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ABSTRACT

This individualized, self-paced course for independent study in soils engineering was adapted from military curriculum materials for use in vocational education. The course is designed to acquaint students with various soil types and their characteristics using various procedures, tests, and recording forms. Some of these duties are determining which soils make the best foundation materials and what effect frost action has on these soils; performing soil surveys and soil classification by field tests and by the Unified Soil Classification System; performing procedures to compact and stabilize soils and design a base course for bituminous roads and airfield runways; and using the proper procedures, equipment, tables, formulas and other aids in tests and soil analysis. The contents are divided into the following eight lessons, each containing objectives, reading assignments, and self-graded tests: basic soil properties; soil surveys, soil classification; soil compaction; strength design using California bearing ratio; flexible pavement structure; frost action and permafrost; improvement of soil characteristics; and construction methods and practices. The text is coded and all questions are keyed to the text. A final examination consisting of 50 multiple choice questions, without answers, is included. (KC)

MILITARY CURRICULUM MATERIALS

The military-developed curriculum materials in this course package were selected by the National Center for Research in Vocational Education Military Curriculum Project for dissemination to the six regional Curriculum Coordination Centers and other instructional materials agencies. The purpose of disseminating these courses was to make curriculum materials developed by the military more accessible to vocational educators in the civilian setting.

The course materials were acquired, evaluated by project staff and practitioners in the field, and prepared for dissemination. Materials which were specific to the military were deleted, copyrighted materials were either omitted or approval for their use was obtained. These course packages contain curriculum resource materials which can be adapted to support vocational instruction and curriculum development.

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- Generating knowledge through research
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- Evaluating individual program needs and outcomes
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Military Curriculum Materials for Vocational and Technical Education

Information and Field Services Division

The National Center for Research in Vocational Education

Military Curriculum Materials Dissemination Is . . .

an activity to increase the accessibility of military-developed curriculum materials to vocational and technical educators.

This project, funded by the U.S. Office of Education, includes the identification and acquisition of curriculum materials in print form from the Coast Guard, Air Force, Army, Marine Corps and Navy.

Access to military curriculum materials is provided through a "Joint Memorandum of Understanding" between the U.S. Office of Education and the Department of Defense.

The acquired materials are reviewed by staff and subject matter specialists, and courses deemed applicable to vocational and technical education are selected for dissemination.

The National Center for Research in Vocational Education is the U.S. Office of Education's designated representative to acquire the materials and conduct the project activities.

Project Staff:

Wesley E. Budke, Ph.D., Director
National Center Clearinghouse

Shirley A. Chase, Ph.D.
Project Director

What Materials Are Available?

One hundred twenty courses on microfiche (thirteen in paper form) and descriptions of each have been provided to the vocational Curriculum Coordination Centers and other instructional materials agencies for dissemination.

Course materials include programmed instruction, curriculum outlines, instructor guides, student workbooks and technical manuals.

The 120 courses represent the following sixteen vocational subject areas:

Agriculture	Food Service
Aviation	Health
Building & Construction Trades	Heating & Air Conditioning
Clerical Occupations	Machine Shop
Communications	Management & Supervision
Drafting	Meteorology & Navigation
Electronics	Photography
Engine Mechanics	Public Service

The number of courses and the subject areas represented will expand as additional materials with application to vocational and technical education are identified and selected for dissemination.

How Can These Materials Be Obtained?

Contact the Curriculum Coordination Center in your region for information on obtaining materials (e.g., availability and cost). They will respond to your request directly or refer you to an instructional materials agency closer to you.

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SOILS ENGINEERING

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Course Description

This course is designed to acquaint students with various soil types and their characteristics using various procedures, tests and recording forms. Some of these duties are:

- Determining which soils make the best foundation materials and what effect frost action has on these soils
- Performing soil surveys and soil classification by field tests and by the Unified Soil Classification System
- Performing procedures to compact the stabilize soils and design base course for bituminous roads and airfield runways
- Using the proper procedures, equipment, tables, formulas and other aids in tests and soil analysis

The contents of the course are divided into eight lessons each containing objectives, reading assignments, and self-graded tests.

Lesson 1 — *Basic Soil Properties* contains information on soil formation, basic properties of soils, liquid limit, plastic limit, shrinkage limit, and soil size and shape.

Lesson 2 — *Soil Surveys; Soil Classification* covers soil surveys, the Unified Soil Classification System, and field unification of soils.

Lesson 3 — *Soil Compaction* covers moisture-density relationships, optimum moisture content, compaction, and trafficability.

Lesson 4 — *Strength Design Using California Bearing Ratio (CBR)* explains the development of the California Bearing Ratio (CBR), laboratory tests, field in-place tests, design CBR and soil CBR.

Lesson 5 — *Flexible Pavement Structure* includes consideration in flexible pavements, airfield categories, airfield design, and road design.

Lesson 6 — *Frost Action and Permafrost* explains the principles of frost action, the effects of frost action, counteractive techniques and design, and permafrost.

Lesson 7 — *Improvement of Soil Characteristics* explains moisture control blending, chemical bituminous stabilization, dust palliatives and waterproofing.

Lesson 8 — *Construction Methods and Practices* covers reconnaissance, site selection, and planning and construction.

This course is designed for student self-study with objectives, text and self-graded tests and answers. The text is coded and all questions are keyed to the text. No support materials are needed other than the charts provided in the text. A final examination consisting of 50 multiple choice questions is included, but no answers to the questions are available.

Developed by:

United States Army

Development and Review Dates

Unknown

Occupational Area:

Building and Construction

Cost:	**Print Pages**
$5.50	225

Availability:
Military Curriculum Project, The Center for Vocational Education, 1960 Kenny Rd., Columbus, OH 43210

Suggested Background:

None

Target Audiences:

Grades 10-adult

Organization of Materials:

Lesson objectives, text readings, self-tests, answers

Type of Instruction:

Individualized, self-paced

Type of Materials:			**No. of Pages:**	**Average Completion Time:**
Soils Engineering				
Lesson 1	–	Basic Soil Properties	30	Flexible
Lesson 2	–	Soil Surveys; Soil Classification	26	Flexible
Lesson 3	–	Soil Compaction	29	Flexible
Lesson 4	–	Strength Design Using California Bearing Ratio (CBR)	19	Flexible
Lesson 5	–	Flexible Pavement Structure	21	Flexible
Lesson 6	–	Frost Action and Permafrost	27	Flexible
Lesson 7	–	Improvement of Soil Characteristics	31	Flexible
Lesson 8	–	Construction Methods and Practices	9	Flexible
Examination			9	

Supplementary Materials Required:

Charts included in text

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ENGINEER
SUBCOURSE 360-1

SOILS ENGINEERING

CORRESPONDENCE COURSE
U.S. ARMY ENGINEER SCHOOL

FORT BELVOIR, VIRGINIA 3-1

EDITION 1 (JUNE, 1974)

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MODIFICATIONS

Pages 1 - 11 of this publication has (have) been deleted in adapting this material for inclusion in the "Trial Implementation of a Model System to Provide Military Curriculum Materials for Use in Vocational and Technical Education." Deleted material involves extensive use of military forms, procedures, systems, etc. and was not considered appropriate for use in vocational and technical education.

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LESSON 1

BASIC SOIL PROPERTIES

CREDIT HOURS ---------------------------2

TEXT ASSIGNMENT ---------------------Attached memorandum.

MATERIAL REQUIRED ------------------None.

LESSON OBJECTIVE -------------------Upon completion of this lesson you should be able to accomplish the following in the indicated topic areas:

1. **Formation of Soils.** Explain the origin and development of soils to include the effects of mechanical and chemical weathering.

2. **Basic Properties of Soils.** Describe the major size divisions, methods of soil analysis, and the effects of variations in gradation and water content upon soil bearing capacity.

3. **Liquid Limit.** Define liquid limits of a soil and outline the procedures by which it may be determined.

4. **Plastic Limit.** Define plastic limit of a soil and outline the preparations and procedures by which it is determined.

5. **Shrinkage Limit.** Define shrinkage limit and outline the preparations and procedures by which it is determined.

6. **Soil Size and Shape.** Explain the effect of size and shape of particles on the stability and bearing strength of soil.

ATTACHED MEMORANDUM

1-1. DEFINITION

The term soil, as used by engineers, refers to the entire unconsolidated material that overlies bedrock and is clearly distinguishable from bedrock. Soil is composed principally of the disintegrated and decomposed products of rock. It contains air and water as well as organic matter derived from the decomposition of plants and animals. Bedrock, for purposes of this definition, is considered to be the solid part of the earth's crust, consisting of massive formations broken only by occasional structural failures.

1-2. FORMATION OF SOILS

a. **Parent material.**

(1) **Principles.** The crust of the earth, generally regarded as a layer 30 to 50 miles thick, is composed of a variety of solid materials. The most common and widespread of these solids is a substance known as rock. Rock is composed of numerous natural compounds called minerals, and is concealed to a large extent by a thin veneer of loose material called soil by the engineer. In a broad sense, rocks are aggregates of minerals. To the engineer, the term rock signifies firm and coherent or consolidated substances that cannot normally be excavated by manual methods alone. Based upon their principal mode of origin, rocks are grouped into three large classes: igneous, sedimentary, and metamorphic.

(2) **Igneous rock.** Igneous rocks are those which are formed from a molten material called magma which is found beneath the earth's crust. This material exists as a dense fluid under extreme conditions of high temperature and pressure. When the shell which confines the magma is ruptured from internal strain, this molten material may then escape until a stable condition is again established. In this process the magma will normally reach the earth's surface, a region of lower temperature, solidify, and form extrusive igneous rock. From the standpoint of formation, perhaps the simplest types of igneous rocks are those produced by this typical volcanic action. Many deposits of this extrusive type are located in the United States (where volcanoes are no longer significantly active) and in many other areas throughout the world. More commonly, however, the magma, forced upward through the crust, becomes trapped before reaching the surface. In these cases, cooling proceeds at a slower rate. This decreased rate of cooling, as well as the different materials with which the molten mass may come into contact during the cooling process, affect the type of grain structure developed. The type of rock formed is named intrusive to differentiate it from the volcanic (extrusive) type. Some examples of igneous rocks are granite, basalt, and rhyolite.

(3) **Sedimentary rock.** Sedimentary rocks are produced by the accumulation and cementation of preexisting rock particles and/or the remains of plants and animals. Particles forming sedimentary rocks are deposited principally by water, but also as a result of wind or ice action. Cementation may be a result of the action of clay or a chemical such as calcium carbonate. Examples of sedimentary rocks include sandstone, shale, limestone, and coral.

(4) **Metamorphic rock.** Existing rocks, subjected to heat and pressure within the earth, may be changed into new and completely different types of rock. Where such a change occurs the resulting material is called a metamorphic rock. Metamorphic rocks include quartzite (metamorphosed sandstone), slate (metamorphosed shale), marble (metamorphosed limestone), and schist.

b. **Weathering.**

(1) **Principles.** When exposed for sufficient length of time to the atmosphere, all rocks undergo disintegration and decomposition and are ultimately converted into a loose mixture of sand, gravel, and finer material. The changes involved in the weathering of rocks are partly physical and partly chemical. The processes involved may be simple or complex and are confined almost entirely to the zone of weathering, which extends from the surface of the ground to the level of ground water. They are wholly the result of atmospheric agents, moisture, and gases which penetrate the rock and cause disintegration and decay.

(2) **Mechanical weathering.**

(a) **Principles.** Mechanical weathering is the name given to a **disintegration** process in which the original rock is broken up into smaller particles without the identity of the particles being lost. The primary agents producing mechanical weathering are water, ice, vegetation, temperature, and wind.

(b) **Water.** The erosive characteristics of water have long been known and understood, and frequently are the principal factors in disintegration of many rocks. In addition to the erosive characteristics of a moving stream, many large particles are rolled along the bottom of the stream by the current. Others are carried in suspension. The moving particles are constantly coming in contact with each other and these impact forces accelerate the breakdown process.

(c) **Ice.** A second process of mechanical weathering occurs with glacial action. Glaciers carry large amounts of material produced by the scouring action accompanying their movement. These deposits frequently provide sources of material for use in construction. They also affect water-supply and foundation problems in which the engineer is vitally interested.

(d) **Temperature.** Temperature is another factor in mechanical disintegration where the differential expansion of the minerals of which the rock is made causes cracks and, eventually, breaks, to occur. Freezing temperatures cause water in the pores and crevices of rocks to freeze. Water undergoes a 9 percent volume increase when it freezes, and increase in volume of the water within the rock exerts high stresses on the rock which may cause it to crack. Repeated cycles of freezing and thawing cause the rock to disintegrate.

(e) **Vegetation.** Vegetation is a factor in physical weathering. The physical action is caused by the penetration of roots into cracks and crevices of rocks. They wedge the rock apart and at times dislodge fragments of parent material. As the plant grows, pressures of several tons per square foot are frequently exerted by its expanding roots.

(f) **Wind.** Wind and small sand particles produce surprisingly rapid abrasion of bare rocks. Wind is therefore one of the agents of mechanical weathering.

(3) **Chemical weathering.** Chemical weathering is the process in which rock is subject to **decomposition** by the chemical agents in the soil. Chemical weathering varies with climatic conditions, types and abundance of vegetation, and chemical content of water. Moisture is an important agent of chemical decay, and acts indirectly as a bearer of certain gases and acids which it brings in contact with exposed materials. Normal atmospheric air consists chiefly of a mixture of nitrogen and oxygen with smaller quantities of other substances such as water vapor and carbon dioxide. Oxygen, carbon dioxide, and water are by far the most important chemically active substances found in the air. Gaseous solutions of oxygen and carbon dioxide which occur in water, in addition to variable amounts of different carbonates of the alkali earths, are effective weathering agents. Disintegration (physical weathering) and decomposition (chemical weathering) are usually concurrent, but in a given locality one may predominate over the other. In arctic regions, and the arid regions of the west, disintegration is the dominant process by which rock masses are broken down. In tropical regions, decomposition is the most important process. In any case, the forces acting on a rock will determine the type and influence the properties of the soil formed.

1-3. GRADATION

a. **Principles.** Soils may be divided into several different groups on the basis of the size of particles included in each group. The scale used in the Unified Soil Classification System is shown in table 1-1. Several methods may be employed to determine the size of soil particles contained in a soil mass and the distribution of particle sizes. Included among these methods are washed sieve analysis, wet mechanical analysis, and combined mechanical analysis.

TABLE 1-1. *Size Divisions*

Size-group name	Definitive Sieve Sizes	
	Passing	Retained On
Cobbles	No max. size	3-inch
Gravel	3-inch	No. 4
Sand	No. 4	No. 200
Fines (silt or clay)	No. 200	To include the smallest particles.

b. **Sieve analysis.** Separation of the soil into its fractions may be done by first weighing the total oven dry soil sample and then shaking the dry loose material through a nest of sieves of increasing fineness, i.e. successively smaller openings, as indicated in figure 1-1. The sieve analysis may be performed directly upon soils which contain little or no fines, e.g. a clean sand, or soils in which the fines may be readily separated from the coarser particles. Soils which have little dry strength and can be crushed easily in the fingers would generally fall in the latter category. If the character of the fines is such that the fine material adheres to the coarser particles and is not removed by dry sieving action, the sample is soaked in water and washed over the No. 200 sieve. The soil re-

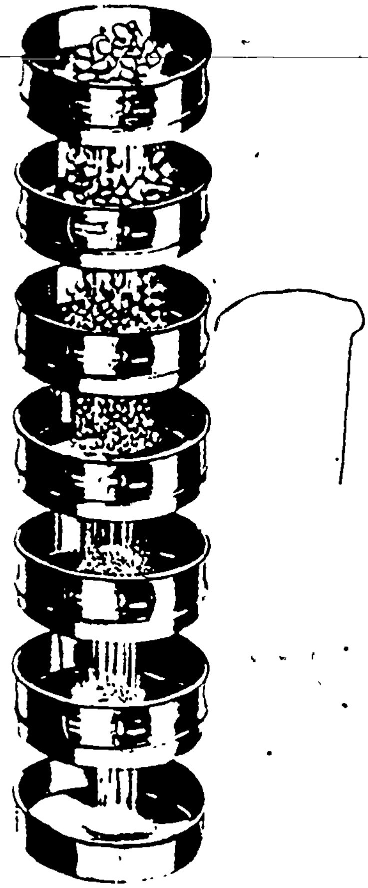

Figure 1-1. Dry sieve analysis.

tained on the No. 200 sieve is then ovendried, the washing loss (fines) determined, and the analysis conducted as before. Sieves which are commonly used by the military engineer have square openings and are designated as 2-, 1½-, 1-, ¾-, and ¼-inch sieves and U. S. Standard No. 4, 10, 40, 60, 100, and 200 sieves. A No. 4 U. S. Standard sieve has 4 openings per lineal inch or 16 openings per square inch, etc.

c. **Wet mechanical analysis.** The practical lower limit for the use of sieves is the No. 200 sieve, which has openings which are 0.074 millimeter square, and a total of 40,000 openings per square inch. However, it is sometimes desirable to determine the distribution of particle sizes below the No. 200 sieve. This may be determined by a process known as decantation or wet mechanical analysis, which employs the principle of sedimentation, i.e. that grains of different sizes fall through a liquid at different velocities. The wet mechanical analysis is not a normal field laboratory test and is not particularly important in military construction except, as will be seen later, that the percentage of particles finer than 0.02 millimeter has a direct bearing on the susceptibility of a soil to frost action.

d. **Combined mechanical analysis.** The procedures which have been described in b and c above are frequently combined to give a more complete picture of grain-size distribution. The procedure is then designated as a combined mechanical analysis.

e. **Recording and reporting.** The results of a sieve analysis may be recorded either in tabular form on DD Form 1206 (fig 1-2) or graphically on DD Form 1207 (fig 1-3). The tabular form is best used when comparing results to a set of specifications. The graphical form permits the plotting of a grain-size distribution curve. This curve affords ready visualization of 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 Unified Soil Classification System to classify sand and gravels as to their gradation characteristics, are determined below.

f. **Gradation and associated factors.**

(1) **Effective size and uniformity coefficient.** The grain size which corresponds to 10 percent on a grain-size distribution curve of the type of figure 1-3 is called Hazen's **effective size** and is designated by the symbol D_{10}. The **uniformity coefficient** is defined as the ratio between the grain diameter corresponding to 60 percent on the curve (D_{60}) and D_{10}. Hence, $C_u = D_{60}/D_{10}$.

(2) **Coefficient of curvature.** Another quantity which may be used to judge the gradation of a soil is the coefficient of curvature, C_c. It is given by the expression:

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

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SIEVE ANALYSIS DATA

DATE: 11 July 1966

PROJECT: FRANCE THEATER

EXCAVATION NUMBER: 2

SAMPLE NUMBER:

DESCRIPTION OF SAMPLE: 25 lb bag sample.

PREWASHED: YES ☐ NO ☒

WEIGHT ORIGINAL SAMPLE (gm.): 359.1

WEIGHT AFTER PREWASHING[1] (gm.):

WASHING LOSS[1] (gm.):

SIEVE OR SCREEN a	WEIGHT RETAINED ON SIEVE (gm) b	PASSING SIEVE WEIGHT (gm.) c	PASSING SIEVE PERCENT d
*	0		100
1½	0	359.1	100.0
3/4	0	359.1	100.0
No. 4	51.0	308.1	85.8
10	40.9	267.2	74.4
20	83.3	183.9	51.2
40	75.4	108.5	30.2
100	49.9	58.6	16.3
NUMBER 200	47.4	11.2	3.1

A. WEIGHT SIEVED THROUGH NO. 200 (gm.): 11.2

B. WASHING LOSS[1] (gm): —

TOTAL PASSING NO. 200 (gm) (A + B): 11.2

TOTAL WEIGHT OF FRACTIONS (Total of all entries in Col b) (gm): 359.1

ERROR (Original weight - total weight of fractions)(gm.): 359.1 - 359.1 = 0.0

PERCENT ERROR $\left(\frac{\text{Error (gm.)}}{\text{Original weight (gm.)}} \times 100\right)$ 0%

REMARKS

TECHNICIAN (Signature): Paul Masain

COMPUTED BY (Signature): Paul Masain

CHECKED BY (Signature): John Stark

[1] For prewashed samples only. * Maximum particle size.

DD FORM 1 AUG 57 1206

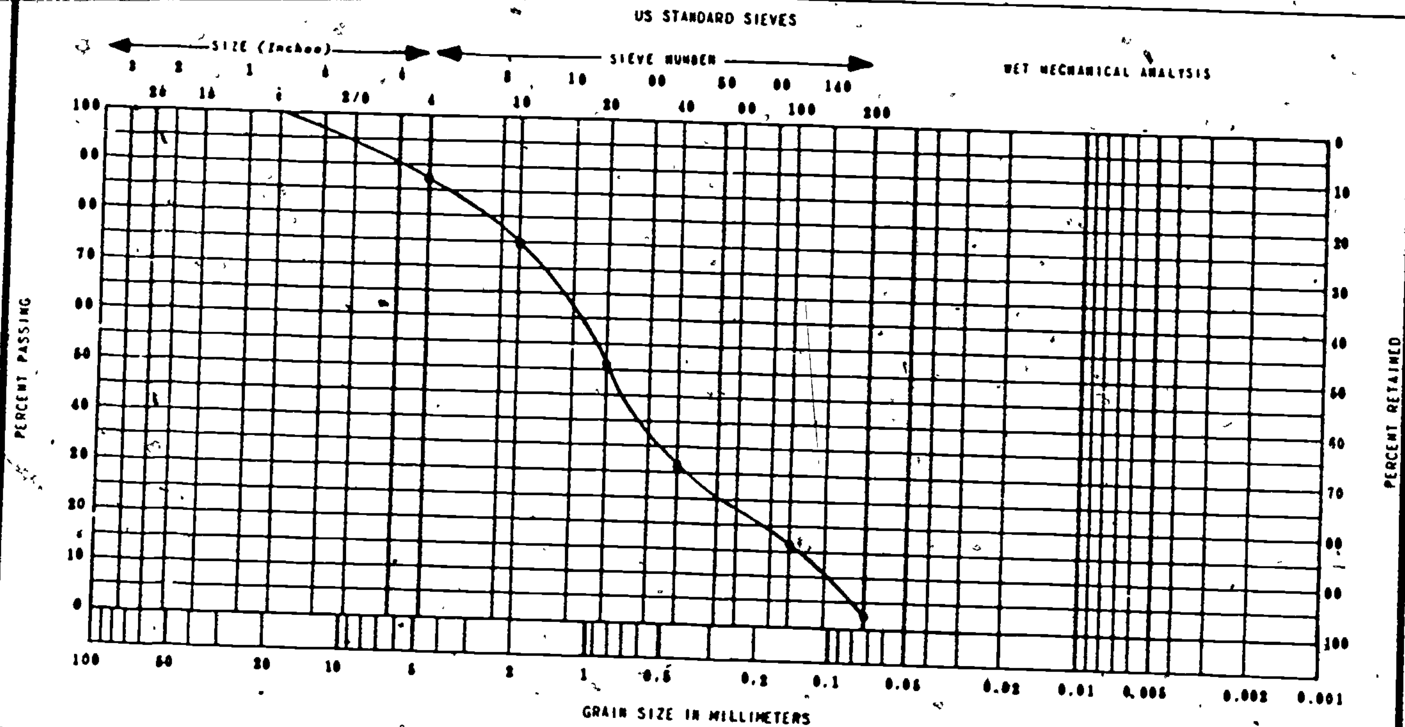
GPO 931156

Figure 1-2. Data sheet, example of dry sieve analysis.

GRAIN SIZE DISTRIBUTION GRAPH - AGGREGATE GRADING CHART

PROJECT: FRAUNCE THEATER

DATE: 11 July 1966



EXCAVATION NUMBER	SAMPLE NUMBER	NATURAL % MOISTURE	w_L	w_P	I_P	CLASSIFICATION	REMARKS
2	FT-P1-V						

TECHNICIAN (Signature): Paul Massain

PLOTTED BY (Signature): Paul Massain

CHECKED BY (Signature): John Stark

DD FORM 1 AUG 57 1207

GPO 931100

Figure 1-3. Grain-size distribution curve from sieve analysis.

D_{60} and D_{10} have meanings previously given, and D_{30} is the grain diameter corresponding to 30 percent on the grain size distribution curve.

g. Well-graded soils. A well-graded soil may be defined as one having a good representation of all particle sizes from largest to smallest and containing few fines. Figure 1-4 illustrates symbolically what is meant by the term well-graded. In the Unified Soil Classification System, well-graded gravels must have a C_u value greater than 4, and well-graded sands must have a C_u value greater than 6. Both sands and gravels must have a value for the coefficient of curvature between 1 and 3 to be well-graded. Those not meeting these conditions are termed poorly graded soils.

h. Poorly graded soils.

(1) **Principles.** There are two types of poorly graded soils, uniformly graded and gap-graded.

(2) **Uniformly graded.** A uniformly graded soil consists predominately of particles nearly uniform in size (fig 1-5).

(3) **Gap-graded.** A gap-graded soil contains some large and some small particles but the continuity of gradation is broken by the absence of some sizes of particles (fig 1-6).

i. Reading of grain-size distribution graph. Figure 1-7 shows typical examples of well-graded and poorly-graded sand and gravel. Well-graded soils (GW and SW curves in figure 1-7) would be represented by a long curve spanning a large size range with a constant or gently varying slope. Uniformly graded soils (SP curve in figure 1-7) would be represented by a steeply sloping curve spanning a narrow size range, and the curve for a gap-graded soil (GP curve in figure 1-7) will flatten out in the area of the grain size deficiency.

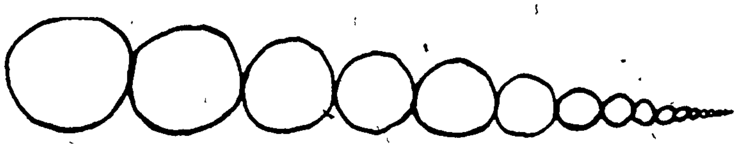


Figure 1-4. Well-graded soil.

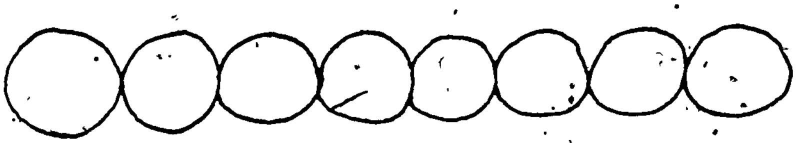


Figure 1-5. Uniformly graded soil.

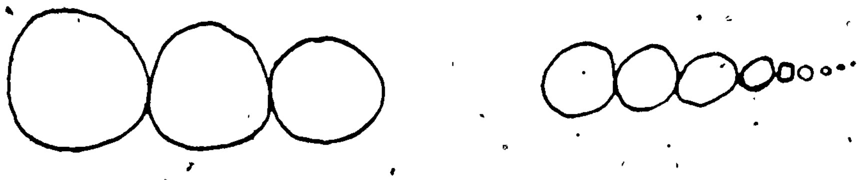
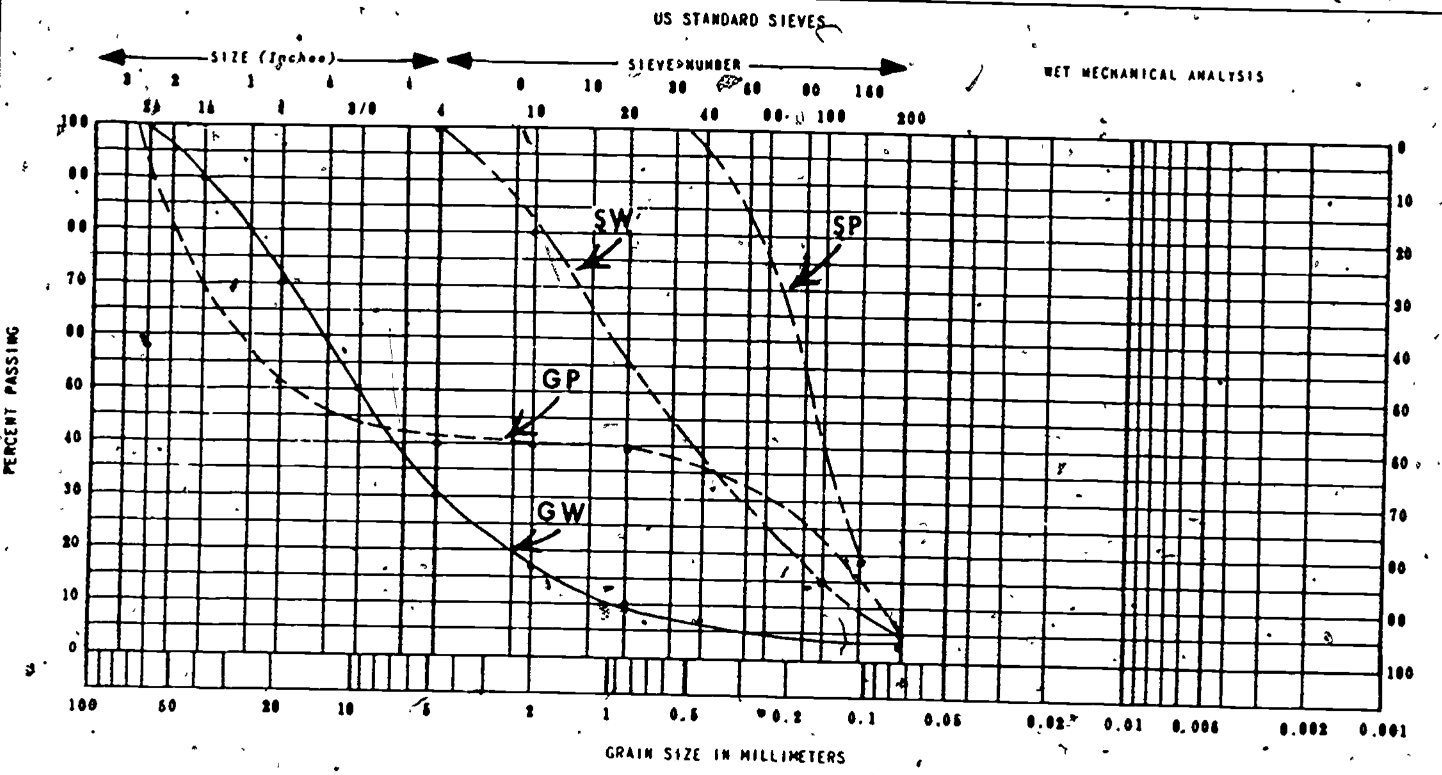


Figure 1-6. Gap-graded soil.

GRAIN SIZE DISTRIBUTION GRAPH - AGGREGATE GRADING CHART | DATE

PROJECT



EXCAVATION NUMBER	SAMPLE NUMBER	NATURAL % MOISTURE	w_L	w_P	I_P	CLASSIFICATION	REMARKS

TECHNICIAN (Signature)	PLOTTED BY (Signature)	CHECKED BY (Signature)

DD FORM 1 AUG 57 1207

Figure 1-7. Typical well-graded and poorly-graded soils.

GPO 943037

j. **Effect of gradation on bearing capacity.** Coarse materials that are well-graded are usually preferable for bearing from an engineering standpoint, since good gradation usually is conducive to high density and stability. Specifications controlling the percentage in the various size groups making up a well-graded soil sample have been established from engineering performance and testing. By proportioning components to obtain a well-graded soil, it is possible to provide for maximum density. Such proportioning develops an "interlocking" of particles with smaller particles filling the voids of larger particles, which makes the soil stronger and more capable of supporting heavier loads. Since the particles are "form-fitted", the best load distribution downward will be realized. When each particle is surrounded and "locked" by other particles the grain-to-grain contact area is increased and the tendency for displacement of the individual grains is minimized.

1-4. SOURCES OF WATER IN SOILS

a. **Surface water.** Water enters through soil surfaces unless some means is taken to seal that surface and prevent such entry. Even sealed surfaces may often have cracks, joints, or fissures through which water may be admitted. In concrete pavements, for instance, the contraction and expansion joints are points of entry for water. After the pavement has been subjected to many cycles of expansion and contraction, the joint filler may become imperfect, thereby allowing water to seep into the base course or subgrade. Surface water may also enter from the sides of a paved road or runway even though the surface itself is entirely waterproof. To reduce this effect to a minimum, shoulders and ditches are usually provided with proper slope to carry the water away at an acceptable rate. Immediately following construction, the slopes are nearly always satisfactory. Often, however, if adequate maintenance is not provided throughout the life of the facility, vegetation or sedimentation may reduce drainage efficiency to such an extent that water will frequently be found standing in the drainage ditch. This water may then be absorbed by the base and subgrade.

b. **Subsurface water.**

(1) **Free or gravitational water (controlled by force of gravity).** Water which percolated down from surface sources eventually reaches a depth at which there exists some medium that restricts, to varying degrees, the further percolation of the moisture. This medium is often bedrock, but it can also be a layer of soil, not wholly solid but with such small void spaces that the water that leaves a zone in the soil is not as great as the volume or supply of water added. In time the accumulating water will completely saturate the soil above the restricting medium and fill all the voids with water. When this zone of saturation is under no pressure except atmospheric pressure, it is called free or gravitational water. It will flow through the soil and is resisted only by the friction between the soil grains and the free water. The upper limit of the saturated zone of free water is called the ground-water table, which varies with climatic conditions. During a wet winter the ground-water table will rise. A dry summer might, however, remove the source of further accumulation of water. This would result in a decreased height of the saturated zone, for the free water would then flow downward, through or along its restricting layer.

(2) **Artesian water.** Entrapped gravitational water will many times accumulate under pressure by various causes and form water sources of an artesian nature. This water is generally enclosed between two layers of impermeable soil (clay). An artificial or deliberate disturbance of the top layer will produce an artesian well or may produce a quick or boiling action.

(3) **Capillary moisture (controlled by forces of capillarity).** Another source of moisture in soils results from what might be termed the capillary potential of a soil. Dry soil grains attract moisture in a manner somewhat similar to the way clean glass does. Outward evidence of this attraction of water and glass is seen by observing the meniscus

that forms around the surface of water confined in a drinking glass. Where the meniscus is more confined, as in a small glass tube, it will support a column of water to a considerable height. The diagram in figure 1-8 shows that the more the meniscus is confined, the greater the height of capillary rise (h_c). Capillary action in a soil results in the "capillary fringe" immediately above the ground water table. The height of capillary rise depends upon numerous factors. One worthy of mention should obviously be the type of soil. Since the pore openings in a soil vary with the grain size, a fine-grained soil will develop a higher capillary fringe area than a coarse-grained soil because it can be imagined to act as a lot of very small glass tubes, each having a greatly confined meniscus. In clays, capillary water rises sometimes as high as 30 feet, and in silts as high as 10 feet. When the capillary fringe extends to the natural ground surface, winds and high temperatures help to carry this moisture away and reduce its effect on the soil. Once a pavement or other watertight surface is applied, however, the evaporating effect of the wind and temperature is eliminated or reduced. This explains the accumulation of moisture often found directly beneath an impervious pavement.

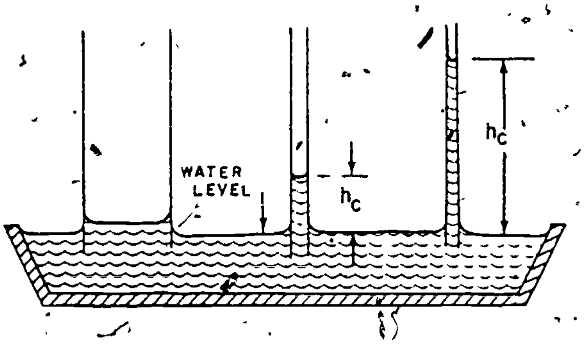


Figure 1-8. Variations in capillary rise.

(4) **Hygroscopic moisture (controlled by forces of absorption).** When wet soil is dried in air in the laboratory, moisture is removed by evaporation until the hygroscopic moisture in the soil is in equilibrium with the moisture vapor in the air. The amount of moisture in air-dried soil, expressed as a percentage of the weight of the dry soil, is called the hygroscopic moisture content. Hygroscopic moisture may be driven off from air-dried soil by heating the material in an oven at 100°C to 110°C (212°F to 230°F) for 24 hours or until constant weight is attained.

1-5. EFFECTS OF MOISTURE ON SOILS

a. Moisture content.

(1) As may be seen from the preceding discussion, some water will be present in most soils. In studies of the effect of water upon soil a term is used that defines the amount of water present in a given soil sample. It is called the "moisture content", and is equal to the weight of water in a soil divided by the weight of the solid mineral grains (referred to as the "weight of dry soil"). When multiplied by 100, the moisture content may be expressed as a percentage. The small letter "w" is usually used to represent this relationship:

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

(2) As an example of how the formula might be applied, suppose a sample of moist soil weighs 110 grams. If the water weighs 10 grams, the weight of dry soil would be 100 grams. The moisture content is then obtained by dividing 10 by 100 and multiplying by 100 to convert to percent.

b. Effects on different types of soils. Coarse-grained soils are much less affected by moisture than are fine-grained soils. First of all coarser soils have larger void openings and, generally speaking, drain more rapidly. Capillarity is no problem in gravels and sands having very little or no fines mixed with them. These soils will not usually retain large amounts of water if they are above the ground-water table. Secondly, since the particles in sandy and gravelly soils are relatively large (in comparison to silt and clay particles) they are, by weight, heavy in comparison to any films of moisture which may surround them. On the other hand, the small, sometimes microscopic particles of a fine-grained

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soil weigh so little that water within the voids has considerable effect on them. Consider the two following phenomena as example of this effect: (1) It is not uncommon for clays to undergo very large volume changes with variations in moisture content. Evidence of this are the shrinkage cracks developed in a lake bed as it dries. (2) Unpaved clay roads, although often hard when sunbaked, will lose stability and turn into mud in a rainstorm. Either of these results is of great importance to an engineer and both are functions of changing water content.

c. **Plasticity.** Not only do clays swell and lose stability when they become wet, but, because of their flat, platelike grain shapes and small size, they also retard the movement or drainage of water through the void spaces between the grains. Since drainage is of great importance (if not of primary importance) in road and airfield construction, the engineer must have some means of determining if clay exists in a soil sample. Plasticity is the property which shows if the sample contains clay.

(1) Plasticity is a property of the fine-grained portion of a soil which permits it, under certain moisture conditions, to be remolded without crumbling or rupturing. A soil or soil fraction is called plastic if, at some water content, it can be rolled out into thin threads. Plasticity could be termed a colloidal property since no mineral possesses plasticity unless it consists of particles of colloidal or clay size. Even then many minerals, such as quartz powder, do not develop plasticity regardless of how small the particles are. All clay minerals, on the other hand, are plastic. Since practically all fine-grained soils contain some clay, most of them are plastic.

(2) The degree of plasticity a soil possesses can be used, therefore, as a general index to its clay content. Sometimes the terms "fat" and "lean" are used to describe the amount of plasticity, a "lean" clay being one that is only slightly plastic because it contains a large proportion of silt or fine sand. In engineering practice the plasticity of a soil is determined by measuring the different states a plastic soil undergoes with changing moisture conditions.

1-6. MOISTURE STATES

a. **Principles.** A fine-grained soil may have different properties at differing moisture contents. A given clay, for instance, may act almost as a liquid, behave as a remoldable plastic, or crumble — each of these characteristics depending only on how much water it contains. Tests were devised by Dr. Atterberg in 1911 to define the water-content ranges of the various states.

b. **Liquid state.** When a typical clay is mixed with a sufficiently large amount of water it will be in a highly saturated, liquid condition, which may be recognized by the fact that the wet soil flows freely under its own weight. The flat, plate-like clay particles then have films of moisture around them that lubricate the particles to the extent that there is practically no shearing resistance or friction between particles.

c. **Plastic state.** Reducing the moisture content of the soil will cause the films of moisture to become thinner. As slight reductions in water content are made, the particles will begin to develop frictional resistance between themselves. Further reductions of moisture content result in still more restriction on the free-flowing character of the particles. As the films of moisture become thinner and thinner their physical properties undergo a definite change. They seem to attach themselves to one another and the particles they surround, giving the soil a plastic character or an ability to be remolded into various shapes without rupturing or crumbling. This plastic condition then can perhaps be pictured as a condition of soil moisture in which the films of water bind the particles together and give the soil mass a cohesive, pliable quality, much the same as modeling clay possesses.

d. **Semisolid state.** Further drying of the clay sample will eventually cause it to become dry to the point that it is no longer pliable. Under deforming pressure applications, the sample will crumble. This condition is called the semisolid state.

e. **Solid state.** The previous discussion stated that one characteristic of clay is that its volume changes with additions and subtractions of water. Shrinkage in soils upon drying is the result of capillary forces which tend to pull the soil grains tightly together. As previously explained, these forces are more effective and of greater magnitude when the pores of the soil are very small. Clays will therefore exhibit the greatest volume variations in drying, since their grains and pores are naturally smaller than those of any other type of soil. As the soil dries out because of evaporation, the surface water disappears and innumerable menisci are created in the voids which are adjacent to the surface of the soil mass. Tensile forces are created in each of these boundaries between water and air. Compressive forces accompanying them act upon the soil structure. In the fine-grained soils, such as clays, the soil structure is compressible and the mass shrinks. As drying continues the mass shrinks to a certain limiting volume, the shrinkage limit. Further drying will not cause a reduction in volume, but may cause cracking as the menisci retreat into the voids. In the clay soils these internal forces become very large and principally account for the rocklike strength of a dried clay mass.

1-7. BOUNDARY MOISTURE CONTENTS

a. **Principles.** The range of moisture contents through which a soil passes while in a particular state or condition has been used by engineers for a considerable time as a basis for estimating the properties of fine-grained soils. In order to determine the magnitude of the water-content range, it becomes necessary to define the boundary moisture contents that divide or separate the various states.

b. **Liquid limit.** A simple way of defining the liquid limit would be to say that it is the moisture content of a fine-grained soil (or the fine-grained portion of a coarse-grained soil) at the point at which it passes from the liquid state into the plastic state. It is the boundary moisture condition between the state in which the soil flows under its own weight and the state in which it can be remolded without crumbling. The test devised by Dr. Atterberg to determine this moisture content value consists of preparing a sample in such a manner that there is a slight resistance to flow developed between the particles and then determining its moisture content. This value for any soil is symbolized by LL or W_L.

c. **Plastic limit.** This limit defines the moisture content at which a fine-grained soil passes into the plastic stage from the semisolid stage or vice versa. The test used here consists of preparing a sample at a water content at which it will begin to crumble after having been rolled into a thread of about ⅛-inch diameter. In this condition the soil does not have enough cohesive quality to consider it as being in the plastic state. The symbol used for the plastic limit is PL or W_P.

d. **Shrinkage limit.** Since this limit is not used nearly as much in engineering practice as the previous two, it will simply be defined here as the water content at which a soil ceases to change in volume upon further loss of water due to drying.

1-8. PRACTICAL VALUE OF UNDERSTANDING MOISTURE STATES AND BOUNDARIES

a. Having available the empirical tests of Dr. Atterberg which enable the determination of the plastic limit and liquid limit, it is possible to determine the magnitude of the moisture content range through which a soil is in the plastic state. This is probably more clearly evident when shown diagramatically. Taking a fine-grained soil sample (A, fig 1-9), consider it to be at varying moisture contents which increase from left to right. From inspection of such a sample, the portion to the left would have dry surfaces and be in the solid state. Moving further to the right the sample would pass into the semisolid, plastic, and liquid states, respectively, as the moisture content of the sample increased. Determination of the boundary water contents could be represented as shown in B, figure 1-9. It should be evident then that the range of moisture contents through which the soil is of plastic consistency would be equal to the difference between the liquid-limit moisture content and the plastic-limit moisture content. In this particular case, this range

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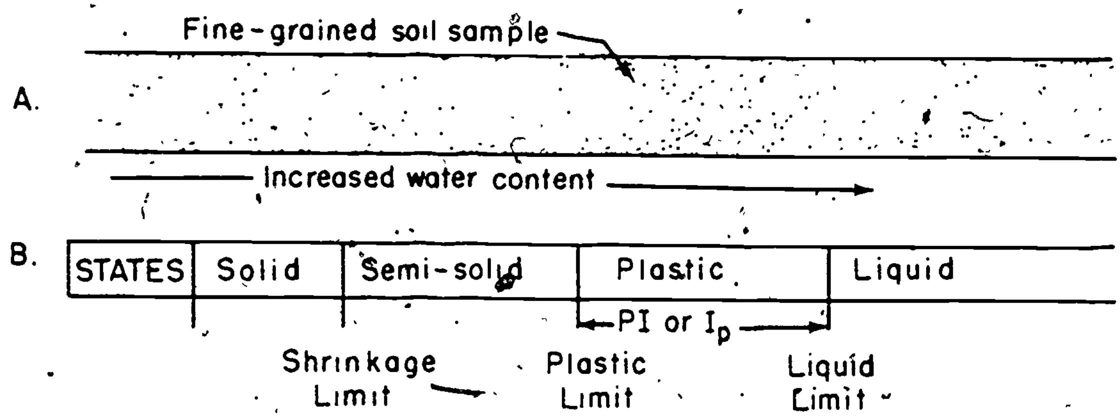

Figure 1-9. Moisture states of a typical fine-grained soil.

of moisture content is defined as the plasticity index:

Plasticity Index = Liquid Limit − Plastic Limit;

$PI = LL - PL$ or $I_P = W_L - W_P$

b. If a soil then has a liquid limit of 25 and a plastic limit of 10, its plasticity index would be 15. In practice, engineers use the plasticity index coupled with the liquid limit as indexes to the properties of a soil. Since clay is the material which causes a soil to be plastic, it would be expected that a fine sand-silt-clay mixture which is predominantly clay would have a larger plasticity index than a similar mixture which is predominantly silt or fine sand. The former would also be expected to produce greater volume changes with varying moisture contents and greater loss of stability when wet than the more "lean" mixture. A nonplastic soil ($PI = 0$), ~~on the other~~ hand, would be even less affected by moisture than a "lean" but plastic soil.

c. There are very few natural soils which properly support heavy aircraft or even vehicular traffic by themselves. The existing soils at a site must be leveled to form the subgrade which then has to be protected by some type of base course and pavement to prevent failure under loading. In order that a base course provide adequate protection for the subgrade it should consist of a well-graded, angular, dense gravel. In addition the base course should drain freely and contain but small amounts of clay, so that it will not lose stability when wet. Provision for the last two requirements can be made by limiting the values of the plasticity index and liquid limit of soils used in constructing the base course. For example, the engineer will find again and again in engineering literature the following specifications for base-course materials for high-quality flexible pavements:

"Base-course materials which are provided with an abrasive wearing surface should have a plasticity index less than 5 and a liquid limit no greater than 25. If no bituminous surface treatment or comparable wearing surface is used, the materials should have a plasticity index between 4 and 9 and a liquid limit not greater than 35." (This latter group of specifications provides for enough clay binder to keep the surface material from ravelling under traffic.)

1-9. LIQUID LIMIT TEST PREPARATION

a. **Definition.** The liquid limit of a soil is the water content at the boundary between the liquid and the plastic states expressed as a percentage of the oven-dried soil and reported as a whole number. This boundary is arbitrarily defined as the water content at which a soil mass, placed in a standard cup and divided into two sections by a standard grooving tool, will make contact for a distance of 0.5 inch when the cup is dropped 25 times for a distance of 1 centimeter (0.3937 inch) at the rate of two drops per second.

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b. **Apparatus.** The apparatus for a liquid limit test consists of the following:

- Balance
- Boxes, stainless steel, 2-inch diameter by 7/8 inch high
- Dishes, evaporating
- Filler, battery
- Guard, wind, for 200-gram balance
- Liquid limit machine, complete with brass cup and grooving tool
- Mortar, porcelain
- Oven, electric
- Pan, sieve, 8-inch frame
- Pestle, rubber-tipped
- Screwdriver
- Sieve, testing, 8-inch frame, full height: U. S. No. 40
- Spatula
- Tongs, crucible

Some of this equipment is shown in figure 1-10.

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

(1) The liquid limit is performed on material finer than the No. 40 sieve.

(2) Samples should be large enough to produce ~~150 to 200~~ grams of material for testing.

(3) The selected sample must not be subjected either to air-drying or oven-drying before testing.

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

(5) 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.

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

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

(8) The combined (passing No. 40) sample is dried to a putty-like consistency by decantation or evaporation, being careful to prevent caking or lumping during the process.

(9) Neither chemical substances nor dry soil are added to hasten settlement nor to speed drying.

(10) The soil sample should be thoroughly mixed just before starting the test.

d. **Checking and adjusting the liquid limit device.** The liquid limit testing machine must be inspected prior to the test to determine that it is in good working condition.

(1) Check the pin connecting the cup for wear. It must not be worn to the point where it permits sideplay of the cup.

(2) Check the screws connecting the cup to the hangar arm. They should be tight.

(3) Check the cup for wear. Long usage will develop a groove through the cup. This can be felt by running the fingers over the inside surface. If worn to this extent, it should be replaced.

(4) Check the grooving tool for wear. The dimensions which control the size of the groove are shown in figure 1-11. The tool should be discarded when the point width exceeds 0.086 inch.

(5) 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. Using the gage, the height is adjusted by means of the adjusting screw (fig 1-11) until the point on the cup that strikes the base is exactly 1 centimeter (0.394 inch) above the base. Tighten screws to secure the adjustment and

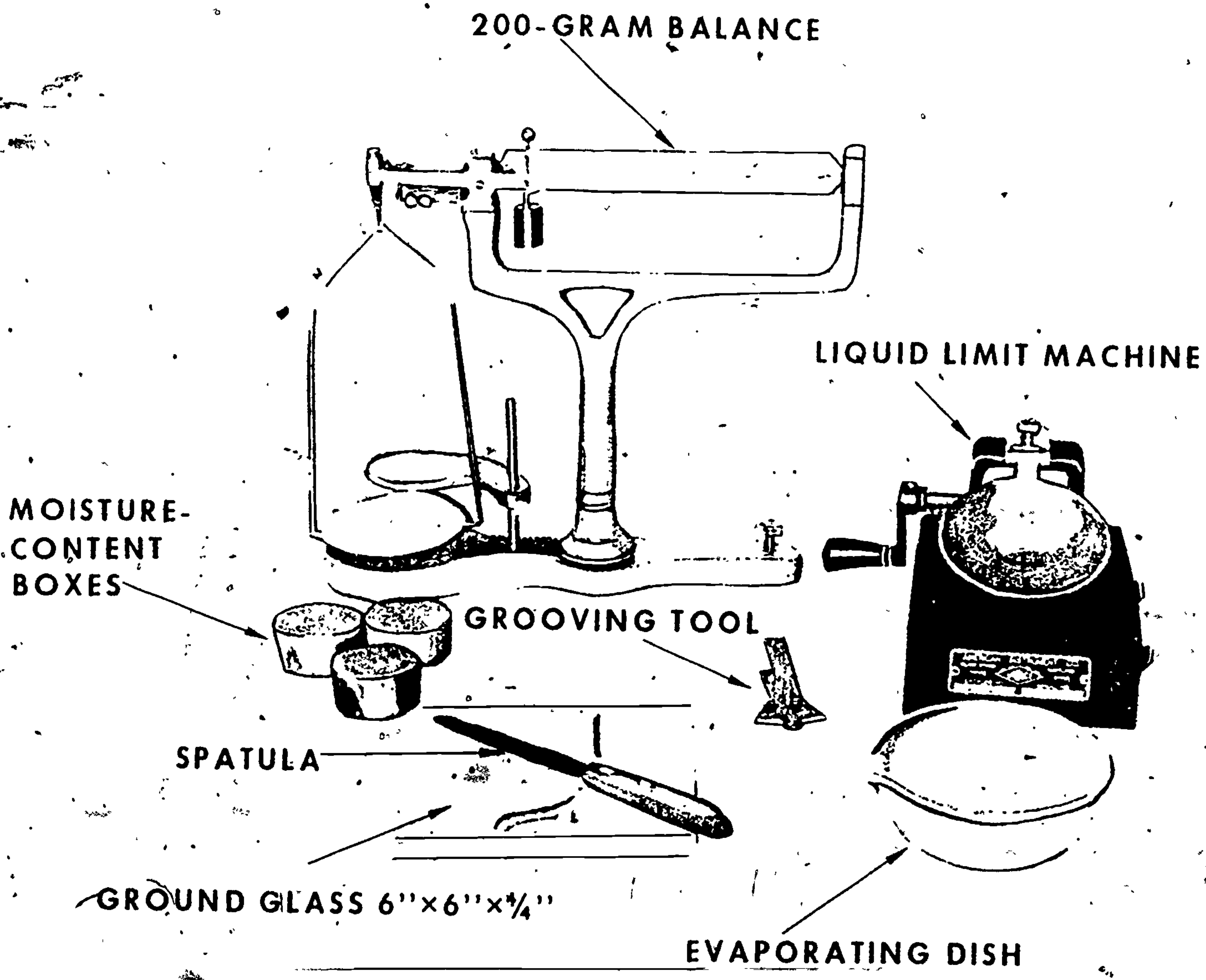
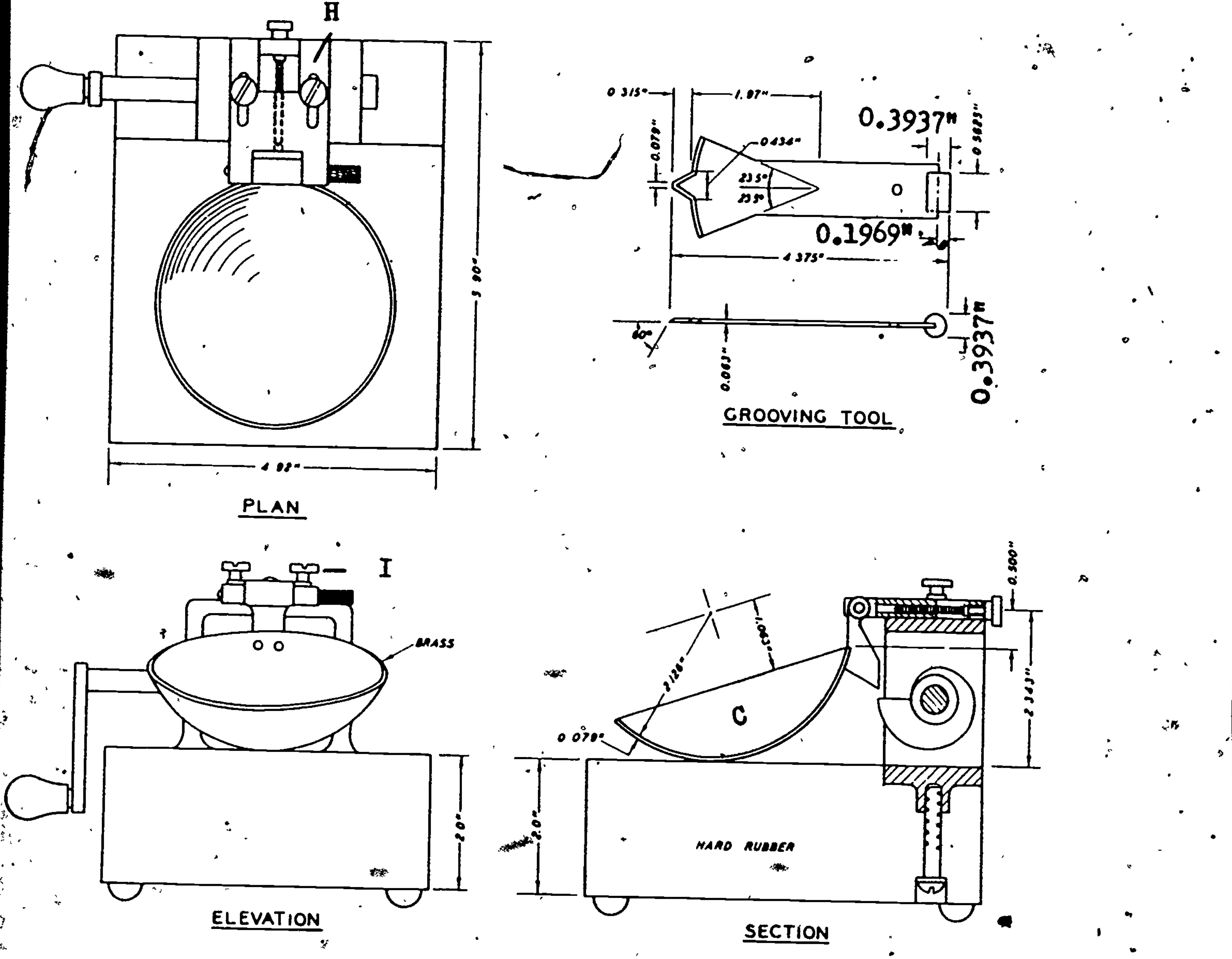


Figure 1-10. Apparatus for determining Atterberg limits.



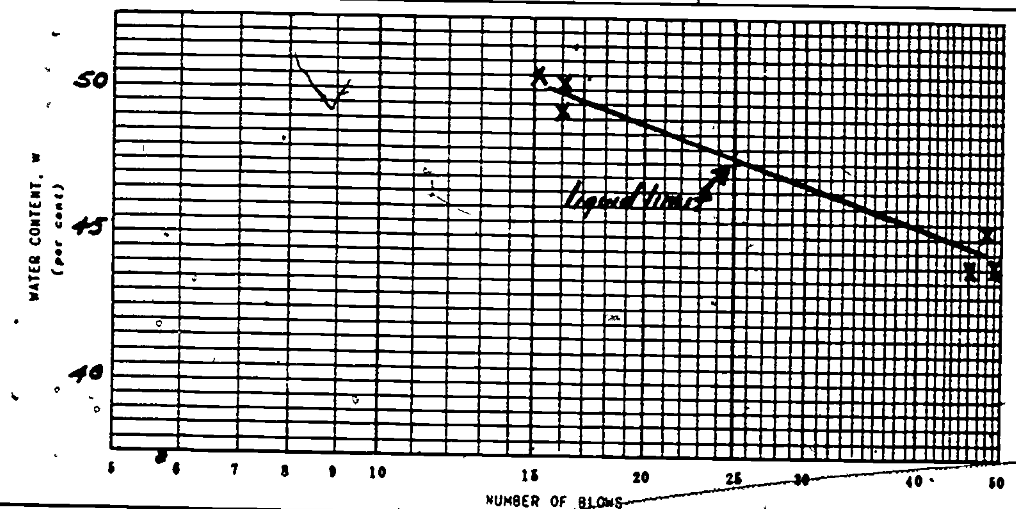
Figule 1-11. Mechanical liquid-limit device.

ATTERBERG LIMITS DETERMINATION		DATE 8 Sept 1968
PROJECT Britt Airfield	EXCAVATION NUMBER 1	SAMPLE NUMBER B-P3-1

LIQUID LIMIT, w_L

RUN NUMBER	1	2	3	4	5	6
TARE NUMBER	41	42	43	44	45	46
A. WEIGHT OF WET SOIL + TARE	41.21	37.40	40.92	45.19	44.60	68.06
B. WEIGHT OF DRY SOIL + TARE	38.38	35.54	38.15	41.52	41.07	63.87
C. WEIGHT OF WATER, W_w (A.-B.)	2.83	1.86	2.77	3.67	3.53	4.19
D. WEIGHT OF TARE	32.06	31.31	31.88	34.29	34.04	55.46
E. WEIGHT OF DRY SOIL, W_s (B.-D.)	6.32	4.23	6.27	7.23	7.03	8.41
WATER CONTENT, $w = (\frac{W_w}{W_s} \times 100)$	44.8	44.0	44.2	50.7	50.2	49.8
NUMBER OF BLOWS	47-49-48	46-46	48-49	14-16-15	15-17-16	15-18-16

w_L 48	w_P 23	I_P (w_L-w_P) 25



PLASTIC LIMIT, w_P

RUN NUMBER	1	2	3		NATURAL WATER CONTENT
TARE NUMBER	47	48	49		50
F. WEIGHT OF WET SOIL + TARE	56.30	55.20	60.71		58.86
G. WEIGHT OF DRY SOIL + TARE	55.90	54.87	60.08		57.82
H. WEIGHT OF WATER, W_w (F.-G.)	0.40	0.33	0.63		1.04
I. WEIGHT OF TARE	54.10	53.40	57.43		55.02
J. WEIGHT OF DRY SOIL, W_s (G.-I.)	1.80	1.47	2.65		2.80
WATER CONTENT, $w = (\frac{W_w}{W_s} \times 100)$	22.2	22.4	23.7		37.1
PLASTIC LIMIT, I_p (Average w)			23		

REMARKS

TECHNICIAN (Signature)	COMPUTED BY (Signature)	CHECKED BY (Signature)
William Ball	William Ball	Joseph Craven

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Figure 1-12. Data sheet Atterberg limits determination.

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.

1-10. LIQUID LIMIT TEST PROCEDURE

With the sample prepared and the device checked, the test procedure consists of the following steps:

a. Record all identifying information for the sample on the top section of the data sheet (fig 1-12).

b. Place 50 to 80 grams of the prepared sample in the cup and level it off to a depth of approximately 1 centimeter (fig 1-13). 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.

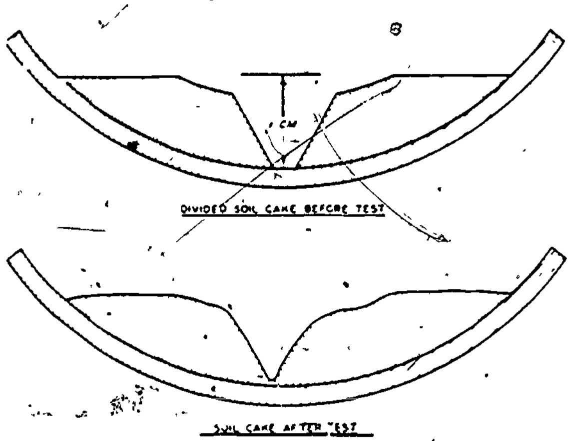

Figure 1-13. Diagram illustrating liquid limit test.

c. Holding the grooving tool and the cup, make the groove by drawing the tool (beveled edge forward) down through the sample. The grooving tool should always be perpendicular to the dish at the point of contact.

