive result may indicate that the sample is not a homogeneous material. A negative result is regarded as favorable to the sample; a positive result is unfavorable and may be used to reject the asphalt.

Considerable importance is attached to the spot test by some asphalt technologists and paving engineers. It is relied on principally as a means of detecting a cracked asphalt, which is a nonhomogeneous material not regarded as a good paving asphalt. The test can also be used to detect an asphalt that has been overheated or coked. Overheating or coking can occur in storage tanks or when the asphalt is added to aggregate that is too hot. In such cases the asphalt is no longer a homogeneous substance, and the spot test will often show a nonuniform (positive) stain.

THIN-FILM OVEN TEST

The thin-film oven test was developed to overcome the deficiencies of the standard loss-on-heating test. The test uses the same oven as the loss-on-heating test except for a modification of the rotating shelf. The test has the same period and temperature of heating (5 hours at 325°F). The significant difference is in the sample. Instead of the 50-gram sample in a 3-ounce ointment can, a ⅛-inch thick layer of asphalt is poured into a wide, shallow aluminum dish. The dish has a flat bottom and is 5½ inches in diameter and ⅜ inch deep. The sample is weighed before and after the heating period and the loss in weight is computed. A penetration or viscosity test may also be conducted on the sample after the heating period to evaluate changes in the asphalt.

Section V. TESTS ON AGGREGATE AND FILLERS

DISCUSSION

The aggregate transmits the load, takes the abrasive wear of traffic, and provides a nonskid surface.

Desired aggregate characteristics include angular shape, rough surface, hardness and gradation. Some of these characteristics, such as shape, surface, and cleanliness, are determined visually. Durability and hardness cannot be seen but require knowledge based either on experience or some form of abrasion testing. Rocks that soak up water will eventually reach the condition where the binder is forced from the surface pores and the cementing action breaks down. Gradation may be established to some extent by observation. However, the grain-size distribution (sieve) tests will define the particle sizes and amounts much more accurately.

In bituminous paving, the aggregate constitutes the bulk of the pavement. Common practice subdivides the bituminous aggregates into a coarse aggregate, a fine aggregate, and a mineral filler. The No. 10 sieve separates coarse from fine aggregate, and the No. 200 sieve size is the lower limit for fine aggregate. Usually 65 percent or more of the mineral filler will pass the No. 200 sieve. The distribution of the different sizes determines how many voids will remain and aids in determining 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 which are defined in the specifications.

SIEVE ANALYSIS

A sieve analysis of the aggregates to be used in a paving mixture is required to determine the particle-size distribution.

Mineral Filler

In bituminous paving, particles finer than the No. 200 sieve are referred to as a mineral filler. To measure the amount of filler in a selected sample, a washed sieve analysis is performed using the No. 40 and No. 200 sieves. After all the material possible has passed the No. 200 sieve and has been discarded, the materials on the sieves are returned to the original washed sample, oven dried, and weighed. The amount of mineral filler is computed as:

Table 3-5. Aggregate sizes and weights

Approximate aggregate size	Approximate dry weight of sample, grams
Fine aggregate	
At least 95% finer than No. 10*	100
At least 90% finer than No. 4 and more than 5% coarser than No. 10*	500
Coarse aggregate	
3/8-inch maximum size	1,000
1/2-inch maximum size	2,500
3/4-inch maximum size	5,000
1-inch maximum size	10,000
1 1/2-inch maximum size	15,000
2-inch maximum size	20,000
2 1/2-inch maximum size	25,000
3-inch maximum size	30,000
3 1/2-inch maximum size	35,000

***Note:** ASTM specifies No. 8, 16, 30, and 50 instead of No. 10, 40 and 60. Tests based on ASTM standards identify the appropriate sieve sizes.

$$\text{percent finer than No. 200} = \frac{\text{original dry weight-washed dry weight}}{\text{original dry weight}} \times 100$$

Fine and Coarse Aggregate (Washed)

When definite amounts or limits of coarse and fine aggregate are specified, the sieve analysis with prewashing must be made using suitable sieves. If no limits have been designated, a range of sieves is selected to give adequate information about gradation. The results are recorded on DD Form (Figure 2-47, page 2-46) and then plotted as a gradation curve on DD Form 1207 (Figure 2-49, page 2-50). In testing aggregates, a representative sample is obtained by quartering, if necessary. The minimum size of the sample depends on the maximum size of particles in the material (Table 3-5.)

SPECIFIC GRAVITY

The specific gravities of aggregates and mineral filler used in bituminous paving mixtures are required to compute the percent of air voids and percent of voids filled with bitumens. Apparent specific gravity used with aggregate blends showing water absorption of less than 2½ percent is based upon the apparent volume of the material, which does not include those pore spaces in the aggregate which are permeable to water. Bulk-impregnated specific gravity is used for aggregate blends with 2½ percent or greater water absorption. The methods for determining absorption of aggregates are described in Chapter 4.

APPARENT SPECIFIC GRAVITY

The apparent specific gravity may be determined using the method described for apparent and bulk specific gravity, or it may be determined using the Dunagan apparatus furnished with the concrete test set.

Apparatus

The apparatus using the concrete test set includes:

Dunagan apparatus (Figure 3-10)

Sieves (2, 1½, ½, ⅜, and Numbers 4, 10, 40, 60, 80, 100, 200)

Evaporating dishes

Electric oven

Pans.

Procedure

- Select approximately 5,000 grams of aggregate from the sample, not including particles smaller than the ⅜-inch sieve. A ⅜-inch sieve is provided.

- Wash the aggregate to remove any dust or other coating and dry it to constant weight in the oven. Record the total weight of the oven-dry aggregate on DD Form 1216 (Figure 3-6).

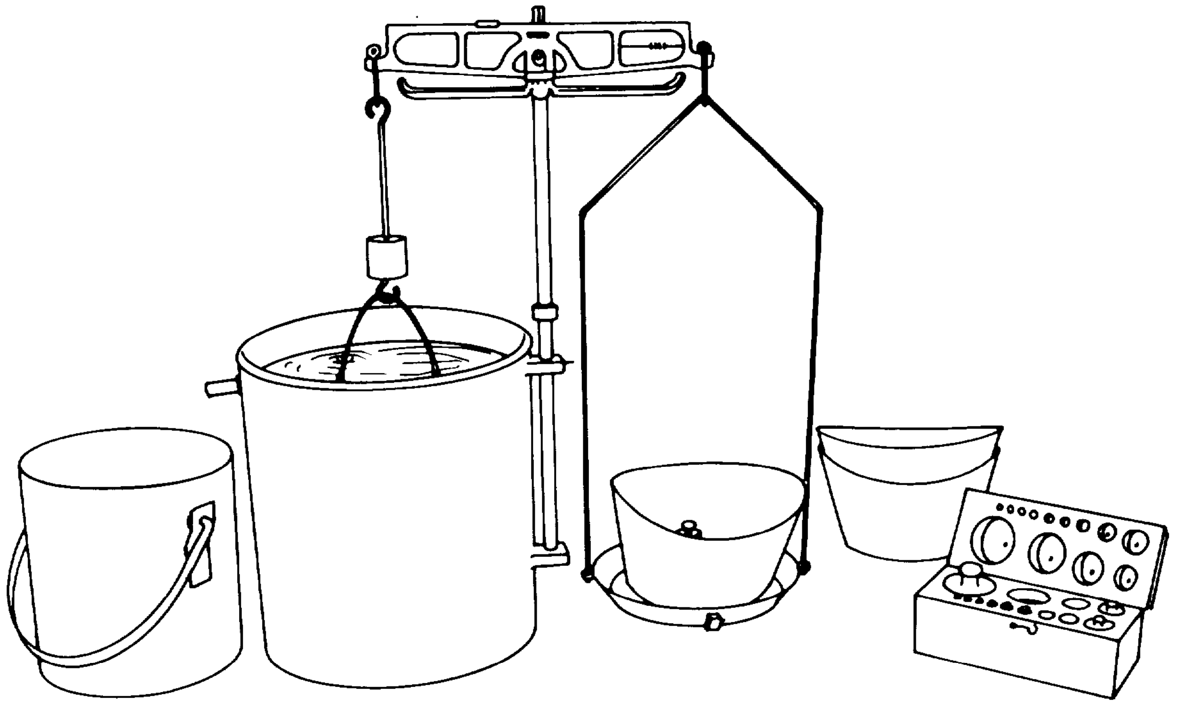


Figure 3-10. Specific gravity test, Dunagan apparatus

Immerse the aggregate in water at 15° to 25°C for a period of 24 hours.

- Soak the sample and place it in the copper bucket which is filled with water. Turn the bucket and aggregate sharply back and forth to assist in removing any air.

- Suspend the bucket from the brass hanger and bring the water level to the overflow pipe.

- Determine the submerged weight using weights placed in the scoop on the right-hand pan. Record the weights.

Calculations

The calculations required to determine the apparent specific gravity of coarse aggregate are shown on DD Form 1216 (Figure 3-6, page 3-21) and are self-explanatory.

APPARENT SPECIFIC GRAVITY OF FINE AGGREGATE (CALIBRATED FLASK)

Apparatus

Balance, 2,000 grams

Evaporating dish

Battery filler

Volumetric flask, 500 milliliters

Electric oven

Pan

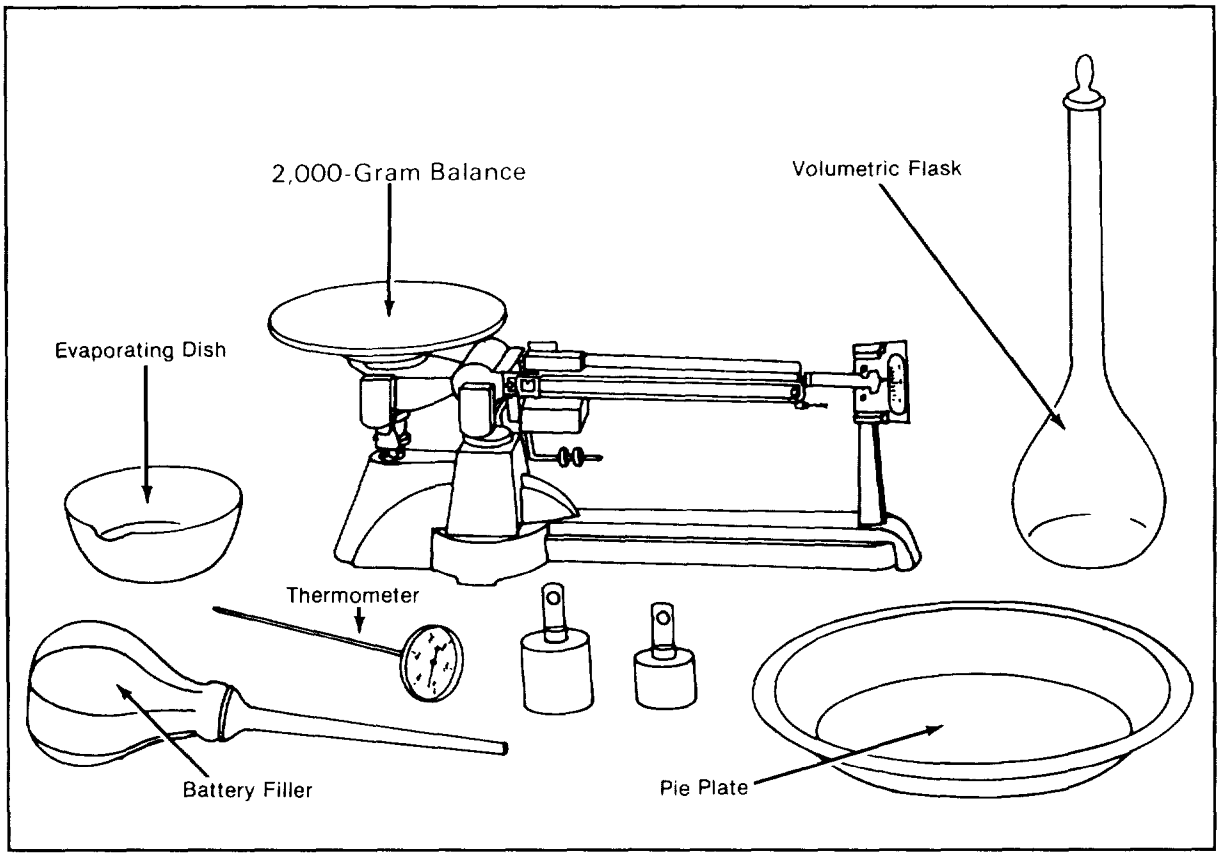


Figure 3-11. Calibrated flask apparatus for determining apparent specific gravity of fine aggregate

Thermometer (0° to 300°F, in 1° gradations)
Absorbent paper or cloth

This equipment, with the exception of the oven, is shown in Figure 3-11.

Procedure

The test procedure for particles finer than the ⅜-inch sieve is as follows (data is recorded on DD Form 1208, Figure 2-52, page 2-56):

- Calibrate a 500-milliliter volumetric flask.
- Dry a representative sample weighing about 500 grams to constant weight in the electric oven.
- **Determine the oven-dry weight of the cooled sample and record it on DD Form 1208, Figure 2-52.**
- Transfer the sample to the 500-milliliter flask, being careful not to lose any of the material.
- Add clean water until the level just reaches the neck of the flask. Allow the sample to soak for 24 hours.
- Hold the flask containing the soaked sample by the neck and roll it back and forth on a smooth surface until air bubbles cease to come from the sample. This procedure is illustrated in Figure 3-12.
- Use the battery filler to bring the water level up in the neck of the flask until the bottom of the meniscus coincides with the calibration mark on the flask. Use absorbent paper or cloth to remove any drops of water from the inside of the neck and on the outside of the flask.
- **Determine the weight of the flast, aggregate, and water and record it on DD Form 1208, Figure 2-52.**
- Measure and record the temperature.

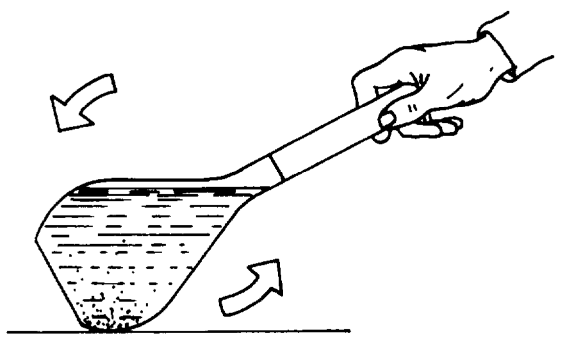


Figure 3-12. Manipulation of calibrated flask to remove air

Calculations

The calculations for apparent specific gravity are the same as those indicated for soil in Figure 2-52, page 2-56.

APPARENT SPECIFIC GRAVITY OF FINE AGGREGATE (UNCALIBRATED FLASK)

When a calibrated flask is not available, the following procedure can be used.

Apparatus

The same equipment is required as for the test specified above, with the addition of a water bath maintained at 68°F (20°C). This procedure cannot be used unless the temperature can be maintained.

Procedure

Conduct the test using the following steps:

1. Obtain a representative sample of aggregate passing the ⅜-inch sieve weighing about 500 grams. Dry it to constant weight in the electric oven maintained at a temperature of 212°F (100°C) to 230°F (110°C).

2. Obtain and record the dry weight after the sample has cooled in air.

3. Transfer the sample to the flask, being careful not to lose any of the material.

4. Add clean water until its level just reaches the neck of the flask, and allow the sample to soak for 24 hours.

5. Roll the flask back and forth on a smooth surface, after the sample has soaked for 24 hours, until air bubbles cease to come from the sample, as illustrated in Figure 3-12, page 3-31.

6. Use the battery filler to bring the water level up in the neck of the flask to slightly above the calibrated mark.

7. Place the flask with water and aggregate in the water bath maintained at a temperature of 68°F (20°C). Use the glass thermometer to check the temperature of the water in the flask from time to time.

8. Bring the water in the flask to a uniform temperature of 68°F (20°C), and 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.

9. Remove the flask from the bath and dry the outside thoroughly. Determine and record the weight of the flask plus the aggregate, plus the water at 68°F (20°C). It does not matter if the level of the water in the neck of the flask changes after removal from the bath. The proper adjustment was made at 68°F (20°C), and the total weight is not affected by the subsequent change in volume.

10. Repeat the procedures in steps 6 through 9 above using water only. Enter the weight of the flask filled with water at 68°F (20°C) on DD Form 1216. This needs to be done only once for a given flask; this value can be tabulated and used in subsequent tests. It is required because the weight of the flask filled only with water at 68°F (20°C) must be known.

Calculations

Indicate calculations necessary to determine the apparent specific gravity of fine aggregate using an uncalibrated flask on DD Form 1216, Figure 3-6, page 3-21.

SPECIFIC GRAVITY OF BULK-IMPREGNATED AGGREGATE

This test is intended for use in determining the specific gravity of the blended aggregates (including filler) used in hot asphaltic mixtures. This method is to be used only when the water absorption for the entire blend of aggregate selected for the job mix formula exceeds 2½ percent. The method is not applicable to determine specific gravity of mineral filler except when included in the blended aggregate. See method 105, MIL-STD-620A, for additional testing details.

Apparatus

The following equipment is required for performing this test:

Dunagan concrete test apparatus

Electric oven (sensitive to ± 5°F in the range of 275°-325°F)

Gallon capacity pails

Balance, 5-kilogram capacity sensitive to 0.1 gram

Bake pans

Heavy sheet metal strip for stirring contents of the pail.

A wire handle is convenient for handling the pail, but is not essential, since the container will be placed in the copper bucket of the Dunagan apparatus to determine weight submerged in water. A No. 10 can (an empty fruit or vegetable can) with the top smoothly cut out is satisfactory, but care must be taken to eliminate air trapped under the bottom when the can is submerged.

Samples should consist of 1,500 grams of blended aggregate (ensuring that the sample represents prototype grading), and 85-100 penetration grade asphalt cement.

Procedure

The procedure is as follows:

- Dry the aggregate sample to constant weight at a temperature not less than 230°F nor greater than 290°F. After cooling the sample in air, weigh it to the nearest 0.1 gram.
- Heat the asphalt to 280° ± 5°F using care to ensure that the temperature never exceeds 285°F, and add a sufficient amount to the 1-gallon pail to fill it about $\frac{1}{3}$ full.
- Insert the sheet metal stirrer and allow the pail and its contents to cool to room temperature. Allow 8 hours for cooling (preferably overnight).
- Weigh the pail plus the asphalt and stirrer in air at room temperature and submerged in water at 72°F ± 2°F.
- Place the pail of asphalt with stirrer and also the sample of aggregate in an oven at 280° ± 5 °F until temperatures of both are equalized. (A minimum of 4 hours is usually required.)
- Remove the aggregate and asphalt from the oven, and gradually add aggregate to the asphalt, stirring thoroughly. After all the aggregate is added, continue stirring until the total elapsed time from the start of mixing to the end of stirring is 2 minutes. During the cooling period, apply a flame to the surface to remove air bubbles. Cool the sample to room temperature (preferably overnight).
- Weigh the sample in air and in water at 72° ± 2°F.

Calculations

Calculate the bulk-impregnated specific gravity as follows:

$$\text{bulk-impregnated specific gravity} = \frac{A}{(D - E) - (B - C)}$$

where:

A = weight of oven dry aggregate in grams
B = weight of pail plus stirrer plus asphalt in air
C = weight of pail plus stirrer plus asphalt in water
D = weight of pail plus stirrer plus asphalt plus aggregate in air
E = weight of soil plus stirrer plus asphalt plus aggregate in water.

Duplicate determinations should check within 0.04. If the values are within the 0.04 tolerance, an average value should be used. If the initial duplicate tests are not within the 0.04 tolerance, repeat tests should be made in duplicate. If the second set of test values is within the tolerance, the first two test values should be discarded, and an average value of the second two tests should be used.

SPECIFIC GRAVITY OF MINERAL FILLER

The specific gravity of mineral filler used in bituminous mixes is required for void computation. The methods described in the specific gravity test, including procedures, calibration, testing and calculations, apply. Note that when the bulk-impregnated specific gravity is used, the mineral filler is included in the blended aggregate.

LOS ANGELES ABRASION TEST

Apparatus

This test requires a special machine consisting of a revolving drum which is rotated at the rate of 30 to 33 revolutions per minute by an electric motor. Inside the drum is a shelf that picks up the aggregate sample along with a charge of steel balls and drops them together on the opposite side of the drum.

Procedure

The test is conducted using various numbers of drum revolutions, sizes of samples, and numbers of steel balls, depending on the grading of the sample. For a sample which passes the $^3/_4$-inch sieve and is retained on the $^3/_8$-inch sieve, 5,000 grams of material is used with 11 balls and 500 revolutions. The sample should be washed and dried to constant weight prior to placing it in the machine. After the 500 revolutions are completed, the sample is removed from the drum and sieved over a No. 12 sieve. The portion retained on the No. 12 sieve is washed, dried to constant weight, and weighed. The difference between the original weight and the final weight of the sample is expressed as a percentage of the original weight of the sample.

Calculations

Job specifications usually require that the loss in weight as determined in the Los Angeles abrasion test shall not be greater than 40 percent for pavement aggregates and 50 percent for base and subbase aggregates.

Section VI. BITUMINOUS MIX DESIGN

DISCUSSION

Hot-mix bituminous concrete for pavements is a mixture of blended aggregate filled with bituminous cement binder. The materials are heated when mixing so that the bitumen becomes fluid and thoroughly covers the aggregate particles. The design of a bituminous concrete mix is an economical blend and gradation of aggregates with bituminous cement. This produces a mixture that will be durable, have the stability to withstand traffic loads, and be workable for placement and compaction with the construction equipment available.

The procedures described in this section are performed during the design of a hot-mix bituminous concrete. They include testing, plotting the results on graphs, and checking the readings against values from the design tables. Testing of the ingredients and the mix is started before and continued throughout the paving operations. TM 5-330 more fully describes the design considerations. TM 5-337 explains the paving operations and the blending and bitumen content criteria. This manual discusses the testing phases.

The selection of the mix ratios of materials is tentative. The bitumen should be the same as the one to be used in the construction. The aggregates and fillers must meet definite requirements. In general, several blends should be considered for laboratory mix-design tests.

At times it will be necessary to shorten the design procedures to expedite military construction. Suggestions for expediting design mix are given at the end of this chapter.

The final step is the preparation of a job-mix formula to be furnished to the construction unit.

GENERAL CONSIDERATIONS FOR HOT-MIX DESIGN

The objective is to determine the most economical blend of components which will produce a final product that meets specifications.

Procedures

- Prepare a sieve analysis of each of the aggregates available.
- Determine the aggregate blend which will achieve the specified gradation (TM 5-337). Plot the selected blend proportions on a graph with the allowable limits to see that the blend conforms.
- Determine the specific gravity of the components.
- Use selected percentages of bitumen (TM 5-337), make trial mixes and determine the design test properties of the mix.
- Plot the test properties on individual graphs using the selected bitumen percentages. Draw smooth curves through the plotted points.
- Select the optimum bitumen content for each test property from the curves (as explained in the criteria tables).
- Average bitumen content values from the previous step above, and from the graphs. Read the test property value corresponding to this average.
- Check these read values with the satisfactoriness-of-mix criteria.

Variables

Gradation specifications are based on limits established as satisfactory by the Corps of Engineers. Within these limits, the following variables will affect the final mix-design.

- Use of mix (surface course, binder course, or road mix).
- Binder (asphalt, cement, or tar).
- Loading (low tire pressure — 100 pounds per square inch and under or high tire pressure — over 100 pounds per square inch).
- Maximum size of aggregate (in stockpile, or based on the thickness of the pavement course).

Blends

Once the gradation specifications have been selected, the available materials are checked to determine how to proportion the blend to meet these specifications. Sieve analysis of the available aggregates are studied, and a series of trial blends is computed. Adjustment of the blend may have to be made after the design and prepared mix is tested. The considerations for establishing and adjusting the blend are explained in TM 5-337.

Optimum Bitumen Content

The determination of optimum bitumen content is based on the gyratory test method or the Marshall test method.

GYRATORY TEST METHOD

Purpose

The purposes of the gyratory test are to:

- Prepare specimens by kneading compaction at a pressure equal to the tire pressure for which the pavement is designed.
- Indicate optimum bitumen content directly by plasticity indicators called gyrograph recordings and by direct readings of shear resistance. The gyrograph recordings begin to widen, and the shearing resistance begins to decrease when

the maximum permissible bitumen content is exceeded.

- Measure shear at the applied tire pressure which is used to calculate a shear strength factor. This factor is used to predict whether the paving mixture will withstand the proposed tire contact pressure.

- Obtain (by direct measurement) unit weight values required to minimize settlement under the design loads. Unit weight calculations are based upon direct measurement of sample height and the known sample diameter.

Definitions

Gyratory test method terms are:

- Gyrograph – a recording of shear strain experienced by the bituminous mixture during the compaction test.

- Gyratory angle – a measure of the magnitude of the gyratory strain. Three pertinent angles are defined as follows:

  Initial gyratory angle or shear strain (machine setting) θ_0.

  Minimum gyratory angle or shear strain (minimum gyrograph band width) θ_i.

  Maximum gyratory angle or shear strain (maximum gyrograph band width) θ_{max}.

- Gyratory stability index (GSI) – the ratio of the maximum gyratory angle to the minimum gyratory angle.

- Gyratory compactibility index (GCI) – the ratio of the unit mass (total mix) at 30 revolutions of the gyratory testing machine, GTM, to the unit mass (total mix) at 60 revolutions of the GTM.

- Gryatory shear strength (S_G) – the shear resistance of the specimen under the imposed loading conditions.

- Gyratory shear factor (GSF) – the ratio of the measured gyratory shear strength to the approximate theoretical maximum induced shear stress.

Apparatus

The following equipment is required:

Gyratory testing machine (GTM) and appurtenances, which is the primary equipment for this test.

Spacer blocks. These are two metal spacer blocks used to zero the equipment that measures the specimen height. They are 2-inch (50.8-millimeter) diameter with one each of the following lengths: 2.50 ± 0.005 inch (63.50 ± 0.013 millimeter) and 3.75 ± 0.005 inch (95.25 ± 0.013 millimeter).

Oven, thermostatically controlled to maintain the required temperature within 5°F (2.8°C).

Hot plate, electric

Mixer, electric. A heavy duty commercial food mixer complete with mixing bowl and beaters.

Balances. Two balances are required, one having a capacity of 5 kilograms or more, sensitive to 1.0 gram; and one having a capacity of 2 kilograms or more, sensitive to 0.1 gram.

Thermometers, armored glass or dial type thermometers with metal stems are recommended. A range from 50° to 400°F (9.9° to 20.4°C) with sensitivity to 5°F (2.8°C) is required.

Beaker, metal, approximately 1,000-milliliter capacity

Tongs for beaker

Pan, metal 12 inches in diameter, 4 inches deep

Spoon, large metal, cooking

Scoop, kitchen

Paper disks, 4 inches (101.6 millimeters) and 6 inches (152.4 millimeters) in diameter

Gloves, work

Rags or paper towels

Kerosene (asphalt solvent)

Creosote (tar solvent).

Procedure

The gyratory method is applicable to mixtures containing asphalt cement, asphalt cutback, asphalt emulsion, tar or rubberized tar, and aggregate up to 1-inch (25.4-millimeter) maximum size in the 4-inch (101.6-millimeter) diameter specimen and 1.5-inch (38.1-millimeter) maximum size in the 6-inch (152.4-millimeter) diameter specimen.

- Selection of bitumen contents. The bitumen content is expressed as a percent of the total weight of the mixture. Using the procedures outlined below, conduct preliminary tests with one specimen each at a minimum of three bitumen contents: one above, one below, and one at the estimated optimum. Once the range of bitumen contents for the design test has been selected, then test at least four specimens at each of the selected bitumen contents. The formulas listed in the surface area method may be used to make a rough estimate of optimum bitumen content. The GTM indicates excessive bitumen by the widening of the gyrograph and the reduction in upper roller pressure during the compaction test. In these preliminary tests, bracket the optimum bitumen content by tests in which these phenomena occur at the higher bitumen contents. The incremental change of the bitumen content should be generally 0.5 percent. For extremely critical mixes, lower the incremental change of bitumen content to 0.3 percent. For highly absorptive aggregate, increase the incremental change of bitumen content to 1.0 percent. The gyratory method does not use voids criteria to select the optimum bitumen content. However, the mix must be sufficiently dense (low in voids) to widen the gyrograph and reduce the roller pressure since these indicate overfilled voids. For this reason, the gyratory method selects mixtures with the most desirable durability properties, the maximum permissible bitumen content, and the minimum acceptable voids content.

- Preparation of aggregates. Procedures for determining particle size distribution and blending to meet specifications requirement have been discussed. The amount of aggregate required is discussed below.

- Preparation of mixtures. For mixes employing penetration grades of bitumen, the temperature of the aggregate and asphalt at the time of mixing should correspond to the temperatures anticipated at the plant during the manufacture of the paving mix. These temperatures will be in the range of 200°F (93° to 177°C).

For tar and rubberized tar, the temperature of the aggregate and the binder at the time of mixing should correspond to the temperature to be used at the plant during manufacture of the paving mix. This temperature will generally not exceed 225°F (107°C) for tar mixes and 250°F (121°C) for rubberized tar mixes.

For mixtures employing liquid asphalts (cutbacks or emulsions), the aggregate should be dried to the moisture content expected during construction (up to a maximum of 2 percent by dry weight). The liquid asphalt should be combined with the aggregate at the temperature recommended for field application. Following mixing, cure the loose mixture in a ventilated oven maintained at 140° ± 5°F (60° ± 2.8°C) for at least 12 hours prior to compaction at this temperature. The mix should be stirred occasionally during curing to accelerate the loss of volatiles.

Combine the aggregates into batches large enough to make specimens approximately 2.50 inches (63.5 millimeters) long in the 4-inch (101.6-millimeter) diameter mold and 3.75 inches (95.3 millimeters) long in the 6-inch (152.4-millimeter) diameter mold. For normal aggregates, this will require approximately 1,200 grams for the 4-inch (101.6-millimeter) diameter specimen and approximately 4,050 grams for the 6-inch (152.4-millimeter) diameter specimen. Heat the aggregate to the proper mixing temperature. Then weigh the required amount of bitumen at the proper temperature into the aggregate mixture. Mix the aggregate and bitumen as thoroughly and rapidly as possible. Mechanical mixing is recommended.

- Compaction and shear test. For this test, set the initial gyratory angle, 0_0, at 1°. Roller positions 2 and 4 are used setting the initial gyratory angle, 0. A trial batch of mix is used in making the 0_0 adjustment. Make certain that the specimen molds are thoroughly clean and free of defects. Excessive wear or grooving in the molds in the area of contact with the upper and lower plates will have an adverse effect upon the compaction as well as the gyrograph (shear strain) recording. Instructions for the compaction temperatures for the laboratory specimens are presented above. Set the GTM at 140°F (60°C) at least 15 minutes before starting the compaction test. Preheat the mold and base plate at 140°F (60°C). Place paper disks in the bottom of the mold and on top of the loose mix to prevent the bitumen from adhering to the end plates. Place the entire batch in the mold. Avoid hand troweling or tamping so that the compaction process will be completely controlled mechanically and will be the most precise and reproducible. Use the mold-carrying tray to load the mold containing the mixture into the machine, after first placing the wall friction yoke in position. Raise the ram and use just enough pressure to retain the specimen while the front of the mold chuck is tightened securely in position. When the mold chuck is securely tightened, increase the vertical pressure to the full compaction test pressure. Now bring the gyrograph recorder pin into contact, actuate the roller carriage, and continue until 29 revolutions have been applied. At the completion of 29 revolutions, stop the carriage and record the specimen height and roller pressure readings at three positions: 1, 3, and 4 (29 to 30 revolutions) thus completing 30 revolutions. Continue to apply additional revolutions until a total of 59 is reached. Again record the height and roller pressure readings at three positions: 1, 3, and 4 (59 to 60 revolutions) thus completing 60 revolutions.

- Wall friction test. Immediately following the compaction and shear test, lower the vertical ram slightly to relieve the pressure on the bottom roller. Lower the bottom roller enough turns to ensure it will be out of contact with the mold chuck. (Keep account of the exact number of turns so the roller can be again reset to exactly the same position.) Reapply the compaction pressure to the ram, and cycle the roller carriage several times to level the specimen. With the compaction pressure still acting on the specimen, loosen the mold chuck bolts and remove the front section of the chuck so that the specimen mold is no longer restrained by the chuck. Install the two wall friction apparatus jacks beneath the wall friction yoke. With the vertical load acting on the specimen, determine the force required to overcome wall friction and move the mold by observing the pressure gage of the jack while actuating the jack. The pressure reading will increase with each thrust of the jack until there is enough force to move the mold. The pressure reading will then stabilize to about the same minimum value after each thrust of the jack. Record the low reading of the wall friction gage in the space provided. Immediately after the wall friction test is completed, remove the test specimen from the GTM and bring the lower roller brought back to the 1°-setting so that the machine is ready for the next test specimen.

- Calibration of machine. In conducting shear tests with the gyratory testing machine, it is necessary to make machine corrections for the gyratory shear value S_G. For this correction, shift the Mohr's diagram for test results on a cohesionless material enough to cause the envelope to pass through the origin of the Mohr's diagram. The cohesionless material used for this test is standard dry ottawa sand, all passing a No. 20 (0.84 millimeter) sieve and retained on a No. 40 (0.42 millimeter) sieve. A correction is needed for each combination of compaction pressure and gyratory angle used in the GTM compaction and shear tests. This correction is determined only once for any combination of vertical pressure and gyratory angle. The dry ottawa sand is first compacted under the same pressure, gyratory angle, and number of revolutions as scheduled for the compaction and shear tests on a given bituminous mixture. The shear test on the dry sand is then conducted for at least three different magnitudes of vertical pressure: starting at some lower value, including an intermediate value, and finally using the same value that was used for compaction. The roller carriage is cycled once after each incremental adjustment in vertical pressure and before reading the upper roller values under that pressure.

Calculations and Presentation of Results

- Calculations for compaction. Calculate the following compaction properties for each specimen:

  Unit mass, total mix
  Unit mass, aggregate only
  Gyratory compactability index (GCI)

- Calculations for shear. Calculate the following gyratory shear properties:

  Gyratory stability index (GSI)
  Gyratory shear strength (S_G)
  Gyratory shear factor (GSF).

- Graphical presentation. For convenience of analysis, the calculations above are plotted against the bitumen content. The graphs may be to any convenient arithmetic scales.

Applying the Gyratory Method

The bitumen content must be as high as possible when using the gyratory method to select the optimum bitumen content and judge the satisfactoriness of the mix.The (GSI) must not be significantly greater than one, and the (GSF) at this bitumen content must exceed one. On the basis of these criteria, this mix is judged to have acceptable shear strength, and the optimum bitumen is taken as 4.7 percent.

THE MARSHALL TEST METHOD

Purpose

The purposes of the Marshall test method are to:

- Prepare specimens by drop-hammer compaction. The number of drop-hammer blows used on the specimens is based on empirical correlations with two different traffic conditions: 50 blows on each end of the specimen for tires with less than 100 pounds per square inch (psi) pressure and 75 blows on each end of the specimen for tires with greater than 100 psi pressure.

- Find optimum bitumen content by averaging four measured properties: the peak of the compaction curve, the peak of the stability curve, the percent of the voids of the total mix at a specified amount, and the percent of the voids filled with bitumen at a specified amount. The values specified above for total mix and bitumen vary with the gradation of the aggregate, the absorption properties of the aggregate, and the compaction effort. There are ten separate sets of criteria to cover these variations. A special exception is made to use only voids total mix when the overall average falls outside the voids total mix limits. This occurs for open-graded mixes or highly porous aggregates. There are no standards for sand-asphalt mixes at 75-blow compaction.

- Measure maximum breaking load (stability) and corresponding deformation (flow) for specimens prepared according to the compaction procedure. These values of stability and flow are empirically correlated for conditions outlined in the paragraph above.

- Obtain the unit weights of specimens from calculations based upon weighing the specimens in air and in water. Porous specimens are coated with paraffin before weighing them in water.

Apparatus

The equipment required for the Marshall test is shown in Figure 3-13:

A - Mixing pan
B - Hot plate
C - Insulated gloves
D - Laboratory spatula
E - Trowel
F - Spoon
G - Insulated tongs
H - Scoop
I - Compaction hammer
J - Compaction mold
K - Filter paper
L - Extractor
M - Chemical resistant gloves
N - Trichloroethane technical
O - Holder, compaction mold
P - Compaction pedestal
Q - Triple beam scale
R - Wire basket
S - Thermometer
T - Hot water bath
U - Marshall stability testing machine
V - Flow indicator
W - Lab towels
X - Beaker with oil
Y - Brush
Z - Stability mold

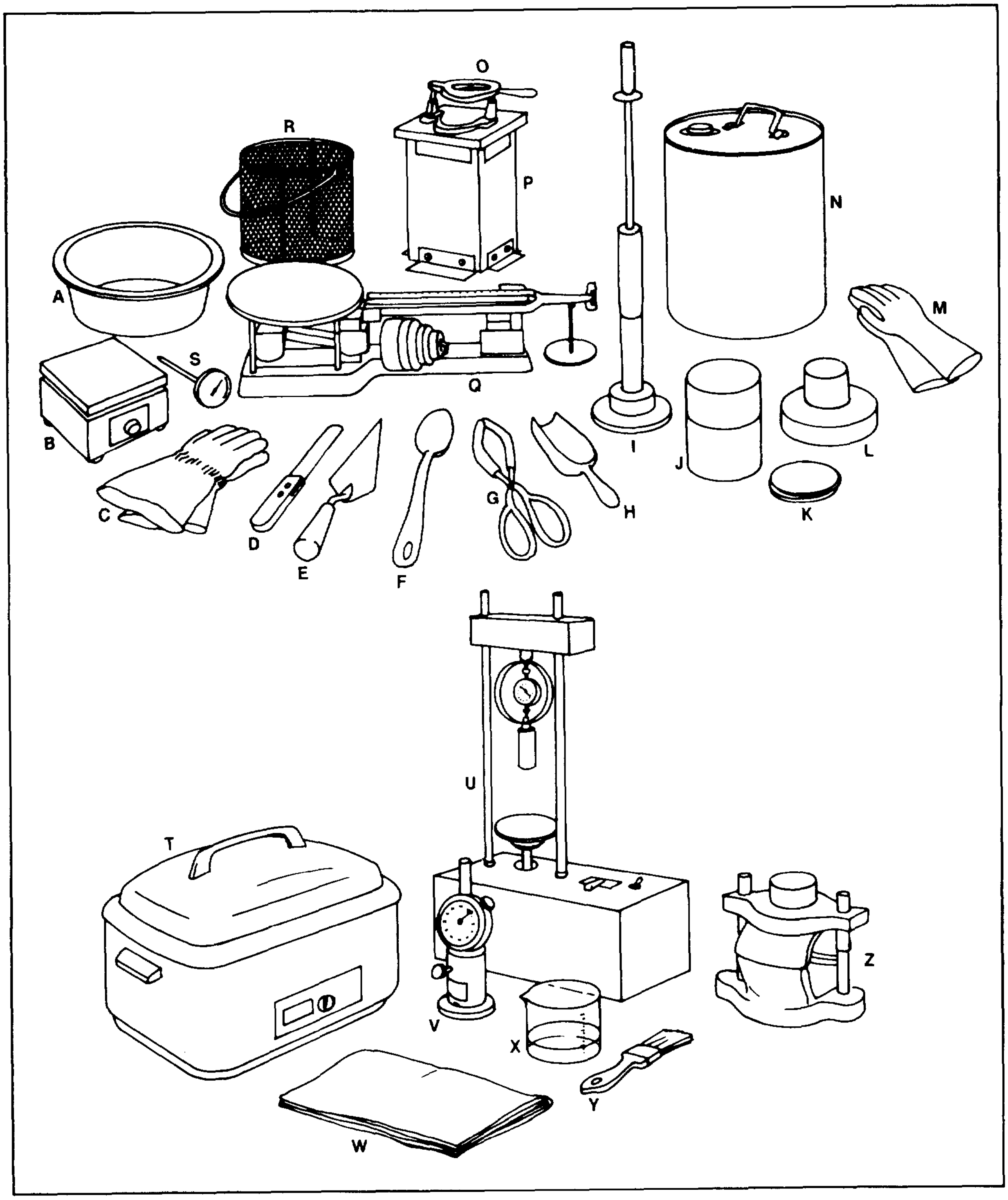

O
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Figure 3-13. Apparatus for bituminous mix design — Marshall method

Procedure

The Marshall method is applicable to hot-mix mixtures using penetration grades of asphalt cement and containing aggregate with not more than 10 percent of the aggregate larger than the 1-inch sieve. The procedure for handling large aggregate and that for cold mix is described later. The following procedure and example provide for the determination of the optimum asphalt content for one particular blend of aggregates.

- Selection of bitumen contents. The optimum amount of bitumen for the aggregate to be tested must be estimated in order to start the laboratory tests. Tests are continued until results show at least two bitumen contents above and two below what would be the optimum content. Since the optimum is not determined until after the results are plotted, specimens are prepared usually for each of six different contents. At least four specimens will be prepared at each bitumen content. One percent incremental changes of bitumen content may be used for preliminary work. However, increments of ½ of 1 percent will be used for final tests. The percent of bitumen will be expressed as a percent of the total weight of the batch of paving mix. TM 5-337 explains how to establish the estimated optimum content.

- Preparation of aggregates. Procedures for determining the particle size distribution and blending to meet specification requirements have already been presented. About 34 kilograms or 75 pounds of the selected blend should be dried to constant weight at 110° ± 5°C (221° to 239°F). This amount of material provides for 1,220 grams per specimen for 24 specimens (four in each of six bitumen contents) with allowance for some loss. The Marshall method uses a 4-inch diameter mold and is not applicable, without special handling, when more than 10 percent of the aggregate is larger than 1 inch. The above total amounts of aggregate required assumes less than 10 percent of the particles will exceed 1 inch. (Special handling of oversize aggregates is covered later.)

- Preparation of hot mixture. The bitumen and the aggregates must be heated to specified temperatures for mixing. These temperatures are based on the bitumen which will be used.

	Mixing Temperature	**Degrees Fahrenheit**
Bitumen Type	Aggregate	Bitumen
Asphalt cement	300 ± 5	270 ± 5
Tar (RT 10, 11, or 12)	225 ±	200 ± 5
Rubberized tar	250 ± 5	225 - 5

Bitumen should not be held at the mixing temperature for more than 1 hour before using. Therefore, the preparation should be planned so that the mixing will be done within this time limit. The mixing pans are preheated to a temperature about 50°F above the mixing temperature. The heated dry aggregate fractions are poured into the pans and mixed thoroughly. A crater is formed in the mixed aggregate, and the required amount of bitumen at the proper temperature is poured into the crater. At this point, the temperature of the ingredients should be within the limits specified above. Mixing of the aggregates and bitumen should be as rapid and as thorough as possible to get a uniform distribution of the bitumen throughout the aggregate.

- Compaction of hot mixtures. Four specimens are to be prepared at each bitumen content, and the molds should be ready to receive the specimens as soon as they are mixed. The striking face of the compaction hammers and the compaction molds must be thoroughly clean and heated to 180° to 250°F. Oiling the mold and other metal in contact with the mix before the mixture is introduced facilitates removal of the specimen after compaction. A silicone spray is convenient for this use. Parts should be wiped with a rag or paper towel prior to use. The mix is placed in the mold (rodding the material as it is added). After this the collar is removed, and the top surface of the mix is smoothed to a slightly rounded shape with a trowel. The thickness of the compacted specimen should be $2\frac{1}{2} \pm \frac{1}{8}$ inch. One or two trials will indicate the quantity of mix required to produce such a specimen. The collar is replaced, and the mold assembly is placed on the compaction pedestal. The temperature at this point must be as specified for compaction.

Bitumen Type	Compaction Temperature Degrees Fahrenheit
Asphalt cement	250 ± 5
Tar (RT 10, 11, or 12)	180 ± 5
Rubberized tar	200 ± 5

The required number of blows are applied with the compaction hammer (Figure 3-14). The base plate and collar are removed, and the mold is reversed and reassembled. The required number of blows are then applied to the other side of the specimen. For example, roads, streets, and facilities for an airfield designed for aircraft whose tires carry 100 psi or less should be compacted by 50 blows on each end of the specimen. If the pavement is being designed for aircraft which carry tires with pressure greater than 100 psi, the compactive effort should be 75 blows per side.

- Cooling of hot specimens. After compaction, the base plate and collar are removed, and the mold and the specimen are either air cooled (normally overnight) or placed in cold water for a minimum of 2 minutes for fast cooling. The cooled specimen is removed from the mold by 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° oven for a few minutes just before ejecting the specimen. The specimen, carefully handled and suitably identified, is placed on a smooth surface until it is ready for testing as described below.

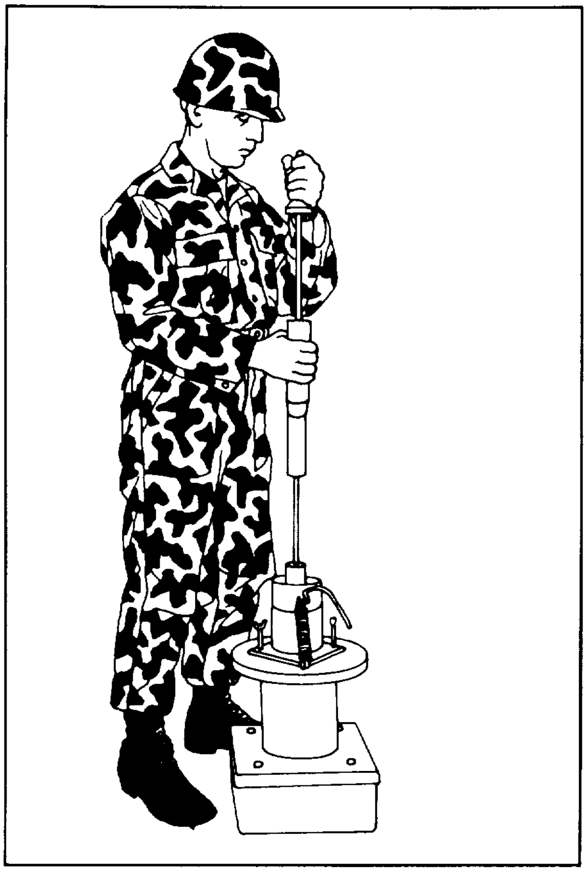


Figure 3-14. Compaction of bituminous trial mix specimens

- Weighing specimen in air and in water. Each specimen is weighed in air and in water (Figure 3-15, page 3-44) in order to obtain the weight and volume measurements used in calculating the unit weight of the compacted mix. The DD Form 1218 (Marshall Method - Computation of Properties of Asphalt Mixture), Figure 3-16, page 3-45, provides space for recording those measurements which are made at room temperature. A direct weight in water of open-textured or porous specimens will give erroneous results because of the penetration and absorption of water. For such specimens, other means must be used to determine the volume of the specimen. One means of measuring the volume of the porous specimens is to coat the specimens with paraffin to seal all the voids and then weigh the coated specimens in air and in water. (See Figure 2-52, page 2-56, for typical data sheet.)

- Measuring stability and flow. The test specimens are brought to the desired temperature for the test by immersing them in a oven for at least 2 hours. The bath temperature for asphalt samples is 140° ± 1°F; for tar samples (RT-10 to RT-12) is 100° ± 1°F; and for tar rubber samples is 120° ± 1°F. Test measurements are recorded on the DD Form 1218, Figure 3-16.

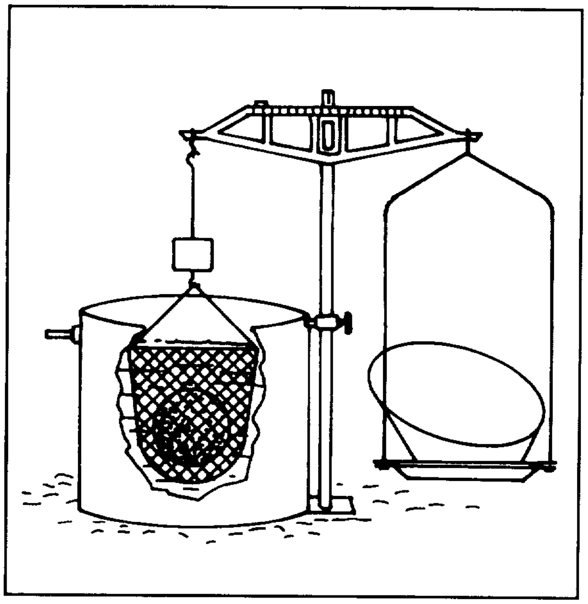


Figure 3-15. Weighing compacted bituminous specimen in water

The inside surfaces of the test heads and the guide rods are cleaned thoroughly prior to making the stability test, and the guide rods are well lubricated so that the upper test head will slide easily over the guide rods on the lower test head.

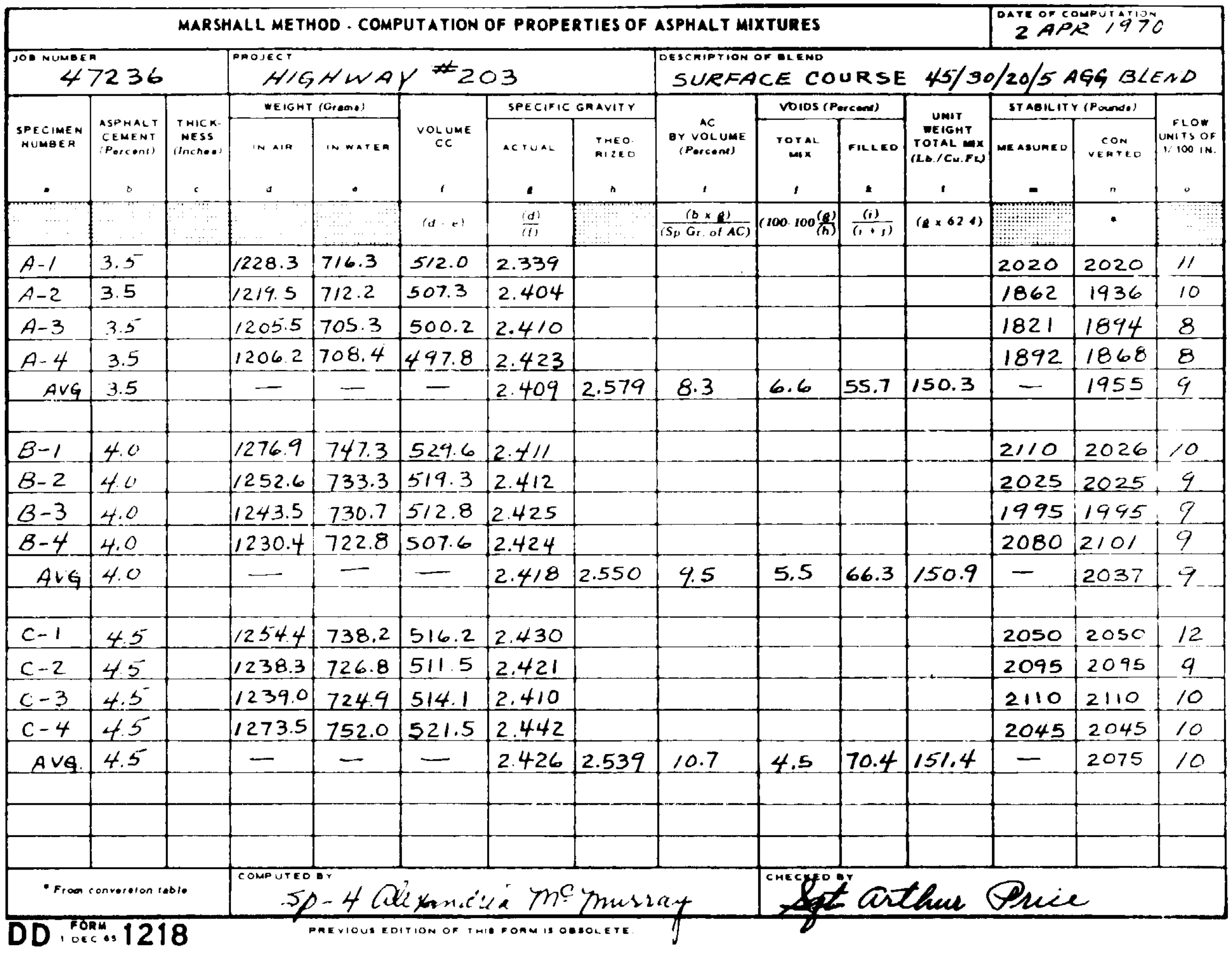
The specimen is removed from the water bath and placed on its side in the lower section of the breaking head. The upper section of the breaking head is positioned on the guide rods and on the specimen, and the complete assembly then placed in position in the testing machine (Figure 3-17, page 3-47). In order to prevent excessive cooling of this specimen with a resulting increase in stability value, the entire procedure from the time the specimen is removed from the water bath should be performed as quickly as possible, normally in 30 seconds.

The flowmeter is placed over one of the guide rods, and an initial reading taken on the flowmeter is estimated to 0.01 inch. The flowmeter is held firmly over the guide rod while loading the specimen. It is read or removed from its position over the guide rod just when the load first begins to decrease, as indicated by the dial gage in the proving ring. The data sheet, Figure 3-16, provides space for recording the flow valve which is the difference between the initial reading and the final reading.

Load is applied to the specimen at a constant rate of strain of 2 inches per minute until failure of the specimen occurs. The load builds up on the typical test as movement occurs, until it reaches a maximum and falls off. The maximum reading of the dial, converted to pounds, is the stability value for the specimen. The data sheet, figure 3-16, provides space for recording this reading.