(1) More than one stroke may be necessary to make the groove. 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.

(2) Enough soil should be in the cup so that when the groove is completed, the shoulder of the grooving tool has removed soil for the length of about 1 inch.

(3) 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 and shaped with the grooving tool.

(4) 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 loss of time in trying to scrape or wash it clean.

d. 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. Record the number of blows.

e. Remove about 10 grams of soil from the cup by drawing the spatula from edge to edge perpendicular to the groove and through the portion of the cake that closed. This sample is used to determine the water content. All weighing should be accurate to $\pm$ 0.01 gram and the water content computed to one decimal place.

f. Transfer the remaining soil in the cup to the mixing dish. Wash and dry the cup and grooving tool. Repeat steps in b through e above for three additional samples, each of which has had the water content adjusted by drying. Drying is accomplished by continued mixing with the spatula and may be 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 result in a series with some more and some less than 25 blows which close the groove. Preferably, the range should be from

15 or more to 35 or less with two tests between 15 and 25 blows and two tests between 25 and 35 blows.

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

h. Computing the liquid limit is done graphically by plotting a "flow curve" with the number of blows and water content as coordinates (fig 1-12). 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 in 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 define a straight line, additional check runs should be made and new points plotted. The **liquid limit** is the water content corresponding to the point where the drawn 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. Example (fig 1-12): After plotting these six points and drawing a straight line as nearly as possible through them, the water content corresponding to the 25-blow coordinate is 48.

1-11 SIMPLIFIED LIQUID LIMIT TEST

a. **Principles.** The simplified 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 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.

b. **Preparation.** The apparatus for the simplified test is the same as that listed previously. The sample is prepared as for a standard test except that its consistency is controlled to result in 20 to 31 blows to cause closure.

c. **Procedure.** Proceed with the test as described previously. 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 the addition of 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 disagreement is greater than two blows, the mixing was insufficient and the entire test must be repeated.

d. **Liquid limit determination.** The liquid limit can be determined graphically by drawing a line through the one plotted point at the predetermined slope or by using the following equation:

$$LL = W_N \left(\frac{N}{25}\right) \text{Tan } \beta$$

Where:

W_N = water content at N blows

N = Number of blows to close the groove

Tan β = slope of the flow line

1-12. PLASTIC LIMIT TEST PREPARATION

a. **Definition.** 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 1/8 inch in diameter.

b. **Apparatus.** The apparatus for the plastic limit test consists of the following:

(1) 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.

(2) Spatula.

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

(4) Balance, sensitive to 0.01 gram.

(5) Oven (preferably automatic controlled for 105° to 110° C).

(6) Evaporating dish.

c. **Preparation.** Approximately 20 grams of material is required for the plastic limit test. The sample is prepared as described for a liquid limit test or it is the material remaining after the liquid limit.

1-13. PLASTIC LIMIT TEST PROCEDURE

The test procedures consists of the following steps:

a. Record all information pertaining to the sample, weights, and computations on a data sheet similar to fig 1-12.

b. 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.

c. Shape the test sample into an ellipsoid and roll it between the fingers and rolling surface (fig 1-14) using enough pressure to roll the soil mass into a thread 1/8 inch in diameter. The rate of rolling should be between 80 and 90 strokes per minute. One stroke is considered a complete forward and back motion returning to the starting position.

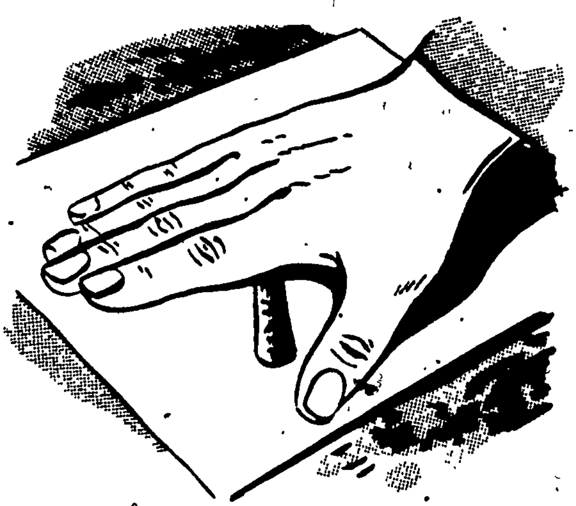

Figure 1-14. Rolling the soil, plastic limit test.

d. When the thread diameter reaches 1/8 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 (fig 1-15). The plastic limit has been reached when the thread crumbles regardless of diameter. Do not speed up crumbling by reducing the rate of rolling, the hand pressure, or both, and do not roll to less than 1/8-inch diameter.

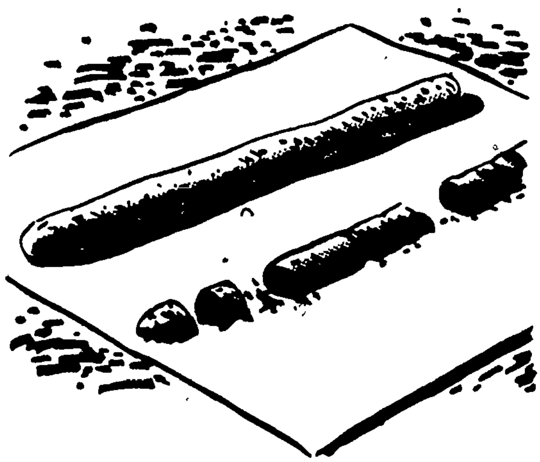

Figure 1-15. Rolled threads, uncrumbled and crumbled.

e. Collect the crumbled portions and determine the water content.

f. Repeat the process with an additional portion of the prepared sample to obtain an average value for the plastic limit. The two test should agree within $\pm$ 1 percent or the tests must be repeated.

g. Record 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.

h. Computing the plastic limit is done mathematically, as follows:
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 (fig 1-12). Plasticity index = liquid limit - plastic limit

or

$PI = LL - PL$

or

$I_P = W_L - W_P$

Example (fig 1-12);

$LL = 48$

$PL = 23$

$PI = LL - PL$

$= 48 - 23 = 25$

1-14. SHRINKAGE LIMIT TEST PREPARATION

a. **Definition.** The shrinkage limit of a soil is the water content expressed as a percentage of ovendried soil, at which further loss in moisture will not cause a decrease in the volume. The shrinkage ratio, R, and linear shrinkage, L_S, are usually determined as part of this test. The shrinkage ratio is defined as the ratio between a given volume change and the corresponding change in water above the shrinkage limit. Linear shrinkage is the decrease in one dimension of a soil mass when the water content is reduced from a given value to the shrinkage limit.

b. **Apparatus.** Equipment for this test, except item (6) below, is not in the soil test set. The apparatus should consist of the following:

(1) Evaporating dish, porcelain.

(2) Shrinkage dish, porcelain or monel metal, 1¾-inch diameter and ½-inch high.

(3) Glass cup, about 2-inch diameter and 1-inch high with a ground-smooth rim.

(4) Glass plate, 3 x 3 x 1/16 inches fitted with three metal prongs for immersing the soil pot in mercury.

(5) Mercury, sufficient to fill the glass cup.

(6) Spatula.

c. **Preparation.** About 30 grams of a thoroughly mixed portion of soil passing through a No. 40 sieve is selected. It is prepared in the same manner as for the liquid limit test.

1-15. SHRINKAGE LIMIT TEST PROCEDURE

The procedure consists of the following steps:

a. Record all identifying information for the sample on a data sheet (fig 1-16).

b. Place the selected portion of the sample in the evaporating dish and mix it thoroughly with distilled water. The amount of water added must be sufficient to wet the soil and make it easy to work into the shrinkage dish without inclosing air bubbles. To achieve the desired consistency, the amount of water is equal to or slightly higher than the liquid limit, with plastic soils requiring up to 10 percent more than the liquid limit.

c. Weigh and record the weight of the empty shrinkage dish.

d. Coat the inside of the shrinkage dish with a thin layer of petroleum jelly or similar compound to prevent the soil from adhering to the dish.

e. Place a volume of the wetted soil equal to about one-third of the volume of the dish into the center of the dish. Tap the dish on a firm surface to cause the soil to flow to the outer edges. Continue tapping until all air bubbles have been eliminated from the soil. Repeat this step with two more layers until the dish is full with a slight excess above the rim of the dish. Strike off the excess with a straightedge and remove all soil adhering to the outside of the dish.

f. Weigh the dish and wet soil immediately and record the weight.

g. Allow the soil to air-dry until a definite color change takes place. Then ovendry it to a constant weight. Record the ovendry weight.

SHRINKAGE LIMIT TEST

Date __________

Project __________

Boring No. __________

Sample or Specimen No.						
Shrinkage Dish No.						
Weight in grams	Dish plus wet soil					
	Dish plus dry soil					
	Water	W_w				
	Shrinkage dish					
	Dry soil	W_s				
	Displaced mercury + evaporating dish					
	Evaporating dish					
	Displaced mercury					
Volume in cc	Shrinkage dish (wet soil pat)	V				
	Volume of dry soil	V_s				
	$V - V_s$					
	$\frac{V - V_s}{W_s} \times 100$					
Water content $= \frac{W_w}{W_s} \times 100$		w	%	%	%	%
Shrinkage limit		SL				
Shrinkage ratio		R				

$$V_s = \frac{\text{weight of displaced mercury}}{\text{specific gravity of mercury (13.53 g/cc)}}$$

$$SL = \text{Water content of wet soil pat} - \left(\frac{\text{volume of wet soil pat} - \text{volume of oven-dry soil pat}}{\text{wt of oven-dry soil pat}}\right)$$

$$= w - \left(\frac{V - V_s}{W_s} \times 100\right)$$

$$R = \frac{\text{wt of oven-dry soil pat}}{\text{volume of oven-dry soil pat}} = \frac{W_s}{V_s}$$

Classification: __________

Remarks __________

Technician __________ Computed by __________ Checked by __________

Figure 1-16. Sample data sheet, shrinkage.

h. Determine the volume of the shrinkage dish by filling it to overflowing with mercury and removing the excess by passing the glass plate firmly over the top. Weigh the amount of mercury required to fill the dish and divide the weight by the density of mercury (13.53 grams/cubic centimeter) to get the volume inside the dish. Record this volume as the volume of the shrinkage dish which is equal to the volume of wet soil placed in the dish.

CAUTION: Mercury may have a toxic effect, particularly if spilled in areas without good ventilation.

i. Place the glass cup in the evaporating dish and fill it to overflowing with mercury. Remove the excess mercury by placing the glass plate with metal prongs over the cup. Be careful not to trap air under the plate. Empty the excess mercury from the evaporating dish. Remove all mercury adhering to the cup and the dish with a brush. Place the mercury-filled cup back into the evaporating dish.

j. Immerse the dry soil pat in the mercury in the cup using the glass plate with the three prongs to hold the pat (fig 1-17). Do not trap air bubbles under the soil pat or the glass plate.

k. Determine the weight of the displaced mercury in the evaporating dish and its volume, h above. This is equal to the volume of the dry soil pat.

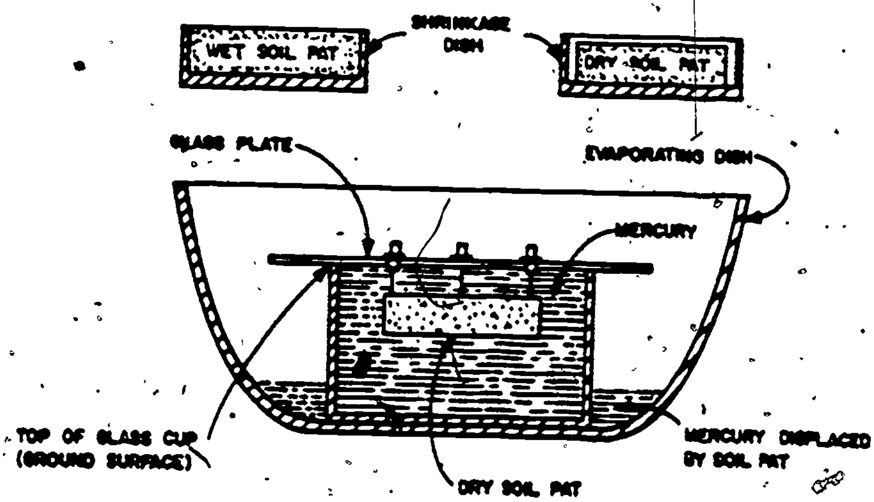

Figure 1-17. Soil pat immersed in mercury.

l. Record all the information pertaining to weights and volumes on the data sheet.

m. Compute the shrinkage value as follows:

(1) Water content $(W) = \frac{W_w}{W_s} \times 100$

Where:

W_w = weight of water (in grams) obtained by subtracting the weight of shrinkage dish plus dry soil from the weight of the dish plus wet soil.

W_s = weight of oven-dried soil (in grams) obtained by subtracting the weight of the shrinkage dish from the weight of the dish plus dry soil.

(2) Shrinkage limits (SL) = $W - \left(\frac{V - V_s}{W_s} \times 100\right)$

Where:

W = water content from (1) above.

V = volume of wet soil pat (in cc).

V_s = volume of oven-dried soil pat (in h above, in cc) (determined by dividing the weight of displaced mercury by the specific gravity of mercury, 13.53 gm/cc).

W_s = weight of oven-dried soil pat (in grams)

(3) Shrinkage ratio (R) = $\frac{W_s}{V_s}$

where V_s and W_s are the same as described in (2) above.

(4) Linear shrinkage (LS) = 100

$$\left(1 - 3\sqrt{\frac{100}{C + 100}}\right)$$

Where:

C = volumetric change from a given water content, w, usually the liquid limit or $C = (w - SL)$ as determined in (1) to (3) above.

1-16. SIZE

a. Size groups.

(1) Particles are defined according to their sizes by the use of sieves, which are simply screens attached to one end of shallow circular containers. If a particle will not pass through the screen with a particular size opening, it is said to be "retained on" that sieve. Particles which pass through a sieve opening are said to be "passing" that sieve size. By passing a soil mixture through several different size sieves, it can be broken into its various components and then defined according to the sieves used.

(2) It is common practice in classifying soils to name the particles of soils according to certain groupings based upon established size definitions as previously discussed.

(3) The fines could be subdivided into their two component parts according to size, but the testing apparatus required (hydrometer analysis) is not often available on the many occasions when these components need to be defined. Therefore, to distinguish between silts and clays the engineer uses plasticity.

(4) Before progressing further, it is well to note the difference between the two nomenclatures denoting the sieve screen sizes, one in "inches", the other designated by "number". Inches refers to the actual linear dimensions of the openings between the wires of a sieve screen. The number designation of a sieve indicates the number of openings existing in the screen per linear inch. Figure 1-18 illustrates the difference.

(5) By measurement the opening in the No. 4 sieve is 4.75 millimeters while that of the ¼-inch sieve is 6.35 millimeters. The difference in the size of the openings of the two sieves illustrated is a result of the thickness of the wires being included with the four openings per linear inch of the No. 4 sieve. Sieves with openings ¼ inch or larger are referenced according to the size of the opening. Sieves with openings smaller than

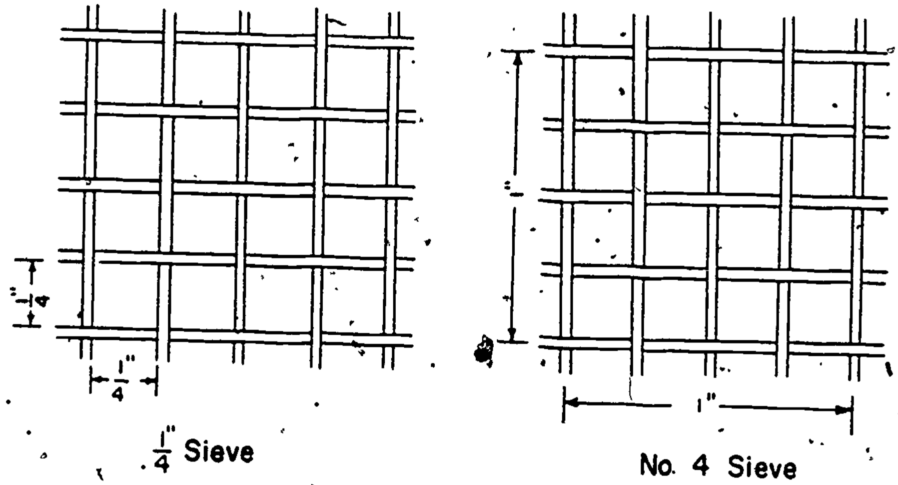

Figure 1-18. Sieve nomenclature.

¼-inch are identified by the number of openings per linear inch.

b. Effect of particle size on the bearing capacity of soil.

(1) One of the factors that affects the bearing capacity of soil is the density of the soil mass. By empirical tests it has been found that, generally speaking, coarse-grained soils can be compacted to greater densities than fine-grained soils. As far as density is concerned, then, a soil containing gravel would be more desirable than one not having the larger particles.

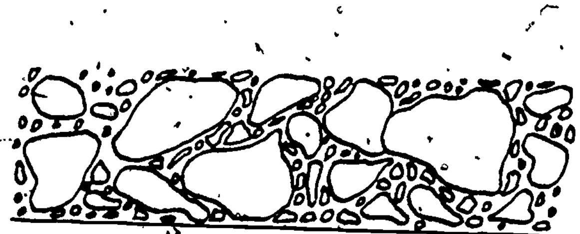

Figure 1-19. Compacted soil layer.

(2) To explain this difference in density on the basis of particle size alone, consider the sketch of a small part of a compacted soil layer shown in figure 1-19. If it were possible to remove one of the large particles without disturbing the surrounding material, and replace it with as large an amount of smaller particles as possible, it would be clear that, in the process, void spaces would be introduced where solids previously existed. This would give a decrease in density proportional to the volume of solids replaced by voids. In short, soils with larger particles generally produce greater densities, because the spaces occupied by the large grains have no voids. The maximum particle size a sample should possess depends on the intended use of the material.

1-17. SHAPE

a. Principles. The shape of the particles composing a soil mass is another factor that is important as an influence on the strength and stability of a soil material. Two general particle shapes are normally recognized: bulky and flaky.

b. Bulky grain shape.

(1) **Principles.** Gravel, sand, and silt particles, although covering a large range of sizes, are all of bulky shape. The term is defined to include particles which are relatively equal in all three of the dimensions as contrasted to flaky grains in which one of the dimensions is small as compared to the

other two. A book is an example of a bulky shaped object. Notice that all of the dimensions are roughly the same size, with the length not more than several times larger than the thickness. A page of a book, by contrast represents a flaky shaped object, where one of the dimensions is very much smaller than the other two. There are four subdivisions of the bulky shape depending on the amount of weathering which has occurred on these particles. The subdivisions in the order of desirability for construction are:

(2) **Angular.** Angular particles are those which have been recently broken up and are characterized by jagged projections, sharp ridges, and flat surfaces. Angular gravels and sands are generally the best materials for construction because of their interlocking characteristics. Such particles are seldom found in nature because weathering processes normally wear them down in a relatively short period of time. Angular material may be produced artificially by crushing, but because of the time and equipment required for such an operation, natural materials with other grain shapes are frequently used.

(3) **Subangular.** Subangular particles are those which have been weathered to the extent that the sharper points and ridges have been worn off. The particles are still very irregular in shape with some flat surfaces.

(4) **Subrounded.** Subrounded particles are those in which weathering has progressed to an even further degree. They are still somewhat irregular in shape but have no sharp corners and few flat areas. Materials with this shape are frequently found in stream beds. They may be composed of hard, durable particles which are adequate for most construction needs.

(5) **Rounded.** Rounded particles are those in which all projections have been removed and few irregularities in shape remain. The particles approach spheres of varying size. Rounded particles are usually found in or near stream beds or beaches. Perhaps the most extensive deposits exist at beaches where repeated wave action has tended to produce, in many cases, almost perfectly rounded particles which are usually uniform in size.

c. **Flaky grain shape.** Flaky particles are those which have flat, platelike grains. As indicated before, these are particles which have one dimension which is relatively very small as compared to the other two. As an illustration of this shape, consider a sheet of paper in which the thickness is greatly exceeded by the length and the width. It should be apparent that in flaky particles the relationship between surface area and the weight of the particles is quite different from that found in bulky particles. This increased surface area provides a greater contact area for moisture and is largely responsible for many of the characteristics which we associate with the flaky particles of clay soils. Clays are flaky because they are formed mostly by chemical weathering (decomposition) whereas the bulky shapes result from mechanical weathering (disintegration). Exceptions to this last statement are the volcanic clays.

d. **Effect of grain shape on bearing capacity.** Aside from the effect of the flaky shape, some significant differences in supporting ability result from use of the various types of bulky shapes. This can be clearly shown by noting the different resistance to penetration exhibited by angular particles and by rounded particles. Angular particles resist penetration much better than rounded particles because:

(1) The angular particles do not roll over one another but have a tendency to interlock because of their sharp peaks and ridges.

(2) Angular particles have rougher surfaces than rounded ones and therefore develop greater frictional resistance to sliding over each other.

SELF TEST

Note: The following exercises comprise a self test. The figures following each question refer to a paragraph containing information related to the question. Write your answer in the space below the question. When you have finished answering all the questions for this lesson, compare your answers with those given for this lesson in the back of this booklet. Do not send in your solutions to these review exercises.

1. Soil is the entire unconsolidated material that overlies bedrock. What is soil principally composed of? (1-1)

2. The crust of the earth is generally regarded to consist of a layer of solid material, mostly rock, from 30 to 50 miles in thickness. What lies beneath the crust and what state is it in? (1-2a(2))

3. Mechanical weathering causes rocks to disintegrate, while chemical weathering causes rocks to decompose. In what climate region is decomposition the dominant process? (1-2b(3))

4. Soils can be divided into several different classifications according to the Unified Soil Classification System. If a soil is retained on a 3-inch sieve, what is it called? (1-3, table 1-1)

5. Figure 1-20 illustrates a grain-size distribution curve plotted from the sieve analysis of a soil sample. What is the uniformity coefficient (C_u) of this soil? (1-3f(1), fig 1-20)

6. Along with the uniformity coefficient, the coefficient of curvature (C_c) indicates the gradation of soil. To indicate a well graded soil, the coefficient of curvature must have a value of between 1 and 3. What is the coefficient of the soil sample represented on figure 1-20? (1-3f (2), fig 1-20)

7. Give a brief definition of a well-graded soil. (1-3g)

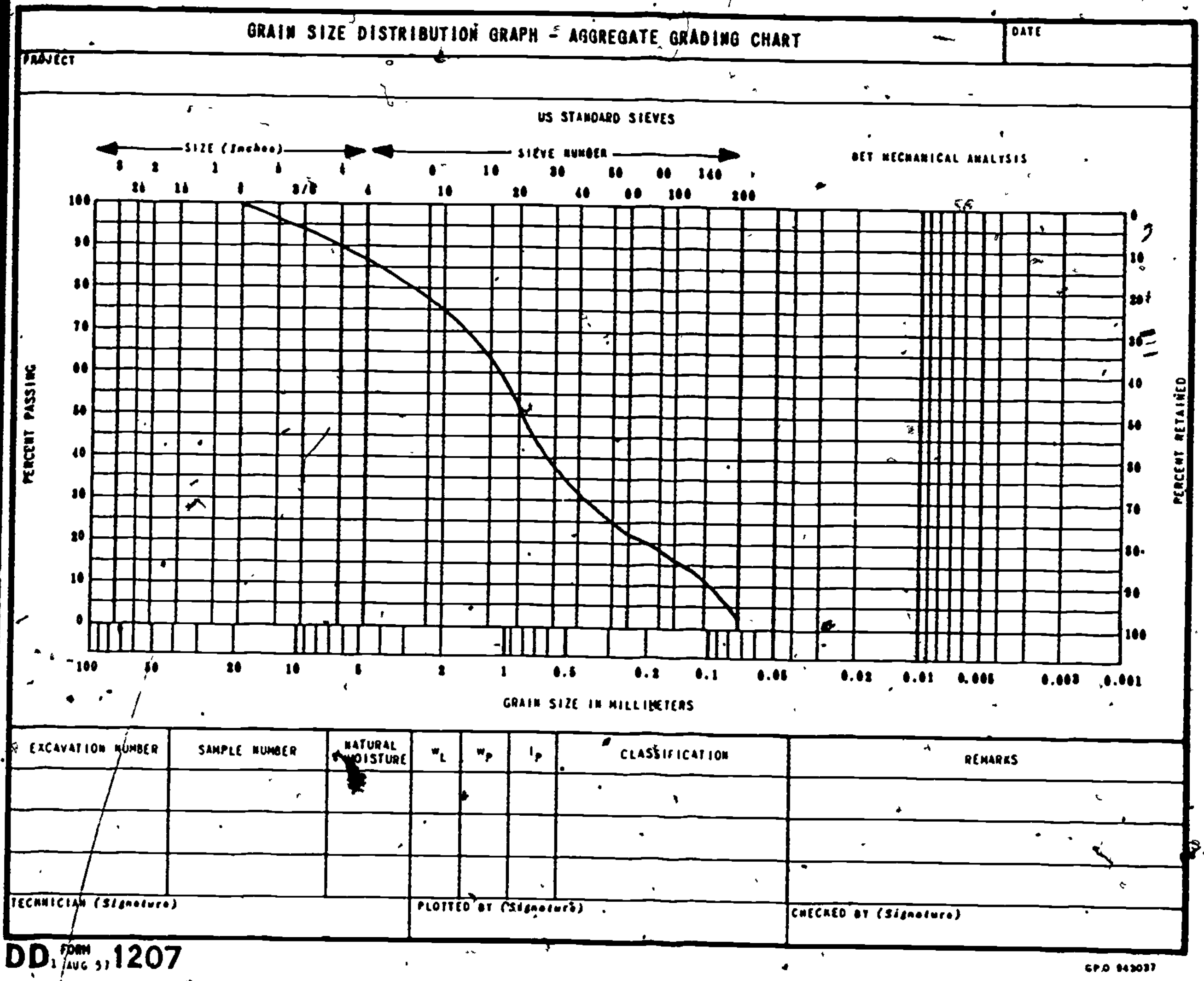

Figure 1-20. For use with exercises 5 and 6.

8. The shape of the grain-size distribution curve will tell you what type of soil you have. What is the shape of the grain-size distribution curve for a uniformly graded soil? (1-3l, fig 1-7)

9. What is meant by the term "moisture content" in soil, and how is it determined? (1-5a)

10. Give the different states a fine grained soil may pass through as the moisture content is increased starting with a very dry sample. (1-6, fig 1-9)

11. The plasticity index of a soil is determined from the liquid limit and the plastic limit. If the liquid limit is 22 and the plastic limit is 10, what is the plasticity index? (1-8a, b)

12. The liquid limit and the plastic limit are two important characteristics of a soil to a construction engineer. Give a definition for liquid limit. (1-9a)

13. In the preparation of a soil sample for the liquid limit test, you must be sure that the entire sample will pass a certain sieve size. What is that sieve size? (1-9c(1))

14. One ounce is equal to 28.35 grams. The test sample for the plastic limit test should weigh approximately how many grams? (1-12c)

15. At what point in the plastic limit test procedure do you determine that the moisture content is such that the plastic limit of the soil has been reached? (1-13d)

16. What is meant by the "shrinkage limit of a soil? (1-14a)

17. When performing the shrinkage limit test, it is necessary to determine the volume of the dry soil pat by displacement. What liquid is used in this part of the test? (1-15j).

18. Sieve screen sizes are given either in inches (size of opening), or in numbers (number of openings per linear inch). What is the dividing point between the two methods of designation? (1-16a(5))

19. Why are course-grained soils generally more desirable in construction than soils containing only fine particles? (1-16b(1))

20. There are four subdivisions of bulky shaped particles. Name them in order of desirability for construction purposes. (1-17b)

LESSON 2

SOIL SURVEYS; SOIL CLASSIFICATION

CREDIT HOURS	3
TEXT ASSIGNMENT	Attached memorandum (Including Identification and Description).
MATERIAL REQUIRED	Chart I, Unified Soil Classification. Chart III, Identification Procedure.
LESSON OBJECTIVE	Upon completion of this lesson you should be able to accomplish the following in the indicated topic areas:

1. **Soil Surveys.** Describe the purpose, methods and procedures for making soil surveys. Make a soil survey and prepare a technical soil report.

2. **Unified Soil Classification System.** Explain how soils are identified and classified under the USCS, use of the master charts, and the scope and value of the system.

3. **Field Identification of Soils.** Explain the procedure and supervise the performance of field tests that can be used to identify and classify soils when lack of time and facilities make laboratory testing impracticable.

ATTACHED MEMORANDUM

2-1. SOIL SURVEYS

a. **Principles.** 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.

b. **Types of Soil Surveys.**

(1) **Principles.** A soil survey consists of gathering soil samples for examination, 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.

(2) **Hasty surveys.** The hasty survey should be preceded by as careful a study of all available sources of information as conditions permit. If possible, a trained person may observe soil conditions in the proposed construction area from the air. Careful aerial observation gives an overall picture which is often difficult to secure at ground level because important features may be obscured 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 modifications in location, design, and construction.

(3) **Deliberate surveys.** 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 beng 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 earthmoving equipment under expedient conditions to permit a hasty approach to the underlying soil.

c. **Objectives of soil surveys.**

(1) **Principles.** 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.

(2) **Location, nature, and classification of soil layers.** Adequate and economic earthwork and foundation design of a structure can only be accomplished when the types and depths of soil to be encountered are known. 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.

(3) **The condition of soils in place.** The moisture content and density of a soil in its natural state sometimes 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 or compact 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, necessitating scarification or rooting before excavation.

(4) **Drainage characteristics.** The 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.

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

(6) **Soil Profile.** The soil profile is a graphical representation of a vertical cross section of the soil layers from the surface of the earth downward. Discussion of the development of the soil profile will follow a more detailed discussion of how the extent of materials within an area is determined.

2-2. SOURCES USEFUL IN PLANNING SOIL SURVEYS

a. **Principles.** There are many sources of information available to the soils engineer.

These should all be used to the fullest extent to eliminate as much detailed investigation as possible. Some of these sources are intelligence reports and information from local inhabitants, and from maps and air photographs. These sources will be used mostly to locate small areas of 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.

b. Intelligence reports and local inhabitants.

(1) Intelligence reports. 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.

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

c. Maps.

(1) Principles. Maps provide valuable information, especially when planning the soil survey. In some cases, maps showing the suitability of terrain for various military purposes, prepared by friendly foreign or enemy agencies, may be of considerable value in planning. There are several kinds of maps which provide different types of information about an area under investigation.

(2) Geological maps. It is apparent that a close relationship exists between geology and soil conditions. Geological maps and brief descriptions of regions and quadrangles have been published in the Folios of the U. S. Geological Survey. Generally, the smallest rock unit mapped is a formation, and geological maps indicate the areal extent of these formations by means of letter symbols, color, or symbolic 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.

(3) Topographic maps. Ordinary topographical maps may be of some use in estimating soil conditions, particularly when used with geologic maps. Topographic maps, especially when the contour interval is 20 feet or more, tend to 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.

(4) Agricultural soil maps. Agricultural soil maps and reports are available for many of the developed agricultural areas of the world. These studies are concerned primarily with surface soils, generally to a depth of about 6 feet. Their value as aids in the engineering study of surface soils is apparent. For example, if the same soil is shown to occur 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

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their texture, color, structure, chemical and physical composition, and morphology.

d. **Air photographs.**

(1) **Principles.** 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 landforms, slopes, drainage patterns, erosional characteristics, soil color or "tone," vegetation and land use. Each of the brief following discussions serves only as an example of the information which may be derived from the examination of air photographs.

(2) **Landform.** The "form" or configuration of the land in different types of deposits is definitely 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 airphoto typically shows sinkholes.

(3) **Slope.** 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.

(4) **Drainage patterns.** A very simple drainage pattern is frequently indicative of pervious soils. A highly integrated (elaborate) 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.

(5) **Erosional patterns.** Considerable information may be gained 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, saucershaped gullies.

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

(7) **Vegetation.** Vegetation may reflect surface soil types, although its significance frequently 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.

(8) **Land use.** Ready identification of soils is frequently facilitated by observing agricultural land use. 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.

2-3. FIELD IDENTIFICATION

a. **Principles.** 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.

b. **Sampling methods.**

(1) **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 but 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 ground water 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.

(2) **Test holes.** Test hole exploration includes the several methods described below. The use of 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 2-1 shows methods of underground exploration and sampling in a condensed table.

TABLE 2-1. *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.

c. **Planning the soil survey.**

(1) **Principles.** The location of auger holes or test pits will depend upon the particular situation. In any case, the method described locates the minimum amount of holes. The completeness of the exploration will depend upon the time available. A procedure for road, airfield, and borrow area investigation will be described.

(2) **Subgrade.**

(a) Since soil tests should be made on samples representing the major soil types in the area, the first step in subgrade exploration is to develop a general picture of the subgrade conditions to assist in determining the representative soils. Field reconnaissance should be made to study landforms and soil conditions in ditches and cuts. Techniques have been developed whereby aerial photographs can be used for delineating areas of similar soil conditions. Full use should also be made of existing data in agricultural soil maps for learning subsurface conditions.

(b) The second step in determining subgrade conditions in the area to be used for runway, taxiway, and apron construction 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 maximum possible information with the least number of borings. In theater of operations cut areas, all holes should extend 4 feet below final subgrade elevation, if possible. In theater of operations fill areas, they should extend 4 feet below the natural ground elevation. The boring requirements stated above usually will result in the borings penetrating beyond 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).

(c) Soil samples should be obtained for classification purposes 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 to be made. 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 to which compaction must extend and to ascertain the presence of any soft layers in the subsoil.

(3) **Borrow areas.** Where material is to be borrowed from adjacent areas, borings should be made in these areas and carried 2 to 4 feet below the anticipated depth of borrow. Samples from the borings should also be classified and tested for water content, density, and strength.

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

(5) **Base and pavement aggregates.** Since these materials are generally crushed and processed materials, a survey should be made of existing producers plus possible other 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 in quantity.

d. **Locating, numbering, and recording samples.** The engineer in charge of the soil survey is responsible for properly surveying, numbering, and recording each auger boring, test pit, or other exploration investigation. 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 is that relating to density of each soil, changes in moisture content, depth to ground water, and depth to rock. A typical boring log is shown as figure 2-1.

2-4. THE SOIL PROFILE

a. **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

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REPORT OF FOUNDATION AND BORROW INVESTIGATION

SITE Airfield Jamock TYPE EXPLORATION Hasty DATE March 66

BORING NO. 1 LOCATION Station 0 + 00 GROUND ELEV. 236

PURPOSE OF EXPLORATION Determine Soil Profile along the Centerline of Runway

DEPTH BELOW SURFACE 1" = 2'	ELEV.	SAMPLE NO, TYPE & DEPTH	GRAPHIC LOG	GROUP SYMBOL	DESCRIPTION, TEST DATA, & REMARKS
1ft	235'	No. 1 at 1/2'		OH	Dark brown and very plastic. Typical top soil of the area.
3ft	233'	No. 2 at 2 1/2'		SM	Soil with low cohesion, some sand with large percentage of silt.
5ft	231'			SC	Coarse sandy soil with a plastic binder material. Light red color.
7ft	229'	No. 3 at 7'		CH	Brown sticky clay, very high plastic qualities. Ribboned out to 4 1/2" with little trouble. Rolled into a thread very readily.
					Bottom of hole

DEPTH TO WATER TABLE 3 ft

SUBMITTED BY SP4 Mc Gurk

Figure 2-1. Typical boring log.

layers encountered in each log, it is desirable to show the natural water contents of fine-grained soils along the side of each log when possible. Also, the elevation of the ground water 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, in order to allow the water to reach its maximum level. The soil profile (fig 2-2) is a graphical representation 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.

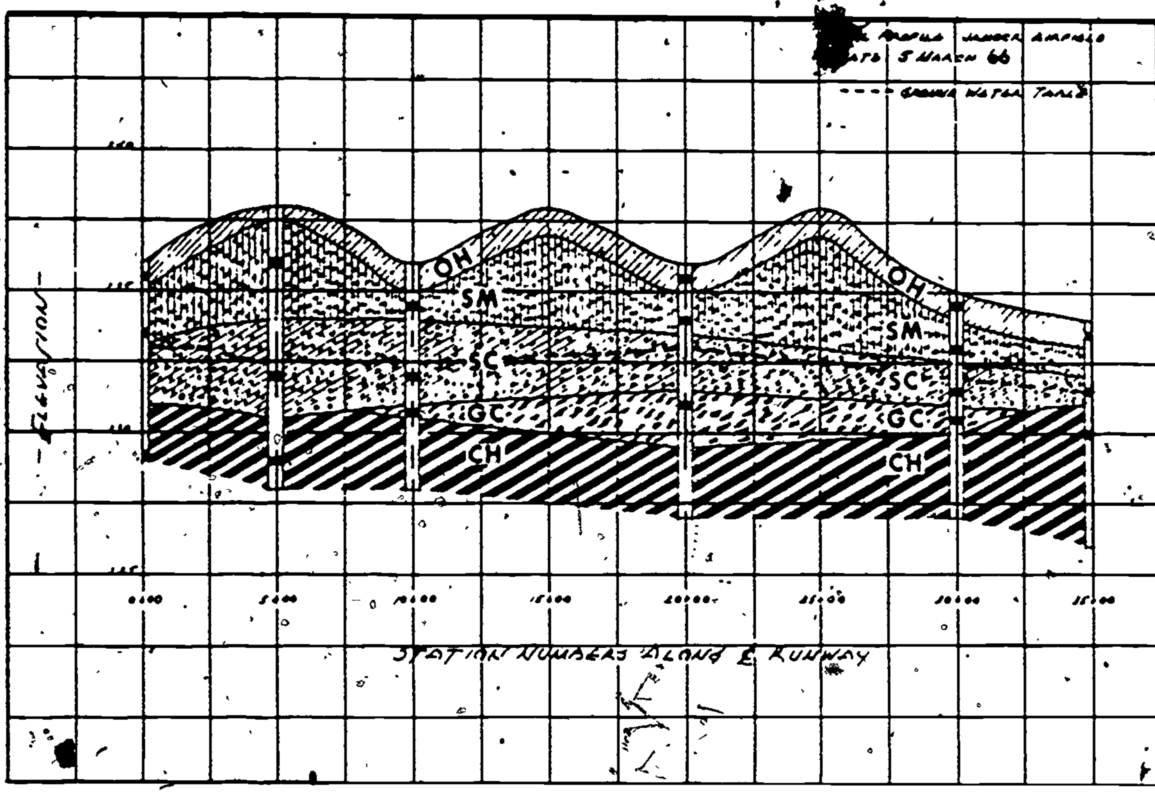

Figure 2-2. Typical soil profile.

b. **Uses of the soil profile.** 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 so that full advantage is taken of the best soils which are 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 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 so that advantage may be taken of the presence of well-draining soils. It may indicate that special drainage installations will be needed with soils which are more difficult to drain, particularly in areas where the water table is high. Considerations relative to capillary and frost action may be particularly important when frost-susceptible soils are shown on the profile.

2-5. RECOMMENDED PROCEDURE FOR SOIL SURVEYS

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

a. Objectives of soil exploration.

(1) Soil types and securing of samples.

(2) Condition of soil in-place, density, and moisture content.

(3) Drainage characteristics.

(4) Depth to ground water and bedrock.

(5) Development of a soil profile for the area.

b. Sources of information.

(1) **Published information.**

(a) Geological and topographic reports and maps.

(b) Agricultural soil bulletins and maps require careful interpretation and knowledge of local terms.

(c) Aerial photographs — used to predict subsurface conditions.

(d) Previous explorations for nearby construction projects.

(2) **Field information.**

(a) General observation of road cuts, streambanks, eroded slopes, earth cellars, mine shafts, existing pits and quarries, etc.

(b) Test holes made with hand auger or power auger if necessary and available.

(c) Test pits, necessary where hand auger cannot penetrate, or large samples are required.

(d) Local inhabitants — preferably trained observers such as contractors, engineers, quarrymen, etc.

c. Planning of general layout.

(1) **Primary objective.** The primary objective is to determine the extent of the various soil types, vertically and laterally, within the zone where earthwork may occur.

(2) **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 condition will be less than in a terrain where the soil conditions change rather frequently. Exploration borings generally 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 generally accepted policy to conduct the exploration borings at the edges of existing pavements, unless these pavements have failed completely, in which case the reason for the failure should be found. Large cuts and fills are the most important areas for detailed exploration.

(3) **Cut and fill sections.** Cut section — 4 feet below subgrade, if possible. Fill section — 4 feet below original ground level, if possible. Effort should be made to locate the ground water table.

d. Procedures in sampling.

(1) Exploration holes or pits must be carefully logged.

(2) Samples must be accurately located and numbered.

(3) The elevation and exact location of each hole should be determined and tied in to the site layout.

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e. Preparation of soil profile.

(1) Shows 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.

(2) Used to locate grade line, for excavation and grading plan, and for drainage plan.

2-6. TECHNICAL SOILS REPORT

The organization and presentation of the soils report is very important. It must be well organized and be presented in a logical and concise format with emphasis on the technical conclusions. The following outline should help to organize a technical soils report successfully:

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 SITE
   - 2—01 Description of Location and Existing Facilities
   - 2—02 Topography, Cultivation, and Drainage
3. GEOLOGY
   - 3—01 Description of Subsurface Materials At and Near Site
   - 3—02 Description of Overburden and Bedrock
4. AGGREGATES
   - 4—01 Field Exploration
   - 4—02 Field Tests
   - 4—03 Laboratory Tests
   - 4—04 Results of Field and Laboratory Investigations
5. FOUNDATION CONDITIONS
   - 5—01 Field Explorations
   - 5—02 Field Tests
   - 5—03 Laboratory Tests
   - 5—04 Results of Field and Laboratory Investigations
6. FILLS AND BORROW MATERIALS
   - 6—01 Field Explorations
   - 6—02 Field Tests
   - 6—03 Laboratory Tests
   - 6—04 Results of Field and Laboratory Investigations

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7. CONCLUSIONS AND RECOMMENDATIONS

7—01 Site Selection

7—02 Economical Design

7—03 Minimum Specifications

ANNEXES

A. General Plan Drawings

B. Location Plan Drawings (existing and proposed features)

C. Profiles

D. Cross-Sections

E. Boring Logs

F. Laboratory Testing Data

G. Field Testing Data

Notes.

1. It can be seen from an inspection of the outline that not every subject will apply to every report and that some of the items must be repeated several times in the same report. For example, items 5, 6, and 7 would have to be repeated for each runway in an airfield and item 4—03 might be omitted for expedient situations.

2. Frequently, portions of the information shown in the outline will be required at different time intervals. 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 greatly simplified.

3. The type of survey conducted will determine the lengths and detail of the report. In the hasty survey, most of the items can be covered in one or two sentences and almost all of the annexes can be omitted. When a deliberate survey is made, it may be necessary to make a detailed soil profile, and detailed plan drawings might be required.

2-7. UNIFIED SOIL CLASSIFICATION SYSTEM

a. **Development.** Soils seldom exist in nature separately as sand, gravel, or any other single component, but usually are found as mixtures with varying proportions of particles of different sizes. Each component contributes its characteristics to the mixture. The Unified Soil Classification System is based on those characteristics of the soil which indicate how it will behave as a construction material.

b. **Major soil categories.** In the Unified Soil Classification System, all soils are divided into three major categories: coarse-grained soils, fine-grained soils, and organic soils. Coarse-grained and fine-grained soils are differentiated by grain size. Organic soils are identified by the presence of large amounts of organic material.

c. **Letter symbols.** The Unified Soil Classification System further divides soils which have been classified into the major soil categories by using arbitrary letter symbols consisting of two letters. For example, the letters for sand, silt, and clay are S, M, and C, respectively, and the symbol for a soil which meets the criteria for a clayey sand would be designated SC. In the case of borderline soils which possess characteristics of two groups and cannot be classified by a single symbol it may be necessary to use four letters such as SM-SC which would describe a sand which contains appreciable amounts of fines on the borderline classification for silt and clay. After the physical characteristics of a soil have been determined by use of the appropriate tests and calculations, these characteristics are used to classify the soil. The criteria for identification are presented in de-

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tail in the following paragraphs and in charts I and III, which have been specially developed to facilitate soil classification, and are bound in the back of this booklet.

d. **Master Charts for Unified Soil Classification System.** Charts I and III are master charts which present information on the Unified Soil Classification System, and procedures which are to be followed in identifying and classifying soils under this system. Principal categories which are 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 are valuable aids in soil classification problems. They provide a simple, systematic means of soil classification.

2-8. COARSE-GRAINED SOILS

a. **Definition.** 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 retained on a No. 4 sieve. It is a sand if more than half the coarse fraction is smaller than a No. 4 sieve. The symbol G is used to denote a gravel and the symbol S to denote a sand. In general practice, there is no clearcut 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 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. For the purpose of systematizing our discussion, it is desirable to further divide coarse-grained soils into three groups on the basis of the amount of fines (materials passing a No. 200 sieve) which they contain.

Note. If fines interfere with free drainage properties, as may occur with plastic fines, use double symbol (i.e., GW-GC, etc.) meaning that such soils will be classed with soils having from 5 to 12 percent fines.

b. **Coarse-grained soils with less than the 5 percent passing No. 200 sieve.**

(1) **Principles.** 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. However, as denoted above, if the fines do interfere with the free drainage properties a dual or double symbol will be used.

(2) **GW and SW groups.** In the GW groups are 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 gravelly sands with little or no nonplastic fines.

(3) **GP and SP groups.** The GP group includes poorly graded gravels and gravel-sand mixtures containing little or no nonplastic fines. In the SP group are contained poorly graded sands and gravelly sands with little or no nonplastic fines. These soils will not meet the gradation requirements established for the GW and SW groups.

c. **Coarse-grained soils containing more than 12 percent passing No. 200 sieve.**

(1) **Principles.** 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 Chart I 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. 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.

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

(3) **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 of these groups, the Atterberg limits must plot above the A-line with a plasticity index of more than 7.

d. **Borderline soils.** 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 (chart I), 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 and, 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 a PI of 9, would be designated as SW-SC. If the Atterberg limits of this soil were such as to plot 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 he is in.

2-9. FINE-GRAINED SOILS

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

b. **Groupings.**

(1) **Principles.** 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 which is designated as the plasticity chart in chart I. This chart was established by the determination of limits for many soils, together with an analysis of the effect of limits upon physical characteristics. Examination of the chart will show that there are two major groupings of fine-grained soils. These are the L groups, which have liquid limits less than 50, and the H groups, which have liquid limits equal to and greater than 50. The symbols L and H have general meanings of low and high compressibility, respectively. Fine-grained soils are further divided with relation to their position above or below the A-line of the plasticity chart.

(2) **ML and MH groups.** Typical soils of the ML and MH groups are inorganic silts; those of low-compressibility in the ML group, others in the MH. All of these soils plot below the A-line. In the ML group are included very fine sands, rock flours, 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. The same statement is true of certain types of kaolin clays which have low plasticity. Plastic silts will fall into the MH group.

(3) **CL and CH groups.** In these groups, the symbol C stands for clay, with L and H denoting low or high liquid limits. These soils plot above the A-line and are principally inorganic clays. In the CL group are included gravelly clays, sandy clays, silty clays, and lean clays. In the CH group are 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 band in the CL and CH groups.

(4) **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 plot below the A-line. Organic silts and organic silt-clays of low plasticity fall into the OL group, while 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 vegetational matter. A liquid limit test, however, is not a true indicator where a considerable portion consists of other than soil matter.

(5) **Borderline soils.** Fine-grained soils which have 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.

2-10. HIGHLY ORGANIC SOILS

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

2-11. FIELD IDENTIFICATION OF SOILS

a. **Introduction.** Lack of time and facilities often make laboratory soil testing impossible in military construction. Even where laboratory tests are to follow, field identification tests must be made during the soil exploration to distinguish between the different soil types encountered so that duplication of samples for laboratory testing will be held to a minimum. Several simple tests that may be used in field identification are described in this lesson. Each test may be performed with a minimum of time and equipment, although seldom will all of them be required to identify any given sample. The number of tests employed will depend on the type of soil, and on the experience of the individual performing them. Using these tests, the soil properties can be estimated and the materials can be classified. Such classification should be recognized as an approximation, since even experienced personnel have difficulty estimating detailed soil properties with a high degree of accuracy. The material which follows is intended as an aid in the identification and classification of soils according to the Unified Soil Classification System.

b. **General procedure.**

(1) **Coarse-grained soils.** An approximate identification of a coarse-grained soil can be made by spreading a dry sample on a flat surface and examining it, paying particular attention to grain size, gradation, grain shape, and hardness of particles. All lumps in the sample must be thoroughly pulverized to expose the individual grains and obtain a uniform mixture when water is added to the fine-grained portion. The use of a rubber-faced or wooden pestle and a mixing bowl is recommended for this purpose. The material also may be pulverized by placing a portion of the sample on a firm, smooth surface and mashing it with the feet. The use of an iron pestle for pulverizing will break up the mineral grains and change the character of the soil.

(2) **Fine-grained soils.** Test for identification of the fine-grained portion 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.

c. **Equipment required.**

(1) **Principles.** Practically all the tests to be described may be performed with no equipment or accessories other than a small amount of water. However, the accuracy and uniformity of results will be greatly increased by the proper use of certain items of equipment. The following listed items of equipment, will meet most requirements, are available in nearly all engineer units or may be improvised, and are easily transported.

(2) **Sieves.** A No. 40 U. S. standard sieve is perhaps the most useful item of equipment. Any screen with about 40 openings per lineal inch could be used, or an approximate separation may be made by sorting the materials by hand. No. 4 and No. 200 sieves are useful for separating gravel, sand, and fines.

(3) **Pioneer tools.** A pick and shovel or a set of entrenching tools is used in obtaining samples. A hand earth auger is useful if samples are desired from depths more than a few feet below the surface.

(4) **Stirrer.** The spoon issued as part of mess equipment serves in mixing materials with water to desired consistency. It also will aid in obtaining samples.

(5) **Knife.** A combat knife, or engineer pocket knife, is useful in obtaining samples and trimming them to the desired size.

(6) **Mixing bowl.** A small bowl with a rubber faced pestle is used in pulverizing the fine-grained portion of the soil. Both may be improvised, such as by using canteen cup and wood pestle.

(7) **Paper.** Several sheets of heavy paper are needed for rolling samples.

(8) **Heating samples.** A pan and heating element are used for drying samples.

(9) **Scales.** Balances or scales are used in weighing samples.

d. **Factors considered in identification of soils.**

(1) **Principles.** The soil properties which form the basis for the Unified Soil Classification System are: the percentages of gravel, sand, and fines; shape of the grain-size distribution curve; and plasticity. These same properties are, therefore, the primary ones to be considered in field identification, but other characteristics observed should be included in describing the soil, whether the identification is made by field or laboratory methods.

(2) **Soil description.** Properties normally included in a description of a soil are as follows:

Color.

Grain size.

- Estimated maximum grain size.
- Estimated percent by weight of fines (material passing No. 200 sieve).

Gradation.

Grain shape.

Plasticity.

Predominant soil type.

Secondary components of soil.

Classification symbol.

Other remarks such as —

- Organic, chemical, or metallic content.
- Compactness.
- Consistency.
- Cohesiveness near plastic limit.
- Dry strength.
- Source — residual, or transported (aeolian, waterborne, glacial, deposit, etc.).

(3) **Examples of soil description.** An example of a soil description using the se-

quence and considering the properties referred to in (2) above, might be as follows:

Dark brown to white.

Coarse-grained soil, maximum particle size 2¾ inches, estimated 60% gravel, 36% sand, and 4% passing No. 200 sieve.

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

Gravel particles subrounded to rounded.

Nonplastic.

Predominantly gravel.

Considerable sand and small amount of nonplastic fines (silt).

GP.

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

(4) **Importance of description.** A complete description with the proper classification symbol obviously conveys much more to the reader than the symbol or any other isolated portion of the description used alone.

2-12. VISUAL EXAMINATION

a. **Principles.** By visual examination, it is possible to determine the color, grain size, and grain shape of the coarse-grained portion of a soil, and estimate the grain size distribution. To observe these properties, a sample of the material is first dried and then spread on a flat surface.

b. **Color.** In field soil surveys, color is often helpful in distinguishing between various soil strata, and with sufficient preliminary experience with local soils, color also may be useful for identifying soil types. Since the color of a soil often varies with its moisture content, the condition of the soil when color is determined must always be recorded. There is generally more contrast in these colors when the soil is in a moist condition, with all the colors becoming lighter as the moisture contents are reduced. In fine-graded soils, certain dark or drab shades of gray or brown, including almost black colors, are indicative of organic colloidal matter (OL, OH). In contrast, clean and bright looking colors, including medium and light gray, olive green, brown, red, yellow, and white generally are associated with inorganic soils. Soil color also may indicate the presence of certain chemicals. Red, yellow, and yellowish brown soil colors may be a result of the presence of iron oxides. White to pinkish colors may indicate presence of considerable silica, calcium carbonate, or aluminum compounds in some cases. Grayish blue, and gray and yellow mottled colors, frequently indicate poor drainage.

c. **Grain size.** The maximum particle size should always be estimated for each sample considered, thereby establishing the upper limit of the grain size distribution curve for that sample. To aid in determining something about the lower limit of the grain size distribution, it is useful to know that the naked eye can normally distinguish the individual grains of soil down to about 0.07 millimeter. This means that all of the particles in the gravel and sand ranges are visible to the naked eye. All of the silt particles and all of the clay particles are smaller than this size and are therefore invisible to the naked eye. Material smaller than 0.07 millimeter will pass the No. 200 sieve.

d. **Approximate grain size distribution.**

(1) **Principles.** The laboratory mechanical analysis must be performed whenever the grain size distribution of a soil sample must be determined accurately. However, an approximation of the grain size distribution can be made by visual inspection. The best method of observing a material for such a determination without using laboratory equipment is to spread a portion of the dry sample on a flat surface; then, using the hands or a piece of paper, attempt to separate the material into its various grain size components. By this method, the gravel particles and some of the sand particles can be separated from the remainder. This will at least give the observer an opportunity to estimate whether the total sample is to be considered coarse-grained or fine-grained, depending on whether or not more than 50 percent of the material would pass the No. 200 sieve. Per-

centage values refer to the dry weight of the soil fractions indicated as compared to the dry weight of the original sample.

(2) **Coarse-grained soil.** If the material is believed to be coarse-grained, then there are two other criteria to consider: first there is less than 5 percent passing the No. 200 sieve; and, second, the fines are nonplastic. If both these criteria can be satisfied and there appears to be a good representation of all grain sizes from largest to smallest, without an excessive amount or a deficiency of any one size, the material may be said to be well graded (GW or SW). If any intermediate sizes appear to be missing, or if there is too much of any one size, then the material is poorly graded (GP or SP). In some cases, it may be possible to take a few of the standard sieves into the field. When such is the case, the No. 4, No. 40, and No. 200 sieves should be included. Using the No. 4 and No. 200 sieves, the sample may be separated into the three man fractions — gravel, sand, and fines. However, if there is a considerable quantity of fines, particularly clay particles, the fines can only be readily separated by washing them through the No. 200 sieve. In such cases, a determination of the percentage of fines is made by comparing the dry weight of the original sample with that retained on the No. 200 sieve after washing. The difference between these two is the weight of the fines lost in the washing process. For determination of plasticity, only that portion of the soil which will pass through a No. 40 sieve should be used.

(3) **Fine-grained soil.** Estimating the grain size distribution of a sample using no equipment at all is probably the most difficult part of field identification and obviously places great importance on the experience of the individual making the estimate. A better approximation of the relative proportions of the components of the finer soil fraction may sometimes be obtained by shaking a portion of this sample into a jar of water and then allowing the material to settle to the bottom. The material will settle in layers; the gravel and coarse sand particles settling out almost immediately, the fine sand particles within a minute, the silt particles requiring as much as an hour, and the clay particles remaining in suspension indefinitely, or until the water is clear. In using this method, it should be kept in mind that the gravel and sand will settle into a much more dense formation than will either the silt or clay.

e. **Grain shape.** The grain shape of the sand and gravel particles can be determined by close examination of the individual grains. The grain shape affects the stability of the soil because of the increased resistance to displacement that is found in the more irregular particles. A material whose grains are rounded has only the friction between the surfaces of the particles to help hold them in place. An angular material has this same friction, which is increased by the roughness of the surface and the area of contact. In action, an interlocking action is developed between the angular particles which gives a much greater stability than friction alone.

f. **Undisturbed soil properties.** A complete description of a soil should include prominent characteristics of the undisturbed materials. The aggregate properties of sand and gravel are described qualitatively by the terms "loose", "medium", and "dense", while those of clays are described by "hard", "stiff", "medium", and "soft". These characteristics usually are evaluated on the basis of several factors, including the relative ease or difficulty of advancing the drilling and sampling tools and the consistency of the samples. In soils that are described as "soft", it should also be indicated whether the material is loose and compressible, as in an area under cultivation, or spongy or elastic, as in highly organic soils. (Moisture conditions will influence these characteristics and should be included in the report.)

2-13. BREAKING OR DRY STRENGTH TEST

a. **Preparation of sample.** The breaking test is performed only on the material passing the No. 40 sieve. This test, as well as the roll test and the ribbon test, is used to measure the cohesive and plastic characteristics of the soil. The test normally is made on a small pat of soil about a ½ inch thick and about 1½ inches in diameter. The pat is prepared by molding a portion of the soil in the wet plastic state into the size and shape desired and then allowing the pat to dry completely. Samples may be tested for dry strength in their natural condition as they are found in the field, but too much reliance must not be given to such tests because of the variations that exist in the drying conditions under field circumstances. Such a test may be used as an approximation, however, and verified later by a carefully prepared sample.

b. **Breaking the sample.** After the prepared sample is thoroughly dry, attempt to break it using the thumb and forefingers of both hands. If it can be broken, try to powder it by rubbing it with the thumb and fingers of one hand.

c. **Typical reactions.**

(1) **Principles.** Typical reactions that are obtained in this test for various types of soils are described below.

(2) **Very highly plastic soils (CH).** Very high dry strength. Samples cannot be broken or powdered by use of finger pressure.

(3) **Highly plastic soils (CH).** High dry strength. Samples can be broken with great effort but cannot be powdered.

(4) **Medium plastic soils (CL).** Medium dry strength. Samples can be broken and powdered with some effort.

(5) **Slightly plastic soils (ML, MH, or CL).** Low dry strength. Samples can be broken quite easily and powdered readily.

(6) **Nonplastic soils (ML or MH).** Very little or no dry strength. Samples crumble and powder on being picked up in the hands.

d. **Precautions.** The test described above is one of the best for distinguishing between plastic clays and nonplastic silts or fine sands. However, a word of caution is appropriate. 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.

2-14. ROLL OR THREAD TEST

a. **Sample.** The roll or thread test is performed only on the material passing the No. 40 sieve. The soil sample used in this test is prepared by adding water until the moisture content is such that the sample may be easily remolded without sticking to the fingers. This is sometimes referred to as being just below the "sticky limit". Using a nonabsorbent surface, such as glass, this sample is rolled rather rapidly into a thread approximately 1/8 inch in diameter.

b. **Rolling.** If a moist soil can be rolled into such a thread at some moisture content, it is said to have some plasticity. Materials which cannot be rolled in this manner are nonplastic, or have very low plasticity.

c. **Typical reactions.**

(1) **Principles.** After reaching the plastic limit, the degree of plasticity may be as described below.

(2) **High plasticity (CH).** The soil may be remolded into a ball and the ball deformed under extreme pressure by the fingers without cracking or crumbling.

(3) **Medium plasticity (CL).** The soil may be remolded into a ball, but the ball will crack and easily crumble under pressure of the fingers.

(4) **Low plasticity (CL, ML, or MH).** The soil cannot be lumped together into a ball without completely breaking up.

(5) **Organic materials (OL or HO).** Soils containing organic materials or mica particles will form soft spongy threads or balls when remolded.

(6) **Nonplastic soils (ML or MH).** These cannot be rolled into a thread at any moisture content.

d. **Description of cohesiveness.** From this test, the cohesiveness of the material near the plastic limit may also be described as weak, firm, or tough. The higher the position of a soil on the plasticity chart (chart I), the stiffer are the threads as they dry out and the tougher are the lumps if the soil is remolded after rolling.

2-15. RIBBON TEST

The ribbon test is performed only on the material passing the No. 40 sieve. The sample prepared for use in this test should have a moisture content that is slightly below the "sticky limit". The "sticky limit" is the lowest water content at which the soil will adhere to a metal tool. Using this material, form a roll of soil about ½ to ¾ of an inch in diameter and about 3 to 5 inches long. Place the material in the palm of the hand and, starting with one end, flatten the roll, forming a ribbon ⅛ to ¼ inch thick by squeezing it between the thumb and forefinger. The sample should be handled carefully to form the maximum length of ribbon that can be supported by the cohesive properties of the material. If the soil sample holds together for a length of 8 to 10 inches without breaking, the material is then considered to be both highly plastic and highly compressive (CH). If the soil cannot be ribboned, it is nonplastic (ML or MH). If it can be ribboned only with difficulty into short lengths, the soil is considered to have low plasticity (CL). The roll test and the ribbon test complement each other in giving a clearer picture of the degree of plasticity of soil.

2-16. WET SHAKING TEST

a. Sample preparation. 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 that the soil will not stick to the fingers upon remolding or just below the "sticky limit".

b. Testing. The sample is then placed in the palm of the hand and shaken vigorously. This is usually done by jarring the hand on the table or some other firm object, or by jarring it against the other hand. The soil is said to have given a reaction 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". Then, upon squeezing the sample between the thumb and forefinger of the other hand, this surface water will quickly disappear, the surface will become dull, the material will become firm, resisting deformation, and cracks will occur as pressure is continued, with the sample finally crumbling like a brittle material. 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 and increase the voids space, and the water is drawn 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.

c. **Typical reactions.**

(1) **Principles.** 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 degrees of reaction to this test. Even a small amount of clay will tend to greatly retard this reaction. Some of the descriptive terms applied to the different rates of reaction to this test are as follows:

(2) **Sudden or rapid.** A rapid reaction to the shaking test is typical of nonplastic, fine sands and silts.

(3) **Sluggish or slow.** 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.

(4) **No reaction.** Obtaining no reaction at all to this test does not indicate a complete absence of silt or fine sand.

2-17. ODOR TEST

Organic soils of the OL and OH groups usually have a distinctive, musty, slightly offensive odor which, with experience, can be used as an aid on their identification. This odor is especially apparent from fresh samples. It is gradually reduced by exposure to air, but can again be made more pronounced by heating a wet sample.

2-18. BITE OR GRIT TEST

a. **Principles:** The bite and grit test is a quick and useful method of identifying sand, silt, or clay. In this test, a small pinch of the soil material is ground lightly between the teeth and the soils are identified as follows:

b. **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 sand.

c. **Silty soils.** The silt grains are so much smaller than sand grains that they do not feel nearly so harsh between the teeth, and are not particularly gritty although their presence is still easily detected.

d. **Clayey soils.** The clay grains are not at all gritty, but feel smooth and powdery like flour between the teeth. Dry lumps on clayey soils will stick when lightly touched with the tongue.

2-19. SLAKING TEST

The slaking test is used to assist in determining the quality of certain soft shales and other soft "rocklike" materials. The test is performed by placing the soil in the sun or in an oven to dry, and then allowing it to soak in water for at least 24 hours. The strength of the soil is then examined. Certain type of shale will completely disintegrate, losing all strength.

2-20. ACID TEST

The acid test is used to determine the presence of calcium carbonate and is performed by placing a few drops of hydrochloric acid on a piece of the soil. A fizzing reaction (effervescence) to this test indicates the presence of calcium carbonate, and the degree of reaction gives an indication of the concentration. Calcium carbonate normally is desirable in a soil because of the cementing action it provides to add to the stability. (In some very dry noncalcareous soils, the absorption of the acid creates the illusion of effervescence. This effect can be eliminated in all dry soils by moistening the soil prior to applying the acid.) Since this cementation normally is developed only after a considerable curing period, it cannot be counted upon for strength in most military construction. The primary use for this test is, therefore, to permit better understanding of what appears to be abnormally high strength values on fine-grained soils which are tested in-place, where this property may exert considerable influence.

2-21. SHINE TEST

The shine test is another means of measuring the plasticity characteristics of clays. A slightly moist or dry piece of highly plastic clay will give a definite shine when rubbed with a fingernail, a pocket knife blade, or any smooth metal surface. On the other hand, lean clay will not display any shine, but will remain dull.

2-22. FEEL TEST

a. **Principles.** The feel test is a general purpose test, and one that requires considerable experience and practice before reliable results can be obtained. The extent of its use will grow with increasing familiarity with soils. Some of the following characteristics can be readily estimated by proper use of this test.

b. **Moisture content.** The natural moisture content of a soil is of value as an indicator of the drainage characteristics, nearness to water table, or other factors which may affect this property. A sample of undisturbed soil is tested by squeezing it between the thumb and forefinger to determine its consistency. The consistency is described by such terms as "hard", "stiff", "brittle", "friable", "sticky", "plastic", or "soft". The soil is then remolded by working it in the hands, and changes, if any, are observed. By this test, the natural water content is estimated relative to the liquid or plastic limit of the soil. Clays which turn almost liquid on remolding are probably near or above the liquid limit. If the clay remains stiff, and crumbles upon being remolded, the natural water content is below the plastic limit.

c. **Texture.** The term "texture", as applied to the fine-grained portion of a soil, refers to the degree of fineness and uniformity. It is described by such expressions as "floury", "smooth", "gritty", or "sharp", depending on the sensation produced by rubbing the soil between the fingers. Sensitivity to this sensation may be increased by rubbing some of the material on a more tender skin area such as the wrist. Fine sand will feel gritty. Typical dry silts will dust readily, and feel relatively soft and silky to the touch. Clay soils are powdered only with difficulty but become smooth and gritless like flour.

2-23. HASTY METHOD

a. **Principles.** With the standard methods of field identification supplemented with a few simplified field tests, an approximate and hasty classification can be obtained of almost any soil. The three simple or hasty tests outlined below will, for the most part, eliminate the need for specialized equipment such as sieves. The results of these tests, when used or supplemented with the results of the tests described in the previous paragraphs, will give at least a tentative classification to almost any soil. The schematic diagram (fig 2-3) may be used as a guide to the testing sequence in the process of assigning a symbol to a sample of soil.

b. **Sedimentation test.**

(1) **Principles.** From the visual observation test described previously, it is relatively simple to approximate the comparative proportions of sand and gravel in a sample of soil by spreading a 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. By placing a small representative amount of soil (such as a heaping tablespoon measure) in a transparent cup or jar, covering with about 5 inches of water, and agitating by stirring or shaking, the soil will be completely suspended in the water. With most cohesive soil 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 size particles, in time periods approximately as indicated in table 2-2.

TABLE 2-2. *Settling Times for Small Particles*

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

(2) **Differentiation of coarse and fine fractions.** The most important use of the sedimentation test is to differentiate the coarse (0.072 mm) fraction from the fine fraction of a soil. Since all of the particles of soil larger than 0.072 mm 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 a #200 sieve.

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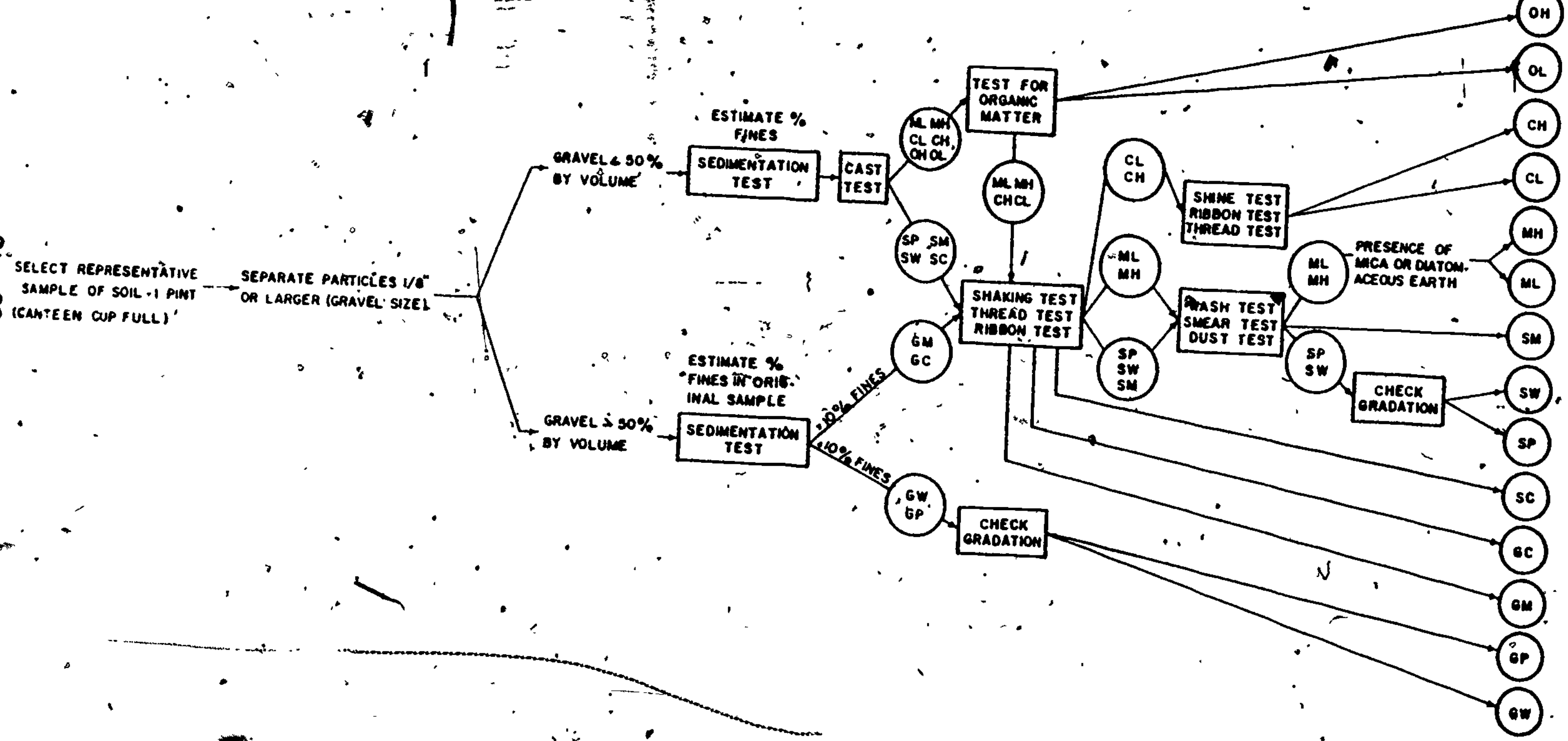

Figure 2-3. Suggested procedure for field identification.

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(3) **Separation of No. 40 sieve soil fraction.** 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 outlined above, 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 the fine sand range.

(4) **Clay lumps.** A difficulty that will be encountered with many clay soils stems from the fact that 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 have a somewhat slippery feel and further mixing and grinding with a wood stick will be necessary to help break up these lumps.

c. **Cast test.** The cast test refers to the strength of a moist soil sample when squeezed in the hand. It is used to indicate the approximate type and quantity of fines present in the sample. The correct amount of water to add to the soil must be estimated by trial and error, although it can generally be stated that maximum cohesion or attraction between the individual soil particles normally will occur when the soil is damp but not sticky. The test consists of compressing a handful of the moist soil into a ball or cigar-shaped cast and observing its ability to withstand handling without crumbling. While experience is desirable in making predictions based upon this test, table 2-3 may serve as a general guide of the behavior of different soil types when formed into a cast and tested.

d. **Wash, dust, and smear tests.** A small amount of silt (less than 5 percent), when intermixed with a coarse-grained soil, normally will not lessen the value of the soil as a construction material. However, increasing quantities of silt will sharply reduce the strength and interfere with the free drainage characteristics of the coarse-grained soil, thus making it less desirable as a road or airfield construction material. To decide what constitutes a harmful concentration of silt by the field identification methods generally requires extensive field experience. If the dust, wash, and smear tests are first practiced on soils of known silt contents, they can be used to produce a fairly accurate result. In the dust test, when a completely dry sample of soil (with the gravel portion removed) is dropped from a height of 1 or 2 feet onto a solid surface, a silt content higher than 10 percent will generally cause a fairly large amount of dust to be produced. In the wash test, an identical soil sample, as above, is placed in the palm of a hand, and covered with about 1/8 inch of water. If the water becomes completely discolored and hides the sand grains, this indicates that the soil sample contains a silt content higher than 5 percent. In the smear test, a sample of soil, again with the gravel portion removed, is moistened to just below the "sticky limit", and then smeared between the thumb and forefinger. When it produces a gritty, harsh feel, this indicates that it contains a silt content of less than 10 percent. A rough, less harsh feel, however, indicates that the sample contains more than 10 percent silt. It should be emphasized that all of the above tests require the testing engineer to have some experience in their use before they can be considered as reliable indicators of the silt contents.

TABLE 2-3. *Cast Test Reactions*

Soil type	Reaction to handling
GP, SP, SW, GW	Cast crumbles when touched.
SM, SC	Cast withstands careful handling.
ML, MH	Cast can be handled freely.
CL, CH	Cast withstands rough handling.

SELF TEST

Note: The following exercises comprise a self test. The figures following each question refer to a paragraph containing information related to the question. Write your answer in the space below the question. When you have finished answering all the questions for this lesson, compare your answers with those given for this lesson in the back of this booklet. Do not send in your solutions to these review exercises.

1. The objective of soil surveys is to gather as much information of engineering significance as possible about the subsurface conditions for a specified area. List as many specific items of engineering significance concerning soils as you can think of. (2-1c)

2. When using air photographs to delineate and identify soils, the drainage pattern is very important. What does a very simple drainage pattern frequently indicate? (2-2d(4))

3. In planning a soil survey in a theater of operations, how deep should preliminary borings be made in cut areas, measured from the final subgrade elevation? (2-3c(2)(b))

4. A soil profile can be developed from the information obtained in a soil survey. Give a brief definition of a soil profile. (2-4a)

5. The information obtained in a soil survey will form the basis for the technical soils report. What part of the technical soils report receives the most emphasis? (2-6)

6. Under the Unified Soil Classification System, all soils are divided into three major categories. What are these three categories? (2-7b)

7. The USCS further divides soils which have been classified into the major soil categories by using letter symbols. What is the symbol used to designate a silty sand? (2-7c, chart I)

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8. Course grained soils are defined as those in which at least half the material by weight is retained on a number 200 sieve. Course grained soils are divided into gravel and sand. Define gravel and sand. (2-8a, chart I)

9. One of the subdivisions of a course-grained gravel is "more than 12 percent passing number 200 sieve." Give three soil groups into which this material could eventually be designated (use chart III). (2-8c(1), chart III)

10. Under course-grained gravel containing more than 12 percent passing the number 200 sieve, you have to run the liquid limit and plastic limit test. If the liquid and plastic limit plot below the A line of the plasticity chart (chart I), what type of soil do you have? (use chart III) (2-8c(2), chart III)

11. Assume a course grained gravel containing more than 12 percent passing a number 200 sieve, and with liquid and plastic limits plotting in the hatched zone on the plasticity chart (chart I). What type soil is indicated? (use chart III) (2-8d, chart III)

12. Assume a fine-grained soil, i.e. more than 50 percent passing a number 200 sieze. If the liquid limit test indicates less than 50, what type soil(s) is it possible to have? (use chart III) (2-9b(1), chart III)

13. Highly organic soils are given the classification Pt. Why are there no laboratory tests established to identify and classify these soils? (2-10)

14. Lack of time and facilities often make laboratory testing of soils impracticable. What is the one most useful item of equipment for field identification of soils? (2-11c(2))

15. To an experienced soils analyst, color may be of considerable help in identifying soil types. What may be indicated if a fine-graded soil appears in dark or drab shades of gray or brown, or almost black?. (2-12b)

16. The breaking test is performed on number 40 sieve material which has been molded into a small pot and allowed to dry completely. If the sample can be broken with great effort but cannot be powdered, what type of soil is it? (2-13c(3))

17. The ribbon test is performed on material passing a number 40 sieve and squeezed between the thumb and forefinger to form a ribbon 1/8 to 1/4 inch thick. If the sample cannot be ribboned, what type soil could it be? (2-15)

18. During the wet shaking test the sample developes a "livery" appearance readily if it consists of fine sands and silts. If this reaction is very sluggish, what can be said of the content of the sample? (2-16c(3))

19. When performing the sedimentation test, a small representative sample of soil is placed in a transparent cup or jar, agitated, and then allowed to settle. What is the approximate time it will take for particles of 0.4 millimeter diameter to settle through 5 inches of water? (2-23b(1), table 2-2)

20. In performing the cast test, a moist soil sample is squeezed in the hand. If the cast crumbles when touched, what type of soil(s) could be present? (2-23c, table 2-3)

LESSON 3

SOIL COMPACTION

CREDIT HOURS ---------------------------- 3

TEXT ASSIGNMENT ---------------------------- Attached memorandum.

MATERIAL REQUIRED ---------------------------- Chart II, Characteristics Pertinent to Roads and Airfields.

LESSON OBJECTIVE ---------------------------- Upon completion of this lesson you should be able to accomplish the following in the indicated topic areas:

1. **Moisture-Density Relationships.** Be able to explain the relationship between moisture content, dry density, and compactive effort.

2. **Optimum Moisture Content.** Determine by test the optimum moisture content for a soil under investigation.

3. **Compaction.** Select type of compaction equipment most suited for each soil condition and control compaction in the field to attain the degree of compaction required for a specific project.

4. **Trafficability.** By use of special tools and instruments developed for the purpose, determine the capability of the soil to support traffic according to type and quantity.

ATTACHED MEMORANDUM

3-1. PRINCIPLES

Compaction is the process of artificially densifying a soil. Densification is accomplished by pressing the soil particles together into a closer state of contact, with air or water being expelled from the soil mass in the process. With relation to compaction, the density of a soil is normally expressed in terms of dry density or dry unit weight, common units of measurement being pounds per cubic foot. Occasionally, use is made of the wet density or wet unit weight.

3-2. MOISTURE-DENSITY RELATIONSHIP

a. **Principles.** Nearly all soils exhibit a similar relationship between moisture content and dry density when subjected to a given compactive effort. This relationship is indicated in figure 3-1. For each soil there is developed a maximum dry density at an optimum moisture content (OMC), for the compactive effort used.

b. **Hydration.** Upon the addition of small increments of water to a completely oven-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 of the solidified type i.e., is adsorbed to the soil particles. This water does not aid compaction by acting as a lubricant, since it is firmly attached to the soil particles.

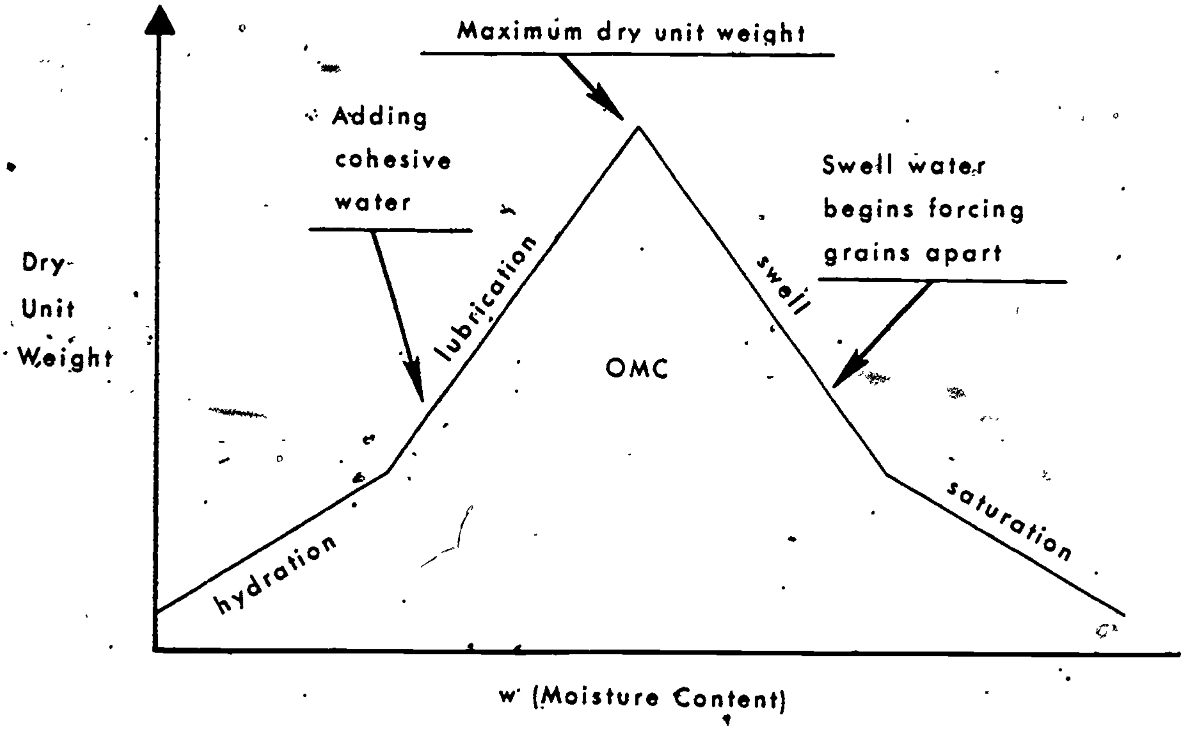


Figure 3-1. Compaction curve.

c. **Lubrication.** Adding more water brings the soil to a point where a slight change in moisture begins to produce a large increase in density. The lubrication phase of the compaction curve has been reached when the additional water produces a cohesive film around the individual soil particles. The lubrication effect of moisture produces greater densities to the point where the cohesive films become so thick that density begins to decrease with further increases in moisture content.

d. **Swell.** The addition of water increases the film around the soil particles thus forcing the soil particles apart, decreasing the dry density. This sharp decrease of dry density indicates that the swell phase of the compaction curve has been reached, and characterizes this phase.

e. **Saturation.** Finally, with further increases in moisture content, free water added to the soil will find the void spaces. This is known as the saturation phase.

f. **Zero air void curve.** This curve represents the dry density and water content of a soil when the voids are completely filled with water and serve as a control to the compaction curve. Any values of the dry density curve which plot to the right of the zero air voids curve are in error. The error may be in the test measurements, the calculations, or the specific gravity. Compute points for plotting the zero air void curve as follows:

$$\gamma d = \frac{62.4\ G}{1 + \frac{wG}{100}}$$

where:

w = water content for 100 percent saturation

γd = dry density of soil

62.4 — unit weight of 1 cubic foot of water, and

G = specific gravity of the soil

Computed values for plotting the zero air voids curve are shown in tabular form on table 3-1.

3-3. COMPACTION REQUIREMENTS

a. **Factors of compaction.**

TABLE 3-1. *Data for Zero Air Voids Curve.*

Specific Gravity of Soil	50	55	60	65	70	75	80	85	90	95	100	105	110	115	120	125	130	135	140	145
	Water Content, w, in Per Cent of Dry Weight for Dry Unit Weight, γ_d, in Pounds per Cubic Foot of																			
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	1.4
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	[illegible]	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	[illegible]	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 per cent.
Data for zero air voids curve computed from the following formula (using a γ_w value of 62.43):

$$S = 100\% \cdot \frac{w}{\frac{\gamma_w}{\gamma_d} - \frac{1}{G_s}}$$

The above equation may also be used to determine curves representing degrees of saturation other than 100 per cent.

The underlined value at the top of each column is the dry unit weight γ_d of the soil in pounds per cubic feet. The body of the table is the water content in percent of dry unit weight at 100% saturation.

(1) **Compactive effort.** For each compactive effort which is used in compacting a given soil there is a corresponding optimum moisture content and a maximum density. 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 any great extent, thus decreasing the dry density. The relationship between compactive effort and density is not linear, and a considerably greater increase in compactive effort may be required to increase the density of a clay soil than is required to bring about the same change in density of a sand. In the field the compactive effort is a function of the weight of the roller and the number of passes for the width and depth of the area of soil which is being rolled. Increasing the weight of the roller or the number of passes increases the compactive effort. Other factors which may be of consequence include lift thickness, contact pressure, and in the case of sheepsfoot rollers, the size of the tamping feet. One should note that in the forthcoming parts of this lesson each roller has a recommended depth of soil to be compacted at one time (lift thickness). The lift thickness should be closely adhered to. Figure 3-2 shows how the dry density will vary with compactive effort. The OMC varies indirectly as the compactive effort. If the water content in a soil is increased, it makes the soil more workable. However, if the compactive effort is increased, the soil does not have to be as workable to obtain the maximum dry density. In other words, the OMC will be decreased, with increasing compactive effort.

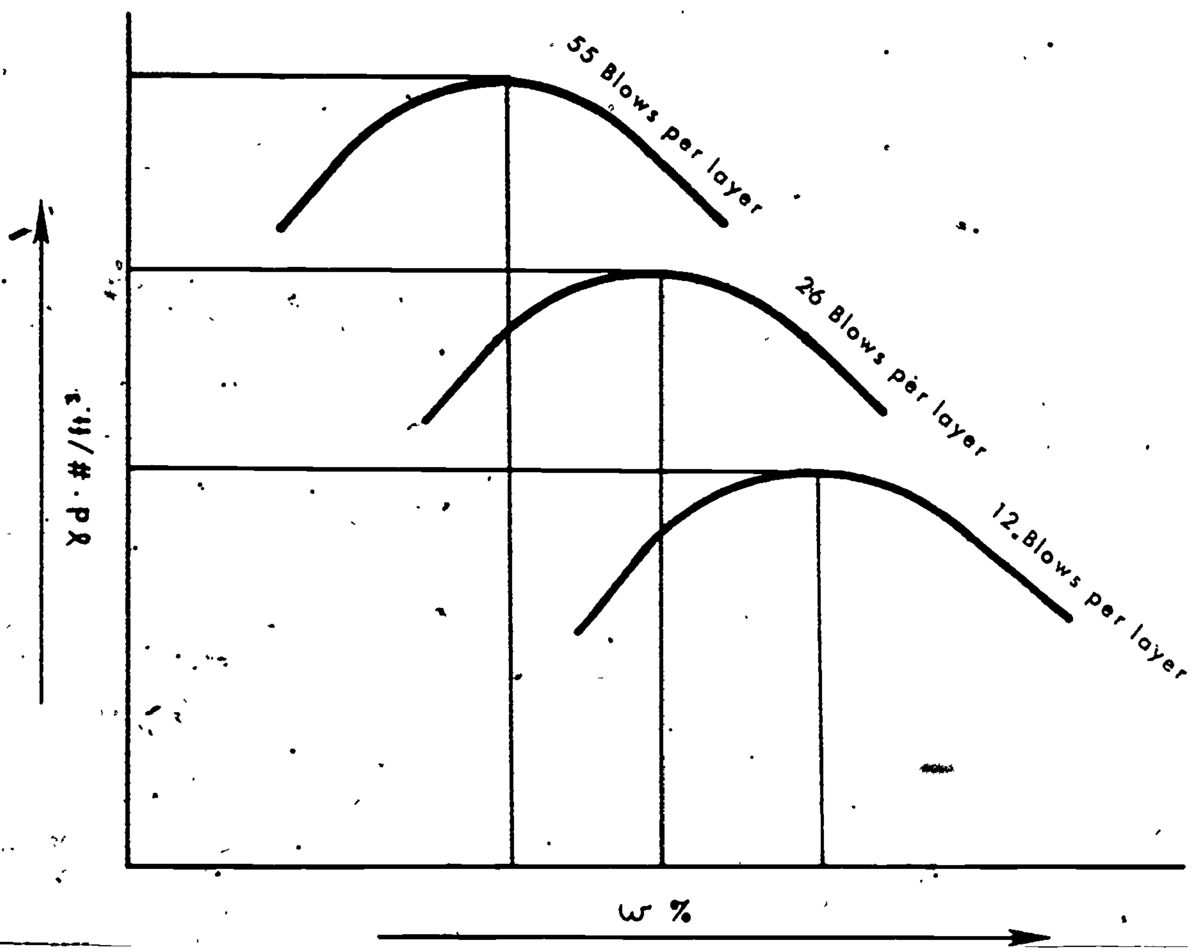


Figure 3-2. Compactive effort.

(2) **Soil types.** Different soils will have varying compactive characteristics. Soils which are very light in weight may have maximum densities under a given compactive effort as low as 60 pounds per cubic foot. Under the same compactive effort, the maximum density of a clay may be in the range of 90 to 100 pounds per cubic foot, while that of a well graded, coarse granular soil may be as high as 135 pounds per cubic foot. Moisture density relationships for seven different soils are shown in figure 3-3. Information relative to the compacted dry unit weights of the soil groups of the Unified Soil Classification System is given in Chart II, Characteristics Pertinent to Roads and Airfields. Chart II is bound in the back of this booklet. Note that moisture content is much more critical in obtaining maximum density for coarse grained soils than for fine grained soils.

(3) **Moisture content.** As explained previously, regulated quantities of water are essential for proper compaction. Excessive water hinders compaction. At OMC the soil particles are lubricated so that they can be compacted into the densest mass possible with a given compactive effort.

b. **Modified AASHO test.**

(1) **Principles.** The compaction control test used by the Corps of Engineers is a modification of the standard AASHO (American Association of State Highway Officials) Method T-99-49, and conforms entirely to ASTM Designation D 1557-58T. This method of test is intended for determining the relationship between the moisture content and density of soils when compacted in a mold of a given size with a 10-pound tamper dropped from a height of 18 inches.

(2) **Apparatus.** The apparatus used in the modified AASHO compaction test is described below.

(a) **Proctor mold.** A cylindrical metal mold of 4.0 ± 0.005-inch diameter and 4.584 ± 0.005-inch height, having 1/30 cubic foot volume. This mold is fitted with a detachable base plate and a removable extension collar approximately $2\frac{1}{2}$ inches high. This mold may be of the "split" type, consisting of two half-round sections or a section of pipe split along one element, which can be securely locked in place to form a cylinder. The 4.0-inch diameter mold is used for samples composed entirely of material passing a U. S. Standard No. 4 sieve (0.18-inch diameter).

(b) **CBR mold.** A cylindrical metal mold (fig 3-4) 6.0 inches in diameter and 4.5 or more inches in height. If the mold is more than 4.5 inches high, generally 7 inches, the height of the compacted sample is controlled to 4.5 or 5 inches by means of a 2.0 or 2.5-inch spacer disk, which is inserted as a false bottom in the mold. This mold is fitted also with a detachable base plate and a removable extension collar approximately $2\frac{1}{2}$ inches high. The 6.0-inch diameter mold is used for compacting samples containing material retained on the No. 4 sieve (0.18-inch diameter). The volume of the mold is 0.075 cubic foot at an internal height of 4.584 ± 0.005 inches and 0.0818 cubic foot at an inside height of 5.00 inch ± 0.005 inch.

(c) **Compaction tamper.** The compaction tamper, is of the sliding weight type as shown in figure 3-5. It consists of a 2-inch diameter steel tamping foot, a $\frac{5}{8}$-inch steel rod, a 10-pound weight with an 11/16-inch hole through the center, and a handle. Construction of the tamping foot and weight are such that tamping blows can be applied adjacent to the sides of the mold. The rod is attached to the tamping foot with a spring cushion. Distance between tamping foot and handle is such that when the weight is raised until it touches the handle, the fall to the tamping foot will be 18 inches. The maximum allowable weight of the assembled compaction tamper is $17\frac{1}{2}$ pounds.

(d) **Laboratory balance.** One 25-kilogram capacity, sensitive to 1 gram, and one 5-kilogram capacity, sensitive to 0.1 gram are needed.

(e) Sieves. 2-inch, $\frac{3}{4}$-inch, and No. 4 sieves conforming to the requirements of the specifications for sieves for testing purposes are necessary.

SOIL TEXTURE AND PLASTICITY DATA

No.	Description	Sand	Silt	Clay	L.L.	P.I.
1	Well Graded Loamy Sand	88	10	2	16	NP
2	Well Graded Sandy Loam	72	15	13	16	0
3	Med. Graded Sandy Loam	73	9	18	22	4
4	Lean Sandy Silty Clay	32	33	35	28	9
5	Loessial Silt	5	85	10	26	2
6	Heavy Clay	6	22	72	67	40
7	Very Poorly Graded Sand	94	6		NP	

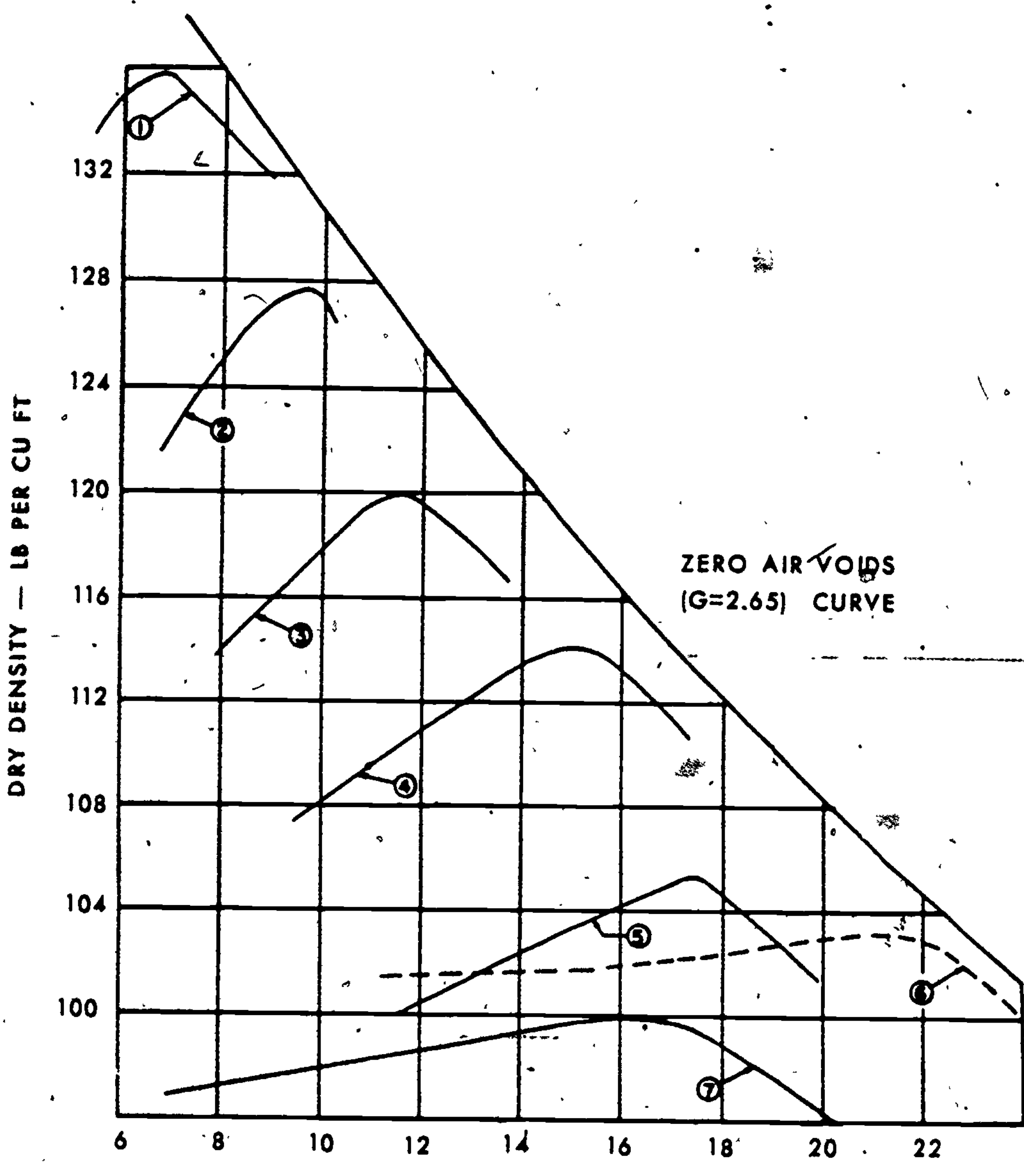


Figure 3-3. Moisture density relationships for seven soils compacted according to the standard AASHO compaction procedure.

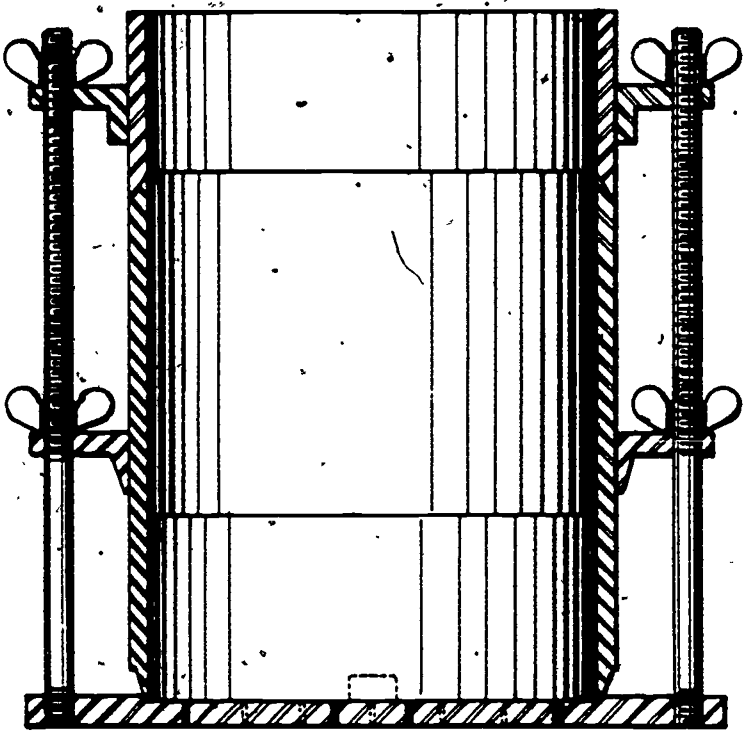


Figure 3-4. CBR mold.

(f) **Straightedge.** A steel straightedge 12 inches in length and having one beveled edge is used.

(g) **Mixing tools.** Miscellaneous tools such as mixing pan, spoon, trowel, spatula, etc., or a suitable mechanical device for thoroughly mixing the sample of soil with increments of water is needed for mixing.

(3) **Preparation of sample.**

(a) **Minus No. 4 materials.** For soils which contain no material retained on a No. 4 sieve, approximately 30 pounds of dry weight (approximately 5 pounds for each Proctor mold) of soil are normally required for the test. First air-dry the soil sufficiently to permit ready passage through a No. 4 sieve. All lumps in the soil should be carefully broken by rolling with a rolling pin. After screening, thoroughly mix the sample, make an initial water-content determination, and store the sample in an airtight container until ready for use.

(b) **Gravelly samples.** For soils containing gravel, the compaction test is performed in the CBR mold on material passing a ¾-inch sieve. First air-dry and screen through a ¾-inch screen. If the soil contains gravel particles larger than ¾-inch, it requires processing for size correction. Approximately 75 pounds of dry weight of material so processed are required for the test. The procedure for this processing is as follows:

Step 1. Separate the material on the ¾ inch and No. 4 mesh screens (0.18-inch).

Step 2. Compute the percentage by weight of material retained on each screen and passing the 0.18-inch screen.

Step 3. Discard the material retained on the ¾-inch screen and replace by an equal percentage by weight of sizes ¾ to 0.18-inch (No. 4 sieve). The percentage of material finer than the No. 4 mesh screen remains constant.

Step 4. Recombine the two constituents of material ¾ inch to 0.18-inch and the material

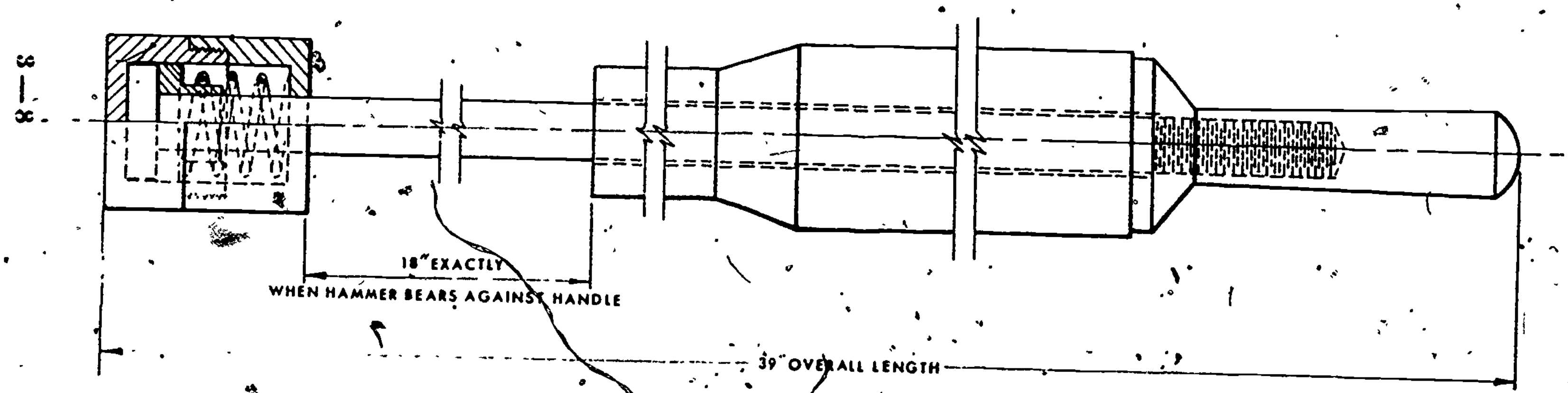


Figure 3-5. - Compaction tamper.

passing the 0.18-inch (No. 4) sieve and mix well.

The above adjustment insures that the proper amount of gravel is in the soil tested and that the maximum particle size is not exceeded.

(c) **Example.** Assume total weight of natural sample airdried = 110 pounds.

Natural sample			Processed sample	
Screen opening in.	Weight retained, lb	% Retained	Desired % retained	Computed weight, lb
3/4	16.0	14.6	0	0
0.18	44.0	40.0	40.0 + 14.6 = 54.6	44.0
(Passing 0.18)	50.0	45.4	45.4	36.5
Total	110.0	100.0	100.0	80.5

An initial moisture content is taken from the processed sample and the sample is stored in an airtight container until ready for use.

(4) **Procedure.**

(a) **Minus No. 4 sieve materials.** The soil is compacted in the 4-inch diameter mold except in cases where the compacted specimens are to be used for CBR test or some other test which requires a different size compacted specimen. The step-by-step procedure for compacting samples in the 4-inch diameter mold is as follows:

Step 1. From the previously prepared material weigh quantity of airdried soil equal to about 2000 grams dry weight.

Step 2. Moisten and thoroughly mix the material with a measured quantity of water sufficient to cause the soil to adhere or ball together slightly when squeezed firmly in the palm of the hand.

Step 3. Store material in an airtight container for approximately 24 hours to permit moisture content to become uniform.

Step 4. Repeat step 1 for at least four additional specimens. For each specimen, increase test water content by approximately 2 percent over that of the previous specimen and repeat step 3 for all specimens.

Step 5. Cover the mold with a light coat of oil or wipe off excess oil. Weigh the compaction mold to the nearest gram and record the weight on a data form similar to that shown in figure 3-6.

Step 6. Attach mold with collar to base plate and place the mold on a concrete floor or other rigid support such as a pedestal. If using the CBR mold, place the 2.0 or 2.5-inch spacer disk in the bottom and insert a filter paper on top of the spacer disk.

Step 7. Place a sufficient amount of the prepared sample in the compaction mold to yield a 1-inch compacted layer.

Step 8. Compact the material in the mold with 25 blows, uniformly distributed over the area, using the 10-pound tamper with height of drop of 18 inches. Using a sharp-pointed instrument, scarify the surface of the compacted layer to a depth of approximately 1/4 inch prior to placing the next layer.

Step 9. Repeat steps 7 and 8 for the next four layers. The fifth layer should extent slightly above the top of the mold.

Step 10. Use a knife or spatula to cut around the inside edge of the collar. Remove collar from the mold and carefully trim the excess portion of the compacted material to the exact level of the top of the mold, using a straightedge.

Step 11. Remove mold, with compacted sample therein, from base plate, weigh the mold

CORPS OF ENGINEERS — U. S. ARMY

COMPACTION CONTROL TEST

PROJECT Ardmore AFB	DATE 9 June 68
SAMPLE NO. 3243	JOB NO. 32181

INITIAL WATER CONTENT, w_o = 4.5 % — AS MOLDED — SOAKED ✓

55 BLOWS PER EACH OF 5 LAYERS 10 -LB HAMMER 18 -IN. DROP

Specimen		A	B	C	Remarks
Desired dry weight	W_s'	5000			
$1 + w_o$		1.045	1.	1.	Subgrade Material
Soil weight $W_s'(1+w_o)$ =	W_o	5225			
Bowl tare	W_b				
$W_o + W_b$					
Test water content	w'	6 %	%	%	
Add water, $W_s'(w' - w_o)$		75			
Mold		4			
Weight mold + Soil	W	14787			
Mold tare	W_m	10742			
Less tare, $W - W_m$ =	W_c	4045			
Average water content	w	5.3 %	%	%	
Mold Constant	C	0.0298	0.	0.	
Wet density, CW_c =	m	120.5			
Dry density, $m/1 + w$ =	d	114.4			

WATER CONTENTS		A		B		C	
Tare		4	29				
Tare + wet soil		159.8	180.3				
Tare + dry soil		153.0	172.5				
Water	W_w	6.8	7.8				
Tare		25.3	25.4				
Dry soil	W_s	127.7	147.1				
Water Content	w	5.3 %	5.3 %	%	%	%	%

Densities in pounds per cubic foot. Weights in grams.

TECHNICIAN	COMPUTED	CHECKED
ACB.	SSW	TH

Figure 3-6. Data form for compaction control.

plus wet soil to the nearest gram, and record the weight.

Step 12. Remove the compacted sample from the mold and obtain two specimens for water-content determinations, one from the top portion of the compacted sample and the other from the bottom. The water-content specimens are weighed to 0.1 gram and the weights recorded on the data sheet.

Step 13. Repeat steps 6 through 12 for the remaining specimens.

In general, five compacted specimens prepared according to the above procedure will completely define a compaction curve. However, in some cases more points are necessary. Compare the wet weights of the various compacted samples to determine if the optimum water content has been reached. The optimum water content and maximum density have been reached if the wettest samples compacted show a marked decline in weight over drier specimens.

(b) **Gravelly samples.** Procedure for compacting gravelly samples is the same as for minus No. 4 sieve material, except that the test is performed in the 6.0-inch diameter CBR mold and the number of blows of the compaction tamper is increased to 55 blows per layer instead of 25 as used in the 4.0-inch diameter Proctor mold. This results in equal compactive efforts for the two molds.

(5) **Calculation.** From the data obtained the following calculations can be made:

(a) **Wet unit weight.** As the five samples are compacted the wet unit weights can be computed. Having the weight of the mold and base plate and the weight of the mold and base plate plus compacted sample, you can take the difference in weight as the weight of the compacted sample. The volume of the compacted sample is 1/30th cubic foot for the Proctor mold or 0.0818 cubic foot for the 7 inch CBR mold with the 2 inch and 0.0750 cubic foot for the 7 inch CBR mold with 2.5-inch spacer disk. The wet unit weight is equal to the wet weight of the compacted soil divided by the volume of the compacted soil.

(b) **Moisture content.** The water content is equal to

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

(c) **Dry unit weight.** After the moisture content samples have been dried in the oven (4 to 24 hours) and the water contents computed, the dry unit weights are computed using the following formula:

$$\text{Dry unit weight} = \frac{\text{Wet unit weight}}{1 + \frac{\text{Moisture content (w)}}{100}}$$

The OMC curve is plotted on arithmetic graph paper after the moisture contents and the dry unit weights are known.

(d) **Recording test data.** A suggested form for use in recording the test data is shown in figure 3-6. The results are plotted in the form of a moisture-density diagram and a curve drawn through the points. Figure 3-7 shows typical moisture-density diagrams for compaction tests, in a CBR mold made with 12, 26, and 55 blows per layer. The maximum point on the curve obtained with 55 blows per layer (Modified AASHO compactive efforts) is designated 100 percent modified AASHO density and the moisture content at this point is the optimum moisture at 100 percent modified AASHO density.

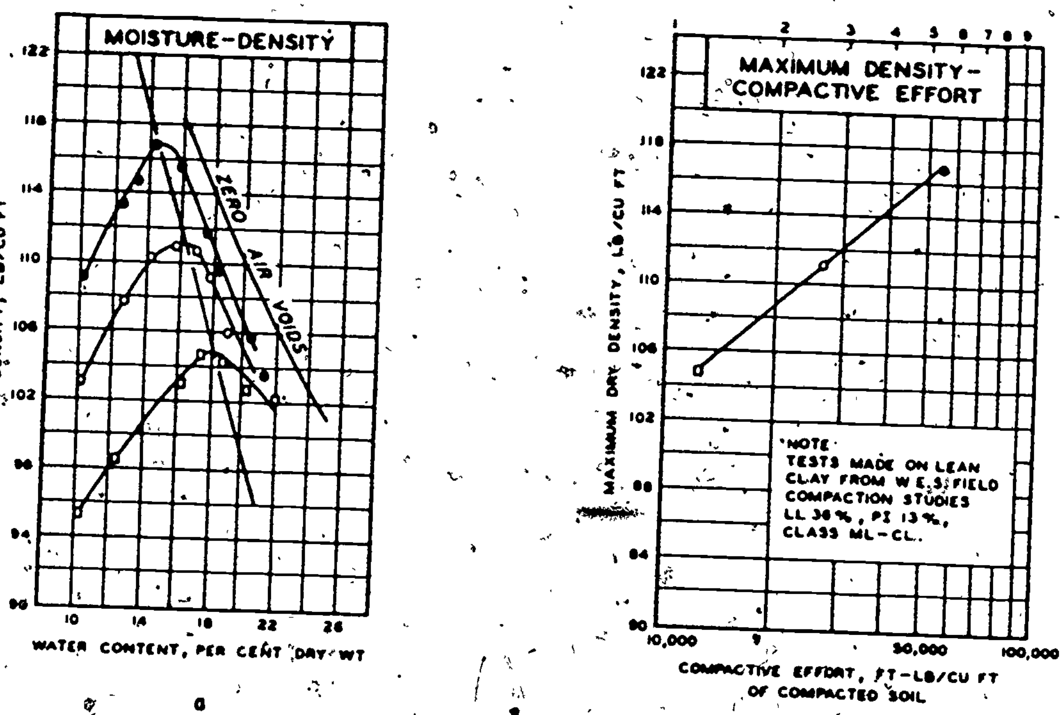
(6) **Adjustment of water content.** Before beginning the discussion on how the water content is varied it is important to remember that the only soil density that can be determined during the period of testing is the wet unit weight. You will recall that the relationship between wet unit weight and dry unit weight is:

$$\text{Dry unit weight or dry density} = \frac{\text{Wet density}}{1 + \frac{w}{100}}$$

Thus, it is necessary that "w" be known in order to convert wet unit weight into dry

CORPS OF ENGINEERS

	LAYERS	BLOWS PER LAYER	WT OF HAMMER	DROP IN INCHES	Theoretical COMPACTIVE EFFORT FT-LB/CU FT for 4.5 inch sample using 2.5 inch spacer disk
●	5	56	10 LB	18	55,900 (MOD. AASHO)
○	5	26	10 LB	18	26,400
□	5	12	10 LB	18	12,200 (STD. AASHO)



TYPICAL PLOTS OF COMPACTION TEST RESULTS

U. S. ARMY

NOTE:
The energy measure in ft-lb/cu.ft. is approximate only and depends entirely on height of sample specimen 4.5 or 5.0 inches as well as on the excess amount of material trimmed off the sample.

Figure 3-7. Typical plots of compaction test results.

unit weight. Since it may take as long as 24 hours to complete the water content tests, the dry unit weights will not be known as the soil samples are compacted; however, the wet unit weight can be computed. The problem of adjusting the water contents of the five or more prepared soil samples is simply one of having enough water added to each of the five samples so that they are at five different water contents ranging from below to well above the optimum. As the Modified AASHO compactive effort is applied to these samples at different water contents, enough test data will be provided to plot the OMC curve. At least two points are needed on each side of OMC to establish the curve. Beginning compaction with the first sample wet enough to just ball together when squeezed and released in the hand, and having each subsequent sample 2-4% wetter, in order that the fourth and fifth samples will produce a decrease in the wet unit weight, should insure that the dry unit weights when computed will provide an acceptable compaction curve. One other hint that will be of aid in selecting the increments by which the water contents of the five samples should be increased (from 2-4%) can be taken from the shapes of typical compaction curves. Generally, the more plastic the soil, the flatter will be the OMC curve and the higher the OMC. By estimating the OMC from experience, it is possible to select appropriate water content increments during the test.

c. **Compaction specifications.**

(1) **Optimum moisture content.** The OMC is that moisture content at which the maximum dry density is obtained. By determining the highest point on the compaction curve (apex) and dropping a vertical down to the horizontal moisture scale or line, it is found that OMC for this particular soil (figure 3-8) is 10%.

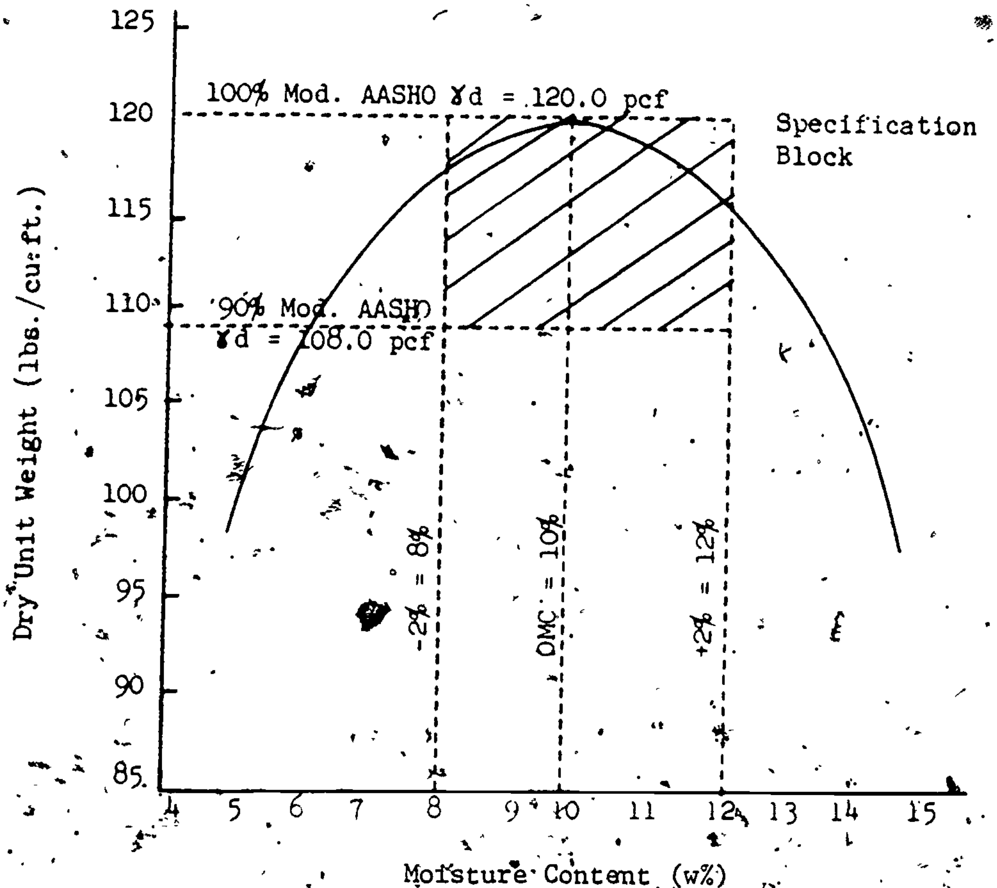


Figure 3-8. Typical compaction curve.

(2) **Maximum modified AASHO dry density.** The maximum modified AASHO dry density or 100% 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 (fig 3-8), in this case 120 pounds/cubic feet.

(3) **Percent moisture.** If moisture content is not kept close to the OMC it will require extra time and equipment effort to obtain the maximum dry density, because more passes will be required of the rollers to compact the soil properly. In the specifications for each job the limits for moisture content should be specified. The limits otherwise should be assumed as ± 2% around OMC. Using figure 3-8, where OMC is 10%, the moisture limits would be from 8% to 12%.

(4) **Percent compaction.** Figure 3-9 gives TO compaction specification. 100% compaction is the maximum modified AASHO dry density.

3-4 COMPACTION EQUIPMENT

a. **Principles.** Some types of compaction equipment work well in one material but are entirely inadequate in others. It may prove necessary to use several types of equipment in conjunction with each other on one project. The equipment must be capable of handling the material needed for an entire depth of lift in one operation.

b. **Rotary-tiller mixer.** The rotary-tiller mixer is used to pulverize or mix materials in place. It is capable of mixing different size aggregates, including gravel, crushed stone, sand, clay, and broken up bituminous paving, with any suitable binder material, to build roads, runways, or parking areas. The action which takes place in the machine is shown in figure 3-10.

c. **Tandem gang disks.** Tandem gang disks are good for all types of material except coarse stone. They are very good for preparing foundations and breaking down chunks. They are also good for blending materials and uniformly mixing moisture into the material. The weighting of the gang disks to penetrate the entire lift of loose material to be compacted is of prime importance.

d. **Gang plow.** The gang plow may be used for all types of materials. It mixes moisture uniformly. It blends materials well but will not break down chunks.

e. **Spring-tooth harrow.** The spring-tooth harrow is used in low-plasticity materials only. Weighting of the harrow is important.

f. **Heavy-duty cultivator.** The heavy-duty cultivator is good for low-plasticity materials.

g. **Grader.** The grader blade is extensively used both for blending subgrade soils and for aerating them to reduce moisture content.

h. **Water truck.** The pressure-tank water truck gives a more uniform flow of water, although the gravity-type tank is still in use. The controls for the spray bars should be in the cab. Spray bars should not leak. To insure uniform water coverage the truck should be moving at uniform speed from the time immediately before the sprays are turned on until after the sprays are turned off.

i. **Sheepsfoot roller.** The sheepsfoot roller compacts from bottom-up and "walks out" as rolling progresses. It gives best results with cohesive material, being practically useless in clean sand. The density increases prior to "walk-out" with the number of passes or with the increased weight of the roller. The weight of the roller may be increased by adding water or oil to the drum. Do not add sand. Density increases with increased foot pressure until the supporting power of the soil is reached. Beyond this point the soil fails in shear, decreasing the bearing capacity, and the roller will not "walk out". Increasing the foot contact of the roller and maintaining the same unit or contact pressure will not produce discernible benefits. The thickness of lift is usually 6 inches with a maximum of 9 inches (compacted), with TOE equipment. The sheepsfoot roller leaves the upper 2 to 3 inches in a loose uncompacted state, but obtains an excellent bond with the

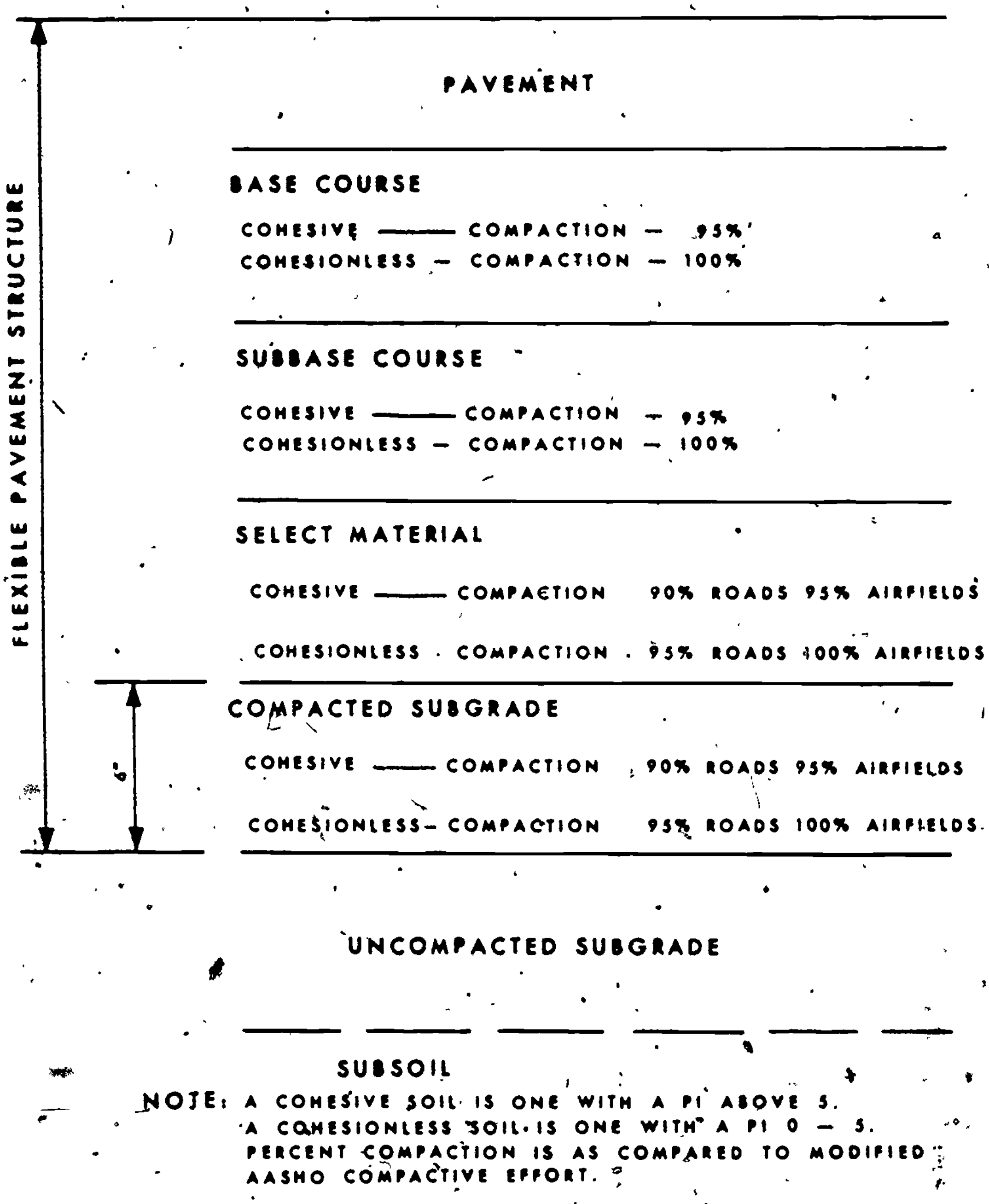


Figure 3-9. Compaction requirements for roads and airfields.