Calculation and Presentation of Results

- The DD Form 1218, Figure 3-16, 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 that there are four duplicate tests for each bitumen content and that these four test values are averaged in each instance. Note also that the thickness of the specimen is not indicated since the volume can be used to find the stability correlation ratio from Table 3-6, page 3-47. The theoretical specific gravity is transferred from



MARSHALL METHOD - COMPUTATION OF PROPERTIES OF ASPHALT MIXTURES

DATE OF COMPUTATION: 2 APR 1970

JOB NUMBER: 47236

PROJECT: HIGHWAY #203

DESCRIPTION OF BLEND: SURFACE COURSE 45/30/20/5 AGG BLEND

SPECIMEN NUMBER	ASPHALT CEMENT (Percent)	THICKNESS (Inches)	WEIGHT (Grams)		VOLUME CC	SPECIFIC GRAVITY		AC BY VOLUME (Percent)	VOIDS (Percent)		UNIT WEIGHT TOTAL MIX (Lb./Cu.Ft.)	STABILITY (Pounds)		FLOW UNITS OF 1/100 IN.
			IN AIR	IN WATER		ACTUAL	THEORIZED		TOTAL MIX	FILLED		MEASURED	CONVERTED	
a	b	c	d	e	f	g	h	i	j	k	l	m	n	o
					(d - e)	$\frac{(d)}{(f)}$		$\frac{(b \times g)}{(\text{Sp Gr. of AC})}$	$(100 - 100\frac{(g)}{(h)})$	$\frac{(i)}{(i + j)}$	(g x 62.4)		*	
A-1	3.5		1228.3	716.3	512.0	2.339						2020	2020	11
A-2	3.5		1219.5	712.2	507.3	2.404						1862	1936	10
A-3	3.5		1205.5	705.3	500.2	2.410						1821	1894	8
A-4	3.5		1206.2	708.4	497.8	2.423						1892	1868	8
AVG	3.5		—	—	—	2.409	2.579	8.3	6.6	55.7	150.3	—	1955	9
B-1	4.0		1276.9	747.3	529.6	2.411						2110	2026	10
B-2	4.0		1252.6	733.3	519.3	2.412						2025	2025	9
B-3	4.0		1243.5	730.7	512.8	2.425						1995	1995	9
B-4	4.0		1230.4	722.8	507.6	2.424						2080	2101	9
AVG	4.0		—	—	—	2.418	2.550	9.5	5.5	66.3	150.9	—	2037	9
C-1	4.5		1254.4	738.2	516.2	2.430						2050	2050	12
C-2	4.5		1238.3	726.8	511.5	2.421						2095	2095	9
C-3	4.5		1239.0	724.9	514.1	2.410						2110	2110	10
C-4	4.5		1273.5	752.0	521.5	2.442						2045	2045	10
AVG.	4.5		—	—	—	2.426	2.539	10.7	4.5	70.4	151.4	—	2075	10

* From conversion table

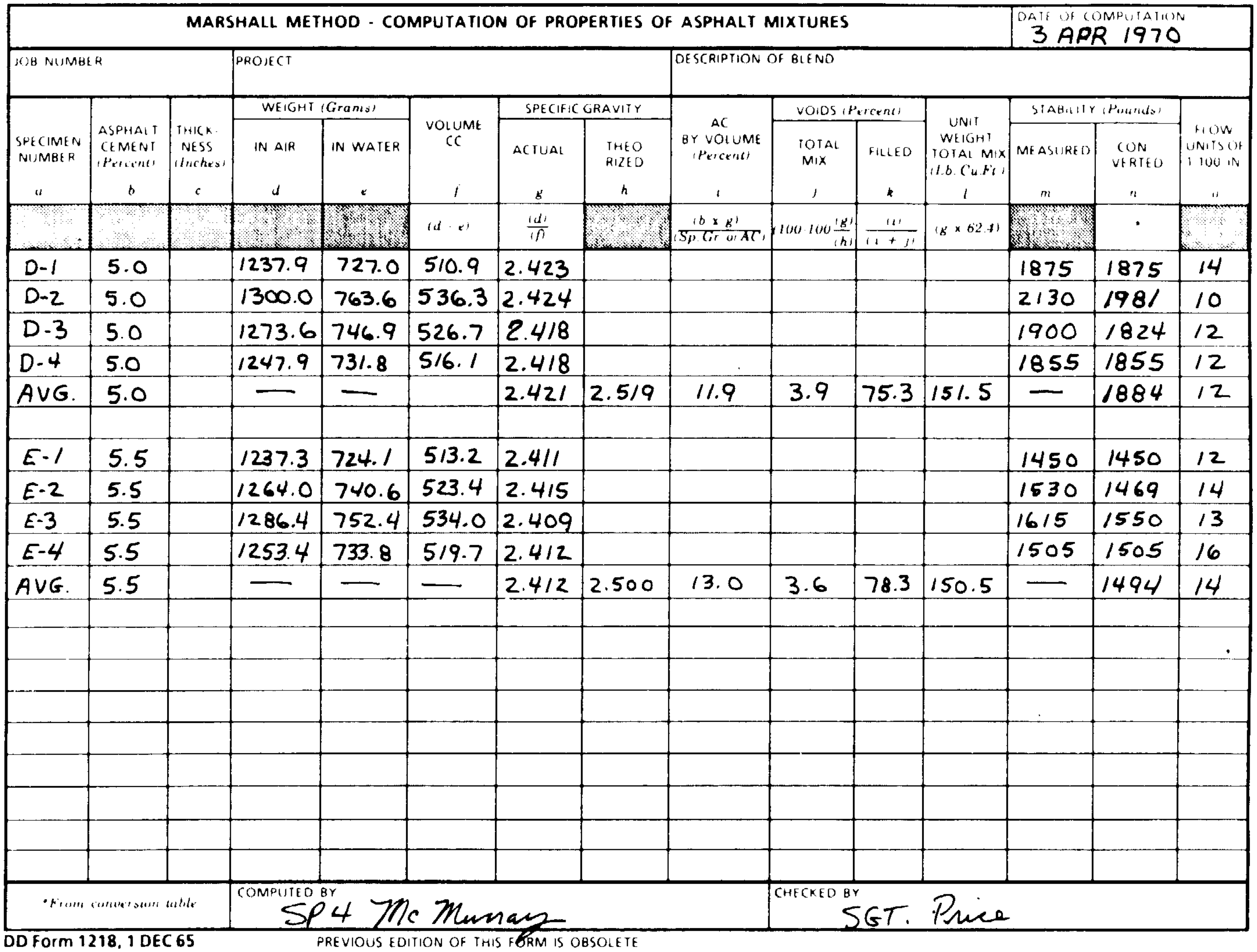
COMPUTED BY: Sp-4 Alexandria McMurray

CHECKED BY: Sgt Arthur Price

DD FORM 1 DEC 65 1218

PREVIOUS EDITION OF THIS FORM IS OBSOLETE.

Figure 3-16. Test results — Marshall stability test



MARSHALL METHOD - COMPUTATION OF PROPERTIES OF ASPHALT MIXTURES

DATE OF COMPUTATION 3 APR 1970

JOB NUMBER

PROJECT

DESCRIPTION OF BLEND

SPECIMEN NUMBER	ASPHALT CEMENT (*Percent*)	THICKNESS (*Inches*)	WEIGHT (*Grams*)		VOLUME CC	SPECIFIC GRAVITY		AC BY VOLUME (*Percent*)	VOIDS (*Percent*)		UNIT WEIGHT TOTAL MIX (*Lb. Cu.Ft.*)	STABILITY (*Pounds*)		FLOW UNITS OF 1/100 IN
			IN AIR	IN WATER		ACTUAL	THEORIZED		TOTAL MIX	FILLED		MEASURED	CONVERTED	
a	*b*	*c*	*d*	*e*	*f*	*g*	*h*	*i*	*j*	*k*	*l*	*m*	*n*	*o*
					$(d - e)$	$\frac{(d)}{(f)}$		$\frac{(b \times g)}{(Sp.Gr\ of\ AC)}$	$100 - 100\frac{(g)}{(h)}$	$\frac{(i)}{(i + j)}$	$(g \times 62.4)$		*	
D-1	5.0		1237.9	727.0	510.9	2.423						1875	1875	14
D-2	5.0		1300.0	763.6	536.3	2.424						2130	1981	10
D-3	5.0		1273.6	746.9	526.7	2.418						1900	1824	12
D-4	5.0		1247.9	731.8	516.1	2.418						1855	1855	12
AVG.	5.0		—	—		2.421	2.519	11.9	3.9	75.3	151.5	—	1884	12
E-1	5.5		1237.3	724.1	513.2	2.411						1450	1450	12
E-2	5.5		1264.0	740.6	523.4	2.415						1530	1469	14
E-3	5.5		1286.4	752.4	534.0	2.409						1615	1550	13
E-4	5.5		1253.4	733.8	519.7	2.412						1505	1505	16
AVG.	5.5		—	—	—	2.412	2.500	13.0	3.6	78.3	150.5	—	1494	14

**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

Figure 3-16. Test results — Marshall stability test — continued

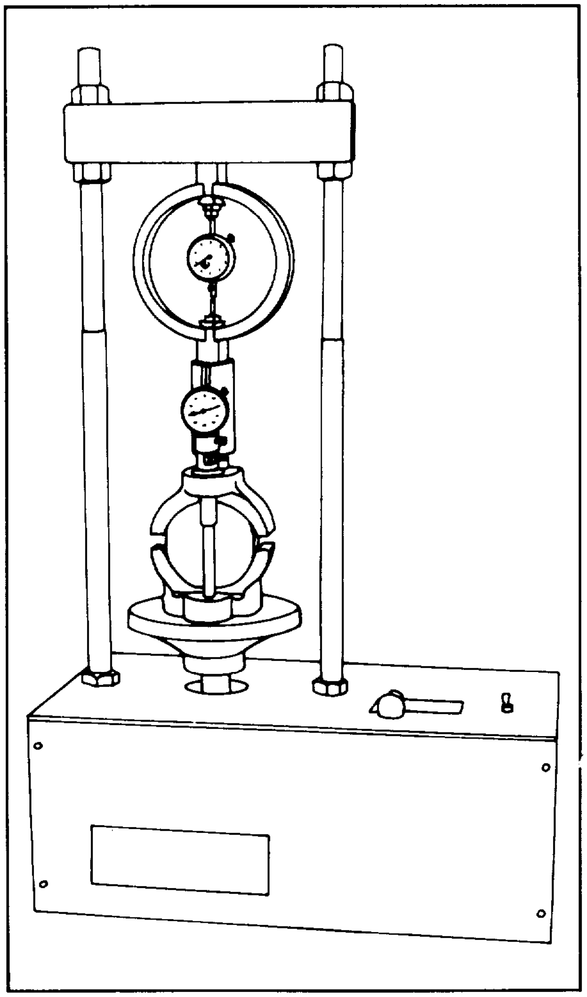
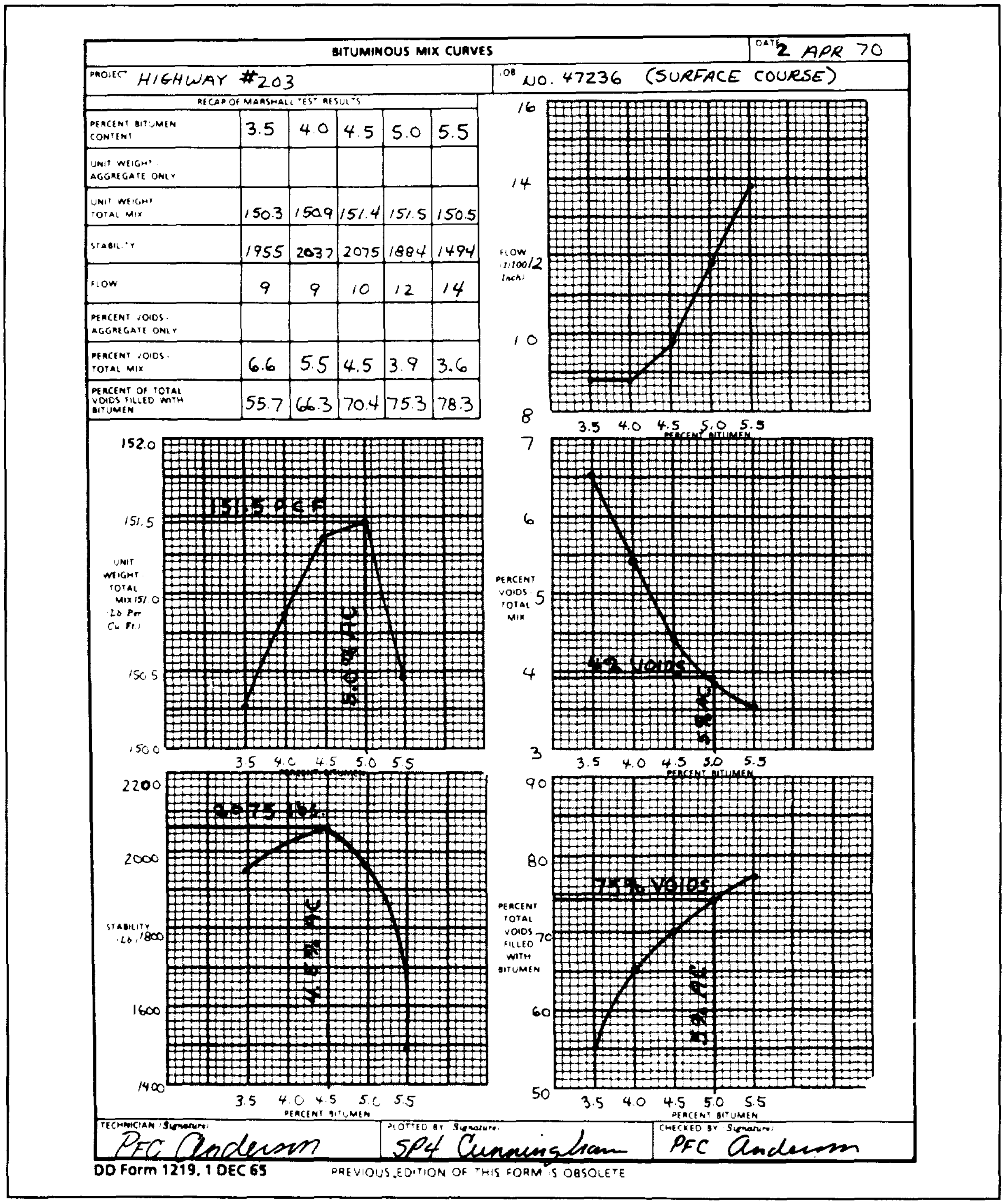


Figure 3-17. Stability test assembly

the DD Form 1218, Figure 3-16, to calculate the voids. Note the stability value is shown directly in pounds. Unless the testing machine provides a load measuring device that reads directly in pounds, it will be necessary to convert this value. Use the calibration factor furnished with the ring dynamometer on the testing machine. The stability value varies directly with the thickness of the specimen. Therefore, it is necessary to correct the stability values for specimens of a thickness greater or less than the standard 2½ inches. Table 3-6 shows the necessary conversion factors for specimens varying in thickness from 1 inch to 3 inches. Table 3-6 also contains data whereby the stability conversion factor can be determined on the basis of the volume of the specimen, since the volume is a direct function of height for a constant 4-inch diameter specimen. All other calculations are indicated directly on this data sheet (Figure 3-16).

Table 3-6. Stability correlation ratios, Marshall stability test

Volume of Specimen in Cubic Centimeters	Approximate Thickness of Specimen in Inches	Correlation Ratio
200-213	1	5.56
214-225	$1\frac{1}{16}$	5.00
226-237	$1\frac{1}{8}$	4.55
238-250	$1\frac{3}{16}$	4.17
251-264	$1\frac{1}{4}$	3.85
265-276	$1\frac{5}{16}$	3.57
277-289	$1\frac{3}{8}$	3.33
290-301	$1\frac{7}{16}$	3.03
302-316	$1\frac{1}{2}$	2.78
317-328	$1\frac{9}{16}$	2.50
329-340	$1\frac{5}{8}$	2.27
341-353	$1\frac{11}{16}$	2.08
354-367	$1\frac{3}{4}$	1.92
368-379	$1\frac{13}{16}$	1.79
380-392	$1\frac{7}{8}$	1.67
393-405	$1\frac{15}{16}$	1.56
406-420	2	1.47
421-431	$2\frac{1}{16}$	1.39
432-443	$2\frac{1}{8}$	1.32
444-456	$2\frac{3}{16}$	1.25
457-470	$2\frac{1}{4}$	1.19
471-482	$2\frac{5}{16}$	1.14
483-495	$2\frac{3}{8}$	1.09
496-508	$2\frac{7}{16}$	1.04
509-522	$2\frac{1}{2}$	1.00
523-535	$2\frac{9}{16}$	0.96
536-546	$2\frac{5}{8}$	0.93
547-559	$2\frac{11}{16}$	0.89
560-573	$2\frac{3}{4}$	0.86
574-585	$2\frac{13}{16}$	0.83
586-598	$2\frac{7}{8}$	0.81
599-610	$2\frac{15}{16}$	0.78
611-625	3	0.76



BITUMINOUS MIX CURVES

DATE 2 APR 70

PROJECT HIGHWAY #203

JOB NO. 47236 (SURFACE COURSE)

RECAP OF MARSHALL TEST RESULTS

PERCENT BITUMEN CONTENT	3.5	4.0	4.5	5.0	5.5
UNIT WEIGHT AGGREGATE ONLY					
UNIT WEIGHT TOTAL MIX	150.3	150.9	151.4	151.5	150.5
STABILITY	1955	2037	2075	1884	1494
FLOW	9	9	10	12	14
PERCENT VOIDS AGGREGATE ONLY					
PERCENT VOIDS TOTAL MIX	6.6	5.5	4.5	3.9	3.6
PERCENT OF TOTAL VOIDS FILLED WITH BITUMEN	55.7	66.3	70.4	75.3	78.3

FLOW (1/100 Inch)
16
14
12
10
8
3.5 4.0 4.5 5.0 5.5
PERCENT BITUMEN
UNIT WEIGHT TOTAL MIX (Lb Per Cu Ft)
152.0
151.5
151.0
150.5
150.0
151.5 PCF
5.0% AC
3.5 4.0 4.5 5.0 5.5
PERCENT BITUMEN
PERCENT VOIDS TOTAL MIX
7
6
5
4
3
4% VOIDS
5% AC
3.5 4.0 4.5 5.0 5.5
PERCENT BITUMEN
STABILITY (Lb)
2200
2000
1800
1600
1400
2075 lbs.
4.5% AC
3.5 4.0 4.5 5.0 5.5
PERCENT BITUMEN
PERCENT TOTAL VOIDS FILLED WITH BITUMEN
90
80
70
60
50
75% VOIDS
5% AC
3.5 4.0 4.5 5.0 5.5
PERCENT BITUMEN

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Figure 3-18. Asphalt mix curves, Marshall test properties

Graphical presentation of the Marshall test properties. The average Marshall test properties from the tabulation (Figure 3-16, page 3-45) for each bitumen content are shown graphically on DD Form 1219, Figure 3-18. The average values for each property are plotted on their respective graphs using the bitumen content as ordinates. A smooth curve is drawn through the plotted points in each instance.

Applying the Marshall Test

Table 3-7, page 3-50, lists the criteria for determining the optimum bitumen content along the Marshall specifications for a satisfactory mix. The optimum bitumen content is determined by averaging the bitumen content read from the curves in Figure 3-18 at the four points indicated for determination of the optimum bitumen content in Table 3-7. Once this average bitumen content is obtained, the Marshall properties at this average are read from curves representing the mix, as in Figure 3-18. These values are compared with the specification limits in Table 3-7 in order to evaluate the satisfactoriness of the mix. Attention is called to the exceptions allowed (notes at bottom of Table 3-7) when the values obtained fail to conform with the specification limits given in the table. Note that the mix in this example conforms with the requirements in Table 3-7 at an optimum content of 4.7 percent.

Modified Marshall Test for Cold-mix Pavement

This method is used as an aid in determining the asphalt content for cold-mix design of light duty pavement. It can be used where asphalt cutbacks will be the binder. The procedures follow 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. Prior to compaction, the mixture is cured for at least 12 hours in an oven set at 140° ± 5°F.

- 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° ± 2°F and tested in the Marshall machine. Heating will normally take about 2 hours.

- Selection of the design amount of asphalt. The asphalt contents at maximum density and maximum stability after averaging are used as the design amount.

Test Variations

These apply to aggregates with 10 percent or more larger than 1-inch maximum size. The procedures described above are applicable where the amount of aggregate larger than the 1-inch sieve is less than 10 percent of the total. When the larger than (plus) 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 plus 1-inch portion.

- Pass the mixed hot batch through a 1-inch sieve. Discard the plus 1-inch portion.

- Make compacted specimens from the portion which passes the 1-inch sieve and perform the Marshall test. Do not calculate the voids of the compacted specimens at this time.

Table 3-7. Marshall test specifications and determination of optimum asphalt content

(1) Property	(2) Course	(3) Criteria		(4) Determination of OAC	
		(75 Blows) ***High Press	(50 Blows) Low Press	High Press	Low Press
Aggregate blends showing water absorption up to 2½% (used with ASTM apparent specific gravity)					
Stability	Surface	1,800 or higher	500 or higher	Peak of curve	Peak of curve
Unit wt	Surface	—	—	Peak of curve	Peak of curve
Flow	Surface	16 or less	20 or less	Not used	Not used
% Voids total mix	Surface	3%-5%	3%-5%	4%-	4%
% Voids filled w/AC	Surface	70%-80%	75%-85%	75%	80%
Stability	Binder	1,800 or higher	500 or higher	Peak of curve*	Peak of curve*
Unit wt	Binder	—	—	Peak of curve*	Peak of curve*
Flow	Binder	16 or less	20 or less	Not used	Not used
% Voids total mix	Binder	5%-7%	4%-6%	6%	5%
% Voids filled w/AC	Binder	50%-70%	65%-75%	60%*	70%*
Stability	Sand asphalt	**	500 or higher	**	Peak of curve
Unit wt	Sand asphalt	—	—	**	Peak of curve
Flow	Sand asphalt	**	20 or less	Not used	Not used
% Voids total mix	Sand asphalt	**	5%-7%	**	6%
% Voids filled w/AC	Sand asphalt	**	65%-75%	**	70%
Aggregate blends showing water absorption greater than 2½% (used with bulk-impregnated specific gravity)					
Stability	Surface	1,800 or higher	500 or higher	Peak of curve	Peak of curve
Unit wt	Surface	—	—	Peak of curve	Peak of curve
Flow	Surface	16 or less	20 or less	Not used	Not used
% Voids total mix	Surface	2%-4%	2%-4%	3%	3%
% Voids filled w/AC	Surface	75%-85%	80%-90%	80%	85%
Stability	Binder	1,800 or higher	500 or higher	Peak of curve*	Peak of curve*
Unit wt	Binder	—	—	Peak of curve*	Peak of curve*
Flow	Binder	16 or less	20 or less	Not used	Not used
% Voids total mix	Binder	4%-6%	3%-5%	5%	4%
% Voids filled w/AC	Binder	55%-75%	70%-80%	65%	75%
Stability	Sand asphalt	**	500 or higher	**	Peak of curve
Unit wt	Sand asphalt	—	—	**	Peak of curve
Flow	Sand asphalt	**	20 or less	Not used	Not used
% Voids total mix	Sand asphalt	**	4%-6%	**	5%
% Voids filled w/AC	Sand asphalt	**	70%-80%	**	75%

*If the inclusion of bitumen contents at these points in the average causes the voids total mix to fall outside the limits, then the optimum bitumen should be adjusted so that the voids total mix are within the limits.

**Criteria for sand asphalt to be used in designing pavement for high pressure tires have not been established.

***High pressure tires are those above 100 psi. Low pressure tires are those with 100 psi or under.

- Determine the bulk specific gravity of the plus 1-inch aggregate, and with the specific gravity of the compacted specimens, compute the adjusted specific gravity (GA) as follows:

$$G_A = \frac{100}{\frac{A}{C} + \frac{B}{D}} \times f$$

where:

A = weight of dry, 1-inch material expressed as a percentage of the total batch weight (bitumen plus aggregate)

B = portion of the total batch remaining after the dry, plus 1-inch portion is removed (100 percent − A percent)

C = bulk specific gravity of the plus 1-inch aggregate

D = actual specific gravity of the compacted specimen

f = empirical factor = 0.995.

- Calculate the voids by using the adjusted specific gravity and apply the design criteria for this value.

- Use stability and flow values as measured on the compacted specimens.

SURFACE AREA METHOD

The following approximation formulas may be used for estimating the optimum bitumen content when the gradation of the aggregate blend is known. These estimates must be considered rough approximations since the optimum bitumen content is a function of the compaction effort as well as the gradation and surface area. The greater the anticipated pavement loading, the greater the compaction effort that must be used. The greater the compaction effort for a given aggregate, the lower the optimum bitumen content. The formulas based upon the surface area of the aggregate are as follows:

For Asphalt Cement

$$P = 0.02a + 0.07b + 0.15c + 0.20d$$

where:

P = percent (expressed as a whole number) of asphalt material by weight of dry aggregate

a = percent (expressed as a whole number) of mineral aggregate retained on No. 50 sieve

b = percent (expressed as a whole number) of mineral aggregate passing No. 50 and retained on No. 100 sieve

c = percent (expressed as a whole number) of mineral aggregate passing No. 100 and retained on No. 200 sieve

d = percent (expressed as a whole number) of mineral aggregate passing the No. 200 sieve.

For Asphalt Emulsion

$$P = 0.05A + 0.1B + 0.5C$$

where:

P = percent (expressed as a whole number) by weight of asphalt emulsion, based on weight of graded mineral aggregate

A = percent (expressed as a whole number) of mineral aggregate retained on No. 8 sieve

B = percent (expressed as a whole number) of mineral aggregate passing No. 8 sieve, retained on No. 200 sieve

C = percent (expressed as a whole number) of mineral aggregate passing No. 200 sieve

NOTE

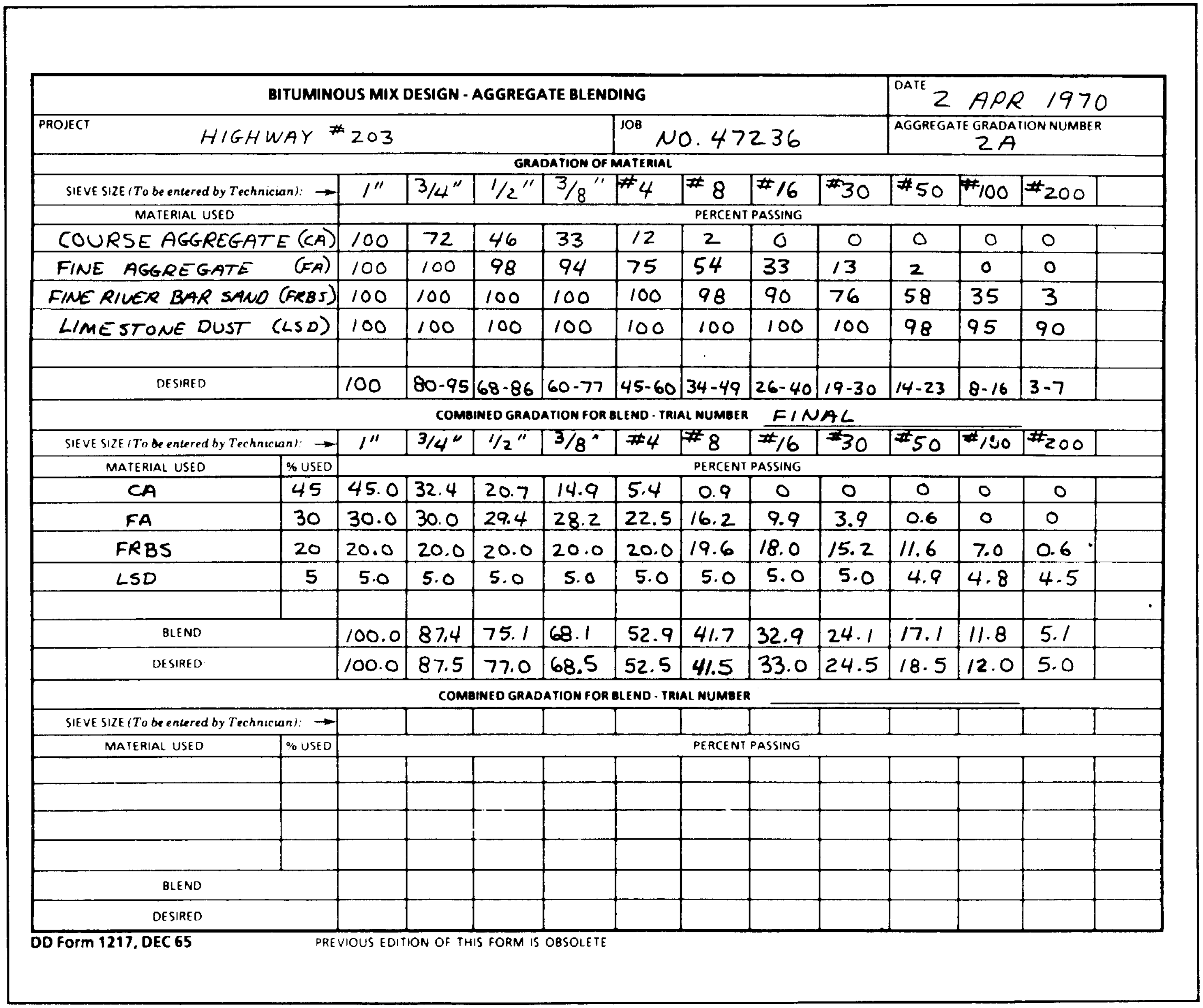
Absorptive aggregate such as slag, limerock, vesicular lava, and coral will require additional asphalt.

JOB-MIX FORMULA

When the necessary laboratory tests have been completed and the optimum bitumen content has been determined, the job-mix formula must be established for use by plant personnel producing the paving mix. Setting up the job-mix formula involves the relative percentages of the available aggregate and the bitumen. In the mix design test already illustrated, the optimum content was found to be 4.7 percent of the total mix. Accordingly, the aggregate portion of the mix will be 95.3 percent of the total mix. Referring to the aggregate blend calculations (Figures 3-19, DD Form 1217 (Bituminous Design - Aggregate Blending), 3-20, and 3-21, DD Form 1207 (Grain Size Distribution Graph - Aggregate Grading Chart) pages 3-53, 3-54 and 3-55) the portions were 45 percent coarse aggregate (CA), 30 percent fine aggregate (FA), 20 percent fine river bar sand (FRBX), and 5 percent limestone dust or mineral filler.

The job-mix formula is then computed as follows:

CA = 95.3 x 45 = 42.9%
FA = 95.3 x 30 = 28.6%
FRBS = 95.3 x 20 = 19.0%
Mineral Filler = 95.3 x 5 = 4.8%
95.3%
Bitumen = 4.7%
Total = 100.0%



BITUMINOUS MIX DESIGN - AGGREGATE BLENDING		DATE 2 APR 1970
PROJECT HIGHWAY #203	JOB NO. 47236	AGGREGATE GRADATION NUMBER 2A

GRADATION OF MATERIAL

SIEVE SIZE *(To be entered by Technician)*: →	1"	3/4"	1/2"	3/8"	#4	#8	#16	#30	#50	#100	#200	
MATERIAL USED	PERCENT PASSING											
COURSE AGGREGATE (CA)	100	72	46	33	12	2	0	0	0	0	0	
FINE AGGREGATE (FA)	100	100	98	94	75	54	33	13	2	0	0	
FINE RIVER BAR SAND (FRBS)	100	100	100	100	100	98	90	76	58	35	3	
LIMESTONE DUST (LSD)	100	100	100	100	100	100	100	100	98	95	90	
DESIRED	100	80-95	68-86	60-77	45-60	34-49	26-40	19-30	14-23	8-16	3-7	

COMBINED GRADATION FOR BLEND - TRIAL NUMBER FINAL

SIEVE SIZE *(To be entered by Technician)*: →		1"	3/4"	1/2"	3/8"	#4	#8	#16	#30	#50	#100	#200	
MATERIAL USED	% USED	PERCENT PASSING											
CA	45	45.0	32.4	20.7	14.9	5.4	0.9	0	0	0	0	0	
FA	30	30.0	30.0	29.4	28.2	22.5	16.2	9.9	3.9	0.6	0	0	
FRBS	20	20.0	20.0	20.0	20.0	20.0	19.6	18.0	15.2	11.6	7.0	0.6	
LSD	5	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	4.9	4.8	4.5	
BLEND		100.0	87.4	75.1	68.1	52.9	41.7	32.9	24.1	17.1	11.8	5.1	
DESIRED		100.0	87.5	77.0	68.5	52.5	41.5	33.0	24.5	18.5	12.0	5.0	

COMBINED GRADATION FOR BLEND - TRIAL NUMBER ____________

SIEVE SIZE *(To be entered by Technician)*: →													
MATERIAL USED	% USED	PERCENT PASSING											
BLEND													
DESIRED													

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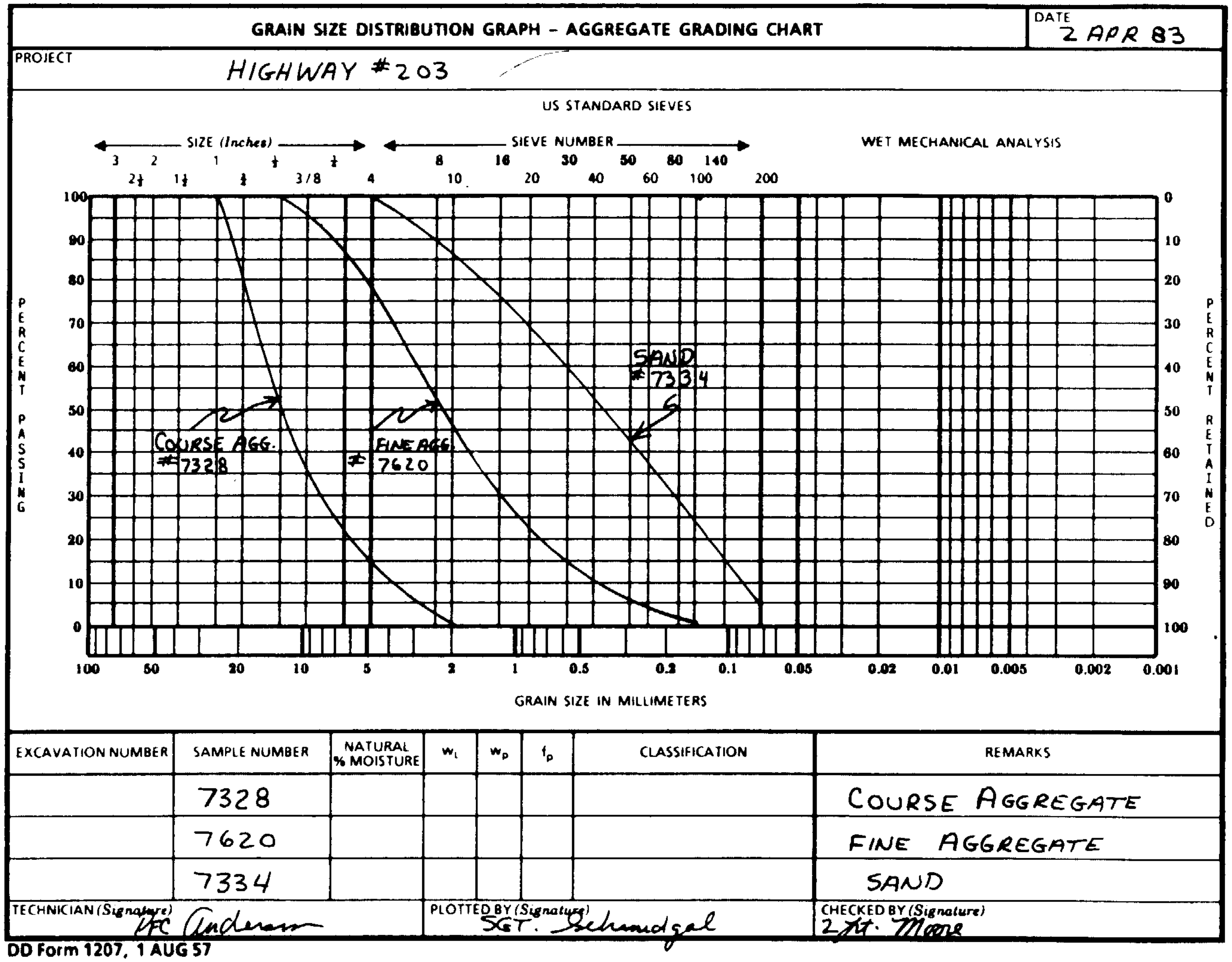
Figure 3-19. Bituminous mix design, aggregate blending, data sheet



GRAIN SIZE DISTRIBUTION GRAPH - AGGREGATE GRADING CHART

DATE 2 APR 83

PROJECT HIGHWAY #203

EXCAVATION NUMBER	SAMPLE NUMBER	NATURAL % MOISTURE	w_L	w_P	I_P	CLASSIFICATION	REMARKS
	7328						COURSE AGGREGATE
	7620						FINE AGGREGATE
	7334						SAND

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PLOTTED BY (Signature) SGT. Schmidgal

CHECKED BY (Signature) 2Lt. Moore

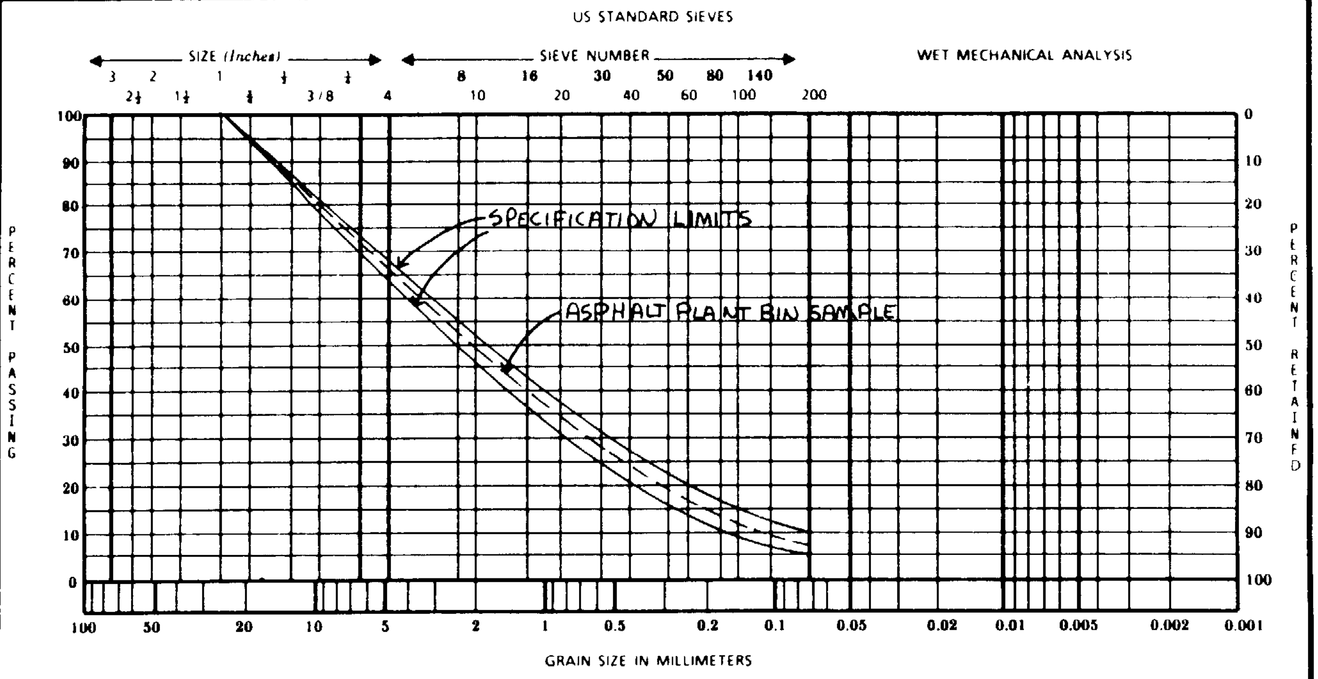
DD Form 1207, 1 AUG 57

Figure 3-20. Aggregate grading chart, stockpile materials

GRAIN SIZE DISTRIBUTION GRAPH - AGGREGATE GRADING CHART

DATE 2 APR 1970

PROJECT HIGHWAY #203



EXCAVATION NUMBER	SAMPLE NUMBER	NATURAL % MOISTURE	w_L	w_P	I_P	CLASSIFICATION	REMARKS

TECHNICIAN (Signature) SP4 Clerici

PLOTTED BY (Signature) SP4 Jones

CHECKED BY (Signature) SSG Haynes

DD Form 1207, 1 AUG 57

Figure 3-21. Aggregate grading chart, specification limits and gradation of blended aggregate

Section VII. PLANT CONTROL

DISCUSSION

Aggregate is mixed in correct proportions for specified construction in high-type bituminous paving plants, and in intermediate-type plants. The type of plant used will depend 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 production for each aggregate specified.

PLANT TYPES

Figures 3-22 and 3-23 are schematic drawings of a batch plant and a drum-mix plant.

High-type Bituminous Paving Plant

In the operation of a high-type bituminous paving plant, aggregates from two or more sources are fed into the aggregate dryer in the approximate proportions required to produce the desired gradation. This initial proportioning generally is accomplished by means of a hopper-type feeder, operating from one or more bins, which feeds the aggregates into a cold elevator which delivers them to the dryer. The mechanical feeder is loaded by a clamshell or other suitable means. The aggregates are heated to the desired temperatures. Upon leaving the dryer, the aggregates pass over vibrating screens where they are separated according to size. The usual screening equipment for a three-bin plant consists of a rejection (scalping) screen for eliminating oversized material and screens for dividing the CA into two separate sizes (bins). FA goes into the third bin. An additional screen is provided for further separation of the CA in a four-bin plant. Additional mineral filler, if required, usually is stored and weighed or proportioned into the mix separately. It may be obtained from the plant's dust collector or from an external source. Plant screens may be changed to provide a variation in size of openings. The sizes used depend largely upon the type of mixture being produced. In some cases it may be necessary to change the size of the screens to obtain a proper balance of aggregate sizes in each bin.

The aggregates must be fed through the plant uniformly, preferably by mechanical feeder, in order to obtain efficient plant operation and produce a desired mixture. It is usually necessary to make some slight adjustments in the plant-bin proportions, since a screen analysis of the hot storage bins will not entirely duplicate the screen analysis used in the laboratory design. This may result from:

Fines lost while passing through the dryer (unless the equipment includes an effective dust collector, and the fines are returned to the mix).

Aggregate degradation in the dryer.

Plant screens which are not completely efficient in the separation of the aggregate, with the result that some fines are carried over into the coarser bins.

Separation of material at hot bins into more or fewer fractions than represented at stockpiles.

Intermediate-type Plants

Intermediate-type plants will not have the refinements of the high-type plant described above. Such items as the gradation control unit, the dryer unit, and even the storage hoppers or bins may not be components of this type of plant. The aggregates from stockpiles or trucks are added directly (in correct proportion) into the elevator and then fed into the mixer. Corrections or changes in the aggregate proportions must be

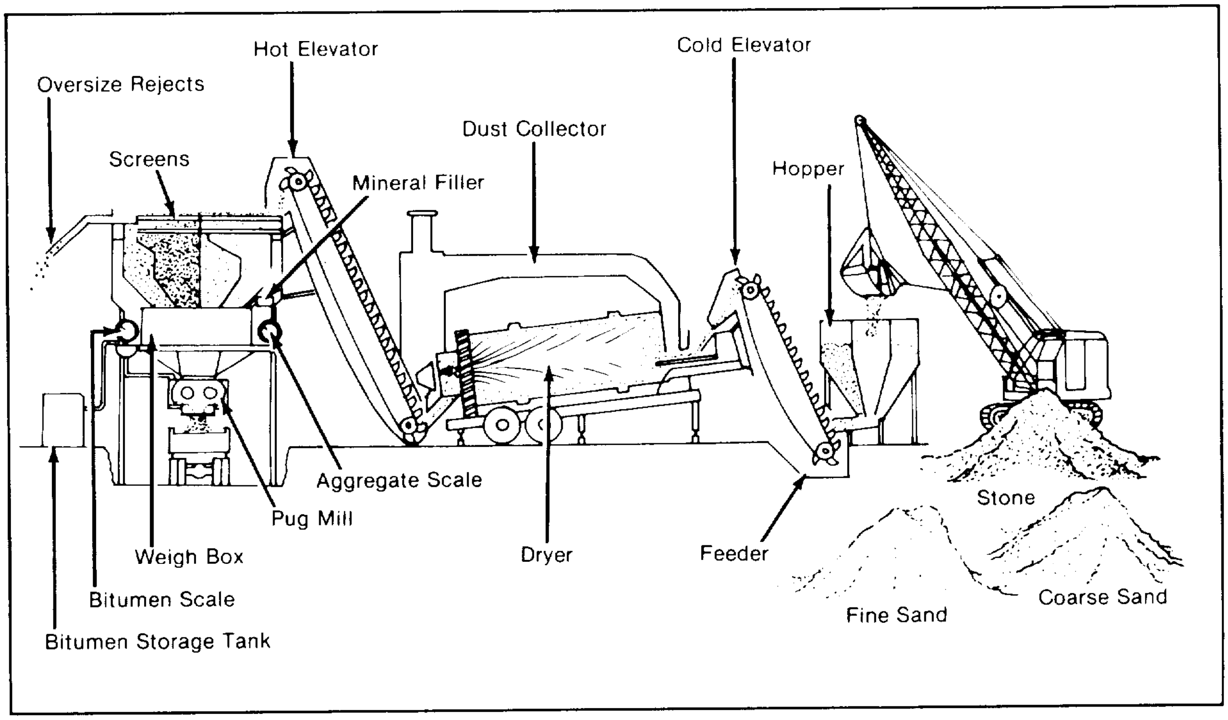

Hot Elevator
Cold Elevator
Oversize Rejects
Dust Collector
Screens
Mineral Filler
Hopper
Aggregate Scale
Stone
Pug Mill
Weigh Box
Dryer
Feeder
Bitumen Scale
Coarse Sand
Fine Sand
Bitumen Storage Tank

Figure 3-22. Bituminous hit-mix batch plant

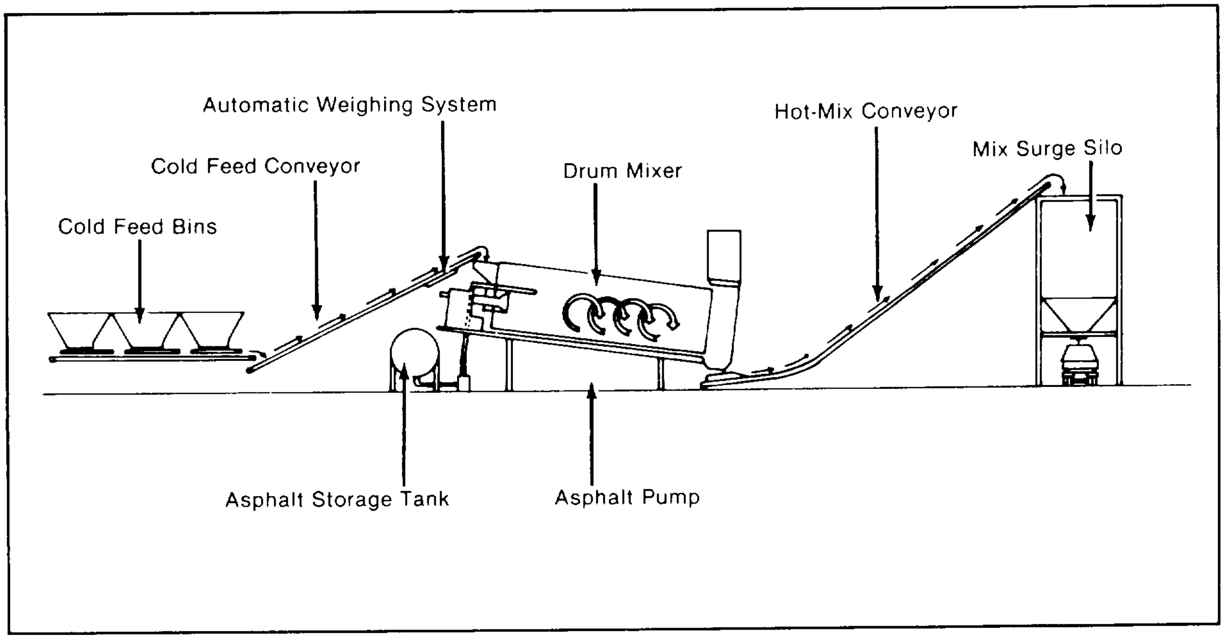

Automatic Weighing System
Hot-Mix Conveyor
Mix Surge Silo
Cold Feed Conveyor
Drum Mixer
Cold Feed Bins
Asphalt Storage Tank
Asphalt Pump

Figure 3-23. Bituminous hot-mix continuous-mix plant

made during the feed. If a dryer is not part of the plant, adjustment to compensate for moisture must be anticipated and made externally.

INITIATING PLANT PRODUCTION

The heaviest demands on laboratory facilities arise at the beginning of plant production. Preliminary computations, which provide the gradation for the mixture design, may be made to determine the weight of material from each bin. Note that the gradation of aggregate supplied by the plant according to computed bin weights may not reproduce precisely the desired gradation. The gradation of the plant-produced aggregates approximates the one used in design, within reasonable tolerances, if initial sampling has been done properly and if the plant is operated efficiently. Certain steps should be taken, however, to ensure that satisfactory mixtures are reproduced from the beginning and throughout the period of plant production. Procedures outlined in this section will ensure satisfactory paving mixtures.

Sieve Analysis

A sieve analysis is made on material from each plant bin. Samples for these sieve analyses are obtained after a few tons of aggregate have been processed through the dryer and screens so that the sample will be representative. Final bin proportions may be based on these sieve analyses.

Provision for Redesign of Mix

The aggregates from the bins sometimes cannot be proportioned to reproduce satisfactorily the gradation of the aggregate used in the laboratory design. It is then necessary to redesign the mix, using plant-produced aggregates. Specimens are prepared and tested for the new design in the same manner as for the original. This gives optimum asphalt content and a satisfactory mix produced by the plant. Occasions may arise in which the gradation of the plant-produced aggregate will differ from the laboratory design so that a part of the aggregates may be wasted. The mix should be redesigned to use all of the available aggregate. Sufficient additional tests should be performed to establish optimum asphalt requirements and ensure that the mix meets applicable criteria.

CONTROLLING PLANT PRODUCTION

Enough paving mix for the preparation of four test specimens is obtained from each of the first four truckloads. The four specimens prepared from each of these samples are compacted and tested in accordance with standard procedures described previously. The tests are conducted as rapidly as possible, and plant production is delayed until data from these tests is available. The data must conform to final design data at the same asphalt content, within reasonable tolerances, before plant production is resumed. If necessary, adjustments are made to secure a conforming mix. Such procedures will delay plant production generally less than 2 hours and assure production of satisfactory mixes. As soon as the data from the test of the plant-produced mix is obtained, it should be compared with corresponding design data for further adjustments of the mix, if necessary. Probable causes of paving-mixture deficiencies for both batch and continuous mixing plants are shown in Figure 3-24, page 3-60. These deficiencies are observed at the plant. Other imperfections and their causes that may be encountered in placing the mix in the pavement are given in Figure 3-25, page 3-61).

Gyratory Test Control

The GSI of the individual test specimens should in no case exceed 1.05. The GSF of the individual test specimens should in no case be less than 1.0. The average value of the other test properties for the four test specimens from any given truckload should not deviate from the final design values by more than the following amounts:

Unit weight total mix	± 1.5 pounds per cubic foot
Gyratory shear (S_G)	± 15 percent

Marshall Test Control

The flow value of the individual test specimens should in no case exceed the specification limits and should not vary more than two points between specimens. The average value of the other test properties for the four test specimens from any given truckload should be within the specification limits and not vary from the final design values by more than the following amounts.

Unit weight total mix	± 1.5 pounds per cubic foot
Marshall stability	± 5 percent
Voids total mix	± 0.5 percent

(No limit is placed on voids filled since control is ensured by the limitations on voids total mix.)

Executing Plant Controls

Plant production should be halted if the measured test properties fail to comply with the limitations cited above.

Production should proceed on a trial basis until the problem has been isolated and corrective measures have been taken. Small variations in aggregate quantities generally do not change test properties significantly, while small changes in bitumen content (a few tenths of a percent) can have a very significant effect on test properties. If trouble is indicated, all calculations should be checked as a first step. The total of the batch weights dumped into the truck should be checked against the total load on the truck. These total weights should not vary from each other by more than 2 percent. Improper weighing or faulty scales may be detected readily, and corrective measures should be taken by maintaining a close check on load weights. Plant scales and gate openings should be checked and recalibrated as necessary. Scales and/or

Probable Causes of Deficiencies in Hot Plant Mix Paving Mixtures

Item	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28
Types of Deficiencies That May Be Encountered in Producing Hot Plant Mix Paving Mixtures	Aggregate scales out of adjustment	Bridging of hot aggregate in bin	Bitumen scales out of adjustment	Lack of proper weighing by operator	Faulty pug mill dump gate	Bitumen temperature too high	Drum feed not uniform	Improper dryer operation	Too little bitumen	Too much bitumen	Sampling method not uniform	Excessive moisture content of aggregate	Bin overflow pipes not functioning	Leaky bins	Segregation of aggregates in bins	Mixing time not uniform	Mineral filler feed not uniform	Aggregate temperature too high	Temperature mechanism out of adjustment	Improper feed of aggregates to dryer	Overrated dryer capacity	Faulty screen operation	Overloaded pug mill	Improper set on pug mill	Bitumen and aggregate feed out of adjustment	Bitumen pump not functioning properly	Not sufficient hot aggregate in storage bins	Aggregate gates not properly set
Bitumen content fails to check job-mix formula	X		X	X	X				X	X	X		X	X		X	X			X				X	X	X	X	X
Gradation fails to check job-mix formula	X	X		X	X						X		X	X	X		X			X		X		X	X		X	X
Poorly mixed loads					X	X										X							X	X				
Fat, rich mixtures	X		X	X	X	X				X	X			X			X			X					X	X		X
Lean or burned mixtures	X		X	X	X	X		X	X		X			X				X	X	X					X	X		X
Mixture temperature fails to check job mix						X	X	X										X	X		X							
Smoking loads							X	X										X	X	X								
Steaming loads							X	X				X							X	X	X							
Overweight or underweight loads		X	X	X	X														X									
Lack of uniformity of mixtures in loads		X	X	X	X	X			X	X		X	X	X	X	X	X		X	X	X	X	X	X	X		X	X

Items 6 to 23 inclusive are applicable to all types plants. Items 1 to 5 inclusive and items 24 to 28 inclusive are applicable to batch plants and volumetric plants respectively.

Figure 3-24. Probable causes of paving mixture deficiencies detected at plant

proportioning devices found to be inaccurate must be adjusted. After the plant components are checked and necessary adjustments have been made, four additional truckloads of plant mix should be produced, sampled, and tested. The plant must not be placed in continuous operation until the test properties conform to the specifications and allowable tolerance. Once the plant has been placed in continuous operation, test specimens should be prepared from approximately each 400 tons of mix produced.

CENTRIFUGAL EXTRACTION METHOD

The procedure for control of plant production (based upon the test properties secured by the use of the Marshall apparatus or the gyratory testing machine) may be supplemented by the determination of asphalt content using a centrifugal extraction apparatus. The measured asphalt content is compared with the optimum asphalt content previously established. Discrepancies may be corrected by plant adjustments, if necessary. After the asphalt has been extracted, the remaining aggregates may be subjected to a sieve analysis for comparison with the previously selected aggregate gradation. Corrections again may be made in the operation of the plant, as necessary. The method is also used to test tar or tar-rubber mixes.

Apparatus

The following equipment is necessary for determining the asphalt content of a bituminous plant mixture by the use of the centrifugal extraction device:

PROBABLE CAUSES OF IMPERFECTIONS IN FINISHED PAVEMENTS																					
EXCESSIVE PRIME COAT	IMPROPER PROPORTIONING	UNSATISFACTORY BATCHES IN LOAD	POOR HANDWORK BEHIND SPREADER	EXCESSIVE SEGREGATION IN LAYING	INADEQUATE ROLLING	POOR SPREADER OPERATION	MIXTURE TOO HOT OR BURNED	MIXTURE TOO COLD	ROLLING MIXTURE WHEN TOO HOT	ROLLING MIXTURE WHEN TOO COLD	POORLY GRADED MIXTURE	UNSTABLE BASECOURSE	FAULTY ALLOWANCE FOR COMPACTION	ROLLER STANDING ON HOT PAVEMENT	MIXTURE TOO COARSE	LACK OF BITUMEN IN MIXTURE	EXCESS OF BITUMEN IN MIXTURE	INADEQUATE CROSS ROLLING	NOT CUT BACK TO UNIFORM THICKNESS	EXCESSIVE MOISTURE IN MIXTURE	TYPES OF PAVEMENT IMPERFECTIONS THAT MAY BE ENCOUNTERED IN LAYING HOT PLANT MIX PAVING MIXTURES
X		X															X				BLEEDING
							X									X				X	BROWN, DEAD APPEARANCE
	X	X		X							X				X						POOR SURFACE TEXTURE
	X	X	X	X	X	X		X	X	X		X		X				X			ROUGH UNEVEN SURFACE
			X					X					X						X		UNEVEN LATERAL JOINTS
			X			X		X		X			X						X		UNEVEN LONGITUDINAL JOINTS
					X					X								X			ROLLER MARKS
	X	X					X	X	X		X						X			X	PUSHING
			X			X		X	X					X							WAVES
	X	X		X				X		X	X				X	X					CRACKING
	X	X	X	X		X		X							X						HONEYCOMB
			X		X	X			X			X						X			DISTORTION
		X				X		X								X					TEARING OF SURFACE DURING LAYING
	X	X															X				RICH OR FAT SPOTS

Figure 3-25. Probable causes of imperfections encountered in laying pavement

Balance, 2,000-gram capacity
Beaker, 600-milliliter capacity
Varnish brush
Graduate, 1,000-milliliter glass
Evaporating dishes
Centrifugal extractor, hand driven with pad
Filter ring (paper-basket type)
Electric hot plate
Electric oven
Bake pan
Solvent (trichloroethane), 5-gallon can
Spatula
Cooking spoon
Crucible tongs

The Dulin-Rotarex centrifugal extraction device and related equipment are shown in Figure 3-26, page 3-62.

Procedure

This test may be performed upon a representative sample taken from an individual batch or upon a composite sample representing several batches.