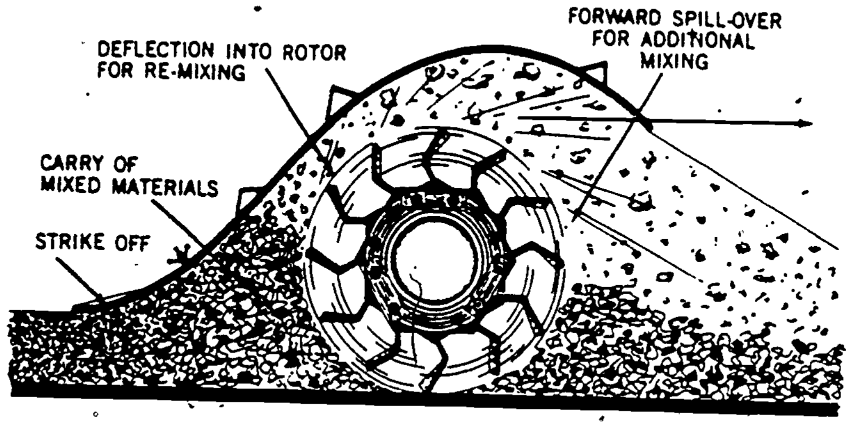


Figure 3-10. Mixing action in rotary-tiller mixer.

preceding construction lifts. Rolling with sheepsfoot rollers should be followed by rolling with a lighter type pneumatic tire roller to compact the upper 2 to 3 inches of the subgrade and subbase soil.

j. **Pneumatic-tired roller (50-, 100-ton, and larger "super" compactors).** Compacts from the top down. It can be used in compacting almost any type soil, provided the values of contact and tire pressure are adjusted for the soil which is being compacted. This roller is **least effective** with fine-grained plastic soils of high compressibility. The density increases with the number of passes. Increased density results from increased tire inflation pressure, just so long as a reasonable minimum wheel load is maintained. Conversely, changing the wheel load without changing the tire pressure produces no discernible results. Generally, 6-inch lifts are recommended; however, in granular soils lifts of 12 to 18 inches have proved satisfactory in some cases. This type of roller is usually supplied with both 90 and 150 psi tires.

k. **Vibratory compactor.** Compacts with a vibratory action which rearranges the soil particles into a denser mass. It gives best results with clean cohesionless sands and gravels. This type of compactor requires more maintenance than most standard rollers. Crawler type tractors produce some of the effects of a vibratory compactor and can in some cases be used for this type of compaction.

l. **Wobble-wheeled roller (pneumatic-tired).** Compacts with a kneading action from the top down. It gives best results in slightly cohesive soils. The thickness of lifts is from 4 to 6 inches. Density increases with passes.

m. **Steel-wheeled roller (three wheels).** Compacts from the top down. The thickness of lift is about 4 inches due to low unit pressure. Generally it is used for compacting cohesionless subgrade, base course, and wearing surfaces. Care must be exercised in rolling base course that material is not crushed by the roller.

n. **Pneumatic-tired roller (15-, 30-ton self-propelled).** Compacts from the top down. It can be used for compaction of almost any type of soil, like the towed and heavier pneumatic-tired rollers described previously.

o. **Equipment list.** Column 13 on Chart II, Characteristics Pertinent to Roads and Airfields, lists equipment that will usually produce the desired density with a reasonable number of passes when moisture conditions and lift thickness are properly controlled.

3-5. FIELD CONTROL OF COMPACTION

a. **Principles.** As has been emphasized previously, specifications for adequate compaction of soils used in military construction generally require the attainment of a certain minimum density in field rolling. This requirement is most often stated in terms of modified AASHO maximum density. With many soils the close control of moisture content is necessary to achieve the stated density with the available equipment. In the discussion which follows, it is assumed that the laboratory compaction curve is available for the soil which is being rolled, so that the maximum density and optimum moisture content are known, that the soil compacted in the laboratory and being rolled in the field are actually the same, and that the required density can be achieved in the field with the equipment available. There are many methods for finding the in-place density. The sand displacement method is presented because it is most commonly used.

b. In-place moisture content of soil.

(1) **Definition.** Moisture content (W) is the ratio, expressed as a percentage, of the weight of water in a soil mass to the weight of the dry soil matter.

(2) **Apparatus.** The apparatus required for this test procedure is: oven, preferably automatically controlled for 105-110° C; balance, sensitive to 0.01 gram; desiccator (this is not necessary if samples are weighed immediately after cooling); sample containers (seamless metal containers with lids are recommended).

(3) **Procedure.** The test procedure consists of the following steps:

(a) First, record on the suggested data sheet (fig 3-11) identifying information such as project, station, pit number, depth, etc., from which sample was taken.

(b) Place the sample in a container, weigh, and record the total weight of the container and wet soil. Also, record the number and empty weight of the containers. If the sample weighs less than about 100 gram, the weight should be recorded to the nearest 0.1 gram.

(c) Place the sample in an open container in an oven heated to 105-110° C. The sample should remain in the oven until the weight becomes constant. Generally, 8 to 12 hours are sufficient.

(d) Remove the container from the oven and place it in a desiccator to cool. Samples in containers too large for the desiccator may be allowed to cool to room temperature in the open air.

(e) After the sample has cooled, weigh it again and record the weight on the data sheet.

(4) **Computations.** The moisture content is equal to:

$$\frac{(\text{Weight of wet soil}) - (\text{Weight of dry soil})}{\text{Weight of dry soil}} \times 100$$

$$\text{or} \quad w = \frac{W_w}{W_s} \times 100$$

where W_w is the weight of the water and W_s is the weight of the dry soil.

(5) **Example.** An example is presented here of the computation of the moisture content of the portion of coarse-grained materials passing a No. 10 sieve when the moisture content of the total sample, gradation, and percentage of absorption of the coarse materials are known. Assume a clay-gravel material with the following characteristics:

Moisture content of total sample	3.6 percent
Material retained on No. 10 sieve	48 percent
Material passing No. 10 sieve	52 percent
Moisture content of material retained on No. 10 sieve	0.6 percent

The portion of the moisture content available to the material passing the No. 10 sieve would be: 3.6 – 0.6 = 3.0 percent of the dry weight of the total sample. On the basis of the dry weight of material passing the No. 10 sieve this would represent a moisture content of

$$\frac{3.0}{0.52} = 5.8 \text{ percent}$$

MOISTURE CONTENT -- GENERAL	Sheet 1 of 1
	Date 12/15/1948

Project Sheppard Air Force Base

NE-SW Runway — Location Sta. 10+50 ℄

Subgrade — Test Pit No. 1

				1					
	Sample No.			1					
1	Tare No.			2					
2	Weight in g	Tare and wet soil		128.3					
3		Tare and dry soil		113.1					
4		Water (2) - (3)	W_w	15.2					
5		Tare		44.5					
6		Dry soil (3) - (5)	W_s	68.6					
7	Moisture content		w	22.2 %	%	%	%	%	%

	Sample No.								
1	Tare No.								
2	Weight in g	Tare and wet soil							
3		Tare and dry soil							
4		Water (2) - (3)	W_w						
5		Tare							
6		Dry soil (3) - (5)	W_s						
7	Moisture content		w	%	%	%	%	%	%

	Sample No.								
1	Tare No.								
2	Weight in g	Tare and wet soil							
3		Tare and dry soil							
4		Water (2) - (3)	W_w						
5		Tare							
6		Dry soil (3) - (5)	W_s						
7	Moisture content		w	%	%	%	%	%	%

$$w\% = \frac{W_w(4)}{W_s(6)} \times 100$$

Remarks Natural Moisture Content

0723568 Technician C.L.B. Computed T.B.S. Checked W.L.S.

Figure 3-11. Data sheet for moisture content.

c. **Sand-displacement method.**

(1) **Principles.** The sand-displacement method of determining the in-place density may be used in either fine- or coarse-grained materials. This test is so named because a calibrated sand is used to determine the volume of the hole from which a sample has been taken. The test consists essentially of digging out a sample of the material to be tested, determining the volume of the hole from which the sample was removed, and determining the dry weight of the sample. There are three requirements that must be met: (1) the volume of the sample must be 0.05 cubic feet or larger, (2) when the sand-displacement method is used, a double cone cylinder must be used that permits calibrating the sand for each sampling operation, (3) the sand moisture content must be constant while performing the test.

(2) **Equipment.**

(a) **Small digging tools.** The small tools used for digging shown in figure 3-12 are: a spatula, a chisel, a kitchen spoon, and a hammer. Other tools such as ice picks, screwdrivers, or pointed trowels may be used effectively also. These tools are useful for digging samples of either coarse or fine-grained materials. An Iwan type auger, 3 or 4 inches in diameter, may be used in fine-grained soils to obtain the sample.

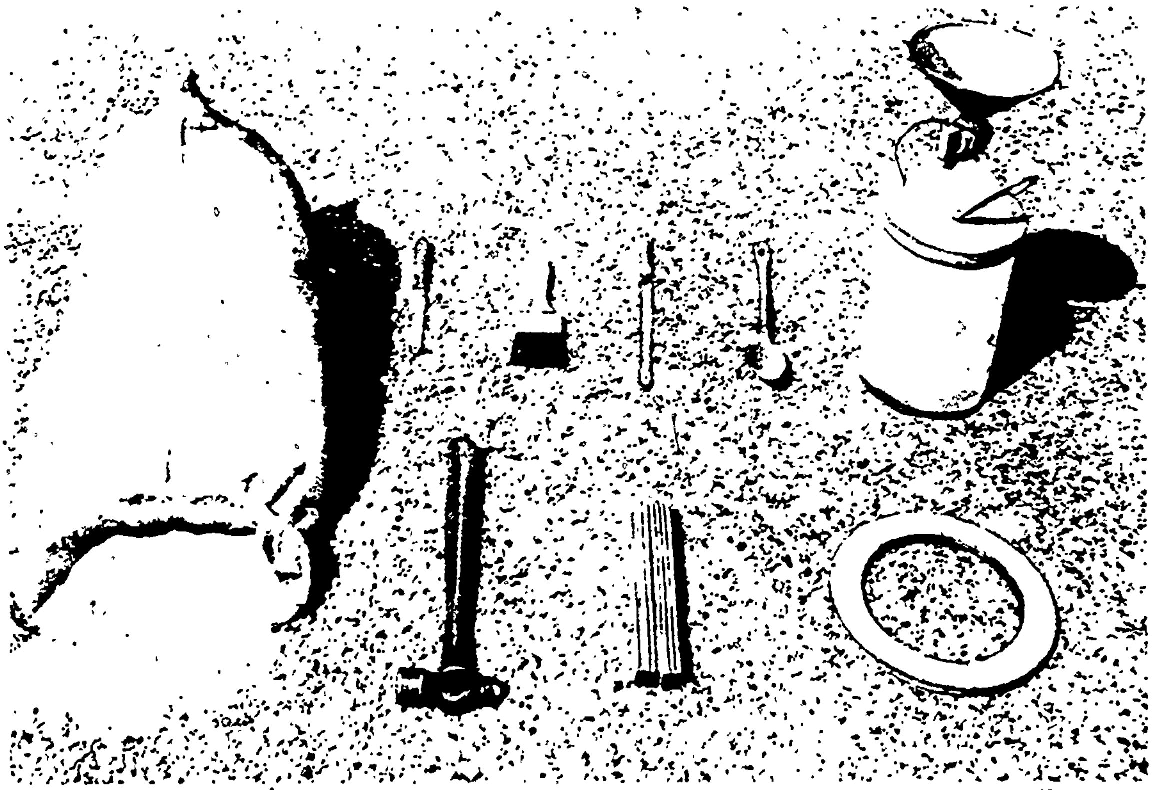


Figure 3-12. Field equipment for sand-displacement method.

(b) **Sample containers.** Almost any type of container with a capacity of about 1 gallon or more, having a close-fitting top or lid, and capable of withstanding the heat of the drying oven is acceptable. One gallon syrup pails are satisfactory.

(c) **Sand.** The sand for use in this test should be composed of clean, airdry, well

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rounded, and hard particles that will pass a No. 20 sieve and be retained on a No. 40. (Some specifications require the sand to be retained on a No. 30 sieve rather than No. 40).

(d) **Sand-density cylinder and seating plate.** The sand-density cylinder developed by the Waterways Experiment Station is shown in figure 3-12 at the upper right and the seating plate used with the cylinder at the lower right. The apparatus is composed of a plexiglas cylinder mounted on a double cone. The cylinder is 10 inches high and 7 inches in diameter. The surfaces of the cones have a slope of 45 degrees and are jointed at the apexes by a ¾-inch valve.

(e) **Modeling clay.** The modeling clay shown in the lower center of figure 3-12 is used to seal underneath the seating plate. Any commercial quality modeling clay is satisfactory for this use.

(f) **Paint brush.** A paint brush about 3 inches wide with moderately long bristles is needed to brush loose material away and help take the sample from the hole.

(g) **Drying oven.** Any oven that can maintain a constant temperature of 105-110° C and has a sufficient sample capacity is satisfactory.

(h) **Balances.** A balance with a capacity of about 20,000 grams sensitive to about 1 gram, and a balance of 500-gram capacity sensitive to 0.1 gram.

(3) **Surface preparation.** The surface of the material to be tested should be prepared by brushing all loose particles away so as to leave a reasonably hard surface. No attempt should be made to level the area with a spatula, trowel, or other tool as this disturbs the surface to be tested. The plate should be seated on the surface and the space under the plate sealed with modeling clay.

(4) **Sand calibration.** The volume of the cylinder and connecting cone up through the valve and the empty weight of the apparatus must be known before the sand can be calibrated. This determination may be made by pouring water into the apparatus until the cylinder, bottom cone, and valve are filled and water stands some distance up in the top cone. Then close the valve, pour off the excess water, dry the cone with a cloth, and weigh. The weight of water in the apparatus in grams is equal to the volume in cubic centimeters. This procedure should be repeated a number of times, the results averaged, and the volume expressed in cubic feet. The airdry sand is calibrated by pouring it into the upper cone with the valve open until the apparatus is filled above the valve. The valve is closed, the excess sand is poured out; and the weight of sand remaining determined. The weight of sand in the apparatus in pounds divided by the volume in cubic feet equals the calibrated density in pounds per cubic foot. The calibration should be repeated for each test as the value will change with temperature and humidity.

Note: Vibration of the sand either by bumping the jar or by heavy equipment shaking the ground during any sand-volume determination may increase the bulk density of the sand and decrease the accuracy of the determination.

(5) **Surface calibration.** The surface irregularities inside the metal seating plate must be taken into account. To do this the volume of the space between the surface of the cone and the surface to be tested is needed. After the sand is calibrated, the apparatus is placed on the metal plate, as shown in figure 3-13. The valve is opened and the space under the cone allowed to fill with sand. When the space is filled, the valve is closed and the weight of sand required to fill the space is determined. It is recommended that as much of the sand be recovered as is feasible without disturbing the seating ring or soil. Then the remaining sand particles should be brushed lightly from within the seating ring, being careful to disturb neither the ring nor the soil to be tested.

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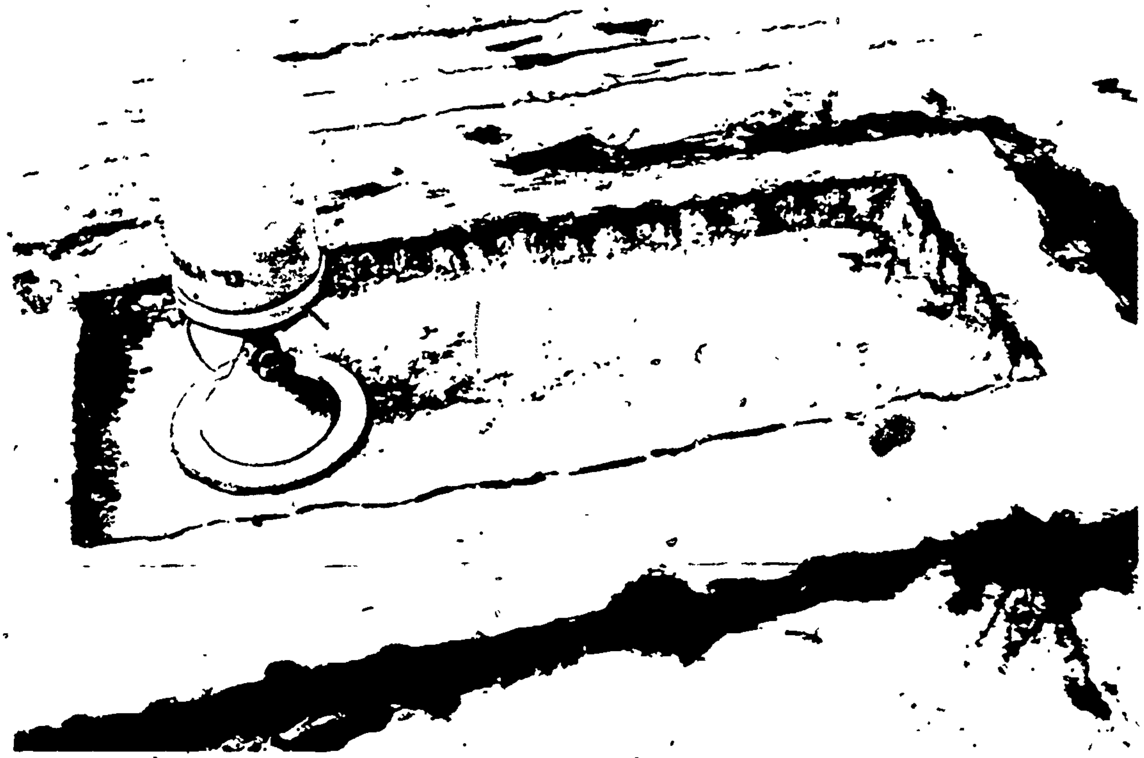

Figure 3-13. Sand density cylinder in test position.

(6) **Digging sample.** All loose particles should be removed and care taken to leave the remaining particles undisturbed. The inside of the hole should be kept as free of pockets and sharp protuberances as possible, since these affect the accuracy of the test. Care should be exercised to remove all loose particles from the hole and to see that all particles removed are included in the sample. All loosened soil should be placed in a container, taking care not to lose any material, the sample weighed immediately, and the wet weight determined.

(7) **Volume and density determination.** After the sample has been removed from the hole, the sand-density cylinder is again placed on the metal plate. The valve is opened and the sand allowed to fill the hole and space under the cone. The weight of the sand required to fill the hole is then equal to the difference between the original and final weights of the apparatus and sand, minus the weight of sand required in the surface calibration. The weight of the sand required to fill the hole divided by the calibrated density of the sand equals the volume of the hole in cubic feet. The dry weight of the sample can be most accurately determined by drying the entire sample in its original container. However, an alternate, less desirable method of determining the moisture content of a small specimen of the density sample and correcting the wet weight of the density sample to give the dry weight may be used. The dry density of the material in place is determined by dividing the dry weight of the sample in pounds by the volume of the hole in cubic feet.

(8) **Volume of sample.** The minimum test hole volumes recommended to use in determining the in-place density of various soil

mixtures are given in table 3-2. This table also shows the minimum weight recommended of the moisture content sample in relation to the maximum particle size in soil mixtures.

TABLE 3-2. *Density of Water and Recommended Minimum Test Hole Volumes*

DENSITY OF WATER

Temperature °C	°F	Volume of water cc per g
12	53.6	1.00048
14	57.2	1.00073
16	60.8	1.00103
18	64.4	1.00138
20	68.0	1.00177
22	71.6	1.00221
24	75.2	1.00268
26	78.8	1.00320
28	82.4	1.00375
30	86.0	1.00435
32	89.6	1.00497

RECOMMENDED MINIMUM TEST HOLE VOLUMES

Maximum particle size in.	Minimum sample volume cu ft	Minimum weight of moisture sample g
0.187 (No. 4 sieve)	0.025	100
0.50	0.050	250
1.00	0.075	500
2.00	0.100	1000

(9) **Calculation.** The volume in cubic centimeters of the density apparatus is equal to the weight of water in grams required to fill the apparatus, multiplied by the water temperature (volume correction shown in table 3-2). The necessary calculations and suggested form for use in recording the data and making the calculations are shown in figure 3-14.

Note: The factor 0.002205 shown in line 5 of figure 3-14 is 1 gram expressed in pounds as follows: 1/454 0.002205.

3-6. TRAFFICABILITY

a. **Principles.** Trafficability is the capacity of a soil to support a vehicle without excessive settlement of the vehicle.

b. **Cone penetrometer.** The cone penetrometer (fig 3-15) is the principal instrument used in evaluating soils trafficability. It consists of a 30-degree cone of ½-square-inch base area, an aluminum staff 19 inches long and ⅜-inch in diameter, a proving ring, a micrometer dial, and a handle. When the cone is forced into the ground, the proving ring is deformed in proportion to the force applied. The amount of force required to move the cone slowly through a given plane is indicated on the dial inside the ring. This force is considered to be an index of the shearing resistance of the soil and is called the cone index of the soil in that plane. The range of the dial is 0 to 300 (150 pounds). The proving ring and handle are used with a ⅜-inch diameter, 19-inch long steel staff and 0.2 square inch cone, for remolding tests in sands and fines, poorly drained.

c. **Huorsleu soil sampler.** A piston type soil sampler (fig 3-16) is used to extract soil samples for remolding tests.

d. **Remolding cylinder and hammer.** The equipment for the remolding test, shown in figure 3-17, consists of a steel cylinder approximately 2 inches in diameter and 8 inches long mounted on an aluminum base, a 2½-pound steel drop hammer sliding on an 18-inch steel staff with handle, and a cone penetrometer. The penetrometer is used to measure soil strength in the cylinder before and after remolding. The sampler is used to obtain the soil sample and place it in the remolding cylinder.

e. **Remolding test.**

(1) **Principles.** Since remolding test techniques for fine-grained soils differ somewhat from those for sands with fines, poorly drained, the operator should be able to recognize the two types of soil for conditions where remolding tests must be made. In such conditions both soil types are wet in appearance and wet to the touch. If squeezed

IN-PLACE DENSITY TEST

Sand Density Cylinder

AIRFIELD	TEST LOCATION *Pit 1*
MATERIAL *Clay gravel Base course*	DEPTH, IN. *3* DATE *17 March 1952*

Sand Calibration	Ground Surface Calibration
1. Weight Filled *12,854* g	6. Weight Before *12,854* g
2. Weight Empty *4,294* g	7. Weight After *10,973* g
3. Weight Sand (1-2) *8,560* g	8. Weight Sand (6-7) *1,881* g
4. Volume Container *0.2024* cu ft	
5. Calibrated Dens. $\left(\frac{3 \times 0.002205}{4}\right)$ *93.2* lb/cu ft	

Hole Volume Determination

9. Weight Before (From 7) *10,973* g
10. Weight After *7,163* g
11. Weight Sand (9-10) *3,810* g
12. Weight Sand Used (11-8) *1,929* g

Density Determination

13. Wet Weight Sample + Container *3,080* g
14. Dry Weight Sample + Container *2,916* g
15. Weight Container *273* g
16. Weight water (13-14) *164* g
17. Wet Weight Sample (13-15) *2,807* g
18. Dry Weight Sample (14-15) *2,643* g
19. Water Content $\left(\frac{16}{18}\right)$ *6.2* %
20. Wet Density $\left(\frac{17 \times 5}{12}\right)$ *135.8* lb/cu ft
21. Dry Density $\left(\frac{18 \times 5}{12}\right)$ *127.8* lb/cu ft

REMARKS: *Material appeared well compacted. Density value may be high due to slight disturbance to walls of hole.*

COMPUTED	CHECKED
ADS	*BET*

Figure 3-14. Data form for sand-displacement method.

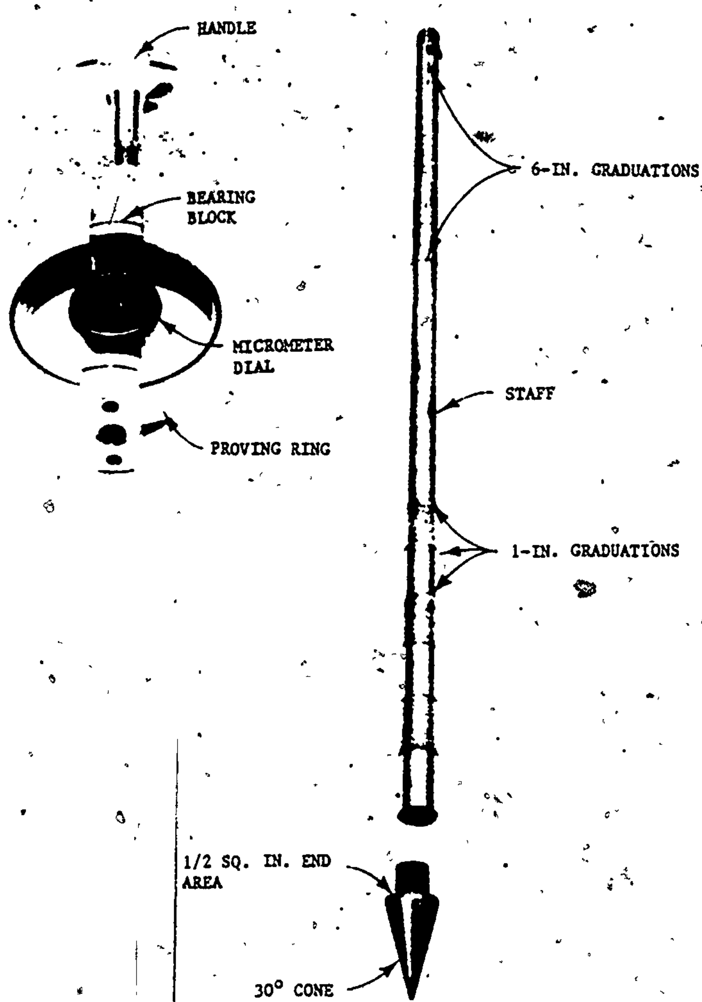

Figure 3-15. Cone penetrometer.

and rolled between the fingertips the fine-grained plastic soil will feel soft and smooth because such soil particles are small and flat in shape. The other soil type will have a definite abrasive feel because of the presence of the larger, rounder particles of sand. However, there will be many cases in which the operator cannot confidently distinguish the two types. In such cases, both types of remolding tests should be made and the remolding indexes obtained compared. If the lower remolding index is the one obtained with the remolding test for sands with fines, poorly drained, it may be assumed that the soil is a sand with fines, poorly drained, and the test for this soil type should be employed throughout the area under investigation. It is emphasized that a good rule to follow in all cases of doubt is to run both types of tests and use the lower remolding index.

(2) **Test procedure for fine-grained soils.** Take a sample with the sampler, eject it directly into the remolding cylinder, and push it to the bottom of the cylinder with the foot of the drop hammer staff. Measure the strength with the penetrometer (aluminum staff) by taking cone index readings as the base of the cone enters the surface of the soil sample and at each successive inch, to a depth of 4 inches. Next, apply 100 blows with the drop hammer falling 12 inches and measure the remolded strength at each 1-inch depth to 4 inches, as was done before remolding. Occasionally a sample is so hard that it cannot be penetrated the full 4 inches,

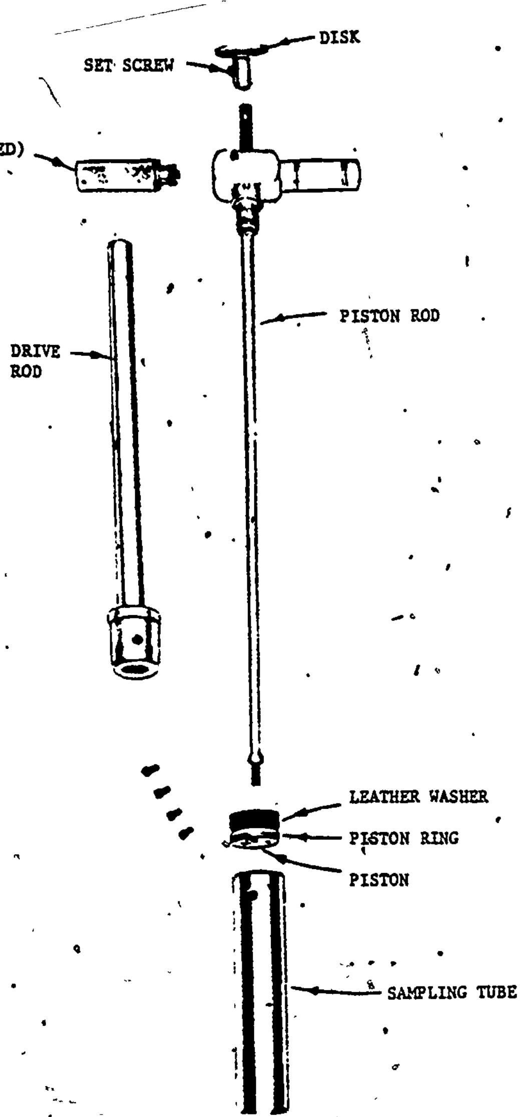

Figure 3-16. Soil sampler.

In such cases the full capacity of the dial (300) on the penetrometer is recorded for each inch below the last reading obtained. The sum of the five cone index readings after remolding divided by the sum of the five cone index readings before remolding gives the remolding index.

(3) **Test procedure for sands with fines, poorly drained.** The procedure is generally the same as that for fine-grained soils except that the cone index measurements are made with the slender staff and small cone, and the sample is remolded by dropping it (along with cylinder and base) 25 times from a height of 6 inches onto a firm surface.

f. **Rating cone index.** The product of the cone index times the remolding index gives the rating cone index for that soil. This value then represents the capacity of that soil to carry traffic under repeated loadings.

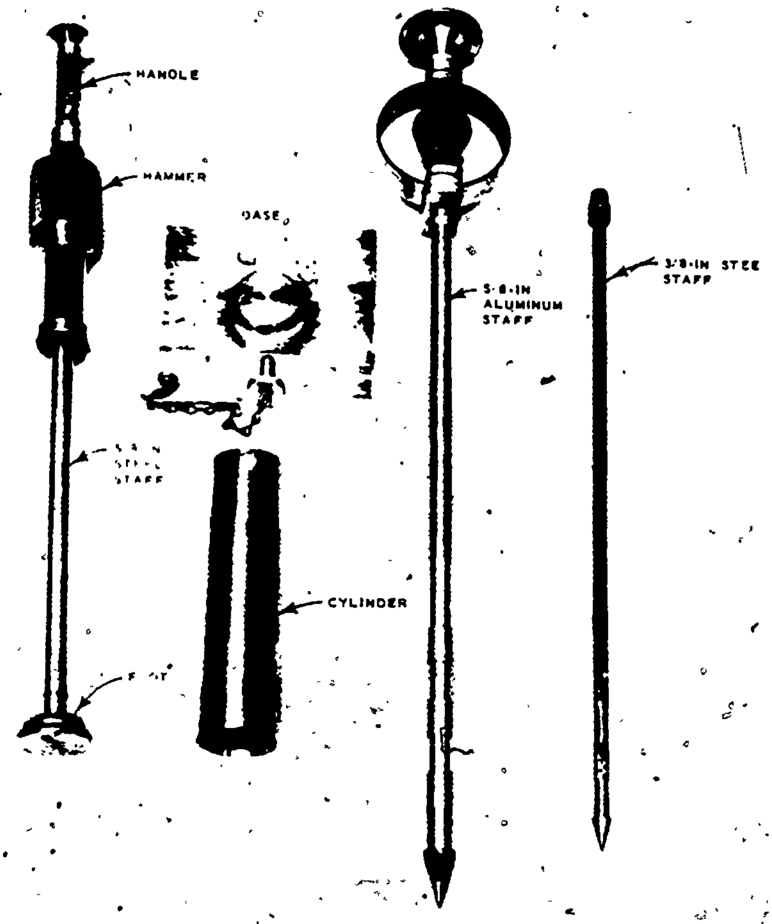

Figure 3-17. Remolding test equipment.

Its value (RCI) must then be compared to the vehicle cone index, or in other words strength of soil must be compared with the strength required to pass a given vehicle. If the soil strength is greater than the strength required by the vehicle, then this type of vehicle may pass over this soil.

g. **Critical layer.** The critical layer is the soil layer in which the rating cone index is considered a significant measure of trafficability, or the layer of soil which is regarded as being most pertinent to establishing relationship between soil strength and vehicle performance. Its depth varies with the weight and type of vehicle and the soil profile, but it is normally the layer lying 6 to 12 inches below the surface.

SELF TEST

Note: The following exercises comprise a self test. The figures following each question refer to a paragraph containing information related to the question. Write your answer in the space below the question. When you have finished answering all the questions for this lesson, compare your answers with those given for this lesson in the back of this booklet. Do not send in your solutions to these review exercises.

1. Compaction is the process of artificially densifying a soil. In the process of densifying a soil, what is normally removed from it? (3-1)

2. Why is it necessary to determine the optimum moisture content for a soil that must to compacted to support a heavy load? (3-2a)

3. The maximum dry density of a soil may be attained by the correct combination of compaction effort and moisture content. What is the normal relationship between compaction effort and the optimum moisture content? (3-3a(1))

4. Within a range of twenty pounds, what is the compacted unit dry weight of well-graded sands or gravelly sands (SW) in pounds per cubic foot? (3-3a(2), chart II)

5. The compaction control test used by the Corps of Engineers is called the "Modified AASHO Test." What information is provided by this test? (3-3b(1))

6. The "Modified AASHO Test" may be run using either a Proctor mold or a CBR mold. How would you determine which mold to use? (3-3b(2)(a), (b))

7. When using the Proctor mold, describe the procedure used in compacting the soil sample. (3-3b(4)(a))

8. Compaction procedure for using the CBR mold is the same as for the Proctor mold except for one major difference. What is this difference? (3-3b(4)(b))

9. What is the dry unit weight in pounds per cubic foot, of a soil having a wet unit weight of 142.6 and a moisture content of 10.3? (3-3b(5)(c))

10. If the maximum modified AASHO dry density of a particular soil is 110 pounds per cubic foot at the OMC of 18 percent, what is the lowest acceptable compactive effort, or density, you could accept for a subgrade for a road in a TO if the material was cohesive? (3-3c(4), fig 3-9)

11. In a road construction project you have a requirement to mix some broken up bituminous pavement with the in-place clay. What piece of equipment would you select for this project? (3-4b)

12. An area proposed for use as open storage has large chunks of earth and excessive surface moisture. What item of construction material is best suited for breaking up these chunks and loosening the surface so the water can penetrate? (3-4c)

13. What type of compaction equipment is most likely to produce the desired compaction in GP soils (poorly graded gravels or gravel-sand mixtures, little or no fines)? (3-4o, chart II)

14. Before determining the in-place moisture content, it is necessary to understand what water content is. What is water content as regards moisture in soil? (3-5b(1))

15. What is the moisture content, in percent, in a soil if the wet soil weight is 123.1 grams and the dry soil weight is 108.2 grams? (3-5b(4))

16. When determining the in-place density of a soil by the sand-displacement method, what purpose is served by the calibrated sand? (3-5c(1))

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17. The sand used in the sand-displacement method should be composed of clean, air-dried, well rounded, and hard particles that will pass a number 20 sieve and be retained on a number 40 sieve. What does it mean to say that this sand must be calibrated? (3-5c(4))

18. How is the volume of the hole, or in other words, the volume of the soil sample, then finally determined by this sand-displacement method? (3-5c(5), (7))

19. What is the function of the proving ring on the cone penetrometer? (3-6b).

20. What mathematical process must be performed to determine the remolding index of a soil after the required number of penetrometer readings have been made? (3-6e(2))

LESSON 4

STRENGTH DESIGN USING CBR

CREDIT HOURS ---------------------------3

TEXT ASSIGNMENT ---------------------------Attached memorandum.

MATERIAL REQUIRED ---------------------------Table 4-3, Aircraft Data Sheet — Air Force Aircraft.

LESSON OBJECTIVE ---------------------------Upon completion of this lesson you should be able to accomplish the following in the indicated topic areas:

1. **Introduction.** Explain the development and use of the California Bearing Ratio (CBR) method for design of flexible pavements.

2. **Laboratory Tests.** Describe the equipment used in a laboratory CBR test, the preparation of soil samples, the performance of the test itself, and the proper recording of the test data.

3. **Field In-Place Test.** Explain the circumstances under which the CBR test is run on undisturbed samples in the field, the procedure, and the recording of the data obtained.

4. **Design CBR and Soil CBR.** Determine the design CBR for the anticipated traffic, utilizing available tables and graphs, and utilizing the data obtained from the laboratory and field CBR tests to determine the moisture content and compactive effort required.

ATTACHED MEMORANDUM

4-1. DEFINITION

The California Bearing Ratio (CBR) method of design for flexible pavements started as an essentially empirical method. Developments and thorough testing have led to the establishment of a distinct pattern for design relations. From this pattern, standard procedures have been evolved for constructing any desired set of CBR curves. The CBR test is essentially a penetration test having the function of measuring the soil resistance to penetration prior to reaching its ultimate shearing value. It is a duplication of the action which wheels exert on a flexible pavement (not a shearing modulus because of the confining effect of the molds). Hence it is defined as the ratio in percentage from 0 to 100, with a standard well graded crushed limestone serving as the 100 percent material.

4-2. PERFORMANCE OF LABORATORY CBR TEST

Experience has shown that minor variations in the CBR test procedures will cause wide variations in test results. For this reason, step-by-step procedures are detailed. Even with these step-by-step procedures, it is realized that difficulties will arise. For materials containing gravel or stones, the procedures have not proven entirely satisfactory, and it will be necessary to conduct a number of tests to assure a reasonable average value. In some cases where gravel or stone is present in such small quantities that it does not affect the stability of the soil, the particles can be removed and inconsistencies in test results will be avoided. For the majority of soils, however, the methods presented have proven satisfactory. In the following paragraphs, procedures and suggested equipment are presented for tests on remolded and compacted samples and undisturbed samples.

4-3. EQUIPMENT FOR PERFORMANCE OF LABORATORY CBR TEST

a. A mold 6 inches in diameter and 7 inches high with a detachable collar and with a perforated base plate shall be used. Perforations in base plate must not be greater than 1/16-inch in diameter. The base plate and collar should be made to clamp on either end of the cylinder. For any group of molds, one extra base plate is desirable as two plates are required when a mold is inverted during the preparation of the test specimen.

b. A spacer disk 5 15/16 inches in diameter and 2 or 2½ inches high is used.

c. A compaction tamper identical with that previously described in Lesson 3 is used.

d. Adjustable stem and perforated plate, tripod, and dial gage (reading to 0.001-inch) are needed, suitable to measure the expansion of the soil. Perforations in the plate should not be greater than 1/16-inch in diameter.

e. One annular weight 5⅞ inches in diameter with center hole 2⅛ inches in diameter is needed. Also needed are several slotted weights weighing 5 pounds each, suitable to apply as surcharge loads on the soil surface during soaking and penetration.

f. A penetration piston having an end area of 3 square inches (1.95 inches in diameter) is used. It must be sufficiently long to pass through the surcharge weights and penetrate the soil (7½ inches long if for use in a large laboratory compression machine).

g. A laboratory testing machine, or screw jack and frame arrangement, is needed, either of which can be used to force the penetration piston into the specimen at a rate of 0.05-inch per minute.

h. Also needed is other general laboratory equipment such as mixing bowls, spatulas, straightedges, scales, soaking tank or buckets, ovens etc.

i. Figure 4-1 illustrates a typical set of equipment needed to prepare and test a soil sample in the laboratory.

4-4. PREPARATION OF REMOLDED SAMPLES

a. **Principles.** The procedure is such that the CBR values are obtained on test specimens which have the same density and moisture content expected in the field. In general, for most materials, the most critical condition in the prototype will exist when the maximum amount of water has been absorbed. For this reason, the CBR is usually made after the specimens have been immersed in water for a 4-day soaking period while contained in molds and confined with a surcharge equal to the weight of the pavement and base that will be above the material. The following procedure has been formulated as a result of studies and should generally be followed.

b. **Size of material.** The soil sample shall first be dried until it becomes friable under a trowel. Drying may be in the air or by use of drying apparatus such that the temperature of the sample does not exceed 140°F. The aggregations shall then be thoroughly broken up in such a manner as to avoid reducing the natural size of the individual particles. An adequate quantity of representative pulverized soil will be screened first over the ¾-inch sieve and then on the No. 4 sieve. Coarse aggregates retained on the ¾-inch sieve will be discarded and replaced by an additional equal portion of the original material passing the ¾-inch sieve and retained on the No. 4 sieve or as explained in lesson 3. The sample is then recombined and thoroughly mixed.

c. **Compacting samples.** Samples shall be prepared and compacted for the CBR test

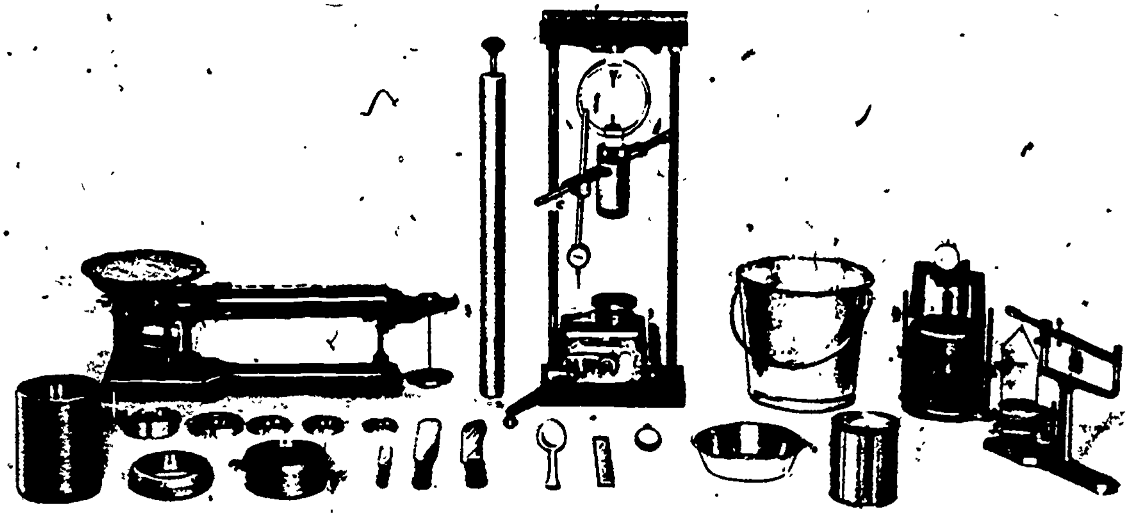

Figure 4-1. Soil test apparatus, laboratory CBR test.

using the same procedures as outlined for compaction control tests (see lesson 3), using the compactive efforts and the molding water content as recommended. Molding moisture content may be obtained by drying a portion of the sample (100 grams for clays or 500 grams for gravelly soils) at the time the specimen is compacted. After compacting the sample, the collar is removed, the specimen is trimmed, a fine wire mesh or coarse filter paper is placed over the top of the specimen, and a perforated base plate is clamped to the top of the mold.

d. **Soaking.** Place the perforated plate with the adjustable stem on the surface of the mold and apply an annular weight to produce an intensity of loading equal to the weight of the base material and pavement within plus or minus 5 pounds, **except that in no case shall the weight be less than 10 pounds.** Immerse mold and weights in water to allow free access of water to top and bottom of the specimen. Take initial measurements for swell and allow to soak for 4 days. A shorter immersion period is permissible if tests show that shorter period does not affect the results for soils that take up moisture readily. Take final swell measurements and compute the swell in percentage of initial specimen height.

e. **Draining.** Remove free surface water and allow the specimen to drain downward for 15 minutes. Care should be taken not to disturb the surface of the specimen during removal of the free water; it may be necessary to tilt the samples. The perforated plate and surcharge weights are removed and the specimen weighed. The specimen is then ready for the penetration test.

4-5. PENETRATION TEST

a. **Principles.** Since the actual penetration test procedure is constant for all types of specimens, it is presented here and will not be repeated for other types. The procedure outlined in the following subparagraphs is also applicable for undisturbed and field in-place tests after the testing surface has been prepared. A suggested form for use in recording the test data is shown in figure 4-2.

b. **Surcharge.** Apply a penetration surcharge on all soils sufficient to produce an intensity of loading equal to the weight of the base material and pavement (within ± 5 pounds), but not less than 10 pounds. If the sample has been soaked previously, the penetration surcharge should be equal to the soaking surcharge. To prevent upheavel of soil into the hole of the surcharge weights, it is advisable to place one 5-pound annular

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CALIFORNIA BEARING RATIO (CBR) TEST
Penetration

PROJECT	DATE
SAMPLE NO.	JOB NO.

NO. BLOWS	AS MOLDED	SOAKED

MOLD NO.	Surcharge weights, in pounds: Soaking________ Penetration________

Date	Time	Days	Reading in.	Swell in.	Swell %	BEARING RATIO DATA (3-sq-in. piston. 0.05 in. per min.)			
		0	0.	0.		Penetration in.	Total Load	Bearing Value* $lb/in.^2$	Corr. CBR
			0.	0.					
			0.	0.		0.025			
			0.	0.		0.050			
			0.	0.		0.075			
			0.	0.		0.100**			
			0.	0.		0.125			

*WATER CONTENTS AFTER SOAKING	Whole Specimen Drained	Top in.	Penetration in.	Total Load	Bearing Value* $lb/in.^2$	Corr. CBR
			0.150			
			0.175			
Tare			0.200†			
Tare + wet soil			0.250			
Tare + dry soil			0.300			
Water			0.350			
Tare			0.400			
Solids			0.450			
Water Content	%	%	0.500			

Note: Standard load at 0.1-in. penetration = 1000 psi.
Standard load at 0.2-in. penetration = 1500 psi.
* (0.734 × total load in kilograms) or (total load in pounds ÷ 3).
** Total load in pounds ÷ 30 = CBR.
† Total load in pounds ÷ 45 = CBR.

TECHNICIAN	CHECKED	DATE

Figure 4-2. Data form for CBR test.

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disk surcharge weight on the soil surface prior to seating the piston and finally applying the remander of the weights.

c. **Seating piston.** Seat the penetration piston with a 1 pound load (or until reasonable pressure) and set both the stress and strain gages at zero. This initial load is required to insure satisfactory seating of the piston and should be considered as the zero load when determining stress-penetration relations.

d. **Application of load.** Apply load uniformly onto the penetration piston through the screw or hydraulic jack so the progressive rate of application is approximately 0.05 inch per minute. Obtain load readings at 0.025, 0.05, 0.075, 0.1, 0.125, 0.150, 0.175, 0.200, 0.250, 0.300, 0.350, 0.400, 0.450 and 0.500 inch depths of penetration. In manually operated loading devices it may be necessary to take load readings at closer intervals to control the rate of penetration.

e. **Moisture.** Determine the moisture content in the upper 1 inch and, in the case of laboratory tests, average moisture content for the entire depth of the sample.

f. **Stress-penetration curve.** The penetration load, in pounds per square inch, is computed and the stress-penetration curve is drawn. To obtain true penetration loads from the test data, the zero point of the curve is adjusted to correct for surface irregularities or disturbances and the initial concave-upward shape of the curve, if present. 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. In figure 4-3, necessary corrections have been made to two of the curves shown. Corrected unit loads are entered in the appropriate places on DD Form 1212.

g. **California bearing ratio.** Determine the corrected load values in psi at 0.1- and 0.2-inch of penetrations. From these the CBR values are obtained by dividing the loads at 0.1- and 0.2-inch by the standard loads of 1,000 and 1,500 pounds per square inch, respectively. The standard loads of 1,000 and 1,500 psi are always constant and represent the load required to penetrate a well graded minus ¾-inch crushed limestone sample at modified AASHO maximum density. Multiply each ratio by 100 to obtain the ratio in percent. The CBR is usually selected at 0.1-inch penetration. If the CBR at 0.2-inch penetration is greater, the test should be rerun. If check tests give similar results, the CBR at 0.2 inch should be used.

h. **Test results.** Test results should contain the following information: compaction procedure (modified AASHO); compaction effort (12, 26, and 55 blows/layer); molding water content (moisture content prior to soaking); density (prior to soaking); soaking and penetration surcharges; expansion of sample (% swell); moisture content after soaking (saturated condition); density after soaking (saturated condition); optimum moisture content and maximum dry density determined by modified AASHO compaction test (55 blows/layer); plot and correction (if necessary) of stress-penetration curve; calculations and selection of a corrected CBR value.

4-6. TEST PROCEDURE FOR REMOLDED SAMPLES

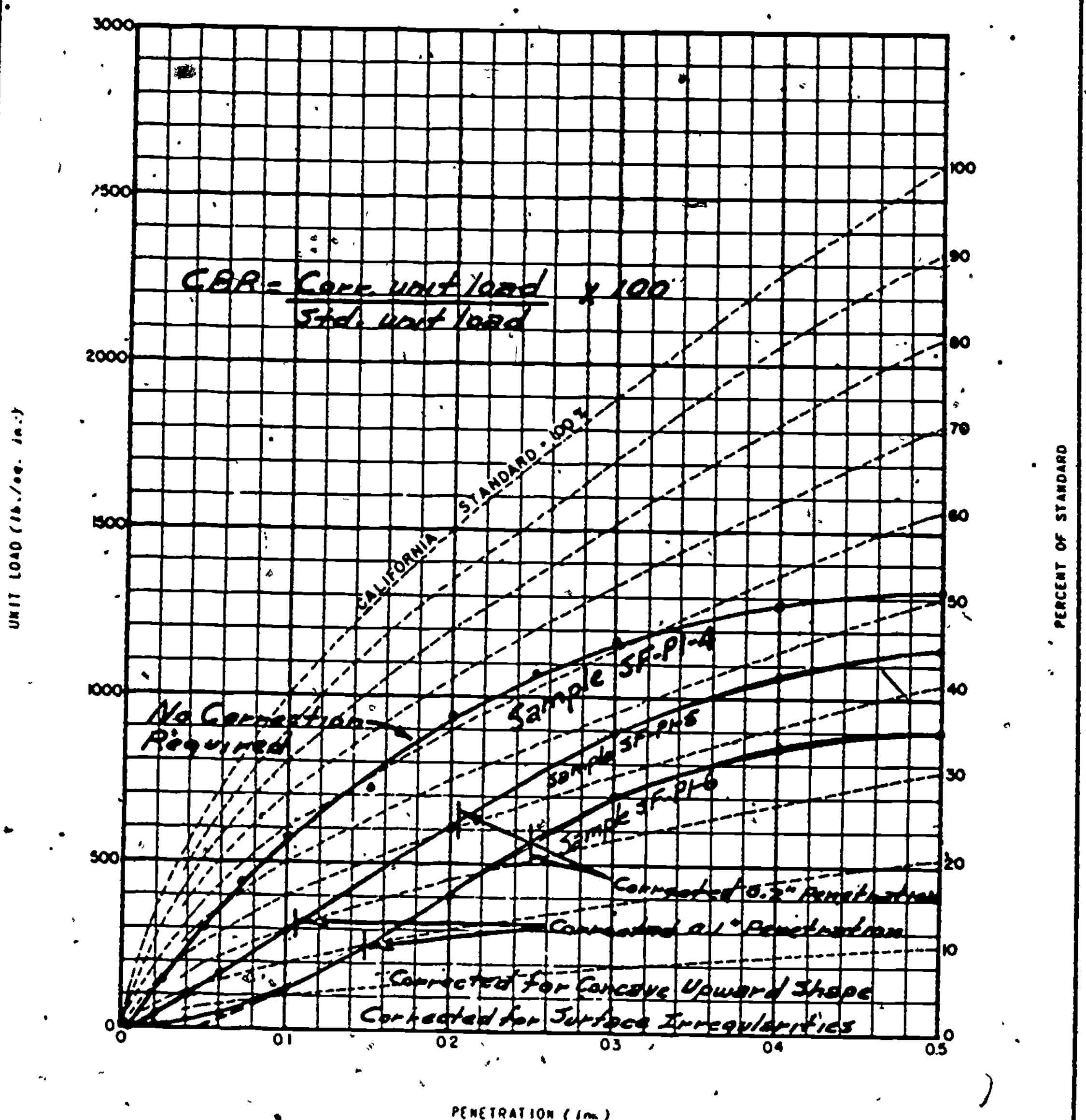
a. **Principles.** In testing remolded specimens for the California method of design, all subgrades and base courses have been grouped into three classes with respect to behavior during the saturation: (a) cohesionless sands and gravels, (b) cohesive soils, and (c) highly swelling soils. The first group usually includes the GW, GP, SW, and SP classifications. The second group is usually in the GM, GC, SM, SC, ML, CL, and OL classifications. Swelling soils usually comprise the MH, CH, and OH classifications. Separate procedures are given for each of these groups below.

b. **Cohesionless sands and gravels.** Cohesionless soils usually compact readily under rollers or traffic and specimens should be prepared at high densities and at a range of water content covering those anticipated in the field, including water contents as high

SWELL DATA					
DATE	TIME	DAYS	GAGE READING	SWELL (in.)	SWELL (%)
23/4		0	0.304		
24/4		1	0.316	0.012	
25/4		2	0.328	0.024	
26/4		3	0.337	0.033	
27/4		4	0.349	0.045	1.0
		5			

CALIFORNIA BEARING RATIO TEST GRAPH

(Plot test curve below to obtain corrected unit load)



TECHNICIAN (Signature)	COMPUTED BY (Signature)	CHECKED BY (Signature)
John Case	John Case	Charles Layne

Figure 4-3. Stress penetration curves.

as practicable. If soaking does not lower the CBR, it may be omitted for further tests on the same material and the material should be placed in an almost saturated condition during the construction phase.

c. **Cohesive soils.** Soils in this group are tested in a manner to develop data that will show their behavior over the entire range of anticipated moisture contents for representative samples. Compaction curves are developed for 55, 26, and 12 blows per layer and each specimen is soaked and penetrated to develop a complete family of curves showing the relationship between density, molding, water content, and corrected CBR.

d. **Swelling soils.** The test procedures for highly swelling soils are the same as previously described for cohesive soils; however, the objectives of the testing program are not exactly the same as for cohesive soils. Tests are performed on soils with expansive characteristics to determine a moisture content and a unit weight which will minimize expansion. The proper moisture content and unit weight are not necessarily the optimum moisture content and unit weight determined by modified AASHO compaction tests. Generally, the minimum swell and highest soaked CBR will occur at a molding moisture content slighly wet of optimum. It may be necessary, when testing highly swelling soils, to prepare samples for a wider range of moistures and densities than normally used in order to establish the relationship between moisture content, density, swell, and CBR for a given soil. A careful study of the test results by an experienced engineer will permit the selection of the proper moisture content and unit weight required in the field. It should be noted that the possibility exists that thickness design may be governed by compaction requirements rather than CBR in some cases.

4-7. PROCEDURE FOR UNDISTURBED SAMPLES

Tests on undisturbed samples will be used for design where the uncompacted condition governs, such as over a highly compressible clay subgrade where normally this type of soil will lose strength upon remolding, and for correlating field in-place tests to the design moisture condition. For the latter condition duplicate samples should be tested to determine the correction necessary for the in-place tests. In this instance, the reduction in CBR that occurs from 4 days of soaking should be applied as a correction to the field in-place test. Considerable care and patience are necessary if disturbance to the relatively undisturbed specimen is to be held to a minimum. Satisfactory undisturbed samples may be obtained by use of steel cylinders, expansible galvanized metal jackets, or box samples. If proper lateral support is not afforded on the sides of the sample, erroneous CBR values will result. Molds and metal jackets are satisfactory for use in fine-grained materials. The annular space between the sample (cut or trimmed from a pedestal) may be filled with paraffin and 10 percent resin to offer support. For gravelly soils the box method is desirable. The sample is covered with wax paper or paraffin to prevent moisture loss during transportation to the laboratory. The soaking and penetration tests are performed as previously outlined after the removal of paper or paraffin from the end of the specimen in the case of molds or metal jackets, or after the surface of the box samples is leveled, with a thin layer of sand if necessary.

4-8. FIELD IN-PLACE TEST

a. **Principles.** The field in-place test is used primarily to check the CBR of subgrade and base courses during construction. Under certain conditions, it is a satisfactory test for determining the load-carrying capacity of a soil. Basically, the penetration phase of the test is the same as described previously.

b. **Conditions for use.** The field in-place test is used under any one of the following conditions:

(1) 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.

(2) When the material is coarse grained and cohesionless so that the changes in moisture content will not greatly affect it.

(3) 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 field in-place test is considered a satisfactory indicator of the load-carrying capacity. The time required for the moisture content to stabilize cannot be stated definitely, but the minimum time is approximately 3 years.

c. **Apparatus.**

(1) **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 soil test set. This in-place test is performed in conjunction with a loaded truck to provide resistance for the CBR screw jack.

(2) **Beam kit, field CBR.** This kit includes a loading beam, a penetration beam, and a beam clamp placed in a beam and weight chest as part of the soil test set.

(3) **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 soil test set.

d. **Preparation.**

(1) **Principles.** 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.

(2) The truck is jacked up so that no weight rests on the springs. A short section of 6-inch I-beam to bridge the bumpers and a clamp for securing the jack to the beam are provided.

(3) The ground surface is leveled and prepared by removing loose material. The apparatus is assembled and positioned under the bumper (fig 4-4). The swivel head must be adjusted so that the penetration piston is plumb, and the device 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 as not to be 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 extend beyond 1 foot with sufficient magnitude to be of any consequence.

(4) The steel plate (as an initial 10-pound surcharge weight) should be seated in a layer of fine sand to distribute the surcharge reaction 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 4-1).

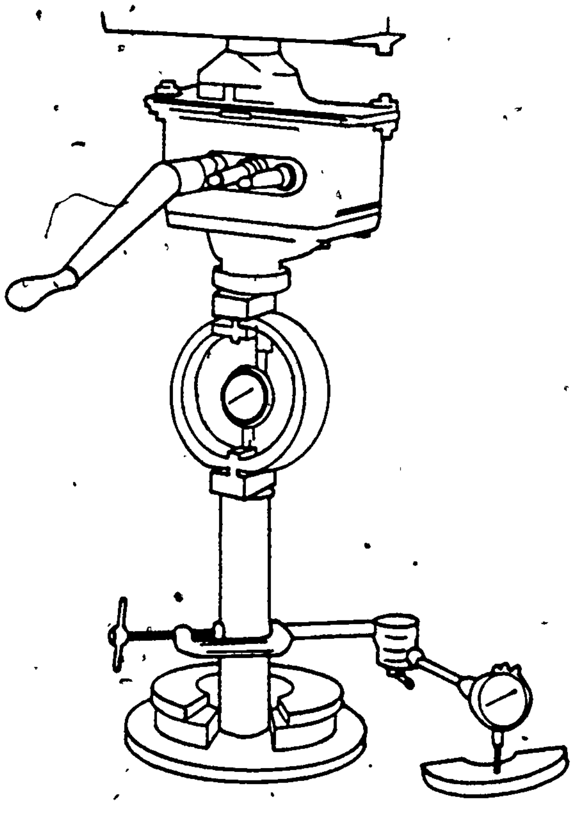


Figure 4-4. Assembled apparatus, field in-place CBR test.

e. **Procedure.**

(1) **Principles.** With the penetration piston and the surcharge weights added, and the load and penetration dials set to zero, the test can be performed.

TABLE 4-1. *Surcharge Weights, Depth Versus Density*

Depth in.	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	114	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		

(Column headings 80–150: Wet Density, lb/cu ft)

* 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 lb has been computed.

(2) Jack the penetration piston at a rate of 0.5-inch per minute.

(3) Read the proving ring (load) readings at the following depths of penetration: 0.025, 0.050, 0.075, 0.100, 0.125, 0.150, 0.175, 0.200, 0.250, 0.300, 0.350, 0.400, 0.450 and 0.500 inch.

(4) Using the proving ring calibration, determine the bearing value (in pounds per square inch).

(5) Plot the penetration versus load deformation curve, with corrections if necessary.

(6) Compute the corrected CBR (in percent).

(7) Obtain a sample of the soil at the point of penetration for a moisture content determination.

(8) Repeat the procedure in (1) through (6) above, two or more times (for a total of three tests) in the same type of soil.

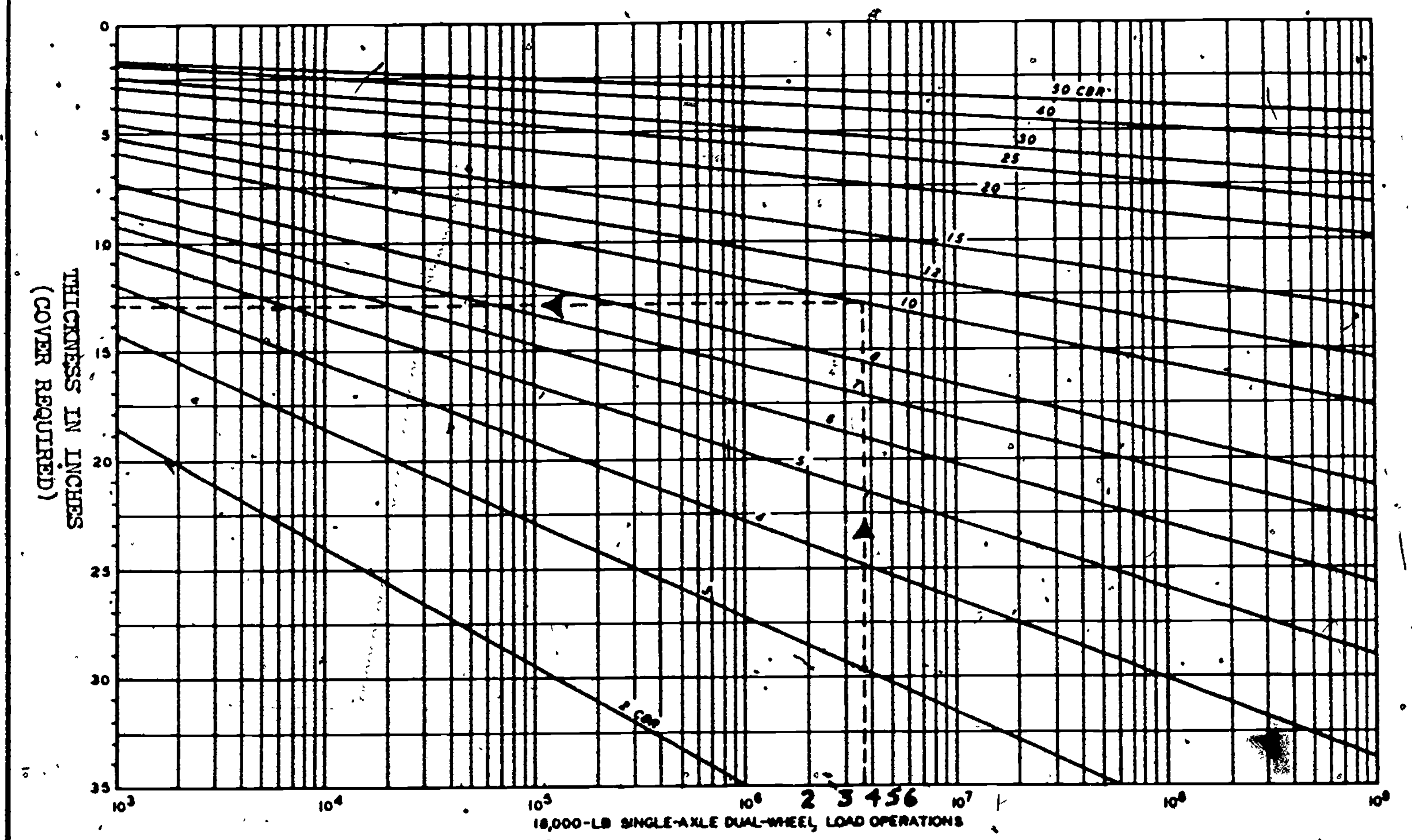
(9) If the results do not show reasonable agreement, three additional tests should be made. The numerical average of the six tests is used as the CBR at that location. Reasonable agreement is designated as the following:

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; 23, 18, and 20 are reasonable and average 20. Values below 20 are rounded off to the nearest unit, and above 20 to the nearest 5 units.