- Upon initiation of the asphalt plant operation, four samples are taken, one from each of the first four trucks to leave the plant. Plant production is then held up until the completion of the test. After the plant is in normal operations,

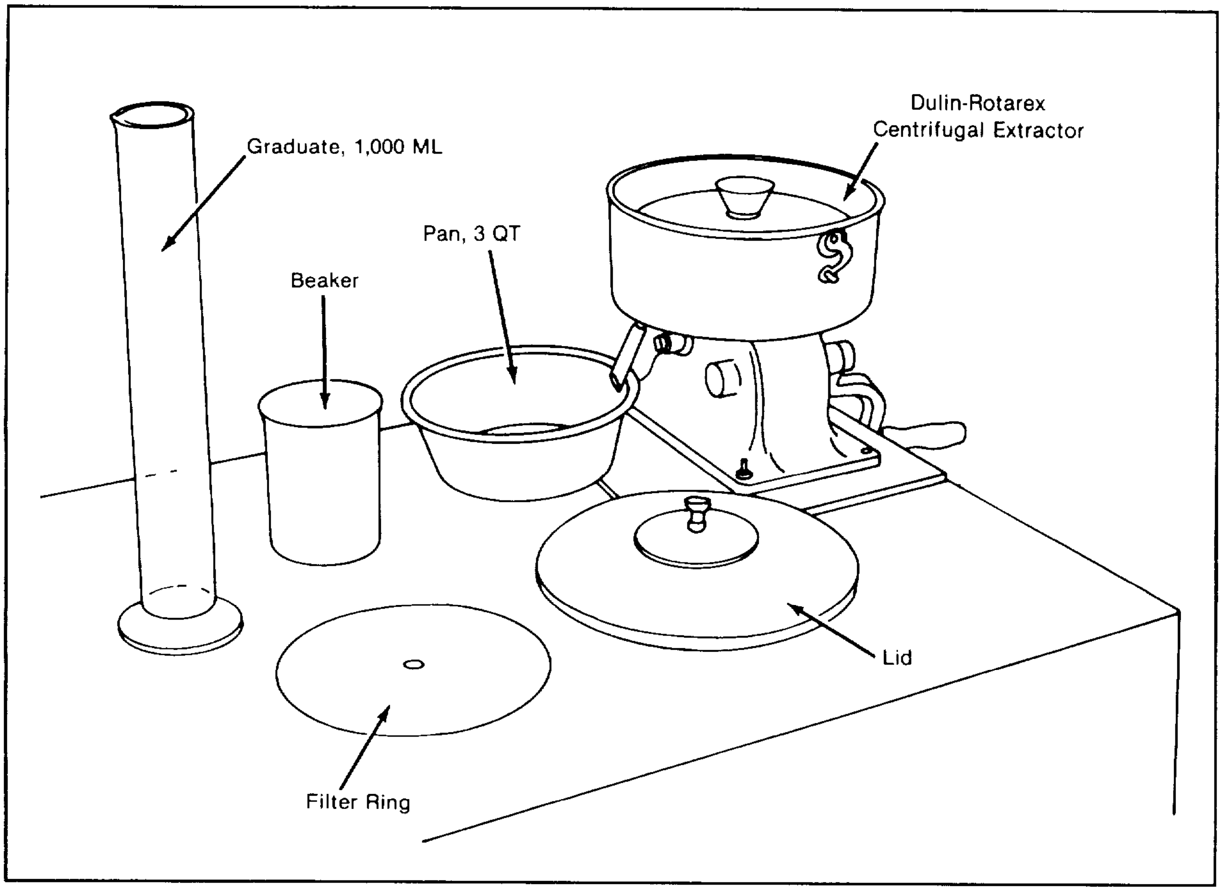


Figure 3-26. Dulin-Rotarex extraction equipment

a suitable composite sample is obtained by combining samples taken every 4 hours or every 400 tons of production, whichever comes first. This is done by using a shovel and cutting completely across the stream of hot mixture as it is discharged from the mixer. The hot samples should be placed in a tin pail or other container with a tight-fitting lid until the last sample for the day is taken.

- The sample is heated in a pan on the hot plate or in the oven until it is soft enough to be easily disintegrated with a large spoon. Care must be taken to see that the individual particles of aggregate are not crushed.

- A representative sample is prepared by the method of quartering and then allow to cool. Not less than 500 grams of a sample containing particles larger than $^{3}/_{4}$ inch or not less than 200 grams of a sample with all particles smaller than $^{3}/_{4}$ inch are accurately weighed and placed in the bowl of the extraction device. The initial weight of the sample is recorded.

- An oven-dried gasket-type filter paper ring is weighed, and the weight is recorded. The filter paper is then fitted on the rim of the bowl. The cover plate is positioned on the bowl and securely locked in place. The bowl is then placed on the shaft of the extractor and fastened securely.

- The empty 1,000-milliliter glass graduate is placed under the spout of the extraction apparatus. Approximately 150 milliliters of solvent is poured into the bowl through the solvent funnel. Flow of the solvent through the weep holes indicates a full bowl. The solvent is allowed to set in the bowl for 10 to 15 minutes before operating the equipment.

- The lid is put on and the bowl is rotated by turning the handcrank until the solvent discharges from the spout in a thin stream. After the first charge is drained out, the lid is removed and additional solvent is added. The lid is replaced and the bowl is again rotated. This operation is repeated several times until the discharged solvent is clean. With a little experience, the operator can soon judge exactly what treatment is necessary for any given material.

- When the last addition of solvent has drained off, the bowl is removed. The bowl is carefully disassembled, and the bowl, cover plate, and the filter paper ring are allowed to air dry. After air drying, the sample is carefully brushed out of the bowl and off the cover plate into the preweighed tare. The sample and the filter paper ring are then oven dried at 110° ± 5°C to a constant weight. The sample and paper filter ring are allowed to cool in a desiccator. The sample is weighed and the weights are recorded.

- The weight of the clean aggregate, plus the weight of any mineral filler retained in the filter paper, is subtracted from the weight of the original sample to determine the amount of asphalt extracted. This value for the amount of asphalt extracted is subject to correction, depending upon the amount of mineral filler contained in the solvent, as indicated below.

- The amount of mineral filler which passed through the filter paper is contained in the solvent extract and may be determined as follows. The total amount of solvent extract (volume in milliliters) is measured and recorded. The solvent is then agitated thoroughly, and 100 milliliters is measured into a preweighed evaporating dish. The evaporating dish is placed on a hot plate and evaporated over low heat until all that remains in the dish is a black residue. The residue is then heated to a dull red ash (500°C to 600°C). A laboratory furnace or blow torch may be used to heat the residue. The heating process will burn off any organic matter (asphalt binder) remaining in the residue. Once the residue has become a dull red glowing ash, it is allowed to cool to room temperature. A saturated solution of ammonium carbonate is prepared and added to the ash in the proportions of 5 milliliters of solution of 1 gram of ash remaining in the dish. (A saturated solution of ammonium carbonate is prepared by dissolving as much ammonium carbonate into water as the water can retain. At 60°F approximately 100 grams of ammonium carbonate will dissolve in 100 milliliters of water.)

NOTE

The solution is prepared at room temperature since ammonium carbonate decomposes at 135°F. The ammonium carbonate solution and ash mixture is allowed to stand for 1 hour. The mixture is then oven dried at 110°C to a constant weight. After oven drying, the mixture is allowed to cool in a desiccator. The mixture (in the evaporating dish) is then weighed, and the weight is recorded.

Calculations

The numerical example (Figure 3-27, page 3-65, illustrates the method of calculating the percentage of asphalt contained in the original sample. The supply of DD Form 1793 (Determination of Asphalt Content) will be requisitioned through normal publications supply channels.

Sieve Analysis of Aggregate

The clean aggregate may be subjected to sieve analysis. The amount of material passing the No. 200 sieve (mineral filler) in the sieve analysis must be increased by the weight of filler retained in the filter paper gasket and in the solvent as determined in calculations.

Testing Tar or Tar-rubber Mixes

For tar or tar-rubber mixes, the above procedure should be modified as follows.

- Soak the sample in crystal-free creosote over night.
- Transfer soaked sample and creosote to the centrifugal apparatus and centrifuge to remove the creosote.
- Wash the sample with benzene until the solvent is a light straw color.

DENSITY TESTS

Density test samples should be taken and tests performed as often as conditions require, but at least once for every 400 tons of mix placed. To obtain a satisfactory specimen, the samples should be taken early in the morning when the pavement is cool. Any additional rolling required as a result of the tests should be done during the heat of the day. The sample for testing may be taken from any portion of the bituminous pavement, provided the area is typical of placing and rolling conditions.

Apparatus

A coring machine or concrete saw may be used for cutting out the samples. Chopped or jackhammered samples should be avoided if possible as these are likely to develop cracks or other disturbances which would lead to erroneous results. Samples should be cut completely through the thickness of the pavement and removed 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.

Procedures

When density samples are taken from a surface course placed on a binder course, it is practically impossible to remove the specimen from the tacked binder course surface. To assist in removing samples of the surface course from the binder course, the spot for the test should be selected before laying the surface course. A piece of wrapping paper about 18 inches square should be placed on the spot selected. On the side of the project least subject to construction traffic, two nails should be driven into the base or pavement a convenient distance apart and equal distance from the center of the paper so that the center of the paper may be readily located after the surface course is laid. The roller should not treat the mix placed over the paper any differently than the remainder of the pavement. The area of pavement over the paper is small, and practice has demonstrated that specimen density secured by this method is about the same as the density in the surrounding pavement. Frames or separators around the proposed sample should not be used.

Calculations

The bulk specific gravity for each specimen is determined and the following formula is used to calculate the density of the specimen:

$$D = G_m \times 62.4$$

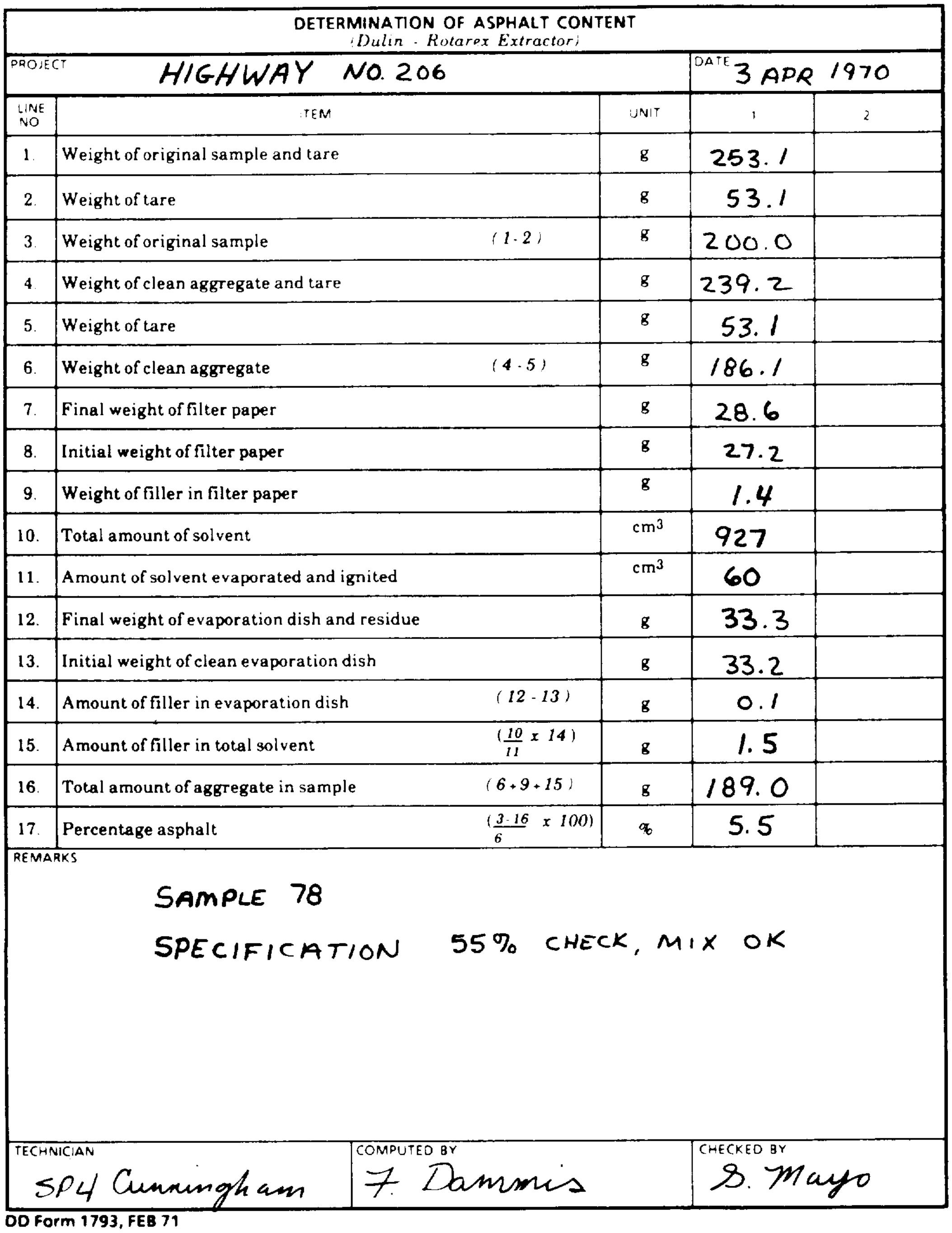
where:

D = density of specimen (pounds per cubic foot)

G_m = bulk specific gravity of specimen

62.4 = density of water (pounds per cubic foot).

The calculated density of the bituminous pavement is compared with the densities obtained during the bituminous mix design and Marshall stability tests. Normally the field density



DETERMINATION OF ASPHALT CONTENT
(Dulin - Rotarex Extractor)

PROJECT HIGHWAY NO. 206

DATE 3 APR 1970

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	($\frac{10}{11}$ x 14)	g	1.5	
16.	Total amount of aggregate in sample	(6+9+15)	g	189.0	
17.	Percentage asphalt	($\frac{3-16}{6}$ x 100)	%	5.5	

REMARKS

SAMPLE 78

SPECIFICATION 55% CHECK, MIX OK

TECHNICIAN	COMPUTED BY	CHECKED BY
SP4 Cunningham	F. Dammis	S. Mayo

DD Form 1793, FEB 71

Figure 3-27. Asphalt content by Dulfin-Rotarex extractor, sample data sheet (DD Form 1793)

must be at least 95 percent of the maximum density calculated in the bituminous mix design. However, individual project specifications do vary, and the acceptable density ranges must be verified for each project. When the density test on samples from the bituminous pavement show that the minimum specified field density has not been obtained, the deficiency should be corrected by additional rolling or the pavement should be removed and replaced. Where constant difficulty is experienced in meeting the specified density, a thorough check should be made of the job mix formula.

EXPEDITING THE DESIGN

When military expediency demands it, the preliminary laboratory mix designs are eliminated, and the mix is designed directly from plant-produced aggregates. In such cases, the engineering officer will, on the basis of sieve analysis or other information or judgment, select the most promising of the available aggregates and start the plant using this aggregate. As pointed out previously, in nearly all cases the aggregate will show some breakdown of the particles because of the heating and screening. Consequently the grading curve for the material which has passed through the plant will differ from the grading curve of the material prior to 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. Manufacture of the paving mix then can be initiated. If the available aggregate must be used, regardless of whether it produces a mix which meets all the design criteria at optimum asphalt content, the asphalt content should be selected to give a mix which meets the criteria for flow and percent of voids in the total mix. The criteria for stability and percent of voids filled with asphalt is given less consideration.

Chapter 4

Concrete

Section I. CHARACTERISTICS AND BEHAVIOR

DISCUSSION

Concrete is one of the most economical, versatile, and universally used construction materials. It is one of the few building materials that can be produced by the user directly on the job to meet the specific requirements. Concrete is an artificial stone which, when first mixed, forms a plastic or putty-like mixture. This mixture can then be placed into a form and allowed to harden or cure for a prescribed length of time. When cured, the finished concrete is a hard, stone-like material. It is used for pavements, foundations, dams and retaining walls, bridges, buildings of all types, and many military works.

Concrete can be described as a mixture of cement, fine and coarse aggregates, and water. During mixing, the cement and water form a fluid paste that contributes to thorough mixing and effective placement of the concrete. The cement and water, when mixed, combine chemically to form a variety of compounds that adhere to aggregate particles and fill the space between the aggregate particles. This combining process, called hydration, results in a rapid development of strength in the first few hours after mixing, followed by less rapid gains in strength during the following weeks.

There are certain curing conditions that are essential to the development of the maximum strength of the concrete. These usually involve some technique for keeping a high percentage of the mixing water from escaping from the concrete, permitting hydration of the portland cement. The strength and desired resistance to weathering deterioration of concrete depend upon an adequate amount of cement-water paste to thoroughly cover all aggregate surfaces and fill the voids between aggregate particles. The amount of water per unit quantity of cement in the paste (water-cement ratio) largely determines the strength of the hardened concrete. For design purposes, the strength at 28 days after mixing usually is used. Beyond this period, the relative increases in strength becomes smaller, and the strength curve flattens out. Some airfield designs use a 90-day strength which can be approximated by increasing the expected 28-day strength by 10 percent.

The total aggregate is a mixture of fine and coarse material, generally sand with either natural gravel or crushed stone. It serves as an inert filler to provide the bulk material required. Well-graded aggregates contain particles of all sizes, from the largest permitted by the dimension of the member to be formed, to sand fines.

The smaller particles fill the spaces between the larger, thus providing a dense material that requires a minimum of cement paste for binder. The aggregate materials must be clean, hard, and at least as strong as the weathering processes.

CEMENT

Cement is a substance which hardens with time and holds or entraps objects or particles in a definite relation to each other. For concrete, portland cement usually is used. Portland cement is manufactured by a standardized process consisting of grinding limestone and clay, mixing them in proportions, heating the mixture to a high temperature to form clinkers, and pulverizing the clinkers so that 95 percent of the material will pass through a No. 200 sieve. A small amount of gypsum is added to control the time of setting. References to cement in this manual mean portland cement. The ASTM specifies eight common types of portland cement (ASTM Designation C 150). These are adequate for most purposes. The various types of portland cement are known as hydraulic cements because they are capable of hardening and developing strength in the presence of water.

Type I

This type is for general construction use when the special properties for any other type are not required.

Type IA

This is air-entraining cement for the same uses as Type I, except that air entrainment is desired. Entrained air improves workability and provides resistance to frost action and freezing and thawing.

Type II

This cement is for general use, especially when moderate sulfate resistance or moderate heat of hydration is desired. It has a lower heat of hydration than the normal Type I, generates heat at a slower rate, and also has improved resistance to sulfate attack. Type II cement is used in locations where a high temperature rise in the concrete is objectionable, as in structures of considerable mass such as large piers, heavy abutments, and heavy retaining walls.

Type IIA

This is air-entraining cement for the same uses as Type II, except that air entrainment is desired.

Type III

This type of cement is used where a high strength is needed quickly. This may be due to a demand for early use, or in cold weather construction to reduce the period of protection against low or freezing temperatures.

Type IIIA

This is air-entrained cement for the same use as Type III, except that air entrainment is desired.

Type IV

This cement is used where a low heat of hydration is desired to keep the amount and rate of heat generated to a minimum. Type IV cement develops strength at a slower rate than Type I cement, but helps prevent the development of high temperatures in the structure with the attendant danger of thermal cracking upon later cooling.

Type V

This cement is for use when high sulfate resistance is desired. Sulfates react chemically with the cement compounds causing undesirable expansion of the mixture. The sulfates may be present in the water used to mix the concrete or may be created by sulfurous gases from nearby industrial areas. The principal source of sulfate attack, however, occurs on foundations and other concrete in contact with the earth in certain regions and is caused by a reaction between the groundwater, containing dissolved reactive minerals or acid, and the hardened cement. Type V cement is low in calcium aluminate and is highly resistant to sulfate attack.

BLENDED CEMENTS

These cements may be the result of additions of various materials to the eight types above or especially designed types. Their specifications are contained in ASTM C 595.

Type IS - Portland Blast-Furnace Slag Cement

This cement is for use in general concrete construction. It is also available in an air-entraining form Type IS-A. This cement uses molten blast-furnace slag, which is rapidly chilled or quenched from its molten state in water to produce a granulated material. The slag (from 25 to 70 percent of the total weight of cement) is interground with the cement clinker during production or is blended in a finely ground form with finished cement.

Type IP - Portland-Pozzolan Cement

This cement contains from 15 to 40 percent pozzolan. Pozzolan is a siliceous or siliceous and aluminous material such as fly ash, volcanic ash, diatomaceous earth, or calcined shale. The strength of concrete made with pozzolanic cements is not as great as concrete made with the same amount of portland cement, but its workability may be better for some uses. The portland-pozzolan cement consists of a blend of portland blast-furnace slag cement and fine pozzolan produced by either intergrinding portland-cement clinker and pozzolan, by blending portland cement or portland blast-furnace slag cement and finely divided pozzolan, or a combination of intergrinding and blending. It is also available in an air-entrained variety called Type IP-A.

Type S - Slag Cement

Slag cement consists of a blend of finely divided granulated blast-furnace slag and portland cement, or hydrated lime, or both, in which the slag content is a least 70 percent of the weight of the slag cement. It is used in combination with portland cement in making concrete and in combination with hydrated lime in making masonry mortar. It is also available in an air-entraining variety (Type SA).

Type I (PM) - Pozzolan-Modified Portland Cement

This cement is for use in general concrete construction. It can be interground or blended, but the pozzolan content is less than 15 percent of the weight of the pozzolan modified portland cement. It is also available in an air-entraining variety (Type I (PM)-A).

Type I (SM) - Slag Modified Portland Cement

This cement is for use in general concrete construction when the special properties specified for other types of cement are not required. It can be interground or blended, but the slag content is less than 25 percent of the weight of the slag-modified portland cement. It is also available in an air-entraining variety (Type I (SM)-A). For both Type I (SM) and Type I (SM)-A cements, moderate sulfate resistance or moderate heat of hydration, or both, may be specified by adding the suffixes (MS) or (MH), or both, to the selected type designation.

Air-Entrained Cement

Concrete made with air-entrained cement is resistant to severe frost action and to salts used for ice and snow removal. It is produced by adding air-releasing materials to the clinker as it is ground. In general, air-entrainment may be controlled to a much greater extent by the use of admixtures with normal cements during mixing. This combination results in a concrete with tiny, distributed, and separated air bubbles (up to millions per cubic foot). The entrained air bubbles improve the workability of fresh concrete. These bubbles reduce the capillary and water channel structure within water. This prevents the buildup of damaging water. Air-entrained concrete has greatly increased durability in outdoor locations exposed to freezing weather. Each of the first three Types (I, II, and III) and Types IS and IP are available as air-entrained. To signify this characteristic, a letter A is added after the type. Thus, air-entrained pozzolan cement is known as Type IP-A.

Other Types of Cement

In addition to the types described above, there is white cement, waterproofed cement, oil well cement, and shrinkage compensating cement (Type K). White cement is made from selected materials to prevent coloring, staining, or darkening of the finished concrete. Waterproofed cement has water repellent material added. The finished and set concrete has a water repellent action. Oil well cement is especially made to harden properly when used under high temperature in deep oil wells. Shrinkage compensating cement (Type K) is a portland cement which, upon mixing with water, forms a paste that during the early hardening period occurring after setting increases significantly in volume. This volume increase, when restrained by reinforcement, assists in reducing shrinkage cracking of concrete. It is specified in ASTM C845.

WATER

Water plays an important part in the concrete mix. Its principal uses are to make the mix workable and to start the chemical reaction. Any material in the water which retards or changes the reaction is detrimental. A good rule of thumb is "if it's good enough to drink, it may be used for concrete."

Ordinary Water

The materials found in some types of water include organic compounds, oil alkali, or acid. Each has its effect on the hydration process.

- Organic material and oil. These compounds tend to coat the aggregate and cement particles and prevent the full chemical action and adherence. The organic material may also react with the cement and create a weakened cementing action, thus contributing to deterioration and structural failure of the concrete.

- Alkalies, acids, and sulfates. Certain limiting amounts of these chemical impurities in the water tend to adversely react with the cement. The result is inadequate cementing and weakened concrete. Water must be substantially free of these chemicals for use in concrete mixing.

Sea Water

The salts in sea water are normally thought of as being corrosive. However, sea water is used sometimes for concrete mixing with satisfactory results. A loss of 10 to 20 percent in compressive strength can be expected when using the same amount of sea water as fresh water. This can be compensated for somewhat by reducing the water-cement ratio.

AGGREGATES

The aggregates commonly used for concrete are natural deposits of sand and gravel, where available, or crushed stone. Crushed aggregate may cost more to produce; however, this may be the only way to obtain substantial quantities of large sized stone. Artificial aggregates such as a blast furnace slag or especially burned shales and clays are used.

Aggregates are divided into two types: fine and coarse. When properly proportioned and mixed with cement, these two groups will yield an almost voidless stone which is strong and durable. Aggregate should be equal to or better in strength and durability than the hardened cement paste if it is to withstand the design loads and effects of severe weather.

Fine Aggregate

Fine aggregate (FA) is defined as the material which will pass a No. 4 sieve and will be predominantly retained on a No. 200 sieve. To increase workability and for economy as reflected by use of less cement, the FA should have a rounded shape. The purpose of the FA is to fill the voids between coarse aggregate (CA) particles and to modify the workability of the concrete. This characteristic of workability is discussed more in the description of finished concrete.

Coarse Aggregate

Coarse aggregate (CA) is a material which will be retained on the No. 4 sieve. In determining the maximum size of CA, other factors must also be considered. The coarser the aggregate use, the more economical the mix, as the aggregate costs less than cement. Larger pieces offer less surface area of the particles than an equivalent volume of small pieces. Use of the largest permissible maximum size of CA permits a reduction in cement and water requirements. One restriction usually assigned to CA is its maximum size. Large pieces can interlock and form arches or obstructions within a concrete form. This restricts the area below to a void, or at best to become filled with the finer particles of sand and cement. This is either a weakened area or a cement-sand concentration which does not leave enough mortar to coat the rest of the aggregate. The maximum size of CA must be no larger than the sizes given in Table 4-1. Capacity of mixing equipment, spacing of reinforcement, or minimum width of formwork may also limit maximum aggregate size.

Gradation

Gradation of aggregate refers to the distribution of particles of aggregate among various sizes. Aggregates having a smooth grading curve and neither a deficiency nor an excess of any one particle size will generally produce mixtures with fewer voids between particles. A too large proportion of CA leaves voids which require more cement paste to fill. This affects the economy of the mix. Too much FA increases the amount of surface area which must be coated with cement paste. This may weaken the concrete and is uneconomical. Good gradation

Table 4-1. Maximum recommended size of coarse aggregate

Structure	Minimum dimension — inches			
	2½-5	6-11	12-29	30 or more
Reinforced walls, beams and columns	½-¾	¾-1½	1½-3	1½-3
Unreinforced walls	¾	1½	3	6
Slabs, heavily reinforced	¾-1	1½	1½-3	1½-3
Slabs, lightly reinforced	¾-1½	1½-3	3	3-6

Note. Maximum size not to exceed one-fifth of minimum dimension of a wall or similar structure, one-third of slab thickness for horizontal slab, or three-fourths of minimum clear spacing between reinforcing bars.

results in a dense mass of concrete with a minimum volume of voids, an economical mix, and a strong structure. Optimum strength, water tightness, and durability in the hardened concrete require careful control of aggregate gradation.

Durability

Durability is the ability of the concrete to resist the elements of weathering and loading. Weak or easily crushed rock or other mineral particles that break down under applied loads will introduce internal stresses causing a breakdown of the concrete. Rocks or mineral particles that are absorptive or susceptible to swelling when saturated will deteriorate when subjected to severe weather conditions. Freezing moisture causes expansion stresses which can easily rupture absorptive rocks. Rocks swollen from the sun's radiant heat and then subjected to shrinkage from sudden cooling by rain or temperature drop may break down from the severe weathering. The concrete aggregate must withstand all these forces of nature.

Deterioration

Deterioration of concrete in many cases can be traced to the aggregate. An excessive amount of organic material, clay, or fine silt in or on the aggregate prevents the cement paste from adequately bonding to the aggregate particles. This results in a structurally weak concrete which is also susceptible to breakdown from weathering. Washing the aggregate to remove the silts, clays, and organic material should prevent this problem.

Chemical Composition

Chemical composition of the aggregate is important. Any chemical reaction between aggregate and cement in the presence of water reduces the hardening and cementing processes occurring in the concrete. Any reduction in the amount of paste, due to the chemical reaction, reduces the amount of paste available for binding the aggregate together. This result is similar to insufficient amount of cement in the mixture.

PROPERTIES OF CONCRETE

To combine the ingredients correctly and to form the required concrete, it is essential to know the required physical properties of both the plastic and the hardened concrete. The hardened concrete must have adequate strength (as required by the design), be watertight, and be resistant to the deteriorating effects of weather or wear. The quality and character of the hardened concrete is greatly influenced by the properties of the mix when plastic. To attain optimum quality, the plastic mix must be uniform, homogeneous, and workable. This permits placing the concrete without developing segregation, honeycombing, or other defects in filling the forms or in producing the desired smooth, hard and resilient surface.

Strength

This is the ability of the concrete to resist a load in compression, bending, or shear (Sections V and VI). The desired design strength is obtained by proportioning the mixture with correctly graded aggregates, an adequate amount of cement to coat the surface area of the particles, and the proper amount of mixing water. The most important influencing factor on strength is the ratio of water to cement (W/C ratio). For plastic and workable mixes lower values of the W/C ratio give higher strengths. Two and one half gallons of water is the minimum amount necessary to adequately hydrate a sack of cement. This minimal amount of water is not sufficient to provide, economically, the needed plasticity and workability for freshly mixed concrete. Additional water must be added to the mixture to improve workability but minimized to obtain the desired strength with an economical cement content. Additional water will thin the paste content and therefore coat more particles. This increases the yield from each sack of cement and produces a more economical mix. Excessive amounts of water (too high a W/C ratio) weakens the paste by allowing the cement particles to hydrate while suspended in water without being in contact with the aggregate or other cement particles. This water eventually evaporates, leaving holes or voids in the hardened concrete which cause additional losses in strength. Minimum and maximum amounts of water are specified to assure an economical mix with no loss in strength. This ranges from 4 gallons to 8 gallons per sack of cement (94 pounds).

Durability

The ability of concrete to resist the elements is called durability. The primary elements affecting concrete are wind, abrasion, freezing and thawing, wetting and drying, and the chemical action of salts. As the W/C ratio is increased (4 gallons per sack), more voids develop in the hardened concrete. Therefore, more surface area is available for the detrimental elements to attack, resulting in a less durable structure.

Watertightness

A well mixed, well proportioned concrete presents a solid surface to water penetration. Superficial voids permit some water to enter below the surface of the concrete but the water soon meets a dense, solid mass which prevents further penetration. As the W/C ratio is increased, the excess water forms more holes or voids which eventually interconnect to form channels into, and throughout, the concrete. The end result is a more porous concrete which permits water to pass. For watertightness, 6 gallons of water or less per sack of cement will meet the requirement.

Workability

Workability is the relative ease or difficulty of placing and consolidating concrete. It is controlled primarily by the amount of each aggregate in proportion to a given quantity of cement paste. As more aggregate is added to a given amount of paste the mixture becomes harsh and stiff. The increased stiffness makes it more difficult to work the concrete into the forms and around the reinforcing bars. The consistency needed depends upon the conditions under which the concrete must be placed and finished. Very dry and stiff mixtures may be placed in most situations where high frequency vibrations are used to assist in consolidating and compacting and fresh concrete. In other situations, difficult placing conditions may require a more fluid concrete mixture to fill narrow forms and to flow around reinforcement.

Homogeneity

Concrete is a fluid mixture containing particles of different size, shape, and mass. Heavier particles have a tendency to settle out through the mixture faster than lighter particles. Often the result is a segregated mixture of a very poor quality. When concrete is properly proportioned, mixed and carefully handled, segregation can be held to a minimum. The mixture must have the proper proportion of cement/sand mortar to prevent the larger coarse-aggregate particles from separating from the batch during mixing, transporting, and placing. When cement is allowed to drop (free fall) over a considerable distance, it can cause segregation of the mixture. To minimize segregation for drops in excess of 3 to 5 feet, bottom dump buckets should be used to place concrete as close to the final location as possible. See TM 5-742 for construction procedures.

Uniformity

Uniformity refers to a single batch of concrete and to all batches for an entire project. The same amount of each ingredient should be mixed into each batch or a nonuniform structure will result. Design would not be met in all sections of the structure and possible failure of these sections could result. Proper supervision in mixing and handling of the concrete will ensure uniformity.

CURING

Concrete does not develop its full strength until the chemical process of curing, called hydration, is complete. Cement must have sufficient water to continue its hydration. Curing is the means of keeping the water available so the hydration can continue. The curing process takes place over an extended period. The most critical portion is the first 7 days. The extent and rate of curing depends upon the presence of moisture and the temperature within the concrete.

Temperature

The ideal temperatures for concrete work are between 55° and 70°F. Above this, rapid evaporation of moisture creates serious problems such as increased water demand, slump loss, decreased setting time, and increased tendency for plastic shrinkage cracking. At lower temperatures the hydration process is delayed. Temperatures below 32°F completely stop the hydration process. Since the chemical reaction gives off some heat, proper methods must be used to keep the heat within the structure during times of low temperatures. Cold weather construction may require heating the individual ingredients or the concrete and covering the emplaced concrete or providing a heated enclosure. In hot weather, extra care is required to prevent a high temperature rise and rapid drying of the fresh concrete. Spraying the aggregate stockpiles with cool water will help lower the concrete temperature. To keep the water as cool as possible, reflective white or aluminum paint is applied to the water supply lines and storage tanks.

On massive construction projects, such as dams and heavy retaining walls, the mixing water is often kept cooled by substituting ice for part of the mixing water. The ice must be melted by the time the concrete is fully mixed and is ready to leave the mixer. Large voids result from unmelted ice in the concrete. Cement replacement materials (such as pozzolans and diatomaceous earth, pumicites, or fly ash) may be used to depress concrete temperature by reducing the heat of hydration in a structure. However, pozzolans vary widely and may have adverse effects on strength, air content, and durability if used in excessive amounts.

Moisture

Concrete curing depends upon a chemical reaction in the presence of water. Moisture lost during the curing process by seepage or evaporation will delay or prevent a complete hydration of the cement and ultimately prevent the development of optimum strength and watertightness. Saturating the subgrade on which the concrete will be placed will delay, if not prevent, seepage from occurring. Impervious membranes (plastic or polyethylene sheets) may also be used to prevent seepage through the subgrade. Wood forms should be thoroughly wetted if they have not been otherwise treated with a moisture sealer. Covering the concrete with a material such as straw, burlap, plastic, or a sprayed-on chemical curing compound as soon after finishing as possible is one method of reducing evaporation. The preferred method of curing is by use of continuous sprays and flowing or ponded water after the concrete has set initially, so it does not damage the finish. This water application can also be part of the temperature control during cold weather and hot weather concreting. The increase of concrete compressive strength with age is shown by the curves in Figure 4-1. Note the long-time gain in strength that occurs when proper temperature and moisture conditions are maintained.

ADMIXTURES

Chemical agents or admixtures are available for almost any purpose, such as increasing

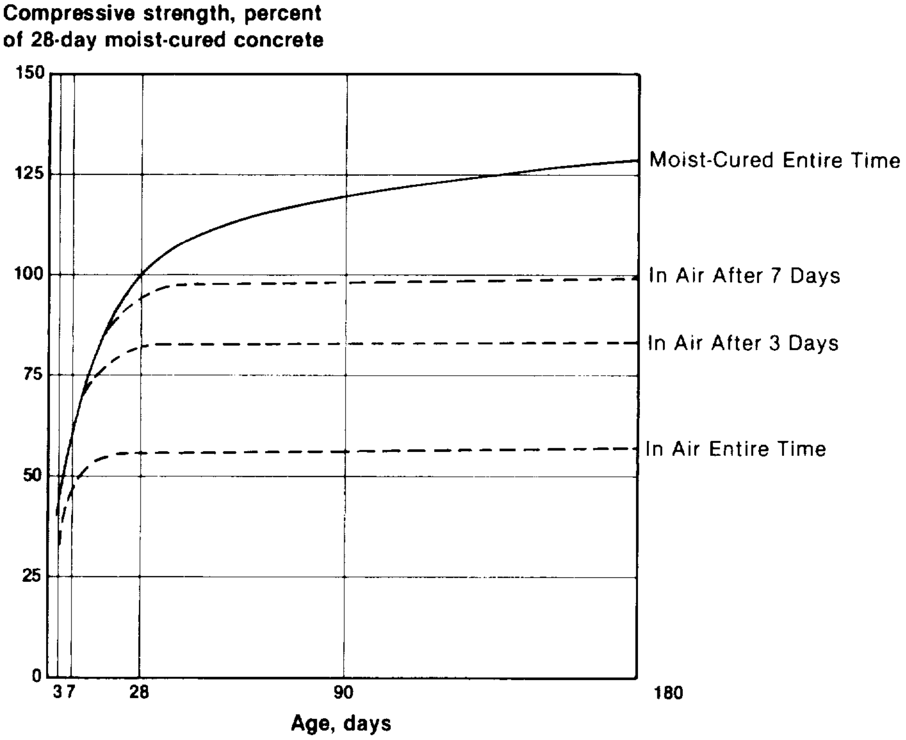


Figure 4-1. Increase of concrete compressive strength with curing age

workability, durability, and strength or compensating for inadequate curing conditions.

Accelerators

Sometimes it is desirable to accelerate the hydration process to obtain high early strength and a high rate of heat production. This combination is useful for cold weather concreting operations. The addition of a chemical accelerating admixture (generally calcium chloride) to the concrete mixture will produce the desired reactions. The recommended maximum dosage for calcium chloride is 2 percent by weight of cement. The ultimate strength of concrete will be slightly lower with the use of an accelerator.

Retarders

Retarders are used when excessively high heat or too rapid setting of concrete will prevent full hydration of the cement. Many materials will retard the setting of concrete. The most common is hydroxylated carboxylic acid salts. Sugar has been used quite successfully.

Air-entraining Agents

The greatest improvement in watertightness and resistance to the disruptive action of freezing and thawing is obtained by incorporating 4 to 6 percent by volume of entrained air into the concrete. Workability of fresh concrete is also enhanced by entraining air. Soaps, oils, acids, wood resins, alkali salts, fine pozzolans, and several proprietary compounds are available for use as air-entraining admixtures with hydraulic cement. These agents form very small, uniformly spaced, discrete air voids that relieve the buildup of damaging pressures from the expansion of freezing water into ice.

Water Reducers (Plasticizers)

The workability of concrete is governed by the proportions of cements, water, and aggregate in a concrete mixture. When a reduction of aggregate or an increase in cement is impractical, concrete workability can be increased by adding a water reducing admixture or plasticizer. Another primary characteristic is the strength gained from a decreased water demand. Less water is required for the same workability which leads to a lower W/C ratio, therefore higher strength. Water requirements may be reduced as much as 10 percent for most water reducing admixtures. Air-entraining agents are also considered as plasticizers because the void system reacts as a lubricant in concrete.

Section II. AGGREGATE TESTING

DISCUSSION

An aggregate must provide maximum strength and durability in a concrete mixture. Fineness, coarseness and gradation of aggregate are factors considered when deriving the correct concrete mix for a specific construction purpose. Specific gravity, absorption, and moisture also affect the ability of aggregate to bind well with cement and water in a concrete mix. The components of the final mix, cement, water and aggregate, must bond adequately for structural strength, and resist weather and loads. Correct aggregate selection also reduces the cost of a project. Engineering analysis determines the aggregate best suited for a particular purpose. Tests will allow the best selection.

For the aggregate tests to be worthwhile, the samples for testing must be representative of the aggregates to be used. Aggregate samples should be taken as close as possible to the finished product to give the best representative sample of the aggregate. Sufficient size and number of samples should be taken from the processing plant discharge point to represent the material in the stockpile. The sample should consist of at least four times as much material as is needed for the tests and should be reduced to the size desired for testing by splitting or quartering. See ASTM D 75 for the standard method of sampling aggregate.

Samples should be taken from near the top, middle, and bottom of the stockpile, and recombined to represent that particular stockpile. A board pushed into the stockpile just above the point of sampling will prevent the material above the sampling point from falling into the sample and causing size contamination.

Pit samples are sources of sand and gravel and should be sampled by channeling exposed faces or channeling in pits if exposures are not available. Care should be taken to ensure that the samples include only materials which are below the overburden or strip zone.

GRADATION

A gradation or sieve analysis indicates whether the particle size distribution of an aggregate meets the requirements for the project. Dense aggregates can result in a concrete that will be denser, more economical, stronger, more watertight, and more resistant. See ASTM C 136

Table 4-2. Desirable gradation for aggregates in concrete

Coarse aggregate

Sieve size, inches	Percent passing indicated sieve										
	4	3½	3	2½	2	1½	1	¾	½	⅜	No. 4
3½ to 1½	100	90-100	—	25-60	—	0-15	—	0-5	—	—	—
2½ to 1½	—	—	100	90-100	35-70	0-15	—	0-5	—	—	—
2 to 1	—	—	—	100	90-100	35-70	0-15	—	0-5	—	—
2 inch to No. 4	—	—	—	100	95-100	—	35-70	—	10-30	—	0-5
1½ inch to ¾	—	—	—	—	100	90-100	20-55	0-15	—	0-5	—
1½ to No. 4	—	—	—	—	100	95-100	—	35-70	—	10-30	0-5
1 inch to No. 4	—	—	—	—	—	100	95-100	—	25-60	—	0-10
¾ inch to No. 4	—	—	—	—	—	—	100	90-100	—	20-55	0-10
½ inch to No. 4	—	—	—	—	—	—	—	100	90-100	40-70	0-15
⅜ inch to No. 4	—	—	—	—	—	—	—	—	100	85-100	10-30

Fine aggregate

Sieve size U.S. standard	Percentage by weight passing
4	95-100
8*	80-100
10	75-95
16*	50-85
20	40-75
30*	25-60
40	20-50
50*	10-30
60	10-25
100	2-10

for analysis methods, and Table 4-2 for acceptable gradation limits.

Apparatus, Test Procedure, and Calculations

The apparatus, test procedure, and calculations required to determine the gradation of aggregate for portland cement concrete are the same as explained for sieve analysis, except that the No. 4 sieve is taken as the dividing line between FA and CA. The minimum sample size required in the sieve analysis of fine aggregate is 500 grams. The result of this test is a gradation curve for the aggregate concerned.

Material Finer Than 75 Milliliters (No. 200 Sieve)

The extremely fine mineral material (clay, silt, dust, or loam) occurring in most aggregates requires relatively large increases in the amounts of mixing water. Fines tend to work to the surface of concrete and cause checking due to shrinkage upon drying. If the fines adhere to the larger aggregate particles they also tend to interfere with the bond between the aggregate particles and cement-water paste. Specifications limit the amount of such material to a small percentage. ASTM C 117 gives the standard test method for fine materials. The apparatus, test procedure, and calculations to determine this percentage are described in tests for impurities.

Fine material, not to exceed 3 to 5 percent of the total aggregate weight, is generally not harmful to concrete. For some purposes a small amount of such fines may improve the workability.

Fineness Modulus

Fineness modulus is an empirical factor that gives a relative measure of the proportional particle size distribution of fine and coarse particles in an aggregate. The fineness modulus does not represent any gradation of the material although the process is similar. A 500 gram sample of sand is sieved through a series of sieves (Numbers 4, 8, 16, 30, 50, and 100). The weight retained on each sieve is converted into a cumulative weight and a cumulative percentage retained, starting with the No. 4 sieve. The sum of the 6 percentages divided by 100 is the fineness modulus. Another procedure for determining the fineness modulus is calculated using the cumulative percentage passing, the usual means of expressing aggregate gradation. The total number of sieves involved times 100 minus the sum of the cumulative percentage passing and divided by 100 gives the fineness modulus. The fineness modulus values range from 2.20 for FA to 7.50 for CA. Typical values are 2.70 FA, 7.40 CA, and 5.80 (35-65 fine-coarse combination).

SPECIFIC GRAVITY, ABSORPTION, AND SURFACE MOISTURE

These tests must be performed on the aggregates before the necessary calculations can be made to design the concrete mixture. For aggregates used in portland cement concrete measurements are made to determine the bulk specific gravity of the aggregates in a saturated, surface-dry condition. Specific gravity is thus based upon 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. See ASTM C 128 for the standard test method.

Apparatus

The following equipment is required to perform tests for bulk specific gravity, percent absorption, and surface moisture:

Balance

Pycnometer, 500-milliliters

Cone, water absorption

Tamper, metal flat head

Basket, wire sample

Pycnometer, 2 to 3 cubic feet

Heat source-oven, hot plate

Container, metal sample

Spatula, metal.

Procedure, Coarse Aggregate

The bulk specific gravity of coarse aggregate in a saturated, surface-dry condition is determined as follows:

- Secure a representative sample weighing approximately 5,000 grams.

- Dry the sample to a constant weight at 110°C.

- Cool the sample, immerse it in water and allow it to soak for 24 ± 4 hours.

- Remove the sample from the water and roll it in a large, absorbent cloth until visible films of water are removed. The surfaces of the particles will still appear to be slightly damp. The larger fragments may be wiped individually. The aggregate sample is now in a saturated, surface-dry condition (SSD). Weigh the sample in air and record the weight to the nearest 0.5 gram.

- Weigh the SSD sample and immediately place into the sample container and determine its weight in water at 23°C. Shake the basket or container while it is immersed to remove any entrapped air. This weight is the immersed weight (or weight in water).

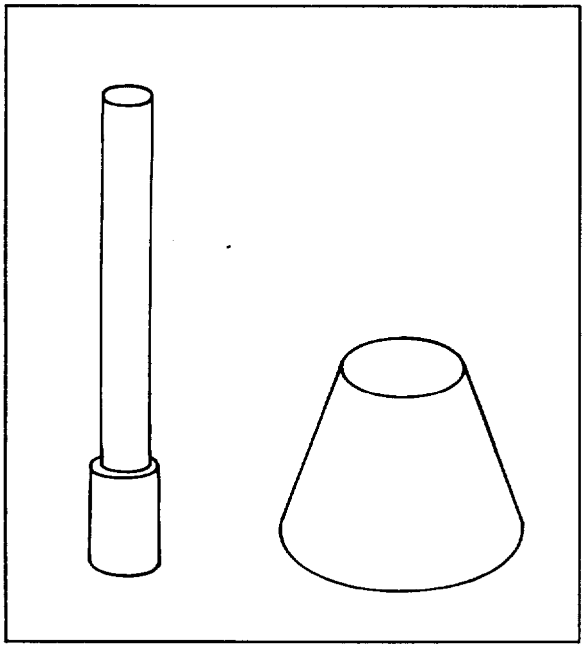


Figure 4-2. Water-absorption cone and tamping rod

- Calculate the SSD bulk specific gravity as follows:

Bulk specific gravity (saturated, surface-dry conditions) $= \frac{B}{B-C}$

where:

B = weight in grams of SSD sample in air

C = weight in grams of SSD sample in water

Procedure, Fine Aggregate

The test procedure for determining the bulk specific gravity of fine aggregate in a saturated, surface-dry condition is as follows.

- Secure a representative sample weighing approximately 1,000 grams.

- Dry the sample to a constant weight at 110°C.

- Cool the sample, immerse it in water and allow it to soak for 24 ± 4 hours.

- Spread the sample, after soaking, on a flat nonabsorbent surface and stir it to obtain uniform drying. Continue drying the sample until it approaches a surface-dry condition.

- Place the water-absorption cone (Figure 4-2) large end down on a smooth surface and fill it loosely with the aggregate. Then tamp the surface of the aggregate lightly 25 times with the metal tamper.

- Lift the cone vertically from the sand. If free moisture is present the cone of fine aggregate will retain its shape.

- Continue the drying, accompanied by constant stirring. Repeat the cone tests at frequent intervals until the cone of fine aggregate slumps slightly upon removal of the cone. This will indicate the fine aggregate has reached a surface-dry condition.

- Weigh exactly 500 grams of the saturated, surface-dry sample and introduce it into a partially water-filled 500-milliliter pycnometer. Agitate the sample to remove all entrapped air bubbles. Then adjust the water temperature to 23°C and fill the pycnometer to its callibrated capacity. Weigh the filled pycnometer and record the weight to the nearest 0.1 gram.

- Calculate the SSD specific gravity as follows:

Bulk specific gravity (saturated, surface-dry)

$$= \frac{500}{B + 500\text{-}C}$$

where:

B = weight in grams of pycnometer filled with water to calibrated capacity

C = weight in grams of pycnometer filled with the sample and water to calibrated capacity.

Procedure, Coarse and Fine Absorption

The test procedure for determining the percent absorption of coarse and fine aggregates follows.

Procedures are continuations of the specific gravity determinations.

- Weigh the coarse aggregate in water and the fine aggregate in the pycnometer.
- Remove the aggregates and dry to a constant weight at a temperature of 100° to 110°C.
- Weigh and record the oven-dry samples.
- Calculate the percent absorption using the following formula:

$$\text{Absorption } P = \frac{B\text{-}A \times 100}{A}$$

where:

P = percent absorption of the aggregate

A = weight in grams of saturated, surface-dry sample.

- The percent absorption represents the moisture content (oven-dry basis) of the aggregate when the aggregate is in a SSD condition.

Procedure One: Surface Moisture

Surface moisture is the water which is present in both the fine and coarse aggregate, exceeding that which corresponds to a saturated, surface-dry condition. This water will become part of the mixing water when the aggregate is used in making concrete. The amounmt of mixing water used must be corrected to allow for its presence. See ASTM C 566 and ASTM C 70.

The test procedure for determining the total moisture content and percent surface moisture of either fine or coarse aggregate is as follows:

- Secure a sample of aggregate representative of the moisture content and of known weight, 0.1 percent.
- Dry the sample to a constant weight at a temperature of 110°C in a metal sample container. Take care to avoid loss of material during testing. The sample is thoroughly dry when further heating causes, or would cause, less than 0.1 percent additional loss in weight.
- Determine again the sample weight to the nearest 0.1 percent and record it.
- Calculate the total moisture content as follows:

$$P = \frac{W\text{-}D}{D} \times 100$$

where:

P = total moisture content, percent

W = weight in grams of original sample

D = weight in grams of dried sample.

The surface moisture is equal to the difference between the total moisture content and the absorption.

Procedure Two: Surface Moisture

An alternate determination of surface moisture in fine aggregate is obtained by displacement of water as follows:

- Secure a representative sample of fine aggregate of known weight, not less than 200 grams. Larger samples will yield more accurate results.

- Determine the weight of the pycnometer filled with water.

- Introduce the sample into the half water-filled pycnometer and refill it with water to the calibration mark. Remove all entrained air.

- Determine the weight of the pycnometer, sample, and water.

- Calculate the weight of the water displaced by the sample using the following equation:

$$V_s = W_c + W_s - W$$

where:

V_s = weight in grams of displaced water

W_c = weight in grams of pycnometer filled with water

W_s = weight in grams of sample

W = weight in grams of pycnometer, sample, and water

- The surface moisture is calculated as follows:

$$P = \frac{V_s - V_d}{W_s - V_s} \times 100$$

where:

P = percent of surface moisture

V_s = weight in grams of displaced water

W_s = weight in grams of sample

V_d = weight in grams of sample divided by the sample bulk specific gravity.

TESTS FOR IMPURITIES

The quality of the aggregate is an important consideration. The presence of organic material, excessive quantities of clay or silt, and shale or other water-absorbing particles can be detrimental to the concrete strength, watertightness, and durability. Recommended permissible limits for deleterious substances in fine and coarse aggregates are given in Tables 4-3 and 4-4, page 4-16.

Table 4-3. Limits for deleterious substances in fine aggregate for concrete

Item	Maximum, percent by weight of total sample
Clay lumps	1.0
Material finer than No. 200 sieve:	
Concrete subject to abrasion	3.0*
All other concrete	5.0*
Coal and lignite:	
Where surface appearance of concrete is of importance	0.5
All other concrete	0.1

* In the case of manufactured sand, if the material finer than the No. 200 sieve consists of the dust of fracture, essentially free from clay or shale, these limits may be increased to 5 and 7 percent, respectively.

TEST FOR MATERIAL FINER THAN 75-MICROMETERS (NO. 200) SIEVE (ASTM C 117)

The presence of fine particles of soil and clay can affect the concrete in two ways. The added surface area of the fine particles picks up the cement paste and reduces the amount available

Table 4-4. Limits for deleterious substances in coarse aggfegate for concrete

Class designation	**Type of location of concrete construction**	**Maximum allowable, percent**						
		Clay lumps and friable particles	**Chart (less than 2.40 sp gr SSD)[3]**	**Sum of clay lumps, friable particles and chart (less than 2.40 sp gr SSD)[3]**	**Material finer than 75-m (No.200) sieve**	**Coal and lignite**	**Abrasion[1]**	**Magnesium sulfate soundness (5-cycles)[2]**
		Severe weathering regions						
1S	Footings, foundations, columns and beams not exposed to the weather, interior floor slabs to be given coverings.	10.0	—	—	1.0[4]	1.0	50	—
2D	Interior floors without coverings.	5.0	—	—	1.0[4]	0.5	50	—
3S	Foundation walls above grade, retaining walls, abutments, piers, girders, and beams exposed to the weather.	5.0	5.0	7.0	1.0[4]	0.5	50	18
4S	Pavements, bridge decks, driveways and curbs,, walks, patios, garage floors, exposed floors and porches, or waterfront structures subject to frequent wetting.	3.0	5.0	5.0	1.0[4]	0.5	50	18
5S	Exposed architectural concrete.	2.0	3.0	3.0	1.0[4]	0.5	50	18
		Moderate weathering regions						
1M	Footings, foundations, columns, and beams not exposed to the weather, interior floor slabs to be given coverings.	10.0	—	—	1.0[4]	1.0	50	—
2M	Interior floors without coverings.	5.0	—	—	1.0[4]	0.5	50	—
3M	Foundation walls above grade, retaining walls, abutments, piers, girders, and beams exposed to the weather[5].	5.0	8.0	10.0	1.0[4]	0.5	50	18
4M	Pavements, bridge decks, driveways and curbs, walks, patios, garage floors, exposed floors and porches, or waterfront structures subject to frequent wetting	5.0	5.0	7.0	1.0[4]	0.5	50	18
5M	Exposed architectural concrete[5].	3.0	3.0	5.0	1.0[4]	0.5	50	18
		Negligible weathering regions						
1N	Slabs subject to traffic abrasion, bridge decks, floors, sidewalks, pavements[5].	5.0	—	—	1.0[4]	0.5	90	—
2N	All other classes of concrete[5].	10.0	—	—	1.0[4]	1.0	50	—

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[1] * Crushed air-cooled blast-furnace slag is excluded from the abrasion requirements. The compact unit weight of crushed air-cooled blast-furnace slag shall be not less than 70 lb/ft^3 (1120 kg/m^3). The grading of slag used in the unit weight test shall conform to the grading to be used in the concrete. Abrasion loss of gravel, crushed gravel, or crushed stone shall be determined on the test size or sizes most nearly corresponding to the grading or gradings to be used in the concrete. When more than one grading is to be used, the limit on abrasion loss shall apply to each.

[2] The allowable limits for soundness shall be 12 percent if sodium sulfate is used.

[3] These limitations apply only to aggregates in which chert appears as an impurity. They are not applicable to gravels that are predominantly chert. Limitations on soundness of such aggregates must be based on service records in the environment in which they are used.

Table 4-4. Limits for deleterious substances in coarse aggregate for concrete (continued)

[4] If the material finer than 75-m (No. 200) sieve is essentially free of clay or shale, this percentage may be increased by 1.5. A greater amount of material passing the 75-m sieve may be permitted, provided the amount passing the 75-m sieve in the fine aggregate is less than the specified maximum. In such case, the sum of the amounts finer than the 75-m sieve from the separate fine and coarse aggregate shall not exceed the sum of the weighted maximum amounts permitted for the coarse plus fine aggregate.

[5] For construction at altitudes exceeding 5,000 feet above sea level, the requirements of the Severe Weather Region shall apply.

to bind and hold the aggregate. The small particles also tend to float up to the surface when the concrete is finished (especially when wet mixes are used). This results in a surface covered by hairline cracks and a tendency for the fines to dust off when dry.

Apparatus

Two sieves, 75 micrometers and 1.18 millimeters (No. 200 and No. 16) and a container large enough to hold the sample covered with water and permit vigorous agitation without loss of sample or water are required.

Procedure

- Oven-dry the sample at 110° ± 5CC (230° ± 9°F), weigh it and record the weight. The sample, after drying, will conform to the following:

Nominal Maximum Size in Millimeters	Minimum Weight in Grams
2.36 (No. 8)	100
4.75 (No. 4)	500
9.5 (3/8 inch)	1,000
19.0 (3/4 inch)	2,500
37.5 (1 1/2 inches)	5,000

- Place the dry sample into the container and cover it with water.

- Agitate the material vigorously to bring the fine portion into suspension.

- Decant the water through the nest of sieves. Return all material retained on the nest of sieves to the sample container.

- Repeat the operation until the water runs clear.

- Oven-dry the sample at 110° ± 5°C (230° ± 9°F) to a constant weight.

- Weigh the dry sample and record the weight.

Calculation

The amount of material passing a 75-micrometer (No. 200) sieve is as follows:

$$A = \frac{B\text{-}C}{B} \times 100$$

where:

A = material finer than 75 micrometers, percent

B = original dry weight of sample, in grams

C = washed dry weight of sample, in grams.

TEST FOR CLAY LUMPS AND FRIABLE PARTICLES IN AGGREGATES (ASTM C 142)

This test is performed on the material remaining after determining the material finer than the 75-micrometer (No. 200) sieve. The cleaned material is oven-dried at 110° ± 5°C (230° ± 9°F), weighed, and separated into sample sizes. For FA, a minimum 25-gram sample of material coarser than the 1.18-millimeter (No. 16) sieve is required. CA are separated into four size ranges as follows: 4.75 to 9.5 millimeters (No. 4 to 3/8 inch), 9.5 to 19.0 millimeters (3/8 to 3/4 inch), 19.0 to 38.1 millimeters (3/4 to 1 1/2 inch), and 38.1 millimeters (1 1/2 inch). Minimum sample weights for these four size ranges are 1,000, 2,000, 3,000, and 5,000 grams respectively. The tests are performed as follows:

Apparatus

Use the apparatus for the fine-material test.

Procedure

Spread the sample in a thin layer on the bottom of a flat pan and cover it with distilled water for 24 ± 4 hours. Roll and squeeze all particles between thumb and forefinger to attempt to break up the particles. Remove from the sample all the broken clay lumps and friable particles by wet sieving them over one of the following sieves: aggregate - 850 micrometers (No. 20), 4.75 to 9.5 millimeters (No. 4 to 3/8 inch) - 2.36 millimeters (No. 8), over 9.5 millimeters (3/8 inch) - 4.75 millimeters No. 4). Remove the retained sample and dry to constant weight at 110° ± 5°C (230° ± 9°F), weigh it, and record the weight.

Calculations

Calculate the percentage of clay lumps and friable particles as follows:

$$P = \frac{W-R}{W} \times 100$$

where:

P = amount of clay lumps and friable particles, percent

W = weight of test sample

R = weight of retained sample after wet sieving and drying.

TEST FOR LIGHTWEIGHT PIECES IN AGGREGATE (ASTM C 123)

Soft, laminated pieces of aggregate such as low-density chert, shale, coal, lignite, and other low density particles are detrimental to concrete. Coal and lignite also are harmful and are distinguished from other lightweight materials by the brownish-black or black color of the particles. Visual examination of the CA will often show these minerals. The amount of undesirable aggregate can be determined by immersing the aggregate in a heavy liquid with a specific gravity between that of the predominant aggregate and that of the low density particles. The sink-float separation procedure for this test is as follows:

The minimum sample size is:

Maximum Size in Millimeters	Minimum Sample Size in Grams
6.35	200
19.0	3,000
38.1	5,000
76.1	10,000

Sink-float Separation Procedure

- Sieve the dried sample over a 297-micrometer (No. 50) sieve for FA, or a 4.76-millimeter (No. 4) sieve for CA and weigh it. Then bring the aggregate to SSD condition.

- Introduce the sample into the heavy liquid whose volume is, at a minimum, three times the volume of the sample. The specific gravity of 1,1, 2,2 - tetrabromethane is 2.97. This, or any other liquid with this specific gravity, can be used.

- Agitate the mixture to allow the lightweight pieces to float to the surface.

- Skim off the lighter particles that are floating, being careful not to remove any other particles.

- Repeat the operation until further agitation does not bring other particles to the surface.

- Remove all the undesirable lightweight material, wash the sample in alcohol, dry it, and weigh the pieces. The difference between the final weight and the original dried weight (expressed as a percentage of the total weight) represents the undesirable material in the aggregate.

Calculations

The percent of lightweight pieces in aggregate is calculated as follows:

$$L = \frac{W_1}{W_2} \times 100$$

where:

L = amount of lightweight pieces, percent

W_1 = dry weight of lightweight pieces

W_2 = dry weight of initial sample retained on 297-micrometer sieve for fine aggregate or 4.76 millimeter sieve for CA.

COLOR TEST FOR ORGANIC MATTER

Any sand that gives a color darker than the standard of this test probably contains an excess of organic matter which will give trouble in concrete. It is possible that the organic matter can be removed by washing. If not, better sand should be obtained. If neither of these things can be done it is necessary to use a lower W/C ratio and control the concrete production carefully in order to obtain the desired strength.

Apparatus

The following items are required to conduct this test (Figure 4-3):

Acid, tannic

Alcohol, ethyl

Bottle, prescription, 12 ounce, graduated

Cylinder, 100 cubic centimeters (cc), graduated

Plate, color

Sodium hydroxide, caustic soda.

Procedure

Prepare a reference standard color solution by adding 2.5 mililiters of 2-percent solution of tannic acid in 10 percent alcohol to 97.5

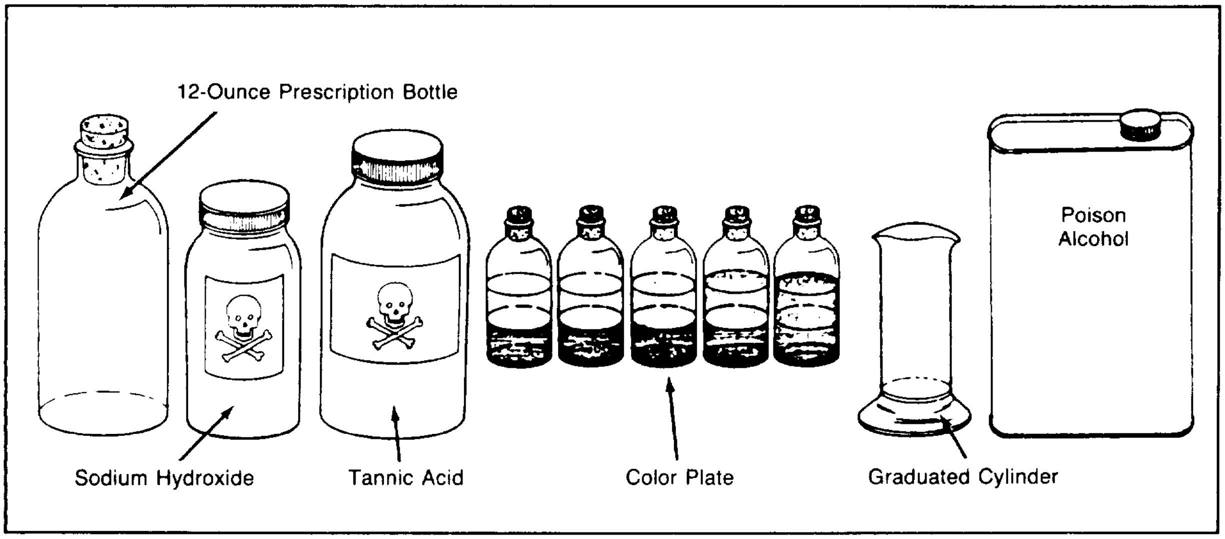


Figure 4-3. Apparatus for color test on organic matter

milliliters of a 3-percent sodium hydroxide solution. Make the three-percent sodium hydroxide solution by dissolving 1 ounce of sodium hydroxide in enough water to make 32 fluid ounces. Make the 2-percent tannic acid solution by adding 10 milliliters of 190 proof alcohol and 2 grams of tannic acid powder to 90 milliliters of water. Pour this solution into a 12-ounce bottle. Stopper and shake it vigorously. Allow it to stand for 24 hours.

- Obtain a representative sample of about 500 grams of the material to be tested.

- Fill a 12-ounce graduated bottle to the 4-1/2 ounce mark with the sample to be tested. Add a 3-percent solution of sodium hydroxide in water until the volume of sand and water after shaking is 7 liquid ounces. Stopper the bottle and shake it vigorously. Allow it to stand for 24 hours.

- After the standard color solution and the test sample have been allowed to stand 24 hours compare the color of the referenced solution with the color of the clear liquid above the test sample.

Evaluation of Results

If the liquid above the sample is darker than the standard color solution, the sand may contain organic impurities which will reduce the strength of the concrete in which the sand is used.

Alternate Test Procedure

After the test sample has been prepared and the sample has been allowed to stand for 24 hours, the color of the liquid above the sample may be compared with the colors given in the standard color plate. A decision may then be made as to whether the sand contains an excessive amount of organic matter.

TEST FOR SOUNDNESS

Soundness is the property of aggregate to resist disintegration due to freezing and thawing. Soundness involves many factors, such as porosity, pore structure, texture, and permeability. Freezing and thawing of aggregate in concrete probably provide the best measure for the soundness of aggregate. Soundness is also measured by means of a sulfate salt, sodium or magnesium, which simulates the volume pressure increase of freezing water.

Procedure for Test for Soundness by Freezing and Thawing

This procedure covers the determination of the aggregate to resist the influences of rapid freezing and thawing in concrete.

- Check that the concrete containing the test aggregate will conform to the following material requirements and characteristics:

Contain Type II cement in amounts ranging from 279 to 390 kilograms per cubic meter (470 to 658 pounds per cubic yard)

Air content of 6.0 ± 0.5 percent

Slump of 64 ± 13 millimeters (2 1/2 ± 1/2 inches)

- Water-cement ratio of 0.49 by weight.

- Fine aggregate grading will comply to the following:

Sieve	Percent Passing
4.75 millimeters (No. 4)	100
2.36 millimeters (No. 8)	85 ± 3
1.18 millimeters (No. 16)	65 ± 5
600 micrometers (No. 30)	45 ± 5
300 micrometers (No. 50)	21 ± 5
150 micrometers (No. 100)	7 ± 2

- Coarse aggregate grading shall comply to the following:

Sieve	Percent Passing
19.0 millimeters (3/4 inch)	97-100
12.5 millimeters (1/2 inch)	66 ± 3
9.5 millimeters (3/8 inch)	33 ± 3
4.75 millimeters (No. 4)	0-3

- Fabricate concrete (89 by 114 by 406 millimeters, or 3-1/2 by 4-1/2 by 16 inches) test beams and cure in moist condition for 14 days. Fundamental transverse frequency will be determined on each specimen which are then placed in storage containers and inundated in water at 40° ± 3°F (4° ± 1.7°C) for not less than one 1.

- Place the specimen and container into the freezing and thawing apparatus and begin cycles.

- Determine the fundamental frequency after cycles 1, 10, and at intervals not exceeding 36 cycles until the value of dynamic E has decreased to 50 percent of the initial measurement, or for a total of 300 cycles.

- When determing the fundamental frequency, remove the specimens (still in containers) from the freeze-thaw apparatus during the thaw cycle and store it in a water bath at 40° ± 3°F. When the specimen is removed from the container, note the appearance before and after washing. Then, determine the fundamental frequency. Turn the specimen end-for-end when replacing the specimen back into its container and replace the container back into the freezing-thawing apparatus.

- Should an interruption occur store the specimens in the freeze cycle until resumption of cycle is started.

- Calculate the average relative dynamic modulus of elasticity (Rel E) as the arithmetic mean for a group of three specimens. The Rel E is calculated for individual specimens as follows:

$$\text{Rel E}_y = \frac{50\text{ N}}{y}$$

where:

y = number of cycles, greater than N, at which Rel E is to be calculated

N = number of cycles at which Rel E = 50.

- The durability factor (DFE) based on relative dynamic modulus of elasticity is calculated as follows:

$$\text{DFE} = \frac{\text{PN}}{\text{M}}$$

where:

P = relative dynamic modulus of elasticity in percentage of dynamic modulus at zero cycles

N = number of cycles at which P reaches the limiting value in percentage or the ultimate number of cycles of the test if P does not reach the limiting value prior to the ultimate cycle

M = ultimate number of cycles of the test.

Procedure for Test for Soundness Using Sodium or Magnesium Sulfates (ASTM C 88)

The freezing-thawing evaluation of aggregates requires considerable equipment and occasionally has been replaced by another test that requires less time and equipment. This procedure involves solutions of special salts in which the aggregate is immersed and saturated. The crystals of these salts are permitted to grow which creates a disruptive force similar to freezing water. The salts used are sodium sulfate and magnesium sulfate. The procedure is as follows:

- Secure a representative sample of aggregate. The sample will be separated into sizes not less than 100 grams. Each size will contain more than 5 percent of the total sample. The sample for each gradation size is given in Table 4-5.

- Immerse the sample fractions to a depth of 1/2 inch in a saturated solution of sodium sulfate or magnesium sulfate for a period of 16 to 18 hours at a temperature of 21° ± 1°C (70° ± 2°F).

- Drain the sample for 15 ± 5 minutes after the immersion period. Then oven-dry the aggregate at a temperature of 110° ± 5°C (230° ± 9°F) until constant weight has been achieved.

- Allow the sample to cool to room temperature and repeat the cycle until the required number of cycles is obtained (generally 5 cycles are sufficient).

- Wash the fractions following the final cycle until all the salt is removed. Add a small amount of barium chloride to the wash water. This will cause a precipitate to form if any salt is left. Continue washing until no precipitate is seen.

- Dry the washed fractions to a constant weight at a temperature of 110° ± 5°C (230° ± 9°F). Then sieve each fraction on its designated sieve. For FA, sieve the fractions over the same size of sieve using same procedure. For CA, sieve the fractions over the following sieves:

Weigh the material retained on each sieve and record the amount. The difference between this amount and the initial weight of each fraction is the loss.

Express the loss of each fraction as a percentage of the initial weight and express the sample as a percentage of the total weight.

Size of Aggregate	Sieve Used to Determine Loss
2½ (63 millimeters) to 1½ inch (37.5 millimeters)	1¼ inch (31.5 millimeters)
1½ to ¾ inch (19.0 millimeters)	5/8 inch (16.0 millimeters)
¾ to ⅜ inch (9.5 millimeters)	5/16 inch (8.0 millimeters)
⅜ inch to No. 4 (4.75 millimeters)	No. 5 (4.0 millimeters)

Table 4-5. Minimum amounts of material for aggregate soundness test

Fine aggregate		
Passing	**Retained**	**Minimum amount**
600 millimeters (No. 30)	300 millimeters (No. 50)	100 grams
1.18 millimeters (No. 16)	600 millimeters (No. 30)	100 grams
2.36 millimeters (No. 8)	1.18 millimeters (No. 16)	100 grams
4.75 millimeters (No. 4)	2.36 millimeters (No. 8)	100 grams
9.5 millimeters (3/8 in)	4.75 millimeters (No. 4)	100 grams

Size (Square-Opening Sieves)	**Weight, grams**
3/8 inch (9.5 millimeters) to No. 4 (4.75 millimeters)	300 ± 5
3/4 (19.0 millimeters) to 3/8 inch	1,000 ± 10
Consisting of:	
1/2 (12.5 millimeters) to 3/8-inch material	330 ± 5
3/4 to 1/2-inch material	670 ± 10
1-1/2 (37.5 millimeters) to 3/4-inch	1,500 ± 50
Consisting of:	
1 (25.0 millimeters) to 3/4-inch material	500 ± 30
1-1/2 to 1-inch material	1,000 ± 50
2-1/2 (63 millimeters) to 1-1/2-inch	3,000 ± 300
Consisting of:	
2 (50 millimeters) to 1-1/2-inch material	2,000 ± 200
2-1/2 to 2-inch material	3,000 ± 300
Larger sizes by 1-inch spread in sieve size, each fraction	7,000 ± 1,000

Section III. IDENTIFICATION OF CEMENTS

DISCUSSION

The positive identification of cement, because of the very wide variety of related or similar appearing materials, requires a complete chemical and physical analysis. The different types of cement have been discussed. Every effort should be made to identify the unknown material directly by obtaining and translating all labels, tags, shipping documents, manufacturing sheets, and other papers which may contain applicable information. When this does not produce results, the simple procedures outlined in the following paragraphs generally will supply enough information to permit a tentative, if not conclusive, identification.

A complete chemical and physical analysis of the cement may be required to completely identify the cement and to determine whether the cement may be used in concrete.

TESTS

The following tests are made to determine whether the material is a cement, and then to attempt to identify its type.

Hardening

A small sample of the material is selected and mixed with enough water to make a plastic paste of a consistency similar to that generally used in cement mortars. It is then molded into a pat about 3 inches in diameter and 3/4 inch thick. The paste should be observed several times an hour to determine whether or not the paste is setting (hardening). The cement has attained a final set when the surface is hard enough to be unmarked when a pencil point or fingernail is pressed against it with moderate force. If it sets within 1 to 10 hours, the material is probably a cement.

Color

If it has been fairly well established that the material in question is a cement, color may serve as a means of further classification. If the material is gray it is likely to be a portland cement. If it is brownish gray it may be a natural cement. If it is black it is an aluminous cement. If it is white it probably is hydraulic lime, plaster, or possibly white portland cement.

AIR-ENTRAINED CEMENT

To determine whether or not a given material contains an air-entraining agent, a sample is placed in a glass cylinder to a depth of about 1 inch. Water is added to a depth of about 6 inches and the cylinder and its contents are shaken vigorously. If a considerable volume of stable, persistent foam forms on the surface, the cement probably contains an air-entraining agent.

HIGH-EARLY STRENGTH CEMENT

High-early strength cement (Type III), may be recognized by making a batch of concrete using the unknown cement in parallel with a similar batch which uses a known cement. Concrete which contains high-early strength cement will usually harden in less time than concrete which contains regular portland cement. High-early strength cement concrete, if molded into standard concrete beams and tested after 3 days for flexural strength, should have a modulus of rupture more than 150 pounds per square inch higher than similar specimens containing regular portland cement concrete.

Section IV. PROPORTIONING OF MIXES

DISCUSSION

Cement, water, and FA and CA are the basic materials used in the production of concrete. Certain admixtures are used occasionally to meet special requirements. Design of a concrete mix consists of determining the correct amount of each material to provide a mixture of concrete having the necessary consistency or workability in the freshly-mixed condition and having the desired strength and durability characteristics in the hardened condition. The workability required depends upon mixing, placing, and finishing conditions of the job and is specific in terms of the slump test. Strength requirements are specified by the engineer for the particular construction, and exposure conditions determine the correct W/C ratio to be used. Gradation of the FA and CA is the factor that controls the proportion of each in the total amount of aggregate. The maximum size of CA particles is limited by the dimensions of the structure being built.

CA size and workability requirements establish the amount of water needed, hence the W/C ratio fixes the required quantity of cement in the mixture. Sometimes the minimum quantity of cement (the cement factor, in 94-pound bags per cubic yard of concrete) is specified. The care and intelligence used in proportioning these ingredients are evident in the finished product of poor or quality concrete. There are three methods of proportioning available to the military engineer; the book method, the trial batch method, and the absolute volume method.

SELECTION OF MIXTURE CHARACTERISTICS

The strength requirements of the concrete, the exposure conditions, and the kind and size of objects to be constructed must be known to design a mixture having the needed characteristics. Selection of the W/C ratio fixes, to a large extent, the determined quality of the hardened concrete. Total quantity of cement and water then is found from the workability requirements. Finally, the aggregate proportions are determined.

Water-cement Ratio

Recommended values of W/C ratio for various strength and exposure requirements are given in Table 4-6 and 4-7, page 4-26, respectively. To attain the desired compressive strength of concrete, the W/C ratio should not exceed the value shown in Table 4-6. The type of structure and degree of exposure also set maximum limits on W/C ratio as given in Table 4-7. The smaller ratio derived from these two tables governs the mixture design.

Table 4-6. Compressive strength of concrete for various water-cement ratios

Water-cement ratio, gal per bag of cement	Probable compressive strength at 28 days, psi	
	Non air-entrained concrete	Air-entrained concrete
4	6,000	4,800
5	5,000	4,000
6	4,000	3,200
7	3,200	2,600
8	2,500	2,000
9	2,000	1,600

Table 4-7. Maximum permissible water-cement ratios for concrete in severe exposure

Type of structures	Exposure conditions*					
	Severe wide range in temperature or frequent alternations of freezing and thawing (air-entrained concrete only) (gallons/sack)			**Mild temperature rarely below freezing, or rainy, or arid (gallons/sack)**		
	In air	**At water line or within range of fluctuating water level or spray**		**In air**	**At water line or within range of fluctuating water level or spray**	
		In fresh water	**In sea water or in contact with sulfates****		**In fresh water**	**In sea water or in contact with sulfates****
Thin sections such as reinforced piles and pipe.	5.5	5	4.5	6	5.5	4.5
Bridge decks.	5	5	4.5	5.5	5.5	5
Thin sections such as railings, curbs, sills, ledges, ornamental or architectural concrete, and all sections with less than 1-in. concrete cover over reinforcement.	5.5	—	—	6	5.5	—
Moderate sections, such as retaining walls, abutments, piers, girders, beams.	6	5.5	5	***	6	5
Exterior portions of heavy (mass) sections.	6.5	5.5	5	***	6	5
Concrete deposited by tremie under water.	—	5	5	—	5	5
Concrete slabs laid on the ground.	6	—	—	***	—	—
Pavements.	5.5	—	—	6	—	—
Concrete protected from the weather, interiors of buildings, concrete below ground.	***	—	—	***	—	—
Concrete which will later be protected by enclosure or backfill but which may be exposed to freezing and thawing for several years before such protection is offered.	6	—	—	***	—	—

This table was adapted from Recommended Practice for Selecting Proportions for Concrete (ACI 613-54).

*Air-entrained concrete should be used under all conditions involving severe exposure and may be used under mild exposure conditions to improve workability of the mixture.

**Soil or groundwater containing sulfate concentrations of more than 0.2 percent. For moderate sulfate resistance, the tricalcium aluminate content of the cement should be limited to 8 percent, and for high-sulfate resistance to 5 percent. At equal cement contents, air-entrained concrete is significantly more resistant to sulfate attack than nonair-entrained concrete.

***Water-cement ratio should be selected on basis of strength and workability requirements, but minimum cement content should not be less than 470 lb per cubic yard.

Consistency

Workability or consistency of freshly-mixed concrete is measured by the slump test. Recommended slumps (in inches) for various types of construction are given in Table 4-8. The stiffness of the mixture is inversely proportional to slump. In absence of any other criteria, concrete should be placed as stiff as possible, yet maintain a homogeneous, voidless mass.

Water Content

The amount of mixing water required for the desired slump depends upon maximum size of aggregate. Table 4-9, page 4-28, gives approximate water requirements in gallons per cubic yard of concrete for concrete mixtures made from different sizes of aggregate for ranges in slump.

BOOK METHOD OF MIXTURE PROPORTIONING

Concrete may be made with a wide variation in quality. The considerations which the proportioning must follow are given under the desirable properties of concrete. The selections of the correct W/C, ratio, the allowable slump, and the type and size of aggregate are all part of the design function. Tables have been derived for aggregates having properties within the specified limits for concrete of satisfactory quality. The design engineer will use the tables to establish the weights of materials to be used and the expected yield for a sack of cement batch. The mixture proportions determined are not always the most economical, however, and the trial batch method should be used to design the mixture when time is available and the size of the job is large enough to warrant the laboratory effort required. The mixture proportions developed by the book method may be modified if the desired workability is not obtained. Such modifications, if needed, are made by changing the proportions of FA to CA or by changing the amount per cubic yard of cement-water paste. Do not change the W/C ratio in making these modifications. Details of selecting mixture characteristics and quantities of materials are given in the following paragraphs.

Water-cement Ratio

Select W/C ratio to meet strength and exposure conditions. Use the smaller of the two W/C ratio values given in Table 4-6, page 4-25, and Table 4-7. Concrete exposed to the weather in temperate climate or colder zones should contain 4.0 to 7.5 percent of entrained air. Determine the maximum size of aggregate from dimensions of members or the limitations of available materials. For economy, use the maximum size aggregate available. Determine by

Table 4-8. Recommended slumps for various types of construction

Types of construction	Slump, inches*	
	Maximum	Minimum
Reinforced foundation walls and footings	5	2
Plain footings, caissons, and substructure walls	4	1
Reinforced slabs, beams, and walls	6	3
Building columns	6	3
Pavements	3	2
Heavy mass construction	3	2
Bridge decks	4	3
Sidewalks, driveways, and slabs on ground	6	3

*When high-frequency vibrators are used, the values may be decreased approximately one-third; in no case should the slump exceed 6 inches.

sieving or estimate the fineness modulus of the sand. Establish the required slump to meet job placement conditions.

Slump

Table 4-10 gives suggested mixtures for air-entrained and nonair-entrained concrete, respectively, with a 3-inch to 4-inch slump. Using the W/C ratio and aggregate size required, select the appropriate weights of water and cement per cubic yard of concrete. Select the weights of FA and CA from the columns in the table that are appropriate for the fineness modulus of the sand. If the slump is to be different from the 3-inch to 4-inch range of Table 4-10, refer to the table footnote for corrections to the book quantities.

Optimum Aggregate Gradation

This depends upon the proportion of FA to CA. The volume of CA per unit volume of concrete is given in Table 4-11, page 4-33, for four values of fineness modulus of FA. This table may be used to estimate mixture quantities when the range in Table 4-10 is exceeded. Determine the W/C ratio (Tables 4-6, page 4-25 and 4-7, page 4-26), water content (Tables 4-8, page 4-27, and 4-9, and percent of entrained air to suit requirements. Find the cubic foot volume of CA per cubic yard from Table 4-11, page 4-33. Calculate absolute volumes in cubic feet, of water, cement, air, and CA per cubic yard. Absolute volume in cubic feet is the weight of the material divided by 62.4 (the weight of 1 cubic foot of water) and the specific gravity of the material. For aggregate, use saturated, surface-dry specific gravity. Subtract this total from 27 cubic feet to get the absolute volume of FA and convert this to weight.

Table 4-9. Approximate mixing water requirements for different slumps and maximum sizes of aggregates

Slump, (stated in inches)	Water, gal per cu yd of concrete, for indicated maximum sizes of aggregate stated in inches							
	3/8	1/2	3/4	1	1 1/2	2	3	6
Nonair-entrained concrete								
1 to 2	42	40	37	36	33	31	29	25
3 to 4	46	44	41	39	36	34	32	28
5 to 6	49	46	43	41	38	36	34	30
Approximate amount of entrapped air in nonair-entrained concrete, percent	3	2.5	2	1.5	1	0.5	0.3	0.2
Air-entrained concrete								
1 to 2	37	36	33	31	29	27	25	22
3 to 4	41	39	36	34	32	30	28	24
5 to 6	43	41	38	36	34	32	30	26
Recommended average total air content, percent	8	7	6	5	4.5	4	3.5	3

Note: These quantities of mixing water are for use in computing cement factors for trial batches. They are maximums for reasonably well-shaped angular coarse aggregates graded within limits of accepted specifications. If more water is required than shown, the cement factor, estimated from these quantities, should be increased to maintain desired water-cement ratio, except as otherwise indicated by laboratory tests for strength. If less water is required than shown, the cement factor, estimated from these quantities, should not be decreased except as indicated by laboratory tests for strength.

Table 4-10. Trial mixtures for concrete of medium consistency

Air-entrained concrete

Water-cement ratio gal per sack	Maximum size of aggregate inches	Air content (entrapped air) percent	Water gal per cu yd of concrete	Cement sacks per cu yd of concrete	With fine sand – fineness modulus = 2.50		
					Fine aggregate percent of total aggregate	Fine aggregate lb per cu yd of concrete	Coarse aggregate lb per cu yd of concrete
4.5	1⅜	7.5	41	9.1	50	1250	1260
	½	7.5	39	8.7	41	1060	1520
	¾	6	36	8.0	35	970	1800
	1	6	34	7.8	32	900	1940
	1½	5	32	7.1	29	870	2110
5.0	⅜	7.5	41	8.2	51	1330	1260
	½	7.5	39	7.8	43	1140	1520
	¾	6	36	7.2	37	1040	1800
	1	6	34	6.8	33	970	1940
	1½	5	32	6.4	31	930	2110
5.5	⅜	7.5	41	7.5	52	1390	1260
	½	7.5	39	7.1	44	1190	1520
	¾	6	36	6.5	38	1090	1800
	1	6	34	6.2	34	1010	1940
	1½	5	32	5.8	32	970	2110
6.0	⅜	7.5	41	6.8	53	1430	1260
	½	7.5	39	6.5	45	1230	1520
	¾	6	36	6.0	38	1120	1800
	1	6	34	5.7	35	1040	1940
	1½	5	32	5.3	32	1010	2110
6.5	⅜	7.5	41	6.3	54	1460	1260
	½	7.5	39	6.0	45	1260	1520
	¾	6	36	5.5	39	1150	1800
	1	6	34	5.2	36	1080	1940
	1½	5	32	4.9	33	1040	2110
7.0	⅜	7.5	41	5.9	54	1500	1260
	½	7.5	39	5.6	46	1300	1520
	¾	6	36	5.1	40	1180	1800
	1	6	36	5.1	40	1180	1800
	1½	5	32	4.6	33	1060	2110
7.5	⅜	7.5	41	5.5	55	1530	1260
	½	7.5	38	5,2	46	1339	1520
	¾	6	36	4.8	40	1210	1800
	1	6	34	4.5	37	1140	1940
	1½	5	32	4.3	34	1090	2110
8.0	⅜	6,5	41	5,1	55	1560	1260
	½	7.5	39	4.9	47	1360	1520
	¾	6	36	4.5	41	1240	1800
	1	6	34	4.3	37	1160	1940
	1½	5	32	4.0	34	1110	2110

Note: This table is for concrete with 3 to 4 inch slump. Increase or decrease water per cubic yard by 3 percent for each increase or decrease of 1 inch in slump, then calculate quantities by absolute volume method. For manufactured fine aggregate, increase percentage of fine aggregate by 3 and water by 17 lb per cubic yard of concrete. For less workable concrete, as in pavements, decrease percentage of fine aggregate by 3 and water by 8 lb per cubic yard of concrete

Table 4-10. Trial mixtures for concrete of medium consistency (continued)

Air-entrained concrete (continued)					
With average sand – fineness modulus = 2.75			With coarse sand – fineness modulus = 2.90		
Fine aggregate percent of total aggregate	Fine aggregate lb per cu yd of concrete	Coarse aggregate lb per cu yd of concrete	Fine aggregate percent of total aggregate	Fine aggregate lb per cu yd of concrete	Coarse aggregate lb per cu yd of concrete
53	1320	1190	54	1360	1150
44	1130	1450	46	1180	1400
35	1040	1730	39	1090	1680
34	970	1870	36	1010	1830
32	950	2030	33	990	1990
54	1400	1190	56	1440	1150
46	1210	1450	47	1260	1400
39	1110	1730	41	1160	1630
36	1040	1870	37	1080	1830
33	1010	2030	35	1050	1990
55	1460	1190	57	1500	1150
46	1260	1450	48	1310	1400
40	1160	1730	42	1210	1680
37	1080	1870	38	1120	1830
34	1050	2030	35	1090	1990
56	1500	1190	57	1540	1150
47	1300	1450	49	1350	1400
41	1190	1730	42	1240	1680
37	1110	1870	39	1150	1830
35	1090	2030	36	1130	1990
56	1530	1190	58	1570	1150
48	1330	1450	50	1380	1400
41	1220	1730	43	1270	1680
38	1150	1870	39	1190	1830
36	1120	2030	37	1160	1990
57	1570	1190	58	1610	1150
49	1370	1450	50	1420	1400
42	1250	1730	44	1300	1680
38	1170	1870	40	1210	1830
36	1140	2030	37	1180	1990
57	1600	1190	59	1640	1150
49	1400	1450	51	1450	1400
43	1280	1730	44	1330	1680
39	1210	1870	41	1250	1830
37	1170	2030	38	1210	1990
58	1630	1190	59	1670	1150
50	1430	1450	51	1480	1400
43	1310	1730	44	1360	1680
40	1230	1870	41	1270	1830
37	1190	2030	38	1230	1990

Table 4-10. Trial mixtures for concrete of medium consistency (continued)

Nonair-entrained concrete

Water-cement ratio gal per sack	Maximum size of aggregate inches	Air content (entrapped air) percent	Water gal per cu yd of concrete	Cement sacks per cu yd of concrete	With fine sand — fineness modulus = 2.50		
					Fine aggregate percent of total aggregate	Fine aggregate lb per cu yd of concrete	Coarse aggregate lb per cu yd of concrete
4.5	⅜	3	46	10.3	50	1240	1260
	½	2.5	44	9.8	42	1100	1520
	¾	2	41	9.1	35	960	1800
	1	1.5	39	8.7	32	910	1940
	1½	1	36	8.0	29	880	2110
5.0	⅜	3	46	9.2	51	1330	1260
	½	2.5	44	8.8	44	1180	1520
	¾	2	41	8.2	37	1040	1800
	1	1.5	39	7.8	34	990	1940
	1½	1	36	7.2	31	960	2110
5.5	⅜	3	46	8.4	52	1390	1260
	½	2.5	44	8.0	45	1240	1520
	¾	2	41	7.5	38	1090	1800
	1	1.5	39	7.1	35	1040	1940
	1½	1	36	6.5	32	1000	2110
6.0	⅜	3	46	7.7	53	1440	1260
	½	2.5	44	7.3	46	1290	1520
	¾	2	41	6.8	39	1130	1800
	1	1.5	39	6.5	36	1080	1940
	1½	1	36	6.0	33	1040	2110
6.5	⅜	3	46	7.1	54	1480	1260
	½	2.5	44	6.8	46	1320	1520
	¾	2	41	6.3	39	1190	1800
	1	1.5	39	6.0	37	1120	1940
	1½	1	36	5.5	34	1070	2110
7.0	⅜	3	46	6.6	55	1520	1260
	½	2.5	44	6.3	47	1360	1520
	¾	2	41	5.9	40	1200	1800
	1	1.5	39	5.6	37	1150	1940
	1½	1	36	5.1	34	1100	2110
7.5	⅜	3	46	6.1	55	1560	1260
	½	2.5	44	5.9	48	1400	1520
	¾	2	41	5.5	41	1240	1800
	1	1.5	39	5.2	38	1190	1940
	1½	1	36	4.8	35	1130	2110
8.0	⅜	3	46	5.7	56	1600	1260
	½	2.5	44	5.5	48	1440	1520
	¾	2	41	5.1	42	1280	1800
	1	1.5	39	4.9	39	1220	1940
	1½	1	36	4.5	35	1160	2110

Note: This table is for concrete with a 3 to 4 inch slump. Increase or decrease water per cubic yard by 3 percent for each increase or decrease of 1 inch in slump, then calculate quantities by absolute volume method. For manufactured fine aggregate, increase percentage of fine aggregate by 3 and water by 17 lb per cubic yard of concrete. For less workable concrete, as in pavements, decrease percentage of fine aggregate by 3 and water by 8 lb per cubic yard of concrete.

Table 4-10. Trial mixtures for concrete of medium consistency (continued)

Nonair-entrained concrete (continued).					
With average sand – fineness modulus = 2.75			With coarse sand – fineness modulus = 2.90		
Fine aggregate percent of total aggregate	Fine aggregate lb per cu yd of concrete	Coarse aggregate lb per cu yd of concrete	Fine aggregate percent of total aggregate	Fine aggregate lb per cu yd of concrete	Coarse aggregate lb per cu yd of concrete
52	1310	1190	54	1350	1150
45	1170	1450	47	1220	1400
37	1030	1730	39	1080	1680
34	980	1870	36	1020	1830
32	960	2030	33	1000	1990
54	1400	1190	56	1440	1150
46	1250	1450	48	1300	1400
39	1110	1730	41	1160	1680
36	1060	1870	38	1100	1830
34	1040	2030	35	1080	1990
55	1460	1190	57	1500	1150
47	1310	1450	49	1360	1400
40	1160	1730	42	1210	1680
37	1110	1870	39	1150	1830
35	1080	2030	36	1120	1990
56	1510	1190	57	1550	1150
48	1360	1450	50	1410	1400
41	1200	1730	43	1250	1600
38	1150	1870	39	1190	1830
36	1120	2030	37	1160	1990
56	1550	1190	58	1590	1150
49	1390	1450	51	1440	1400
42	1240	1730	43	1290	1680
39	1190	1870	40	1230	1830
36	1150	2030	37	1190	1990
57	1590	1190	59	1630	1150
50	1430	1450	51	1480	1400
42	1270	1730	44	1320	1680
39	1220	1870	41	1260	1830
37	1180	2030	38	1220	1990
57	1630	1190	59	1670	1150
50	1470	1450	52	1520	1400
43	1310	1730	45	1370	1600
40	1260	1870	42	1300	1830
37	1210	2030	39	1250	1990
58	1670	1190	60	1710	1150
51	1520	1450	53	1560	1400
44	1350	1730	45	1400	1680
41	1290	1870	42	1330	1830
38	1250	2030	39	1280	1990

Table 4-11. Volume of coarse aggregate per cubic yard of concrete

Maximum size of aggregate, in	Fineness modulus of fine aggregate			
	2.40	2.60	2.80	3.00
	Coarse aggregate, cu ft per cu yd*			
3/8	13.5	13.0	12.4	11.9
1/2	15.9	15.4	14.8	14.3
3/4	17.8	17.3	16.7	16.2
1	19.2	18.6	18.1	17.6
1 1/2	20.2	19.7	19.2	18.6
2	21.1	20.5	20.0	19.4
3	22.1	21.6	21.1	20.5

*Volumes are based on aggregates in dry-rodded condition. These volumes are selected from empirical relationships to produce concrete with a degree of workability suitable for usual reinforced construction. For less workable concrete such as required for concrete pavement construction, they may be increased about 10 percent. When placement is to be by pump, they should be decreased about 10 percent.

TRIAL BATCH METHOD OF MIXTURE PROPORTIONING

The trial batch method is a simple field method which incorporates many of the best features of several other methods. It may be used under almost any circumstances. The trial batch method is capable of producing, with a minimum number of tests, a satisfactory mixture using the materials used in the project. Such trial mixtures are relatively small batches made with laboratory precision to establish the proportions of materials that will produce a mixture of required workability characteristics, density, and economy. The minimum cement content or W/C ratio, maximum size of aggregate, air content, and consistency (slump) are selected to meet job requirements. Then mixtures are made in which the aggregate proportions are adjusted through trial batches until the most economical proportions that provide a dense, workable mixture are found. The final quantities of materials for the field mixtures are calculated by using a ratio from the quantities used in the selected trial batch.

Preparation of Aggregates

Some of the physical characteristics of the aggregates to be used must be determined before a mixture design can be carried out. Tests will be conducted to provide the following necessary information:

Gradation of FAs and CAs

Bulk specific gravities and absorption of both FAs and CAs

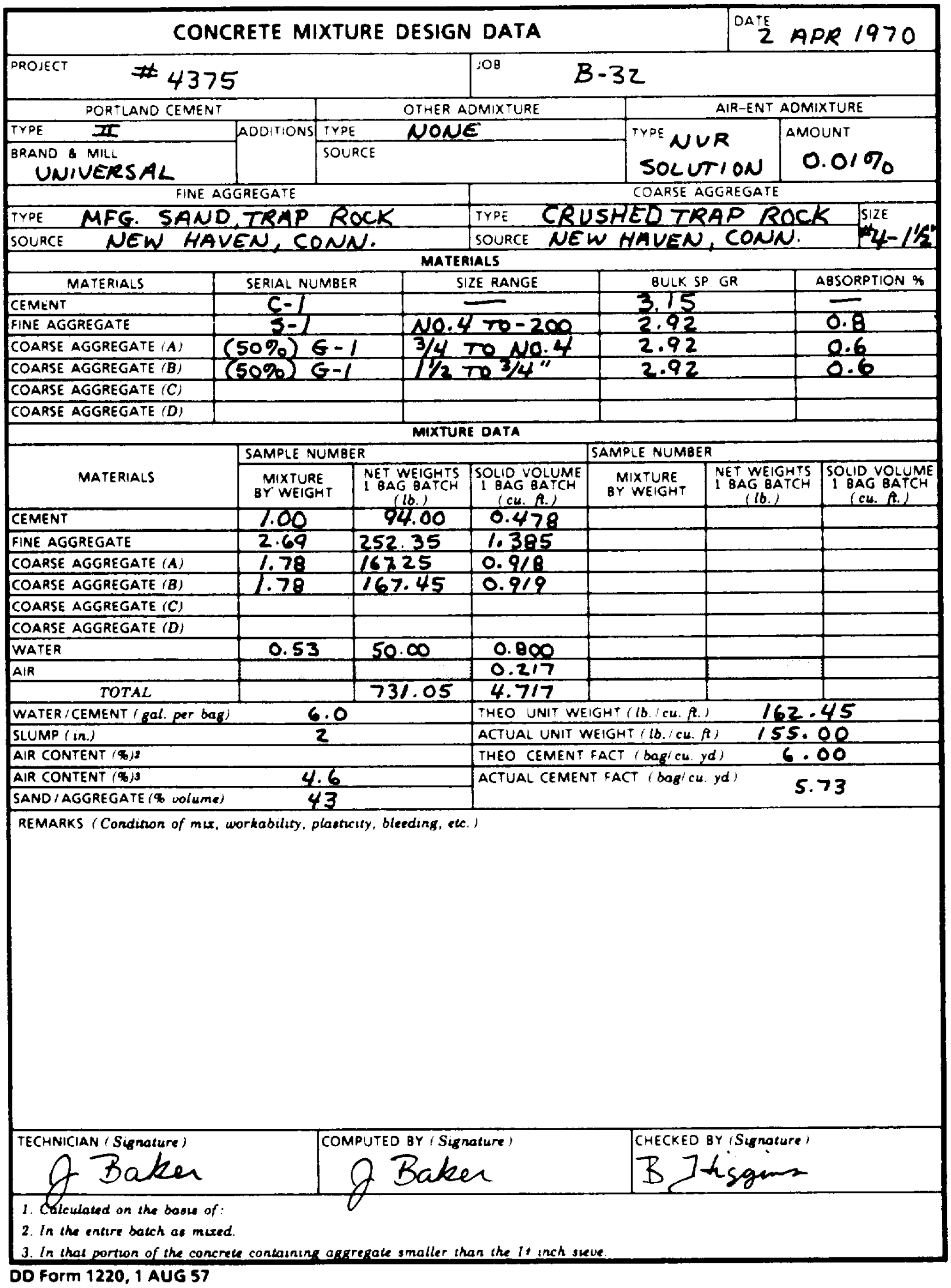
Surface moisture of both FAs and CAs.

If time permits, soak the aggregates for several hours prior to use, allow them to dry to a saturated, surface-dry condition, and place them in covered containers to maintain this condition until use. Correction for surface moisture then will not be required.

If the CA to be used contains particles larger than 3/4 inch, it should be separated into two or more fractions. For example, it may be divided into the following sizes: No. 4 to 3/4 inch, 3/4 to 1-1/2 inches, 1-1/2 to 3 inches. The two or more fractions are then recombined in the proportions

Table 4-12. Example of calculations for aggregate blending

Sieve size	Separate sizes individual % retained		50%	50%	Combined grading 50-50			Desired grading	
	No. 4 to ¾ in	¾ to 1½ in	No. 4 to ¾ in	¾ to 1½ in	Individual % retained	Cumulative %		Individual % retained	Cumulative % passed
						Retained	Passed		
1½	——	——	——	——	——	——	100.0	0.0	100.0
1 in	——	60.0	——	30.0	30.0	30.0	70.0	28.4	71.6
¾ in	——	36.0	——	18.0	18.0	48.0	52 0	16.9	54.7
½ in	34.0	4.0	17.0	2.0	19.0	67.0	33.0	20.1	34.6
⅜ in	24.0	——	12.0	——	12.0	79.0	21.0	13.2	21.4
No. 4	40.0	——	20.0	——	20.0	99.0	1.0	21.4	0.0
Pan	2.0	——	1.0	——	1.0	100.0	——	——	——
Total	100.0	100.0	50.0	50.0	100.0	——	——	100.0	——



CONCRETE MIXTURE DESIGN DATA

DATE 2 APR 1970

PROJECT #4375

JOB B-32

PORTLAND CEMENT		OTHER ADMIXTURE	AIR-ENT ADMIXTURE	
TYPE II	ADDITIONS	TYPE NONE	TYPE NVR SOLUTION	AMOUNT 0.01%
BRAND & MILL UNIVERSAL		SOURCE		

FINE AGGREGATE	COARSE AGGREGATE	
TYPE MFG. SAND, TRAP ROCK	TYPE CRUSHED TRAP ROCK	SIZE #4-1½"
SOURCE NEW HAVEN, CONN.	SOURCE NEW HAVEN, CONN.	

MATERIALS

MATERIALS	SERIAL NUMBER	SIZE RANGE	BULK SP. GR	ABSORPTION %
CEMENT	C-1	—	3.15	—
FINE AGGREGATE	S-1	NO. 4 TO -200	2.92	0.8
COARSE AGGREGATE (A)	(50%) G-1	3/4 TO NO. 4	2.92	0.6
COARSE AGGREGATE (B)	(50%) G-1	1½ TO 3/4"	2.92	0.6
COARSE AGGREGATE (C)				
COARSE AGGREGATE (D)				

MIXTURE DATA

MATERIALS	SAMPLE NUMBER			SAMPLE NUMBER		
	MIXTURE BY WEIGHT	NET WEIGHTS 1 BAG BATCH (lb.)	SOLID VOLUME 1 BAG BATCH (cu. ft.)	MIXTURE BY WEIGHT	NET WEIGHTS 1 BAG BATCH (lb.)	SOLID VOLUME 1 BAG BATCH (cu. ft.)
CEMENT	1.00	94.00	0.478			
FINE AGGREGATE	2.69	252.35	1.385			
COARSE AGGREGATE (A)	1.78	16725	0.918			
COARSE AGGREGATE (B)	1.78	167.45	0.919			
COARSE AGGREGATE (C)						
COARSE AGGREGATE (D)						
WATER	0.53	50.00	0.800			
AIR			0.217			
TOTAL		731.05	4.717			

WATER/CEMENT (*gal. per bag*)	6.0	THEO. UNIT WEIGHT (*lb./cu. ft.*)	162.45
SLUMP (*in.*)	2	ACTUAL UNIT WEIGHT (*lb./cu. ft*)	155.00
AIR CONTENT (%)2		THEO. CEMENT FACT (*bag/cu. yd*)	6.00
AIR CONTENT (%)3	4.6	ACTUAL CEMENT FACT (*bag/cu. yd*)	5.73
SAND/AGGREGATE (*% volume*)	43		

REMARKS (*Condition of mix, workability, plasticity, bleeding, etc.*)

TECHNICIAN (*Signature*)	COMPUTED BY (*Signature*)	CHECKED BY (*Signature*)
J Baker	J Baker	B J Higgins

1. Calculated on the basis of:

2. In the entire batch as mixed.

3. In that portion of the concrete containing aggregate smaller than the 1½ inch sieve.

DD Form 1220, 1 AUG 57

Figure 4-4. Data sheet, concrete mixture design data

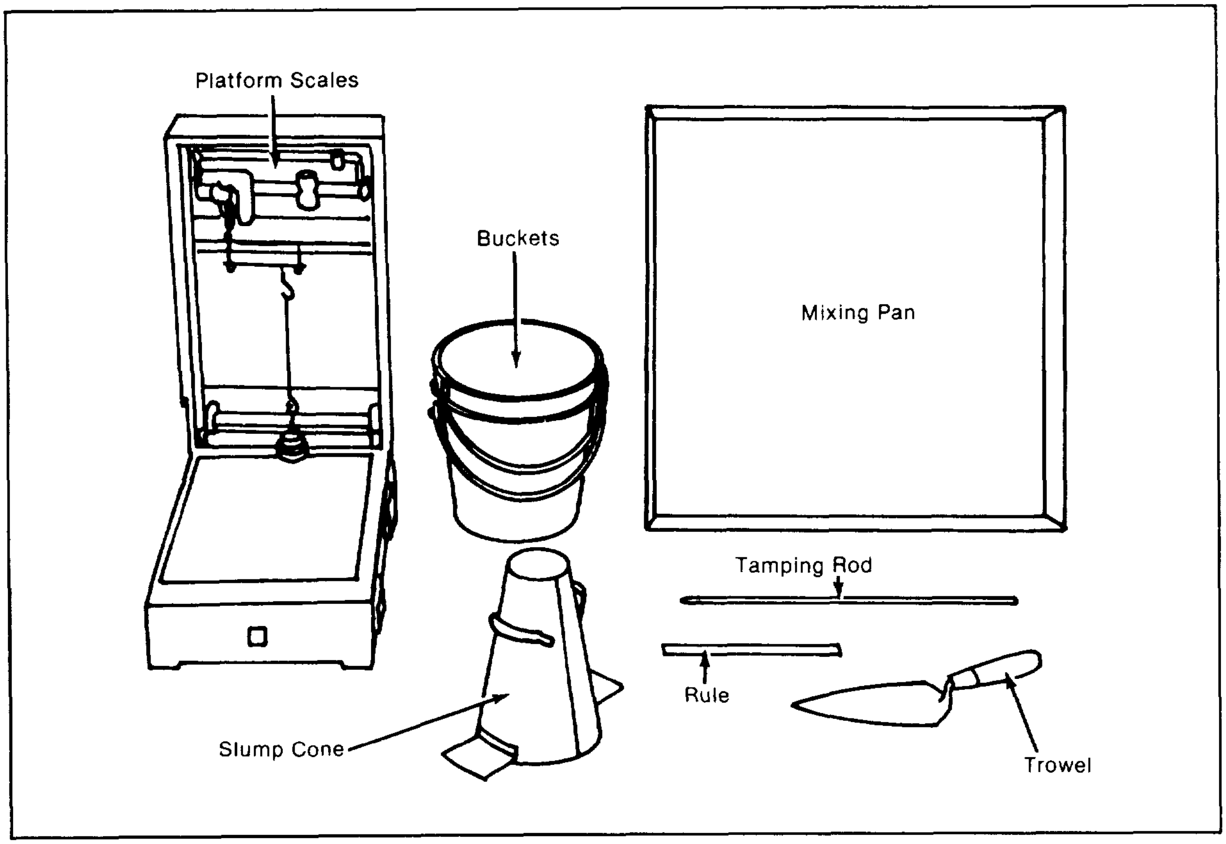


Figure 4-5. Apparatus for trial batch method of mixing portland cement concrete

necessary to produce the best possible gradation. Desirable CA gradings are given in Table 4-2, page 4-11. For example, assume that the CA to be used contains material from the No. 4 sieve to 1-1/2 inches in size. It should be divided into two size ranges: No. 4 to 3/4 inch and 3/4 to 1-1/2 inches. The two fractions are then recombined in proportions which most nearly give the grading called for in Table 4-12, page 4-34. The results of the preliminary tests are entered in the appropriate spaces on DD Form 1220 (Concrete Mixture Design Data), Figure 4-4, page 4-35.

Apparatus

The test procedures require the following equipment:

Bucket, 14 quart

Slump-cone molds

Electric oven

Concrete-mixing pan, 24 x 31 x 6 inches

Steel tamping rod, 3/8-inch diameter by 24-inches long

Steel rule

Scales, 300-pound capacity

Kitchen scoop

Square point shovel

Brick trowels

Wood float

Graduates, 25 milliliters.

Trial Batch Procedure

The mix proportions are to be determined for concrete which will be used in a retaining wall that will be exposed to fresh water in a severe climate. A compressive strength of 3,000 psi at 28 days is required. The minimum thickness of the wall is 8 inches, and 2 inches of concrete must cover the reinforcement. All trial mix data will be entered in the appropriate blanks on the trial mix data worksheet, Figure 4-6, page 4-40.

- Select the W/C ratio, which must satisfy the requirements of strength, durability, and watertightness. Tables 4-6, page 4-25, and 4-7, page 4-26, list suggested W/C ratios. Since the W/C ratio is the most important factor, it must be held constant with the other variables used to control the other requirements. It is extremely important to remember that a change in the W/C ratio will alter the characteristics of the hardened concrete.

Water-cement Ratio. Item 4 (under type of structures) Table 4-13, page 4-38, indicates that a maximum W/C ratio of 5.5 gallons per sack will satisfy the exposure requirements. Using Type IA (air entrained) portland cement and a compressive strength of 3,450 psi (3,000 + 15 percent), Figure 4-7 indicates that a maximum W/C ratio of approximately 5.75 gallons per sack will satisfy the strength requirements. To meet both specifications, a W/C ratio of 5.5 gallons per sack is selected. Since the maximum size of CA must not exceed one-fifth the minimum thickness of the wall, nor three-fourths of the clear space between reinforcement and the surfaces, the maximum size of CA is chosen as 1-1/2 inches. Because of the severe exposure conditions, the concrete should contain entrained air. From Table 4-14, page 4-39, the recommended air content is 5 ± 1 percent. Assuming that the concrete will be consolidated by vibration, Table 4-15, page 4-39, indicates a recommended slump of from 2 to 4 inches. The trial batch proportions are now determined.

One-tenth of a sack batch means 9.4 pounds of cement is recorded along with 1/10 of W/C ratio (5.5 gallons x 8.33 pounds per gallon = 45.82 = pounds of water divided by 10 = 4.58 pounds of water for a 1/10 sack batch). This is recorded as the initial weight of water.

- Fine and coarse aggregates. Representative samples of FAs and CAs are selected and weighed. This is recorded in column (2) of Figure 4-4. All of the measured quantities of cement, water, and air-entraining admixture are used. FAs and CAs are then added until a workable mixture having the proper slump is produced. The unit weight is determined, specimens are made, and the air percent is checked. The weight of material actually used is recorded in column (4), Figure 4-6. The weights for a 1-bag batch per cubic yard are calculated and recorded in columns (5) and (6) respectively. The cement factor (CF) in bags per cubic yard is calculated and recorded as indicated in Figure 4-6, page 4-40.

$$\text{CF} = \frac{\text{volume (27 cubic feet)}}{\text{(8.68 yield)}}$$

The CF, multiplied by each amount entered in column 5 (weight for 1-bag sack), equals the amount of each component to produce a cubic yard of concrete. Several trial batches may be designed to produce the best and most economical mix.

- Slump test. The next step is to determine the workability or consistency desired. Consistency is measured by the slump test and is expressed in inches of slump. Slump (in inches) is inversely proportional to the stiffness of the mix. See Table 4-8, page 4-27, for recommended values of slump for different types of construction. In the absence of any other criteria, concrete should be placed as stiff as possible, yet maintain a homogeneous voidless mass. Revise the mix by adding aggregates, as needed, until satisfactory proportions are obtained. Weigh the remaining aggregates and subtract this weight

Table 4-13. ACI recommended maximum permissible water-cement ratios for different types of structures and degrees of exposure

	Exposure conditions*					
Type of structures	**Severe wide range in temperature or frequent alternations of freezing and thawing (air-entrained concrete only) (gallons/sack)**			**Mild temperature rarely below freezing, or rainy, or arid (gallons/sack)**		
	In air	**At water line or within range of fluctuating water level or spray**		**In air**	**At water line or within range of fluctuating water level or spray**	
		In fresh water	**In sea water or in contact with sulfates****		**In fresh water**	**In sea water or in contact with sulfates****
Thin sections such as reinforced piles and pipe	5.5	5	4.5	6	5.5	4.5
Bridge decks	5	5	4.5	5.5	5.5	5
Thin sections such as railings, curbs, sills, ledges, ornamental or architectural concrete, and all sections with less than 1-in concrete cover over reinforcement	5.5	—	—	6	5.5	—
Moderate sections, such as retaining walls, abutments, piers, girders, beams	6	5.5	5	***	6	5
Exterior portions of heavy (mass) sections	6.5	5.5	5	***	6	5
Concrete deposited by tremie under water	—	5	5	—	5	5
Concrete slabs laid on the ground	6	—	—	***	—	—
Pavements	5.5	—	—	6	—	—
Concrete protected from the weather, interiors of buildings, concrete below ground	***	—	—	***	—	—
Concrete which will later be protected by enclosure or backfill but which may be exposed to freezing and thawing for several years before such protection is offered	6	—	—	**	—	—

This table was adapted from Recommended Practice for Selecting Proportions for Concrete (ACI 613-54).