4-9. COVER REQUIREMENTS

a. **Roads.** The cover requirements for roads are computed using figure 4-5. To use figure 4-5, enter the 18,000-pound single-axle dual-wheel load operations on the bottom of the chart, move straight up to the correct



CBR DESIGN CURVES FOR 18,000-LB SINGLE-AXLE DUAL-WHEEL LOAD
THICKNESS VS OPERATIONS

Figure 4-5. Flexible pavement design curves for roads, thickness versus operations for 18,000-pound, single-axle, dual-wheel load.

CBR line and across to the cover required in inches. These cover requirements must be used in conjunction with table 4-2 which indicates the minimum thickness of pavement and base courses.

b. **Airfields.** The cover requirements for airfields are computed using a series of curves. Table 4-3, which is bound in the back of this booklet, contains a list of USAF aircraft with tire and weight data and recommended pavement and base thickness. Figure 4-6 is a typical curve for computing the cover requirements for airfields. To use these curves, make sure the right curve for the applicable aircraft is used. Enter the applicable chart on the left at the correct assembly load in kips obtained from table 4-3. Note that when entering the chart from the left, the type of traffic area must be known. Go across to the correct CBR curve and down to the cover requirement in inches.

TABLE 4-2. *Recommended Minimum Thickness of Pavement and Base*

Line No.	Equivalent 18,000 lb single-axle, dual-wheel load operations	100 CBR BASE		80 CBR BASE		50 CBR BASE	
		Pavement in	Base in	Pavement in	Base in	Pavement in	Base in
1	$3x10^3$ or less	ST[a]	4	MST[b]	4	1½	4
2	$3x10^3 - 1.5x10^4$	ST	4	MST	4	2	4
3	$1.5x10^4 - 7x10^4$	MST	4	1½	4	2½	4
4	$7x10^4 - 7x10^5$	MST	4	1½	4	3	4
5	$7x10^5 - 7x10^6$	1½	4	2	4	3½	4
6	$7x10^6 - 7x10^7$	1½	4	2½	4	4	4
7	$7x10^7 - 7x10^8$	2	4	3½	4	4½	4
8	$7x10^8 - 7x10^9$	3	4	3½	4	5	4

[a] Single bituminous surface treatment

[b] Multiple bituminous surface treatment

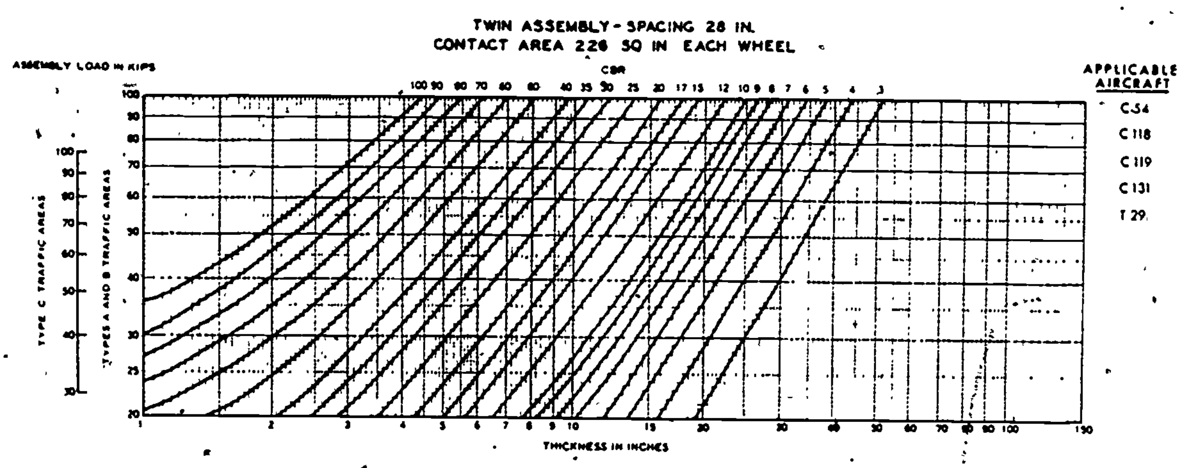


Figure 4-6. Typical airfield flexible pavement design and evaluation curves, tricycle type landing gear, twin, tandem assemblies, full operational.

4-10. DETERMINATION OF CBR

a. **Principles.** The data from the CBR tests and the resulting family of CBR curves represent characteristics for a wide range of field conditions. The CBR data, the molding water content, and the dry density can be combined to present the test results in a highly usable form. This combination is an extension of the compaction curve data and is illustrated in A, B, and C, figure 4-7.

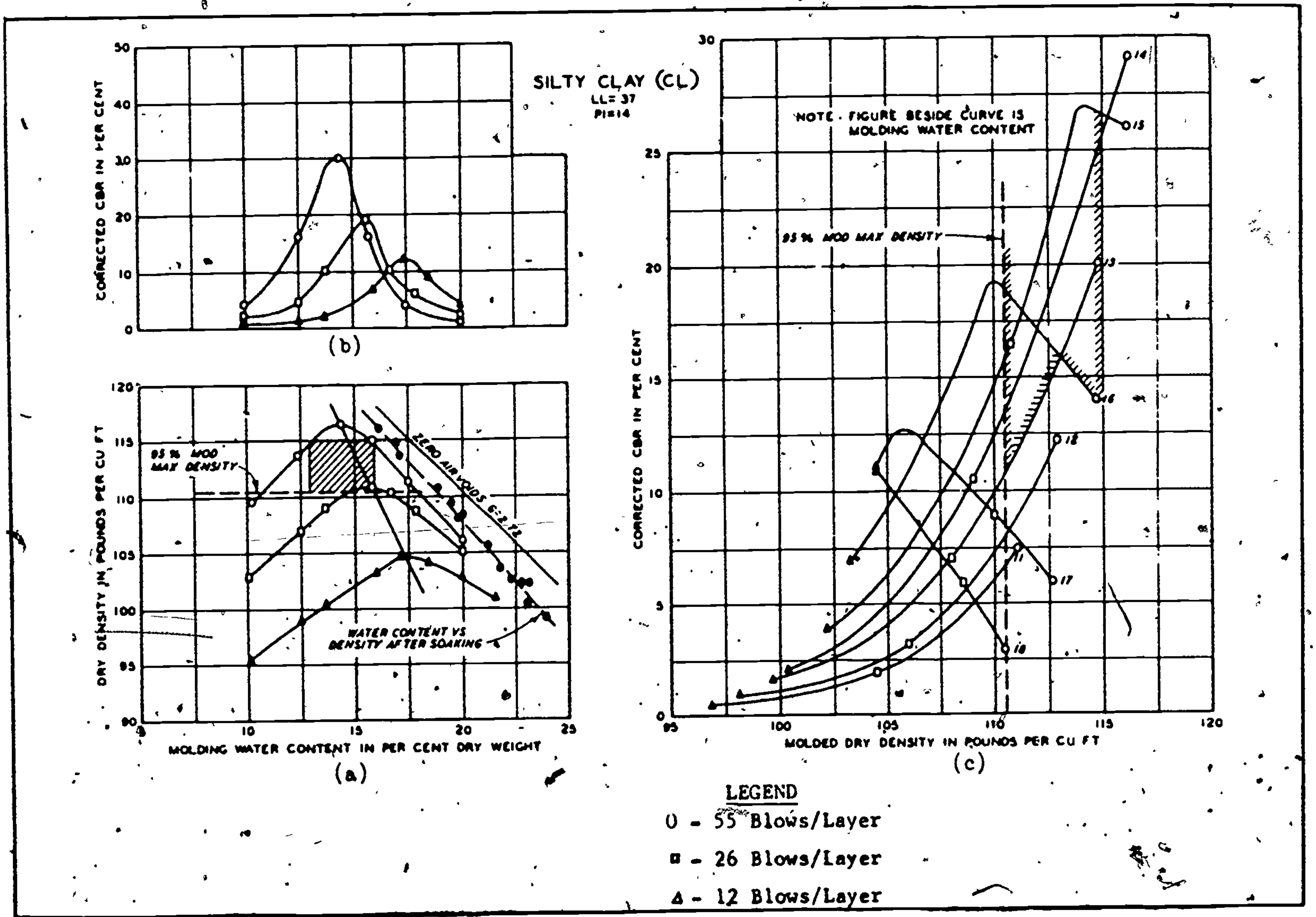


Figure 4-7. Presentation of CBR data.

b. **Procedure.**

(1) **Figure a.** Figure 4-7a, is the compaction graph using the moisture content and density as coordinates. A curve is drawn for each of the three compactive efforts; 55, 26, and 12 blows. The optimum moisture content (OMC) and maximum density are determined as explained. The 95 percent modified AASHO maximum density is computed ($116.5 \times .95 = 110.58$) and the result plotted on the graph. A horizontal line (dashed on the figure) is drawn through the plotted point until it intersects the drawn curve. Values of $\pm$ 1½ percent are plotted either side of the OMC ($14.1 \pm 1½ = 12.6$ or 15.6). For ease in plotting, the lines have been drawn at 13 and 16 percent until they intersect the 55 blow curve and the 95 percent density line. The shaded area in a, figure 4-7, represents values greater than 95 percent modified AASHO density and within about $\pm$ 1½ percent of the OMC. These values will be used later as explained in (3) below.

(2) **Figure b.** Figure 4-7b, is a graph using the relation of corrected CBR values for the same moisture contents as used in graph a. For ease in plotting, the CBR versus water content graph is drawn directly above the density graph and uses the same abscissa as the graph a.

(3) **Desired presentation.** The desired presentation is a graph showing the relation of molded density to corrected CBR (c, fig 4-7). The graph is plotted by selecting some whole percent value of moisture content below the lower one computed in (1) above (11 percent in the example). The density (from graph a) and the CBR (from graph b) corresponding to this selected value are determined for each of the three compactive efforts.

Step 1. To determine the values, place a straightedge or draw a line vertically through the point along the bottom edge corresponding to the selected moisture content.

Step 2. Move up the line until it intersects the 12-blow curve on the density graph (a, fig 4-7) and read the density corresponding to this value.

Step 3. Move up the line to the 12-blow curve on graph b, and read the CBR corresponding to this value.

Step 4. On graph c, plot the CBR versus density (read in steps 1 and 2 above) and mark the point.

Step 5. Using the same selected moisture line, determine the CBR versus density for the 26- and the 55-blow efforts and plot these values on graph c.

Step 6. Draw a smooth curve through the three plotted points.

Step 7. Repeat this process (steps 1 to 6 above) for additional moisture contents using increments of 1 percent until the curve shows a definite falling off. Note the 18 percent curve in c, figure 4-7.

Step 8. Transfer the 95 percent line (110.6 lb per cu ft) and the maximum allowable density line (115 lb per cu ft) from graph a. The hatched vertical lines in graph c indicate these values.

(4) **Interpretation.** To interpret the test results from graph c, consider the hatched portion. This illustrates that the CBR can vary from 11 (13 percent moisture; 95 percent density) to 26 (15 percent moisture; maximum allowable density). For design purposes, a CBR of 11 or 12 would be used and the moisture content specified to stay between 13 and 16 percent. Thus, when 95 percent density (or better) is attained in the field with a moisture content maintained between 13 and 16 percent, the CBR would definitely be above its design value.

(5) **Illustration.** Note how the graph illustrates the effect of moisture content variations on the CBR. A drop to 12 percent would require a maximum compactive effort to reach a CBR of 12. Maintaining this same effort and achieving this same density at 13 percent moisture increases the CBR to 16, and at 14 percent moisture to above 20. At the upper limit, as little as 1 percent increase in moisture (16 to 17 percent) will cause a

drop in CBR from 18 to 8 at 95 percent density. At 18 percent moisture, the CBR would decrease to 3. This presentation illustrates the importance of maintaining the moisture content within the specified range until the 95 percent density is reached.

SELF TEST

Note: The following exercises comprise a self test. The figures following each question refer to a paragraph containing information related to the question. Write your answer in the space below the question. When you have finished answering all the questions for this lesson, compare your answers with those given for this lesson in the back of this booklet. Do not send in your solutions to these review exercises.

1. Define and give the function of the CBR test. (4-1)

2. If gravel and stones are present in considerable quantity, what is the procedure recommended for obtaining a reasonably reliable CBR test result? (4-2)

3. Why is it desirable to have one more base plate than you have molds in a laboratory CBR test set? (4-3a)

4. Test equipment must be available which can be used to force the penetration piston into the specimen at a certain rate. What is the rate of penetration, in inches per minute? (4-3g)

5. Test specimens for the laboratory CBR test should have the same density and moisture content as that expected in the field. How is this condition attained in the laboratory? (4-4a)

6. In the preparation of a remolded sample for the laboratory CBR test, all material retained on a certain sieve size must be eliminated. What is this sieve size? (4-4b)

7. After the soil specimen has soaked for 4 days, it is removed from the water. What further action must then be taken before the specimen is ready for the penetration test? (4-4e)

8. In setting up the equipment for the penetration test, why is an initial load of one pound placed on the penetration piston? (4-5c)

9. During the penetration test, load readings are taken at several predetermined depths of penetration. What use is made of these figures? (4-5f, fig 4-3)

10. In order to determine the CBR of the specimen, values from the plotted curve must be compared to related values from a standard curve. How is the standard curve determined? (4-5g, fig 4-3)

11. For the purpose of testing remolded specimens for the California method of design, all soils have been grouped into three classes: cohesionless sands and gravels, cohesive soils, and highly swelling soils. Why is this done? (4-6a)

12. The objective for tests on highly swelling soils is not the same as for tests on cohesive soils. What is the objective? (4-6d)

13. When the penetration test is to be performed on undisturbed samples, much care must be exercised to obtain specimens which are relatively undisturbed. What method is recommended for gravelly soil? (4-7)

14. What is the primary purpose for making a field in-place CBR test? (4-8a)

15. When performing a field in-place CBR test, some precaution must be exercised in placement of the dial foot. What considerations must be given in this respect? (4-8d(3))

16. When performing the in-place CBR test, three tests are made in the same type of soil providing they are within tolerance of each other. What is the permissable tolerance between the three results in a CBR range from 30-60? (4-8e(9))

First special requirement. Exercises 17 through 21 will give you practical work in the selection of cover requirements and base and pavement thickness for roads assuming the following special situation: You are to design a road for 18,000 pound single-axle dual-wheel load operations of 2.3×10^6. Two borrow pits are available, one with a CBR of 30 and the other with a CBR of 50. The subgrade has a CBR of 15. You have the equipment and material for a pavement.

17. What is the cover, in inches, required above a CBR 15 subgrade? (4-9a, fig 4-5)

18. What is the cover, in inches, required above a CBR 30 subbase? (4-9a, fig 4-5)

19. What is the cover, in inches, required above a CBR 50 base course? (4-9a, fig 4-5)

20. What is the minimum thickness of the base course in inches? (4-9a, fig 4-2)

21. What is the minimum thickness of pavements, in inches? (4-9a, fig 4-2)

Second special requirement. Exercises 22 through 25 will give you practical work in the selection of pavement and base thickness for airfields. These exercises are based upon the following special situation: A design is required for a rear area full operational airfield, capable of handling C118A aircraft. Frost is not a problem. Tests indicate the CBR as: subgrade CBR 8, select material CBR 15, subbase CBR 50, base course CBR 80. The design of type "B" traffic areas will be covered here. Note: use figure 4-6 and 4-8, and table 4-3.

22. What is the design load in kips for the C-118A aircraft? (4-9b, table 4-3)

23. What is the total thicknesses in inches of cover required above the CBR 8 subgrade? (62.8 kips design gear load) (4-9b, fig 4-6)

24. What is the total thickness in inches of cover required above the CBR 50 subbase? (62.8 kips design gear load) (4-9b, fig 4-6)

25. What is the thickness in inches of pavement (in final design)? (62.8 kips design gear load) (4-9b, fig 4-6)

Third special requirement. Exercises 26 through 30 will give you practical work in determining minimum and maximum water content and design CBR using the following special situation: You have a family of CBR curves and a family of compaction curves illustrated in figure 4-9. You have to combine the two graphs to obtain the corrected CBR versus molded dry density in order that you may pick a design CBR and specify a moisture content range.

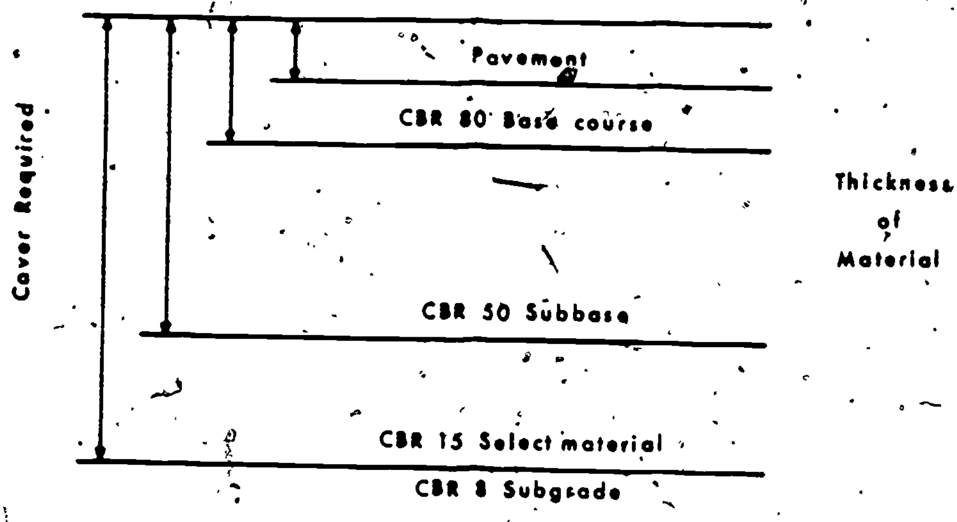
26. What is the CBR for a water content of 10 percent dry weight and a 55 blow/layer compactive effort? (4-10b(3))

27. What is the minimum design water content (percent?) (4-10b(3))

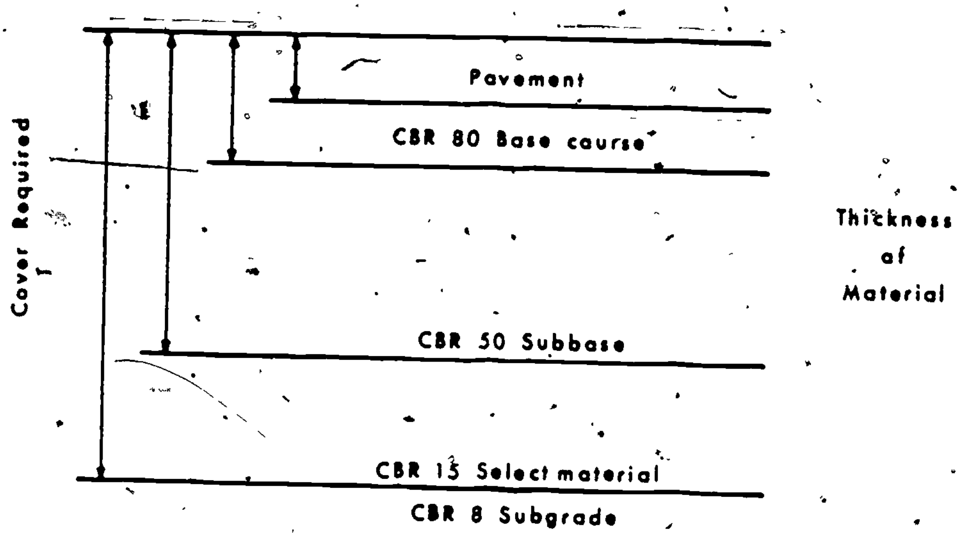
28. What is the maximum design water content (percent?) (4-10b(4))

29. What is the minimum design CBR? (4-10b(4))

30. What is the maximum design CBR? (4-10b(4))



RAW DESIGN



FINAL DESIGN

Figure 4-8. For use with exercises 23 through 25.

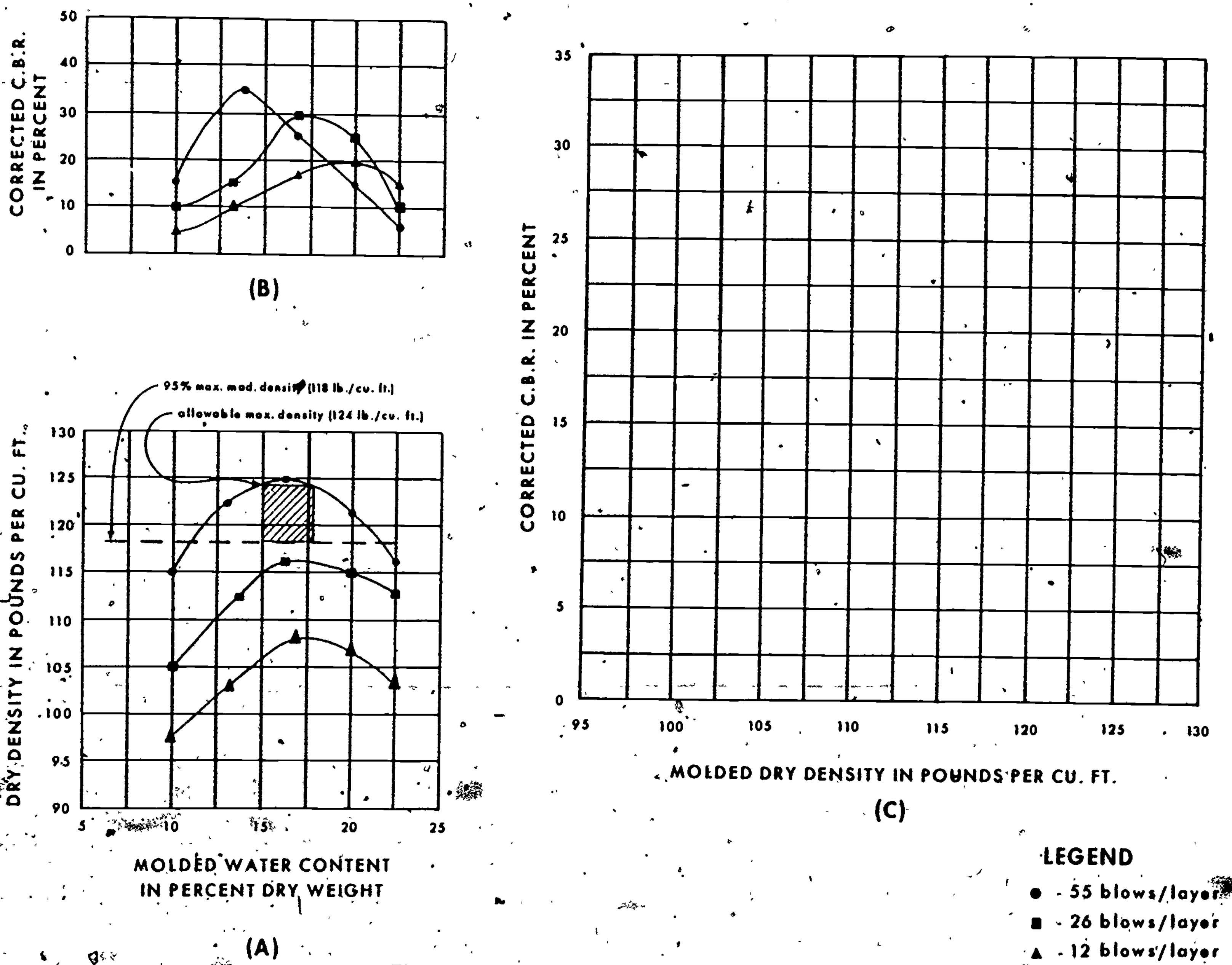


Figure 4-9. For use with exercises 26 through 30.

LESSON 5

FLEXIBLE PAVEMENT STRUCTURE

CREDIT HOURS 4

TEXT ASSIGNMENT .... Attached memorandum.

MATERIAL REQUIRED .... None.

LESSON OBJECTIVE .... Upon completion of this lesson you should be able to accomplish the following in the indicated topic areas:

1. **Considerations in Flexible Pavements.** List and define the major factors which must be considered in the plans and design for a flexible pavement construction project.

2. **Airfield Categories.** Explain the different airfield categories, the reason for different categories in a theater of operations, and the basic design differences.

3. **Airfield Design.** Design a flexible pavement for an airfield utilizing the design curves and other data and criteria as given in this lesson.

4. **Road Design.** Design a flexible pavement for a road utilizing the data and criteria as presented in this lesson.

ATTACHED MEMORANDUM

5-1. FACTORS AFFECTING DESIGN OF A FLEXIBLE PAVEMENT STRUCTURE

a. **Tests.** Soils are subjected to classification tests (Lesson 3) to permit selection of representative samples for more detailed tests to determine compaction characteristics, CBR value, and other properties needed for designing the flexible pavement structure. Subbase and base course materials are tested for compliance with specification requirements, to determine suitability of materials, and also to ascertain CBR values in certain instances. When the explorations and tests are completed, limiting conditions in the subgrade and subsoil must be determined, materials selected, and CBR or other design values selected for the various layers. These procedures require good judgment on the part of the military engineer.

b. **Pavement composition.** Figure 5-1 shows two typical sections of flexible pavements, one with thick and one with thin base course. In either case the subgrade is the foundation which eventually carries any load applied at the surface. The airfield or road usually must be leveled and shaped; consequently, the subgrade is customarily defined as the natural soil which is compacted or otherwise treated to receive the base and wearing courses. The base and subbase are composed of higher quality material than the subgrade, either imported or selected at the site. The design of flexible pavement is based on the principle that the magnitude of stress induced by a wheel load decreases with depth below the surface. Consequently, the stresses induced in a given subgrade material can be decreased by increasing the thickness of the

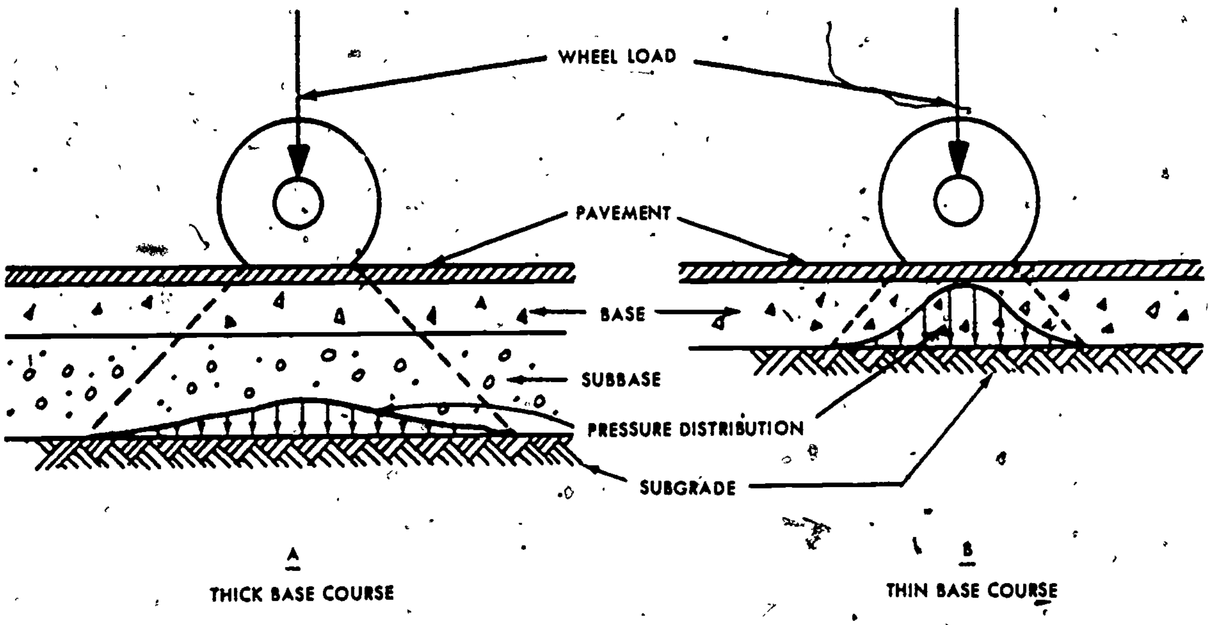


Figure 5-1. Distribution of pressure under single-wheel loads.

superimposed base and pavement. Figure 5-1 illustrates this point. In the diagram at the left, the base (including subbase) is thick, the load at the subgrade level is spread over a wide area, and the pressures there are small. In the diagram at the right the base is thin and the load at the subgrade level is confined to a much smaller area, with the result that the pressures at subgrade level are correspondingly higher. This pattern of decreasing stresses with increasing depth is the basis of the conventional flexible pavement design in which subgrade materials, of low bearing capacity are covered with thick base courses, whereas thin base courses are adequate for subgrade materials with high bearing capacities.

c. **Effects of wheel assemblies and tire pressure.**

(1) **Wheel assembly.** The diagram in figure 5-1 illustrates the distribution of pressure under a single-wheel load. Multiple-wheel assemblies are beneficial in the case of flexible pavements having high subgrade strength and a thin base course because the stresses produced by the tires of multiple-wheel assemblies do not overlap appreciably at shallow depths. This is illustrated by plane A-A in figure 5-2. In the case of flexible pavements with low subgrade strength and thick base course, the stresses produced overlap (plane B-B, fig 5-2), and less benefit is gained from the use of multiple-wheel assemblies. Criteria are given herein for designing and evaluating multiple-wheel assemblies for both road and airfield design.

(2) **Tire pressure.** The intensity of stress at a given point in a flexible pavement is directly affected by the tire-contact area and the tire pressure. The major difference in stress intensities caused by variation in tire pressure occurs near the surface; consequently, the pavement and upper base course are most seriously affected by high tire pressures.

5-2. COMPONENTS OF FLEXIBLE PAVEMENT STRUCTURE

A typical flexible pavement structure is shown in figure 5-3, which illustrates the terms used in this lesson to refer to the various layers. All the layers shown in figure 5-3 are not present in every flexible pavement. For example, a two-layer structure consisting of only a compacted subgrade and a base course is a complete flexible pavement. Also the word "pavement" when used by itself refers to only the leveling, binder, and surface course, while the words "flexible pavement" refer to the entire pavement structure from the subgrade up.

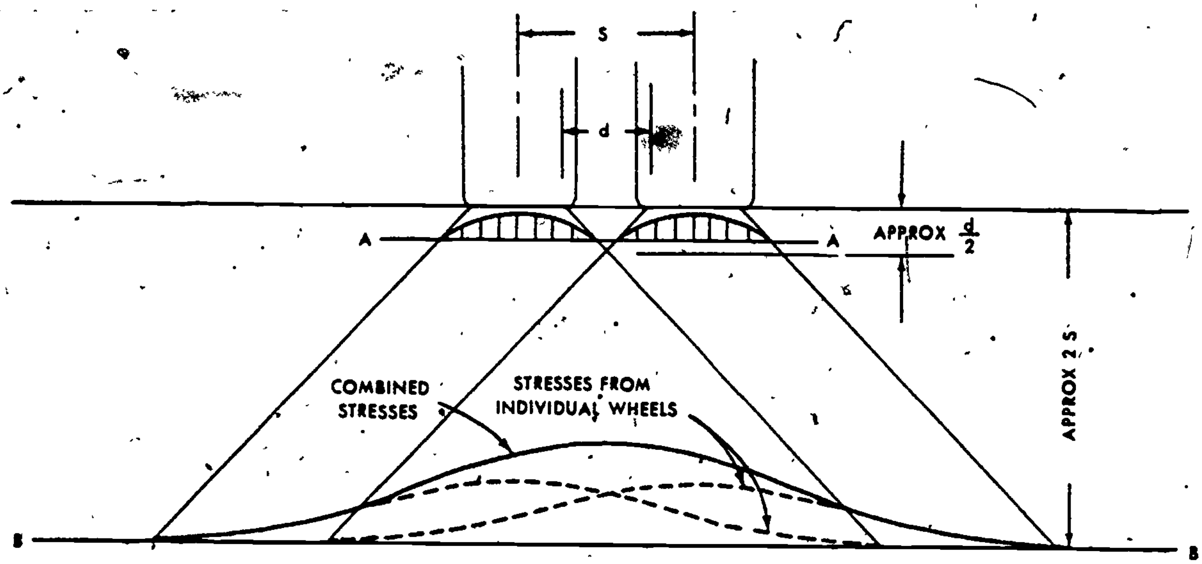
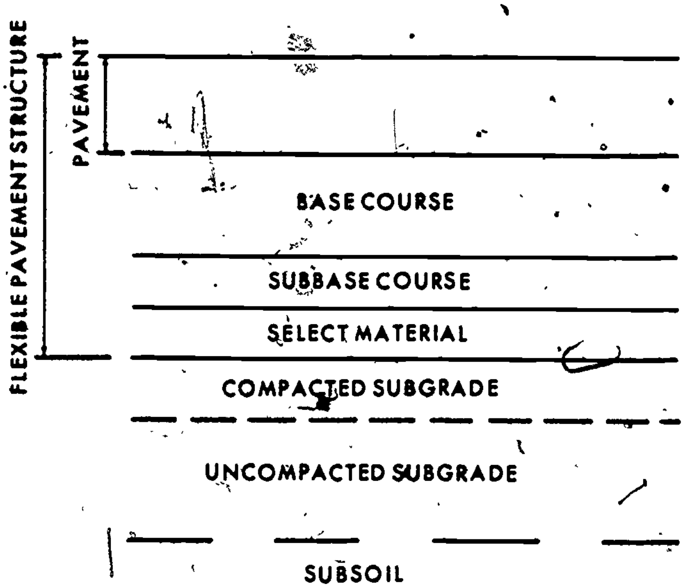


Figure 5-2. Distribution of pressures produced by multiple-wheel assemblies.



NOTE:

1. THE WORD "STRUCTURE" IS OFTEN DELETED FROM THE PHRASE "FLEXIBLE PAVEMENT STRUCTURE." ALSO THE WORD "COURSE" IS OFTEN DELETED FROM "BASE COURSE" AND "SUBBASE COURSE."
2. ALL LAYERS AND COATS ARE NOT PRESENT IN EVERY FLEXIBLE PAVEMENT STRUCTURE.
3. DEMARCATION BETWEEN SUBBGRADE AND SUBSOIL IS INDEFINITE.

Figure 5-3. Typical flexible pavement.

5-3. CONSIDERATIONS RELATIVE TO EACH MATERIAL MAKING UP FLEXIBLE PAVEMENT

a. Subgrade.

(1) Factors to be considered. The information obtained from the explorations and tests previously referred to should be adequate to enable full consideration of all factors affecting the suitability of the subgrade and subsoil. The primary factors are as follows: the general characteristics of the subgrade soils; depth to ledge rock; depth to water table; the compaction that can be attained in the subgrade and the adequacy of the existing density in the layers below the zone of compaction requirements; the CBR that the compacted subgrade, uncompacted subgrade, and subsoil will have under future conditions; the presence of weak or soft layers in the subsoil; susceptibility to detrimental frost action.

(2) Selection of subgrade and subsoil design CBR values. The CBR test described in Lesson 4 includes procedures for making tests on samples compacted in test molds to the design density and soaked 4 days, for making in-place CBR tests, and for making tests on undisturbed samples. These tests are used to estimate the CBR that will develop in the prototype structure except that where the design CBR is above 20 the subgrade must meet the requirements for subbases.

b. Select materials and subbase courses.

(1) Procedures. It is common practice in flexible pavement design to use locally available or other relatively cheap materials between the subgrade and base course for economy. Those layers are designated as select materials or subbases. Those with design CBR values below 20 are arbitrarily called select materials, while those with CBR values of 20 and above are called subbases. Minimum thicknesses of pavement and base have been established to eliminate the need for subbases with design CBR values above 50. Where the design CBR value of the subgrade without processing is in the range of 20 to 50, select materials and subbases may not be needed. However, the subgrade cannot be assigned design CBR values of 20 or higher unless it meets the gradation and plasticity requirements for subbases. In some cases, where subgrade materials meet plasticity requirements but are deficient in grading requirements, it may be possible to treat an existing subgrade by blending in stone, limerock, sand, etc., to produce an acceptable subbase; however, it is emphasized that "blending in" cohesionless materials to lower the plasticity index will not be allowed in any case.

(2) Materials.

(a) Select material. Select materials will normally be locally available coarse-grained soils (prefix G or S), although fine-grained soils in the ML and CL groups may be used in certain cases. Limerock, coral, shell, ashes, cinders, caliche, disintegrated granite, and other such materials should be considered when they are economical. Recommended plasticity requirements are listed. These are suggested to insure a material that can be processed readily. Materials not meeting these requirements may be considered where it can be shown that they can be processed readily. A maximum size of 3 inches is suggested to aid in meeting grades.

(b) Subbase materials. Subbase materials may consist of naturally occurring coarse-grained soils or blended and processed soils. Materials such as limerock, coral, shell, ashes, cinders, caliche, and disintegrated granite may be used as subbases when they meet the requirements described. As noted in the preceding paragraph, the existing subgrade may meet the requirements for a subbase course or it may be possible to treat the existing subgrade to produce a subbase. Also, as noted, admixing native or processed materials will be done only when the subgrade, unmixed, meets the liquid limit and plasticity index requirements for subbases, because it has been found by experience that "cutting" plasticity in this way does not work out satisfactorily. Material stabilized with commercial admixes may be economical as subbases in certain instances. Portland cement, cutback asphalt, emulsified asphalt, and tar are commonly employed for this purpose, also, it may be possible to decrease the plasticity of some materials by use of lime or portland cement sufficiently to make them suitable as subbases.

(3) **Selection of design CBR for select material and subbases.** Tests are usually made on remolded samples; however, where existing similar construction is available, CBR tests should be made in-place on material when it has attained its maximum expected water content or on undisturbed soaked samples. The procedures for selecting test values described for subgrades apply to select materials and subbases. The CBR tests are supplemented by the following gradation and Atterberg limits requirements for subbases as indicated in table 5-1. Suggested limits for select materials are also indicated. In addition to the requirements shown in the table, the material must also show in the laboratory tests a CBR equal to or higher than the CBR assigned to the material for design purposes. Cases may occur in which certain natural materials that do not meet the gradation requirements may develop satisfactory CBR values in the prototype. Exceptions to the gradation requirements are permissible when supported by adequate in-place CBR tests on construction that has been in service for several years. The CBR test is not applicable for use in evaluating materials stabilized with chemical admixtures, and they must be rated by judgment in terms of an equivalent CBR. Ratings as high as 50 can be assigned these materials when proper construction procedures are followed.

TABLE 5-1. *Recommended Maximum Permissible Values of Gradation and Atterberg Limit Requirements in Subbase and Select Materials*

Material	Maximum design CBR		Size inches		Gradation requirements % passing				Atterberg limits			
					No. 10		No. 200		LL		PI	
	Airfields	Roads	Airfields	Roads	Airfields	Roads	Airfields	Roads	Airfields	Roads	Airfields	Roads
Subbase	50	50	3	2	50	50	15	15	25	25	5	5
Subbase	40	40	3	2	80	80	15	15	25	25	5	5
Subbase	30	30	3	2	100	100	15	15	25	25	5	5
Select Material	Below 20	20	3	3	-----	-----	25	-----	35	35	12	12

c. **Base course.**

(1) **Essential features.** The purpose of a base course or courses is to distribute the induced stresses from the wheel load so that they will not exceed the strength of the subgrade. Figure 5-4 shows an idealized representation of the distribution of stress through two base courses. When the subgrade strength is low, the stress must be reduced to a low value and a substantial thickness of base is needed. Where the subgrade strength is higher, a lesser thickness will provide adequate distribution. Since the stresses in the base course are always higher than in the subgrade (fig 5-4) it stands to reason that the base course must have higher strength. Similarly, where two or more different types of base courses are used, the better quality material is placed on top.

(2) **Base course requirements.**

(a) **Principles.** Careful attention should be given to the selection of materials for base courses and to their construction. The materials should be dense and uniformly compacted so no differential settlement occurs in adjacent areas. For continuous stability, all base courses should meet the requirements listed below.

(b) **Gradation requirements.** Gradation of particle size must, whenever feasible, be within specified limits as determined

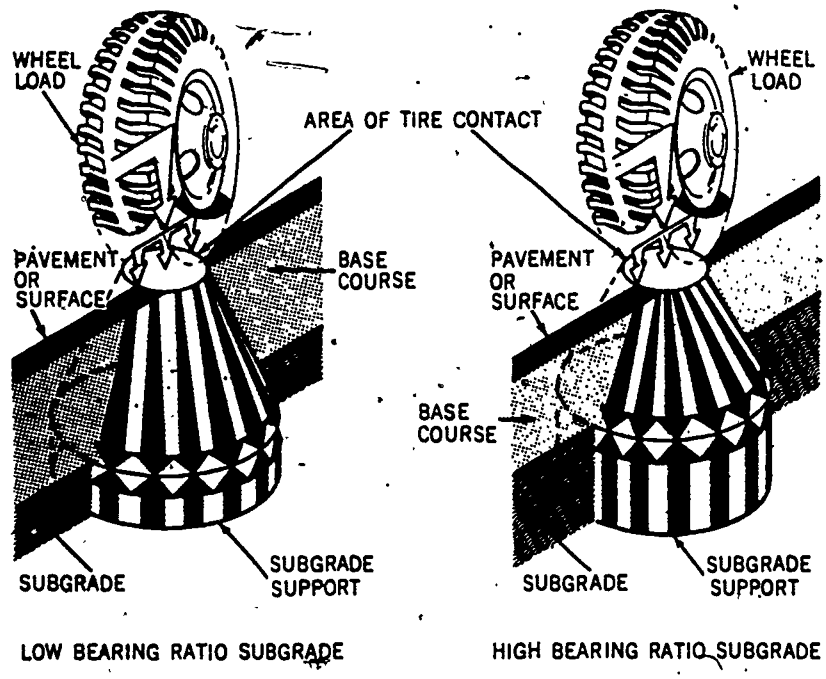


Figure 5-4. Distribution of stress in base courses and effects of subgrade strength on base course thickness.

by mechanical analysis. However, in construction in forward areas it may not be practicable to hold to close gradation requirements. For deliberate construction, base course material should contain no more than 15 percent passing a No. 200 sieve.

(c) **Plasticity requirements.** Material passing the No. 40 sieve which acts as a binder in a base course material must have desirable properties. Requirements for mechanically stabilized soil bases are given in lesson 7. No material which has a liquid limit greater than 25 or a plasticity index greater than 5 should be used for a base course in deliberate construction.

(d) **Compaction and strength requirements.** Thickness of layers in constructing base courses must be within the limits which will insure proper compaction. Thickness of layers depends upon type of material, equipment used, and method of construction used. All base courses must be compacted. Compaction should meet the requirements given in lesson 3. The CBR of the finished base course must conform to that used in the design, and the total compacted thickness must equal that obtained from the design curves as discussed. Table 5-2 lists nine types of materials and/or processes that may be used as base courses for roads and airfields. A design CBR is given for each type.

TABLE 5-2. *Assigned CBR Ratings for Base Course Material*

No.	Type	Design CBR
1	Graded crushed aggregate	100
2	Water-bound macadam	100
3	Dry-bound macadam	100
4	Bituminous base course, central plant, hot mix	100
5	Limerock	80
6	Bituminous macadam	80
*7	Stabilized aggregate	80
8	Soil cement	80
9	Sand shell or shell	80

*It is recommended that stabilized aggregate base course material not be used for tire pressures in excess of 100 psi.

5-4. DESIGN CATEGORIES (AIRFIELDS)

a. Basis of design.

(1) **Principles.** Each of the different types of airfields in the basic airfield complex is included for a specific purpose. Since the type, volume, composition, and character of anticipated traffic is much greater in the rear area, a different pavement structure and a resilient, waterproof, load-distributing medium which protects the base course from the detrimental effects of water and the abrasive action of traffic may be required. This section then will only deal with the following types of airfields that may require a flexible pavement structure: the heavy lift, rear area; the tactical, rear area; and the Army rear area.

(2) **Heavy lift, rear area.** This is a facility that must accept a high volume of C-141, C-135, or other heavy transport aircraft carrying the strategic intertheater tonnage from the continental United States to the theater of operations. These airfields must, in most cases, have an all-weather surface of adequate strength to fulfill their mission.

(3) **Tactical, rear area.** This is a facility that must support high performance fighter aircraft for fighting the air war and conducting aerial mapping and reconnaissance missions. The design life is expected to range from 6 months to 2 years. The surfacing may be a landing mat on an adequate base or a flexible pavement, if an adequate existing surface is not available.

(4) **Army rear area.** This is a facility that must accept a high volume of Army aircraft to perform command and control, observation and surveillance, and logistics support. The surfacing may be a landing mat on an adequate base or a flexible pavement, if an adequate existing surface is not available.

b. Pavement thickness. The full design thickness of a pavement for any aircraft is known as the "full operational" thickness. This thickness will support traffic of the design aircraft equivalent to 1,000 coverages with only moderate maintenance. For greater traffic intensity the maintenance will be heavier. A coverage is defined as the equivalent of tracking the full width of a runway or taxiway pavement by successive nonoverlapping passes of aircraft tires. The term "cycle" is defined as one landing and one takeoff operation by an aircraft. When traffic is less than the equivalent of 1,000 coverages, the pavement thickness required is less than full operational. There are two pavement design standards which are lower than the full operational. These are the "minimum operational" design, suitable for traffic equivalent to 200 coverages, and the "emergency" design, suitable for traffic equivalent to 40 coverages. Pavements will rarely be designed for the "emergency" category which anticipates only a 2-week life. This category, however, may be used for rear area airfields as an initial construction effort, or for use while longer life fields are under construction. The general relationship between pavement thickness, anticipated life, traffic, and maintenance is shown in table 5-3. The terms "full

TABLE 5-3. *Relation Between Traffic and Pavement Design Thickness*

Classification by construction type	Maximum No. coverage	Approximate No. of cycles	Anticipated pavement life	Pavement thickness	Maintenance
Emergency	40	100-500 Cargo 800 Fighter	2 weeks	60% Full operational	Heavy and continuous
Minimum operational	200	500-1800 Cargo 4,000 Fighter	6 months	80% Full operational	Daily
Full operational	1,000	2,500-12,000 Cargo; 20,000 Fighter	2 years	Full operational	Weekly

operational", "minimum operational", and "emergency" are used in this subcourse only to refer to the pavement thickness and the related traffic intensity and design curves.

c. **Traffic areas.**

(1) **Principles.** On a theater of operations airfield, the pavements can be grouped into two traffic areas, designated as types B and C, defined below and shown in figure 5-5.

(2) **Type B traffic area.** Type B traffic areas are those in which the traffic is more evenly distributed over the full width of the pavement facility but which received the full design weight of the aircraft during traffic operations. Pavement facilities considered to be type B traffic areas are as follows: end 1,000 feet of runways; primary taxiways; all parking aprons, warmup pads, hardstands, and aircraft power-check pads.

(3) **Type C traffic area.** Type C traffic areas are those in which the volume of traffic is low or the weight of the operating aircraft is generally less than the design weight. In the interior portion of runways, there is enough lift on the wing of the aircraft at the speed at which the aircraft passes over the pavements to reduce considerably the stresses applied to the pavements. Thus the pavement thickness can be reduced in these portions of the runways. Pavement facilities considered to be type C traffic areas are as follows: interior portion of runway between 1,000-foot ends; maintenance aprons; ladder taxiways.

(4) **Smoothness requirement for jet aircraft.** Smoothness design for all paved surfaces has become extremely critical with high performance jet aircraft, which have takeoff and landing speeds approaching 200 miles per hour. On uneven surfaces at high speeds, jet aircraft have a tendency to "porpoise" and "vibrate". Therefore, it is recommended that no pavement surface should depart more than ½ inch from the design grade and that local smoothness should have no more than ⅛ inch deviation from a 12-foot straightedge (longitudinal). As a minimum the smoothness should be such as to insure that no damage will occur to operating aircraft.

5-5. DESIGN PROCEDURE (AIRFIELDS)

a. **Design curves.** The design curves were developed by the Corps of Engineers. In the basic correlation of the curves with traffic records, single wheels at 100 psi tire pressures were used. In general, the correlation shows that traffic records are in substantial agreement with the indicated thicknesses where the overlying layers were of adequate quality so that no shear deformation occurred in them. The curves for the loadings and traffic areas were obtain by resolving the single-wheel 100 psi curves into curves for high pressure tires and multiple wheel landing gears by theoretical methods. These curves have been spot-checked by accelerating traffic tests and correlated with aircraft traffic. These curves are available from numerous sources although only an example of one will be given here. Table 4-3 in back of this booklet, contains a list of USAR aircraft with their gear characteristics, tire, and weight data, and recommended minimum pavement and base thickness. The design curves represent the recommended required thickness of stable material overlying the layer under consideration. There may be some difference in the stress-distributing characteristics of subbases and bases with different CBR values. However, until further investigation shows the need for a change, the same total thickness will be used for designs that incorporate low CBR materials in subbase as for designs that are composed of material with a high CBR for the full depth. Experience has shown that thin layers of material may not act as independent structural elements adding their proper strength to the entire construction. Accordingly it is recommended that no structural layer except the pavement shall be less than 6 inches thick. This is not, however, to be construed as a limitation on life thickness within a layer. In regions where the annual precipitation is less than 15 inches and the water table (including perched water table) will be at least 15 feet below the finished pave-

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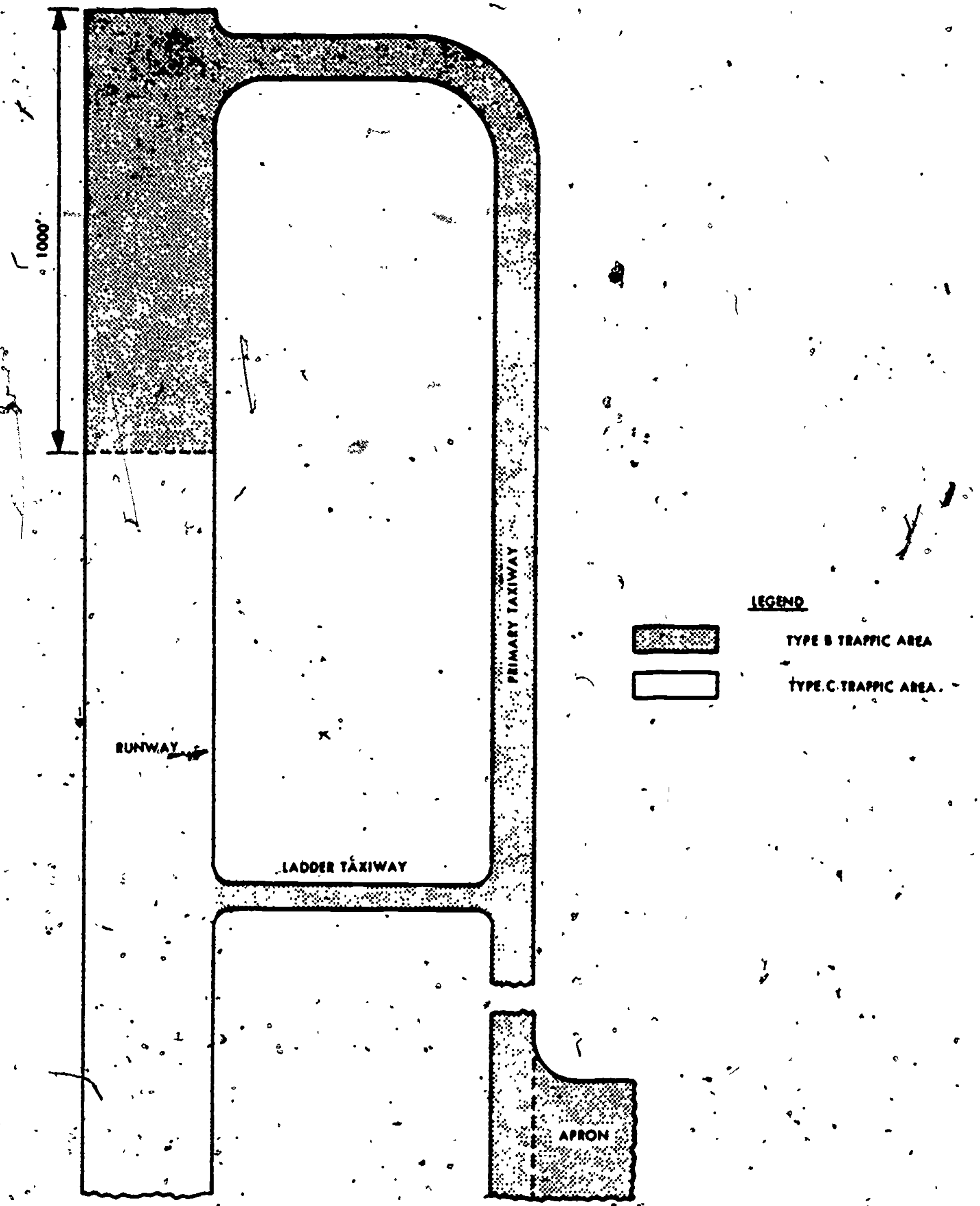


Figure 5-5. Typical layout pavement for theater of operations airfields.

ment surface, the danger of high moisture content in the subgrade is reduced. Where information on existing construction in these regions indicates that the water content of the subgrade will not increase above optimum as determined by the modified AASHO compaction test, the total thickness above the subgrade as determined by CBR tests on **soaked samples, may be reduced 20 percent.** The reduction will be effected in the select material or in the subbase courses having the lowest CBR value. The reduction applies to the total thickness dictated by the subgrade CBR and in no case will be additionally applied to that portion of the total thickness required by the CBR of higher layers. When only limited rainfall records are available, or the annual precipitation is close to the 15 inch criterion, careful consideration will be given to such factors as the number of consecutive years in which the annual precipitation exceeded 15 inches, and the sensitivity of the subgrade to small increases in moisture content, before any reduction in thickness is made.

b. **Air Force airfield design examples.**

(1) **Statement of requirement.** A design is required for a rear area full operational airfield, capable of handling C-141A aircraft. Frost is not a problem. Tests indicate the CBR as: subgrade, CBR 5, cohesive; select materials, CBR 15, cohesive; subbase materials, CBR 40, cohesionless; base course materials, CBR 80, cohesionless.

(2) **Thickness requirements.**

Step 1. Table 4-3 indicates that a C-141A aircraft has a design load of 149,500 pounds. This table further indicates that for a base course of CBR 80 the recommended thickness of base course is 6 inches and pavement thickness is 3½ inches.

Step 2. Figure 5-6 indicates that for 149,500 pounds (say 150,000 pounds) design load and a subgrade of CBR 5, the total thickness for type B traffic area is 42 inches, and for type C traffic area is 34 inches. Using the same design curve and assuming that the next layer above the subgrade will be the select material with a CBR of 15, the curve indicates that the cover required for the select material is 18 inches for type B areas and 14.5 inches for type C areas. The next layer will be the subbase material with a CBR of 40. The curve indicates that for the type B area the thickness above this material must be 8 inches and for the type C area 6 inches. For the 80 CBR base course the curve indicates a cover of 3.5 inches or 3.5 inches of pavement for the type B area and 2.5 inches for the type C area. A tentative sketch (raw design) can now be made for the traffic areas.

Step 3. Raw design and final design sketches (figs 5-7 and 5-8) are prepared for type B and type C areas, respectively.

Step 4. The raw design for the type B traffic area must now be revised to conform to all the requirements for thickness of pavement and base course as given in table 4-3, minimum thickness requirements of 6 inches, and the indicated compaction requirements for each material.

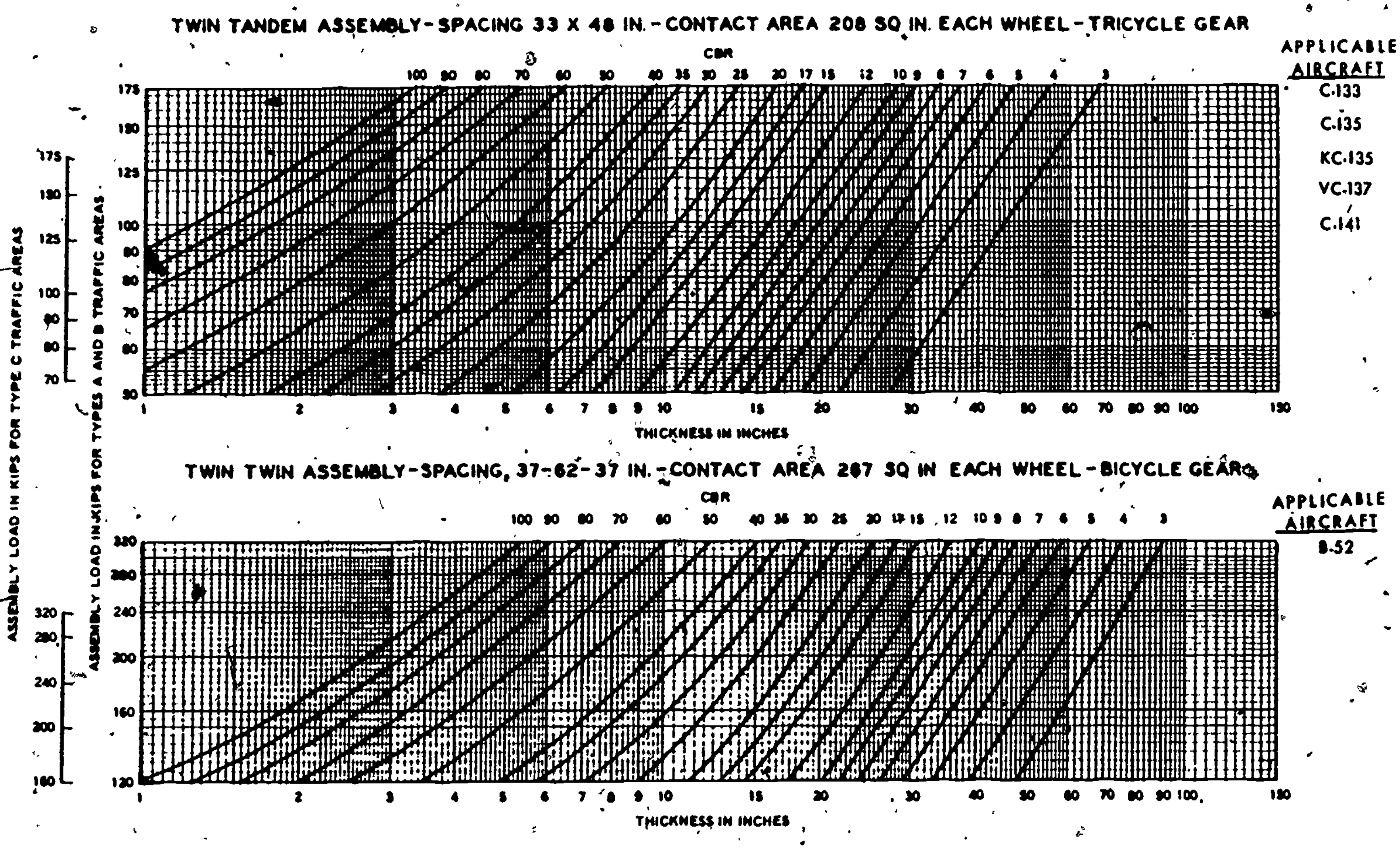
Step 5. Table 4-3 for this aircraft for a full operational airfield and a CBR 80 base course states that the pavement must be at least 3.5 inches in thickness and the base course 6 inches in thickness. The final design is then computed as shown in figure 5-7.

Step 6. The amount of cover over the CBR 40 material is therefore 9.5 inches to comply with the criteria of 3.5 inches of pavement and 6 inches of base course. This decreases the thickness of the CBR 40 subbase material from 10 inches to 8.5 inches, which is still greater than the minimum of 6 inches.

Step 7. For type C traffic areas, raw design and final design are shown in figure 5-8.

5-6. DESIGN CATEGORIES FOR ROADS

a. *Step 1.* Estimate the number of operations each type of vehicle is expected to use the proposed road during its design life. This may be based on knowledge of traffic



5—11

TYPE A TRAFFIC AREA - USE IN-PLACE THICKNESSES WITH THESE CURVES FOR MINIMUM OPERATION

TYPES B AND C TRAFFIC AREAS - USE IN-PLACE THICKNESSES WITH THESE CURVES FOR FULL OPERATION

Figure 5-6. Flexible pavement evaluation curves, tricycle type landing gear, twin tandem and twin twin assemblies, full operational category.

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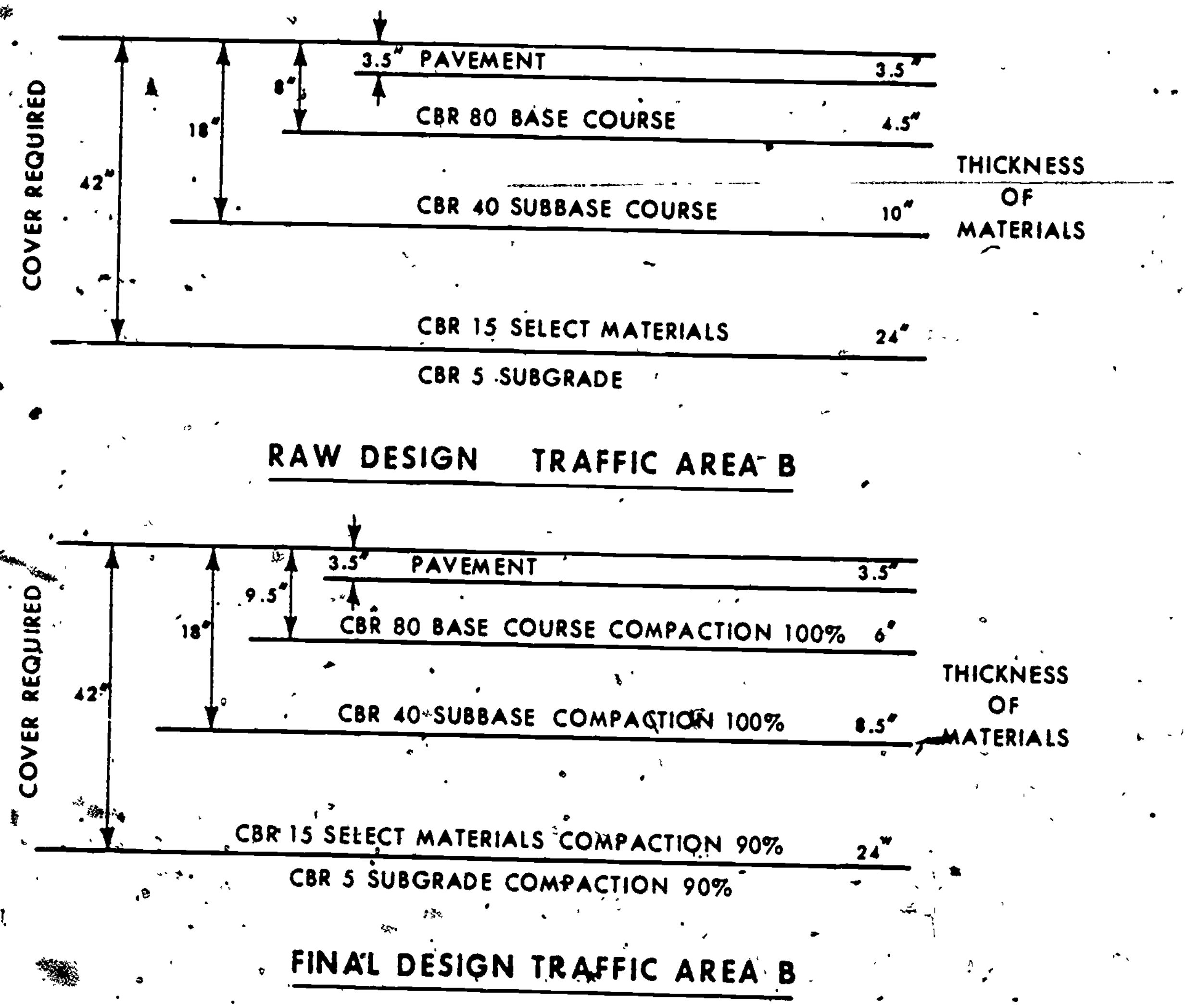


Figure 5-7. Raw and final design and compaction requirements for type B traffic areas for C-141A aircraft.

using similarly used roads or on anticipated traffic. It must be borne in mind that the greatest influence on thickness design is the type and number of operations of very heavy vehicles. This will be made clear in the following example problem.

b. *Step 2.* Estimate how long a period of time the proposed road will be needed. This is called its "design life".

c. *Step 3.* Convert the operations of each type of vehicle into equivalent 18,000-pound, single-axle, dual-wheel load operations through the use of equivalent operation factors shown in figures 5-9 and 5-10.

d. *Step 4.* Sum up total number of equivalent 18,000-pound, single-axle, dual-wheel load operations and apply to the design curves shown in figure 4-5.

e. **Design example.**

(1) **Problem.** To illustrate thickness design by the CBR method, assume that a main supply route is to be designed for a 2-year design life on a subgrade with a design

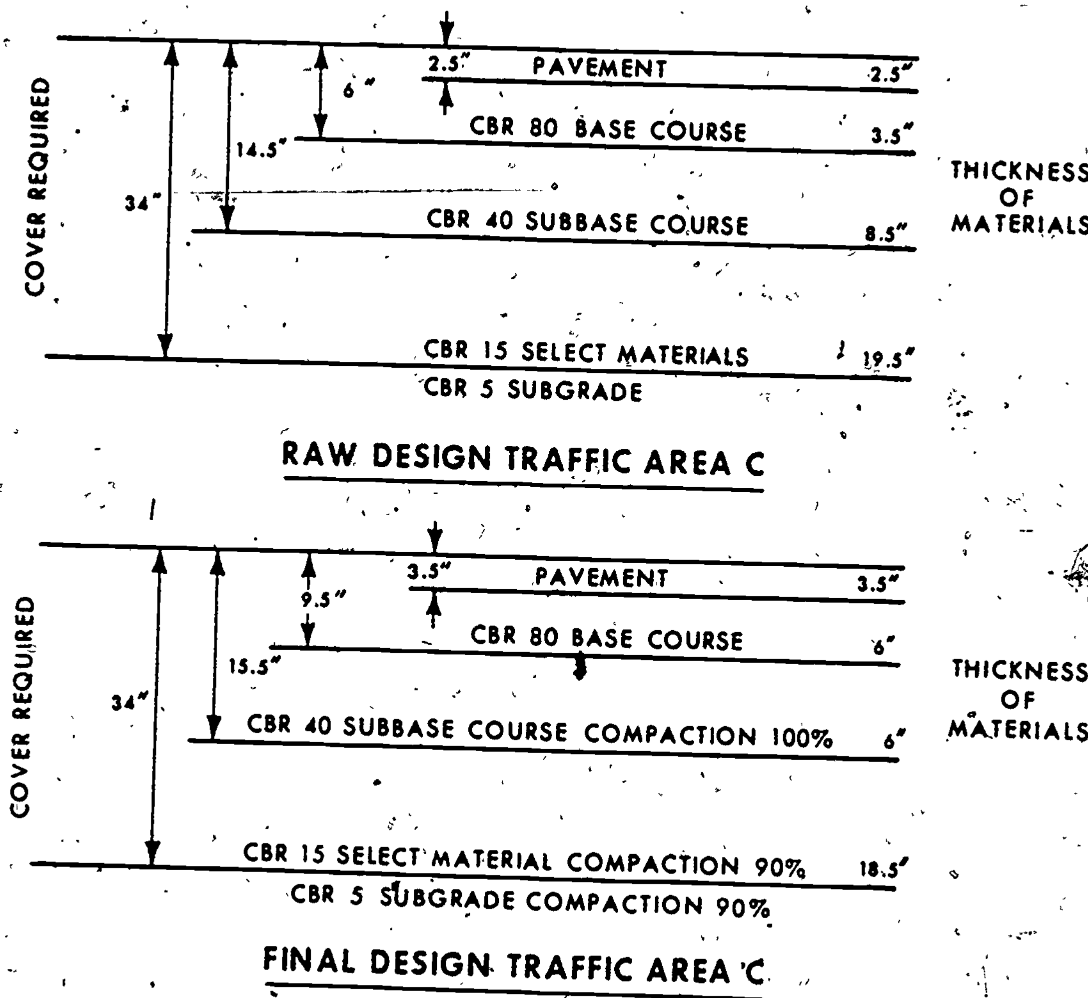


Figure 5-8. Raw and final design and compaction requirements for type C traffic areas for C-141A aircraft.

CBR value of 10. The road is to be an all-weather road and materials and equipment are available to construct an asphalt concrete surface. Ample quantity of base course material with a CBR value of 50 is available. In addition, a pit has been located adjacent to the road site that will furnish material for subbase with CBR value of 40.

(2) *Step 1.* Assume further that as a result of study of previous main supply routes, it is estimated that this road will be subjected to the average daily volume of traffic (step 3).

(3) *Step 2.* The estimated design life is given as 2 years.

(4) *Step 3.* Convert operations of these axle and gross loads into equivalent 18,000-pound, single-axle, dual-wheel load operations through use of the equivalent operations factor curves shown on figures 5-9 and 5-10 follows:

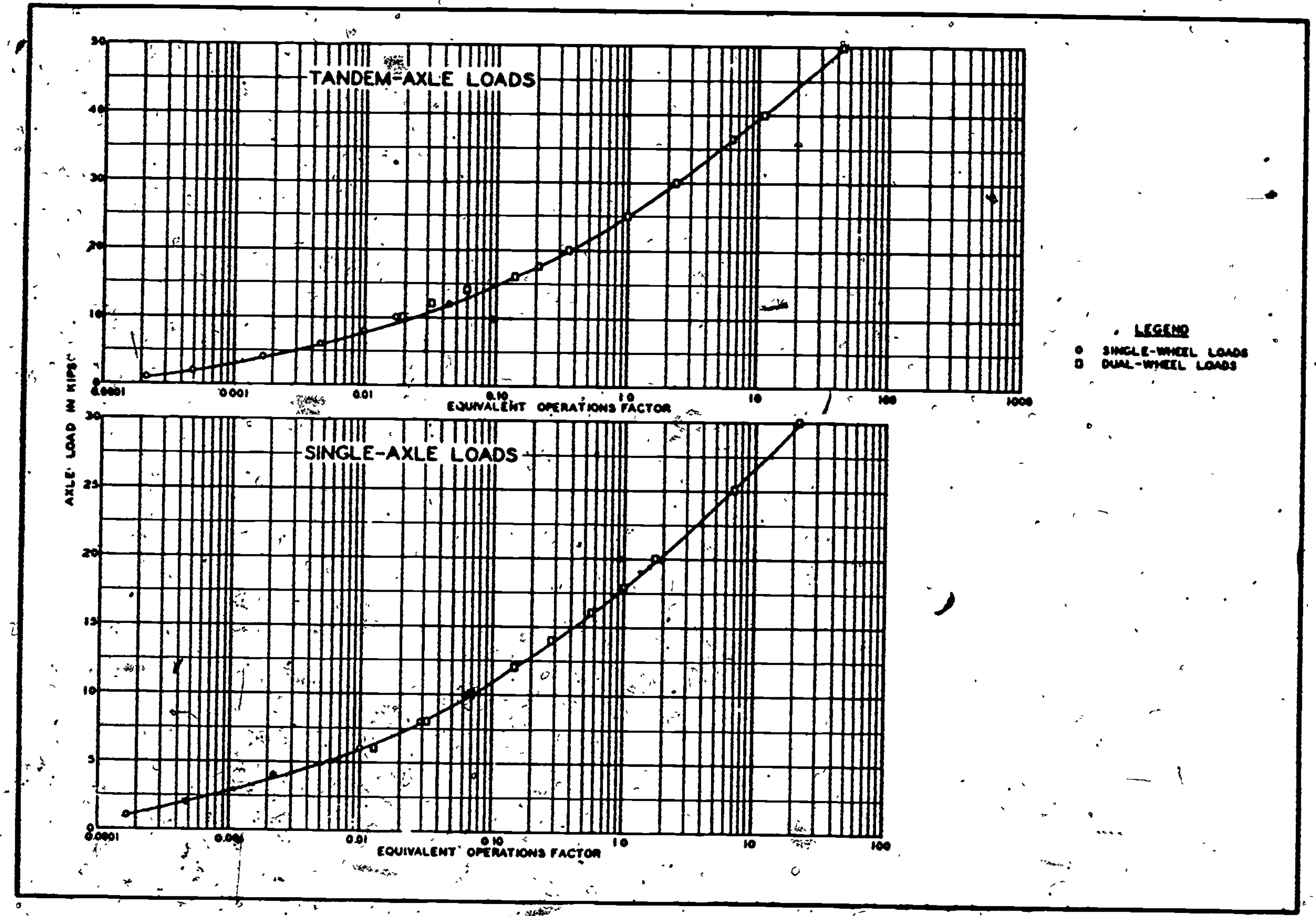
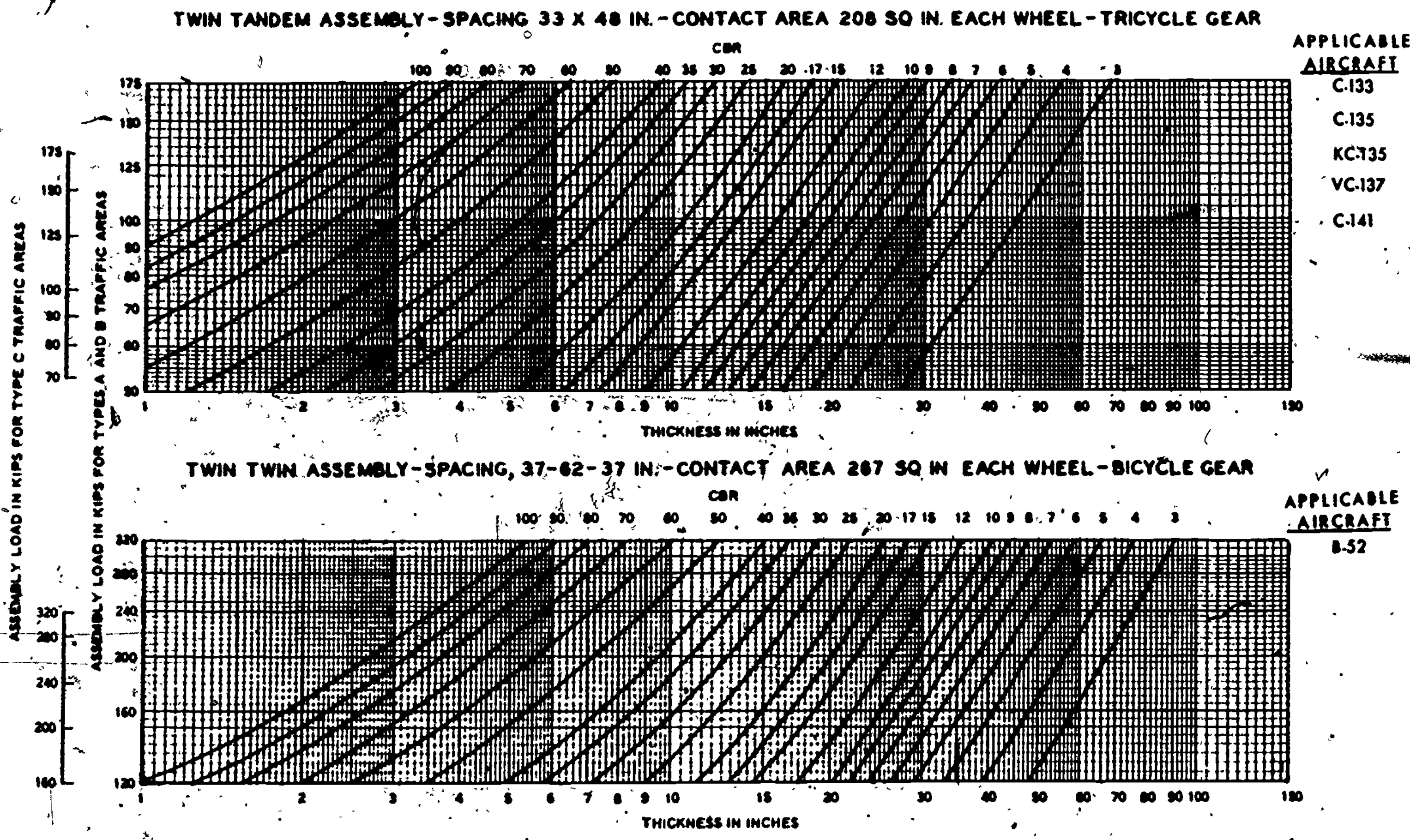


Figure 5-9. Equivalent operations factors for tandem-axle loads and single-axle loads.



TYPE A TRAFFIC AREA	- USE IN-PLACE THICKNESSES WITH THESE CURVES FOR MINIMUM OPERATION.
TYPES B AND C TRAFFIC AREAS	- USE IN-PLACE THICKNESSES WITH THESE CURVES FOR FULL OPERATION.

Figure 5-6. Flexible pavement evaluation curves, tricycle type landing gear, twin tandem and twin twin assemblies, full operational category.

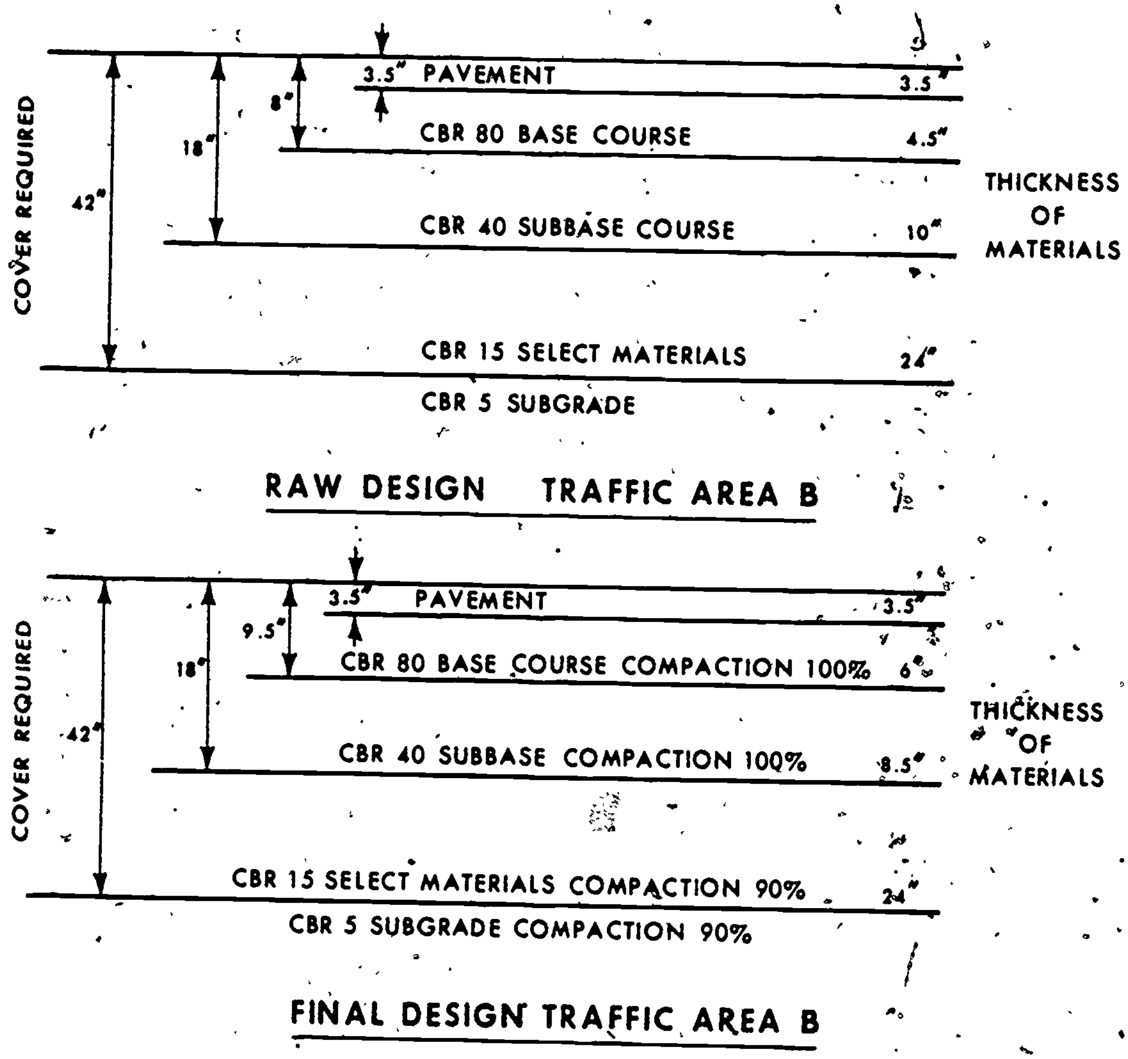


Figure 5-7. Raw and final design and compaction requirements for type B traffic areas for C-141A aircraft.

using similarly used roads or on anticipated traffic. It must be borne in mind that the greatest influence on thickness design is the type and number of operations of very heavy vehicles. This will be made clear in the following example problem.

b. *Step 2.* Estimate how long a period of time the proposed road will be needed. This is called its "design life".

c. *Step 3.* Convert the operations of each type of vehicle into equivalent 18,000-pound, single-axle, dual-wheel load operations through the use of equivalent operation factors shown in figures 5-9 and 5-10.

d. *Step 4.* Sum up total number of equivalent 18,000-pound, single-axle, dual-wheel load operations and apply to the design curves shown in figure 4-5.

e. **Design example.**

(1) **Problem.** To illustrate thickness design by the CBR method, assume that a main supply route is to be designed for a 2-year design life on a subgrade with a design

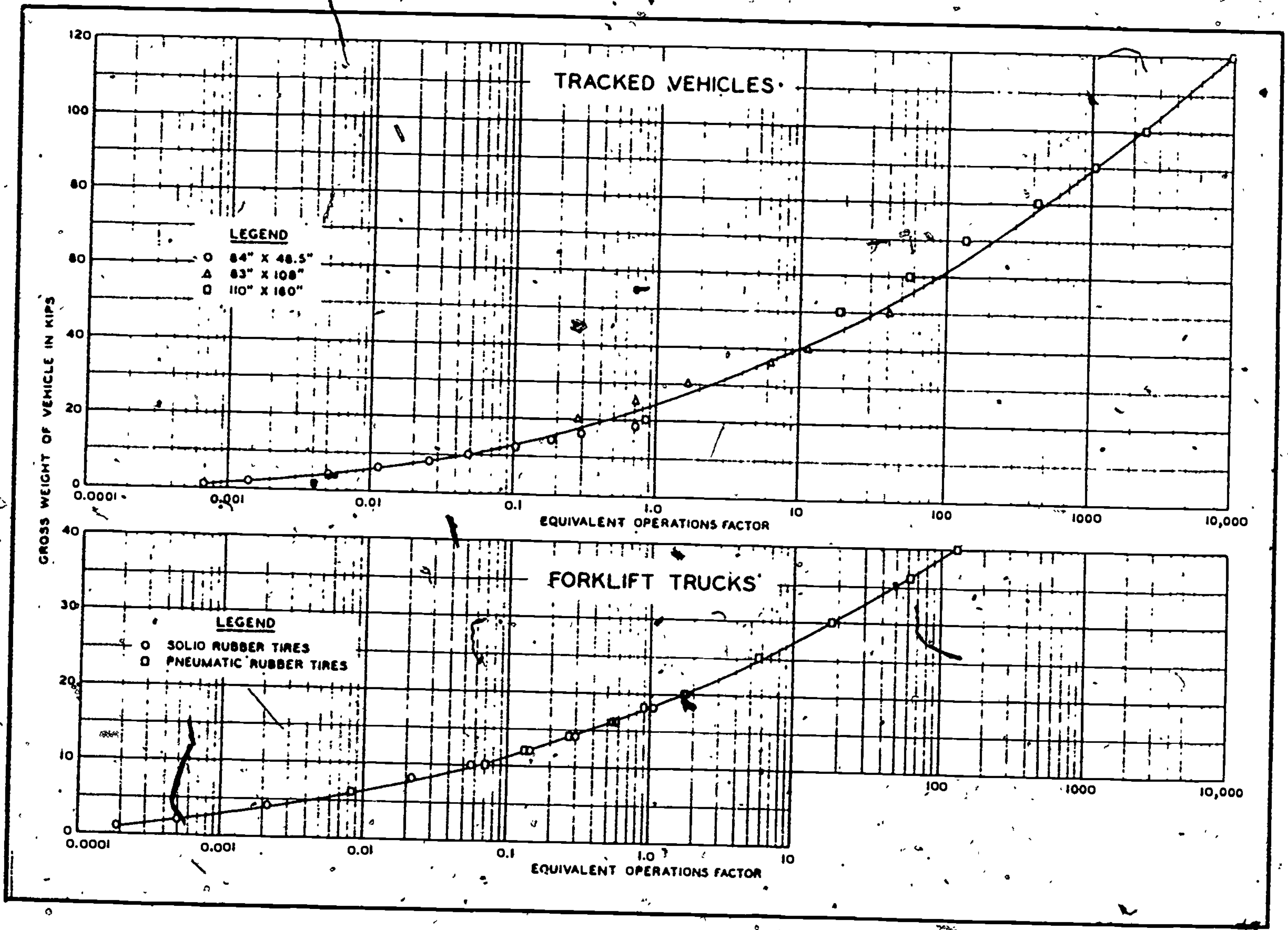


Figure 5-10. Equivalent operations factors for tracked vehicles and forklift trucks.

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Axle or gross load lb	Single axle operations per day	Tandem axle operations per day	Tracked vehicle operations per day
1,000	500	---	---
1,500	400	---	---
3,000	300	---	---
4,000	100	---	---
5,000	---	20	---
10,000	50	100	25
20,000	20	200	---
30,000	---	50	---
32,000	---	---	15
80,000	---	---	10

Axle or gross load lb*	Operation factor**		Operations per day		Equivalent operations per day
Single axle					
1,000	0.00016	×	500	=	0.08
1,500	0.0003	×	400	=	0.12
3,000	0.0012	×	300	=	0.36
4,000	0.0028	×	100	=	0.28
10,000	0.070	×	50	=	3.50
20,000	1.80	×	20	=	36.00
					40.34
Tandem axle					
5,000	0.0031	×	20	=	0.062
10,000	0.024	×	100	=	2.400
20,000	0.360	×	200	=	72.000
30,000	2.30	×	50	=	115.000
					189.462
Tracked vehicle					
10,000	0.05	×	25	=	1.25
32,000	2[illegible]0	×	15	=	40.50
80,000	480.00	×	10	=	4800.00
					4841.75
Total equivalent operations per day					5071.55

*Axle loads apply to wheeled vehicles. Gross loads apply to tracked vehicles and forklift trucks.

**Values read from figures 5-9 and 5-10.

Equivalent operations in the 2-year life: 365 days × 2 years × 5071 total equivalent operations = 3,701,830 or 3.7×10^6. (It can be seen that the equivalent operations per day for the single axle vehicles could have been disregarded without affecting the result. Note, however, the tremendous value obtained from tracked vehicles.)

(5) *Step 4.* The first step is to determine the amount of cover required over the CBR 10 subgrade. Entering figure 4-5 at 3.7×10^6 of equivalent operations, going vertically up to CBR 10 value, thence horizontally to the left, it is found that the CBR 10 material must have a cover of 13 inches. (See dotted line and arrows figure 4-5.) Table 4-2, "Recommended Minimum Thickness of Pavement and Base", indicates that the minimum thickness of pavement and base course worth CBR value of 50 with an anticipated traffic of 3.7×10^6 equivalent operations is 3½ inches of pavement a[illegible]4 inches of base course, which is in excess of cover requirement of 4 inches over the CBR 40 material. These values of thickness of base and pavement were determined as follows: since the equivalent operations of 3.7×10^6 (3,700,000) falls within the value of 7×10^5 (700,000) and 7×10^6 (7,000,000), line 5 of the above table is used. Entering line 5 and proceeding horizontally to the right and reading under the column of 50 CBR base, the values listed above are found. Figure 5-11 indicates the proposed design for the flexible pavement structure.

(6) **Alternate approach.** The most common type of bituminous wearing surface that can be expected to be constructed in the theater of operations is surface treatments. From experience in the use of surface treatments on base courses with CBR less than 80 it has been found that constant maintenance is a necessity. Assume now in the above example that pavement could not be placed due to lack of materials and equipment and surfacing would be limited to multiple surface treatment. The construction in this example problem is limited to the use of materials of CBR 40 subbase and CBR 50 base course. When figure 4-5 was used in above example it was found that a minimum of 3½ inches of pavement was required over the 50 CBR material. However, neither pavement material nor material better than CBR 50 is available so it is necessary to use the CBR 50 to fill this gap, and accept the fact that heavy maintenance will be required. The design for use of multiple surface treatment is as

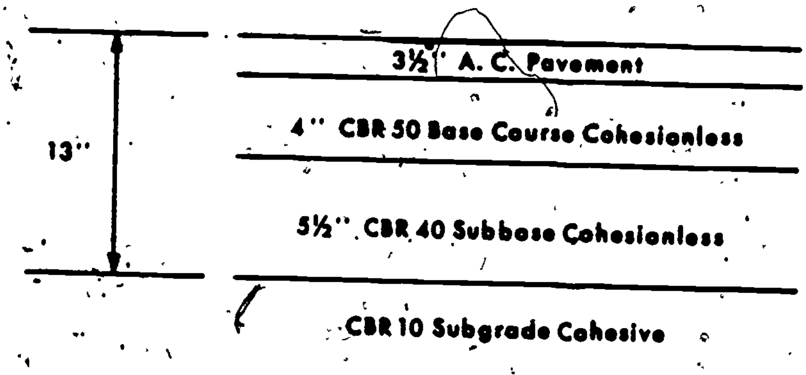

Figure 5-11. Thickness design.

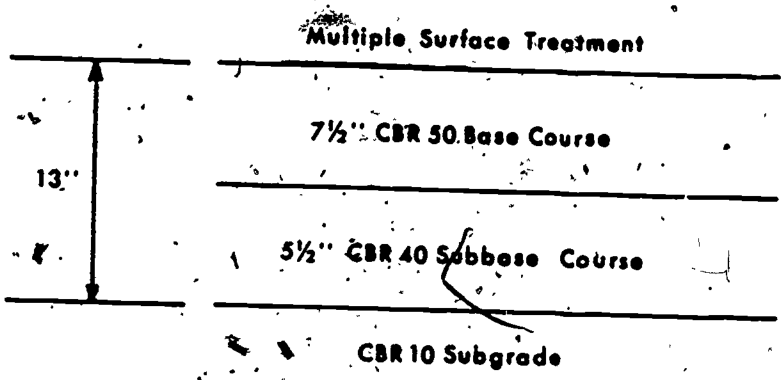

Figure 5-12. Thickness design (alternate approach).

shown in figure 5-12. Note that the design requirements also include the compaction requirements given in lesson 3.

5-7. SIMILARITIES OF DESIGN BETWEEN ROADS AND AIRFIELDS

The design of roads and airfields is based upon the California Bearing Ratio. The curves for thickness design using the CBR are almost identical for roads and airfields. These curves use as their criteria wheel configuration and tire contact area in both cases. In the case of airfields the wheel configuration and tire pressure are taken care of by specifying the type of aircraft. In roads, the different types of wheel configurations and tire pressures are changed to 18,000 pound single-axle dual-load equivalents and then applied to one set of curves. Once the design category has been determined for either basic design, it is then just a matter of entering the correct chart and reading the cover required. The cover required must be in correlation with the minimum base and pavement thickness and design loads set up in tables provided.

SELF TEST

Note: The following exercises comprise a self test. The figures following each question re-refer to a paragraph containing information related to the question. Write your answer in the space below the question. When you have finished answering all the questions for this lesson, compare your answers with those given for this lesson in the back of this booklet. Do not send in your solutions to these review exercises.

1. In any flexible pavement, the subgrade is the foundation which eventually must carry the load. In the design of a flexible pavement what is done to compensate for variations in the bearing strength of a subgrade? (5-1b)

2. The distribution of pressure depends to a large extent upon the wheel assembly. When are multiple-wheel assemblies most beneficial? (5-1c(1))

3. Explain the meaning of the two terms, "pavement" and "flexible pavement." (5-2)

4. Relatively cheap, locally available materials are frequently used as a course between the subgrade and the base course of a flexible pavement. If the material has a CBR above 20, what is it called? (5-3b(1))

5. Another course sometimes used between the subgrade and the base course is called the select material course. How is this soil normally classified under the Unified Soil Classification System? (5-3b(2)(a))

6. Select materials and subbases have certain recommended maximum permissable values of gradation and Atterberg limits. What is the maximum liquid limit of select material for airfields? (5-3b(3), table 5-1)

7. Explain briefly the purpose of a base course in flexible pavement construction. (5-3c(1))

8. In the tactical rear area certain types of airfields are constructed. What is their minimum design life? (5-4a(3))

9. The full design thickness of airfield pavement is called "full operational". What number of coverages will this thickness support? (5-4b)

10. The emergency classification for pavement thickness anticipates only a 2-week life. What percent of full operational is the pavement thickness? (5-4b, table 5-3)

11. Pavements are grouped into two traffic areas, types B and C. What is considered to be the type B traffic area? (5-4c(2))

12. Smoothness requirements for jet aircraft are extremely critical. What is the maximum recommended deviation (inch) from design grade? (5-4c(4))

Special situation. A design (exercises 13-15) is required for a rear area full operational airfield capable of handling C-133 aircraft (145.0 kips design gear load). Frost is not a problem. Tests indicate the CBR as: subgrade CBR 5, select material CBR 10, subbase CBR 40, base course CBR 80. The design of type C traffic areas will be used here. Use figure 5-6.

13. What is the (final design) thickness of pavement (inches)? (5-5a, b, fig 5-6, table 4-3)

14. What is the (final design) thickness of base course (inches)? (5-5a, b, fig 5-6, table 4-3)

15. If the minimum thickness of base course is 5 inches and the minimum thickness of pavement is 2 inches, what is the thickness of subbase (inches)? (5-5a, b, fig 5-6)

Special situation. You are to design a road for a 3-year life span on a subgrade with a CBR of 15. Two borrow pits are available, one with a CBR of 30 and the other with a CBR of 60. You have the equipment and material for a multiple surface treatment for a pavement. You have an estimate of the average daily volume of traffic (table 5-4).

TALBE 5-4. *For Use with Exercises 16 through 18*

Axle or gross load lbs	Single-axle operations per day	Tandem axle operations per day	Tracked vehicle operations per day
1,000	250	---	---
2,000	250	---	---
3,000	200	---	---
4,000	100	---	---
5,000	150	---	---
10,000	---	300	40
20,000	---	300	---
30,000	50	50	---
80,000	---	---	30

16. What is the single-axle total equivalent operations per day in 18,000-pound, single-axle, dual-wheel load equivalents? (5-6c, fig. 5-9, table 5-4)

17. What is the tandem-axle total equivalent operations per day in 18,000-pound, single-axle, dual-wheel load equivalents? (5-6c, fig 5-9, table 5-4)

18. What is the tracked vehicles total equivalent operations per day in 18,000-pound, single-axle, dual-wheel load equivalents? (5-6c, fig 5-10, table 5-4)

19. The previous total equivalent operations for the design life (3 years) in 18,000-pound, single-axle, dual-wheel load is assumed 9×10^{6}. The cover required above a CBR 15 subgrade is 10¼ inches. If the tracked vehicle (80,000 lb. gross load) is increased by 15 operations per day, what is the new cover required above a CBR 15 subgrade (inches)? (5-6c, d, figs 5-9, 5-10)

20. The previous thickness of base is assumed 7 inches and thickness of pavement is assumed 3 inches. A multiple surface treatment has to be placed due to lack of materials for a pavement. What is the new (final design) thickness of base course (inches)? 5-6e(6), figs 5-9, 5-10)

LESSON 6

FROST ACTION AND PERMAFROST

CREDIT HOURS ---------------------------- 2

TEXT ASSIGNMENT ---------------------- Attached memorandum.

MATERIAL REQUIRED -------------------- None.

LESSON OBJECTIVE ---------------------- Upon completion of this lesson you should be able to accomplish the following in the indicated topic areas:

1. **Principles of Frost Action.** Explain the theory of frost action, the conditions under which frost action is most likely to occur and the effect of degree days, freezing index, and water source upon the problem.

2. **Effects of Frost Action.** Describe the effect of frost action on the subgrade and upon rigid and flexible pavements.

3. **Counteractive Techniques and Design.** Determine the most effective counteractive measures that can be taken to reduce or prevent frost action damage.

4. **Permafrost.** Define permafrost and explain the special problems permafrost presents in construciton. Present accepted methods for meeting these problems.

ATTACHED MEMORANDUM

6-1. PRINCIPLES

Frost action refers to any process which, as a result of either freezing or thawing, affects the ability of the soil to support a structure. One of the most difficult problems related to frost action which the engineer encounters is that pavements are frequently broken up or severely damaged as subgrades freeze during the winter and thaw out in the spring. When frozen subgrades are thawing out, they may become extremely unstable. In some severely affected areas, it has been necessary to completely close a facility to traffic until the subgrade recovers its stability.

6-2. FROST ACTION PROCESS

a. **Freezing.** Early theories attributed frost heaves to the expansion of the water contained in soil voids upon freezing. The facts, however, indicate that heaving is due to the freezing of additional water that is attracted from the nonfrozen soil layers. The process of ice segregation may be pictured as follows: The thin layers of water adsorbed to soil grains (which do not act like water) become supercooled, meaning this water remains liquid below 32°F. A strong attraction exists between this water and the ice crystals which form in larger void spaces, and this supercooled water flows toward the already formed crystals and freezes on contact. Continued crystal growth leads to the formation of an ice lens which continues to grow in thickness and width until the source of water is cut off or the temperature rises above the normal freezing point (fig 6-1).

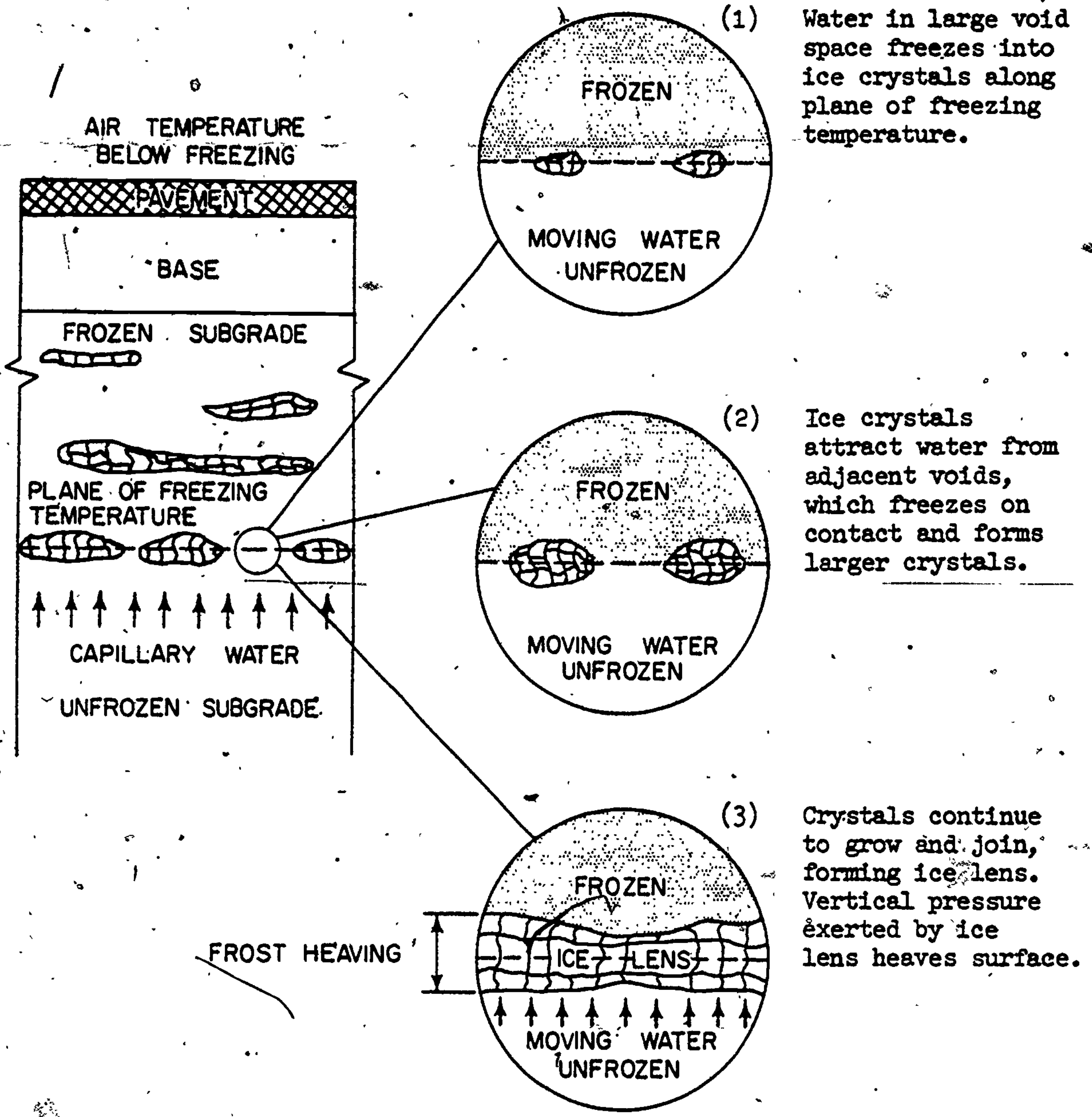

Figure 6-1. Formation of ice lenses.

b. **Thawing.** The second phase of frost damage occurs toward the end of winter, or in early spring when thawing commences. The frozen subgrade thaws both from the top and from the bottom. (1) If the air temperature remains barely below the freezing point for a sufficeint length of time, deeply frozen soils will gradually thaw from the bottom upwards because of the outward conduction of heat from the earth's interior. An insulating blanket of snow tends to encourage this type of thawing. Such a thawing condition is highly desirable, because it permits the melted water from the thawed ice lenses to seep back through the lower soil layers to the water table from whence it was drawn during the freezing process. Such dissipation of the melted water places no load on the surface drainage system, and no tendency exists to reduce subgrade stability by reason of saturation, thus, there is little difficulty in maintaining unpaved roads in a passable condition. (2) Thawing occurs from the top downward if the air temperature suddenly rises from below the freezing point to well above that point and remains there an appreciable time. It leaves a frozen layer beneath the thawed subgrade. The thawed soil between the pavement and this frozen layer contains an excess amount of moisture resulting from the melting of the ice which it contained. Since the frozen layer of soil is impervious to the water, adequate drainage is almost impossible. The poor stability of the resulting supersaturated road and airfield subgrades accounts for many pavement failures. Earth roads may become impassable when supersaturated. (3) Thawing from the top and bottom occurs when the air temperature remains barely above the freezing point for a sufficient time. Such thawing results in reduced soil stability, the duration of which, however, would be less than for the case where thaw is only from the top downward.

6-3. CONDITIONS NECESSARY FOR FROST ACTION

a. **Frost-susceptible soil.** The potential intensity of ice segregation in a soil is dependent to a large degree on the void sizes which may be expressed as an empirical function of grain size. Thus, inorganic soils with 3% or more of grains by weight finer than 0.02 millimeter in diameter are considered frost susceptible. Frost-susceptible soils have been classified in the following four groups: F-1, F-2, F-3, and F-4, (table 6-1), listed approximately in the order of increasing susceptibility to frost heaving and/or weakening as a result of frost melting. The order of listing of subgroups under groups F-3 and F-4 does not necessarily indicate the order of susceptibility to frost heaving or weakening of these subgroups. There is some overlapping of frost susceptibility between groups. The soils in group F-4 are of especially high frost susceptibility. Soil names are defined in the Unified Soil Classification System.

TABLE 6-1. *Class of Frost-Susceptable Soils*

Frost Group	Kind of soil	Percentage finer than 0.02 mm. by weight	Typical soil types under Unified Soil Classification Systems
F-1	Gravelly soils	3 to 10	GW, GP, GW-GM, GP-GM
F-2	(a) Gravelly soils	10 to 20	GM, GW-GM, GP-GM
	(b) Sands	3 to 15	SW, SP, SM, SW-SM, SP-SM
F-3	(a) Gravelly soils	Over 20	GM, GC
	(b) Sands, except very fine silty sands	Over 15	SM, SC
	(c) Clays, PI > 12	—	CL, CH
F-4	(a) All silts	—	ML, MH
	(b) Very fine silty sands	Over 15	SM
	(c) Clays, PI < 12	—	CL, CL-ML
	(d) Varved clays and other fine-grained banded sediments	—	CL and ML; CL, ML, and SM; CL, CH, and ML; CL, CH, ML, and SM

b. **Freezing temperatures.** Temperatures below 32°F must penetrate the soil. In general, the thickness of ice layers (and amount of consequent heaving) is inversely proportional to the rate of penetration of freezing temperature into the soil. Thus, winters with fluctuating air temperatures at the beginning of the freezing season will produce more damaging heaves than extremely cold, harsh winters where the water is more likely to be frozen in place before ice segregation can take place. The daily average temperature is commonly obtained by averaging the maximum and minimum temperatures for one day, but in some cases by averaging several temperature readings taken at equal time intervals during the day, generally hourly. The degree days for any one day equal the difference between the average daily air temperature and 32°F. The degree days are minus when the average daily temperature is below 32°F (freezing degree days) and plus when above (thawing degree days). Figure 6-2 shows a typical curve for a specific winter obtained by plotting cumulative degree days against time. The freezing index is the number of degree days between highest and lowest points on a curve of cumulative degree days versus time for one freezing season. It is used as a measure of the combined duration and magnitude of below freezing temperatures occurring during any given freezing season. The index determined for air temperatures at 4.5 feet above the ground is commonly designated as the air freezing index, while that determined for temperature immediately below a surface is known as the surface freezing index. The freezing index determined for a specific winter is illustrated in figure 6-2.

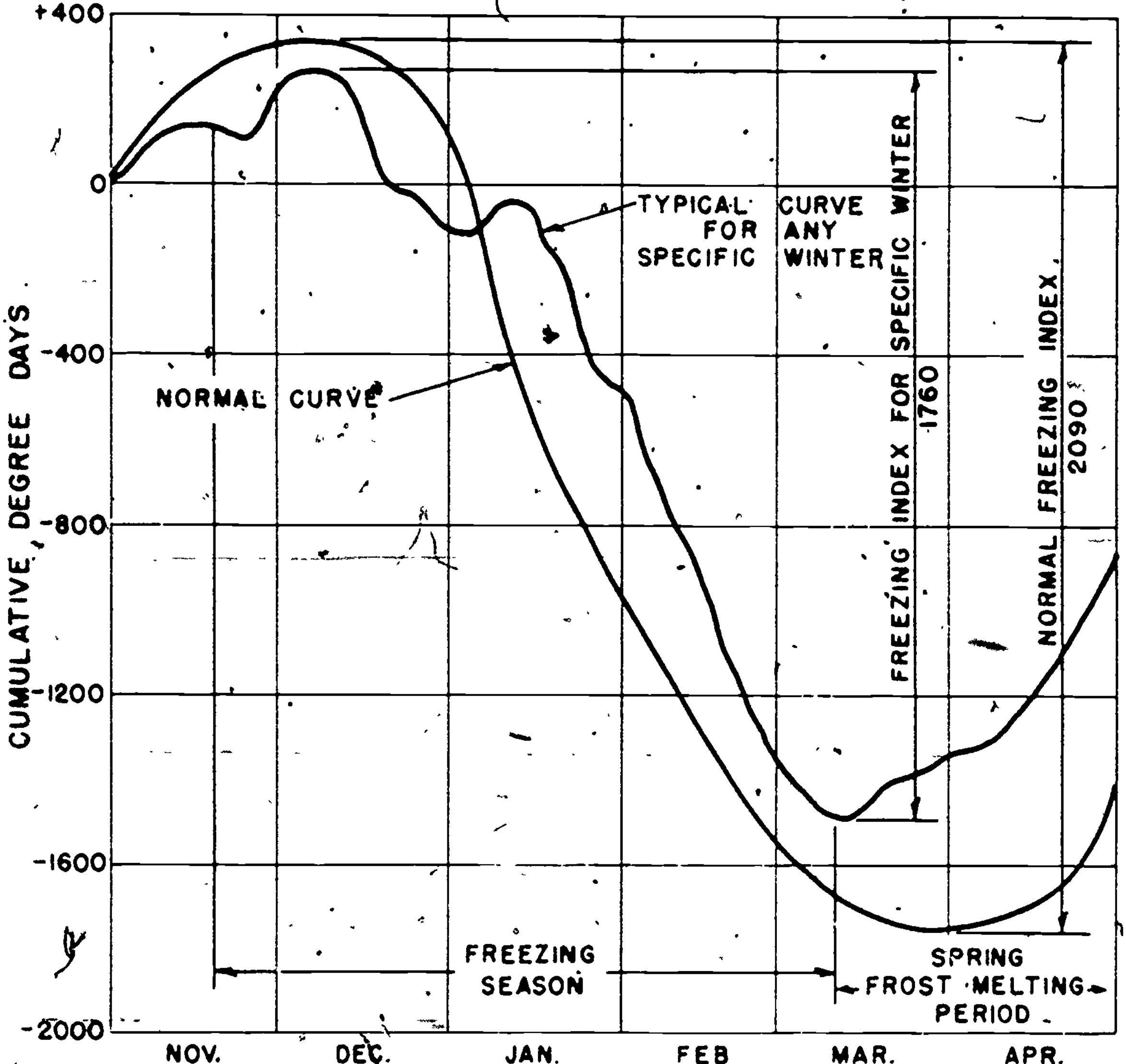

Figure 6-2. Determination of freezing index.

c. Depth of frost penetration. The depth to which freezing temperatures penetrate below the surface of a pavement depends principally on the magnitude and duration of below freezing air temperatures and on the amount of water which is subject to being frozen in the subgrade. A relationship between air-freezing index and depth of frost penetration is shown in figures 6-3 and 6-4.

d. **Water.** A source of water must be available to promote the accumulation of ice lenses. Examples are: from a high ground water table; from a capillary supply from an adjoining water table; from infiltration at the surface; from a water bearing system (aquifer); or from the water held in the voids of the fine-grained soils. A potentially troublesome water supply for ice segregation is present if the highest ground water at any time of the year is within 5 feet of the proposed subgrade surface or the top of any frost susceptible base materials. When the depth to the uppermost water table is in excess of 10 feet throughout the year, a source of water for substantial ice segregation is usually not present. In addition to the conditions stated above, it is necessary to consider all reliable information concerning past frost heaving and performance during the frost melting period of airfield and highway pavements constructed in the area being investigated, with a view toward modifying or increasing the frost design requirements.

6-4. DETRIMENTAL EFFECTS OF FROST ACTION

a. **Heaving.** Frost heave, indicated by the raising of the pavement, is directly associated with ice segregation and is visible evidence on the surface that ice lenses have formed in the subgrade and/or base material. Conditions conducive to uniform heave may exist, for example, in a section of pavement constructed with a fairly uniform stripping or fill depth, uniform depth to ground water table, and uniform soil characteristics. Conditions conducive to irregular heave occur typically at locations where subgrades vary between clean sand and silty soils, or at abrupt transitions from cut to fill sections with ground water close to surface. Lateral drains, culverts, or utility lines placed under pavements on frost-susceptible subgrades frequently cause abrupt differential heaving. Wherever possible the placing of such facilities beneath pavements should be avoided, or transitions should be provided so as to moderate the roughening of the pavement during the period of heave.

b. **Loss of strength of subgrade pavement.** When ice segregation has occurred in a frost-susceptible soil, reduction of its strength with a corresponding reduction in load supporting capacity of the pavement develops during prolonged frost-melting periods. This occurs particularly in the spring, because the melting of the ice from the surface downward releases an excess of water which cannot drain through the still-frozen soil below and in the shoulders, or redistribute itself readily, thus softening the soil. Supporting capacity may be reduced in clay subgrades even though significant heave has not occurred because water for ice segregation is extracted from the voids of the uniform clay below, and the resulting shrinkage of the latter largely balances the volume of the ice lenses formed. Further, traffic may cause remolding or develop hydrostatic pressure within the pores of the soil during the period of weakening, thus resulting in further reduced subgrade strength. The degree to which a soil loses strength during a frost-melting period and the length of the period during which the strength of the soil is reduced depends on the type of soil, temperature conditions during freezing and thawing periods, the amount and type of traffic during the frost-melting periods, the availability of water during the freezing and thawing periods, and drainage conditions.

c. **Effects of frost action on pavements.**

(1) **Rigid concrete pavements.** Concrete has little tensile strength and a slab is designed to resist loads from above while receiving uniform support from the subgrade and base course. Hence, slabs have a tendency to break up as a result of the upthrust from nonuniform heaving soils causing point bearing. As a rule, if rigid pavements survive the

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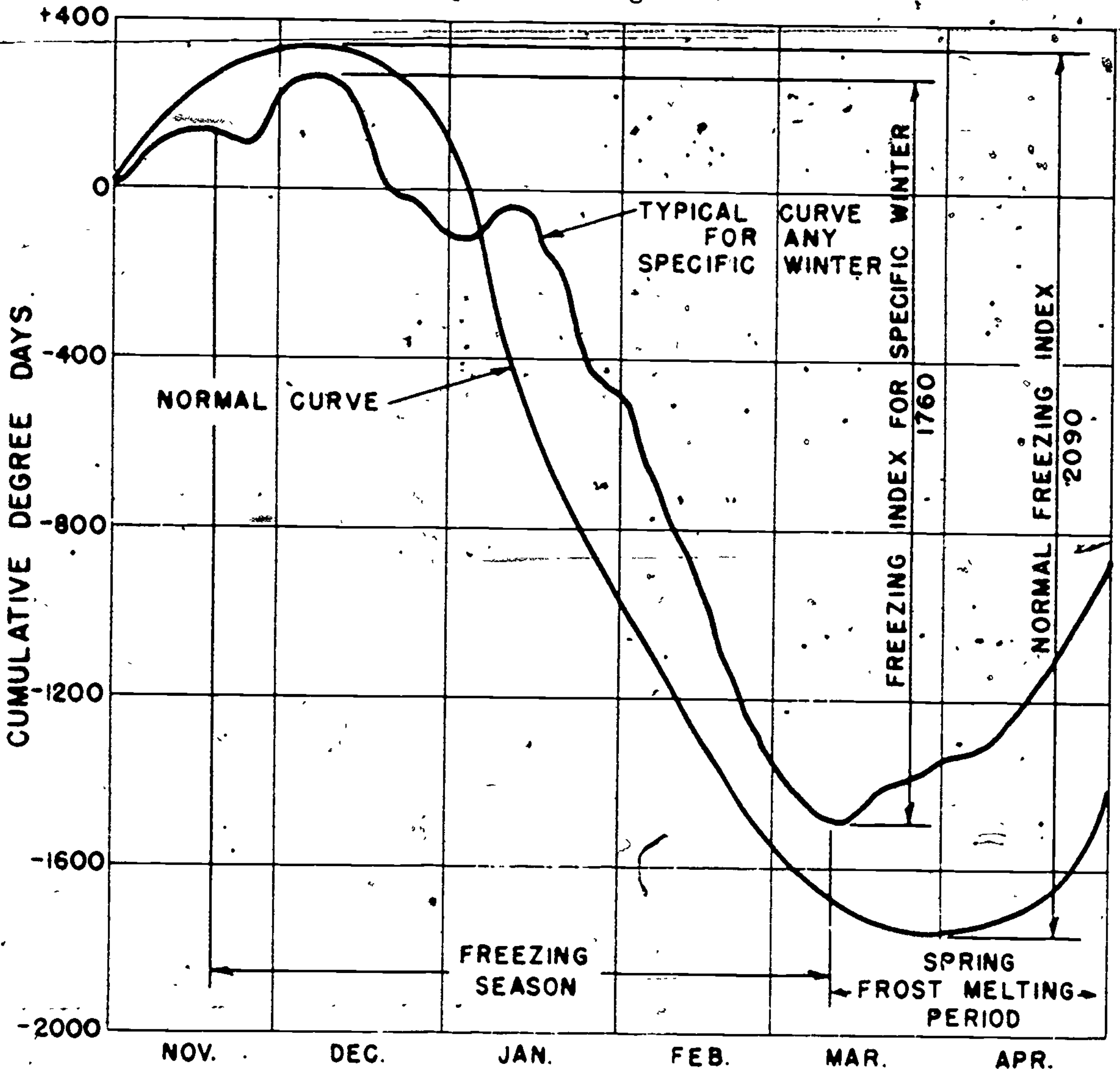

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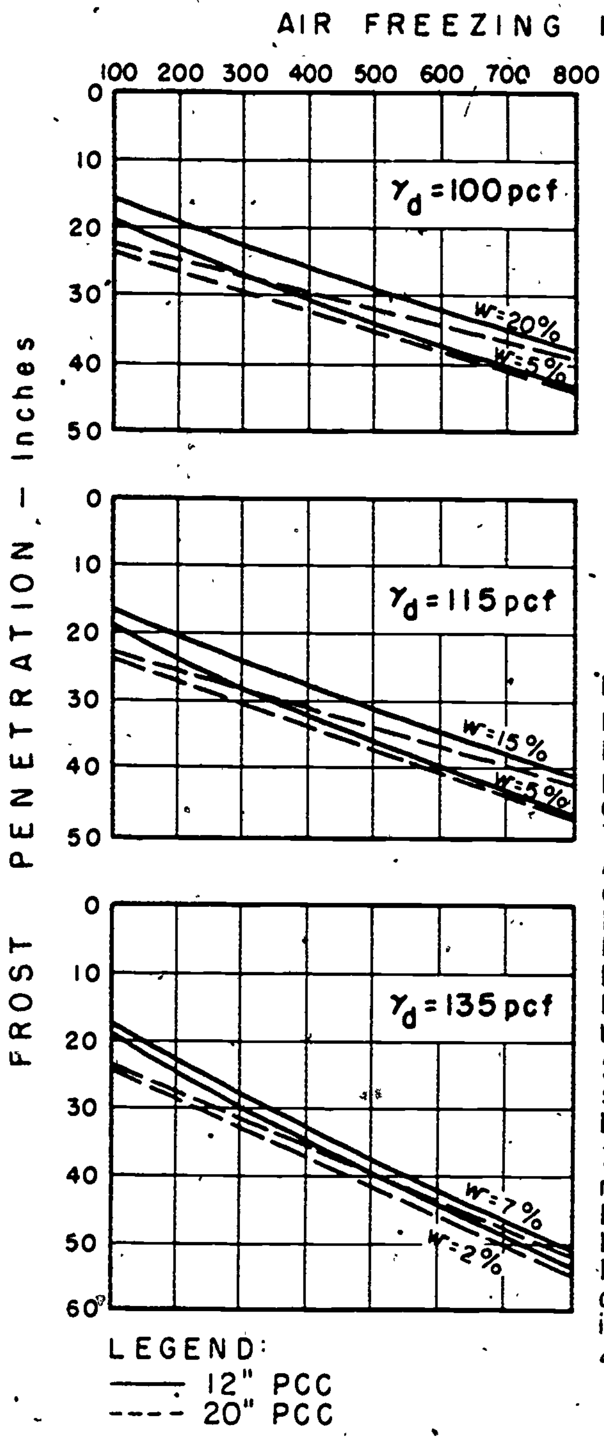
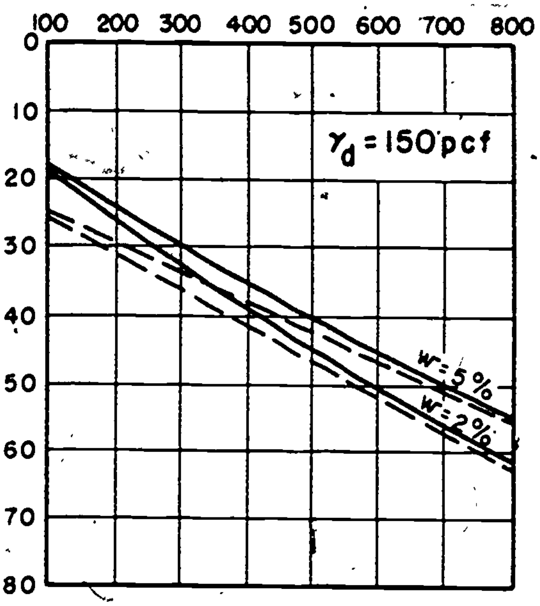




NOTES:

1. Frost penetration depths based on Modified Berggren Equation and computation procedures outlined in the following reports:
   a. "Analytical Studies of Freezing and Thawing of Soils", H.P. Aldrich and H.M. Paynter, Arctic Construction and Frost Effects Laboratory, Corps of Engineers, U.S. Army, June 1953.
   b. "Frost Penetration in Multi layer Soil Profiles" MIT Dept. of Civil and Sanitary Engineering, Soil Engineering Division., Arctic Construction and Frost Effects Laboratory, Corps of Engineers, U.S. Army, June 1957.
2. It was assumed in computations that all of soil moisture freezes when soil is cooled below 32°F.
3. Frost penetration depths shown are measured from pavement surface. Depths for 12" PCC pavements are good approximations for bituminous pavements over 6 to 9 in. high quality base. For a given locality, depths may be computed with the Modified Beggren Equation if necessary data are available.
4. γ_d = dry unit weight.
   w = moisture content in percent based on dry unit weight.

Figure 6-3. Relationships between air freezing index and frost penetration into granular non-frost-susceptible soil beneath pavements kept free of snow and ice for freezing indexes below 800.