*Air-entrained concrete should be used under all conditions involving severe exposure and may be used under mild exposure conditions to improve workability of the mixture.

**Soil or groundwater containing sulfate concentrations of more than 0.2 percent. For moderate sulfate resistance, the tricalcium aluminate content of the cement should be limited to 8 percent, and for high-sulfate resistance to 5 percent. At equal cement contents, air-entrained concrete is significantly more resistant to sulfate attack than non air-entrained concrete.

***Water-cement ratio should be selected on basis of strength and workability requirements, but minimum cement content should not be less than 470 lb per cubic yard.

Table 4-14. Approximate mixing water requirements for different slumps and maximum sizes of aggregates

Maximum size of aggregate, inches	Air-entrained concrete				Nonair-entrained concrete			
	Recommended average total air content, percent*	Slump, inches			Approximate amount of entrapped air, percent*	Slump, inches		
		1 to 2	3 to 4	5 to 6		1 to 2	3 to 4	5 to 6
		Water, gal per cu. yd. of concrete**				Water, gal per cu yd of concrete**		
⅜	7.5	37	41	43	3.0	42	46	49
½	7.5	36	39	41	2.5	40	44	46
¾	6.0	33	36	38	2.0	37	41	43
1	6.0	31	34	36	1.5	36	39	41
1½	5.0	29	32	34	1.0	33	36	38
2	5.0	27	30	32	0.5	31	34	36
3	4.0	25	28	30	0.3	29	32	34
6	3.0	22	24	26	0.2	25	28	30

This table was adapted from Recommended Practice for Selecting Proportions for Concrete (ACI 613-54)

*Plus or minus 1 percent.

**These quantities of mixing water are for use in computing cement factors for trial batches. They are maximums for reasonably well-shaped angular coarse aggregates graded within limits of accepted specifications.

*Table 4-15. Recommended slumps for various types of construction**

Type of construction	Slump, inches	
	Maximum	Minimum
Reinforced foundation walls and footings	6	3
Unreinforced footings, caissons, and substructure walls	4	1
Reinforced slabs, beams, and walls	6	3
Building columns	6	4
Pavements	3	1
Heavy mass construction	3	1
Bridge decks	4	3
Sidewalk, driveway, and slabs on ground	6	3

*When high-frequency vibrators are used, the values may be decreased approximately one-third, but in no case should the slump exceed 6 inches.

CONCRETE TRIAL MIX DATA

1. **PROJECT NO:** LL-50
2. **STRUCTURE:** RETAINING WALL
3. **EXPOSURE CONDITION:**
   Severe or Moderate XX **Mild** ______
   In Air ______
   In Fresh Water X
   In Sea Water
4. **TYPE OF STRUCTURE (A-1):** D
   MAX W/C FOR EXPOSURE: 5.5 **gal/sack**
   MAX W/C FOR WATERTIGHTNESS 5.5 **gal/sack**
5. **TYPE OF CEMENT** 1A
6. **FINENESS MODULUS OF SAND** 2.75
7. **SPECIFIC GRAVITY:**
   Sand 2.60
   Gravel 2.65
8. **MAXIMUM SIZE AGGREGATE:** 1½"
9. **AIR CONTENT** 5 **% ± 1%**
10. **DESIRED SLUMP RANGE**
    MAX. 4 **in** **MIN.** 2 **in**
11. **STRENGTH REQUIREMENT:** 3.45 **psi**
    W/C FOR STRENGTH: 5.75 **gal/bag**
    USE W/C 5.5 **gal/sack**

DATA FOR TRIAL BATCH

(Saturated, surface-dry aggregates)

(1) Material	(2) Initial Wt (lb)	(3) Final Wt (lb)	(4) Wt Used (lb)	(5) Wt for 1-bag Batch	(6) Wt per cu yd	(7) Remarks
Cement	9.4	0	9.4	94.0 lbs	292.34	Cement Factor = 3.11 bag/cu yd
Water	4.58	0	4.58	45.8 lbs	142.44	47.74 gal/cu yd
Fine Agg	66.0	29.0	37.0	370 lbs	1150.7	88.9% of total aggregate
Coarse Agg	89.0	15.0	74.0	740 lbs	2301.4	
Air-Entraining Admixture	.30 oz		TOTAL (T) = 1249.3		3886.9	

Measured Slump: 3 **in** **Air Content** 5.4 **%** **Workability:** Good

Wt Container concrete (lb)	**42.60**
Wt Container (lb)	**6.60**
Wt Concrete = A (lb)	**36.00**
Vol Container = B (cu ft)	**.25**

Unit Wt of concrete = $w = \frac{A}{B}$

$\frac{A}{B} = \frac{36.0}{.25}$ = 144.0 **lb/cu ft**

Yield = $\frac{T}{W} = \frac{1249.8}{144}$ = 8.68 **cu ft/bag**

Cement Factor = $\frac{\text{(Capactity)}}{\text{(Yield)}} = \frac{27}{8.68}$ = 3.11

Figure 4-6. Worksheet for concrete trial mix data

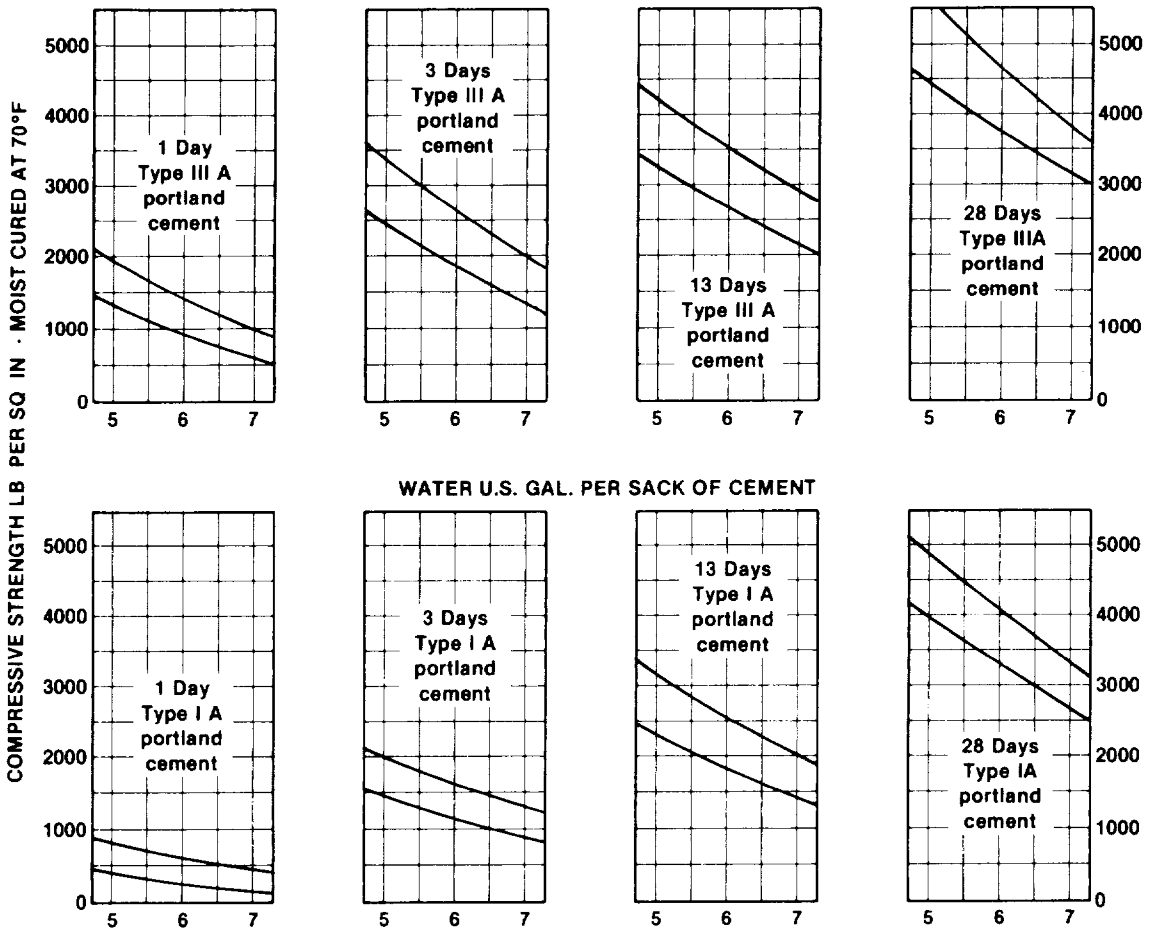


Figure 4-7. Age-compressive strength relationships for Types I and III air-entrained portland cement

from the original weight to obtain the amounts used for each aggregate. Record all weights of materials on the trial mixture data form.

- Air-entraining admixture. If an air-entraining agent is to be used in the concrete, the proper amount of this material is obtained and placed in a suitable container. The amount of admixture required varies considerably, depending upon the type used. If neutralized vinsol resin is used, approximately 0.05 percent by weight of the cement will be required. This is 20 milliliters per sack of cement.

- Trial batch mixing. Trial batches may be as large as the size of the mixer, but smaller quantities are more convenient (1/10-sack batches may be used). If the W/C ratio is 5 gallons per sack, then 0.5 gallons of water and 1/10 sack of cement (9.4 pounds) would be used. The cement and water are mixed to form a paste. Sand and gravel are then mixed with the paste until the desired consistency is obtained. Both the FA and CA should be in a saturated surface-dry condition. The weights of the sand and gravel used in the batch are obtained by weighing each container filled with aggregate before the trial batch is run and by weighing the container with the remainder of the aggregate after the run. The difference in weight is the weight of the aggregate used in the trial batch. Consistency of the trial batch is tested with the slump test. When the weights for the sand, gravel, and water required for 1/10 sack of cement have been determined, the weight of each ingredient required for one sack is obtained by multiplying by 10.

- Adjustments. Usually a single trial batch does not supply enough information, since it is almost certain to be either oversanded or too harsh, or it misses the desired slump or air content. After calculating the results of the first batch, make additional batches using different weights of material obtained by adjusting the quantities used in the initial batch. Mix each batch according to the procedure previously described. Make adjustments in the mixture until the best possible mixture is obtained.

Calculations

Determine the yield after determining the unit weight. The ingredients used for the one-sack batch must be converted to pounds and totalled. The total weight of a one-sack batch is then divided by the unit weight of the fresh concrete, to equal the volume in cubic feet that a one-sack batch will produce.

To determine the quantity of each ingredient necessary to batch a mixer or to estimate the total amount of each material required, the CF must be established. The CF is obtained by dividing the capacity of the mixer or job (volume) in cubic feet by the yield in cubic feet. For example, to determine the amount of each ingredient required to batch a 16S mixer, the CF equals the capacity of the mixer divided by the yield. Capacity of the 16S mixer is 16 cubic feet with no overload. Yield is assumed to be 4 cubic feet.

$$\text{CF} = \frac{\text{volume (mixer capacity)}}{\text{yield (in cubic feet)}}$$

$$\text{CF} = \frac{16}{4} = 4$$

With this method, total requirements for cement, water, sand, and gravel for any project can be obtained by multiplying the appropriate cement factor by one-sack batch quantities. The CF for a cubic yard batch of concrete would be as follows.

$$\text{CF} = \frac{\text{volume (27 cubic feet per cubic yard)}}{\text{yield (in cubic feet)}}$$

ABSOLUTE VOLUME METHOD OF MIXTURE PROPORTIONS

Concrete mixtures may be proportioned by using absolute volumes. This method is detailed in the American Concrete Institute (ACI) Report, Recommended Practice for Selecting Proportions for Concrete (ACI 613-54).

Preparation

The W/C ratio, slump, air content, and maximum size of aggregate are selected. In addition, the water requirement is estimated from Table 4-9, page 4-41. Certain additional items must be known before calculations can be made. These are the specific gravities of FAs and CAs and the dry-rodded unit weight of CA. If the maximum size of aggregate is known, the volume of dry-rodded CA per cubic yard can be estimated from Table 4-11, page 4-33. Then the quantities per cubic yard of water, cement, CA, and air can be calculated. The sum of the absolute volumes of these materials in cubic feet is then subtracted from 27 to give the specific volume of FA.

The flexibility of the absolute volume method lends itself to the many distinct characteristics of each project. The cubic yard is a standard measure for most concrete construction projects. The purpose of the absolute volume method of mix proportioning is to determine, as closely as possible, the amount of cement, water, sand, gravel, and air required to completely fill a one cubic yard container with quality concrete.

Example Using Absolute Volume Method

The mix proportions are to be determined for the following conditions:

Maximum W/C ratio	5.5 gallons per sack
Maximum size of aggregate	3/4 inch
Air content	6 ± 1 percent
Slump	2 to 3 inches
Fineness modulus of FA	2.75
Specific gravity of portland cement	3.15
Specific gravity of FA	2.66
Specific gravity of CA	2.61
Dry-rodded unit weight of CA	104 pounds per cubic foot

The water requirement is estimated at about 34 gallons per cubic yard (from Table 4-9, page 4-28). Since the maximum W/C ratio is 5.5 gallons per sack, the cement factor must be at least 6.2 sacks per cubic yard. The volume of dry-rodded CA is estimated to be 16.9 cubic feet per cubic yard from Table 4-11, page 4-33. Thus the weight of CA is 16.9 x 104 = 1,758 pounds. The absolute volumes of these quantities of materials is calculated using the following formula.

$$\text{absolute volume} = \frac{\text{weight of material}}{\text{specific gravity x unit weight of water}}$$

The unit weight of water is 62.4 pounds per cubic foot, and water weighs approximately 8.33 pounds per gallon. Thus the absolute volumes are:

Cement

$$\frac{6.2 \times 94}{3.15 \times 62.4} = 2.97 \text{ cubic feet}$$

Water

$$\frac{34 \times 8.33}{1 \times 62.4} = 4.54 \text{ cubic feet}$$

Coarse aggregate

$$\frac{1{,}758}{2.61 \times 62.4} = 10.80 \text{ cubic feet}$$

Air

0.06 x 27 = 1.62 cubic feet

Total Absolute Volumes = 19.93 cubic feet

The absolute volume of the fine aggregate is:

27.00 - 19.93 + 7.07 cubic feet

and its weight is:

7.07 x 2.66 x 62.4 = 1,174 pounds

These estimated quantities should be used for the first trial batch. Subsequent batches should be adjusted by maintaining the W/C ratio constant and achieving the desired slump and air content.

TESTS OF FRESH CONCRETE

After the concrete is first mixed, several tests are used to determine its quality.

Slump Test

When the mixture appears to have reached the desired consistency, a slump test should be made. This method of testing covers the procedure to be used in the laboratory and in the field for determining consistency of concrete. It is not an exact method, but it gives sufficiently accurate results. This test is not applicable when there is a considerable amount of aggregate over 2 inches in the concrete.

- Preparation. Samples of concrete for test specimens will be taken at the mixer or, in the case of ready-mixed concrete, from the transportation vehicle during discharge. The sample of concrete from which test specimens are made will be representative of the entire batch. Two or more samples will be obtained by repeatedly passing a scoop or pail through the discharging stream of concrete from the middle portion of the batch. The samples thus obtained will be transported to the testing site. To counteract segregation, the concrete will be mixed with a shovel until the concrete is uniform in appearance. The location in the work of the batch of concrete thus sampled will be noted for future reference. In the case of paving concrete, samples may be taken from the batch immediately after depositing on the subgrade. At least five samples will be taken from different portions of the pile, and these samples will be thoroughly mixed to form the test specimen.

- Apparatus. The following apparatus is required and is available in the mobile laboratory.

A mold of No. 16 gage galvanized metal in the form of the lateral surface of the frustum of a cone with the base 8 inches in diameter, the top 4 inches in diameter, and 12 inches in height. The base and the top are open and parallel to each other and at right angles to the axis of the cone (Figure 4-8).

Tamping rod 5/8 inch in diameter and 24 inches long having one end rounded.

- Procedure. The mold will be dampened and placed on a flat, moist nonabsorbent surface. From the sample of concrete obtained the mold will immediately be filled in three layers, each approximately one-third the volume of the mold. In placing each scoopful of concrete, the scoop will be moved around the top edge of the mold as the concrete slides from it to ensure symmetrical distribution of concrete within the mold.

Each layer will be rodded with 25 strokes of the rounded end of the 5/8-inch round rod. The strokes will be distributed uniformly over the cross section of the mold and will penetrate into the underlying layer. The bottom layer will be rodded throughout its depth. After the top layer has been rodded, the surface of the concrete will be struck off with a trowel so that the mold is exactly filled. The mold will be removed at a rate of 5 ± 2 seconds for the height of the mold from the concrete by raising it carefully

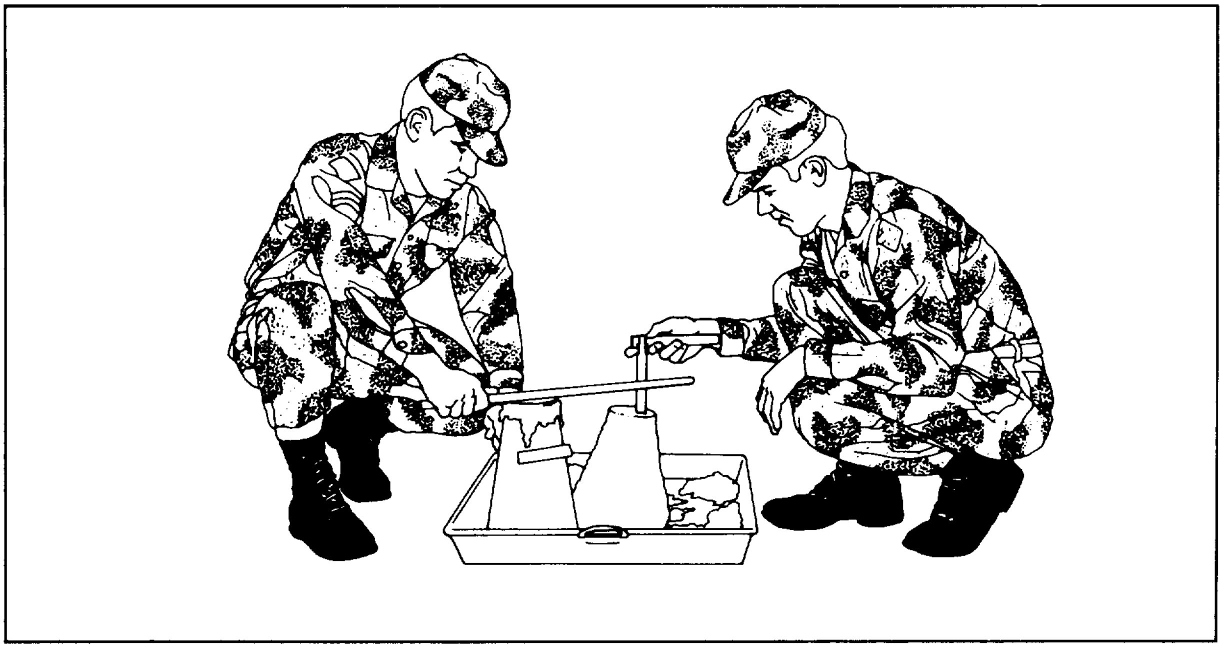


Figure 4-8. Measuring slump of fresh concrete

in a vertical direction. The slump will then be measured immediately by determining the difference between the height of the mold and the height at the vertical axis of the concrete.

- Slump. The consistency will be recorded in terms of inches of subsidence of the specimen during the test, which shall be known as the slump.

Slump = 12 inches − height of specimen after subsidence

- Supplementary test procedure. After the slump measurement is completed, the side of the specimen frustrum should be tapped gently 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.

Air-Content Testing

An air-entraining admixture is added to the concrete mix so that enough air will be entrained to improve workability and durability of the mixture, but not enough to substantially reduce the strength. Air-entrained cements may also be available for use in some military situations. The desired amount of air is generally from 4.0 to 7.5 percent of the total mix.

- Apparatus. The basic equipment for determining the percentage of entrained air is the pressure-type indicator. Typical test apparatus is shown in Figure 4-9, page 4-46. Before the air content of a concrete mixture can be determined, it is necessary that the entrained-air indicator is calibrated accurately, and that the correction factor for the aggregate contained in the concrete is determined.

- Procedure. The measuring bowl of the apparatus is filled with three equal layers of the concrete to be tested. Each layer is consolidated by 25 strokes of the rodding tool and evenly distributed over the surface of the layer. After each layer is rodded, the sides of the bowl are tapped sharply 10 to 15 times with the mallet until the holes left by the rodding are leveled and no large bubbles of air appear on the surface of the layer. While rodding the first layer, care should be taken not to strike the bottom of the measuring bowl too hard. The second and third layers should be rodded with only enough force for the rod to penetrate the surface of the previous layer. The bowl should be overfilled by the third layer. After rodding, the excess concrete should be removed by sliding the strike-off bar across the top flange with a sawing motion until the concrete is just level with the top of the bowl.

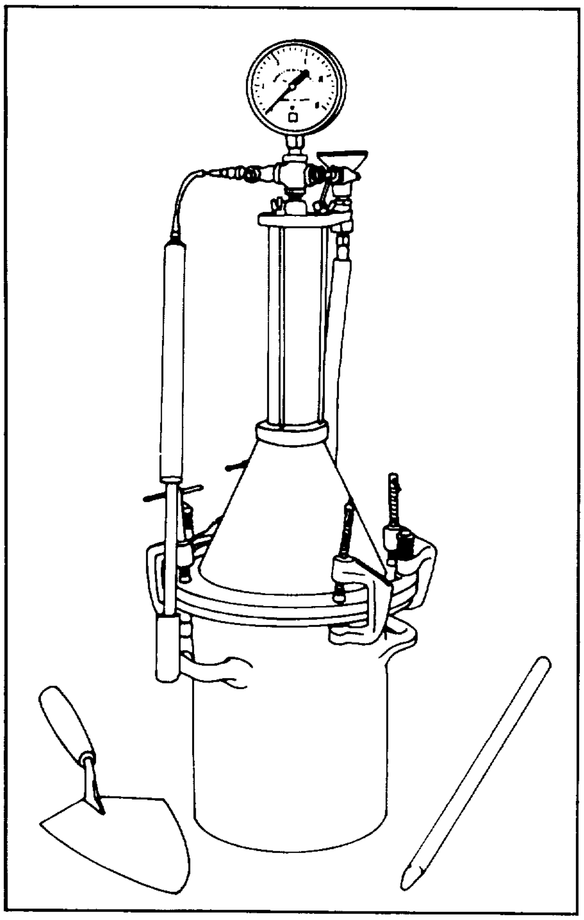


Figure 4-9. Apparatus for air-content test

The remainder of the test will depend on the type of meter furnished. The detailed procedure is outlined in the instruction book furnished with each meter.

The calculations for determining the entrained-air content are given in the instruction book furnished with the air meter.

Unit Weight

A cylindrical metal measure of either 1/10, 1/5, or 1/2-cubic foot capacity is filled with the fresh concrete and rodded in three layers as described for the air content test. After each layer is rodded, the sides of the container are tapped 10 to 15 times with the rod to remove any air pockets. The containers are rodded 25 times per layer. After filling and consolidating, the top surface is struck off with a heavy, flat metal cover plate to ensure that the measure is level full. All excess concrete is cleaned from the surface of the container and weighed. The tare weight of the clean, dry container is measured. The measure is calibrated if necessary to determine its volume by accurately filling it with water, weighing it, and dividing by the unit weight of water as shown in Table 4-13, page 4-38, to get the volume of container.

CALCULATIONS

The volume of concrete produced per sack of cement is called the yield. Yield for concrete made without an air-entraining agent is computed by assuming that the ingredients form a solid mass with no voids present. From the standpoint of economy the yield should be as high as possible, consistent with other desired properties of the concrete mixture. The yield is also useful in estimating the quantity of each material required to produce a given amount of concrete.

Correct the actual weights used in the trial mix for free surface moisture if present in the aggregates. Record corrected values (SSD weights). Multiply the weights of materials used by 100 and divide by the weight of the cement used to get the weights for the trial batch. Calculate the yield for a batch by dividing the total weight of a material by the unit weight of concrete. The CF is the number of bags of cement required for a cubic yard of concrete. It is calculated by dividing the yield per sack into 27 (cubic feet per cubic yard). Find the weights of all materials needed for a cubic yard by multiplying the weights for the trial batch by the CF. The FA expressed in percent by weight of total aggregate is found as follows:

$$\frac{\text{weight of fine aggregate}}{\text{weight of fine aggregate} + \text{weight of coarse aggregate}}\ (100)$$

The CF will'vary with the W/C ratio and the maximum size and particle shape of the CA used. The CF might be as high as 9.0 hundredweight (cwt) per cubic yard with pea-gravel concrete and as low as 5.0 or 5.5 cwt per cubic yard with angular, 1½-inch stone, both having a W/C ratio of 0.53. CFs for structural and paving concrete containing 1½-inch aggregate and a W/C ratio of 0.49 gallons per bag might vary between 5.5 and 7.0 cwt of cement per cubic yard.

Section V. FLEXURAL STRENGTH TEST (MODULUS OF RUPTURE)

DISCUSSION

The purpose of the test described in this section is to determine the flexural strength of concrete. A determination of the flexural strength is frequently necessary as a part of the design of concrete mixtures to check compliance with established specifications or to provide information necessary to the design of an engineering structure.

TEST BEAMS

Beam forms for casting test beams from fresh concrete are available in many sizes (usually 6 x 6 inches). Although equipment for obtaining sawed specimens may not be available, the test may be performed upon beams sawed from existing concrete structures for evaluation purposes.

Apparatus

The equipment furnished may include a tester designed to obtain an index of the flexural strength. Equipment which is necessary to perform this test, aside from the loading frame is:

Concrete beam tester, third-point loading

Wire brush

Wood float

Collapsible mold, for 6·x 6 x 21-inch beams

Mixing pan, 24 x 24 x 3 inches

Proving ring, complete with gage and ring clamps, 10,000-pound capacity

Tamping rod

Square-point shovel

Steel straightedge

Brick trowel.

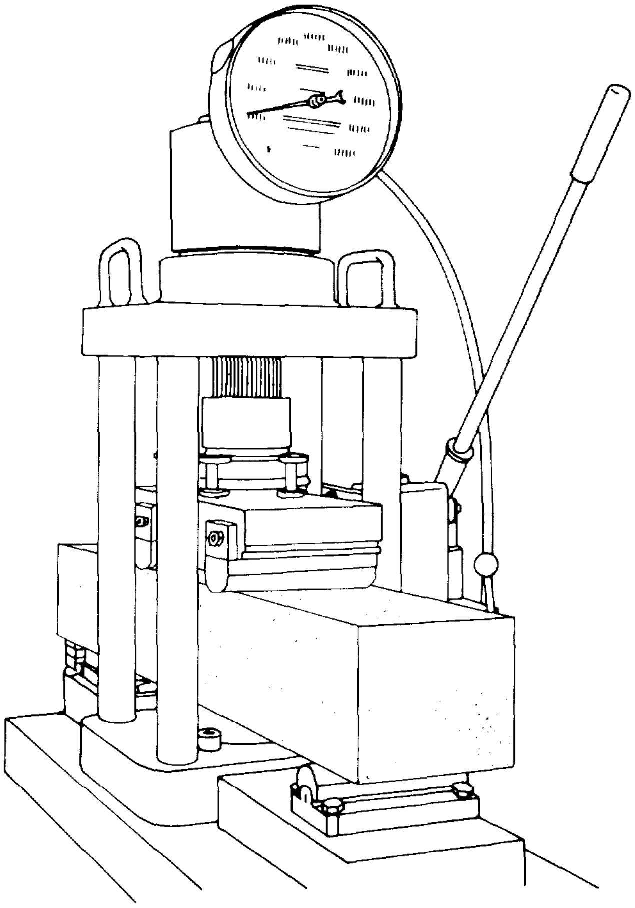


Figure 4-10. Flexural strength testing assembly for 6 x 6 x 21-inch concrete beam ready for testing

The flexural strength testing assembly is shown in Figure 4-10. An alternate testing assembly can be made from the loading frame and certain attachments provided with the CBR test set, the breaker (third-point loading), and the 10,000-pound capacity proving ring. This assembly is shown in Figure 4-11 (alternate testing assembly).

Procedure, Formation of Beams

The concrete from which the test beams are to be made is mixed in accordance with the procedure described, except that three 1/5-bag batches are required. This will produce approximately 3 cubic feet of concrete. After the concrete has been mixed, the slump test is conducted in accordance with the procedure described. The air content is determined by the procedure. If the slump and air content are within the desired limits, the concrete used to conduct these tests is returned to the mixing pan. The entire batch is remixed enough to produce a homogeneous mass. Flexure test beams are used

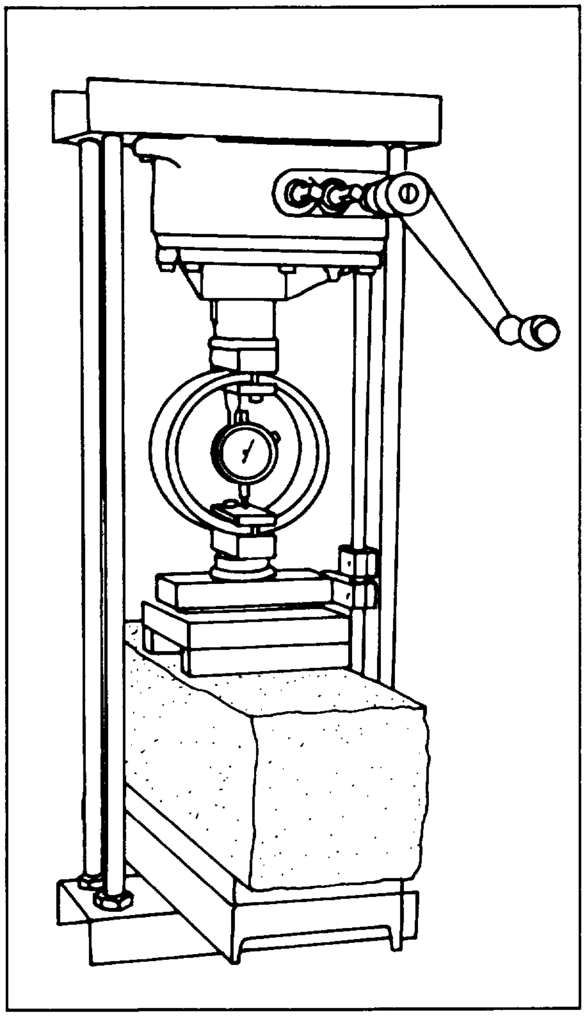


Figure 4-11. Alternate testing assembly using CBR test equipment

to check the quality of concrete being produced in the field, particularly in constructing pavements. When the concrete has cured to produce an adequate beam strength, traffic may use the pavement. Concrete is sampled from the production batches as required. Beams are molded, cured, and tested as described in the following procedures.

Forming beams. A standard 6 x 6-inch concrete-beam mold is assembled, and the inside of the mold is lightly oiled. The mold is filled with two layers of concrete, each approximately 3 inches in depth. Each layer is consolidated by rodding, using one stroke per 2 square inches evenly distributed over the surface of the layer. Tap the sides lightly to close the voids left by rodding. A light spading of the concrete along the sides of the mold with a trowel helps in the removal of surface voids. The top surface is struck off with a straightedge and finished with a wood or magnesium float.

- Number of specimens. At least five specimens are made for each mixture design being evaluated. Two of the beams are broken at 7 days. This will permit an early evaluation of the mix design. The remaining three beams are broken at 28 days. Additional specimens may be made for future testing. Specimens containing standard portland cement generally will develop about 80 percent of their 28-day strength during the first 7 days of curing.

- Curing. A suitable identifying label is placed on the finished surface of the specimens and the entire specimen, still in the mold, is covered with a double thickness of wet burlap. A range of 60° to 80°F (16° to 27°C) with 73.4° ± 3°F (23° ± 1.7°C) is the most satisfactory curing temperature for concrete. Therefore, an effort should be made to store the test specimens at approximately that temperature. The specimens are removed from the molds after 24 hours of curing. The beams are stored under a double thickness of wet burlap so that they will be kept continuously wet until they are tested.

Procedure, Testing of Beams

The loading device is assembled as shown in Figure 4-10. The test beam is turned so that the finished surface is to the side and centered in the loading assembly. The testing apparatus is operated until the loading blocks are brought into contact with the upper surface of the beam. Care should be taken to secure full contact between the loading (and supporting) surfaces and the beam. If the surface of the specimen is so rough that full contact is not secured, the

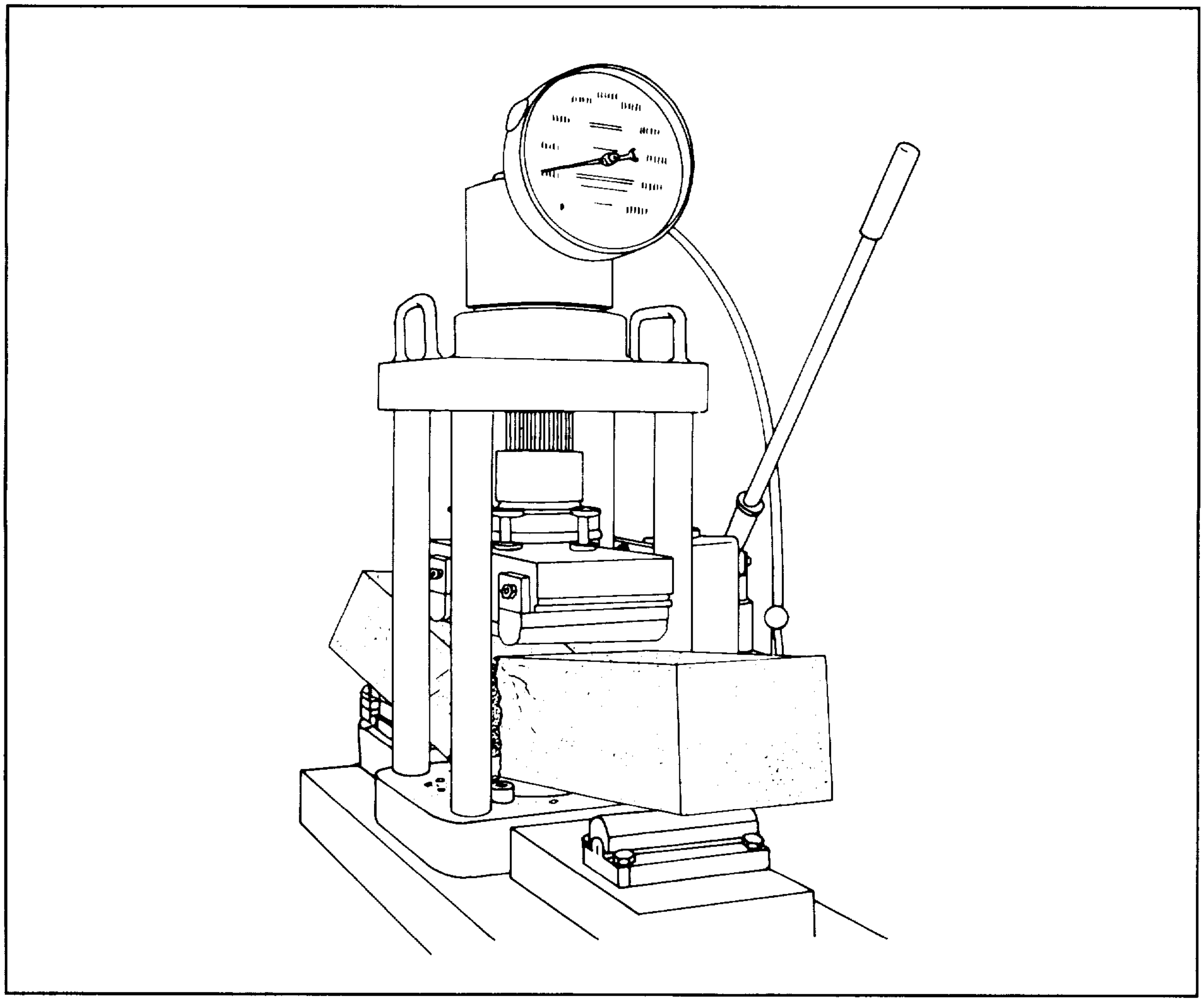


Figure 4-12. Beam broken by third-point loading

specimen should be ground to secure full contact. Load may be applied at a rate so that the increase in extreme fiber stress in the beam is between 125 and 175 pounds per square inch per minute. The extreme fiber stress corresponding to any load may be estimated from the equation to be given. Readings are obtained on the proving ring dial and are converted to corresponding total loads in pounds by application of the proving ring constant. Aside from controlling the rate of application of the load, the only reading necessary is that which corresponds to the maximum load applied to the test beam. After the specimen has been broken (Figure 4-12), the dimensions of the cross section at which failure occurred are obtained to the nearest 0.05 inch. These dimensions are taken to be the average of three widths and depths at the section of failure.

Calculations of Modulus of Rupture

If the specimen is broken within the middle third of the span length, the modulus of rupture is calculated as follows:

$$R = \frac{P\ (L)}{b\ (d)^2} \text{ or } R = \frac{3\ PA}{b\ (d)^2}$$

where:

R = modulus of rupture in pounds per square inch

P = maximum applied load pounds
L = span length in inches
b = average width of specimen in inches
d = average depth of specimen in inches.

- If the specimen is broken outside the middle third of the span length by not more than 5 percent of the span length, the modulus of rupture is calculated as follows:

$$R = \frac{3\ Pa}{b(d)^2}$$

where:

a = the distance, in inches, between the line of fracture and the nearest support, measure along the centerline of the tension surface of the beam

- If the specimen is broken outside the middle third of the span length by more than 5 percent of the span length, the results of the test will be discarded.

CONCLUSIONS

The report of the test for flexural strength should include the following:

Identification number

Average width to the nearest 0.05 inch

Average depth to the nearest 0.05 inch

Span length, in inches

Maximum applied load, in pounds

Modulus of rupture calculated to the nearest 5 pounds per square inch

Defects in specimen

Age of specimen

Values of the modulus of rupture vary widely, depending on the concrete tested. Specifications relative to concrete pavements frequently require a modulus of rupture in excess of 600 to 650 pounds per square inch (28-day curing, third-point loading). The flexural strength (modulus of rupture) are expected to be approximately 15 to 20 percent of the compressive strength for comparable conditions of age and curing.

An approximate relationship between modulus of rupture and compressive strength is as follows:

$$f_c' = \frac{(R)^2}{K}$$

f_c = compressive strength, pounds per square inch

R = modulus of rupture, pounds per square inch

K = constant, usually 100.

Section VI. COMPRESSIVE STRENGTH TEST

DISCUSSION

The compressive strength of hardened concrete, as measured by compression tests on standard forms of cylindrical specimens, is used in the design of structures.

Compressive strength tests are made on concrete trial mixtures to evaluate the performance of available materials and to establish mixture proportions that will give the required strength. Strength tests are used also to control the quality of concrete being manufactured in the field. Compressive strength is defined as the average of the strengths of all cylinders of the same age made from a sample taken from a single batch of concrete. At least two cylinders, or preferably three, are required to constitute a test. Hence four or six specimens are required if tests are to be made at 7 and 28 days.

CONCRETE CYLINDERS

The standard test specimen is a cylinder 6 inches in diameter by 12 inches long, capped with a suitable material to provide smooth, bearing surfaces on each end. Load is applied to the end surfaces through metal platens on the testing machine (cylinder breaker) causing compressive stress in the longitudinal direction of the cylinder.

Apparatus

The following equipment is required to mold and test concrete cylinders:

Concrete compression tester, 250,000-pound capacity

Mixing equipment for trial mixtures

Concrete cylinder molds, 6 x 12 inches

Mixing pan, 24 x 24 x 3 inches

Scoop

Tamping rod

Steel straightedge

Trowel

Square point shovel

Polyethylene sheeting

Burlap

Calipers, 6-inch opening

Steel scale, inches and hundredths.

Preparation of Cylinders

- Make the cylinders as near as possible to the place where they will be stored for the first 24 hours. Sufficient concrete (about 1 cubic foot) for the desired number of cylinders must be available in the trial mixture or field sample. Material must not be taken from the air content test, since this may be contaminated with excess water. Take the sample from a stationary mixer by passing a receptacle completely through the discharge stream of the mixer at about the middle of the batch, or by diverting the stream into a container. The contents of a paving mixer should be discharged into a pile and sample material taken by a shovel from at least five different portions of the pile. Take samples from revolving-drum truck mixers or agitators by repeatedly passing a receptacle through the entire discharged stream until sufficient concrete is collected into the pan (Figure 4-13). Thoroughly mix the sample before filling the cylinder molds.

- Clean and dry the molds. Give the metal molds, if these are used, a light coat of oil before filling. Fill the molds in three equal layers, rodding each layer 25 times, as described for the slump test. Tap the side of the mold with the rod after each layer to close holes that may be left by the rod. Strike off the top surface level with

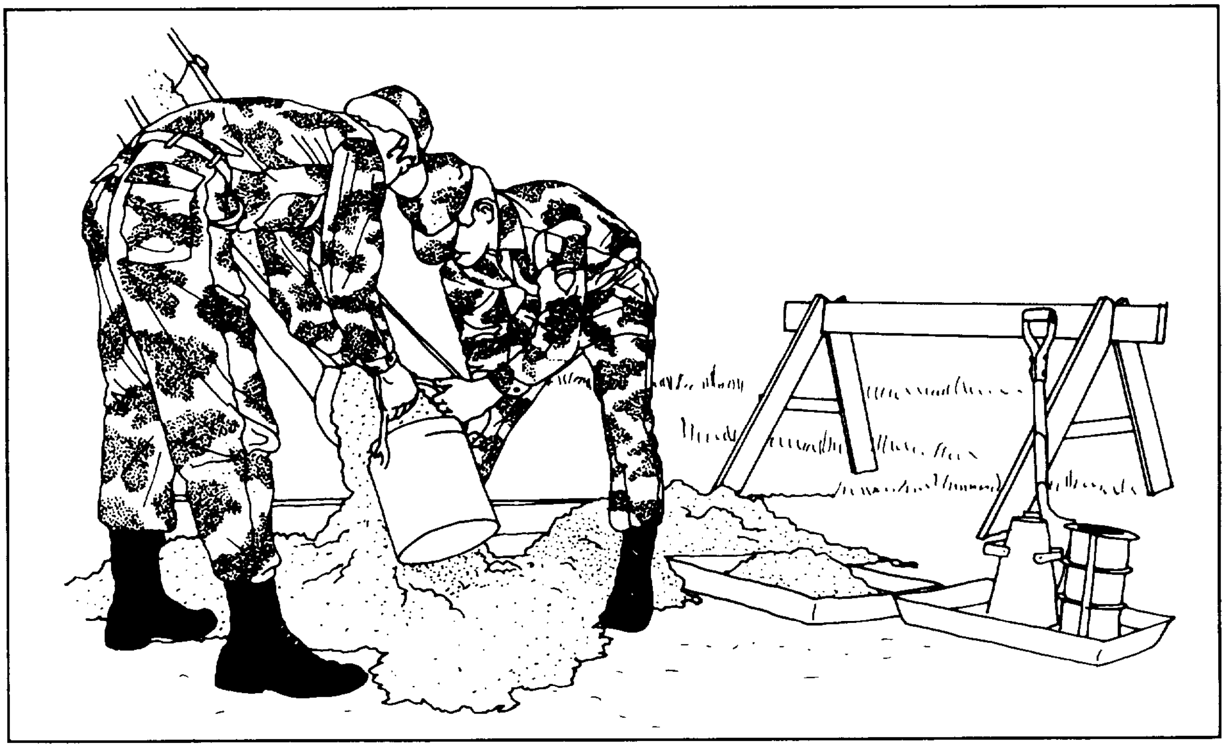


Figure 4-13. Sampling concrete from truck mixer

the top of the mold. Store the cylinders on a level, flat, hard surface free from vibration. In moving the cylinders of fresh concrete in cardboard molds, lift them on a flat trowel to prevent bulging of the lower end of the mold. Tag the specimens or molds for identification.

- Cover the specimens in the molds with polyethylene sheeting and place wet burlap over them (Figure 4-14, page 4-54). Maintain the temperature of the specimens between 60° to 80°F (16° to 27°C8) with 73.4° ± 3°F (23° ± 1.7°C) as the optimum curing temperature. Stoves or electric light bulbs may be used for heating. Cooling is accomplished, if needed, by evaporation of excess water from the wet burlap. A fan may be used to cause lower temperatures than will occur from natural evaporation. Keep the burlap wet' if it is allowed to dry, the concrete curing will be adversely affected.

At the end of 24 hours, remove the specimens from the molds and store them to continue curing until testing. Mark the cylinders with tags, wax, or permanent ink for identification. Individual handling of samples from different sources is as indicated.

Curing and Storage of Cylinders

- Specimens intended for checking the strength of laboratory trial mixtures or to serve as the basis for acceptance or quality control of field concrete are moist cured at 73.4° ± 3°F (23° ± 1.7°C) until the test. They are stored in moist rooms, damp sand or sawdust, or in lime water to maintain free water on all surfaces of the specimen at all times.

- Test specimens made in the field to determine when forms may be removed should

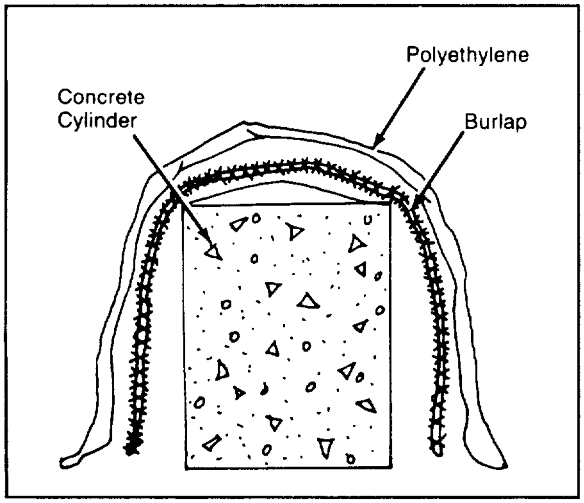


Figure 4-14. Curing of test cylinders

receive, 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. They are stored in or on the structure as near as possible to the point of use. They are tested 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 time of removing formwork from the structure.

- Specimens shipped from the field to the laboratory for testing are packed in a sturdy wooden box or other suitable container surrounded by wet sawdust or wet sand. Protection from freezing during storage or shipment must be provided. Moist curing is continued when the specimens are received in the laboratory.

Capping of Cylinders

The ends of compression-test specimens must be planed within 0.002 inch and within 0.5 degrees of being perpendicular to the axis of the cylinder.

- Specimens formed in strong metal molds having accurately flat base plates can be capped with neat cement at 2 to 4 hours after molding. A stiff paste of portland cement and water is made at the time the cylinder is molded so that the capping mixture will have shrunk before application. Any free water or laitance (layer of fine particles on the surface) is removed from the end of the specimen. The paste is applied to the top of the concrete and worked with a flat plate until it is smooth and level with the top of the mold.

- Hardened concrete specimens may be ground to place ends or capped with a material having greater compressive strength than the concrete. Prepared mixtures of sulfur and granular materials, special high-strength gypsum plasters, and neat high-early strength cement are satisfactory capping materials (ordinary low-strength plaster of paris, compressible rubber, or fibrous materials are not suitable for caps). These materials are applied in a plastic state and finished to the desired plane surface by applying glass or metal plates and squeezing out excess material to provide a cap that is as thin as possible. Sulfur caps may be applied in time to harden for at least 2 hours before testing. Plaster caps cannot be stored over 4 hours in the moist room. Neat cement caps must be aged 6 days or more in the moist room (2 days when Type II cement is used). During capping, moist, cured specimens should be protected against drying by covering them with wet burlap.

COMPRESSION TEST

The procedure for conducting the compression test is as follows:

Preparation

Prepare the testing machine (Figure 4-15) by cleaning the bearing plates and, if needed, cleaning and lubricating the spherical seat. Check the operation of the machine. Keep the specimens moist by covering them with wet burlap during the period between removal from the curing environment and testing.

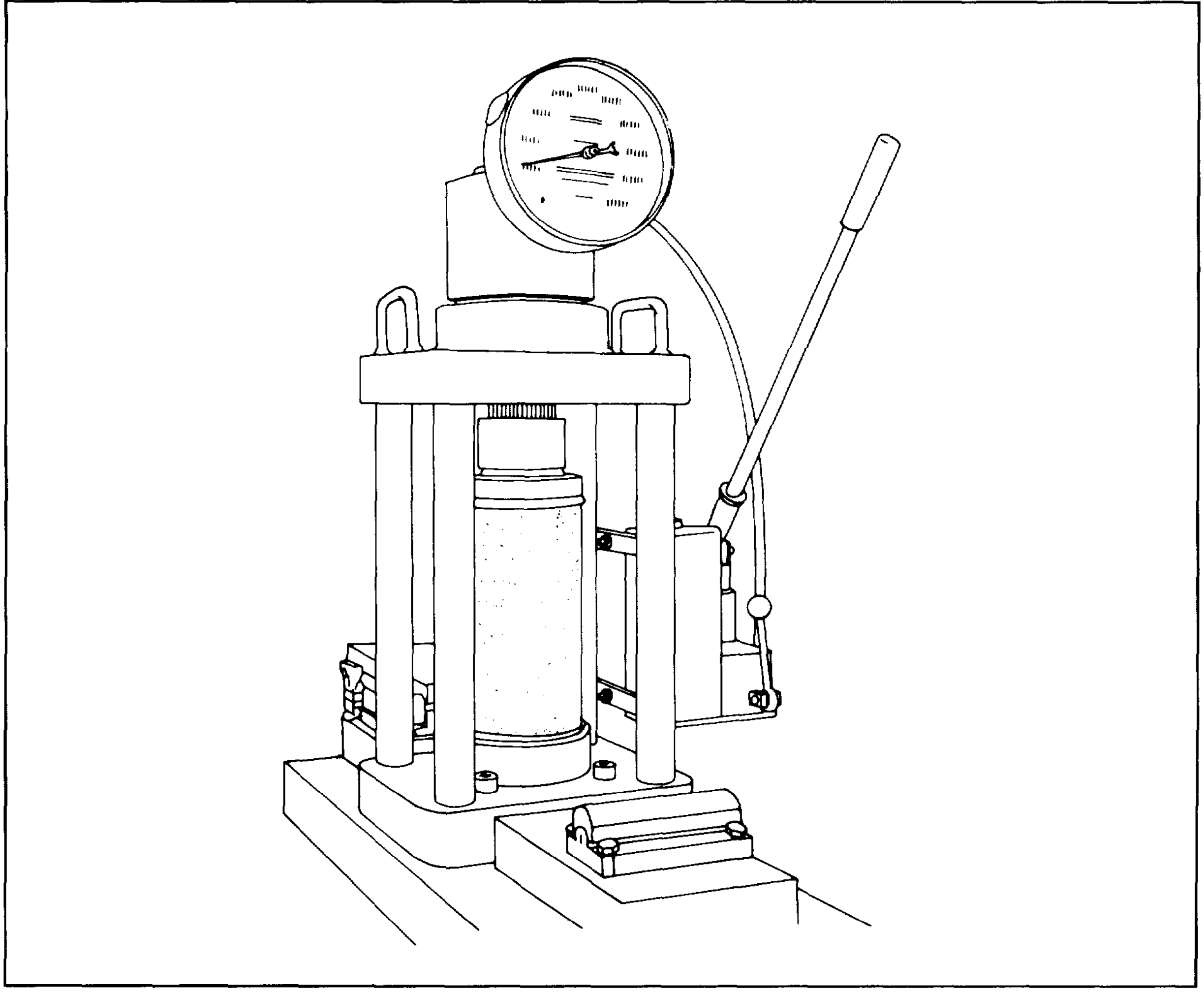


Figure 4-15. Compression testing machine with concrete test cylinder in place

Determine the diameter of the specimen to the nearest 0.01 inch by averaging two diameters measured at right angles to each other at midheight of the cylinder. Measure the length, including caps, to the nearest 0.1 inch. Record the dimensions on a prepared data sheet.

Procedure

Place the specimen on the lower bearing block, bring the upper block almost to contact, and align the axis of the specimen with the center of thrust of the spherical head. Carefully and slowly bring the spherical head into contact with the specimen, rotating the movable portion gently by hand so that a uniform seating is obtained. Apply the test load continuously and without shock at a rate of 20 to 50 pounds per square inch per second. Observe and record the maximum load during the test. Observe the type of fracture and record any unusual features. The normal cone-type fracture is shown in Figure 4-16, page 4-56. A load of 154,000 pounds exerted over a 28.3 square inch area determines the strength to be 5,400 pounds per square inch.

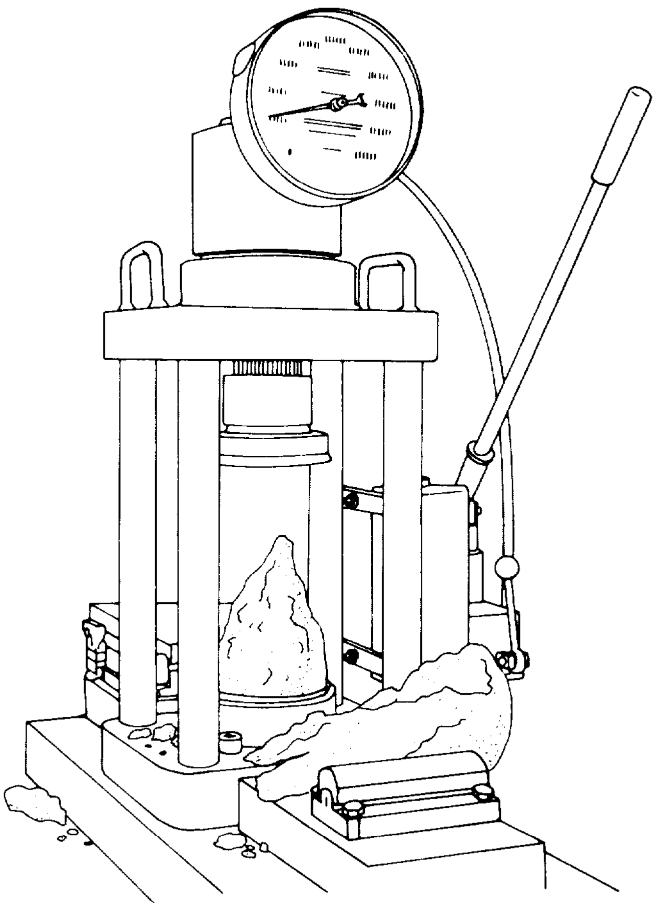


Figure 4-16. Normal fracture of concrete test cylinder in compression

Calculation

The compressive strength of the concrete is the maximum load divided by the cross-sectional area of the cylinder:

where:

$$f'_c = \frac{P}{A}$$

f'_c = compressive strength, pounds per square inch

P = maximum load in test, pounds

A = cross-sectional area, square inches.

Report the identification, diameter and length, maximum load, compressive strength, irregular fracture or defect in caps or specimen, and age of the specimen at the time of the test. Report the average compressive strength of all cylinders from the same concrete sample.

Chapter 5

Soil Stabilization

Section I. MECHANICS OF STABILIZATION

DISCUSSION

This chapter presents criteria and tests for improving the structural quality and workability of soils used for base courses, subbase courses, select materials, and subgrades for pavements. It is applicable to all elements responsible for Army, Air Force, or Navy pavement construction. 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 used also to provide control of dust.

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 the blending of soils to achieve a desired gradation or the mixing of commercially available additives that may alter the gradation, change the strength and durability, or act as a binder to cement the soil.

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 soil strength and durability.