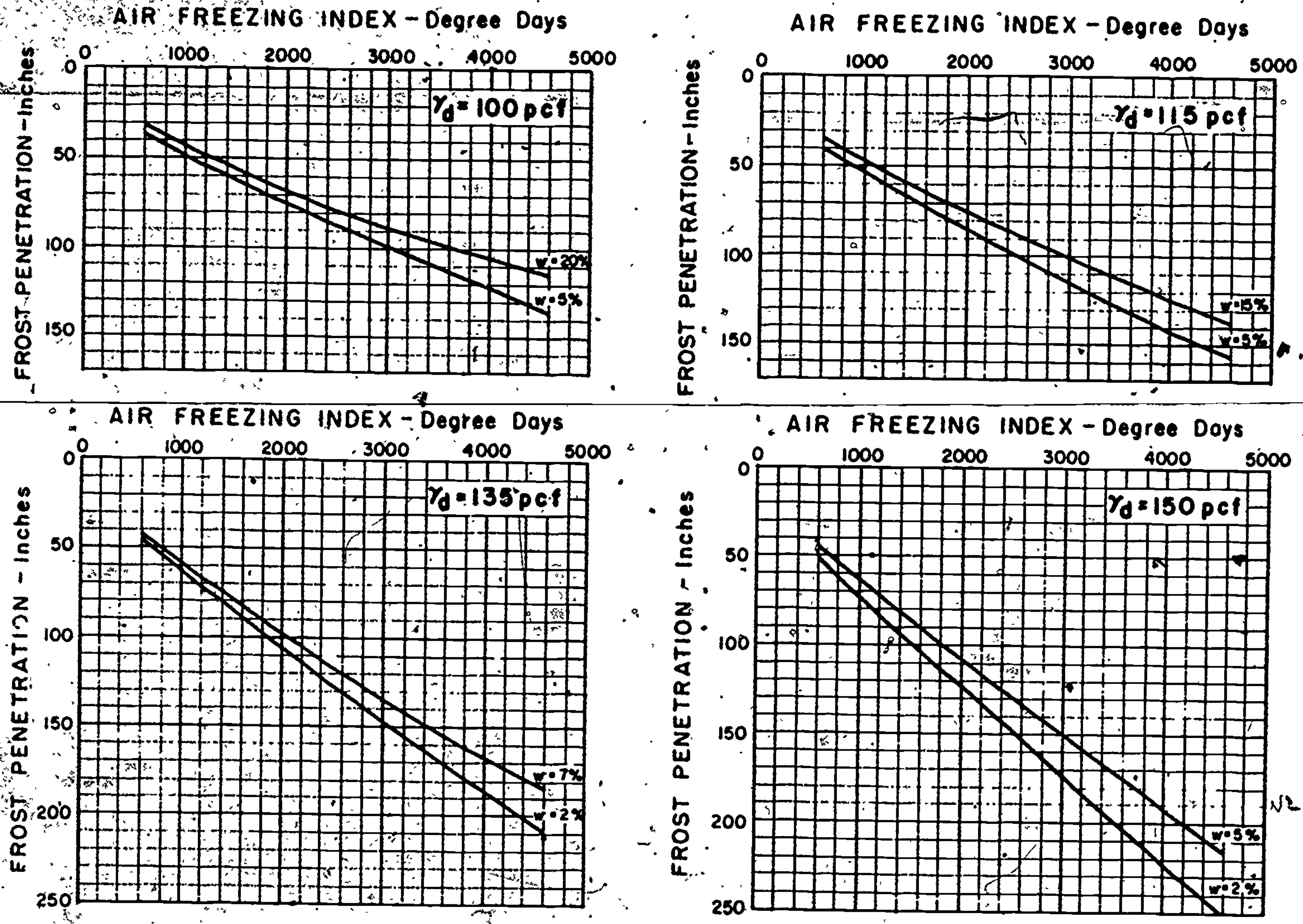


Figure 6-4. Relationships between air freezing index and frost penetration into granular non-frost-susceptible soil beneath pavements kept free of snow and ice for freezing indexes above 800.

ill effects of upheaval, they will generally not fail during thawing. Reinforced concrete will carry a load by beam action over a subgrade having both frozen and supersaturated areas. The average rigid pavement will carry a limited amount of traffic, but less than the design load, over an area which has become entirely supersaturated and semiliquid.

(2) Flexible bituminous pavements. The ductility of flexible pavements helps them to deflect with heaving and later resume their original positions. While heaving may produce severe bumps and cracks, generally it is not too serious for flexible pavements. By contrast, a load applied to a poorly supported flexible pavement during the thawing period will usually produce small mounds called frost boils, at the weak spots in the pavement. Sustained traffic over areas afflicted with frost boils sets up a pumping action that results in complete failure of the road in the immediate vicinity of the frost boil.

(3) **Slopes.** Exposed back slopes and side slopes of cuts and fills in fine grained soil have a tendency to slough off during the thawing process. The additional weight of water plus the soil exceeds the shearing strength of the soil and the hydrostatic head of water exerts the greatest pressure at the foot of the slope. This causes a sloughing off of the soil at the toe, multiplying the failure by consecutive shear failure due to inadequate stability of the altered slopes.

6-5. COUNTERACTIVE TECHNIQUE

a. **Lowering water table.** Every effort should be made to lower the ground water table in relation to the grade of the road or runway. This may be accomplished by the installation of subsurface drains, or open side ditches, provided suitable outlets are available and provided also that the subgrade soil is drainable; or it may be accomplished by raising the grade line in relation to the water table. Whatever means are employed for producing the condition, the distance from the top of the proposed subgrade surface (or any frost-susceptible base material used) to the highest probable elevation of the water table should not be less than 10 feet. When the depth to the highest level of the water table is in excess of 10 feet throughout the year, a source of water for appreciable ice segregation is usually not present.

b. **Preventing upward water movement.** Treatments which successfully prevent the rise of water include placing a 6-inch layer of pervious, coarse grained soil 2 or 3 feet beneath the surface. This layer must be designed as a filter to prevent clogging the pores with finer material and subsequent defeat of the original purpose. If the depth of frost penetration is not too great, it may be cheaper to backfill completely with granular material. Another method, successful though expensive, is to excavate to the frost line, lay prefabricated bituminous surfacing (PBS), and backfill with granular material. In some cases, soil-cement and asphalt stabilized mixtures, 6 inches thick, have been used effectively to cut off the upward movement of water.

c. **Removal of frost susceptible soil.** Even though the site selected may be on ideal soil, invariably on long stretches of roads, or on wide expanses of runways, there will be localized areas subject to frost action. These must be recognized, removed, and replaced with select granular material. Unless this is meticulously carried out, differential heaving or frost boils, upon thawing, may result.

d. **Insulating the subgrade against frost.** The most generally accepted method of preventing subgrade failure due to frost action is to provide a suitable insulating cover so that freezing temperatures do not penetrate the subgrade to any significant depth. This insulating cover consists of a suitable thick pavement and a thick non-frost-susceptible base course.

e. **Snow removal.** During freezing weather, if the wearing surface is cleared of snow, it is very important that the shoulders also be kept free of snow. Where this is not the case, freezing will set in first beneath the wearing surface. This permits water to be drawn into, and accumulated in, the subgrade from the unfrozen shoulder area which is protected by the insulating snow. If both areas are free of snow, then freezing will begin in the shoulder areas because it is not protected by a pavement. Under this condition water is drawn from the subgrade to the shoulder area. As freezing progresses to include the subgrade, there will be little frost action unless more water is available from ground water or seepage.

6-6. FROST ACTION DESIGN

a. Limited subgrade frost penetration design:

Step 1. Determine the freezing index. (If in the arctic or subarctic regions also determine the thawing index.)

Step 2. Conduct combined mechanical analysis and strength determination of subgrade and borrow soils and establish heave characteristics of the natural subgrade soil, locate depth of the ground water table and any other necessary characteristics.

Step 3. Determine design load, tire contact area or pressure, gear configuration and wheel spacing, and any other classification characteristics of the type field required.

Step 4. Classify subgrade soil into group of frost susceptibility (F-1 through F-4).

Step 5. Estimate or compute the average moisture contents in the granular non-frost-susceptible base course material and the natural subgrade soil at the start of the freezing period. Also determine the dry unit weight of the granular non-frost-susceptible base course material at maximum modified AASHO compactive efforts.

Step 6. Determine the total frost penetration which would occur in a granular non-frost-susceptible base course material, assumed of unlimited depth in the design freezing index year, figures 6-3 and 6-4. Use straight line interpolation where necessary. The solid moisture content lines shown on the charts represent 12 inches of portland cement concrete pavement or a bituminous pavement over 6 to 9 inches of high quality non-frost-susceptible base course material. The dotted moisture content lines represent 20 inches of portland cement concrete pavement. The minimum thickness of asphaltic concrete pavement and high quality base course materials required in a temperate climate are tabulated on tables 6-2 and 6-3.

Step 7. Get familiar with the pavement cross section on upper left of figure 6-5, and then compute the base course thickness (c) required for zero frost penetration into the subgrade soil using a pavement thickness (p) as required by the tabulations on tables 6-1 and 6-2 as follows: $c = a - p$

Where:

a = the total thickness of pavement and non-frost-susceptible base found in step 6 above.

p = the minimum required thickness of asphaltic concrete pavement tabulated on tables 6-1 and 6-2.

Step 8. Compute the ratio (r) of the in-place subgrade moisture content to the moisture content of the non-frost-susceptible base course material after drainage as follows:

$$r = \frac{\text{moisture content in percent of the natural subgrade}}{\text{moisture content in percent of the drained base course}}$$

TABLE 6-2. *Pavement and Base Thickness Design Criteria for Single-Wheel Loads*

Single wheel
Tire inflation, 100 psi

LOAD (KIPS)	THICKNESS, IN.					
	80-CBR			100-CBR		
	Pavement	Base	Total	Pavement	Base	Total
10	2	6	8	2	6	8
20	2	6	8	2	6	8
30	3	6	9	2	6	8
40	3	6	9	2	6	8
50	3	6	9	2	6	8
60	4	6	10	3	6	9
70	4	6	10	3	6	9

Single wheel
Contact area, 100 sq in. each wheel

LOAD (KIPS)	THICKNESS, IN.					
	80-CBR			100-CBR		
	Pavement	Base	Total	Pavement	Base	Total
10	2	6	8	2	6	8
15	3	6	9	2	6	8
20	3	6	9	3	6	9
25	4	6	10	3	6	9
30	5	6	11	4	6	10

It is important to note that:

If the computed "r" value above exceeds 2.0 use 2.0 for r. The reason for limiting r to a maximum of 2.0 is that not all of the moisture content in a fine grained soil will actually freeze at the temperature which will be reached in the portion of the subgrade into which freezing temperatures will penetrate.

Step 9. Enter figure 6-5 using the c value, as computed in step 7 above, as abscissa and progressing vertically until intercepting the computed or applicable r value, then progress horizontally to both left and right scales giving you a direct reading of the design base thickness b and allowable subgrade frost penetration s respectively. This is the design for "limited subgrade frost penetration". A double check should be made using figure 6-6 of values b and s, as this plot illustrates the basic subgrade penetration assumption on which this whole design procedure is based.

TABLE 6-3. ***Pavement and Base Thickness Design Criteria for Multiple-Wheel Loads***

Heavy Load Design--Assembly Load, 265 kips
TWIN-TWIN ASSEMBLY, BICYCLE
SPACING, 37-62-37 IN. CENTER-TO-CENTER
CONTACT AREA, 267 SQ IN. EACH WHEEL

TRAFFIC AREA	MINIMUM THICKNESS, IN.[1]					
	100-CBR BASE			80-CBR BASE[2]		
	PAV	BASE	TOTAL	PAV	BASE	TOTAL
A	5	10	15	6	9	15
B	4	9	13	5	8	13
C	4	9	13	5	8	13
D	3	6	9	3	6	9
Hangar floors and access aprons[3]	3	6	9	3	6	9

TWIN-TWIN ASSEMBLY, BICYCLE
SPACING, 37-62-37 IN. CENTER-TO-CENTER
CONTACT AREA, 267 SQ IN. EACH WHEEL

LOAD KIPS	MINIMUM THICKNESS, IN.[1]											
	TYPES B AND C TRAFFIC AREAS						TYPE A TRAFFIC AREAS					
	100-CBR BASE			80-CBR BASE[2]			100-CBR BASE			80-CBR BASE[2]		
	PAV	BASE	TOTAL	PAV	BASE	TOTAL	PAV	BASE	TOTAL	PAV	BASE	TOTAL
160	3	6	9	3	6	9	3	8	11	4	8	12
200	3	7	10	4	6	10	4	8	12	5	8	13
230	4	8	12	5	7	12	5	8	13	6	8	14
265	4	9	13	5	8	13	5	10	15	6	9	15
300	5	9	14	6	8	14	6	10	16	7	9	16
330	6	10	16	7	9	16	7	11	18	8	10	18

TWIN ASSEMBLY, BICYCLE
SPACING, 37 IN. CENTER-TO-CENTER
CONTACT AREA, 267 SQ IN. EACH WHEEL

LOAD KIPS	MINIMUM THICKNESS, IN.[1]											
	TYPES B AND C TRAFFIC AREAS						TYPE A TRAFFIC AREAS					
	100-CBR BASE			80-CBR BASE[2]			100-CBR BASE			80-CBR BASE[2]		
	PAV	BASE	TOTAL	PAV	BASE	TOTAL	PAV	BASE	TOTAL	PAV	BASE	TOTAL
50	3	6	9	3	6	9	3	8	11	3	8	11
75	3	6	9	3	6	9	3	8	11	4	8	12
100	3	6	9	4	6	10	4	8	12	5	8	13
125	4	8	12	5	7	12	5	9	14	6	8	14
150	5	9	14	6	8	14	6	10	16	7	9	16

TWIN-TANDEM ASSEMBLY, TRICYCLE
SPACING, 31-63 IN. CENTER-TO-CENTER
CONTACT AREA, 267 SQ IN. EACH WHEEL

LOAD KIPS	MINIMUM THICKNESS, IN.[1]											
	TYPES B AND C TRAFFIC AREAS						TYPE A TRAFFIC AREAS					
	100-CBR BASE			80-CBR BASE[4]			100-CBR BASE			80-CBR BASE[2]		
	PAV	BASE	TOTAL	PAV	BASE	TOTAL	PAV	BASE	TOTAL	PAV	BASE	TOTAL
[illegible]	2	6	8	3	6	9	2	6	8	3	6	9
[illegible]	3	6	9	3	6	9	3	6	9	3	6	9
135	3	6	9	3	6	9	3	6	9	3	6	9
150	3	6	9	3	6	9	3	6	9	4	6	10
175	3	6	9	3	6	9	3	6	9	4	6	10

TWIN ASSEMBLY, TRICYCLE
SPACING, 37 IN. CENTER-TO-CENTER
CONTACT AREA, 267 SQ IN. EACH WHEEL

LOAD KIPS	MINIMUM THICKNESS, IN.[1]											
	TYPES B AND C TRAFFIC AREAS						TYPE A TRAFFIC AREAS					
	100-CBR BASE			80-CBR BASE[4]			100-CBR BASE			80-CBR BASE[2]		
	PAV	BASE	TOTAL	PAV	BASE	TOTAL	PAV	BASE	TOTAL	PAV	BASE	TOTAL
40	2	6	8	2	6	8	2	6	8	3	6	9
60	3	6	9	3	6	9	3	6	9	3	6	9
80	3	6	9	3	6	9	3	6	9	4	6	10
100	3	6	9	4	6	10	4	6	10	5	6	11
120	3	7	10	4	6	10	5	7	12	6	6	12

1. These minimum thicknesses apply when layer directly under the base course has a design CBR of not more than 50; when the underlying layer has a design CBR of 80, the minimum thickness of base course shall be 6 inches.
2. Restricted to Florida limerock except that stabilized aggregate will be permitted in Type D traffic areas.
3. Applicable in other than cold climates.
4. Florida limerock or stabilized aggregate permitted.

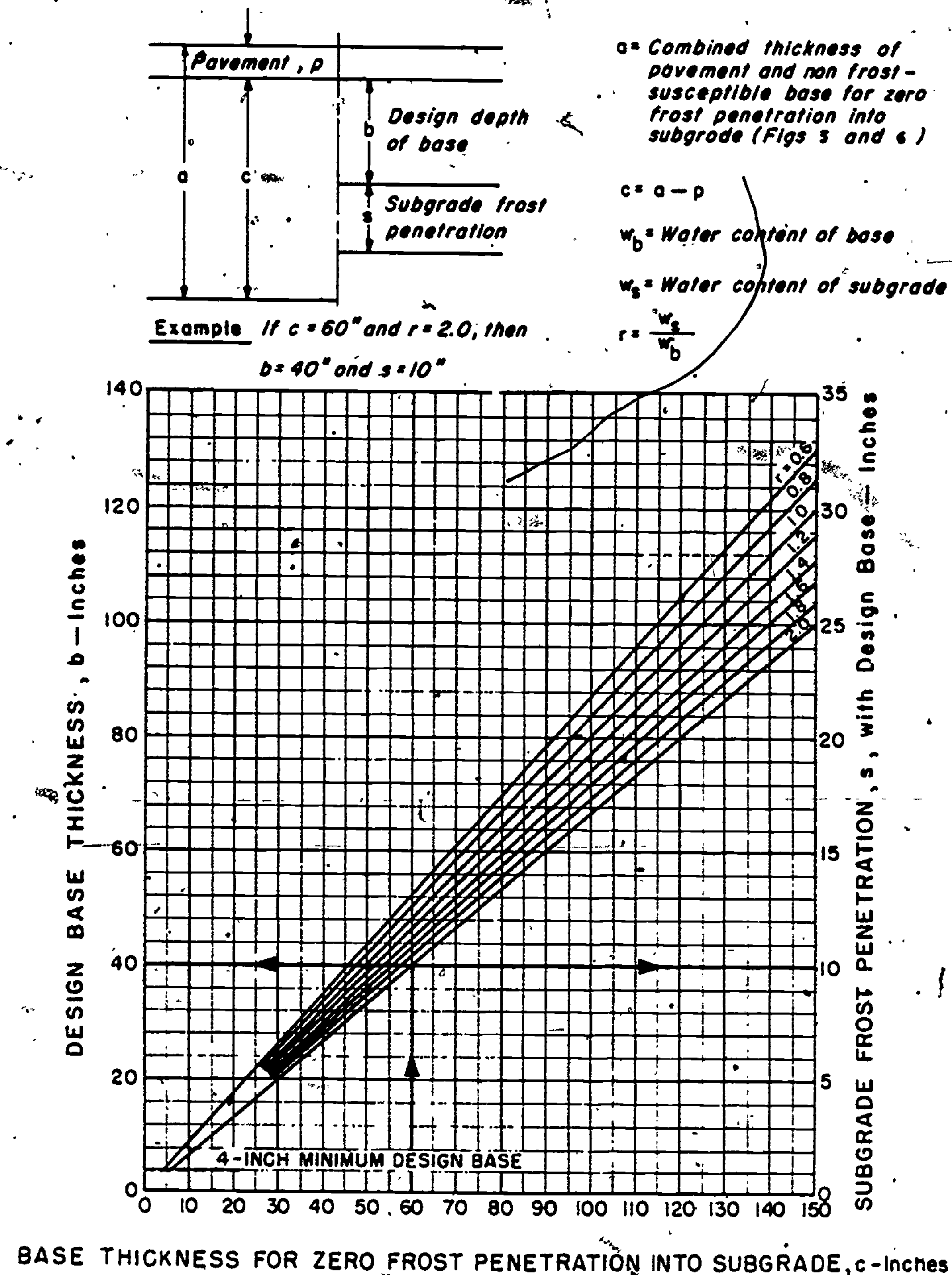


Figure 6-5. Design depth of non-frost-susceptible base for limited subgrade frost penetration.

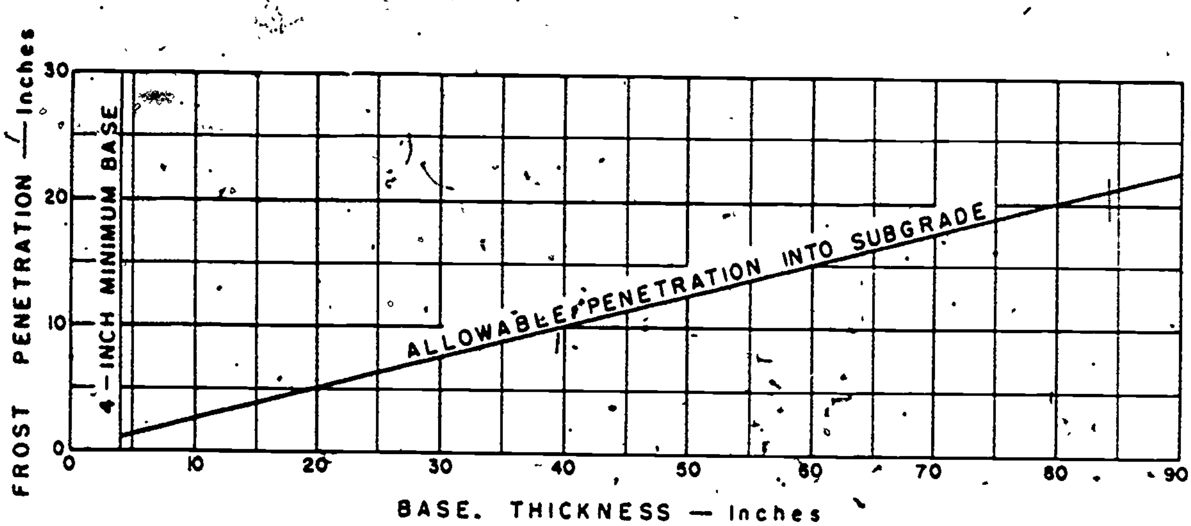


Figure 6-6. Allowable subgrade frost penetration in design freezing index year.

It is important to note that:

(a) Only this method of frost design may be used to determine the combined thickness of both rigid and flexible pavements and non-frost-susceptible base in the following cases:

1. Over all group F-4 subgrade soils.

2. Over other frost-susceptible subgrade soils, when cracking of rigid pavements or unacceptable pavement roughness caused by nonuniform frost heave may be expected, with lesser design thicknesses.

(b) The limited subgrade frost penetration design should always be investigated over other groups of frost-susceptible subgrade soils as well and should be selected for use based on economy of design.

(c) Under average field conditions, the above procedures will result in sufficient thickness of material between the frost-susceptible subgrade and the pavement so that frost penetration of the amount s **should not result in excessive differential heave** and cracking of the pavement surface during the design freezing index year.

Step 10. Design the bottom 4 inches of the design base as a filter material satisfying both of the following conditions:

$$\frac{\text{15 percent passing size of filter blanket}}{\text{85 percent passing size of subgrade soil}} \lessapprox 5$$

and

$$\frac{\text{50 percent passing size of filter blanket}}{\text{50 percent passing size of subgrade soil}} \lessapprox 25$$

It is important to note that in the above case, the filter blanket prevents the frost-susceptible subgrade soil from penetrating through the filter; however, the filter material itself must also not penetrate the non-frost-susceptible base course material. Therefore, the following additional requirement

$$\frac{\text{15 percent passing size of base course}}{\text{85 percent passing size of filter blanket}} \leqq 5$$

and

$$\frac{\text{50 percent passing size of base course}}{\text{50 percent passing size of filter blanket}} \leqq 25$$

Where: $\leqq$ means equal to or smaller than

In addition to the above two requirements, the filter material will, in all cases, have less than 3 percent by weight finer than 0.02 millimeter.

For rigid pavements, the 85 percent size of filter or regular base course material placed directly beneath the pavements shall be equal to or greater than ¼ inch in diameter. This requirement prevents loss of support by the pumping of soil through the joints of the rigid pavements.

Step 11. When the maximum combined thickness of pavement and base obtained by this procedure exceeds 72 inches, special study should be made of the following, possibly more economical, alternatives:

(a) Limit the total combined thickness of pavement and non-frost-susceptible base to 72 inches and use steel reinforcement to minimize and limit the cracking that will occur.

(b) Reduce the slab dimensions to as little as 12.5 or 15 feet without reinforcement.

It is important to note that both of the above alternatives will produce greater surface roughness than obtained using the basic design method because of greater subgrade frost penetration.

b. Reduce subgrade strength design.

Step 1. Investigate the reduced subgrade strength design for both flexible and rigid pavements on subgrade soils of groups F-1, F-2, and F-3 when subgrade conditions are sufficiently uniform to assure that objectionable differential heaving or unacceptable cracking of pavements will not occur or where subgrade variations are correctable to this condition. On less important slow speed flexible pavements this design may be used over nonuniform heaving and F-4 subgrade soils.

Step 2. Use the applicable charts, figures 6-7 through 6-12; enter the charts with the design load on the bottom horizontal scale; progress vertically up to intersect the appropriate soil group (F-1, F-2, or F-3); then progress horizontally to your left and read directly the minimum combined thickness of pavement and non-frost-susceptible base course material required above this subgrade in inches. It is important to note that:

(a) In no case should the thickness of pavement and non-frost-susceptible base be less than 9 inches where frost action is a consideration, regardless of loading.

(b) Figures 6-7 through 6-11 state that the thickness will be reduced 10 percent for the runway interior as follows:

1. Light load pavement and theater of operations pavements: Traffic Area "C"

~~*2.* Heavy load pavements: Traffic areas "C" and "D"~~

Step 3. Design the bottom 4 inches of the non-frost-susceptible base as a filter blanket, as outlined in step 10a, above, if the total depth of frost penetration (a), found in step 6, exceeds the total combined thickness of pavement and base required in the reduced subgrade strength design, found in step 2.

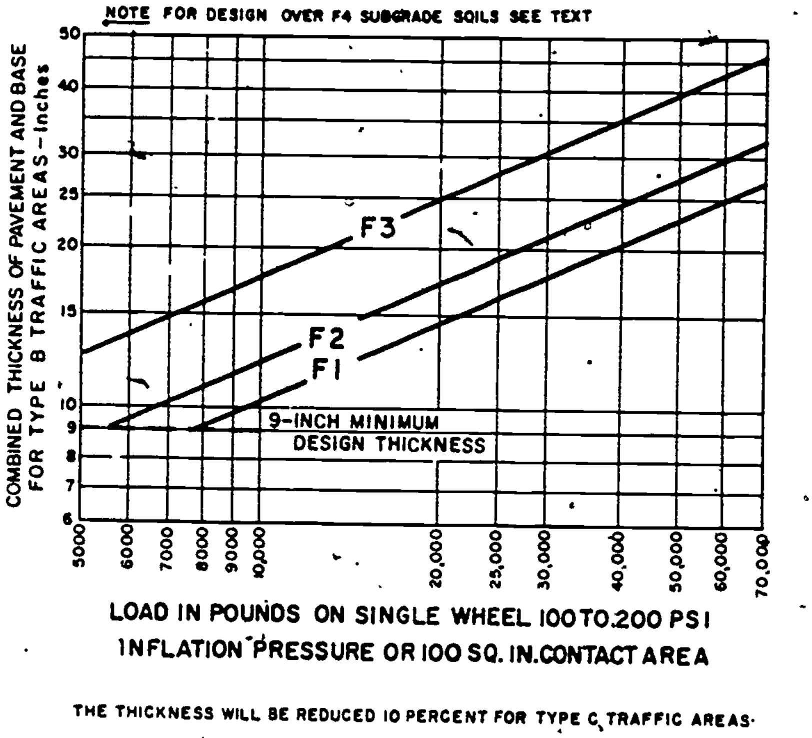


Figure 6-7. Frost condition reduced subgrade strength design curves for flexible pavements.

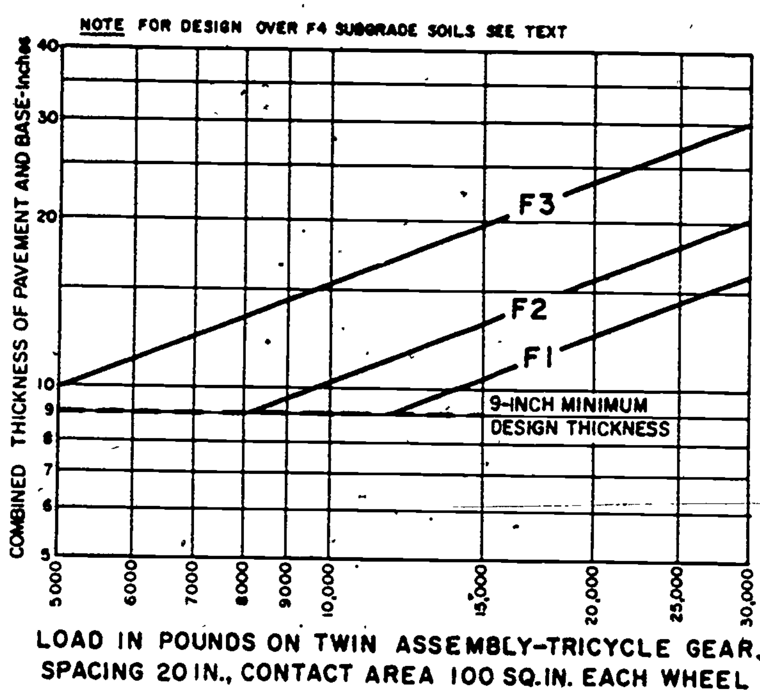
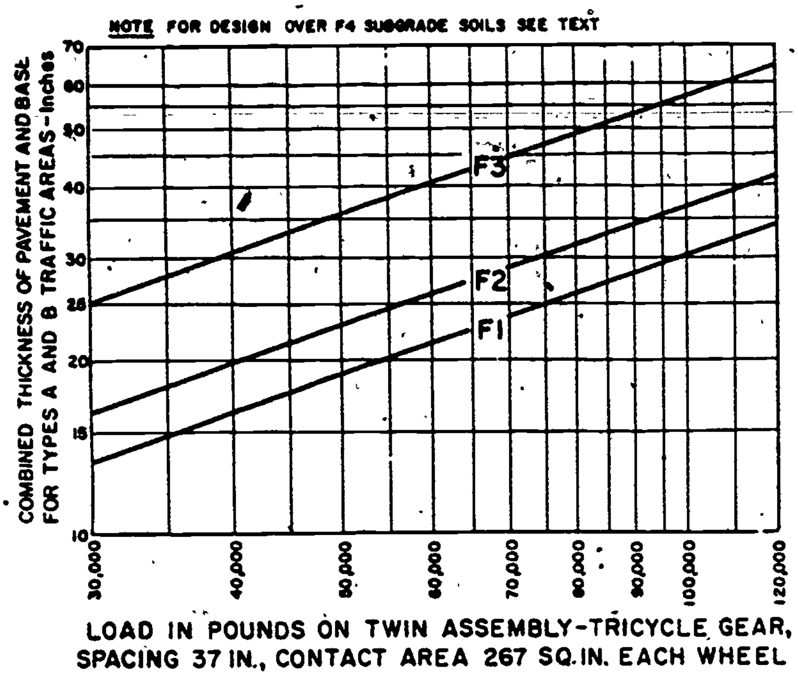
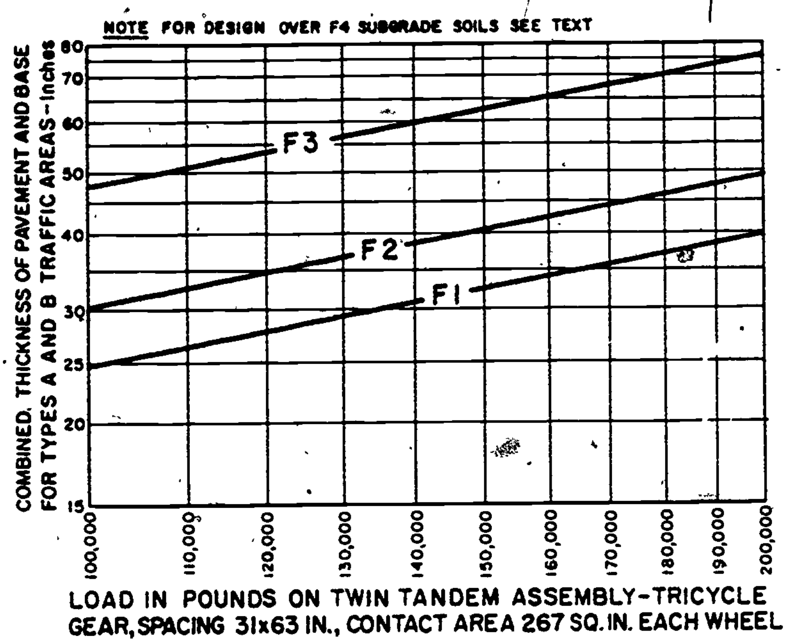


Figure 6-8. Frost condition reduced subgrade strength design curves for Army heliport flexible pavements.



THE THICKNESS WILL BE REDUCED 10 PERCENT FOR TYPE C TRAFFIC AREAS

Figure 6-9. Frost condition reduced subgrade strength design curves for flexible pavements.



THE THICKNESS WILL BE REDUCED 10 PERCENT FOR TYPE C TRAFFIC AREAS

Figure 6-10. Frost condition reduced subgrade strength design curves for flexible pavements.

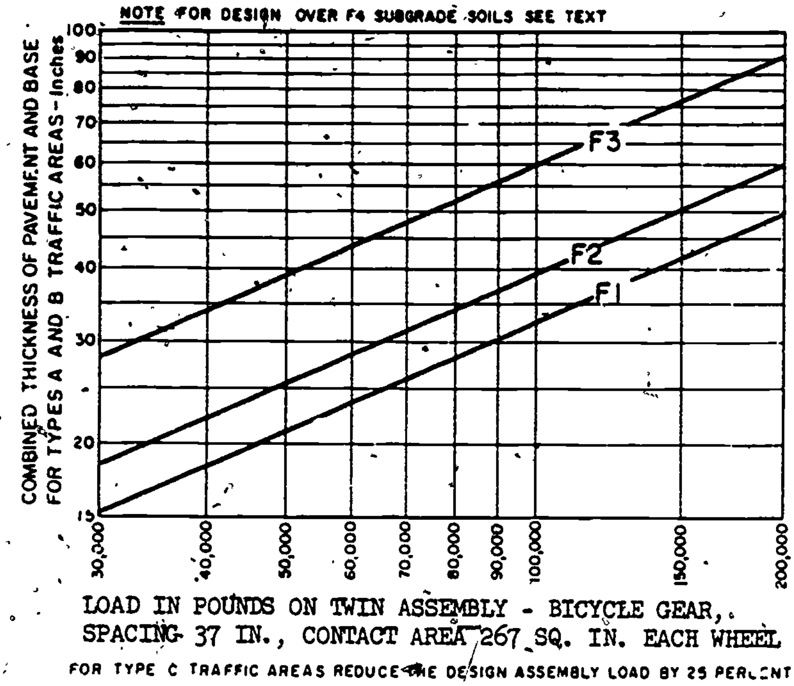


Figure 6-11. Frost condition reduced subgrade strength design curves for flexible pavements.

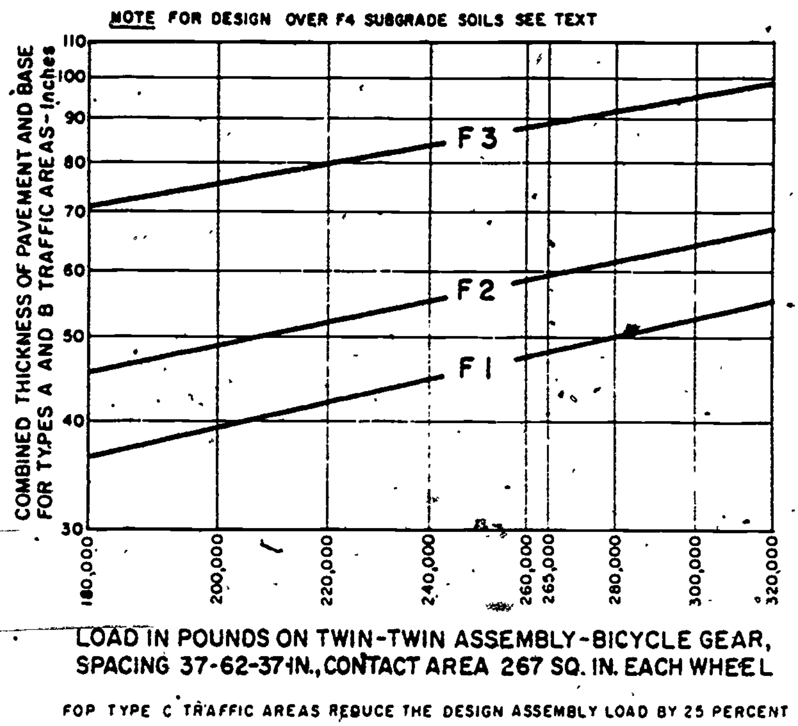


Figure 6-12. Frost condition reduced subgrade strength design curves for flexible pavements.

Step 4. Compare the two designs "limited subgrade frost penetration" and "reduced subgrade strength" and select the most economical of the two designs (meaning the one design requiring the least expenditure of materials and effort, etc.) as your frost design.

Step 5. (Arctic and subarctic regions)

Use the thawing index found in step 1 and enter figure 6-13 at the upper scale; progress down until intersecting the curve; then progress horizontally left and read directly the thickness of non-frost-susceptible granular base required in inches; multiply this number with the appropriate correction factor for the various moisture contents tabulated. This will give you a total depth of thaw.

Step 6. Compare the total depth of thaw, from step 5, with the reduced subgrade strength design, from steps 1-3, and select the more economical of the two, then select your final design based on the choice as outlined in step 4 above.

Step 7. Incorporate the final frost design into the regular "temperate climate design" as follows:

(a) The required thickness of asphaltic concrete pavement for the "temperate climate design" will remain unchanged.

(b) Change the depth of the non-frost-susceptible base course, if necessary, to fulfill the requirements of both the "temperate climate design" and the appropriate selected "frost" design. (This may in many instances require that the total pavement structure of base, subbase, and select material be changed to a non-frost-susceptible granular base course all the way through and may in addition require an increase in depth of the total pavement structure).

(c) If at any time the depth of frost penetration (a), found in step 6, exceeds the total depth of asphaltic concrete pavement and granular non-frost-susceptible base course material, a 4-inch filter blanket, as specified in step 10, must be placed in the bottom of the non-frost-susceptible base at the transition between frost-susceptible and non-frost-susceptible materials. (This may occur between base and subgrade, but may also occur between base and subbase, as well as between base and select material.)

Step 8. Add the compaction requirements as tabulated in table 6-4. Make sure that all of the requirements written in the upper portion of the table are fulfilled prior to entering the table below.

The table is based on two types of subgrade soil, "Cohesionless", meaning a soil having a PI from 0 to 5, and "Cohesive", meaning a soil having a PI of more than 5. Also be aware of the difference in requirements between areas of cut or fill.

6-7. PERMAFROST DEFINITIONS

Permafrost is perennially frozen ground. Note that this is a condition of soil temperature and does not necessarily imply an unsatisfactory foundation condition (fig 6-14). Suprapermafrost is the entire layer of ground above the permafrost table. This may or may not coincide with the "active layer", which is defined as the top stratum that is exposed to seasonal freezing and thawing. The thickness of the active layer and of the permafrost layer depend primarily on the mean annual temperature and the ambient air temperature. The farther north you go, the lower the mean annual temperature and the fewer the number of thawing days; therefore, the thicker will be the permafrost layer and the thinner the

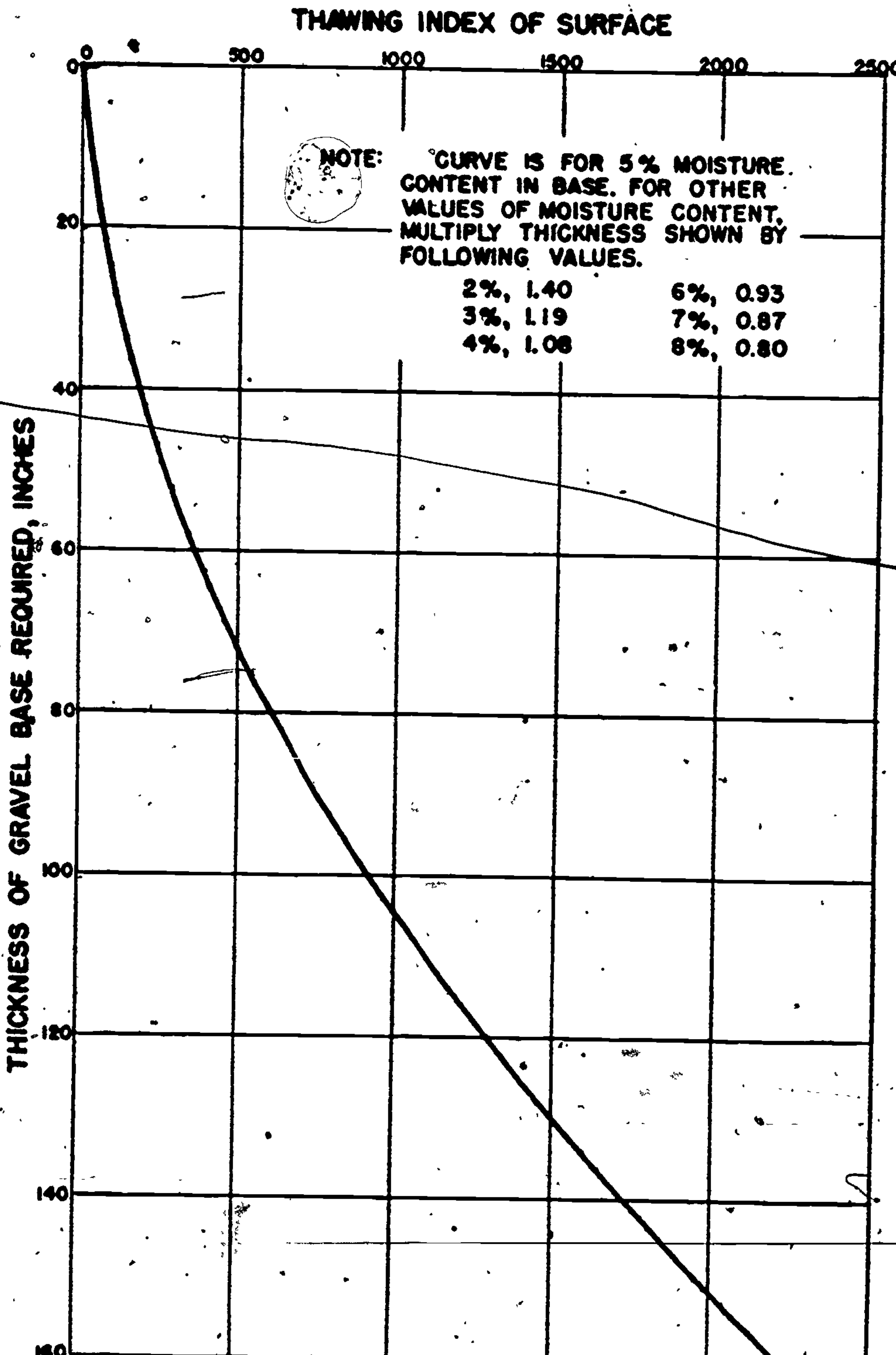


Figure 6-13. Thickness of base required to prevent thawing of subgrade.

TABLE 6-4. *Compaction Requirements for Types A, B, C, and D Traffic Areas*

Material	Percentage Compaction
	Materials with Design CBR Values of 20 and above
Base courses	Maximum that can be obtained, generally in excess of 100% of modified AASHO maximum and never less than 100%. Proof-rolling* in Type A traffic areas and central 75 ft of runways for heavy load pavements.
Subbases and subgrades	100% of modified AASHO maximum except where it is known that a higher density can be obtained practicably, in which case the higher density should be required.
	Materials with Design CBR Values below 20
Select material and subgrades in fills	As shown below except that in no case will cohesionless fill be placed at less than 95% nor cohesive fill at less than 90%.
Subgrade in cuts	Subgrade in cuts must have natural densities equal to or greater than the values listed below. Where such is not the case, the subgrade must (a) be compacted from the surface to meet the tabulated densities, (b) be removed and replaced, in which case the requirements given above for fills apply, or (c) be covered with sufficient select material subbase and base so that the uncompacted subgrade is at a depth where the in-place densities are satisfactory.

Depth of Compaction for Select Materials and Subgrades

Depth of Compaction in Feet for Percent Modified AASHO Compaction Shown

Cohesionless Materials

Type of Assembly	Gear Load kips	100 A	100 B	100 C	100 D	95 A	95 B	95 C	95 D	90 A	90 B	90 C	90 D	85 A	85 B	85 C	85 D
Heavy Load Pavements																	
Twin-twin, bicycle	160**		3				5.5				8				10		
Spacing, 37-62-37 in. Contact area, 267 sq in.	265	4.5	4	3.5	2	8	7.5	6.5	4	11.5	10.5	9	5.5	14	13	11	6.5
Light Load Pavements																	
Single wheel	10		1	1			1.5	1.5			2	2			3	2.5	
Contact area, 100 sq in.	20		1.5	1			2	1.5			3	2.5			4	3.5	
	25		1.5	1.5			2.5	2			3.5	3			4	4	
	30		1.5	1.5			2.5	2.5			4	3.5			4.5	4	
Miscellaneous																	
Twin-twin, bicycle	160	3	3	2.5	1.5	6	5.5	4.5	2.5	8.5	8	6.5	4	10.5	10	8	5
Spacing, 37-62-37 in.	200	3.5	3.5	3	1.5	6.5	6.5	5.5	3	9.5	9	7.5	4.5	12	11	9.5	5.5
Contact area, 267 sq in.	230	4	4	3	2	7.5	7	6	3.5	10.5	10	8	5	13.5	12.5	10.5	6
	265	4.5	4	3.5	2	8	7.5	6.5	4	11.5	10.5	9	5.5	14	13	11	6.5
	300	5	4.5	4	2.5	8.5	8	7	4	12.5	11.5	9.5	6	15.5	14	11.5	7
	330	5.5	5	4	2.5	9	9	7.5	4.5	13	12	10	6	16.5	15	12.5	7.5
Twin, bicycle	50	2	2	1.5	1	4	3.5	3	1.5	5	4.5	3.5	2.5	7	6	5	3
Spacing, 37 in.	75	2.5	2.5	2	1	4.5	4	3.5	2	6.5	5.5	4.5	3	8.5	7	6	3.5
Contact area, 267 sq in.	100	3	3	3	1.5	5.5	5	4	2.5	7.5	6.5	5.5	3.5	10	8.5	7	4.5
	125	3.5	3.5	3	1.5	6	5.5	4.5	3	8.5	7.5	6.5	4	11	9.5	8	5
	150	4	4	3	2	7	6	5	3	9.5	8	7	4	12	10.5	9	5.5
Twin tandem, tricycle	100	2.5	2	2		4.5	4	3.5		6.5	6	5.5		9	8	7	
Spacing, 31-63 in.	120	2.5	2.5	2		5	4.5	4		7.5	6.5	6		9.5	9	8	
Contact area, 267 sq in.	135	3	2.5	2.5		5.5	5	4.5		8	7	6.5		10.5	9.5	8.5	
	150	3	3	2.5		6	5	4.5		8.5	7.5	7		11	10	9	
	170	3.5	3	3		6.5	5.5	5		9	8.5	7.5		12	11	10	
Twin, tricycle	40	1.5	1.5	1.5		3	2.5	2.5		4.5	4	3.5		5.5	5	4.5	
Spacing, 37 in.	60	2	2	1.5		4	3.5	3		5.5	5	4.5		7	6.5	6	
Contact area, 267 sq in.	80	2.5	2.5	2		4.5	4	3.5		6.5	5.5	5		8	7.5	7	
	100	3	2.5	2.5		5	4.5	4		7	6.5	5.5		9.5	8.5	8	
	120	3.5	3	2.5		5.5	5	4.5		8	7	6.5		10.5	9.5	8.5	
Single wheel	10		1	1			1.5	1.5			2	2			2.5	2.5	
100-psi tire inflation	30		1.5	1.5			2.5	2.5			3.5	3.5			4.5	4	
pressure	50		2	2			3.5	3			4.5	4			6	5.5	
	70		2.5	2			4	3.5			5.5	5			7	6.5	

Cohesive Materials

Type of Assembly	Gear Load kips	100 A	100 B	100 C	100 D	95 A	95 B	95 C	95 D	90 A	90 B	90 C	90 D	85 A	85 B	85 C	85 D	80 A	80 B	80 C	80 D
Heavy Load Pavements																					
Twin-twin, bicycle	160**		1.5				3				4				5.5				7		
Spacing, 37-62-37 in. Contact area, 267 sq in.	265	2.5	2.5	2	1.5	4.5	4	3.5	2	6.5	6	5	3	8.5	8	6.5	4	10.5	9.5	8	4.5
Light Load Pavements																					
Single wheel	10		0.5	0.5			1	1			1	1			1.5	1.5			2	1.5	
Contact area, 100 sq in.	20		1	1			1.5	1			1.5	1.5			2	2			2.5	2.5	
	25		1	1			1.5	1.5			2	1.5			2.5	2			3	3	
	30		1	1			1.5	1.5			2	2			3	2.5			3.5	3	
Miscellaneous																					
Twin-twin, bicycle	160	2	1.5	1.5	1	3	3	2.5	1.5	4.5	4	3.5	2	6	5.5	5	3	7.5	7	6	3.5
Spacing, 37-62-37 in.	200	2	2	1.5	1	3.5	3.5	3	1.5	5.5	5	4	2.5	7	6.5	5.5	3.5	8	8	7	
Contact area, 267 sq in.	230	2.5	2	2	1	4	4	3	2	6	5.5	4.5	2.5	8	7	6	3.5	9.5	9	7.5	[illegible]
	265	2.5	2.5	2	1.5	4.5	4	3.5	2	6.5	6	5	3	8.5	8	6.5	4	10.5	9.5	8	4.5
	300	3	3	2.5	1.5	5	4.5	4	2.5	7	6.5	5.5	3	9	8	7	4	11	10	8.5	5
	330	3	3	2.5	1.5	5.5	5	4.5	2.5	7.5	7	6	3.5	9.5	9	7.5	4.5	11.5	11	9	5.5
Twin, bicycle	50	1.5	1	1	0.5	2	2	1.5	1	3	2.5	2	1.5	4	3.5	3	2	5	4	3.5	2
Spacing, 37 in.	75	1.5	1.5	1	0.5	2.5	2.5	2	1	3.5	3.5	3	1.5	5	4.5	3.5	2	6	5	4	2.5
Contact area, 267 sq in.	100	2	2	1.5	1	3	3	2.5	1.5	4.5	4	3.5	2	5.5	5	4.5	2.5	7	6	5	3
	125	2.5	2	1.5	1	3.5	3.5	2.5	1.5	5	4.5	3.5	2.5	6.5	6	5	3	8	7	5.5	3.5
	150	2.5	2.5	2	1	4	3.5	3	2	5.5	5	4	2.5	7	6.5	5.5	3.5	8.5	7.5	6.5	4
Twin tandem, tricycle	100	1	1	1		2.5	2	2		3.5	3	2.5		5	4.5	4		6	5.5	5	
Spacing, 31-63 in.	120	1.5	1.5	1		2.5	2.5	2		4	3.5	3		5.5	5	4.5		6.5	6	5.5	
Contact area, 267 sq in.	135	1.5	1.5	1.5		3	2.5	2.5		4.5	4	3.5		6	5	4.5		7	6.5	6	
	150	2	1.5	1.5		3	3	2.5		4.5	4	3.5		6.5	5.5	5		7.5	7	6.5	
	170	2	2	1.5		3.5	3	3		5	4.5	4		6.5	6	5.5		8	7.5	7	
Twin, tricycle	40	1	1	0.5		1.5	1.5	1.5		2.5	2	2		3	3	2.5		4	3.5	3	
Spacing, 37 in.	60	1.5	1	1		2	2	1.5		3	2.5	2.5		4	3.5	3.5		5	4.5	4	
Contact area, 267 sq in.	80	1.5	1.5	1.5		2.5	2.5	2		3.5	3	3		5	4.5	4		6	5	4.5	
	100	2	1.5	1.5		3	2.5	2.5		4	3.5	3.5		5.5	5	4.5		6.5	6	5.5	
	120	2	2	1.5		3.5	3	2.5		4.5	4	3.5		6	5.5	5		7.5	6.5	6	
Single wheel	10		0.5	0.5			1	1			1.5	1			1.5	1.5			2	2	
100-psi tire inflation	30		1	1			1.5	1.5			2	2			3	2.5			3.5	3	
pressure	50		1	1			2	2			2.5	2.5			3.5	3			4.5	4	
	70		1.5	1.5			2.5	2			3	3			4	4			5	4.5	

* Proof-rolling shall consist of 30 coverages of a heavy rubber-tired roller (150-psi, 30,000-lb minimum tire load) on each layer of base where the required CBR is in excess of 50 and on the top of the layer immediately under these layers.

** Hangar floors.

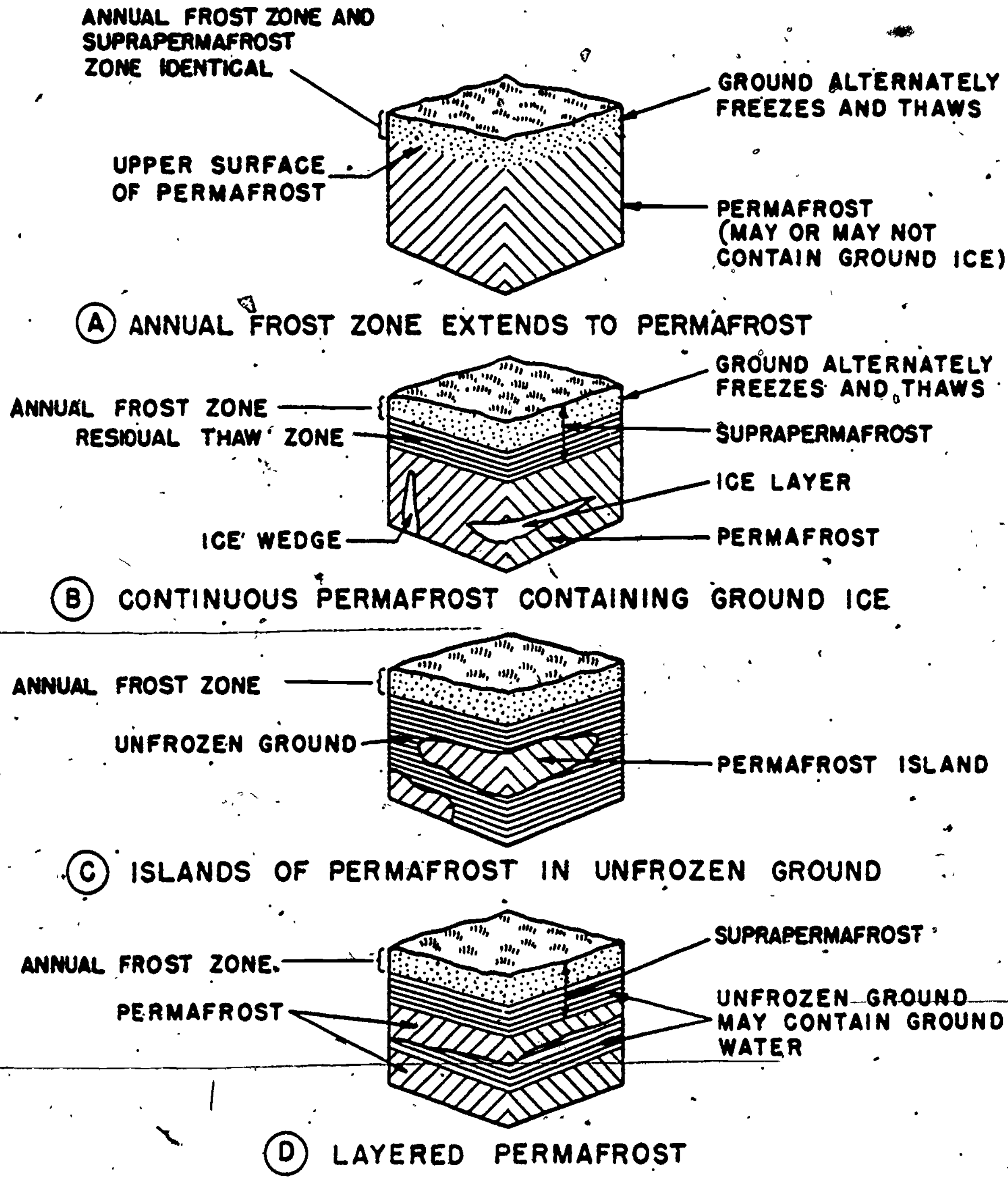


Figure 6-14. Typical sections through ground containing permafrost.

active layer. The permafrost layer varies from a foot or less in the southern part of the Arctic, to an estimated 1,600 feet in the vicinity of Thule, Greenland. Likewise, the active layer may vary from 8 to 10 feet in the more southerly latitudes, to 2 to 4 feet in the vicinity of Thule. Permafrost occurs in regions where the mean annual temperature is below freezing. The southern limit of permafrost generally coincides with the mean annual 32°F isotherm. Permafrost is common in northern North America and northern Asia. It underlies several million square miles of the arctic and subarctic regions. About a fifth of the world's land area is involved — practically all of Greenland four-fifths of Alaska, half of Russia, and large portions of Northern Canada.

6-8. PERMAFROST GROUNDWATER

a. **Principles.** Ground water may exist above, within, or below the permafrost layer depending on physiography, climate, and seasonal weather. Since free ground water is above freezing temperatures, its unaccustomed presence may thaw permafrost. Ground water is thus an important factor to consider in construction, apart from just the usual water supply question.

b. **Warm season behavior.** When the annual frost zone is unfrozen, ground water above the permafrost behaves like any ground water above an impervious stratum. In fine grained peaty soils, common in arctic regions, the water movement is extremely slow.

c. **Cold season behavior.** When seasonal freezing begins, the top surface of the annual frost zone becomes impervious. This confines the ground water and often puts it under pressure. As pressure increases by deepened freezing, the ground water seeks to escape. Often it is forced up through weak spots to the surface where it spreads and freezes into fields of surface ice. This action is found especially in hilly, terraced, and rolling terrain where the gradient of the ground water table is exceedingly steep and surface icefields continue to grow rapidly as long as water is available to feed them.

6-9. CONSTRUCTION PROBLEMS

a. Permafrost is a construction problem because of its effect upon the foundations of structures and pavements. The problem of the supporting strength of frozen soil is difficult to evaluate and only general guide lines have been developed. The general characteristics are known and are described as below.

b. Soil with a high bearing value when frozen may lose considerable strength when thawed, due to the escape of water which is released upon thawing.

c. Resultant settlement is invariably nonuniform because of the unequal distribution of soil moisture and/or segregated ice in the soil.

d. In the frozen state, the bearing capacity is directly related to ground temperature which, in turn, varies with mean surface temperature and numerous other factors.

e. Soil moisture is dependent on soil type, annual precipitation, water table, and glacial geology of the area under study.

f. The complexity of this problem is increased by the fact that a soil profile frequently does not exist in the Arctic, making a design based on the weakest known condition mandatory.

g. One of the interesting things about this problem of construction on permafrost is that, generally, the farther north one goes the less difficult the foundation construction becomes. When the permafrost temperature is near the freezing point (as is frequently the case in the subarctic) it becomes very difficult to predict the thermal behavior after construction and provide an adequate design.

6-10. GENERAL DESIGN APPROACHES FOR FOUNDATIONS

a. **Principles.** There are three general design approaches for foundations in permafrost areas. The first of these is to retain the thermal regime, to avoid upsetting the pattern of freeze and thaw established by nature over a long period of time; the second is to prethaw and establish a new thermal regime; and the third is to find a stable soil location. The first, to retain the thermal regime, is the design approach commonly used in the more northerly regions where soil temperature in permafrost is generally well below the freezing point. Design for a facility here aims at retaining the frozen ground at or near its original temperature following the construction of the facility.

b. **First method.** There are three common methods of designing construction so as **to retain the thermal regime.** The first is to raise the structure above the ground surface, allowing free passage of air beneath. The air provides an insulating blanket to prevent heat from infiltrating to the ground during the summer. Likewise, this provides free passage of cold air to insure the normal backfreeze during the winter. **This is the method commonly used for relatively light-load buildings.**

c. **Second method.** The second method of **retaining the thermal regime is to provide a fill of non-frost-susceptible material on top of the subgrade — a granular fill with a minimum of fine** so as to prevent ice segregation and detrimental frost action, thus preventing a heaving or buckling action during the winter freeze and settlement during the summer thaw. This is the method commonly used for airfields and roads in the Arctic where no influx of heat, as from a heated building, occurs during the winter months and seasonal backfreeze is readily obtained. The depth of non-frost-susceptible material necessary **to retain seasonal thaw** should be computed. In the subarctic, where the thawing index may be higher than 1,000, it may be entirely feasible to provide complete thaw protection. In this case, the design must be made for seasonal weakening during the frost melting period. In moderate climates, airfield construction design normally follows a cut versus fill balance, wherein areas of cut are balanced as nearly as possible by areas of fill. In arctic and subarctic regions, **cut should be eliminated to the maximum extent practicable.** In so doing, advantage is taken of the existing suprapermafrost which already forms a generally trouble-free insulative layer. Special consideration must be given to drainage. Subsurface drainage is almost impossible in the Arctic. In many cases, it is mandatory to orient a runway, for example, in the same direction as the existing drainage pattern to **eliminate harmful cross-drainage.** Moving surface or subsurface water is probably the worst enemy of a thermal regime in arctic soil. As an example, water seepage under a taxiway in Greenland caused a 20-foot depth of thaw in the permafrost. The surfacing of airfields and roads is of particular importance. Flexible pavement is normally employed for airfield surfacing for two chief reasons: First, it is almost impossible to provide a base where no vertical movement will occur. Bituminous pavement provides enough flexibility to allow for this slight movement without requiring undue maintenance. Second, the lower ambient temperatures during the summer season prohibit the placement and curing of rigid pavement during the inherently short construction season. Bituminous surfacing has one decided disadvantage in the Arctic. The color of this pavement absorbs the sun's rays and produces surface temperatures sometimes as much as 25°F in excess of ambient air temperature. Since the depth of thaw is in direct proportion to surface temperature rather than air temperature, the effect of a black pavement on thaw penetration can be readily seen.

d. **Third method.** The third method of retaining the thermal regime is a sort of combination of these two previous methods — the granular fill and the passage of cold air underneath the floor. **It is used for large structures which are required to support heavy floor loads such as hangars and heat-and-power plants.** It is essentially a slab-on-grade construction in which the under-floor cooling is accomplished either by natural or forced air circulation, or mechanical refrigeration. In this method a subflooring is placed on top of a granular fill. Corrugated metal pipes or a series of metal pans run through the subflooring and connect to plenum chambers on opposite sides of the buildings, and extend vertically to produce a "chimney effect" to provide a continuous flow of cold

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outside air through the subflooring. Above the pipes is a layer of concrete, a layer of foam glass insulation, and then the finished floor. This was the method used in constructing foundations for hangars and other heavy-floor-load buildings at Thule Air Force Base. On the Nike installations surrounding Thule Air Force Base mechanical refrigeration is used to keep the material supporting portions of the launcher buildings frozen year round. To meet ordnance safety requirements, the launcher box at each installation had to be placed below the surface, with earth mounded above the exposed portion. The area was excavated to the required grade, exposing the permafrost, and was immediately backfilled with a thin layer of non-frost-susceptible material to retard melting. A refrigeration pipe grid system was installed in the foundation to prevent heat transfer from the building. All of the above methods can be used for foundations where the soil temperature is well below the freezing point. The purpose is to prevent extension of the active layer, or to prevent thawing of it at all (as in the refrigerated foundations) so as to prevent degradation of the permafrost table with resultant harmful effects.

e. **New thermal regime.** Throughout large areas of the subarctic it is almost impossible to design for retention of the existing thermal regime because ground temperatures of the permafrost are very close to the freezing point. In these areas a different design approach is necessary. If it were possible to design for the bearing capacity of the soil in a thawed condition, that would be the solution, but since most buildings need greater support, foundations must be placed either on a fill of satisfactory granular material after excavation of the permafrost, or on piles set deep into the permafrost. Frequently a combination of these two methods is used in actual design. Where piles are used they are generally set into the permafrost to a **minimum depth equal to twice the thickness of the active layer.** Because of the rock-like characteristics of permafrost, it is seldom practical to drive piles. They are usually placed by one of three methods: drilling, jetting, or trenching.