USES OF STABILIZATION

Pavement design is based on the premise that specified levels of quality will be achieved for each soil layer in the pavement system. Each layer must resist shearing within the layer, avoid excessive elastic deflections that would result in fatigue cracking within the layer or in overlying layers, and prevent excessive permanent deformation through densifications. As the quality of a soil layer is increased, the ability of that layer to distribute the load over a greater area is generally increased enough to permit a reduction in the required thickness of the soil and surface layers.

Improve Quality

Stabilization commonly is used for better soil gradation, reduction of plasticity index or swelling potential, and increased durability and strength. Soils stabilized by additives often provide an all-weather working platform for construction operations. These types of soil quality improvement are referred to as soil modifications.

Reduce Thickness

The tensile strength and stiffness of a soil layer can be improved through the use of additives and 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, Chapter 3; TM 5-822-6/AFM 88-7, Chapter 1; TM 5-824-3/AFM 88-6, Chapter 3; and TM 5-825-2/AFM 88-6, Chapter 2. Before a stabilized layer can be used to reduce the required thickness in the design of a pavement system, the stabilized material must meet the durability requirements of various types of additive stabilization and the minimum strength requirements. Generally, as the percent of fines and the plasticity index (PI) increase pulverization becomes more difficult, and is harder to obtain uniform distribution of the stabilizing additive. For these types of soils, preprocessing or pretreatment with other additives may be necessary. For example, fine-grained soils may be pretreated with lime to aid in their pulverization, making the mixing of a bitumen or cement additive more successful.

METHODS OF STABILIZATION

The two general methods of stabilization are mechanical and additive. The effectiveness of stabilization depends upon the ability to obtain uniformity in blending the various materials. Mixing in a stationary or traveling plant is preferred. However, other means of mixing such as scarifiers, plows, disks, graders, and rotary mixers have been satisfactory.

The method of soil stabilization is determined by the amount of stabilizing required and the conditions encountered on the project. An accurate soil description and classification is essential to the selection of the correct materials and procedure. Table 5-1 lists the most suitable treatments for various soil types to stabilize these soils for different objectives.

Mechanical

Mechanical stabilization is accomplished by mixing or blending soils of two or more gradations to obtain a material meeting the required specification. The soil blending 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 required densities by conventional means.

Additive

Additive refers to a manufactured commercial product that, when added to the soil in the proper quantities, will improve the quality of the soil layer. This chapter is directed towards the use of portland cement, lime, lime-cement-fly ash, and bitumen, alone or in combination, as additives to stabilize soils. The selection and determination of the percentage of additives depend upon the soil classification and the degree of improvement in soil quality desired. Generally, smaller amounts of additives are required to alter soil properties, such as gradation, workability, and plasticity, than to improve the strength and durability sufficiently to permit a thickness reduction design. After the additive has been mixed with the soil, spreading and compaction are achieved by conventional means.

- Stabilization by Cementing Action. This requires the addition of chemical agents to the soil to produce the hardened product. There are three main stabilizing agents which can be added, and the method of treatment bears the name of these agents: Soil-cement, soil-lime, and lime-fly ash. The methods of chemical stabilization have much in common and involve somewhat similar construction practices. They depend upon hydration, pozzolanic action of lime with silica and alumina, alteration of the clay material, or a combination of these actions. The result is a semirigid, fairly brittle material with considerable compressive strength and moderate flexural strength when tested either statically or dynamically. The ultimate strength depends to a great degree on the density which is achieved during compaction and before the mix cures.

Table 5-1. Stabilization methods most suitable for specific applications

Purpose	Soil type	Method
Subgrade stabilization Improved load carrying and stress distribution characteristics	Fine granular	SA, SC, MB, C
	Coarse granular	SA, SC, MB, C
	Clays of low PI	C, SC, CMS, LMS, SL
	Clays of high PI	SL, LMS
Reduce frost susceptibility	Fine granular	CMS, SA, SC, LF
	Clays of low PI	CMS, SC, SL, LMS
Waterproofing and improved runoff	Clays of low PI	CMS, SA, LMS, SL
Control of shrinkage and swell	Clays of low PI	CMS, SC, C, LMS, SL
	Clays of high PI	SL
Reduce resiliency	Clays of high PI	SL, LMS
	Elastic silts or clays	SC, CMS
Base course stabilization Improvements of substandard materials	Fine granular	SC, SA, LF, MB
	Clays of low PI	SC, SL
Improved load carrying and stress distribution characteristics	Course granular	SA, SC, MB, LF
	Fine granular	SC, SA, LF MB
Reduction of pumping	Fine granular	SC, SA, LF, MB, membranes
Dust palliative	Fine granular	CMS, SA, Oil or bituminous surface spray, APSB
	Plastic soils	CMS, SL, LMS, APSB, DCA 70

LEGEND:

Where the methods of treatment are:

APSB = Asphalt penetration surface binder
C = Compaction
CMS = Cement modified soil
DCA70 = Polyvinylacetate emulsion
LF = Lime fly ash
LMS = Lime modified soil
MB = Mechanical blending
SA = Soil-asphalt
SC = Soil-cement
SL = Soil-lime

- Bituminous Stabilization. In bituminous treatment, the end product performs differently at least initially, the product is much less brittle; and its behavior depends on the nature of the loading (static or dynamic) and the temperature when the load is applied.

Modification

Soil stabilization by modification usually results in something less than a thoroughly cemented, hardened or semihardened material. This type of stabilization may be accomplished by compacting, by mechanical blending, by adding cementing materials in small amounts, or by adding chemical modifiers. Cement and lime modifiers (cement-modified soil and lime-modified soil) are used in quantities too small to provide high-strength cementing action. They reduce the plasticity of clay soils. Calcium chloride or sodium chloride are added to the soil to retain moisture (and also control dust), to hold fine material for better compaction, and to reduce frost heave by lowering the freezing point of water in the soil. Bituminous materials, such as cutback asphalts or asphaltic penetrative soil binder (APSB), and certain chemicals, such as polyvinyl acetate emulsion (DCA-70), are used to waterproof the soil surface and to control dust.

Section II. TYPE AND AMOUNT OF STABILIZING AGENT

DISCUSSION

This section provides a method for determining the type or types of stabilizers and the amount of stabilizer to be used with a particular soil and considers stabilization of soils with lime, cement, fly ash, and bituminous materials. In order to select the proper stabilizer type for a particular soil, a sieve analysis and Atterberg limits tests must be performed, in accordance with the procedures given in this manual.

GENERAL REQUIREMENTS FOR USE OF STABILIZERS

Lime

Experience shows that lime will react with many medium, moderately fine, and fine-graned soils to produce decreased plasticity, increased workability, reduced swell, and increased strength. Soils classified according to the Unified Soil Classification System as CH, CL, MH, ML, OH, OL, SC, SM, GC, GM, SW-SC, SP-SC, SM-SC, GW-GC, GP-GC, ML-CL, and GM-GC should be considered as potentially capable of being stabilized with lime.

Cement

Cement can be used as an effective stabilizer for a wide range of materials. In general, 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.

Fly Ash

Fly ash, when mixed with lime, can be used effectively to stabilize most coarse and medium grained soils. However, the PI should not be greater than 25. Soils classified by the USCS as SW, SP, SP-SC, SW-SC, SW-SM, GW, GP, GP-GC, GW-GC, GP-GM, GW-GM, GC-GM, SC-SM can be stabilized with fly ash.

Bituminous

Most bituminous soil stabilization has been performed with asphalt cement, cutback asphalt, and asphalt emulsions. Soils that can be stabilized effectively with bituminous materials usually contain less than 30 percent passing the No. 200 sieve and have a PI less

Table 5-2. Estimated time required for test procedures

Type of construction	Stabilizing agent	Time required*
Expedient	Lime LCF Cement Bitumen	None None None None
Nonexpedient	Lime LCF Cement Bitumen	30 days 30 days 6-9 days 1 day

*These criteria do not include time required for gradation and classification tests on the untreated soil.

than 10. Soils classified by the USCS as SW, SP, SW-SM, SP-SM, SW-SC, SP-SC, SM, SC, SM-SC, GW, GP, SW-GM, SP-GM, SW-GC, GP-GC, GM, GC, and GM-GC can be effectively stabilized with bituminous materials provided the above mentioned gradation and plasticity requirements are met.

Combination

Combination stabilization is specifically defined as lime-cement, lime-asphalt, and lime-cement-fly ash (LCF) stabilization. Combinations of lime and cement often are acceptable expedient stabilizers. Lime can be added to the soil to increase the workability and mixing characteristics of the soil as well as reduce its plasticity. Cement can then be mixed into the soil to provide rapid strength gain. Combinations of lime and asphalt are often acceptable stabilizers. The lime addition may prevent stripping at the asphalt-aggregate interface and increase the stability of the mixture.

TIME REQUIRED FOR TESTING

The more thorough a testing program, the more assurance there is for the long-term success of the project. Time is often of primary concern to the 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 5-2). Because of this, the method presented allows for a rapid or expedient approximation along with a more precise laboratory determination of the type and quantity of stabilizer. An estimate for testing time is presented in Table 5-2.

SELECTION OF STABILIZER

In the selection of a stabilizer additive, the factors that must be considered are the type of soil to be stabilized, the purpose for which the stabilized layer will be used, the type of soil quality improvement desired, the required strength and durability of the stabilized layer, and the cost and environmental conditions.

The solid gradation triangle in Figure 5-1, page 5-6 is based upon the pulverization characteristics of the soil. When these characteristics are combined with certain restrictions relative to liquid limit (LL), and soil gradation contained in Table 5-3 (page 5-8), they provide guidance for the selection of the additive best suited for stabilization. Figure 5-2 (page 5-13),is entered with the percentage of gravel (percent material retained on the No. 4 sieve), sand (percent material passing the No. 4 sieve and retained on the No. 200 sieve, and fines, (percent material passing the No. 200 sieve) to determine the area in which

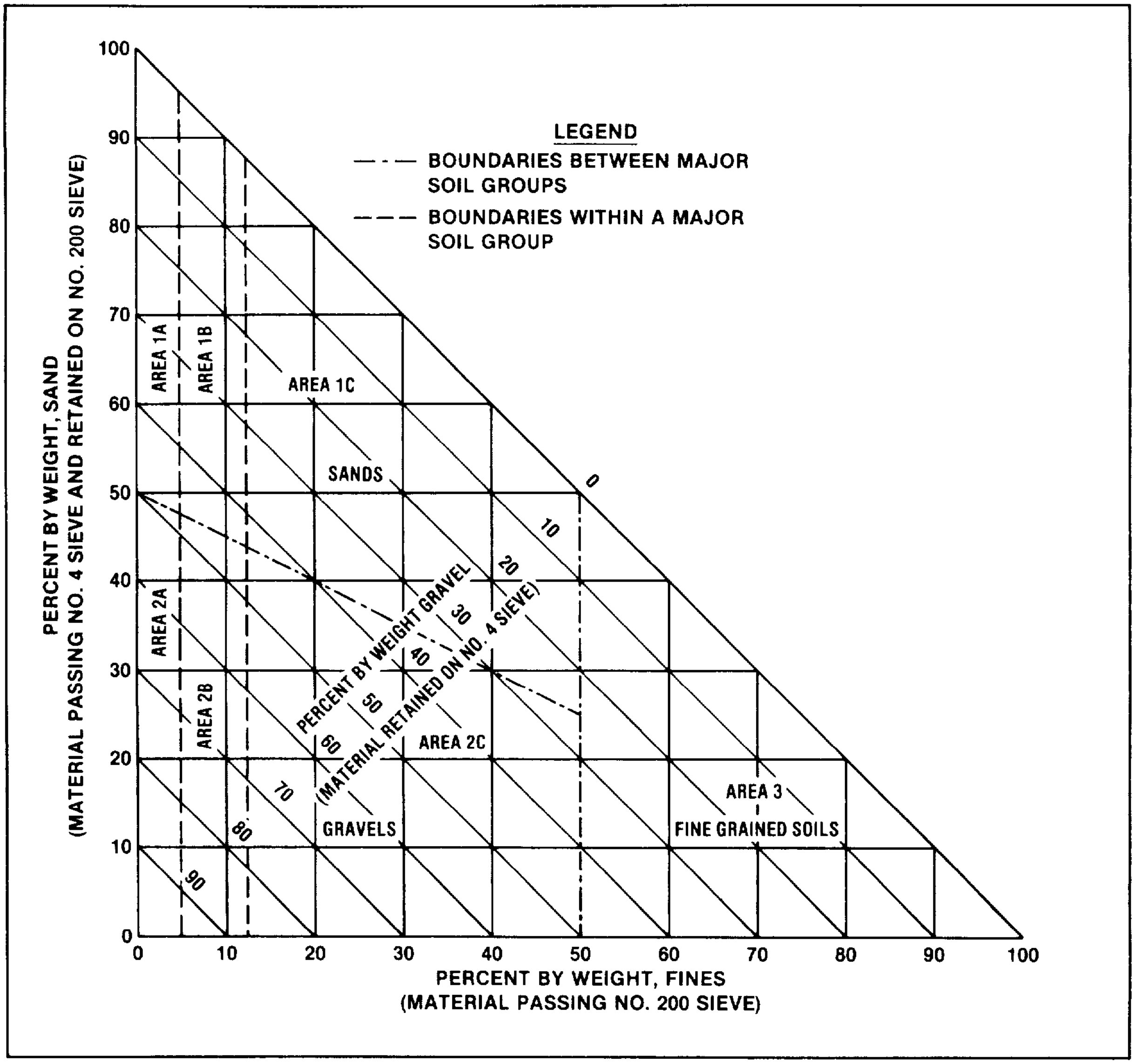


Figure 5-1. Soil gradation triangle

the soil gradation falls. The area (1A, 2C, and 3) indicated at the intersection of the three material percentages is used to enter Table 5-3, page 5-8, to select the type of stabilizing additive based on the various restrictions and remarks. For example, a soil having a PI of 15 and 57 percent gravel, 26 percent sand, and 7 percent fines fall in Area 2B of Figure 5-1. Table 5-3 indicates that cement, lime, LCF, or bitumen could be considered. Howver, the PI of 15 eliminates bitumen, and the fact that only 33 percent of the material passes the No. 4 sieve indicates that lime or a combination of LCF will be the best additive for stabilization.

STABILIZATION WITH PORTLAND CEMENT

Portland cement can be used either to modify and improve the quality of the soil or to transform the soil into a cemented mass, which significantly increases its strength and durability. The amount of cement additive will depend upon whether the soil is to be modified or stabilized. The only limitation to the amount of cement to be used to stabilize or modify a soil pertains to the treatment of the base courses to be used in flexible pavement systems. When a cement-treated base course is to be surfaced with asphaltic concrete, the percent of cement by weight will be limited to 4 percent in Air Force pavements and 5 percent for Navy pavements.

Cement Content for Modification of Soils

- The amount of cement required to improve the quality of the soil through modification is determined by trial and error. If it is desired to reduce the PI of the soil, successive samples of soil-cement mixtures must be prepared at different treatment levels, and the PI of each mixture must be determined. The minimum cement content that yields the desired PI is selected, but since it was determined based upon the minus 40 fraction of the material, this value must be adjusted to find the design cement content based upon total sample weight expressed as:

$$A = 100Bc$$

where:

A = design cement content, percent total weight of soil

B = percent cement required to obtain the desired PI of minus 40 material, expressed as a decimal

C = percent cement required to obtain the desired PI of minus 40 material, expressed as a decimal.

- If the objective of modification is to improve the gradation of granular soil through the addition of fines, the analysis should be conducted on samples at various treatment levels to determine the minimum acceptable cement content. The determination of cement content to reduce the swell potential of fine-graded plastic soils can be accomplished by molding several samples at various cement contents and soaking the specimens, along with untreated specimens, for 4 days. The lowest cement content that eliminates the swell potential or reduces the swell characteristics to the minimum becomes the design cement content. The cement content determined to accomplish soil modification should be checked to see whether it provides an unconfined compressive strength great enough to qualify for a reduced thickness design in accordance with criteria established for soil stabilization (Tables 5-4 and 5-5, page 5-9).

- Cement-modified soil also may be used in frost areas. In addition to the procedures for mixture design described above, cured specimens should be subjected to the 12 freeze-thaw cycles test (but omitting wire brushing) or other applicable freeze-thaw procedures. This should be followed by frost-susceptibility tests and should meet the requirements set forth for the base course. If cement-modified soil is used as the subgrade, its frost susceptibility should be used as the basis of the pavement thickness design if the reduced subgrade strength design method is applied.

Cement Content for Cement-Stabilized Soil

The following procedure is recommended for determining the design cement content for cement-stabilized soils, using the following steps:

1. Determine the classification and gradation of the untreated soil. The soil must meet the gradation requirements shown in Table 5-6, page 5-9, before it can be used in a reduced thickness design.

Table 5-3. Guide for selecting a stabilizing additive

Area	Soils class [1]	Type of stabilizing additive recommended	Restriction on LL and PI of soil	Restriction on percent passing No. 200 sieve[1]	Remarks
1A	SW or SP	Bituminous			
		Portland cement			
		Lime-cement-fly ash	PI not to exceed 25		
1B	SW-SH or SP-SH or SW-SC or	Bituminous	PI not to exceed 10		
		Portland cement	PI not to exceed 30		
		Lime	PI not less than 12		
1C	SM or SC or SM-SC	Bituminous	PI not to exceed 10	Not to exceed 30% by weight	
		Portland cement	See footnote 2		
		Lime	PI not less than 12		
		Lime-cement-fly ash	PI not to exceed 25		
2A	GW or GP	Bituminous			Well-graded material only
		Portland cement			Material should contain at least 45% by weight of material passing No. 4 sieve
		Lime-cement-fly ash	PI not to exceed 25		
2B	GW-GH or GP-GH or GW-GC or GP-GC	Bituminous	PI not to exceed 10		Well-graded material only
		Portland cement	PI not to exceed 30		Material should contain at least 45% by weight of material passing No. 4 sieve
		Lime	PI not less than 12		
		Lime-cement-fly ash	PI not to exceed 25		
2C	GM or GC or GM-GC	Bituminous	PI not to exceed 10	Not to exceed 30% by weight	Well-graded material only
			See footnote 2		Material should contain at least 45% by weight of material passing No. 4 sieve
		Lime	PI not less than 12		
		Lime-cement-fly ash	PI not to exceed 25		
3	CH or CL or HII or ML or OII or OL or ML-CL	Portland cement	LL less than 40 and PI less than 20		Organic and strongly acid soils falling within this area are not susceptible to stabilization by ordinary means
		Lime	PI not less than 12		

[1] Soil classification corresponds to MIL-STD-619B. Restriction on liquid limit (LL) and plasticity index (PI) is in accordance with Method 103 in MIL-STD-621A.

[2] $PI \leq 20 + \frac{50 - \text{percent passing No. 200 sieve}}{4}$

Table 5-4. Minimum unconfined compressive strengths

Stabilized soil layer	Minimum unconfined compressive strength, psi*		
	Flexible pavement		Rigid pavement all
	Air Force and Army	Navy	
Base course	750	750	500
Subbase course, select material or subgrade	250	300 (cement) 150 (lime)	200

*Unconfined compressive strength determined at 7 days for cement stabilization and 28 days for lime or lime-cement-fly ash stabilization.

Table 5-5. Durability requirements

Type of soil stabilized	Maximum allowable weight loss after 12 wet-dry or freeze-thaw cycles percent of initial specimen weight	
	Army and Air Force	Navy
Granular, PI < 10	11	14
Granular, PI > 10	8	14
Silt	8	14
Clays	6	14

Table 5-6. Gradation requirements

Type course	Sieve size	Percent Passing	
		Army and Air Force	Navy*
Base	2-in	100	——
	1½-in	70-100	——
	1-in	45-100	100
	¾-in	——	90-100
	½-in	30-90	——
	No. 4	20-70	40-70
	No. 10	15-60	——
	No. 30	——	12-40
	No. 40	5-40	——
	No. 200	0-20	3-15
Subbase	3-in	100	100
	No. 4	——	45-100
	No. 10	——	36-60
	No. 100	——	3-20
	No. 200	0-25	0-3

*Base course aggregate to have a sand equivalent of not less than 18 (ASTMD 2419).

Table 5-7. Estimated cement requirements for various soil types

Soil classification	Initial estimated cement requirement percent dry weight
GW-SW	5
GP, SW-SM, SW-SC, SW-GM, SW-GC	6
GM, SM, GC, SC, SP-SM, SP-SC, GP-GM, GP-GC, SM-SC, GM-GC	7
SP, CL, ML, ML-CL	10
MH, OH	11
CH	10

Table 5-8. Average cement requirements for granular and sandy soils

Material retained on No. 4 sieve percent	Material smaller than 0.05 mm percent	Cement content, percent by weight maximum dry density, lb/cu ft (treated material)					
		116-120	121-126	127-131	132-137	138-142	143 or more
0-14	0-19	10	9	8	7	6	5
	20-39	9	8	7	7	5	5
	40-50	11	10	9	8	6	5
15-29	0.19	10	9	8	6	5	5
	20-39	9	8	7	6	6	5
	40-50	12	10	9	8	7	6
30-45	0-19	10	8	7	6	5	5
	20-39	11	9	8	7	6	5
	40-50	12	11	10	9	8	6

Note: Base course goes to 70 percent retained on the No. 4 sieve.

2. Select an estimated cement content from Table 5-7 using the soil classification. For expedient design, use this cement content.

3. Use the estimated cement content and determine the compaction curve of the soil-cement mixture.

4. Use the untreated soil gradation characteristics, cement content, and maximum dry density determined in steps one, two, and three and verify the estimated cement content using Tables 5-8 or 5-9, page 5-12, and Figure 5-2, page 5-13, depending upon the soil classification. If the estimated cement content from step two varies by more than ± 2 percent from the value in Tables 5-8 or 5-9, conduct additional compaction tests, varying the cement content until the value from Table 5-8 or 5-9 is within ± 2 percent of that used for the moisture-density test.

5. Prepare samples of the soil-cement mixture for unconfined compression and durability tests at the dry density, at the cement content, at 2 percent above, and at 2 percent below the cement content determined in step four. The samples should be prepared in accordance with Chapter two except that when more than 35 percent of the material is retained on the No. 4 sieve a CBR mold should be used to prepare the specimens. Cure the specimens for 7 days in a humid room before testing. Test three specimens using the unconfined compression test and subject three specimens to durability tests, either wet-dry tests for pavements located in nonfrost areas or freeze-thaw tests for pavements located in frost areas.

6. Compare the results of the unconfined compressive strength and durability tests with the requirements shown in Tables 5-4 and 5-5, page 5-9. The lowest cement content which meets the required unconfined compressive strength requirement and demonstrates the required durability is the design cement content. If the mixture should meet the durability requirements but not the strength requirements, the mixture is considered to be a modified soil.

STABILIZATION WITH LIME

Lime can be used either to modify some of the physical properties and improve the quality of a soil or to transform the soil into a stabilized mass and increase its strength and durability. The amount of lime additive will depend upon whether the soil is to be modified or stabilized. The lime used may be either hydrated or quicklime, although hydrated lime is better since quicklime is highly caustic and dangerous to use. The design lime content determined from the criteria presented below are for hydrated lime. As a guide, the lime contents determined for hydrated lime should be reduced by 25 percent to determine a design content for quicklime.

LIME CONTENT FOR LIME-MODIFIED SOILS

The amount of lime required to improve the quality of a soil is determined through the same trial-and-error process used for cement-modified soils.

LIME CONTENT FOR LIME-STABILIZED SOILS

Strength Requirements

To take advantage of the thickness reduction criteria, the lime-stabilized soil must meet the unconfined compressive strengths shown in Table 5-4, page 5-9, as well as the durability requirements in Table 5-5, page 5-9.

Procedures

When lime is added to a soil, a combination of reactions begins to take place immediately and is nearly complete within an hour, although substantial strength gain is not reflected for some time. These reactions result in a change in both the chemical composition and physical properties. Most lime, when placed in a water

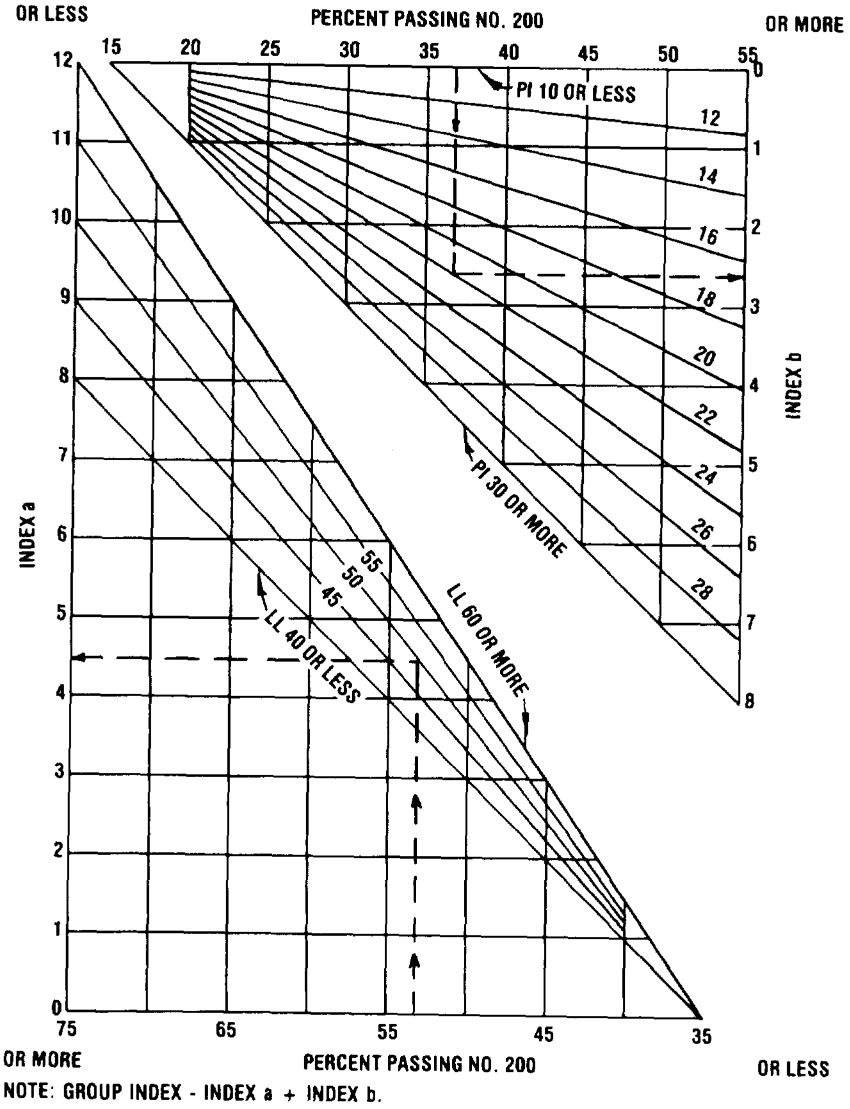
Table 5-9. Average cement requirements for silty and clayey soils

Group index	Material between 0.05 and 0.005 mm percent	Cement content, percent by weight maximum dry density, lb/cu ft (treated material)						
		99-104	105-109	110-115	116-120	121-126	127-131	132 or more
0-3	0-19	12	11	10	8	8	7	7
	20-39	12	11	10	9	8	8	7
	40-59	13	12	11	9	9	8	8
	60 or more	—	—	—	—	—	—	—
3-7	0-19	13	12	11	90	8	7	7
	20-39	13	12	11	10	0	8	8
	40-59	14	13	12	10	10	9	8
	60 or more	15	14	12	11	10	9	9
7-11	0-19	14	13	11	10	9	8	8
	20-39	15	14	11	10	9	9	9
	40-59	16	14	12	11	10	10	9
	60 or more	17	15	13	11	10	10	10
11-15	0-19	15	14	13	12	11	9	9
	20-39	16	15	13	12	11	10	10
	40-59	17	16	14	12	12	11	10
	60 or more	18	16	14	13	12	11	11
15-20	0-19	17	16	14	13	12	11	10
	20-39	18	17	15	14	13	11	11
	40-59	19	18	15	14	14	12	12
	60 or more	20	19	16	15	14	13	12

solution, has a pH of about 12.4. Therefore, the pH is a good indicator of the desirable lime content of a soil-lime mixture. The reaction that takes place when lime is introduced to a soil generally causes a significant change in the plasticity of the soil. Therefore, the changes in the plastic and liquid limits also become an indicator of the desired lime content. Two methods are presented for the determination of the initial design lime content.

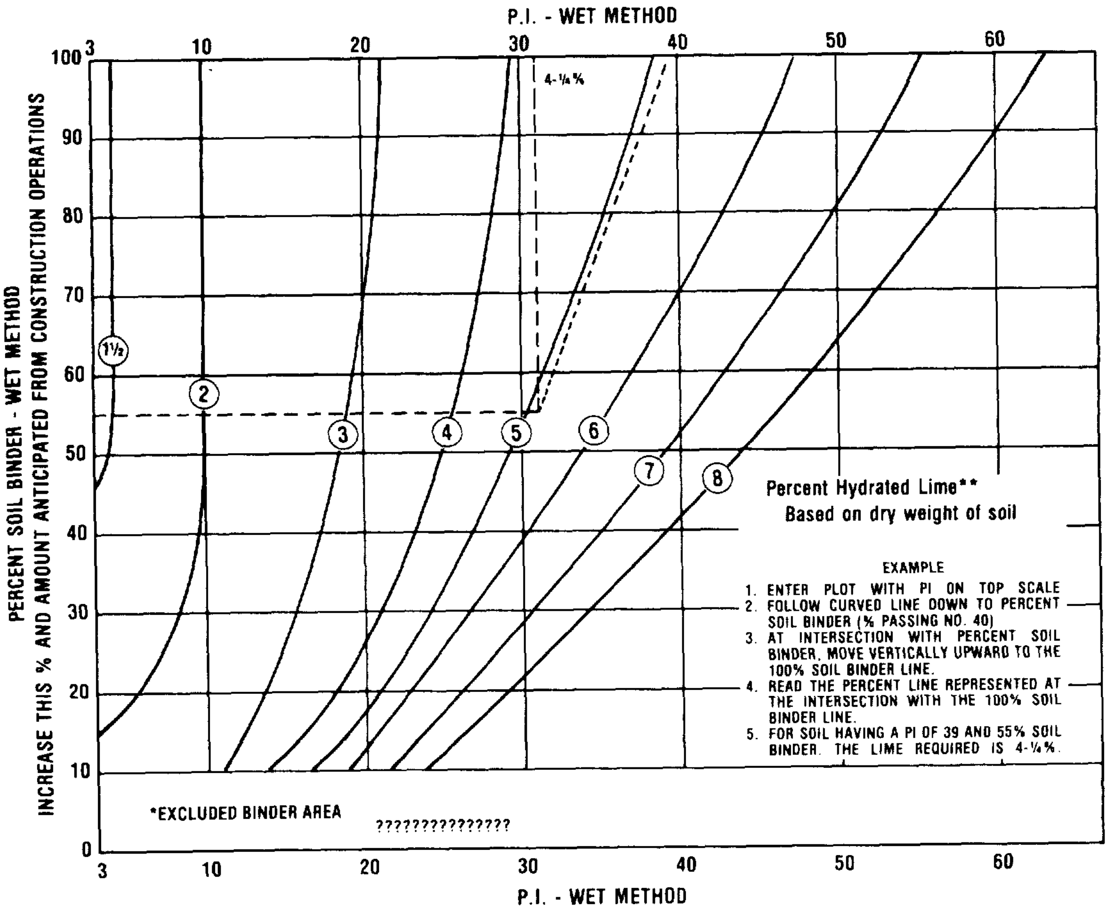
- The preferred method is to prepare several mixtures at different lime treatment levels and determine the pH of each mixture after 1 hour. The lowest lime content producing the highest pH of the soil-lime mixture is the initial design lime content. Procedures for conducting a pH test on lime-soil mixtures have been described. In frost areas, specimens must be subjected to the freeze-thaw test. An alternate method of determining an initial design lime content is by the use of Figure 5-3, page 5-14. This method should be used for expedient lime determination. Specific values required to use Figure 5-3 are the PI and the percent of material passing the No. 40 sieve. These properties are determined from plastic limit and graduation tests on the untreated soil.

- After the initial lime content has been estimated, conduct a compaction test with the lime-soil mixture. Following the same procedures for soil-cement except that the mixture



Adapted from **Soil-Cement Laboratory Handbook** (EB052.06S)

Figure 5-2. Average cement requirements



EXAMPLE

1. ENTER PLOT WITH PI ON TOP SCALE
2. FOLLOW CURVED LINE DOWN TO PERCENT SOIL BINDER (% PASSING NO. 40)
3. AT INTERSECTION WITH PERCENT SOIL BINDER, MOVE VERTICALLY UPWARD TO THE 100% SOIL BINDER LINE.
4. READ THE PERCENT LINE REPRESENTED AT THE INTERSECTION WITH THE 100% SOIL BINDER LINE.
5. FOR SOIL HAVING A PI OF 39 AND 55% SOIL BINDER. THE LIME REQUIRED IS 4-¼%.

* Exclude use of chart for materials with less than 10% - No. 40 and cohesionless materals (P.I. less than 3)

** Percent of relatively pure lime usually 90% or more of Ca and/or Mg hydroxides and 85% or more of which pass the No. 200 sieve. Percentages shown are for stabilizing subgrades and base courses where lasting effects are desired. Satisfactory temporary results are sometimes obtained by the use of as little as ½ of above percentages. Reference to cementing strength is implied when such terms as "Lasting Effects" and "Temporary Results" are used.

Adapted from The National Lime Association, Washington, D.C.

Figure 5-3. Estimated lime requirements

should be allowed to cure no less than 1 hour and no more than 2 hours in a sealed container before molding. Compaction will be accomplished in five layers using 55 blows of a 10-pound hammer having an 18-inch drop. The moisture-density should be determined at lime contents equal to design +2 and +4 percent for the preferred method and at design ± 2 percent for the alternate method. In frost areas, cured specimens should be subjected to the 12 freeze-thaw cycles (but omitting wire brushing) or other applicable freeze-thaw procedures, followed by frost susceptibility determinations in standard laboratory freezing tests. For lime-stabilized or lime-modified soil used in lower layers of the base course, the frost susceptibility should meet the requirements set forth for the base course. If lime-stabilized or lime-modified soil is used as the subgrade, its frost susceptibility should be used as the basis of the pavement thickness design if the reduced subgrade strength design method is applied.

- Unconfined compression tests should be performed at the design percent of maximum density on three specimens for each lime content tested. The design value would then be the minimum lime content yielding the required strength. Procedures for the preparation of lime-soil specimens are similar to those used for cement-stabilized soils, with two exceptions. After mixing, the lime-soil mixture should be allowed to mellow for not less than 1 hour or more than 2 hours. After compaction, each specimen should be wrapped securely to prevent moisture loss and should be cured in a constant temperature chamber at 73° ± 2°F for 28 days. Procedures for conducting unconfined compression tests are similar to those used for soil-cement specimens except that in place of moist curing, the lime-soil specimens should remain securely wrapped until testing.

- Compare results of the unconfined compressive tests with the criteria in Table 5-4 page 5-9. The design lime content must be the lowest lime content of specimens meeting the strength criteria indicated.

LIME AND OTHER ADDITIVES

If lime is used as a preliminary additive to reduce the PI or to alter the gradation of soil before adding the primary stabilizing agent such as bitumen or cement, then the design lime content is the minimum treatment level that will achieve the desired results. For nonplastic and low-PI materials in which lime alone generally is not satisfactory for stabilization, the addition of fly ash may be needed to produce the necessary reaction.

THE pH TEST

Lime to be used for soil stabilization is used to determine lime requirements in the pH test.

Apparatus

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

50-milliliter plastic breakers

Distilled water that is free of carbon dioxide (CO_2)

Balance

Oven

Moisture cans

Procedure

- Standardize the pH meter with a buffer solution having a pH of 12.45.

- Weigh to the nearest 0.01 gram representative samples of air-dried soil, passing the No. 40 sieve and equal to 20.0 grams of oven-dried soil.

- Pour the soil samples into 150-milliliter plastic bottles with screw-top lids.

- 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.)

- Mix the soil thoroughly and dry the lime.

- Add 100 milliliters of distilled water that is free of carbon dioxide (CO_2) to the soil-lime mixtures.

- Shake the soil-lime and water for a minimum of 30 seconds or until there is no evidence of dry material on the bottom of the bottle.

- Shake the bottles for 30 seconds every 10 minutes.

- Transfer part of the slurry, after 1 hour, to a plastic breaker and measure the pH.

- 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.

STABILIZATION WITH LCF

Stabilization of coarse-grained soils having little or no fines can often be accomplished by the use of the LCF combination. Fly ash is a pozzolanic material, consisting mainly of silicon and aluminum compounds. When mixed with lime and water, fly ash hardens into a mass capable of obtaining high compressive strengths. Thus, lime and fly ash in combination can often be used successfully in stabilizing granular materials having few fines. In addition to lime and fly ash, a small amount of portland cement is also added to accelerate and increase strength gain. The use of LCF mixtures is not permitted for Air Force airfield pavements if the cement content exceeds 4 percent, but may be permitted for Navy and Marine Corps airfields, subject to approval by Headquarters, Naval Facilities Engineering Command.

Materials

Types of materials suitable for the LCF stabilization are coarse-grained soils having no more than 12 percent of the material passing the No. 200 sieve. In addition, the PI of the minus 40 fraction should not exceed 25.

Proportions

Initial mix will be 1 percent portland cement, 4 percent lime, 24 percent fly ash, and 71 percent soil. Minimum unconfined compressive strength requirements are indicated in Table 5-4 page 5-9. If test specimens do not meet strength requirements, add cement in increments of 1/2 percent until strengths are adequate. The total quantity of additives should not exceed 15 percent by weight. In frost areas, the LCF mixture should meet the weight loss criteria specified for cement-stabilized soils. The specimens should be compacted in the CBR mold.

STABILIZATION WITH BITUMEN

Bituminous-Stabilized Soils

- Soil-bitumen. A cohesive soil system made water-resistant with a bituminous admixture.

- Sand-bitumen. A system in which sand is cemented together by bituminous material.

- Oiled earth is an earth-road system made resistant to water absorption and abrasion by means of a sprayed application of slow-curing or medium-curing liquid asphalts.

- Bitumen-waterproofed, mechanically stabilized soil is a system in which two or more soil materials are blended to produce a good gradation of particles from coarse to fine.

Comparatively small amounts of bitumen are needed, and the processed soil is compacted.

- Bitumen-lime blend is a system in which small percentages of lime are blended with fine-grained soils to facilitate the penetration and mixing of bitumens into the soil.

Soil Gradation

The recommended soil gradations for subgrade materials and base or subbase materials are shown in Tables 5-10 and 5-11, respectively. Mechanical stabilization may be required to bring soil to proper gradation.

Types of Bitumen

Bituminous stabilization is generally accomplished using asphalt cement, cutback asphalt, or asphalt emulsions. The type of bitumen to be used depends upon the type of soil to be stabilized, method of construction, and weather conditions. In frost areas, the use of tar as a binder should be avoided because of its high-temperature susceptibility. Asphalts are affected less by temperature changes, but a grade of asphalt suitable to the prevailing climate should be selected. As a general rule, the most satisfactory results are obtained using the most viscous liquid asphalt that can be readily mixed into the soil. For higher quality mixes in

Table 5-10. Recommended gradations for bituminous-stabilized subgrade materials

Sieve size	Percent passing
3 in	100
No. 4	50-100
No. 30	38-100
No. 200	2-30

Table 5-11. Recommended gradations for bituminous-stabilized subbase materials

Sieve size	1½-in Maximum	1-in Maximum	¾-in Maximum	½-in Maximum
1½ in	100	——	——	——
1 in	84 ± 9	100	——	——
¾ in	76 ± 9	83 ± 9	100	——
½ in	66 ± 9	73 ± 9	82 ± 9	100
⅜ in	59 ± 9	64 ± 9	72 ± 9	83 ± 9
No. 4	45 ± 9	48 ± 9	54 ± 9	62 ± 9
No. 8	35 ± 9	37 ± 9	41 ± 9	47 ± 9
No. 16	27 ± 9	28 ± 9	32 ± 9	36 ± 9
No. 30	20 ± 9	21 ± 9	24 ± 9	28 ± 9
No. 50	14 ± 7	16 ± 7	17 ± 7	20 ± 7
No. 100	9 ± 5	11 ± 5	12 ± 5	14 ± 5
No. 200	5 ± 2	5 ± 2	5 ± 2	5 ± 2

Table 5-12. Bituminous requirements

Open-graded aggregate
Rapid- and medium-curing liquid asphalts RC-250, RC-800, and MC-3000
Medium-setting asphalt emulsion MS-2 and CMS-2
Well-graded aggregate with little or no material passing the No. 200 sieve
Rapid- and medium-curing liquid asphalts RC-250, RC-800, MC-250, and MC-800
Slow-curing liquid asphalts SC-250 and SC-800
Medium-setting and slow-setting asphalt emulsions MS-2, CMS-2, SS-1, and CSS-1
Aggregate with a considerable percentage of fine aggregate and material passing the No. 200 sieve
Medium-curing liquid asphalts MC-250 and MC-800
Slow-curing liquid asphalts SC-250 and SC-800
Slow-setting asphalt emulsions SS-1, SS-1h, CSS-1, and CSS-1h
Medium-setting asphalt emulsions MS-2 and CMS-2
The simplest type of bituminous stabilization is the application of liquid asphalt to the surface of an unbound aggregate road. For this type of operation, the slow- and medium-curing liquid asphalts SC-70, SC-250, MC-70, and MC-250 are used.

which a central plant is used, viscosity-grade asphalt cements should be used. Much bituminous stabilization is performed in place. The bitumen is applied directly on the soil or soil-aggregate system, and the mixing and compaction operations are conducted immediately thereafter. For this type of construction, liquid asphalts such as cutback and emulsions are used. Emulsions are preferred over cutbacks because of energy constraints and pollution control efforts. The specific type and grade of bitumen will depend on the characteristics of the aggregate, type of construction equipment, and climate conditions. Generally, the types of bituminous materials that will be used for the soil gradation are indicated in Table 5-12.

Mix Design

Guidance for the design of bituminous-stabilized base and subbase courses is contained in TM 5-822-8. For subgrade stabilization, the following equation may be used for estimating the preliminary quantity of cutback asphalt to be selected:

$$p = \frac{0.02(a) + 0.07(b) + 0.15(c) + 0.20(d)}{(100 - S)} \times 100$$

where:

p = percent of cutback asphalt by weight of dry aggregate

a = percent of mineral aggregate retained on No. 50 sieve b = percent of mineral aggregate passing No. 50 and retained on No. 100 sieve

c = percent of mineral aggregate passing No. 100 and retained on No. 200 sieve

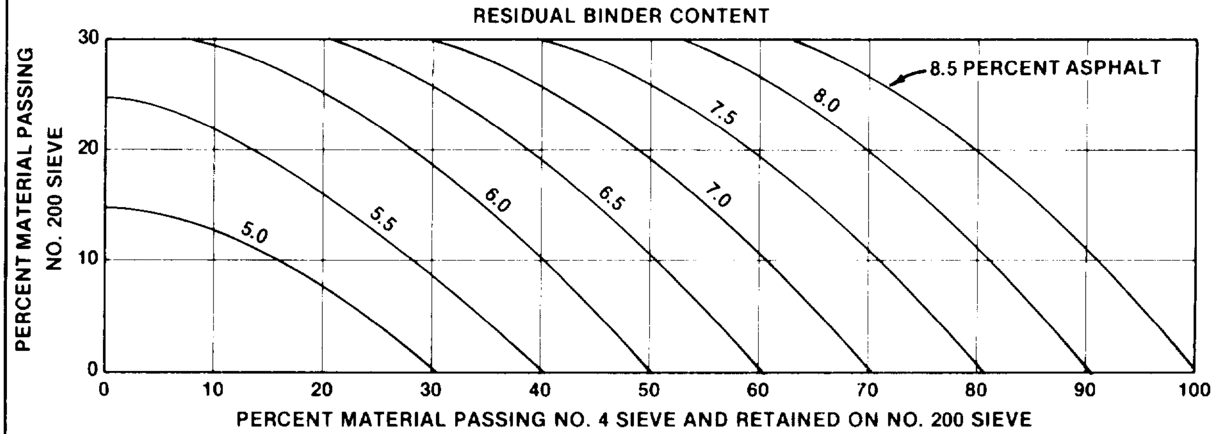
d = percent of mineral aggregate passing No. 200 sieve

s = percent solvent.

The preliminary quantity of emulsified asphalt to be used in stabilizing subgrades can be determined from Table 5-13, page 5-20. Figure 5-4 can be used to find mix design for asphalt cement. The final design content of asphalt should be selected based upon the results of the Marshall stability test procedure. The minimum Marshall stability recommended for subgrades is 500 pounds; for base courses, 750 pounds. If a soil does not show increased stability when reasonable amounts of bituminous materials are added, the gradation of the soil should be modified, or another type of bituminous material should be used. Poorly graded materials may be improved by the addition of suitable fines containing considerable material passing the No. 200 sieve. The amount of bitumen required for a given soil increases with an increase in percentage of finer sizes.

CLIMATIC ZONES	ASPHALT GRADE, PENETRATION
ARCTIC	100 TO 120
TEMPERATE	85 TO 100
TROPIC	60 TO 70
DESERT	40 TO 50

DETERMINATION OF ASPHALT GRADE
FOR EXPEDIENT CONSTRUCTION



SELECTION OF ASPHALT CEMENT CONTENT

Figure 5-4. Selection of grade and quantity of asphalt cement

Table 5-13. Emulsified asphalt requirements

Percent passing No. 200 sieve	Pounds of emulsified asphalt per 100 pounds of dry aggregate at percent passing No. 10 sieve					
	≤ 50	60	70	80	90	100
0	6.0	6.3	6.5	6.7	7.0	7.2
2	6.3	6.5	6.7	7.0	7.2	7.5
4	6.5	6.7	7.0	7.2	7.6	7.7
6	6.7	7.0	7.2	7.5	7.7	7.9
8	7.0	7.2	7.5	7.7	7.9	8.2
10	7.2	7.5	7.7	7.9	8.2	8.4
12	8.5	7.7	7.9	8.2	8.4	8.6
14	7.2	7.5	7.7	7.9	8.2	8.4
16	7.0	7.2	7.5	7.7	7.9	8.2
18	6.7	7.0	7.2	7.5	7.7	7.9
20	6.5	6.7	7.0	7.2	7.5	7.7
22	6.3	6.5	6.7	7.0	7.2	7.5
24	6.0	6.3	6.5	6.7	7.0	7.2
25	6.2	6.4	6.6	6.9	7.1	7.4

USE OF STABILIZED SOILS IN FROST AREAS

Additives

Bitumen, portland cement, lime, and combinations of lime, portland cement, and fly ash (LCF) are the most common additives for use in stabilized soils. Other stabilizers may be used for pavement construction in frost areas only with approval obtained from the Headquarters, Department of the Army (DAEN-MPE-D). Washington, DC 20314 (for Army projects); Headquarters, Air Force Engineering and Services Center (AFESC/DEM), Tyndall AFB, FL 32401 (for Air Force projects); or Headquarters, Naval Facilities Engineering Command, Alexandria, VA 22332 (for Navy or Marine Corps projects).

Limitations

In frost areas, stabilized soil will be used only in a layer or layers comprising one of the upper elements of a pavement system and directly beneath the pavement surfacing layer, where the added cost of stabilization is compensated for by its structural advantage in reducing the required thickness of the pavement system. Treatment with a lower degree of chemical stabilization should be used in frost areas only with caution and after intensive tests, because weakly cemented material usually has less capacity to endure repeated freezing and thawing than firmly cemented material. A possible exception is the use of 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 moderate amounts of a stabilizing additive. Materials that are modified by small amounts of a chemical additive to improve certain properties of the soil without significant cementation also should be tested to determine that the desired improvement is durable through repeated freeze-thaw cycles. The improvement should not be achieved at the expense of making the soil more susceptible to ice segregation.

Construction Cutoff

For materials stabilized with cement, lime, or LCF whose strength increases with time of curing, it is essential that the stabilized layer be constructed sufficiently early in the season to allow the development of adequate strength before the first freezing cycle begins. The rate of strength gain is substantially lower at 50°F than at 70° or 80°F. Chemical reactions will not occur rapidly for lime-stabilized soils when the soil temperature is less than 60°F and is not expected to increase for one month, or for cement-stabilized soils when the soil temperature is less than 40°F and is not expected to increase for one month. In frost areas, it is not always sufficient to protect the mixture from freezing during a 7-day curing period as required by the applicable guide specifications. A construction cutoff date well in advance of the onset of freezing conditions may be essential.

Weather

Hot dry weather is preferred for all types of bituminous stabilization. When asphalt cements are used for stabilization, proper compaction must be obtained. If thin lifts of asphalt-stabilized material are being placed, the air temperature should be 40°F and rising, and the compaction equipment should be used immediately after lay-down operations. Adequate compaction can be obtained at freezing temperatures if thick lifts are used. When cutbacks and emulsions are used, the air and soil temperatures should be above freezing. Heavy rains on mixed, but uncompacted, material may be detrimental.

Example

- A unit is given the mission of constructing an airfield facility that will sustain operations of C-130 aircraft (gross weight of 150,000 pounds), along with smaller aircraft. Due to unsatisfactory soil strength requirements and availability of chemical stabilizing agents, stabilization is to be considered. This facility also will be a short-term field in the forward area and will be considered expedient in design.

- A site reconnaissance and a few soil samples indicate the natural soil strength to be 8 CBR. Sieve analysis and Atterberg limits show 20 percent of the material passing a No. 200 sieve, 300 percent passing a No. 40 sieve, a PI of 28, a LL of 30, and 45 percent of the material being retained on a No. 4 sieve as being reasonably representative of the soil at the proposed site. The soil classifies as GC according to the USCS.

- Determine type of stabilizer needed, percent of stabilizer required (by weight) and design thickness.

PICK AND CLICK TESTS

Specimens covering a wide range of cement contents (for example: 10 percent, 14 percent, and 18 percent) are molded at optimum moisture and maximum density. After at least 1 or 2 days of hardening while kept moist, and after a 3-hour soaking period, the specimens are inspected by picking — done with a pointed instrument such as a dull ice pick or bayonet — and by sharply clicking each specimen against a hard object such as concrete or another sound specimen, to determine their relative hardness when set. If the specimen cannot be penetrated more than 1/8 to 1/4 inch by picking and if it produces a clear or solid tone upon clicking, an adequate cement factor is indicated. When a dull thud or punky sound is obtained, there is inadequate cement even though the specimen may resist picking. The age of the specimens is a factor, and a specimen which 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 that too little cement will not produce a satisfactory stabilization.

WET-DRY AND FREEZE-THAW TESTS

After determining the maximum density and OMC, specimens must be molded for the wet-dry and freeze-thaw tests.

Preparation

These specimens are prepared using the computed OMC and the cement contents described above for the different soil classification. The cement contents are selected in 2 percent increments either side of the median value. Two specimens are molded for each of the three cement contents; one for the wet-dry test, and one for the freeze-thaw test. The same procedure is used to mold the specimens as was used for the OMC determination. Special care must be used to scarify the surfaces between layers to assure a good bond. When the second layer is being placed, a 750-gram sample should be taken for a moisture determination. The molded specimens are placed in a moisture cabinet in an atmosphere of high humidity for 7 days to permit cement hydration before testing.

Procedure, Wet-Dry Test

After the 7 day curing period, the specimens are submerged in tap water at room temperature for a period of 5 hours and removed. Then the specimens are dried in an oven at 71°F (160°F) for 42 hours and removed. The entire surface area is wire brushed to remove all material loosened during wetting and drying. Use two firm strokes on each portion of the surface. These strokes are applied 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. The operation is repeated for a total of 12 cycles. After 12 cycles of test, the specimens are dried to constant weight at 110°C (230°F) and weighed to determine the oven-dry weights.

Procedure, Freeze-Thaw Test

After the curing period, water-saturated felt pads about 1/4 inch thick, blotters, or similar absorptive materials are placed between the specimens and specimen carriers, and the assembly is placed in a freezing cabinet with a constant temperature not warmer than −23°C (−10°F) for 24 hours and removed. The assembly is then allowed to thaw in a moist room or in suitable covered containers with a temperature of 21°C (70°F) and a relative humidity of 100 percent for 23 hours. Free water is made available to the absorbent pads to permit the specimens to absorb water by capillarity during the thawing period. The specimens are next given two firm strokes on all areas with the wire brush to remove material loosened during freezing and thawing. Scale that has formed should be removed with a sharp pointed instrument if necessary. One cycle consists of 24 hours of freezing, 23 hours of thawing, and 1 hour of handling (total 48 hours). After being brushed at the end of each thawing period, the specimens are turned over end-for-end before they are replaced on the water-saturated pads. The test is continued for a total of 12 cycles after which the specimens are dried to constant weight at 110°C (230°F) and weighed to determine their oven-dry weights.

Calculations and Criteria

The results of the wet-dry and freeze-thaw cycles are indicated as soil-cement losses. These losses are computed by using the original dry weights and final corrected dry weights.

- Water-of-hydration correction. The final oven-dry weight of the specimen includes some water used for cement hydration that cannot be driven off at 110°C (230°F). The average amount of this water retained in the specimen is based on the type of soil; gravels ± 1 1/2 percent sand ± 2 1/2 percent, silts ± 3 percent, and clays ± 3 1/2 percent. This correction is computed by the following formula:

$$\text{corrected oven-dry weight} = \frac{\text{measured oven-dry weight of specimen}}{\text{percent water retained} + 100} \times 100$$

Example: Sample composed mostly of sand weighs 3.77 pounds at the end of the test. Water of hydration is 2.5 percent.

$$\text{Corrected oven-dry weight} = \frac{3.77}{2.5 + 100} \times 100 = \frac{3.77}{1.025} = 3.68 \text{ pounds}$$

- Soil-cement loss. The soil-cement loss can now be calculated, as a percentage of the original dry weight, or:

$$\text{Soil-cement loss} = \text{original oven-dry weight} - \frac{\text{final corrected oven-dry weight}}{\text{original oven-dry weight}} \times 100$$

Example: A sample of soil has an original weight of 3.99 pounts.

$$\text{soil-cement loss} = \frac{3.99 - 3.68}{3.99} \times 100 = 7.8 \text{ percent}$$

This value would be reported to the nearest whole number or as 8 percent.

- Weight loss criteria. The minimum cement content recommended for use is the one for which losses of specimen weight during 12 cycles of the wet-dry test or freeze-thaw test conform to the following standards:

GW, GM, GC, SW, SM, SC, and SP soils, not over 14 percent

ML, and MH soils, not over 10 percent

GL, CH, OH, and OL soils, not over 7 percent.

- Strength criteria. The strength of soil-cement specimens tested in compression at various ages should increase with age and with increases in cement. The ranges of cement contents should produce results meeting the requirements above. A sample that has an unconfined compression strength of approximately 300 psi after curing 7 days and shows increasing strength with age can be considered adequately stabilized.

- Cement weight-to-volume conversion. The required cement content by weight must be converted to the equivalent cement content by volume for control during construction, since this is the easier quantity to use in the field. The following formula illustrates the calculation:

$$\text{Volume of cement (percent)} = \frac{D-(D/C)}{94} \times 100$$

where:

D = oven-dry density of soil-cement (pounds per cubic foot)

$$C = \frac{100 + \text{percent cement (by weight)}}{100}$$

94 = weight of 1 cubic foot of cement.

The nomograph in Figure 5-5 makes the conversion without computation. Use a straightedge placed at the soil-cement density and at the percent by weight of cement. Read the percent of cement by volume on the right-hand scale.

MODIFIED MIX DESIGN FOR SANDY SOILS

Sandy soils are generally the most readily and economically stabilized because they require the least amount of cement for adequate hardening and because they contain a minimum amount of material which prevents intimate mixing of soil and cement. The following shortcut testing procedures for sandy soils will not always indicate the minimum cement contents required, but the results will be close enough to be on the safe side and economical. If time permits, the procedures for freeze-thaw test are followed for greater economy of design.

Two procedures are used; one for soils not containing material retained on the No. 4 sieve, and the other for soils containing material retained on the No. 4 sieve. The procedures can

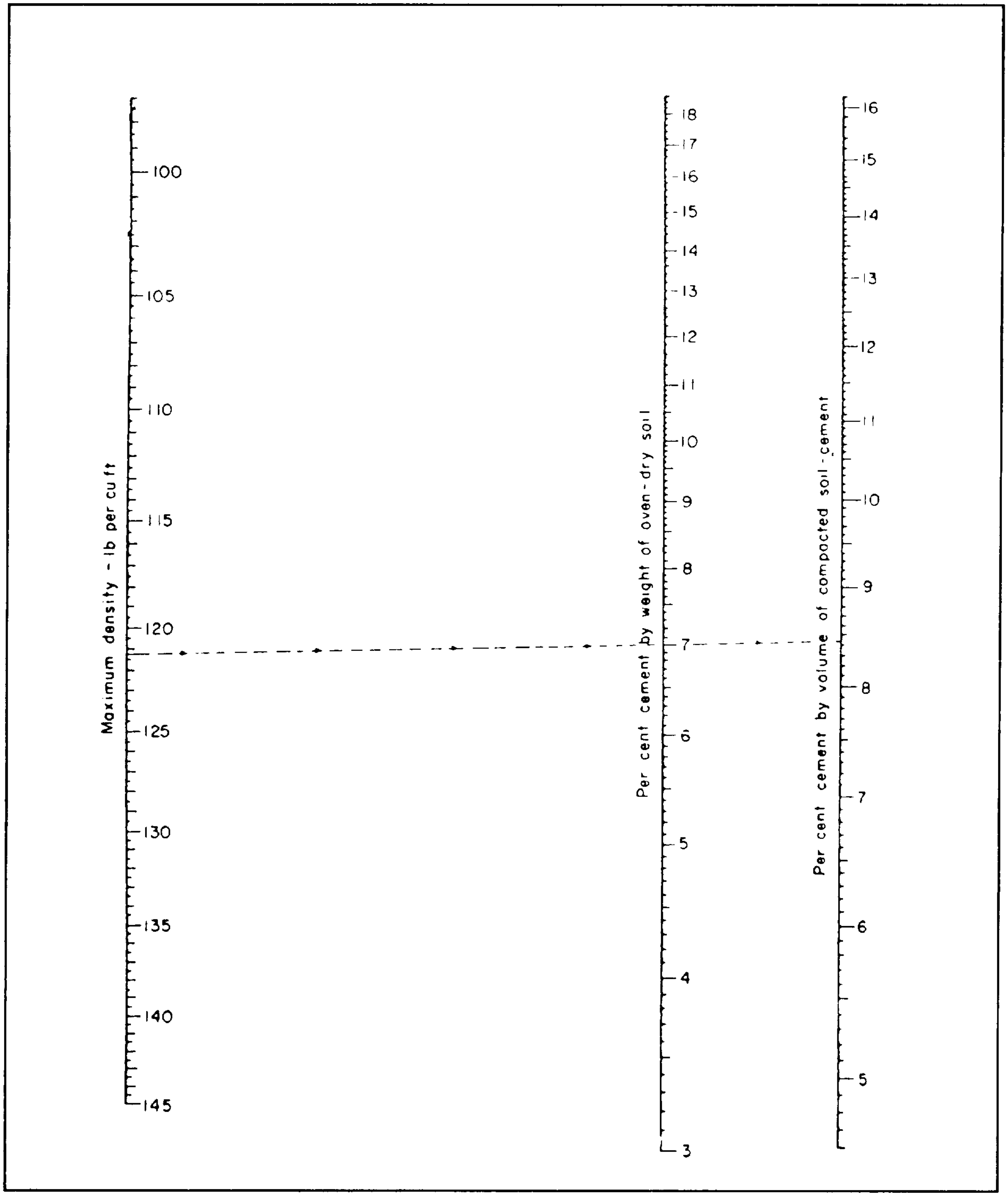


Figure 5-5. Relation of cement content by weight to cement content by volume

be used only with soils containing less than 50 percent of material smaller than 0.05 millimeter (silt and clay) and less than 20 percent smaller than 0.005 millimeter (clay). Dark gray to black sandy soils obviously containing appreciable organic impurities together with miscellaneous granular materials (such as cinders, caliche, chat, chart, marl, red dog, scoria, shale, and slag) should be tested using the full procedures and not tested by the modified methods for sandy soils. When coarse-grained or sandy soils (generally of groups GW, GP, GM, SW or SM) are encountered, they may be classified for testing purposes using either the first or the second procedure. There is one other exception. Granular soils with materials retained on the No. 4 sieve whose bulk specific gravity is less than 2.45 cannot be tested.

Procedures Sequence

- Determine the soil gradation.
- Determine the bulk specific gravity of the material retained on the No. 4 sieve.
- Perform the moisture-density test of an estimated soil-cement mixture.
- Locate the indicated cement requirements from the charts.
- Perform compressive-strength tests to verify the cement requirement.

Procedure for Soils with no Material Retained on the No. 4 Sieve

- Determine the maximum density and OMC for a mixture of soil and cement. (Figure 5-6 will give an estimated density. This value and the percentage of material smaller than 0.05 mm are used with Figure 5-7, page 5-28, to determine an indicated cement content).
- Use the maximum density value and Figure 5-7, page 5-28, to determine an indicated cement requirement.
- Mold three compressive-strength specimens at maximum density and OMC.
- Moist-cure the specimens for 7 days and test for strength.
- Plot the value of the averaged compressive strength on Figure 5-8, page 5-28. If this plot is above the curve, the cement factor is probably too low and needs adjusting. Two new test specimens are prepared; one at the cement content as computed above, and the second with a 2 percent higher cement content. The full freeze-thaw test is performed on these two specimens.

Procedure for Soils with Material Retained on the No. 4 Sieve

- Determine the maximum density and OMC for a mixture of soil and cement. Use Figure 5-9, page 5-29, for an estimated maximum density, and Figure 5-10, page 5-30, with this density, percentage of material retained on the No. 4 sieve, and percentage smaller than 0.05 mm to determine moisture content. The 45 percent maximum retained on No. 4 sieve still applies. Also any material larger than ¼ inch must be replaced with an equivalent weight of the material passing the ¼-inch sieve and retained on the No. 4 sieve.
- Using the maximum density from above and Figure 5-10, page 5-30, determine the indicated cement requirement.
- Mold test specimens at maximum density and OMC.
- Moist-cure for 7 days and test for compressive and average.
- Use Figure 5-11, page 5-31, to determine the allowable compressive strength for the

soil-cement mixture. Connect 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, the cement content is adequate. If the strength is too low, the cement factor is also too low, and a full test should be performed.

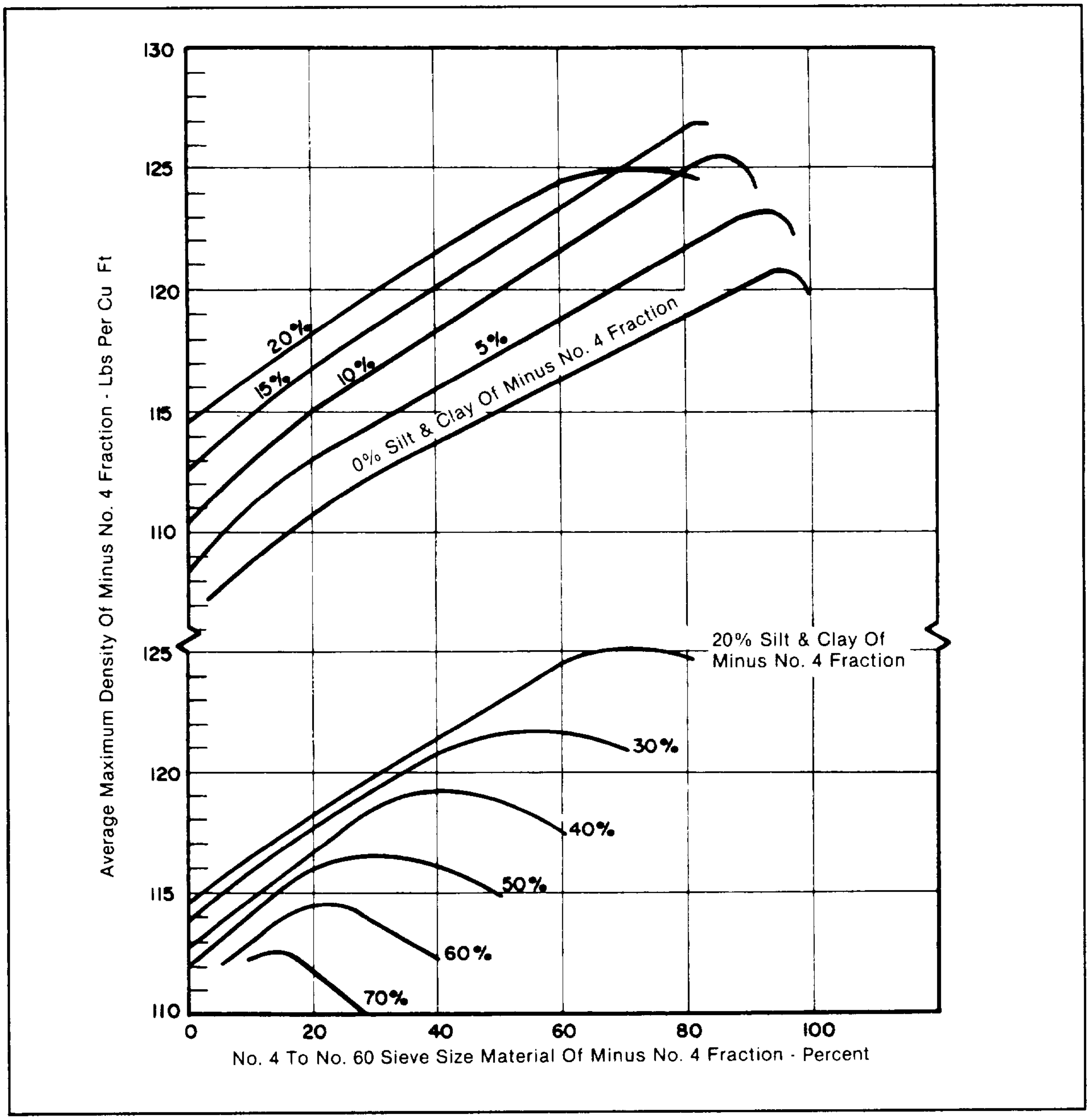


Figure 5-6. Average maximum densities of the minus No. 4 fraction of soil-cement mixtures

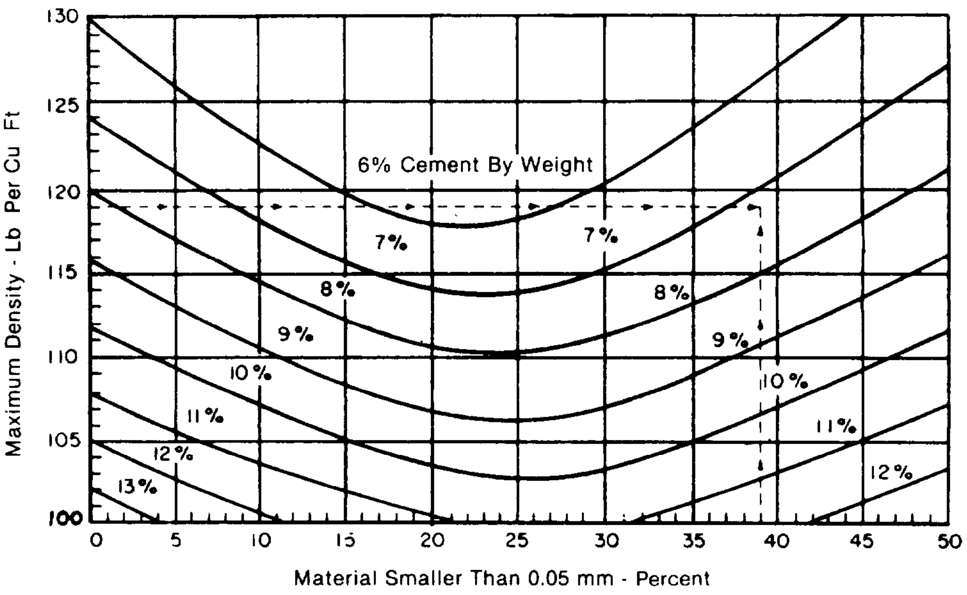


Figure 5-7. Indicated cement contents of soil-cement mixtures not containing material retained on the No. 4 sieve

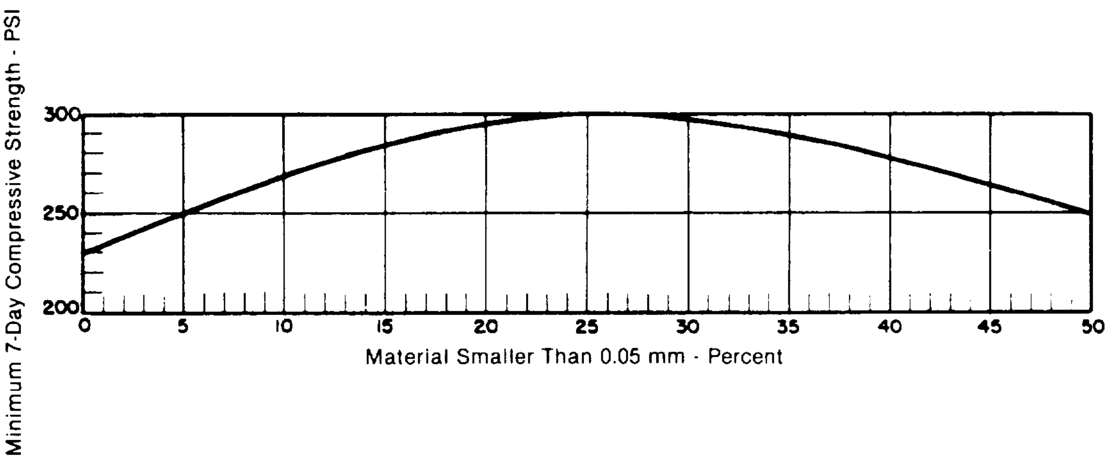


Figure 5-8. Minimum 7-day compressive strengths required for soil-cement not containing material retained on the No. 4 sieve

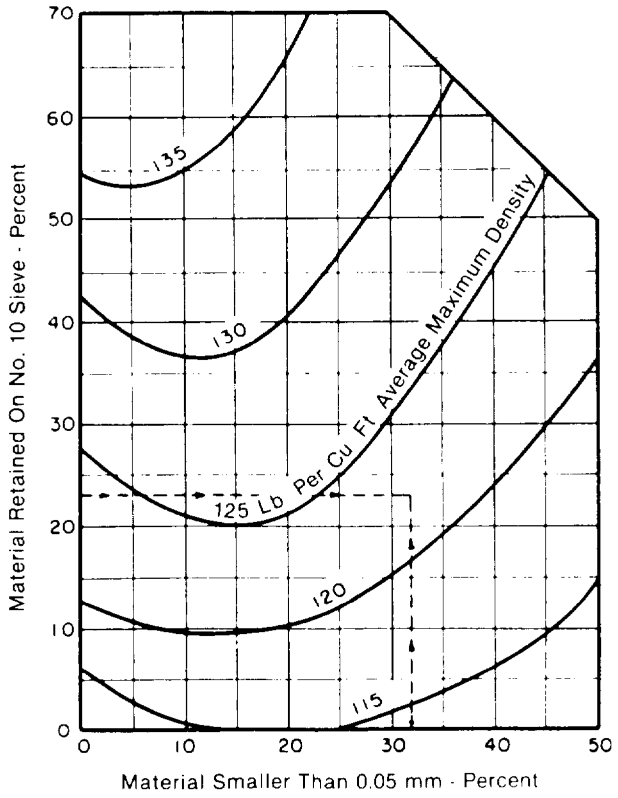


Figure 5-9. Average maximum densities of soil-cement mixtures containing material retained on No. 4 sieve

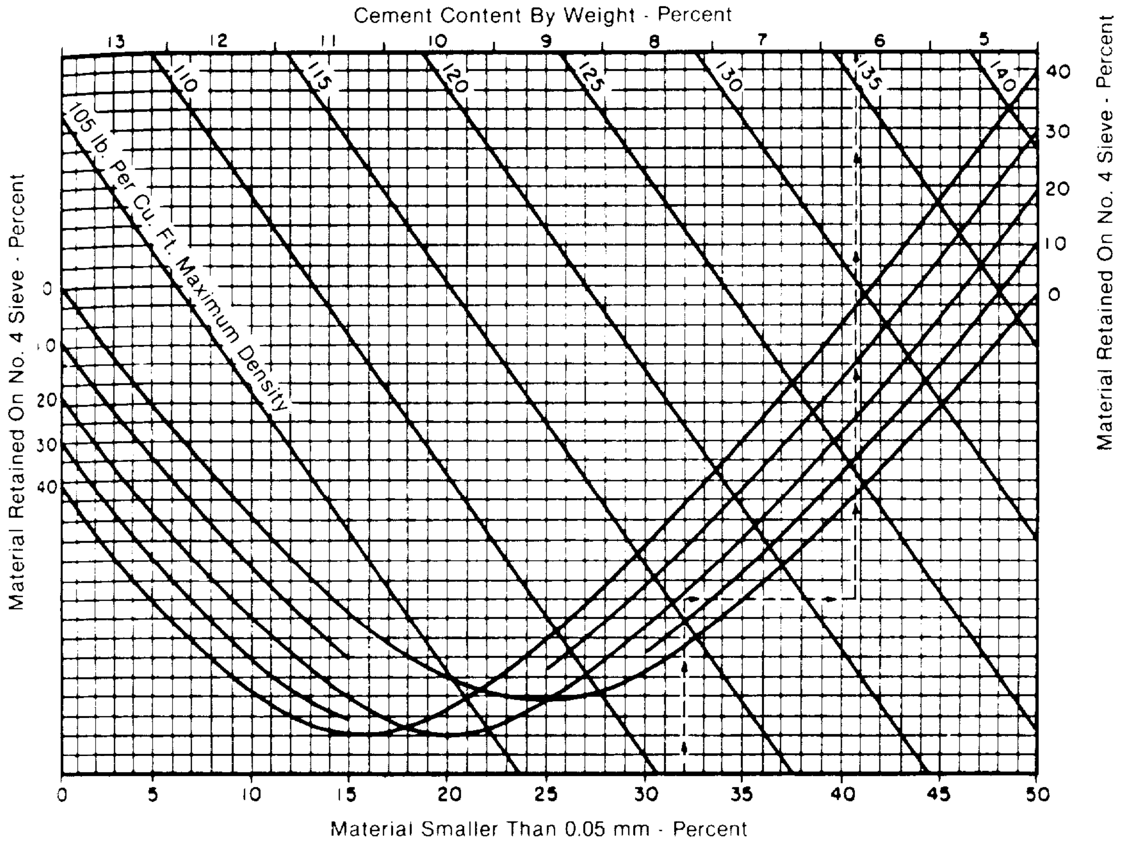


Figure 5-10. Indicated cement contents of soil-cement mixtures containing material retained on No. 4 sieve

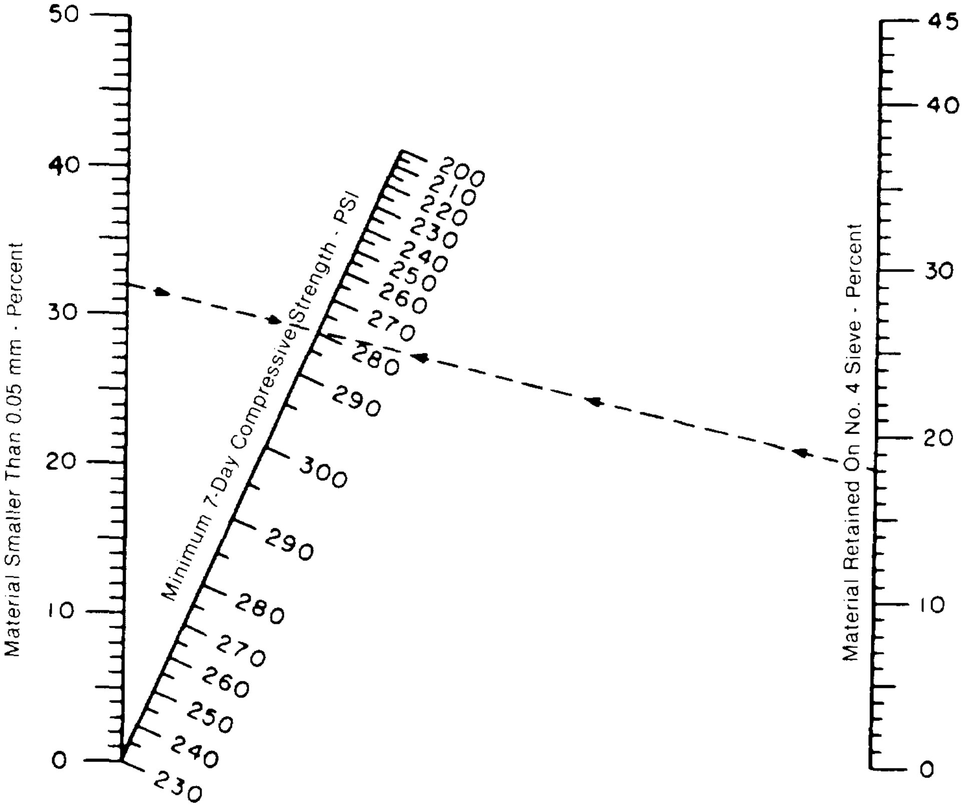


Figure 5-11. Minimum 7-day compressive strengths required for soil-cement mixtures containing material retained on No. 4 sieve

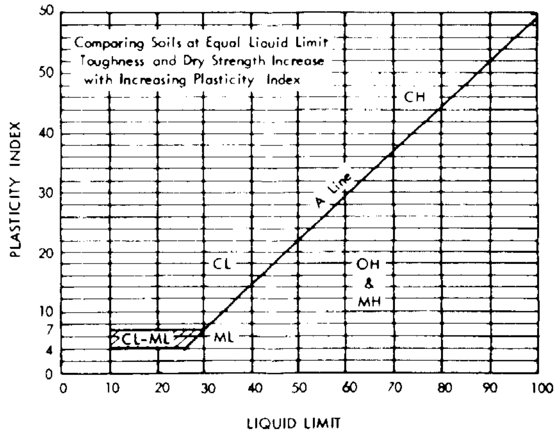
APPENDIX A

UNIFIED SOIL CLASSIFICATION SYSTEM

The Unified Soil Classification System is a method of describing the different soils based on the soils' physical characteristics. The system was adopted by the US Army Corps of Engineers (and the Department of Interior, Bureau of Reclamation) in January 1952. The three tables, A-1, A-2, and A-3, are included to aid in identifying the soils from the different test results and to illustrate the engineering characteristics pertinent to roads and airfields and to embankments and foundations. The tables list some of the limiting values associated with the tests described in this manual. The values should guide the technician in testing. A great discrepancy between a test result and the values given in the tables for that particular soil type should alert the technician to the possibility of some inaccuracy or error either in the testing or in the identification of the soil. Figure A-1 illustrates the process of identifying soils through the tests.

Table A-1. Unified Soil Classification System

UNIFIED SOIL CLASSIFICATION
(Including Identification and Description)

Major Divisions			Group Symbols	Typical Names	Field Identification Procedures (Excluding particles larger than 3 in. and basing fractions on estimated weights.)			Information Required for Describing Soil	Laboratory Classification Criteria			
1	2		3	4	5			6	7			
Coarse-grained Soils More than half of material is larger than No. 200 sieve size. The No. 200 sieve size is about the smallest particle visible to the naked eye.	Gravels More than half of coarse fraction is larger than No. 4 sieve size (For visual classification, the 1/4-in. size may be used as equivalent to the No. 4 sieve size)	Clean Gravels (Little or no fines)	GW	Well-graded gravels, gravel-sand mixtures, little or no fines.	Wide range in grain sizes and substantial amounts of all intermediate particle sizes.			For undisturbed soils add information on stratification, degree of compactness, cementation, moisture conditions, and drainage characteristics. Give typical name; indicate approximate percentage of sand and gravel, maximum size, angularity, surface condition, and hardness of the coarse grains; local or geologic name and other pertinent descriptive information; and symbol in parentheses. Example: Silty sand, gravelly; about 20% hard, angular gravel particles 1/2-in. maximum size; rounded and subangular sand grains, coarse to fine; about 15% nonplastic fines with low dry strength; well compacted and moist in place, alluvial sand; (SM)	Use grain-size curve in identifying the Fractions as given under field identification.	Determine percentages of gravel and sand from grain-size curve. Depending on percentage of fines (fraction smaller than No. 200 sieve size) coarse-grained soils are classified as follows: Less than 5% GW, GP, SW, SP, More than 12% GM, GC, SM, SC, 5% to 12% Borderline cases requiring use of dual symbols.	$C_u = \frac{D_{60}}{D_{10}}$ Greater than 4 $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3	
			GP	Poorly graded gravels or gravel-sand mixtures, little or no fines.	Predominantly one size or a range of sizes with some intermediate sizes missing.						Not meeting all graduation requirements for GW	
		Gravels with Fines (Appreciable amount of fines)	GM	Silty gravels, gravel-sand-silt mixture.	Nonplastic fines or fines with low plasticity (for identification procedures see ML below)						Atterberg limits below "A" line or PI less than 4	Above "A" line with PI between 4 and 7 are borderline cases requiring use of dual symbols.
			GC	Clayey gravels, gravel-sand-clay mixtures.	Plastic fines (for identification procedures see CL below.)						Atterberg limits above "A" line with PI greater than 7	
	Sands More than half of coarse fraction is smaller than No. 4 sieve size	Clean sands (Little or no fines)	SW	Well-graded sands, gravelly sands, little or no fines	Wide range in grain size and substantial amounts of all intermediate particle sizes.						$C_u = \frac{D_{60}}{D_{10}}$ Greater than 6 $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3	
			SP	Poorly graded sands or gravelly sands, little or no fines	Predominantly one size or a range of sizes with some intermediate sizes missing.						Not meeting all graduation requirements for SW	
		Sands with Fines (Appreciable amount of fines)	SM	Silty sands, sand-silt mixtures.	Nonplastic fines or fines with low plasticity (for identification procedures see ML below)						Atterberg limits below "A" line or PI less than 4	Limits plotting in hatched zone with PI between 4 and 7 are borderline cases requiring use of dual symbols.
			SC	Clayey sands, sand-clay mixtures.	Plastic fines (for identification procedures see CL below.)						Atterberg limits above "A" line with PI greater than 7	
Fine-grained Soils More than half of material is smaller than No. 200 sieve size.					Identification Procedures on Fraction Smaller than No. 40 Sieve Size			For undisturbed soils add information on structure stratification, consistency in undisturbed and remolded states, moisture and drainage conditions. Give typical name; indicate degree and character of plasticity; amount and maximum size of coarse grains; color in wet condition; odor, if any; local or geologic name and other pertinent descriptive information; and symbol in parentheses. Example: Clayey silt, brown; slightly plastic; small percentage of fine sand; numerous vertical root holes; firm and dry in place; loess; (ML).		 PLASTICITY CHART For laboratory classification of fine-grained soils		
					Dry Strength (Crushing characteristics)	Dilatancy (Reaction to shaking)	Toughness (Consistency near PL)					
	Silts and Clays Liquid limit is less than 50		ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity.	None to slight	Quick to slow	None					
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.	Medium to high	None to very slow	Medium					
			OL	Organic silts and organic silts clays of low plasticity.	Slight to Medium	Slow	Slight					
	Silts and Clays Liquid limit is greater than 50		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.	Slight to medium	Slow to none	Slight to medium					
			CH	Inorganic clays of high plasticity, fat clays	High to very high	None	High					
			OH	Organic clays of medium to high plasticity, organic silts.	Medium to high	None to very slow	Slight to mdeium					
Highly Organic Soils			Pt	Peat and other highly organic soils.	Readily identified by color, odor, spongy feel and frequently by fibrous texture.							

(1) Boundary classification: Soils possessing characteristics of two groups are designated by combinations of group symbols. For examples GW-GC, well-graded gravel-sand mixture with clay binder. (2) All sieve sizes on this chart are U.S. standard.

FIELD IDENTIFICATION PROCEDURES FOR FINE-GRAINED SOILS OR FRACTIONS

These procedures are to be performed on the minus No. 40 sieve size particles, approximately 1/64 in. For field classification purposes, screening is not intended, simply remove by hand the coarse particles that interfere with the tests.

Dilatancy (reaction to shaking)

After removing particles larger than No. 40 sieve size, prepare a pat of moist soil with a volume of about one-half cubic inch. Add enough water if necessary to make the soil soft but not sticky.

Place the pat in the open palm of one hand and shake horizontally, striking vigorously against the other hand several times. A positive reaction consists of the appearance of water on the surface of the pat which changes to a livery consistency and becomes glossy. When the sample is squeezed between the fingers, the water and gloss disappear from the surface, the pat stiffens, and finally it cracks or crumbles. The rapidity of appearance of water during shaking and of its disappearance during squeezing assist in identifying the character of the fines in a soil.

Very fine clean sands give the quickest and most distinct reaction whereas a plastic clay has no reaction. Inorganic silts, such as a typical rock flour, show a moderately quick reaction.

Dry Strength (crushing characteristics)

After removing particles larger than No. 40 sieve size, mold a pat of soil to the consistency of putty, adding water if necessary. Allow the pat to dry completely by oven, sun, or air-drying, and then test its strength by breaking and crumbling between the fingers. This strength is a measure of the character and quantity of the colloidal fraction contained in the soil. The dry strength increases with increasing plasticity.

High dry strength is characteristic for clays of the CH group. A typical inorganic silt posses only very slight dry strength. Silty fine sands and silts have about the same slight dry strength, but can be distinguished by the feel when powdering the dried specimen. Fine sand feels gritty whereas a typical silt has the smooth feel of flour.

Toughness (consistency near plastic limit)

After particles larger than the No. 40 sieve size are removed, a specimen of soil about one-half inch in cube size, is molded to the consistency of putty. If too dry, water must be added and if sticky, the specimen should be spread out in a thin layer and allowed to lose some moisture by evaporation. Then the specimen is rolled out by hand on a smooth surface or between the palms into a thread about one-eighth inch in diameter. The thread is then folded and rerolled repeatedly. During this manipulation the moisture content is gradually reduced and the specimen stiffens, finally loses its plasticity, and crumbles when the plastic limit is reached.

After the thread crumbles, the pieces should be lumped together and a slight kneading action continued until the lump crumbles.

The tougher the thread near the plastic limit and the stiffer the lump when it finally crumbles, the more potent is the colloidal clay fraction in the soil. Weakness of the thread at the plastic limit and quick loss of coherence of the lump below the plastic limit indicate either inorganic clay of low plasticity, or materials such as kaolin-type clays and organic clays which occur below the A-line.

Highly organic clays have a very weak and spongy feel at the plastic limit.

Table A-2. Characteristics pertinent to roads and airfields

Major Divisions (1)	(2)	Letter (3)	Symbol Hatching (4)	Symbol Color (5)	Name (6)	Value as Foundation When Not Subject to Frost Action (7)	Value as Base Directly under Bituminous Pavement (8)	Potential Frost Action (9)	Compressibility and Expansion (10)	Drainage Characteristics (11)	Compaction Equipment (12)	Unit Dry Weight Lb Per Cu Ft (13)	Field CBR (14)	Subgrade Modulus k_5 Lb Per Cu In. (15)
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	GW		Red	Well-graded gravels or gravel-sand mixtures, little or no fines	Excellent	Good	None to very slight	Almost none	Excellent	Crawler-type tractor, rubber-tired equipment, steel-wheeled roller	125-140	60-80	300 or more
		GP			Poorly graded gravels or gravel-sand mixtures, little or no fines	Good to excellent	Poor to fair	None to very slight	Almost none	Excellent	Crawler-type tractor, rubber-tired equipment, steel-wheeled roller	110-130	25-60	300 or more
		GM d		Yellow	Silty gravels, gravel-sand-silt mixtures	Good to excellent	Fair to good	Slight to medium	Very slight	Fair to poor	Rubber-tired equipment, sheepsfoot roller; close control of moisture	130-145	40-80	300 or more
		u				Good	Poor	Slight to medium	Slight	Poor to practically impervious	Rubber-tired equipment, sheepsfoot roller	120-140	20-40	200 to 300
		GC			Clayey gravels, gravel-sand-clay mixtures	Good	Poor	Slight to medium	Slight	Poor to practically impervious	Rubber-tired equipment, sheepsfoot roller	120-140	20-40	200-to 300
	SAND AND SANDY SOILS	SW		Red	Well-graded sands or gravelly sands, little or no fines	Good	Poor	None to very slight	Almost none	Excellent	Crawler-type tractor, rubber-tired equipment	110-130	20-40	200 to 300
		SP			Poorly graded sands or gravelly sands, little or no fines	Fair to good	Poor to not suitable	None to very slight	Almost none	Excellent	Crawler-type tractor, rubber-tired equipment	100-120	10-25	200 to 300
		SM d		Yellow	Silty sands, sand-silt mixtures	Good	Poor	Slight to high	Very slight	Fair to poor	Rubber-tired equipment sheepsfoot roller; close control of moisture	120-135	20-40	200 to 300
		u				Fair to good	Not suitable	Slight to high	Slight to medium	Poor to practically impervious	Rubber-tired equipment, sheepsfoot roller	105-130	10-20	200 to 300
		SC			Clayey sands, sand-clay mixtures	Fair to good	Not suitable	Slight to high	Slight to medium	Poor to practically impervious	Rubber-tired equipment, sheepsfoot roller	105-130	10-20	200 to 300
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	Fair to poor	Not suitable	Medium to very high	Slight to medium	Fair to poor	Rubber-tired equipment, sheepsfoot roller; close control of moisture	100-125	5-15	100 to 200
		CL			Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays	Fair to poor	Not suitable	Medium to high	Medium	Practically impervious	Rubber-tired equipment, sheepsfoot roller	100-125	5-15	100 to 200
		OL			Organic silts and organic silt-clays of low plasticity	Poor	Not suitable	Medium to high	Medium to high	Poor	Rubber-tired equipment, sheepsfoot roller	90-105	4-8	100 to 200
	SILTS AND CLAYS LL > 50	MH		Blue	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts	Poor	Not suitable	Medium to very high	High	Fair to poor	Sheepsfoot roller	80-100	4-8	100 to 200
		CH			Inorganic clays of high plasticity, fat clays	Poor to very poor	Not suitable	Medium	High	Practically impervious	Sheepsfoot roller	90-110	3-5	50 to 100
		OH			Organic clays of medium to high plasticity, organic silts	Poor to very poor	Not suitable	Medium	High	Practically impervious	Sheepsfoot roller	80-105	3-5	50 to 100
HIGHLY ORGANIC SOILS		Pt		Orange	Peat and other highly organic soils	Not suitable	Not suitable	Slight	Very high	Fair to poor	Compaction not practical	-	-	-

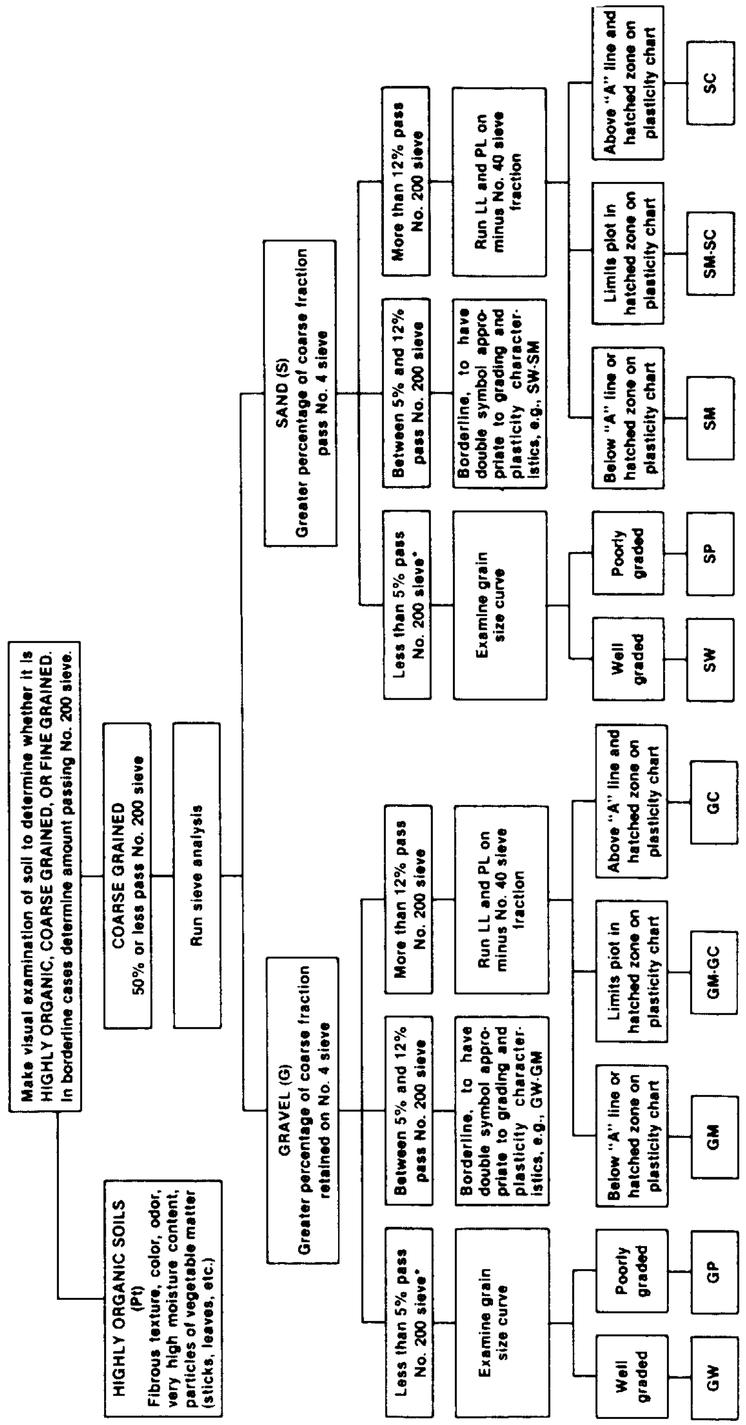
Notes:

1. Column 3, Division of GM, and SM groups into subdivisions of d and u are for roads and airfields only; subdivision is on basis of Atterberg limits; suffix d (e. g., GMd) will be used when the liquid limit is 28 or less and the plasticity index is 6 or less; the suffix u will be used when the liquid limit is greater than 28.
2. Column 7, values are for subgrades and base courses except for base course directly under bituminous pavement.
3. In column 8, the term "excellent" has been reserved for base materials consisting of high quality processed crushed stone.
4. In column 9, these soils are susceptible to frost as indicated under conditions favorable to frost action described in the text.
5. In column 12, the equipment listed will usually produce the required densities with a reasonable number of passes when moisture conditions and thickness of lift are properly controlled. In some instances, several types of equipment are listed, because variable soil characteristics within a given soil group may require different equipment. In some instances, a combination of two types may be necessary.
   a. Processed base materials and other angular materials. Steel-wheeled rollers are recommended for hard angular materials with limited fines or screenings. Rubber-tired equipment is recommended for softer materials subject to degradation.
   b. Finishing. Rubber-tired equipment is recommended for rolling during final shaping operations for most soils and processed materials.
   c. Equipment size. The following sizes of equipment are necessary to assure the high densities required for airfield construction:
      Crawler-type tractor -- total weight in excess of 30,000 lb.
      Rubber-tired equipment -- wheel load in excess of 15,000 lb, wheel loads as high as 40,000 lb may be necessary to obtain the required densities for some materials (based on contact pressure of approximately 65 to 150 psi).
      Sheepsfoot roller -- unit pressure (on 6- to 12-sq-in. 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 per cent of the total peripheral area of the drum, using the diameter measured to the faces of the feet.
6. **Column 13, unit dry weights are for compacted soil at optimum moisture content for modified AASHO compactive effort.**

Table A-3. Characteristics pertinent to embankments and foundations

Major Divisions (1)	(2)	Letter (3)	Symbol		Name (6)	Value for Embankments (7)	Permeability Cm Per Sec (8)	Compaction Characteristics (9)	Std AASHO Max Unit Dry Weight Lb Per Cu Ft (10)	Value for Foundations (11)	Requirements for Seepage Control (12)
			Hatching (4)	Color (5)							
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	$k > 10^{-2}$	Good, tractor, rubber-tired, steel-wheeled roller	125-135	Good bearing value	Positive cutoff
		GP	●		Poorly-graded gravels or gravel-sand mixtures, little or no fines	Reasonably stable, pervious shells of dikes and dams	$k > 10^{-2}$	Good, tractor, rubber-tired, steel-wheeled roller	115-125	Good bearing value	Positive cutoff
		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}	Good, with close control, rubber-tired, sheepsfoot roller	120-135	Good bearing value	Toe trench to none
		GC	▨		Clayey gravels, gravel-sand-clay mixtures	Fairly stable, may be used for impervious core	$k = 10^{-6}$ to 10^{-8}	Fair, rubber-tired, sheepsfoot roller	115-130	Good bearing value	None
	SAND AND SANDY SOILS	SW	⁘	Red	Well-graded sands or gravelly sands, little or no fines	Very stable, pervious sections, slope protection required	$k > 10^{-3}$	Good, tractor	110-130	Good bearing value	Upstream blanket and toe drainage or wells
		SP	⁘		Poorly-graded sands or gravelly sands, little or no fines	Reasonably stable, may be used in dike section with flat slopes	$k > 10^{-3}$	Good, tractor	100-120	Good to poor bearing value depending on density	Upstream blanket and toe drainage or wells
		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}	Good, with close control, rubber-tired, sheepsfoot roller	110-125	Good to poor bearing value depending on density	Upstream blanket and toe drainage or wells
		SC	▨		Clayey sands, sand-silt mixtures	Fairly stable, use for impervious core for flood control structures	$k = 10^{-6}$ to 10^{-8}	Fair, sheepsfoot roller, rubber tired	105-125	Good to poor bearing value	None
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}	Good to poor, close control essential, rubber-tired roller, sheepsfoot roller	95-120	Very poor, susceptible to liquefaction	Toe trench to none
		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}	Fair to good, sheepsfoot roller, rubber tired	95-120	Good to poor bearing	None
		OL	▦		Organic silts and organic silt-clays of low plasticity	Not suitable for embankments	$k = 10^{-4}$ to 10^{-6}	Fair to poor, sheepsfoot roller	80-100	Fair to poor bearing, may have excessive settlements	None
	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}	Poor to very poor, sheepsfoot roller	70-95	Poor bearing	None
		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}	Fair to poor, sheepsfoot roller	75-105	Fair to poor bearing	None
		OH	▨		Organic clays of medium to high plasticity, organic silts	Not suitable for embankments	$k = 10^{-6}$ to 10^{-8}	Poor to very poor, sheepsfoot roller	65-100	Very poor bearing	None
HIGHLY ORGANIC SOILS		Pt	▤	Orange	Peat and other highly organic soils	Not used for construction		Compaction not practical		Remove from foundations	

Notes:
1. Values in columns 7 and 11 are for guidance only. Design should be based on test results.
2. In column 9, the equipment listed will usually produce the desired densities with a reasonable number of passes when moisture conditions and thickness of lift are properly controlled.
3. Column 10, unit dry weights are for compacted soil at optimum moisture content for Standard AASHO (Standard Proctor) compactive effort.


Make visual examination of soil to determine whether it is HIGHLY ORGANIC, COARSE GRAINED, OR FINE GRAINED. In borderline cases determine amount passing No. 200 sieve.
HIGHLY ORGANIC SOILS (Pt) Fibrous texture, color, odor, very high moisture content, particles of vegetable matter (sticks, leaves, etc.)
COARSE GRAINED 50% or less pass No. 200 sieve
Run sieve analysis
GRAVEL (G) Greater percentage of coarse fraction retained on No. 4 sieve
SAND (S) Greater percentage of coarse fraction pass No. 4 sieve
Less than 5% pass No. 200 sieve*
Between 5% and 12% pass No. 200 sieve
More than 12% pass No. 200 sieve
Examine grain size curve
Borderline, to have double symbol appropriate to grading and plasticity characteristics, e.g., GW-GM
Borderline, to have double symbol appropriate to grading and plasticity characteristics, e.g., SW-SM
Run LL and PL on minus No. 40 sieve fraction
Well graded
Poorly graded
Below "A" line or hatched zone on plasticity chart
Limits plot in hatched zone on plasticity chart
Above "A" line and hatched zone on plasticity chart
GW
GP
GM
GM-GC
GC
SW
SP
SM
SM-SC
SC

Note: Sieve sizes are U.S. Standard.

* If fines interfere with free draining properties use double symbol such as GW-GM.

Figure A-1. *Identification of soils under soil classification system*

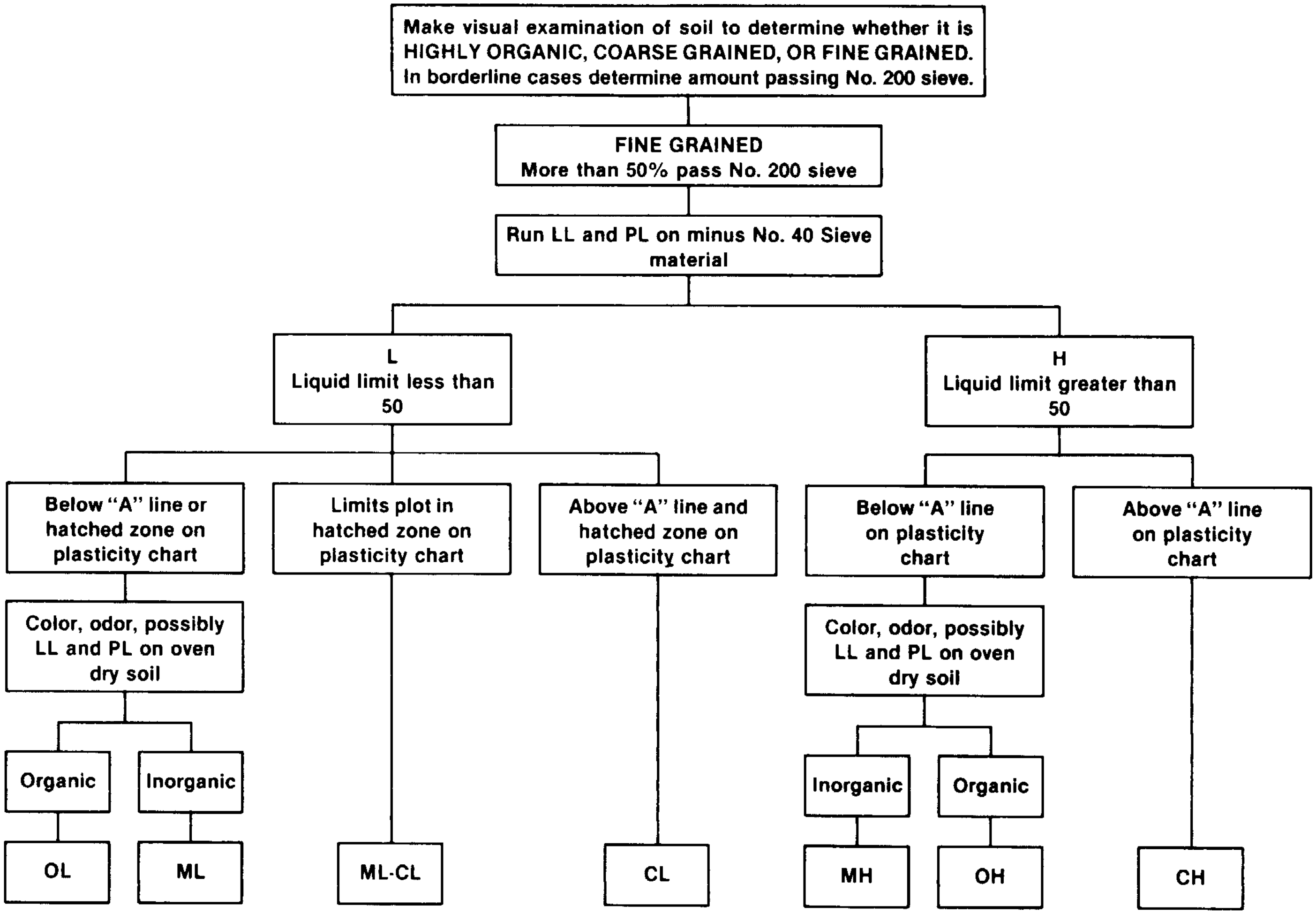

Make visual examination of soil to determine whether it is HIGHLY ORGANIC, COARSE GRAINED, OR FINE GRAINED. In borderline cases determine amount passing No. 200 sieve.
FINE GRAINED More than 50% pass No. 200 sieve
Run LL and PL on minus No. 40 Sieve material
L Liquid limit less than 50
H Liquid limit greater than 50
Below "A" line or hatched zone on plasticity chart
Limits plot in hatched zone on plasticity chart
Above "A" line and hatched zone on plasticity chart
Below "A" line on plasticity chart
Above "A" line on plasticity chart
Color, odor, possibly LL and PL on oven dry soil
Color, odor, possibly LL and PL on oven dry soil
Organic
Inorganic
Inorganic
Organic
OL
ML
ML-CL
CL
MH
OH
CH

Figure A-1. Identification of soils under soil classification system (continued)

APPENDIX B

USEFUL INFORMATION

1 inch = 2.54 centimeters = 25.4 millimeters

12 inches = 1 foot = 0.3048 meter

1 square foot = 144 square inches

1 square yard = 9 square feet = 1,296 square inches

1 acre = 43,560 square feet

1 acre, if square = approximately 208.7 feet on each side

1 cubic foot = 1,728 cubic inches = 7.48 United States gallons

1 cubic yard = 27 cubic feet = 0.7646 cubic meter

1 cubic meter = 1.308 cubic yards = 35.31 cubic feet = 1,000 liters

1 United States gallon = 231 cubic inches = 0.1337 cubic foot = 4 quarts = 8 pints

1 United States gallon of water weighs 8.33 pounds

1 British Imperial gallon = 1.2 United States gallons

1 gross or long ton = 2,240 pounds = 1.12 net tons

1 net or short ton = 2,000 pounds

1 pound = 16 ounces = 256 drams = 7,000 grains = 453.6 grams

1 kilogram = 1,000 grams

1 atmosphere = 14.7 pounds per square inch = 30 inches mercury = 33.95 feet of water at 62°F

1 pound per square inch = 144 pounds per square foot = 0.068 atmosphere = 2.042 inches mercury = 2.1 feet of water at 62°F

1 cubic foot of water at 62°F weighs 62.36 pounds

Water freezes at 32°F or at 0°C

Water boils at 212°F or at 100°C (sea level and 14.7 pounds per square inch pressure)

°F = 9/5(°C) + 32

Circumference of a circle = diameter x 3.14

Area of a circle = diameter squared by 0.7854

1 foot-pound = unit of work or the work required to lift 1 pound through a vertical distance of 1 foot

Relation between API (air position indicator) degrees and specific gravity (liquids - lighter than water)*

API Gravity Degrees	Specific Gravity, 60°/60°
10	1.0000
15	0.9655
29	0.9333
25	0.9032
30	0.8750
35	0.8485
40	0.8235
45	0.8000

$$\text{*API Gravity Degrees} = \frac{141.5}{\text{specific gravity } 60^\circ/60^\circ} - 131.5$$

Multiply	By	To obtain
Atmospheres	14.70	Pounds per square inch
Centimeters	0.3937	Inches
Cubic feet	7.481	United States gallons
Cubic feet	1,728	Cubic inches
Cubic meters	35.51	Cubic feet
Cubic yards	27	Cubic feet
Feet	0.3048	Meters
Feet	30.48	Centimeters
Gallons	231	Cubic inches
Gallons	0.134	Cubic feet
Inches	2.54	Centimeters
Kilograms	1,000	Grams
Kilograms	2.205	Pounds
Liters	0.03531	Cubic feet
Liters	0.2642	Gallons
Liters	1,000	Cubic centimeters
Miles	5,280	Feet
Pounds	453.6	Grams

APPENDIX C

METRIC CONVERSION TABLE

1 centimeter = 0.03821 foot

1 centimeter = 0.3937 inch

1 cubic centimeter = 0.06102 cubic inch

1 inch = 2.540 centimeters

1 cubic inch = 16.39 cubic centimeters

1 gallon = 3,785 cubic centimeters

1 gallon = 3.785 liters

1 liter = 61.02 cubic inches

1 liter = 0.2642 gallon

1 kilogram = 2.205 pounds

1 pound = 453.6 grams

APPENDIX D

TEST PROCEDURES FOR SOIL-CEMENT STABILIZATION

STEPS

The steps, which are described in more detail in the following paragraphs, are suggested.

- Determine the maximum density and optimum moisture content for the soil-cement mixture.

- Mold specimens for inspection of hardness.

- Inspect specimens using "pick and click" procedures.

- Perform unconfined compression test on best soil-cement mixture.

MOISTURE-DENSITY TEST

The maximum density and optimum moisture content are determined at an estimated cement content using the compaction test procedure described in Chapter 2, Section IX, for the 4-inch diameter mold.

MOLDING SPECIMENS

Specimens for inspection of hardness are molded at the optimum water content determined in the moisture-density test using the same procedure described in Chapter 2. It is best if these specimens can be molded in the standard Proctor mold and then be removed from the mold and placed in a high humidity atmosphere for hydration or curing.

INSPECTING SPECIMENS

After at least 2 days of hardening (with the specimens being kept moist) and after a 4-hour soaking, the samples are inspected by picking with a sharp-pointed instrument and by sharply clicking each specimen against a hard object such as concrete to determine relative hardness.

PICK TEST

In the pick test, the specimen is held in one hand, and a relatively sharp-pointed instrument, such as a dull ice pick, is lightly jabbed into the specimen (or the end of a specimen molded in a can) from a distance of 2 or 3 inches. If the specimen resists this light picking, the force of impact is increased until the pick is striking the specimen with considerable force. Specimens that are hardening satisfactorily will definitely resist the penetration of the pick, whereas specimens that are not hardening properly will offer little resistance. To pass the pick test, a specimen that is not over 7 days old and that has been soaked in water must resist the penetration of the ice pick, which is under considerable force, to a distance greater than 1/8 to 1/4 inch.

CLICK TEST

- Application to Specimens Passing the Pick Test. The click test is then applied to water-soaked specimens that are apparently hardening satisfactorily and that have passed the pick test. In the click test, the specimens, one in each hand, are held perpendicular to each other and 4 inches apart. They are then lightly clicked together a number of times. The force of impact is increased with each click. Specimens that are hardening satisfactorily will click together with a ringing or solid tone. As the force of impact is increased, one of the specimens may break transversely even though it is hardening adequately. The internal portion of a satisfactory specimen should then pass the pick test. After two or three hard specimens are obtained, they may be saved, and one may be used in the click test with a soil-cement specimen of a soil being tested.

- Sound and Breakage. When a poorly hardened specimen is clicked with a satisfactory specimen, a dull thud is obtained rather than the solid sound obtained with two satisfactory specimens. After the first or second click, the inferior specimen will generally break, and its internal portion will not pass the pick test.

- Age. At the time the click test is conducted, the age of the specimens must be taken into account. For instance, specimens that are not properly hardened at an age of 4 days may be satisfactorily hardened at an age of 7 days.

- Repeat Procedure. The pick and click procedures are then repeated after the specimens have been dried out and again after a second soaking to test their relative hardness at both extremes of moisture content.

- Duplicate Procedure. It is suggested that duplicate specimens be molded and tested in compression at the age of 7 days and after soaking in water for 4 hours. Minimum compressive strengths of 400 pounds per square inch (psi) are generally considered adequate.

GENERAL REMARKS

There is a distinct difference between satisfactorily hardened soil-cement specimens and inadequately hardened specimens. Even an inexperienced tester will soon be able to differentiate between them and to select a safe cement content to harden the soil. It is important to remember that an excess of cement is not harmful, but that a deficiency of cement will result in inferior stabilization.

GLOSSARY, ACRONYMS AND ABBREVIATIONS

AASHTO	American Association of State Highway Transportation Officials
ACI	American Concrete Institute
AGG	Aggregate
AP	Asphalt petroleum
AP	Asphalt penetration numbers
APSB	Asphalt penetrative soil binder
APSP	Asphalt penetration surface binder
ASTM	American Society for Testing Materials
Avg	Average
C	Celcius
C	Circumference
C	Compaction
CA	Coarse aggregate
CBR	California Bearing Ratio
cc	Cubic centimeter
CE	Corps of Engineers
CF	Cement factor
CMS	Cement modified soil
Cor	Corrected
cwt	Hundred weight
D	Density
D	Diameter
DA	Department of Army
DD	Department of Defense
DCA 70	Polyvinyl acetate emulsion
Diam	Diameter
El	Elevation
f	Foot, feet
F	Fahrenheit
FA	Fine aggregate
FRBS	Fine river bar sand
ft	Foot, feet
G	Gram
G	Gravity
G	Specific gravity
Gal	Gallon
Ga	Adjusted specific gravity
GA	Adjusted specific gravity
Gavg	Average specific gravity
GCI	Gyratory compactibility index
Gm	Bulk specific gravity
gm	Gram
Gs	Solids specific gravity
GSF	Gyratory shear factor
GSI	Gyratory stability index
GTM	Gyratory testing machine
Gw	Specific gravity of liquid
in	Inch
lb	Pound
LCF	Lime-cement fly ash
LF	Lime fly ash
LL	Liquid limit
LMS	Lime-modified soil
LSD	Limestone dust
MB	Mechanical blending
MC	Medium curing cutback
MDD	Maximum dry density
min	Minute
Min	Minimum
MH	Moderate heat of hydration

ml	Milliliter
mm	Millimeter
MS	Medium setting
MS	Moderate sulfate resistance
N	Number
v	Viscosity
N/A	Not applicable
OAC	Optimum asphalt content
Omax	Maximum gyratory angle
oz	Ounce
Pass	Passing
PCF	Wet density
pcf	Pounds per cubic foot
PI	Plasticity index
PL	Plastic limit
psi	Pounds per square inch
Ret	Retained
RC	Rapid curing rate
RelE	Relative dynamic modulus of elasticity
RS	Rapid setting
RT	Road tar
RTCB	Road tar cutbacks
SA	Soil-asphalt
SC	Slow curing cutbacks
SC	Soil-cement
Sg	Gyratory shear strength/value
SIG	Materials Quality Specialist
SL	Soil-lime
SM	Slag modified
Spec	Specific
Sq	Square
SS	Slow setting
SSD	Saturated surface-dry condition
T	Temperature
Temp	Temperature
TM	Technical manual
USCS	Unified Soil Classification System
w/	With (word or number)
W/C	Water-cement ratio
wt	Weight
%	Percent
′	Foot, feet
″	Inch
≤	Equal to or less

REFERENCES

REQUIRED PUBLICATIONS

Required Publications are sources that users must read in order to understand or to comply with this publication.

Engineer Manual (EM)

EM 1110-2-1906	Engineering and Design: Laboratory Soil Testing

Field Manuals (FM)

FM 5-34	Engineer Field Data
FM 5-335	Drainage

Technical Bulletins (TB)

TB 385-103	Safety Procedures For Tester, Density, and Moisture Nuclear Method (Soil and Asphalt) Campbell Pacific Nuclear Corp. Model MC-1 (CCE) (NSN 6635-01-030-6896)

Technical Manuals (TM)

TM 5-330	Planning and Design of Roads, Airbases, and Heliports in the Theater of Operations (AFM 86-3, Vol II)
TM 5-332	Pits and Quarries
TM 5-337	Paving and Surfacing Operations
TM 5-545	Geology
TM 5-822-4	Soil Stabilization for Pavements (AFM 88-7, Chap 4)
TM 5-6635-386-12&P	Operator's and Organizational Maintenance Manual for Tester, Density, and Moisture (Soil and Asphalt) Nuclear Method

Supply Catalogs (SC)

SC 6635-97-CL-E01	Test Set, Soil Trafficability
SC 6635-98-CL-E02-HR	Hand Receipt Catgalog Covering Content of Test Set, Soil and Map Use
SC 6635-98-CL-E02	Test Set, Soil
SC 6635-98-CL-E03	Test Set, Asphalt
SC 6635-98-CL-E04	Test Set, Concrete

DD Forms

DD Form 1205	Soil Moisture Content
DD Form 1206	Sieve Analysis Data

DD Form 1207	Grain Size Distribution Graph - Aggregate Grading Chart
DD Form 1208	Specific Gravity Tests
DD Form 1209	Atterberg Limits Determination
DD Form 1210	Soil Compaction Test Data
DD Form 1211	Soil Compaction Test Graph
DD Form 1212	California Bearing Ratio Test Data
DD Form 1213	Unconfined Compression Test
DD Form 1215	Unit Weight Determination "Volume of Hole" Methods
DD Form 1216	Specific Gravity of Bituminous Mix Components
DD Form 1217	Bituminous Mix Design - Aggregate Blending
DD Form 1218	Marshall Method - Computation of Properties of Asphalt Mixtures
DD Form 1219	Bituminous Mix Curves
DD Form 1220	Concrete Mixture Design Data
DD Form 1793	Determination of Asphalt Content
DD Form 1794	Hydrometer Analysis
DD Form 1795	Combined Mechanical Analysis
DD Form 2463	California Bearing Ratio (CBR) Analysis
DD Form 2464	Report of Foundation and Borrow Investigation

RELATED PUBLICATIONS

Related Publications are sources of additional information. They are not required in order to understand this publication.

Army Regulations (AR)

AR 310-25	Dictionary of United States Army Terms (Short Title: AD)
AR 310-50	Catalog of Abbreviations and Brevity Codes

The American Society of Testing and Materials (ASTM)

ASTM D75-82	Sampling Aggregates, Practice for
ASTM C136-84	Sieve Analysis of Fine and Course Aggregates, Methods for (Rev. A)
ASTM C117-84	Materials Finer Than 75-um (No. 200) Sieve in Mineral Aggregates by Washing, Test Method for
ASTM C128-84	Specific Gravity and Absorption of Fine Aggregates, Test Method for
ASTM C566-84	Total Moisture Content of Aggregate by Drying Test Method for
ASTM C70-79	Surface Moisture in Fine Aggregate, Test Method for (R 1985)
ASTM C142-78	Clay Lumps and Friable Particles in Aggregates, Test Method for (R 1984)

ASTM C88-83	Soundness of Aggregates by Use of Sodium Sulfate or Magnesium Sulfate, Test Method for
ASTM C845-80	Expansive Hydraulic Cement, Spec. for
ASTM D2419-74	Sand Equivalent Value of Soils and Fine Aggregate, Test Method for (R 1979)

Field Manual (FM)

FM 5-541	Military Soils Engineering

Military Standards (MIL STD)

MIL-STD 619B	Unified Soil Classification System for Roads, Airfields Embankments and Foundations
MIL-STD-620A Notice 1	Test Methods for Bituminous Paving Methods
MIL STD-620A Notice 2	Test Methods for Pavement Subgrade, Subbase, and Base Course Material

Navy Documents (NAVDOC)

NAVFAC DM-7.1	Soil Mechanics
NAVFAC DM-7.2	Foundations and Earth Structures

Technical Manuals (TM)

TM 5-241-14	Universal Transverse Mercator Grid Table for Latitudes 0 Degree to 80 Degree: Bessel Spheroid (Meters); Coordinates for 5-Minutes Intersections
TM 5-241-15	Universal Transverse Mercator Grid Table Latitudes 0 Degree to 45 Degree 05': Clarke 1880 Spheroid (Meters); Coordinates for 5-Minute Intersections
TM 5-241-16	Universal Transverse Mercator Grid Table for Latitudes 0 Degree to 40 Degree 05': Everest Spheroid (Meters); Coordinates for 5-Minutes Intersections
TM 5-241-17	Latitude Functions: Hayford Spheroid (International Spheroid): Natural Values of the Meridional ARC A, B, C, D, E, and F Factors, Radii of Curvature R
TM 5-241-19	Latitude Functions: Bassel Spheroid; Natural Values of the Meridional ARC A, B, C, D, E, and F Factors, Radii of Curvatures R and N
TM 5-241-20	Latitude Functions: Clarke 1880 Spheroid; Natural Values of the Meridional ARC A, B, C, D, E, and F Factors, Radii of Curvature R and N
TM 5-241-21	Latitude Function: Everest Spheroid; Natural Values of the Meridional ARC A, B, C, D, E, and F Factors, Radii of Curvatures R and N

TM 5-331A	Utilization of Engineer Construction Equipment: Volume A: Earthmoving, Compaction, Grading, and Ditching Equipment
TM 5-331D	Utilization of Engineer Construction Equipment: Volume D-1: Asphalt and Concrete Equipment
TM 5-349	Arctic Construction
TM 5-624	Maintenance and Repair of Surface Areas
TM 5-805-1	Standard Practice for Concrete Military Structures (AFM 88-3,Chap 6)
TM 5-818-1	Soils and Geology: Procedures for Foundation Design of Buildings and Other Structures (Except Hydraulic Structures) (AFM 88-3, Chap 7)
TM 5-822-5	Engineering and Design: Flexible Pavements for Roads, Streets, Walks, and Open Storage Areas (AFM 88-7, Chap 3)
TM 5-822-6	Rigid Pavements for Roads, Streets, Walks, and Open Storage Areas (AFM 88-7, Chap 1)
TM 5-822-8	Bituminous Pavements-Standard Practice
TM 5-823-2	Army Airfields and Pavements: Airfield-Heliport Flexible Pavement Design
TM 5-824-3	Rigid Pavements of Airfields Other than Army (AFM 88-6, Chapa 3)
TM 5-824-4	Airfields Other than Army: Airfield Operational and Maintenance Facilities
TM 5-825-2	Flexible Pavement Designs for Airfields (NAVFAC DM-21-3, AFM 88-6, Chap 2)

OTHER

American Concrete Institute, PO Box 19150, Redford Station, Detroit, MI 48219

Manual of Concrete Practice, Part I

Reinforced Concrete Design Handbook

American Association of State Highway and Transportation Officials, 444 N. Capitol Street, Washington, D.C. 20001

Standard Specifications for Transportation Materials and Methods of Sampling and Testing, Part II, 1982, 13th ed

American Society for Testing and Materials, 1916 Race Street, Philadelphia, PA 19103

ASTM Book of Standards, Vol I, Vol II

Significance of Tests and Properties of Concrete and Concrete-Making Materials, STD 169B 1978

Department of the Interior, 1981 Washington, D.C.

Concrete Manual

McGraw-Hill Book Company, 1221 Avenue of the Americas, New York, NY 10020

- Composition and Properties of Concrete, 1974, G.E. Troxell, H. E. Davis and J. W. Kelly
- Concrete Construction Handbook, 1968, J. J. Waddell
- Handbook of Heavy Construction, 1971, Havers and Stubbs

The Asphalt Institute, Asphalt Institute Building, College Park, Md 20740

- Manual Series No. 4 (MS-4): The Asphalt Handbook
- Manual Series No. 10 (MS-10): Soils Manual for Designed Asphalt Pavement Structure
- Manual Series No. 13 (MS-13): Asphalt Surface Treatments and Asphalt Penetration Macadam
- SS-1; Specifications and Construction Method for Hot Mix Asphalt Paving

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SOIL MOISTURE CONTENT	DATE
PROJECT	

EXCAVATION NUMBER	SAMPLE NUMBER	FORMULA Water Content, $w = \frac{W_W}{W_S} \times 100$

TEST						
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	%	%	%	%	%	%
TEST						
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	%	%	%	%	%	%
TEST						
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	%	%	%	%	%	%
TEST						
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	%	%	%	%	%	%
TEST						
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	%	%	%	%	%	%

REMARKS *(Continue on reverse if additional space is required)*

TECHNICIAN *(Signature)*	COMPUTED BY *(Signature)*	CHECKED BY *(Signature)*

DD Form 1205, 1 FEB 62 EDITION OF 1 AUG 57 IS OBSOLETE

SIEVE ANALYSIS DATA

1. DATE STARTED

2. PROJECT

3. EXCAVATION

4. DATE COMPLETED

5.

6. SAMPLE NUMBER

7. PREWASHED *(x one)*

☐ YES ☐ NO

8. ORIGINAL SAMPLE WEIGHT

9. + #200 SAMPLE WEIGHT

10. −#200 SAMPLE WEIGHT

11. SIEVE SIZE	12. WEIGHT OF SIEVE	13. WEIGHT OF SIEVE SAMPLE	14. WEIGHT RETAINED	15. CUMULATIVE WEIGHT RETAINED	16. PERCENT RETAINED	17. PERCENT PASSING

18. TOTAL WEIGHT RETAINED IN SIEVES *(Sum Column 14)*

19. ERROR *(8 23)*

20. WEIGHT SIEVED THROUGH #200 *(Weight in pan)*

21. WASHING LOSS *[8 - (9 + 10)]*

22. TOTAL WEIGHT PASSING #200 *(20 + 10)*

23. TOTAL WEIGHT OF FRACTIONS *(18 + 22)*

24. REMARKS

USCS ________________

PERCENT - G __________

PERCENT - S __________

PERCENT - F __________

25. ERROR *(Percent)*

$$\frac{\text{ERROR } (19)}{\text{ORIGINAL WT } (8)} \times 100 =$$

26. TECHNICIAN

27. COMPUTED BY *(Signature)*

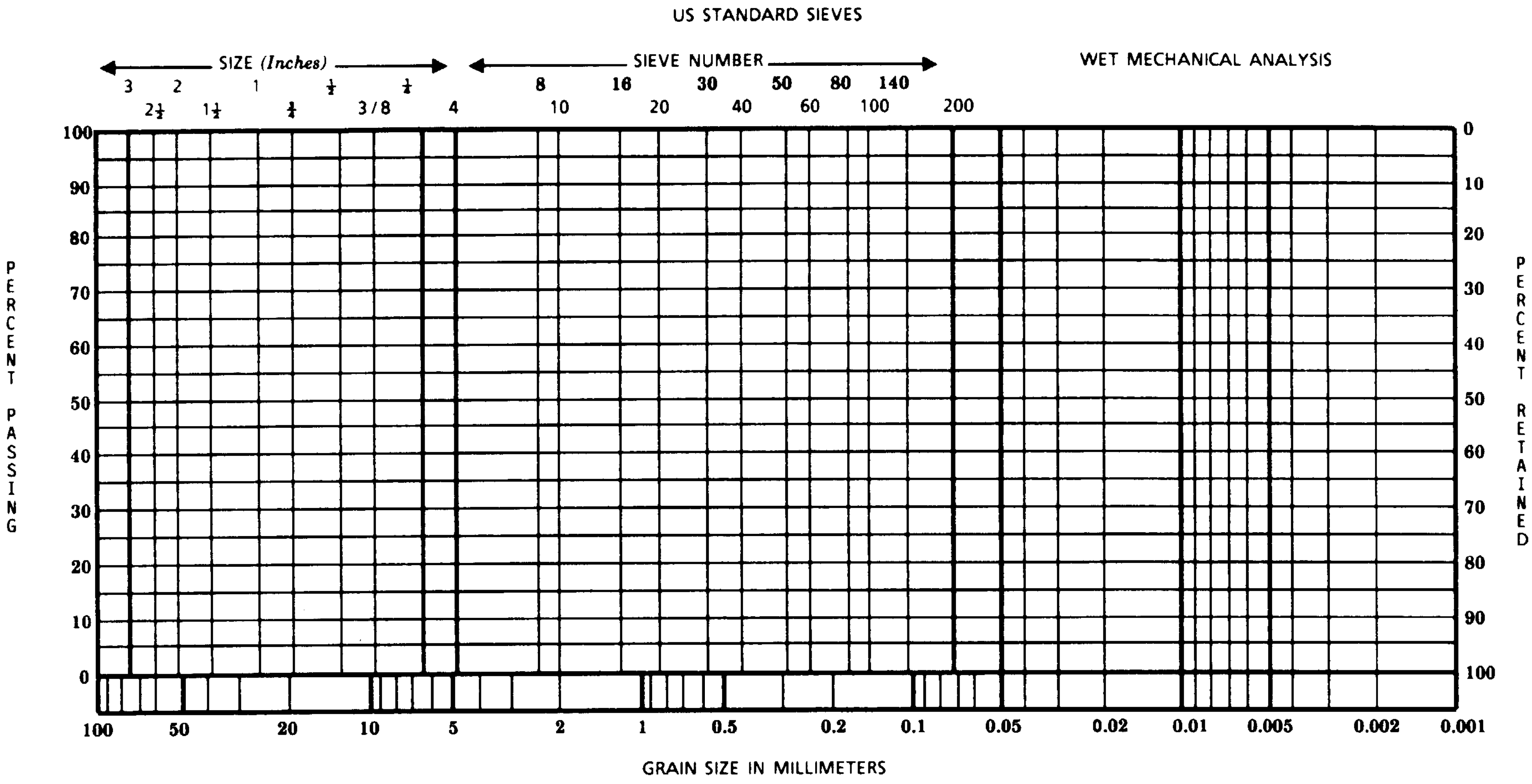
28. CHECKED BY *(Signature)*

DD Form 1206, DEC 86 *Previous editions are obsolete*

GRAIN SIZE DISTRIBUTION GRAPH – AGGREGATE GRADING CHART

DATE

PROJECT



EXCAVATION NUMBER	SAMPLE NUMBER	NATURAL % MOISTURE	w_L	w_p	f_p	CLASSIFICATION	REMARKS

TECHNICIAN *(Signature)*

PLOTTED BY *(Signature)*

CHECKED BY *(Signature)*

DD Form 1207, 1 AUG 57

SPECIFIC GRAVITY TESTS

1. DATE

2. PROJECT

3. BORING NUMBER

4. JOB NUMBER

5. EXCAVATION NUMBER

6. SPECIFIC GRAVITY OF SOLIDS (G_s)

	a.	SAMPLE OR SPECIMEN NUMBER				
	b.	FLASK NUMBER				
	c.	TEMPERATURE OF WATER AND SOIL (T, °C)				
	d.	DISH NUMBER				
WEIGHT IN GRAMS	e.	DISH + DRY SOIL				
	f.	DISH				
	g.	DRY SOIL	W_s			
	h.	FLASK + WATER AT T, °C)	W_{bw}			
	i.	$W_s + W_{bw}$				
	j.	FLASK + WATER + IMMERSED SOIL	W_{bws}			
	k.	DISPLACED WATER, $W_s + W_{bw} - W_{bws}$				
	l.	CORRECTION FACTOR	K			
	m.	$(W_sK) \div (W_s + W_{bw} - W_{bws})$	G_s			

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

	a.	SAMPLE OR SPECIMEN NUMBER				
	b.	TEMPERATURE OF WATER AND SOIL (T, °C)				
	c.	TARE + SATURATED SURFACE - DRY SOIL				
WEIGHT IN GRAMS	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.	CORRECTION FACTOR	K			
	m.	(AK) ÷ (A − C) (APPARENT)	G_a			
	n.	(AK) ÷ (B − C) (BULK)	G_m			

8. REMARKS

9. TECHNICIAN *(Signature)*

10. COMPUTED BY *(Signature)*

11. CHECKED BY *(Signature)*

Previous editions are obsolete.

ATTERBERG LIMITS DETERMINATION

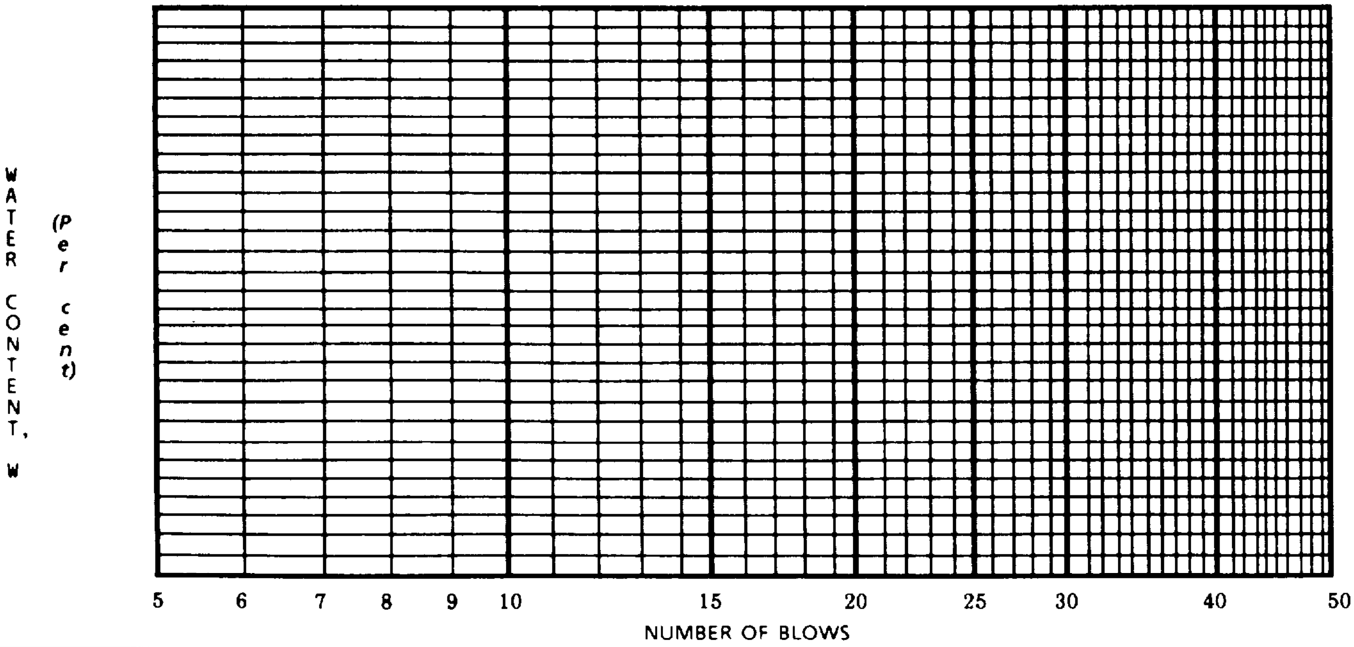
DATE

PROJECT	EXCAVATION NUMBER	SAMPLE NUMBER

LIQUID LIMIT, W_L

RUN NUMBER						
TARE NUMBER						
A. WEIGHT OF WET SOIL + TARE						
B. WEIGHT OF DRY SOIL + TARE						
C. WEIGHT OF WATER, W_W *(A. - B.)*						
D. WEIGHT OF TARE						
E. WEIGHT OF DRY SOIL, W_S *(B. - D.)*						
WATER CONTENT, $w = (\frac{W_W}{W_S} x\,100)$						
NUMBER OF BLOWS						

w_L	w_P	$I_P\ (w_L - w_P)$



PLASTIC LIMIT, W_P

					NATURAL WATER CONTENT
RUN NUMBER					
TARE NUMBER					
P. WEIGHT OF WET SOIL + TARE					
G WEIGHT OF DRY SOIL + TARE					
H. WEIGHT OF WATER, W_W *(F. - G.)*					
I. WEIGHT OF TARE					
J. WEIGHT OF DRY SOIL, W_S *(G. - I.)*					
WATER CONTENT, $w = (\frac{W_W}{W_S} x\,100)$					
PLASTIC LIMIT, I_P *(Average w)*					

REMARKS

TECHNICIAN *(Signature)*	COMPUTED BY *(Signature)*	CHECKED BY *(Signature)*

DD Form 1209, AUG 57

SOIL COMPACTION TEST DATA

DATE

PROJECT

EXCAVATION NUMBER

SAMPLE NUMBER

NUMBER OF LAYERS

CONVERSION FACTORS
1728 cu. in. per cu. ft.
453.6 gm. per lb.

NUMBER OF BLOWS PER LAYER

WEIGHT OF TAMPER

HEIGHT OF DROP

MAXIMUM PARTICLE SIZE

SPECIFIC GRAVITY

DIAMETER OF MOLD *(in.)*

HEIGHT OF SOIL SAMPLE *(in.)*

VOLUME OF SOIL SAMPLE *(cu. ft.)*

RUN NUMBER	UNITS												
WEIGHT OF WET SOIL + MOLD													
WEIGHT OF MOLD													
WEIGHT OF WET SOIL													
WET UNIT WEIGHT, $\gamma = \frac{\text{Weight of wet soil (lb.)}}{\text{Vol of soil sample (cu. ft)}}$	LB. PER CU. FT.												
TARE NUMBER													
A. WEIGHT OF WET SOIL + TARE													
B. WEIGHT OF DRY SOIL + TARE													
C. WEIGHT OF WATER, W_W *(A. - B.)*													
D. WEIGHT OF TARE													
E. WEIGHT OF DRY SOIL, W_S *(B. - D.)*													
WATER CONTENT, $W = (\frac{W_W}{W_S} x\, 100)$	PERCENT												
AVERAGE WATER CONTENT	PERCENT												
DRY UNIT WEIGHT, $\gamma\ d = (\frac{\gamma}{1 + w/100})$	LB. PER CU. FT.												

TECHNICIAN *(Signature)*

COMPUTED BY *(Signature)*

CHECKED BY *(Signature)*

DD Form 1210, AUG 57

Mar applicable block ☐ **SOIL COMPACTION TEST GRAPH** ☐ **SWELL DATA**		1. DATE
2. PROJECT		**RESULTS** 6. MAXIMUM DRY UNIT WEIGHT
3. EXCAVATION NUMBER	4. SAMPLE NUMBER	
5. REMARKS *(Use reverse side, if more space is needed)*		7. OPTIMUM MOISTURE *(Water)* CONTENT

☐
PERCENT
OF SWELL

☐
DRY UNIT
WEIGHT
(Pounds per cubic foot)

MOISTURE *(Water)* CONTENT, W *(Percent of dry weight)*

8. TECHNICIAN *(Signature)*	9. PLOTTED BY *(Signature)*	10. CHECKED BY *(Signature)*

DD Form 1211, DEC 86

Previous editions are obsolete.

CALIFORNIA BEARING RATIO TEST DATA	DATE

PROJECT

☐ FIELD-IN-PLACE	TEST SITE

☐ LABORATORY	EXCAVATION NUMBER	SAMPLE NUMBER	TYPE ☐ UNSOAKED ☐ SOAKED ☐ DRAINED ☐ UNDRAINED

CONDITION ☐ UNDISTURBED ☐ DISTURBED	NUMBER OF LAYERS	NUMBER OF BLOWS PER LAYER	WEIGHT OF HAMMER *(lb.)*	DROP *(inches)*

MOLD NUMBER	MOLD DIAMETER *(in.)*	MOLD HEIGHT *(in.)*	DEPTH, TOP TO SAMPLE *(in.)*	SAMPLE HEIGHT *(in.)*	SAMPLE VOLUME *(cu. ft.)*

			SURCHARGE WEIGHTS	
PROVING RING NUMBER	PROVING RING CONSTANT	PROVING RING CAPACITY	SOAKING *(lb.)*	PENETRATING *(lb.)*

FORMULAS

Total Load = Corrected Dail Reading x Proving Ring Constant

$$\text{Unit Load} = \frac{\text{Total Load}}{3} \qquad \text{CBR}(\%) = \frac{\text{Corrected Unit Load}}{\text{Standard Unit Load}} \times 100$$

CONVERSION FACTORS

1728 cu. in. per cu. ft.

453.6 gm. per lb.

PENETRATION DATA

PENETRATION *(inches)*	STD. UNIT LOAD *(psi)*	PROVING RING DIAL READING	CORRECTED RING DIAL READING	TOTAL LOAD *(pounds)*	UNIT LOAD *(psi)*	CORRECTED UNIT LOAD *(psi)*	CBR *(%)*
0.025	**250**						
0.050	**500**						
0.075	**750**						
0.100	**1000**						
0.150	**1250**						
0.200	**1500**						
0.250	**1700**						
0.300	**1900**						
0.400	**2300**						
0.500	**2600**						

WATER CONTENT AND UNIT WEIGHT DATA

WEIGHT OF SOIL + MOLD *(lb.)* (gm)*		
WEIGHT OF MOLD *(lb.)* (gm)*		
WEIGHT OF WET SOIL *(lb.)* (gm)*		
VOLUME OF SAMPLE *(cu. ft.)*		
WET UNIT WEIGHT *(lb./cu. ft.)*		
TARE NUMBER		
WEIGHT OF WET SOIL + TARE *(gm.)*		
WEIGHT OF DRY SOIL + TARE *(gm.)*		
WEIGHT OF WATER, W_w *(gm.)*		
WEIGHT OF TARE *(gm.)*		
WEIGHT OF DRY SOIL, W_s *(gm.)*		
WATER CONTENT, w *(%)*		
AVERAGE w *(%)*		
DRY UNIT WEIGHT *(Density)* *(lb./cu. ft.)*		

OPTIMUM MOISTURE CONTENT OF SOIL, w_{opt} *(%)*	THEORETICAL MAXIMUM UNIT WEIGHT *(Density)* *(lb./cu. ft.)*
ACTUAL WATER CONTENT OF SOIL, w *(%)*	ACTUAL UNIT WEIGHT *(lb./cu. ft.)*
DIFFERENCE *(%)*	PERCENT OF MAXIMUM $\left(\frac{\text{Actual}}{\text{Maximum}} \times 100\right)$

REMARKS

*Strike out unit weight not applicable.

USE REVERSE SIDE FOR SWELL DATA AND GRAPH

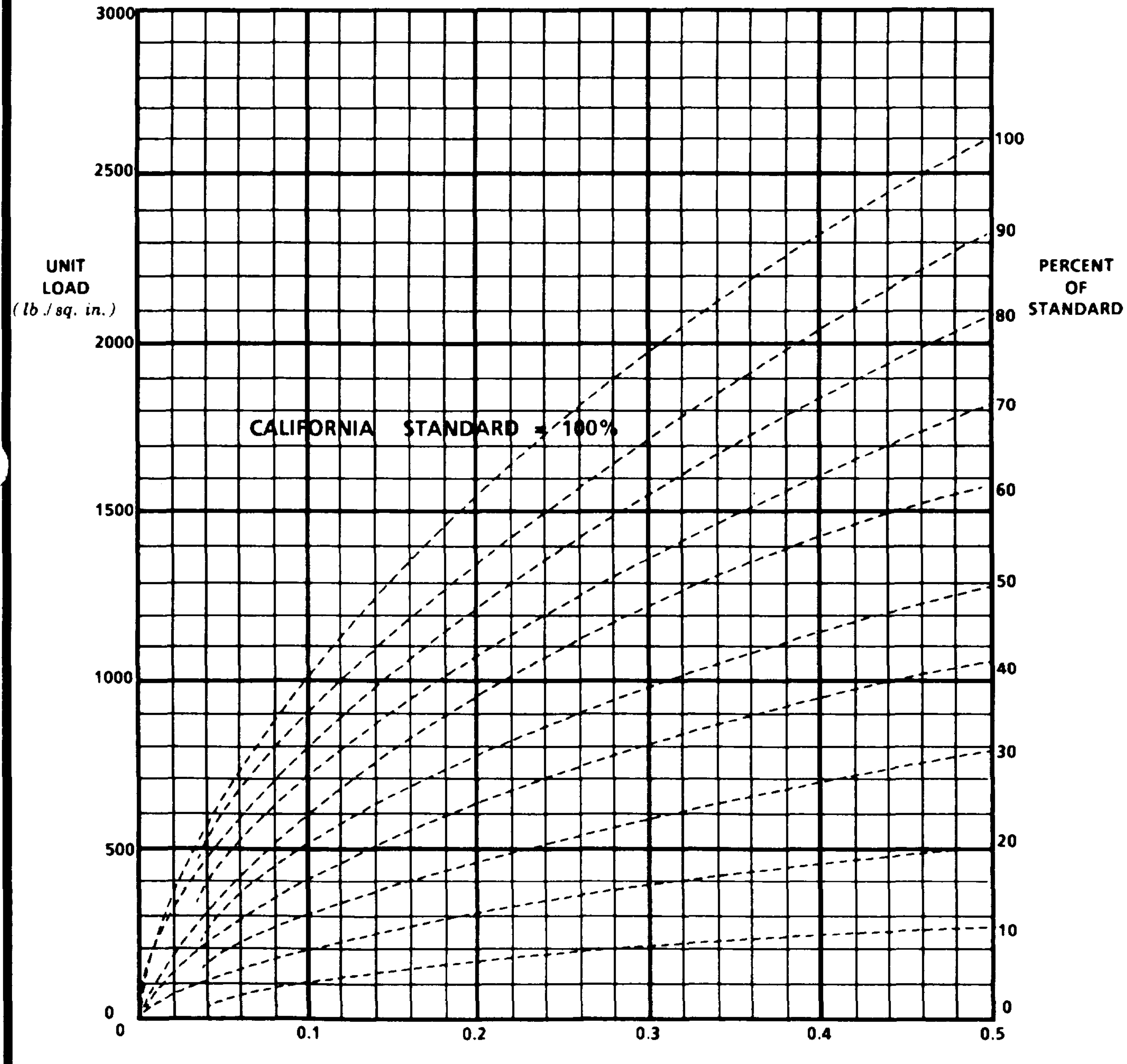
DD Form 1212, AUG 57

SWELL DATA

DATE	TIME	DAYS	GAGE READING	SWELL *(in.)*	SWELL *(%)*
		0			
		1			
		2			
		3			
		4			
		5			

CALIFORNIA BEARING RATIO TEST GRAPH

(Plot test curve below to obtain corrected unit load)



PENETRATION *(in.)*

TECHNICIAN *(Signature)*	COMPUTED BY *(Signature)*	CHECKED BY *(Signature)*

DD Form 1212 Reverse, AUG 57

UNCONFINED COMPRESSION TEST

DATE

PROJECT

EXCAVATION NUMBER

SAMPLE NUMBER

PROVING RING DIAL NUMBER

PROVING RING NUMBER

CALIBRATION CURVE NUMBER

PROVING RING CONSTANT , K_r

VERTICAL DIAL NUMBER

RATE OF LOAD APPLICATION

FORMULAS

$$\text{Area} = \frac{\pi D^2}{4} = \frac{C^2 D}{4} \qquad \text{Corrected Area, } A = \frac{A_0\,(sq.\,ft.)}{1 - E} \qquad \text{Unit Strain, } E = \frac{\Delta L}{L_0} \qquad \text{Unit Stress} = \frac{P\,(lb.)}{A\,(sq.\,ft.)}$$

INITIAL MEASUREMENTS ON SAMPLE			
	DIAMETER, D *(in.)*	CIRCUMFERENCE , C *(in.)*	AREA *(sq. in.)*
TOP			
CENTER			
BOTTOM			

AVERAGE AREA, A_0	
SQ. IN.	SQ. FT.

HEIGHT, L_0 *(in.)*

ELAPSED TIME *(min.)*	PROVING RING DIAL READING *(0.0001 in.)*	LOAD, P *(lb.)* *(Ring Dial x K_r, or from calibration curve)*	VERTICAL DIAL READING *(in.)*	VERTICAL DIAL DIFFERENCE, ΔL *(in.)*	UNIT STRAIN, E *(in./in.)*	CORRECTED AREA, A *(sq. ft.)*	UNIT STRESS *(lb./sq. ft.)*

WATER CONTENT *(%)*

UNCONFINED COMPRESSIVE STRENGTH, q_u *(lb./sq. ft.)*

REMARKS AND CALCULATIONS *(Use reverse side, if more space is needed)*

TECHNICIAN *(Signature)*

COMPUTED BY *(Signature)*

CHECKED BY *(Signature)*

DD Form 1213, AUG 57

UNIT WEIGHT DETERMINATION **"VOLUME OF HOLE" METHODS**	DATE

PROJECT	TEST SITE	SAMPLE NUMBER

ADDITIONAL SPECIFICATIONS

CONVERSION FACTORS

1 in. = 2.54 cm.
1 lb. = 453.6 gm.
1 gm./cc. or 62.4 lb./cu. ft. = Unit weight of water
1 cu. ft. = 1728 cu. in.

CALIBRATION OF STANDARD MATERIAL	STANDARD MATERIAL *(Check one)* ☐ SAND ☐ OIL ☐ OTHER *(Specify)*				
	UNITS				
APPARATUS OR TARE NUMBER					
1. WEIGHT OF APPARATUS OR TARE FILLED					
2. WEIGHT OF APPARATUS OR TARE EMPTY					
3. WEIGHT OF MATERIAL *(1. -2.)*					
4. VOLUME OF APPARATUS OR TARE					
5. UNIT WEIGHT OF MATERIAL ($\frac{3.}{4.}$)					
6. AVERAGE UNIT WEIGHT OF MATERIAL	LB /CU FT /				

CALIBRATION OF APPARATUS		TEMPLATE NUMBER		CONE NUMBER	
	UNITS				
7. INITIAL WEIGHT OF APPARATUS + SAND					
8. FINAL WEIGHT OF APPARATUS + SAND					
9. WEIGHT OF SAND IN TEMPLATE AND/OR CONE					

"VOLUME OF HOLE"					
	UNITS				
10. INITIAL WEIGHT OF APPARATUS + MATERIAL					
11. FINAL WEIGHT OF APPARATUS + MATERIAL					
12. WEIGHT OF MATERIAL RELEASED *(10. -11.)*					
13. WEIGHT OF MATERIAL IN HOLE *(For oil, same as 12. For sand, 12. -9.)*					
14. VOLUME OF HOLE ($\frac{13.}{6.}$)					

DD Form 1215, AUG 57

WATER CONTENT DETERMINATION

	UNITS						
TARE NUMBER							
15. WEIGHT WET SOIL & TARE							
16. WEIGHT DRY SOIL & TARE							
17. WEIGHT WATER *(15. -16.)*							
18. WEIGHT TARE							
19. WEIGHT DRY SOIL *(16. -18.)*							
20. WATER CONTENT $(\frac{17.}{19.} \times 100)$							
21. AVERAGE WATER CONTENT	PERCENT						

UNIT WEIGHT DETERMINATION

	UNITS			
TARE NUMBER				
22. WEIGHT WET SOIL & TARE				
23. WEIGHT TARE				
24. WEIGHT WET SOIL *(22. -23.)*				
25. WET UNIT WEIGHT *(24. /14.)*	LB./CU. FT.			
26. DRY UNIT WEIGHT $(25. \times \frac{100}{100 + 21.})$	LB./CU. FT.			

REMARKS

TECHNICIAN *(Signature)*	COMPUTED BY *(Signature)*	CHECKED BY *(Signature)*

DD Form 1215 Reverse, AUG 57

SPECIFIC GRAVITY OF BITUMINOUS MIX COMPONENTS

DATE

PROJECT

JOB

COARSE AGGREGATE	**UNITS** *(Grams)*			
MATERIAL PASSING __________ SIEVE AND RET AINED ON _______ SIEVE				
SAMPLE NUMBER				
1. WEIGHT OF OVEN - DRY AGGREGATE				
2. WEIGHT OF SATURATED AGGREGATE IN WATER				
3. DIFFERENCE *(Line 1 minus 2)*				
APPARENT SPECIFIC GRAVITY, G = $\frac{(Line\ 1)}{(Line\ 3)}$				
FINE AGGREGATE	**UNITS** *(Grams)*			
MATERIAL PASSING NUMBER _________ SIEVE				
SAMPLE NUMBER				
4. WEIGHT OF OVEN - DRY MATERIAL				
5. WEIGHT OF FLASK FILLED WITH WATER AT 20° *C*				
6. SUM (Line 4 + 5)				
7. WEIGHT OF FLASK + AGGREGATE + WATER AT 20° *C*,				
8. DIFFERENCE *(Line 6 minus 7)*				
APPARENT SPECIFIC GRAVITY, G = $\frac{(Line\ 4)}{(Line\ 8)}$				
FILLER	**UNITS** *(Grams)*			
SAMPLE NUMBER				
9. WEIGHT OF OVEN - DRY MATERIAL				
10. WEIGHT OF FLASK FILLED WITH WATER AT 20° *C*,				
11. SUM *(Line 9 + 10)*				
12. WEIGHT OF FLASK + AGGREGATE + WATER AT 20° *C*,				
13. DIFFERENCE *(Line 11 minus 12)*				
APPARENT SPECIFIC GRAVITY, G = $\frac{(Line\ 9)}{(Line\ 13)}$				
BINDER	**UNITS** *(Grams)*			
SAMPLE NUMBER				
14. WEIGHT OF PYCNOMETER FILLED WITH WATER				
15. WEIGHT OF EMPTY PYCNOMETER				
16. WEIGHT OF WATER *(Line 14 minus 15)*				
17. WEIGHT OF PYCNOMETER + BINDER				
18. WEIGHT OF BINDER *(Line 17 minus 15)*				
19. WEIGHT OF PYCNOMETER + BINDER + WATER TO FILL PYCNOMETER				
20. WEIGHT OF WATER TO FILL PYCNOMETER *(Line 19 minus 17)*				
21. WEIGHT OF WATER DISPLACED BY BINDER *(Line 16 minus 20)*				
APPARENT SPECIFIC GRAVITY, G = $\frac{(Line\ 18)}{(Line\ 21)}$				

TECHNICIAN *(Signature)*

COMPUTED BY *(Signature)*

CHECKED BY *(Signature)*

DD Form 1216, 1 DEC 65 PREVIOUS EDITION OF THIS FORM IS OBSOLETE.

BITUMINOUS MIX DESIGN - AGGREGATE BLENDING		DATE
PROJECT	JOB	AGGREGATE GRADATION NUMBER

GRADATION OF MATERIAL

SIEVE SIZE (*To be entered by Technician*): →												
MATERIAL USED	PERCENT PASSING											
DESIRED:												

COMBINED GRADATION FOR BLEND - TRIAL NUMBER ____________

SIEVE SIZE (*To be entered by Technician*): →													
MATERIAL USED	% USED	PERCENT PASSING											
BLEND:													
DESIRED:													

COMBINED GRADATION FOR BLEND - TRIAL NUMBER ____________

SIEVE SIZE (*To be entered by Technician*): →													
MATERIAL USED	% USED	PERCENT PASSING											
BLEND:													
DESIRED:													

DD Form 1217, DEC 65 PREVIOUS EDITION OF THIS FORM IS OBSOLETE.

COMBINED GRADATION FOR BLEND - TRIAL NUMBER ____________

SIEVE SIZE *(To be entered by Technician)*: →													
MATERIAL USED	% USED	PERCENT PASSING											
BLEND:													
DESIRED:													

COMBINED GRADATION FOR BLEND - TRIAL NUMBER ____________

SIEVE SIZE *(To be entered by Technician)*: →													
MATERIAL USED	% USED	PERCENT PASSING											
BLEND:													
DESIRED:													

REMARKS

TECHNICIAN *(Signature)*	COMPUTED BY *(Signature)*	CHECKED BY *(Signature)*

DD Form 1217 Reverse, DEC 65

MARSHALL METHOD - COMPUTATION OF PROPERTIES OF ASPHALT MIXTURES	DATE OF COMPUTATION

JOB NUMBER	PROJECT	DESCRIPTION OF BLEND

SPECIMEN NUMBER	ASPHALT CEMENT *(Percent)*	THICK-NESS *(Inches)*	WEIGHT *(Grams)*		VOLUME CC	SPECIFIC GRAVITY		AC BY VOLUME *(Percent)*	VOIDS *(Percent)*		UNIT WEIGHT TOTAL MIX *(Lb./Cu.Ft.)*	STABILITY *(Pounds)*		FLOW UNITS OF 1/100 IN.
			IN AIR	IN WATER		ACTUAL	THEO-RIZED		TOTAL MIX	FILLED		MEASURED	CON-VERTED	
a	*b*	*c*	*d*	*e*	*f*	*g*	*h*	*i*	*j*	*k*	*l*	*m*	*n*	*o*
▨	▨	▨	▨	▨	*(d - e)*	$\frac{(d)}{(f)}$	▨	$\frac{(b \times g)}{(Sp.\ Gr.\ of\ AC)}$	$(100-100\frac{(g)}{(h)})$	$\frac{(i)}{(i+j)}$	*(g x 62.4)*	▨	*	▨

**From conversion table*	COMPUTED BY	CHECKED BY

DD Form 1218, 1 DEC 65 PREVIOUS EDITION OF THIS FORM IS OBSOLETE.

BITUMINOUS MIX CURVES

DATE

PROJECT

JOB

RECAP OF MARSHALL TEST RESULTS

PERCENT BITUMEN CONTENT					
UNIT WEIGHT - AGGREGATE ONLY					
UNIT WEIGHT - TOTAL MIX					
STABILITY					
FLOW					
PERCENT VOIDS - AGGREGATE ONLY					
PERCENT VOIDS - TOTAL MIX					
PERCENT OF TOTAL VOIDS FILLED WITH BITUMEN					

FLOW *(1/100 Inch)*

PERCENT BITUMEN

UNIT WEIGHT - TOTAL MIX *(Lb. Per Cu. Ft.)*

PERCENT BITUMEN

PERCENT VOIDS - TOTAL MIX

PERCENT BITUMEN

STABILITY *(Lb.)*

PERCENT BITUMEN

PERCENT TOTAL VOIDS FILLED WITH BITUMEN

PERCENT BITUMEN

TECHNICIAN *(Signature)*

PLOTTED BY *(Signature)*

CHECKED BY *(Signature)*

DD Form 1219, 1 DEC 65 PREVIOUS EDITION OF THIS FORM IS OBSOLETE

CONCRETE MIXTURE DESIGN DATA

DATE

PROJECT

JOB

PORTLAND CEMENT		OTHER ADMIXTURE	AIR-ENT ADMIXTURE	
TYPE	ADDITIONS	TYPE	TYPE	AMOUNT
BRAND & MILL		SOURCE		

FINE AGGREGATE	COARSE AGGREGATE	
TYPE	TYPE	SIZE
SOURCE	SOURCE	

MATERIALS

MATERIALS	SERIAL NUMBER	SIZE RANGE	BULK SP. GR.	ABSORPTION %
CEMENT				
FINE AGGREGATE				
COARSE AGGREGATE *(A)*				
COARSE AGGREGATE *(B)*				
COARSE AGGREGATE *(C)*				
COARSE AGGREGATE *(D)*				

MIXTURE DATA

MATERIALS	SAMPLE NUMBER			SAMPLE NUMBER		
	MIXTURE BY WEIGHT	NET WEIGHTS 1 BAG BATCH *(lb.)*	SOLID VOLUME 1 BAG BATCH *(cu. ft.)*	MIXTURE BY WEIGHT	NET WEIGHTS 1 BAG BATCH *(lb.)*	SOLID VOLUME 1 BAG BATCH *(cu. ft.)*
CEMENT						
FINE AGGREGATE						
COARSE AGGREGATE *(A)*						
COARSE AGGREGATE *(B)*						
COARSE AGGREGATE *(C)*						
COARSE AGGREGATE *(D)*						
WATER						
AIR	▨	▨		▨	▨	
TOTAL						

WATER/CEMENT *(gal. per bag)*	THEO. UNIT WEIGHT *(lb./cu. ft.)*
SLUMP *(in.)*	ACTUAL UNIT WEIGHT *(lb./cu. ft)*
AIR CONTENT *(%)*[2]	THEO. CEMENT FACT. *(bag/cu. yd)*
AIR CONTENT *(%)*[3]	ACTUAL CEMENT FACT. *(bag/cu. yd)*
SAND/AGGREGATE *(% volume)*	

REMARKS *(Condition of mix, workability, plasticity, bleeding, etc.)*

TECHNICIAN *(Signature)*	COMPUTED BY *(Signature)*	CHECKED BY *(Signature)*

1. Calculated on the basis of:

2. In the entire batch as mixed.

3. In that portion of the concrete containing aggregate smaller than the 1½ inch sieve.

DD Form 1220, 1 AUG 57

DETERMINATION OF ASPHALT CONTENT

(Dulin - Rotarex Extractor)

PROJECT

DATE

LINE NO.	ITEM		UNIT	1	2
1.	Weight of original sample and tare		g		
2.	Weight of tare		g		
3.	Weight of original sample	*(1-2)*	g		
4.	Weight of clean aggregate and tare		g		
5.	Weight of tare		g		
6.	Weight of clean aggregate	*(4-5)*	g		
7.	Final weight of filter paper		g		
8.	Initial weight of filter paper		g		
9.	Weight of filler in filter paper		g		
10.	Total amount of solvent		cm^3		
11.	Amount of solvent evaporated and ignited		cm^3		
12.	Final weight of evaporation dish and residue		g		
13.	Initial weight of clean evaporation dish		g		
14.	Amount of filler in evaporation dish	*(12 - 13)*	g		
15.	Amount of filler in total solvent	$(\frac{10}{11} \times 14)$	g		
16.	Total amount of aggregate in sample	*(6+9+15)*	g		
17.	Percentage asphalt	$(\frac{3-16}{6} \times 100)$	%		

REMARKS

TECHNICIAN	COMPUTED BY	CHECKED BY

DD Form 1793, FEB 71

HYDROMETER ANALYSIS

PROJECT			DATE
BORING NO.	SAMPLE OR SPECIMEN NO.	CLASSIFICATION	
DISH NO.	GRADUATE NO.		HYDROMETER NO.
DISPERSING AGENT USED			QUANTITY
DISPERSING AGENT CORRECTION C_d =		MENISCUS CORRECTION C_m =	

TIME	ELAPSED TIME MIN	TEMP °C	HYDRO. READING *(R')*	CORRECTED READING *(R)*	PARTICLE DIAMETER *(D)*, mm	TEMP CORRECTION *(m)*	*R + m*	PERCENT FINER	
								PARTIAL	TOTAL

WEIGHT IN GRAMS	Dish plus dry soil		
	Dish		
	Dry soil	W_o	

Specific gravity of solids,
G_s =

Corrected hydrometer reading *(R)*
= hydrometer reading *(R')* + C_d + C_m

The particle diameter *(D)* is calculated from Stokes' equation using corrected hydrometer reading. Use nomographic chart for solution of Stokes' equation.

Hydrometer graduated in specific gravity

$$\text{Partial percent finer} = \frac{G_s}{G_s - 1} \times \frac{100}{W_o} (R + m)$$

Hydrometer graduated in grams per liter

$$\text{Partial percent finer} = \frac{100}{W_o} (R + m)$$

$$\text{Total percent finer} = \text{partial percent finer} \times \frac{W_s - W_1}{W_s}$$

W_s = total oven-dry wt of sample used for combined analysis

W_o = oven-dry wt in grams of soil used for hydrometer analysis

W_1 = oven-dry wt of sample retained on No. 200 sieve

REMARKS

TECHNICIAN	COMPUTED BY	CHECKED BY

DD Form 1794, FEB 71

CALIFORNIA BEARING RATIO (CBR) ANALYSIS

1 DATE

2. PROJECT

3 SOIL CLASSIFICATION

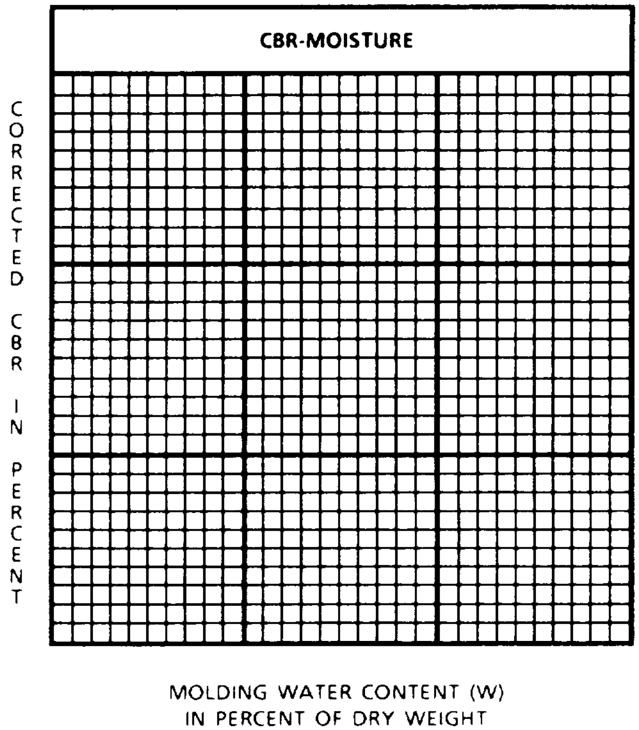
4. LOCATION OF SOIL

5. GROUP NUMBER

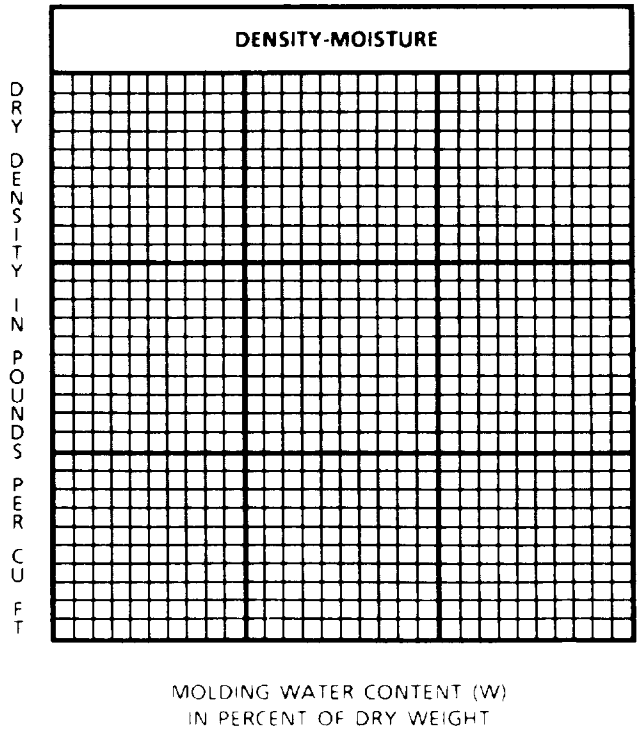
DATA SUMMARY

MOLD NUMBER	BLOWS PER LAYER	MOISTURE CONTENT W *(Percent)*	DRY DENSITY	CORRECTED CBR *(Percent)*

CBR-MOISTURE



DENSITY-MOISTURE



6 TECHNICIAN

7 COMPUTED BY

8 CHECKED BY

DD Form 2463, DEC 86

CBR FAMILY OF CURVES DATA

W *(Percent)*		BLOWS / LAYER	BLOWS / LAYER	BLOWS / LAYER
	DRY DENSITY IN LB / CU FT			
	CORRECTED CBR IN PERCENT			
	DRY DENSITY IN LB / CU FT			
	CORRECTED CBR IN PERCENT			
	DRY DENSITY IN LB / CU FT			
	CORRECTED CBR IN PERCENT			
	DRY DENSITY IN LB / CU FT			
	CORRECTED CBR IN PERCENT			
	DRY DENSITY IN LB / CU FT			
	CORRECTED CBR IN PERCENT			
	DRY DENSITY IN LB / CU FT			
	CORRECTED CBR IN PERCENT			
	DRY DENSITY IN LB / CU FT			
	CORRECTED CBR IN PERCENT			
	DRY DENSITY IN LB / CU FT			
	CORRECTED CBR IN PERCENT			
	DRY DENSITY IN LB / CU FT			
	CORRECTED CBR IN PERCENT			
	DRY DENSITY IN LB / CU FT			
	CORRECTED CBR IN PERCENT			

CBR FAMILY OF CURVES

CORRECTED CBR PERCENT

MOLDED DRY DENSITY IN POUNDS PER CU FT

DESIGN CBR

COMPACTION RANGE

________ % TO ________ %

________ LBS/FT³ TO ________ LBS/FT³

W *(Percent)*	LOWEST CBR	W PERCENT RANGE	ASSURED CBR
		% TO %	
		% TO %	
		% TO %	
		% TO %	
		% TO %	
		% TO %	
		% TO %	
		% TO %	
		% TO %	
		% TO %	
		% TO %	

		DESIGN CBR (HIGHEST ASSURED) ________
		DESIGN MOISTURE RANGE ________ % TO ________ %

REPORT OF FOUNDATION AND BORROW INVESTIGATION					1. DATE
2. SITE	3. TYPE OF EXPLORATION	4. BORING NUMBER		5. LOCATION	6. GROUND ELEVATION
7. PURPOSE OF EXPLORATION					

8. DEPTH BELOW SURFACE	9. ELEVATION	10. SAMPLE NUMBER	11. GRAPHIC LOG	12. GROUP SYMBOL	13. DESCRIPTION, TEST DATA, AND REMARKS

14. DEPTH TO WATER TABLE	15. SUBMITTED BY

DD Form 2464, DEC 86

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NAVFAC MO-330
AFM 89-3
17 AUGUST 1987

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