(1) **Drilling.** In the drilling method, the pile is placed in a predrilled hole and embedded in a soil slurry which quickly backfreezes in the permafrost portion of the soil.

(2) **Jetting.** Jetting is accomplished with steam point or water jets. This method can only be used where the soil type is of the sandy silty or sandy type.

(3) **Trenching.** Sometimes the most economical method of setting piles, especially where clusters are involved, is to excavate a trench to the desired depth, place the piles, and backfill. In all cases, a suitable period for backfreeze around the piles must be allowed before construction can proceed.

f. **Satisfactory soil location.** The possibility that an entirely satisfactory soil location can be located in arctic or subarctic regions should not be overlooked. The cost of an extensive subsurface investigation can be completely justified by locating an area where little, if any, special treatment is required to develop a suitable foundation for structures or airfields. A typical example of this condition would be an outwash gravel terrace located along a stream bed. The complete lack of patterned ground, as determined from a thorough study of aerial photographs, is a good indication that a suitable subsurface condition may exist.

SELF TEST

Note: The following exercises comprise a self test. The figures following each question refer to a paragraph containing information related to the question. Write your answer in the space below the question. When you have finished answering all the questions for this lesson, compare your answers with those given for this lesson in the back of this booklet. Do not send in your solutions to these review exercises.

1. Frost action is a process which results from freezing and thawing of soil. This can present difficult problems to an engineer. Describe the nature of these problems. (6-1)

2. In the freezing process ice formations, called ice lenses, form in the subgrade and seem to continue to grow. Explain this phenomenon briefly. (6-2a, fig 6-1)

3. In late winter and early spring, the ice lenses thaw either from the top down due to warming of the atmosphere, or from the bottom up due to outward conduction of heat from the earth interior. Which is desirable from an engineer's position, and why? (6-2b)

4. What conditions must be present in order to have significant frost action? (6-3a, b, c, d)

5. Frost heave, indicated by raising of the pavement, may be either uniform or irregular; the latter causing the most severe problems. What conditions generally tend to cause irregular heaving? (6-4a)

6. Explain briefly the theory of loss of subgrade supporting strength during the spring thawing period. (6-4b)

7. Explain how frost boils are developed in flexible pavements. (6-4c(2))

8. At what depth should the highest level of the water table be below the top of the proposed subgrade surface (or any frost-susceptible base material used), to insure that there is only a minimum source of water for ice segregation? (6-5a)

9. A counteractive technique to prevent frost action is the pacing of a layer of pervious, coarse grained soil 2 to 3 feet beneath the surface. What is the recommended thickness of this layer? (6-5b)

10. One of the first things to calculate in limited subgrade frost penetration design is total frost penetration. If dry unit weight is 115 pounds/cubic foot, moisture content is 15 percent, and the air freezing index is 500 degree days, what is the depth of frost penetration (inches)? (6-6a, step 6, fig 6-3)

11. In limited subgrade frost penetration design, the results of the design are the base thickness b and allowable subgrade frost penetration s. If r = 1.4 and c = 50 (base thickness for zero frost penetration into subgrade), what are b and s? (6-6a, step 9, fig 6-5)

12. For both flexible and rigid pavements at least the bottom 4 inches of the base will be designed as a filter. This filter material will, in no case, have more than a certain percent by weight finer than 0.02 millimeter. What is this percentage? (6-6a, step 10)

13. In reduced subgrade strength design one should not use F-4 material except for lesser important slow speed flexible pavements. If the subgrade material is F-2, type B traffic and 45,000 pound load in pounds on twin assembly tricycle gear, spacing 37 inches, contact area 267 square inches each wheel, what is the combined thickness of pavement and base? (6-6b, step 2, fig 6-9)

14. No matter what the loading, there is a minimum for the thickness of pavement plus non-frost-susceptible base. What is this minimum (inches)? (par 6-6b, step 2)

15. Permafrost is perennially frozen ground. Does this imply an unsatisfactory foundation condition? (6-7)

16. Why does the presence of ground water in a permafrost area present special and important considerations in construction? (6-8a, 6-9b)

17. Of the three general design approaches for foundations in permafrost area, retaining the thermal regime is one. What method of retaining the thermal regime would you use for a relatively light-load building? (6-10b)

18. Retaining the thermal regime avoids upsetting the pattern of freeze and thaw established by nature over a long period of time. What method of retaining the thermal regime would you use for roads and airfields in the Arctic where there is no influx of heat? (6-10c)

19. What method of retaining the thermal regime would you use for a large structure which is required to support heavy floor loads? (6-10d)

20. If ground temperatures of the permafrost are very close to the freezing point it is almost impossible to design for retention of the thermal regime. Another method is piles. What is the accepted minimum depth to which the piles should be placed? (6-10e)

LESSON 7

IMPROVEMENT OF SOIL CHARACTERISTICS

CREDIT HOURS ---------------------------2

TEXT ASSIGNMENT ---------------------Attached memorandum.

MATERIAL REQUIRED --------------------None.

LESSON OBJECTIVE ---------------------Upon completion of this lesson you should be able to accomplish the following in the indicated topic areas:

1. **Moisture Control and Blending.** Determine the reason for stabilization and the most appropriate method under the circumstances existing. Describe the procedure for adjusting moisture content and for blending of available soils for improved compaction.

2. **Chemical and Bituminous Stabilization.** Define the situations under which soil stabilization by the application of chemicals such as portland cement and lime, and bituminous materials, may be effective in improving the construction qualities of the natural soils.

3. **Dust Palliatives and Waterproofing.** Explain the five general classifications of materials available to the engineer for dust control and/or soil waterproofing, how they are best used, and the advantages and disadvantages of each.

ATTACHED MEMORANDUM

7-1. SOIL CHARACTERISTICS

Improvement of soil characteristics is accomplished by the alteration or preservation of one or more properties of a soil to improve its engineering characteristics and performance. A wide selection of specific processes and materials is available to the engineer, none of which is capable of solving all soils problems. The selection of a specific process or material depends upon the particular reason why you need to improve the soil characteristics. There are three primary reasons why one needs to mechanically or chemically stabilize a soil. They are: strength improvement — increase the stability of the existing soil to enhance its load-carrying capacity; dust control — eliminate or alleviate dust during dry weather or in arid climates; soil waterproofing—preserve the natural strength of a soil by preventing the ingress of surface water.

7-2. MOISTURE CONTROL

a. **Compaction.**

(1) **Description.** The process of compaction of a soil is perhaps the oldest and most important means of soil stabilization. By compaction, particles are rearranged to form a denser mass with a resulting change in engineering properties such as strength, permeability, and compressibility. The extent of alteration which can be achieved by compaction depends on the soil type, moisture content, amount and method of compaction. Generally, fine-grained soils are more difficult

to compact than coarse-grained soils and are more sensitive to the effects of overcompaction.

(2) **Uses.** In the construction of a theater-of-operations airfield, compaction alone might suffice to increase the bearing strength of an existing soil from a borderline condition to an acceptable one for unsurfaced operations or for the placement of landing mat.

(3) **Procedures.** To achieve the best results from compaction, a soil should be at or near its optimum moisture content. Since water content may or may not be controllable in expedient airfield construction, good judgment should be used in each instance to establish the feasibility of compaction. In general, plastic soils cannot be compacted if the moisture is more than about one-fourth above optimum. Cohesionless soils should be placed at a sufficiently high moisture content to prevent bulking. Depending on the soil involved, its condition, and the type of compaction applied, densification will result in depths ranging from 6 to 12 inches. In general, rubber-tired and sheepsfoot rollers are best equipped to compact cohesive soils. Cohesionless soils can best be compacted by vibratory compactors, heavy tractors, and to a lesser extent by rubber-tired rollers. Smooth, steel-wheel rollers are used normally to maintain a smooth surface during compaction and finishing operations.

b. **Water content above optimum.** Scarify and dry out. Do not compact when too wet. This will cause settlement when drying out and load is applied. Soil particles will take the place of water.

c. **Water content below optimum.**

(1) **Principle.** Scarify and add water; recompact the soil. This will cause slight settlement under the right conditions when water is added and the load is applied.

(2) **Quantitative moisture control.** If the moisture content of the soil is less than optimum, the amount of water which must be added for efficient compaction generally is computed in gallons per 100 feet of length (station). The computation is based upon the dry weight of soil contained in a compacted layer. For example, assume that the soil is to be placed in a layer 6 inches in compacted thickness at a dry unit weight of 120 pounds per cubic foot. The moisture content of the soil is determined to be 5 percent, while the optimum moisture content is 12 percent. Assume that the strip to be compacted is 40 feet wide or 40 station feet per station. The wieght of water required per cubic foot (with no allowance for evaporation) is equal to $120\,(0.12-0.05) = 120(0.07) = 8.4$ pounds. Allowance for evaporation varies from 5 percent to 10 percent depending on temperature, humidity, work conditions, type of soil, and length of time between placing and rolling. In this example an allowance of 10 percent is made for evaporation. Then, $8.4 \times (1.0 + 0.1) = 8.4 \times 1.1 = 9.24$ pounds of water required to be added per cubic foot of soil to obtain optimum moisture content and allow 10 percent for evaporation. Since water weights 8.33 pounds per gallon, the number of gallons required per cubic foot of soil $= 9.24 \div 8.33 = 1.109$. The volume of compacted soil per station $= 40(100)\ 6/12 = 2000$ cubic feet. The amount of water required per station for the 40 foot wide strip is then $1.109 \times 2000 = 2{,}218$ gallons. Table 7-1 can be used in lieu of computations of the type discussed (except allowances for evaporation cannot be used). Adjustments for the effect of rain during the laydown may be made by use of table 7-2.

7-3. BLENDING

a. **Principle.** The overall objective of mechanical stabilization is simply to blend available soils in such a fashion that, when properly compacted, they will give the desired stability. In certain areas, for example, the natural soil at a selected location may be of low supporting power because of an excess of clay, silt, or fine sand. Within a reasonable distance there may exist suitable granular materials which may be blended with the existing soil to effect a marked improvement in stability of the soil at a very much lower cost in manpower and material than is involved in applying imported surfacing, and it may even produce better results in the end.

TABLE 7-1. *Moisture Content — U. S. Gallons/Sq Yd or Station Foot for 6-inch Compacted Lift*

Moisture content %,	Gallons	50	55	60	65	70	75	80	85	90	95	100	105	110	115	120	125	130	135	140	145	150
		Dry unit weight (Pounds/cubic foot)																				
1	Sq yd	0.27	30	0.32	0.35	0.38	0.41	0.43	0.46	0.49	0.51	0.54	0.57	0.59	0.62	0.65	0.68	0.70	0.73	0.76	0.78	0.81
	Station foot	3.0	3.3	3.6	3.9	4.2	4.5	4.8	5.1	5.4	5.7	6.0	6.3	6.6	6.9	7.2	7.5	7.8	8.1	8.4	8.7	9.0
2	Sq yd	0.54	0.60	0.65	0.70	0.76	0.81	0.86	0.92	0.97	1.03	1.08	1.14	1.18	1.24	1.30	1.36	1.40	1.46	1.51	1.57	1.62
	Station foot	6.0	6.7	7.2	7.8	8.4	9.0	9.6	10.2	10.8	11.4	12.0	12.7	13.1	13.8	14.4	15.1	15.6	16.2	16.8	17.4	18.0
3	Sq yd	0.81	0.89	0.97	1.05	1.13	1.22	1.30	1.38	1.46	1.54	1.62	1.70	1.78	1.86	1.94	2.03	2.11	2.19	2.26	2.35	2.43
	Station foot	9.0	9.9	10.8	11.7	12.6	13.6	14.4	15.3	16.2	17.1	18.0	18.9	19.8	20.7	21.6	22.6	23.4	24.3	25.1	26.1	27.0
4	Sq yd	1.08	1.19	1.30	1.40	1.51	1.62	1.73	1.84	1.94	2.05	2.16	2.27	2.38	2.48	2.59	2.70	2.81	2.92	3.02	3.13	3.24
	Station foot	12.0	13.2	14.4	15.6	16.8	18.0	19.2	20.4	21.6	22.8	24.0	25.2	26.4	27.6	28.8	30.0	31.2	32.4	33.6	34.8	36.0
5	Sq yd	1.35	1.49	1.62	1.76	1.89	2.03	2.16	2.30	2.43	2.57	2.70	2.84	2.97	3.11	3.24	3.38	3.51	3.65	3.78	3.92	4.05
	Station foot	15.0	16.6	18.0	19.5	21.0	22.5	24.0	25.5	27.0	28.5	30.0	31.6	33.0	34.6	36.0	37.6	39.0	40.6	42.0	43.6	45.0
6	Sq yd	1.62	1.78	1.94	2.11	2.27	2.43	2.59	2.75	2.29	3.08	3.24	3.40	3.56	3.73	3.89	4.05	4.21	4.37	4.54	4.70	4.86
	Station foot	18.0	19.8	21.6	23.4	25.2	27.0	28.8	30.6	32.4	34.2	36.0	37.8	39.6	41.4	43.2	45.0	46.8	48.6	50.4	52.2	54.0
7	Sq yd	1.89	2.08	2.27	2.46	2.65	2.84	3.02	3.21	3.40	3.59	3.78	3.97	4.16	4.35	4.54	4.73	4.91	5.10	5.29	5.48	5.67
	Station foot	21.0	23.1	25.2	27.3	29.4	31.5	33.6	35.7	37.8	39.9	42.0	44.1	46.2	48.3	50.4	52.5	54.6	56.7	58.8	60.9	63.0
8	Sq yd	2.16	2.38	2.59	2.81	3.02	3.24	3.46	3.67	3.89	4.10	4.32	4.54	4.75	4.97	5.18	5.40	5.62	5.83	6.05	6.26	6.48
	Station foot	24.0	26.4	28.8	31.2	33.6	36.0	38.4	40.8	43.2	45.6	48.0	50.4	52.8	55.2	57.6	60.0	62.4	64.8	67.2	69.5	72.0
9	Sq yd	2.43	2.67	2.92	3.16	3.40	3.65	3.89	4.13	4.37	4.62	4.86	5.10	5.35	5.59	5.83	6.08	6.32	5.56	6.80	7.05	7.29
	Station foot	27.0	29.7	32.4	35.1	37.8	40.5	43.2	45.9	48.6	51.3	54.0	56.7	59.4	62.1	64.8	67.5	70.2	72.9	75.5	78.3	81.0
10	Sq yd	2.70	2.97	3.24	3.51	3.78	4.05	4.32	4.59	4.86	5.13	5.40	5.67	5.94	6.21	6.48	6.75	7.02	7.29	7.56	7.83	8.10
	Station foot	30.0	33.0	36.0	39.0	42.0	45.0	48.0	51.0	54.0	57.0	60.0	63.0	66.0	69.0	72.0	75.0	78.0	81.0	84.0	87.0	90.0

TABLE 7-2. *Effect of Rainfall*

Inches of rain	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0
Gal/sq yd.	0.56	1.12	1.68	2.24	2.80	3.36	3.93	4.49	5.05	5.61
Gal/station foot	6.22	12.4	18.7	24.9	31.1	37.3	43.7	49.9	56.1	62.3

b. Requirements for mechanically stabilized soil bases.

(1) Experience in civil highway construction has indicated that best results will be obtained with this type of mixture if the fraction passing the No. 200 sieve is not greater than two-thirds of the fraction passing the No. 40 sieve. The size of the largest particles should not exceed two-thirds of the thickness of the layer in which they are incorporated. The mixture should be well graded from coarse to fine.

(2) A basic requirement of soil mixtures which are to be used as base courses is that the plasticity index should not exceed 6. Under certain circumstances, this requirement may be relaxed if a satisfactory bearing ratio is developed. Experience also indicates that under ideal circumstances the liquid limit should not exceed 28. It may be possible to relax these requirements in theater-of-operations construction. The requirements may be lowered to a liquid limit of 36 and a plasticity index of 10 for full-operational airfields, and to a liquid limit of 45 and a plasticity index of 15 for emergency and minimum-operational airfields, when good drainage is provided.

(3) If the base is to function satisfactorily, other requirements than those relating to the soil mixture alone must be met. The base must normally have a high bearing value. Density requirements and those relating to frost action are also of particular importance.

c. Requirements for mechanically stabilized soil surfaces.

(1) Preference should be given to mixtures which have a maximum size of aggregate equal to 1 inch, or perhaps 1½ inches, as experience has indicated that particles larger than this tend to work themselves to the surface over a period of time under the action of traffic. Somewhat more fine soil is desirable in a mixture which is to serve as a surface, as compared with one for a base, so that it will be more resistant to the abrasive effects of traffic, more resistant to the penetration of precipitation which falls directly upon the surface, and can, to some extent, replace by capillarity moisture which is lost by evaporation.

(2) Emergency airfields which have surfaces of this type require a mixture which has a plasticity index between 5 and 10. Civil experience indicates that road surfaces of this sort should have a liquid limit not in excess of 35, and that the plasticity index should be between 4 and 9. The surface should be made as tight as possible and good surface drainage should be provided. It is to be noted that, for best results, the plasticity index of a stabilized soil which is to function first as a wearing surface and then as a base, with a bituminous surface being provided at a later date, should be held within very narrow limits.

(3) Considerations relative to compaction, bearing value, and frost action are as important for surfaces of this type as for bases.

d. Rule-of-thumb proportioning. A rough estimate of the proper proportions of available soils in the field is possible, and depends upon manual and visual inspection. Each soil should be moistened to a point where it is moist, but not wet; in a wet soil the moisture can be seen as a shiny film on the surface. What is desired is a mixture which will feel gritty and in which the sand grains can be seen. When the soils are combined in the proper proportion a cast formed by squeezing the moist soil mixture in the hand will be able to withstand normal handling without breaking. With practice, the optimum blend of two soils can be reasonably accurately established. This procedure is most applicable to the blending of clays or silty clays with sands or moderately coarse soils. If gravel is available, this may be added, although there is no real rule-of-thumb to tell how much should be added. It is always desirable to have too little gravel rather than too much.

e. The problem. This process of proportioning will now be illustrated by a numerical example. Two materials are available, material B in the roadbed and material A from a nearby borrow source. The mechanical analysis of each of these materials is given below, together with the liquid limit and plasticity index of each. The desired grading of the combnation is also shown, together with the desired plasticity characteristics. Results of laboratory tests are shown in table 7-3.

TABLE 7-3. *Results of Laboratory Tests on Soil Samples*

Sieve designation	Percent passing, by weight Material A	Material B	Specified
	Mechanical analysis		
1-inch	100	100	100
¾-inch	92	72	70-100
⅜-inch	83	45	50-80
No. 4	75	27	35-65
No. 10	67	15	20-50
No. 40	52	5	15-30
No. 200	33	1	5-15
	Plasticity characteristics		
		Percent by weight	
Liquid limit	32	12	Not more than 28.
Plasticity index	9	0	Not more than 6.

f. Proportioning to meet specified gradation.

(1) Figure 7-1a shows the first steps in plotting the data obtained from mechanical analysis of soils A and B. The percentage by weight of soil A passing each sieve is plotted on the vertical axis along the left margin of the graph; soil B is plotted on the vertical axis along the right margin. For each sieve, a line is drawn across the graph connecting the plotted values of percent passing by weight for each soil. These diagonals are referenced with the sieve size.

(2) So that the percentage of each soil, in any combination of the two, may be read directly from the graph, percentages of soil A are referenced in the top margin in descending values from left to right; percentages of soil B are shown in the lower margin in descending values from right to left. Reading directly from the graph, you will note that when a mixture has 80 percent of soil A, it will have 20 percent of soil B.

(3) Desired gradations of the combination (specification limits) are drawn in the margin to the right of the graph (fig 7-1b). This desired gradation also corresponds to specifications given in table 7-4.

(4) Now that the basic data has been plotted, we must graphically establish the limits of combination of the two soils that will meet specified gradation requirements. This is done by first extending the upper limit of the specified percent for each sieve size until it intersects its respective sieve line. Figure 7-1c shows these lines extended and intersecting the sieve lines at points A,B,C,D, and E. Through the point of intersection which is nearest to the right side of the graph (point E in this case) a perpendicular is drawn to intersect the top and bottom percentage scales. This perpendicular is the left limit line and shows the maximum amount of soil A and the minimum amount of soil B that the mixture should include in order to satisfy grading requirements. This procedure is for soil-mixture graphs in which the diagonals descend from left to right. When the diagonals ascend from left to right the left limit line is determined by the intersection of the lower limit of the specified percent with

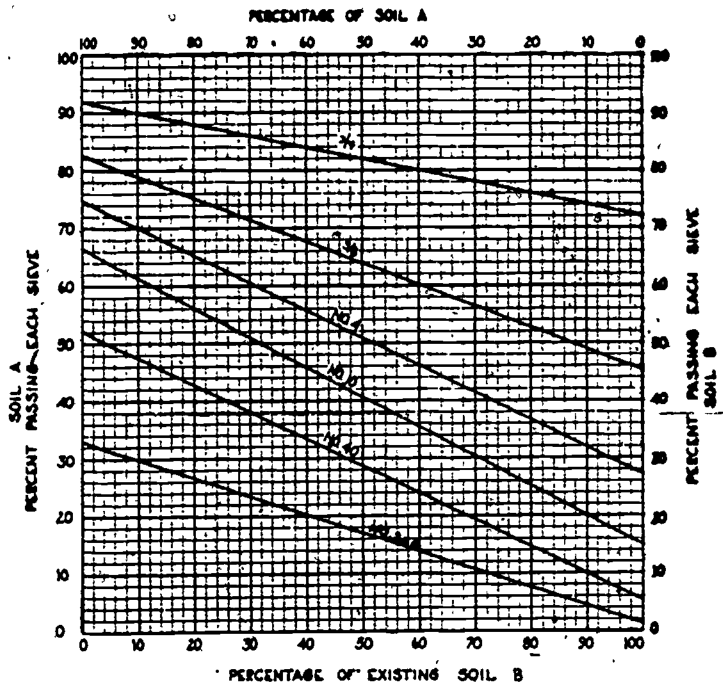

Figure 7-1a. Graphical method for proportioning soils.

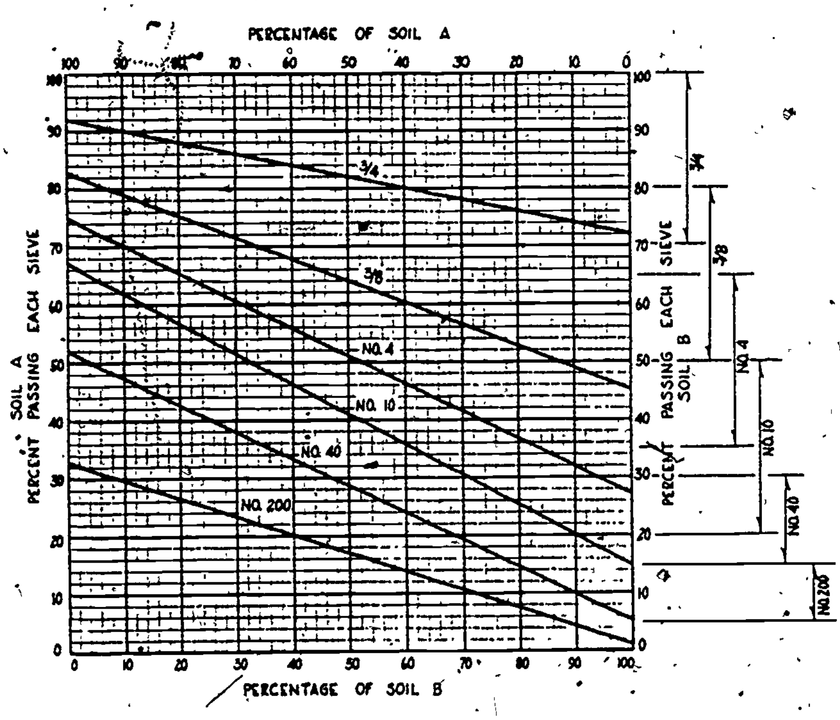

Figure 7-1b. Graphical method for proportioning soils.

Table 7-4. *Desired Gradation for Crushed Rock, Gravel, or Slag and Uncrushed Sandy and Gravelly Aggregates for Base Courses*

Sieve designation	Percent passing each sieve, by weight				
	Maximum aggregate size				
	3-inch	2-inch	1½-inch	1-inch	1-inch
3-inch	100				
2-inch	65-100	100			
1½-inch		70-100	100		
1-inch	45-75	55-85	70-100	100	100
¾-inch		50-80	60-90	70-100	
⅜-inch	30-60	40-70	45-75	50-80	
No. 4	25-50	30-60	30-60	35-65	
No. 10	20-40	20-50	20-50	20-50	65-90
No. 40	10-25	10-30	10-30	15-30	33-70
No. 200	3-10	5-15	5-15	5-15	8-25

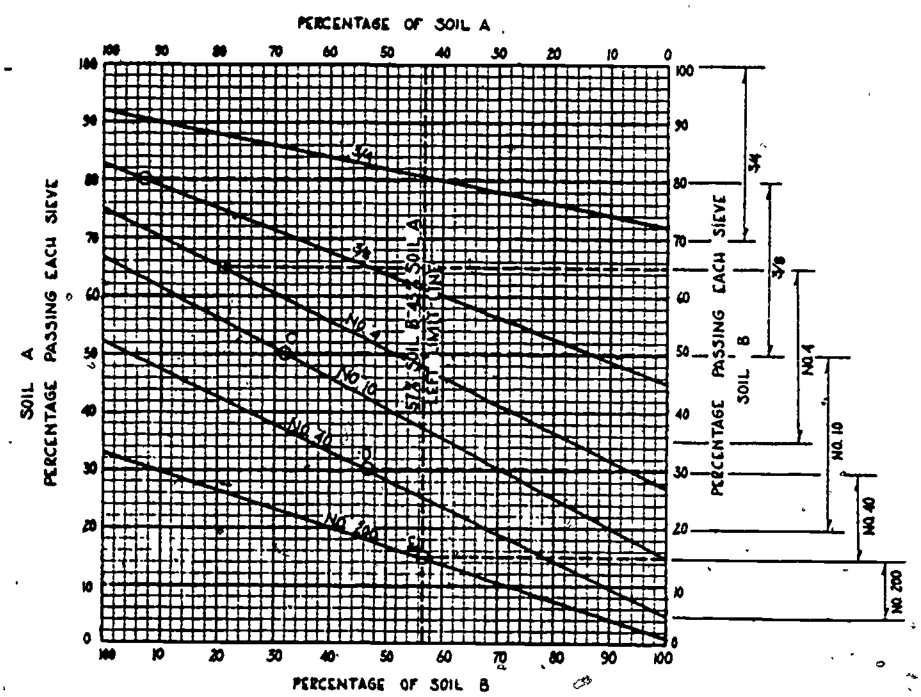

Figure 7-1c. Graphical method for proportioning soils.

its respective diagonal sieve line which lies furthest to the right on the graph.

(5) Figure 7-1d shows the right limit line established by extending the lower limits to the specified percents for each sieve size until they intersect their respective sieve lines. The right limit line is defined by the perpendicular passing through the point which is farthest to the left on the graph (point 1 in this case). The right limit line

shows the minimum amount of soil A and the maximum amount of soil B that can be included in a mixture meeting specifications. Any combination of the two soils with percentages of each soil falling within the right and left limits will meet gradation requirements. As in 7-1d, above, this method is for graphs in which the diagonals descend from left to right. When the diagonals ascend from left to right, the right limit line is established by extending the upper limits of the specified percents until they intersect their respective sieve lines, and is the perpendicular through the intersection which is farthest to the left.

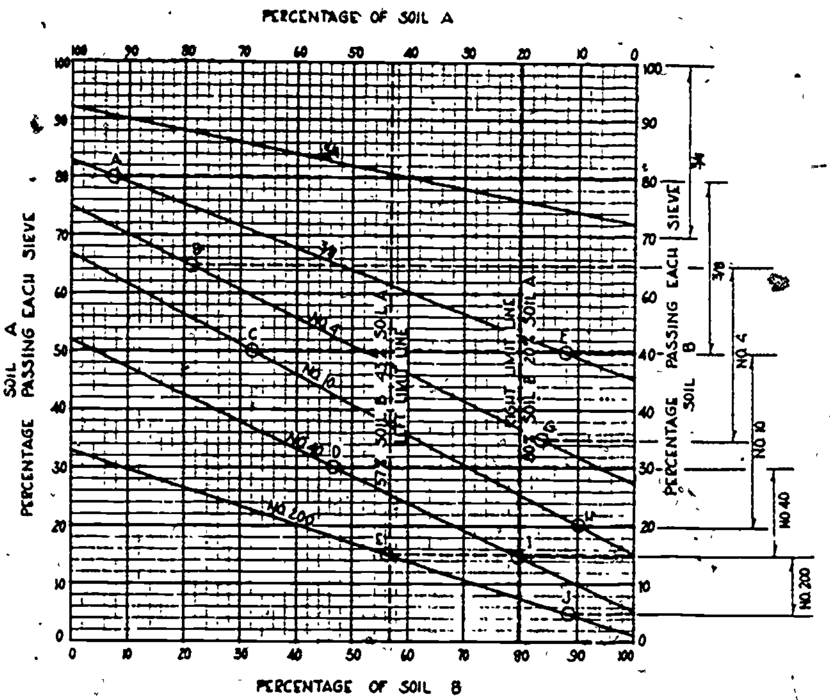

Figure 7-1d. Graphical method for proportioning soils.

(6) With some soils (diagonal sieve lines descending from left to right) the upper limit of a specified sieve size may fail to intersect its sieve line. This condition means that the soil plotted on the vertical axis on the left side of the graph will satisfy gradation requirements for that sieve size without any addition of the other soil. If a lower limit of a specified sieve size fails to intersect its sieve line, the soil plotted on the vertical axis on the right will satisfy grading requirements by itself. When the diagonal sieve lines ascend from left to right, failure of an upper limit to intersect its sieve line means the soil plotted on the right side will be satisfactory for that sieve size by itself; failure of a lower limit to intersect its sieve line indicates that the soil plotted on the left side will satisfy the specifications for that sieve size by itself. If both upper and lower limits of a specified sieve size fail to intersect the sieve line, either soil, by itself, will satisfy specifications for that sieve size.

g. Proportionng to meet plasticity requirements. An approximate method of determining the plasticity index (PI) and liquid limit of the combined soils will serve to indicate if the proposed trial mixture is satisfactory, pending the performance of laboratory tests. This may be done either arithmetically or graphically. A graphical method for getting these approximate values is shown in figure 7-2. The values shown in figure 7-2 require additional explanation, as follows: Consider 500 pounds of the mixture tentatively selected (30% A, 70% B). Of this

500 pounds, 150 pounds will be material A and 350 pounds material B. Within the 150 pounds of material A, there will be 150(0.52) = 78 pounds of material passing the No. 40 sieve. Within the 350 pounds of material B, there will be 350(0.05) = 17.5 pounds of material passing the No. 40 sieve. The total amount of material passing the No. 40 sieve in the 500 pounds of blend = 78 + 17.5 = 95.5 pounds. The percentage of this material which has a PI of 9 (material A) is (78/95.5) 100 = 82. As shown in figure 7-2, the approximate PI of the mixture of 30% material A and 70% material B is 7.4 percent. By similar reasoning, the approximate liquid limit of the blend is 28.4 percent. These values are somewhat higher than permissible under the specifications. An increase in the amount of material B will somewhat reduce the plasticity index and liquid limit of the combination.

h. Limitations of mechanical stabilization. Without minimizing the importance of mechanical stabilization, limitations of this method should also be realized. The principles of mechanical stabilization have frequently been misused, particularly in areas where frost action is a factor in the design. For example, clay has been added to granular subgrade materials in order to "stabilize" them, when in reality all that was needed was adequate compaction to provide a strong, easily drained base which would not be susceptible to detrimental frost action. An understanding of the compaction which can be achieved by modern rolling equipment should prevent a mistake of this sort.

7-4. METHODS OF APPLICATION OF SOIL STABILIZING MATERIALS

Before describing specific chemical stabilization processes and materials it is desirable that the engineer be cognizant of certain basic methods by which stabilizers are applied. One of two general application techniques is common to all soil stabilization processes; these are (a) admix application and (b) surface penetration application. The particular technique which will be used depends on the stabilization process involved, the type of soil to be stabilized, and the function to be accomplished by the stabilization. Prior to selecting a specific stabilization process, the implications of the two methods of applications to the particular job requirements should be considered.

a. Admix applications. An admix application is the blending of a soil with another material to achieve a uniform mixture of the dissimilar materials. An admix application must always be used where it is necessary to incorporate solid forms of soil treatment materials with an existing or imported soil. It may be used to incorporate liquid materials when it is desired to achieve treatment to a depth greater than that which can be achieved by a surface penetration application. Where stabilization for strength improvement is required, the admix method always must be used in any soil blending operation or to incorporate a chemical stabilizer. It may or may not be required in applying materials for dust control or soil waterproofing. In the application of dust palliatives or soil waterproofers, admixing should be used only where it is essential to provide a layer of 3 inches or greater in thickness. This will seldom be required on nontraffic areas, but may be necessary in traffic areas where rutting will occur under traffic. Admixing can be accomplished either in place or off site.

(1) In-place mixing. In-place mixing is the blending of soil on the site. It is accomplished by loosening (if necessary) the soil surface to a depth approximately equivalent to the desired thickness of the treated layer, adding the soil treatment material in the desired quantity, blending and mixing, and recompacting the mixture by rolling. Equipment which can be used for an in-place mixing operation includes rotary tillers, pulverizer-mixers, graders, scarifiers, disk harrows, or plows. Where conventional stabilization traveling mixers are available, good blending generally will be achieved with two or three coverages over the area to be treated. If the ground surface is easily tilled (e.g., sandy or silty soils) the soil treatment materials can be dispersed on the soil without any initial pulverization. If the ground is

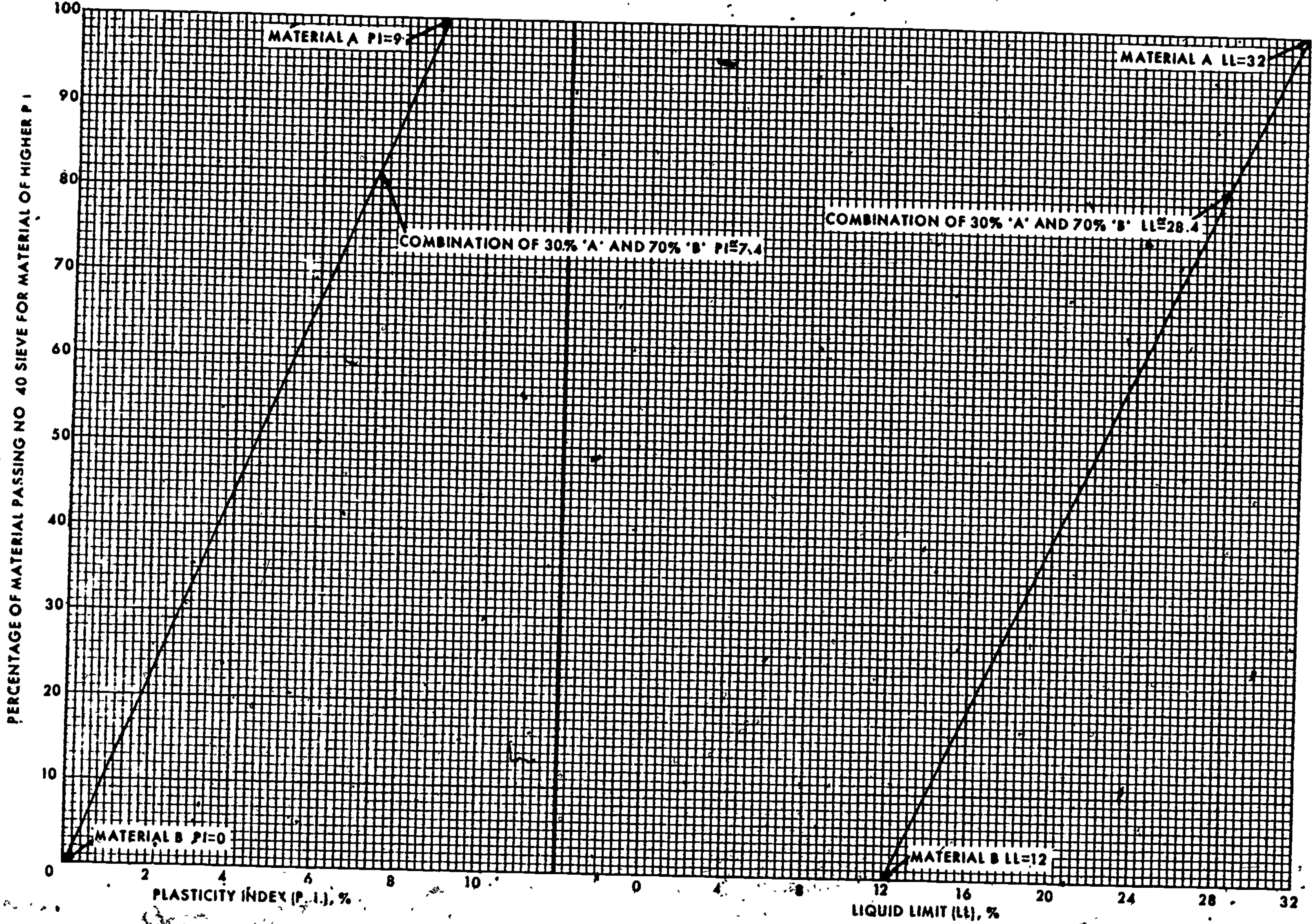

Figure 7-2. Graphical method of estimating plasticity characteristics of a combination of two soils.

hard or firm (e.g., dry clay) it will be necessary to loosen the surface and break down the soil prior to applying the treatment material. Solid materials may be spread either by hand or by mechanical spreaders. Liquid materials should be placed with distributor equipment. Similar procedures can be used with equipment such as disk harrows or plows, but the effectiveness of mixing will generally be very poor. A grader will do a reasonably good mixing job by alternately windrowing and blading a soil-treatment mixture back and forth until the mix appears uniform. The blend is then spread and compacted.

(2) **Off-site mixing.** Off-site mixing is generally used where in-place construction is not desirable and soil from another source would provide a more satisfactory treated surface. Off-site mixing may be accomplished with stationary mixing plants, or can be done expediently by windrow mixing with graders (as described in (1) above) in a central working area. The major disadvantage of any off-site operation is the necessity to transport and spread the mixed material.

b. Surface penetration applications.

(1) A surface penetration application is the placing of a soil treatment material directly on the existing ground surface by spraying or other means of distribution. This is the most rapid and efficient method of application for placing liquid materials, and will be used only for the placement of certain dust palliatives and soil waterproofers. Depending on the material, surface penetration applications may be accomplished by a liquid pressure distributor, a gravity flow water distributor, or by hand hosing or spraying. Hoses or hand sprayers should be used only for small areas, maintenance, or where good uniformity of a treatment is not critical.

(2) The effectiveness of the surface penetration method depends primarily on the viscosity of the treatment material and the permeability of the soil surface. Soils which are predominantly coarse grained are more readily penetrated than soils which are primarily fine grained. Where the soil is extremely clayey, it may be virtually impossible to achieve any penetration with a liquid material. For such a contingency, application as an admixture may be the most desirable solution, or treatment may be accomplished by lightly scarifying the surface to a shallow depth (1 to 2 inches), following by spraying and lightly rolling. **In some soils, such as silts or silty clays, penetration by certain liquid materials can be enhanced by a light initial sprinkling with water.**

(3) Because of the limited depths of penetration which can be achieved by any liquid material, only a thin treated surface layer will be achieved. For nontraffic areas of airfields subjected to the maximum intensity of propeller blast, shallow penetration treatments may be adequate. However, nontraffic areas of airfields subjected to a maximum back blast of jet aircraft will likely be eroded very readily and no penetration material would suffice for long. Shallow-depth penetration treatments of areas subjected to direct tire traffic (either aircraft or ground vehicles) can be worn away by traffic or, if a small area of untreated soil is exposed, can be stripped from the surface by subsequent traffic. Also, if rutting occurs as a result of traffic, the effectiveness of a surface penetration treatment will be destroyed rapidly.

7-5. CHEMICAL STABILIZATION

a. Material used. Of the various materials which can be considered for chemical stabilization, portland cement and lime have the most applicability to the problem of expedient airfield construction where strength development is the primary function. Bituminous materials are not generally suitable for application to initially weak, fine-grained soils; however, a brief description of the process will be provided with respect to its possible utility in special cases. Although many other chemical stabilizing systems have been developed and studied by the Corps of Engineers and others, none have proven to be as generally effective as the more commonly known conventional additives and thus, will not be considered herein.

b. Selection of a stabilizer. The choice of stabilizer and ultimate mix design depends on several factors some of which include: type of soil to be treated, initial condition of the soil, desired strength and durability of the stabilized soil, logistical and cost requirements, and climatic environment in which the stabilizer is used. Certain problem soils may be encountered which are totally unresponsive to stabilization due to their chemical nature. Such soils may not readily be distinguished on the basis of their classification or general physical properties. Very often, previous stabilization experience by others in a particular locale can be used advantageously in establishing the merits of a particular stabilizer and also may provide guidance for an appropriate mix design.

7-6. PORTLAND CEMENT STABILIZATION

a. Description. A soil treated with portland cement will result in a mass generally referred to as soil-cement. Cement influences soil both by reducing its plasticity and, more importantly, developing cementation which imparts early and long-term strength improvement. Portland cement is regarded as the most generally suitable stabilizer for a wide range of soil types, but its effectiveness diminishes rapidly as the plasticity index of a soil exceeds about 15 to 20. This is principally because of the increased surface area of the finer soils and the difficulty of achieving an intimate blend of soil and cement in a wet, cohesive soil. A soil-cement layer is reasonably resistant to attack by water, but does not provide a good wearing surface.

b. Use. In reference to the expedient airfield requirements, cement can be used in all instances to produce the minimum strengths needed for unsurfaced operations or to improve the subgrade under landing mat. Generally, if a soil is responsive to cement treatment, the minimum strengths given in table 7-5 can be achieved in from 1 to 3 days after placement with quantities of cement ranging from 3 to 8 percent by dry soil weight. Compositional soil factors which may influence the extent of stabilization with cement include decayed organic matter or the presence of sulfates. Also, the susceptibility of laterite or lateritic soils to stabilization varies from excellent to poor depending on the organic content and degree of laterization.

c. Mix design. The mix design or determination of the amount of cement necessary for use with a particular soil to achieve a desired result is not ordinarily a simple one. Usually it is based on a rather elaborate series of laboratory tests. Such designs are primarily concerned with satisfying long-range durability requirements and therefore are not pertinent to the expedient airfields problem. As a guide, the following treatments will generally be appropriate for the indicated soils:

Soil type	Approximate treatment % by dry soil weight
Gravels	3-4
Sands	3-5
Silts and clayey silts	4-6
Clays	6-8

Not less than 3 percent cement should be considered for any treatment. To check the amount of cement necessary to stabilize a particular soil or to examine the adequacy of a treatment, the following simple field procedure can be used. Using soil (approximately 50 pounds dry soil weight) obtained from the site and at the existing moisture content, prepare mixes at varying cement contents by blending with a shovel. Place the mixtures in 1-foot-square molds constructed of two-by-fours (open at top and bottom), and compact the mixture as densely as possible by the most convenient means available. Cover the mold with a wet cloth and let stand for 24 hours. After this cure period, the strength of the soil-cement can be determined by testing with the airfield penetrometer.

d. Construction procedures. Soil cement stabilization will be constructed by use of the admixing techniques described previously.

TABLE 7-5. *Summary of Soil Stabilizers for Strength Improvement Function*

(1) Material	(2) Form of material	(3) Applicable soil range	(4) Estimated range of quantity requirements (%) †	(5) Minimum curing time requirements
Portland cement	Powder	Gravels	3-4	24 hours
		Sands	3-5	
		Silts, clayey silts	4-8	
		Clays	6-8	
Lime				
1. Hydrated	Powder	Clayey gravels	2-4	7 days
		Silty clays	5-10	
		Clays	3-8	
2. Quickline	Powder	Clayey gravels	2-3	4 hours
		Silty clays	3-8	
		Clays	3-6	
Bituminous material				
1. Asphaltic cutbacks				
a. RC-70 to RC-800	Liquid	Sands	5-7 ††	1-3 days
		Silty sands	6-10	
		Clayey sands	6-10	
b. MC-70 to MC-800	Liquid	Sands	5-7	1-3 days
		Silty sands	6-10	
		Clayey sands	6-10	
2. Asphaltic emulsions	Liquid	Sands	5-7	1-3 days
		Silty sands	6-10	
		Clayey sands	6-10	

† Based on dry density of existing soil.
†† All quantities listed for asphalts are actual bitumen requirements, exclusive of volatiles.

(1) For construction of soil-cement layers no greater than 8 to 10 inches, the existing soil is scarified to the desired depth of treatment. The soil is then pulverized as best as possible at the prevailing moisture content. If the soil is a wet clay, this operation may be particularly difficult. The approximate amount of cement is then spread over the loosened soil surface either by hand or mechanical means. Since cement will usually be supplied in 94-pound sacks, these can be spotted along the surface in rows of predetermined spacing. This spacing can be determined by estimating the dry density of the existing soil and computing the cement weight requirement for a prescribed area and depth of treatment. The soil and cement are then thoroughly blended by admixing and the resulting mass compacted to the maximum densification possible. Compaction should be accomplished as quickly as possible after completion of the mixing, preferably no longer than 1 hour afterward. Water content will generally not be controllable in expedient construction. Where bearing strength of a clay is the predominant problem, the existing soil moisture content will usually be greater than optimum for compaction. In silts or clayey silts, situations may occur in which water is deficient. In these instances, water should be added to achieve a condition suitable for good compaction. Water is added with a water distributor, and must be thoroughly mixed with the soil and cement mass. If sands are stabilized with cement, the addition of water will almost always be necessary. Following compaction, the surface is shaped, wetted down with water, and allowed to cure for 1 to 3 days.

(2) Where thicknesses in excess of 6 to 8 inches are needed, the soil cement must be placed in multiple lifts. The existing soil must be excavated to a depth of the required thickness minus 6 to 8 inches and removed to the side. The exposed subgrade is then scarified or loosened to a 6- to 8-inch depth, cement is applied in the approximate quantity, the soil and cement are thoroughly blended, and the mixture is compacted to maximum densification. Subsequent lifts of 6 to 8 inches are similarly constructed using the material which has been excavated.

7-7. LIME STABILIZATION

a. **Description.** Lime stabilization is the process of stabilizing soil in which the additive is lime. The term "lime" is used to denote either hydrated lime (hydrous or slaked) or quicklime (anhydrous). There are two chemically different types of lime: (a) calcitic or high calcium lime and (b) dolomitic or high magnesium lime. Although both types of lime have been used successfully for stabilization, the calcitic type (calcium hydroxide or calcium oxide) is the most widely used and known of the two basic types. While there is no reason to suspect that one type is better than the other, if both are available it might be well to evaluate each one by a simple test similar to that described previously for soil-cement. Lime may be obtained in different granular sizes, but is most often used in the "pulverized" state (i.e., essentially 100 percent passing a No. 20 sieve and 85 to 95 percent passing a No. 100 sieve for quicklime; 95 percent or more passing the No. 200 sieve for hydrated lime). Lime influences a soil through the combined actions of ion exchange, flocculation, and a sementitious mechanism.

b. **Uses.**

(1) **Hydrated lime.** Through its reaction with soil, hydrated lime can influence: (a) grain size distribution, (b) soil plasticity, (c) volume change characteristics, (d) compaction properties, (e) strength, and (f) durability. In addition, hydrated lime has a drying effect on a soil which improves its workability. In this respect, hydrated lime has been used successfully as a pretreatment of wet, cohesive soils prior to stabilization with portland cement. Hydrated lime is most applicable for stabilization of clay soils or coarse soils containing excessive clay fines. It has limited application in predominantly silty soils and no application for sandy soils, unless used in combination with a pozzolanic material such as flyash. The use of pozzolanic material will generally not be feasible since quantities ranging from 8 to 20 percent by

soil weight are normally required, and no further discussion of this technique will be given herein. The cementing action of hydrated lime with soil is a slow one and requires considerably more time than that required with portland cement. Generally, 7 days or longer of curing are required before any significant strength changes are achieved. Hence, the use of hydrated lime would be limited only to a situation where time may not be a critical factor. Hydrated lime-soil surfaces will generally be more resistant to the detrimental effects of water, but are abraded readily by traffic.

(2) **Quicklime**. The reactions of quicklime and soil and the functions which can be performed are the same as those with hydrated lime. Quicklime has two significant advantages over hydrated lime in the treatment of very wet soils in that strength benefits will occur very rapidly (in a matter of 2 to 4 hours) and that a very marked drying effect of the soil will be achieved almost immediately upon blending with a wet soil. Since quicklime is simply a more concentrated form of hydrated lime, less additive is required to accomplish a more effective job. Perhaps the major deterrent to the field use of quicklime is its burn hazard due to its caustic nature. Quicklime can produce severe burns quickly when in contact with perspiring or moist skin. Prolonged exposure to hydrated lime may also cause minor skin irritation, especially in hot, humid climates.

c. **Safety precautions.** If proper protective precautions are taken, both forms of lime can be used safely:

(1) **Clothing.** A long-sleeved shirt should always be worn. High shoes or laced boots should be worn and trouser legs should be tied over the shoe tops. A hat and gauntlet type gloves should be worn. Clothing should not bind too tightly around the neck or wrists.

(2) **Face protection.** Safety glasses or goggles with side shields should be worn at all times when working with lime. A protective nose and mouth filter mask should be worn. A protective cream should be applied to all exposed parts of the body when prolonged exposure to lime dust is expected.

(3) **Cleansing.** After a day of exposure to lime, the body should be cleansed thoroughly with soap and water. Generally, the personnel most vulnerable to lime burns are those handling the bags of lime, emptying the bags, and spreading the lime. Under no circumstances should an open bag of lime be dropped on the ground, since the impact can cause a dense cloud of lime dust to rise directly into the face. Should lime dust ever get into the eyes, an immediate, thorough flushing with water should be done and medical attention obtained as soon as possible.

d. **Mix design.** Many factors will influence the developed properties of a soil-lime mixture and thus no single mix design can be specified that would be applicable for a particular soil or situation. Generally, a rapid field procedure similar to that prescribed for soil-cement could be used to establish a suitable mix design. The only exception is in the length of time for curing. For hydrated lime, no strength evaluation should be made before 7 days of curing. For quicklime, strengths may be measured after 4 hours or longer if desired. As a general guide, the following range of treatments may be considered:

	Approximate treatment, % by dry soil weight	
Soil type	**Hydrated lime**	**Quicklime**
Clayey gravels	2-4	2-3
Silty clays	5-10	3-8
Clays	3-8	3-6

e. **Construction procedures.** The procedures used in lime stabilization are the same as those described previously for soil-cement, with the following exceptions or additional considerations. Since the hydrated lime-soil reaction is a slow one, there is no need to have a strict limitation on the elasped time between mixing and compaction. For heavy clays, a time lapse of as much as 24 hours may not be detrimental to the ultimate developed properties; however, a greater compaction effort will generally be required to achieve good densification. Also, overexposure to air can result in carbonation of the lime which is detrimental to its effectiveness. In quicklime, certain reactions occur extremely rapidly and to obtain the benefit of these reactions, compaction should be done no longer than 2 to 3 hours, and preferably sooner, after mixing. In the curing of hydrated lime-soil, the surface should be kept moist during the total time of cure or about 7 days. For the quicklime-soil layer, the surface should be moistened immediately after compaction and as necessary thereafter to prevent surface spalling and peeling due to shrinkage. If desirable, a light application of an oil, low viscosity bituminous cutback, or an asphalt emulsion can be used as a curing seal.

7-8. BITUMINOUS STABILIZATION

a. Description. Bituminous stabilization is the process in which soil and bitumen are combined to achieve a more stable soil mass.

b. Uses. The primary application of bituminous materials in expedient airfield construction is to provide dust control or serve as a soil waterproofer. The use of bituminous materials for these functions will be discussed later. Few problem situations will exist where bituminous stabilization might be useful in satisfying a strength improvement function in expedient airfield construction. However, where a sand or essentially cohesionless soil may pose a problem in areas of limited traffic use, such as shoulders or overruns, bituminous stabilization certainly can be considered as a possible solution. As a general rule, only those soils which can readily be pulverized by mix-in-place construction equipment are satisfactory for bituminous stabilization. Soils having more than 30 percent by weight of particles passing the No. 200 sieve and/or a plasticity index greater than 10 generally are not adaptable to bituminous stabilization. Micaceous soils and those containing high organic content do not respond favorably to bituminous treatment. Some of the major disadvantages to stabilization with a bituminous material are: (a) a comparatively large quantity of bitumen generally is required, (b) the range of soil types which can be treated effectively is limited, (c) the ultimate results are highly dependent on the mix design (too little asphalt will accomplish nothing and too much will result in an overly "rich" mxture); and (d) long curing times are required.

c. Types of bitumens. The most suitable types of bituminous materials for soil stabilization for strength development purposes are cutback asphalts and emulsified asphalts. Of the cutback asphalts, grades RC-70 to RC-800 (rapid curing) or grades MC-70 to MC-800 (medium curing) are most frequently used. Soils in which MC cutback is used should have some fines or natural binder and must be well graded. The RC grades of cutback asphalts are primarily applicable to relatively cohesionless soils. Of the emulsified asphalts, the slow-setting anionic emulsion SS-1 (for cool weather use) or SS-1h (for warm weather use) can be used. Asphalt emulsions will generally be diluted with water in proportions of from 3:1 to 7:1 (water to emulsified asphalt by volume). The grade of bituminous material which can be used depends on the type of soil, method of construction used, and weather conditions.

d. Mix design. The necessary amount of bituminous material to be added to a soil can be determined expediently by the procedures outlined previously for soil-cement. Curing times before testing of the trial specimens will range from 1 to 3 days for the RC and emulsified asphalts and from 3 to 5 days for the MC asphalts. Generally, the amount of bituminous material required increases with an increase in percentage of the finer sizes. Unlike other stabilizers, the quantity requirements of a bituminous material can vary significantly. Normally, no less than a 5 percent treatment by dry soil weight of asphalt (exclusive of voltatiles) will be used and it may be necessary to provide as much as 10 or 12 percent. Since there can be no simple rule-of-thumb guide for the design of a soil-bituminous mixture, trials should be made until a mixture is produced which will give the desired stability.

e. **Moisture content.** For proper use of asphalts, the soil moisture contents must be held to certain maximum limits. For cutback asphalts, the moisture content of the soil normally should be less than 2 percent, although some sands may be handled successfully at moisture contents slightly in excess of this. For emulsified asphalt the moisture content should be such that, when the material is mixed with soil, the particles will retain a uniform coating of bitumen. Some moisture to assist in coating the soil particles is desirable, but the moisture content of the finished mixture should be low enough to permit the emulsion to break prior to compaction.

f. **Construction procedures.** Bituminous materials may be added to soils by mixing in place, or by the use of a traveling plant or stationary plant. The traveling and stationary plants generally will produce the best results. The usual sequence for construction is as follows: (a) pulverize the soil to the required depth, (b) adjust moisture content, if necessary, to that required for proper mixing, (c) add bitumen and mix thoroughly, (d) aerate to proper consistency for compaction, (e) compact with sheepsfoot or rubber-tired roller, and (f) fine grade the surface.

7-9. SUMMARY OF SOIL STABILIZATION SYSTEMS FOR STRENGTH IMPROVEMENT FUNCTION

A summary of the soil stabilization materials and quantity requirements for use for the strength improvement function is given in table 7-5. From knowledge of the areas which may be stabilized, the depths to which stabilization is necessary, and the determined or estimated dry density of the existing soil, the total material requirements can be determined by the information contained in table 7-5. More reliable quantity estimates can be obtained once a specific mix design has been established. A simple formula procedure applicable to the different stabilizers is as follows:

a. For cement and lime:

$$Q = A \times \frac{T}{12} \times D \times \frac{S}{100}$$

where

Q = total quantity of material required, pounds

A = area to be treated, square feet

T = required thickness of stabilized layer, inches

D = dry density of existing soil, pounds per cubic foot

S = percent treatment specified

b. For asphalts:

$$Q = A \times \frac{T}{12} \times D \times \frac{S}{100} \times \frac{100}{R}$$

where

R = percent residual asphalt in cutback or emulsion forms

The quantity R in the formula for asphalts varies with the particular asphalt type and grade. The following values may be used as an estimate of R:

Type and grade	R	Type and grade	R
MC-70	50	RC-250	65
MC-250	60	RC-800	75
MC-800	70	MS-2, SS-1 or	
RC-70	55	SS-1h	60

7-10. DUST PALLIATIVES

a. **The problem of dust.**

(1) **Definition and effects.** A major problem which can exist during operations of aircraft on unsurfaced or limited-surfaced airfields is that of dust. The term "dust" can be defined simply as particles of soil which have become airborne. As a general rule, dust will consist predominantly of soil of particle size finer than 0.074 millimeter (i.e., passing the No. 200 sieve). The presence of dust can have significant adverse effects on the overall efficiency of aircraft by increasing down-time and maintenance requirements, shortening engine life, reducing visibility, and affecting health and morale of personnel. In addition, dust clouds can aid the enemy by

revealing positions and the scope of operations.

(2) **Factors influencing dust formation.** The presence of, or relative amount of dust-size particles in a soil surface is not necessarily indicative of a dust problem or of the severity of dust which will result in various situations. Some of the factors which contribute to the generation, severity, and perpetuity of dust from a potential ground source include overall soil gradation, moisture content, density, and smoothness of the ground surface, presence of salts or organic matter, vegetation, wind velocity and direction, and air humidity. When conditions of soil and environment are favorable, the imposition of an external force to a ground surface will generate dust which will exist in the form of clouds of various density, size, and height above ground. In the case of aircraft, dust may be generated as a result of erosion by propeller wash, engine exhaust blast, jet-blast, and the draft of moving aircraft. Further, the kneading and abrading action of tires can loosen particles from the ground surface which may become airborne.

(3) **Sources of dust.** On unsurfaced airfields the source of dust may be the runway, taxiways, shoulders, overruns, and parking areas. In areas of open terrain and prevailing winds, soil particles can be blown in from distant locations and deposited on an airfield. This can contribute to the dust potential despite adequate initial control measures for the soil within the constructed area. Where such a condition exists and is severe enough to be a problem, it may be necessary to make additional applications of dust palliatives to an airfield site to maintain control.

b. **Dust palliatives.** The primary objective of a dust palliative is to prevent soil particles from becoming airborne. Dust palliatives may be required for control of dust on nontraffic or traffic areas, or both. If prefabricated landing mat, membrane, or conventional pavement surfacing is used in the traffic areas of an airfield, the use of dust palliatives would be limited to nontraffic areas. For nontraffic areas, a palliative thus is needed which is capable of resisting the maximum intensity of airblast impingement by using aircraft. Where dust palliatives are considered for traffic areas, they must withstand the abrasion of the wheels in addition to blast impingement. Although a palliative may provide the necessary resistance against air impingement, it may be totally unsuitable as a wearing surface. An important factor limiting the applicability of a dust palliative in traffic areas is the extent of surface rutting which will occur under traffic. If the bearing capacity is such that the soil surface will rut under traffic, the effectiveness of a shallow-depth palliative treatment could be destroyed rapidly by breakup and subsequent stripping from the ground surface. Some palliatives will tolerate deformations better than others, but normally ruts of 1½ to 2 inches will result in the virtual destruction of any thin layer or shallow-depth penetration dust palliative treatment.

7-11. WATERPROOFING

a. **The problem of water.**

(1) **Effects.** The ability of an airfield to sustain operations depends on the bearing strength of the ground. Although an unsurfaced facility may possess the necessary strength when initially constructed, exposure to water can result in a loss of strength to the detriment of traffic operations. Fine-grained soils or granular materials which contain an excessive amount of fines generally are more sensitive to water changes than coarse-grained soils. Surface water also may contribute to the development of dust by eroding or loosening material from the ground surface which can become dust during dry weather conditions.

(2) **Sources of water.** Water may enter a soil either by the leaching of precipitation or ponded surface water, by capillary action of underlying ground water, by a rise in the level of the water table, or by condensation of water vapor and accumulation of moisture under a vapor-impermeable surface. As a general rule, an existing ground water table

at shallow depths will have a low bearing strength and will be avoided wherever possible. Discussion of the various means to protect against moisture entry from sources other than the ground surface will not be considered herein. In most instances, the problem of surface water can be lessened considerably through good grading, compaction, and drainage practices.

b. **Soil waterproofers.**

(1) The objective of a soil surface waterproofer is to protect a soil against attack by water and thus preserve its in-place or as-constructed strength during wet weather operations. The use of soil waterproofers generally will be limited to traffic areas. There may be instances where it is desired to prevent excessive softening of areas such as shoulders or overruns which are normally considered as nontraffic or limited traffic areas. Also, soil waterproofers may be useful in preventing soil erosion resulting from surface water runoff. As in the case of dust palliatives, a thin or shallow-depth soil waterproofing treatment will lose its effectiveness when damaged by excessive rutting and thus can be used efficiently only in areas which are initially firm.

(2) Many soil waterproofers also function well as dust palliatives, and therefore a single material might be considered as a treatment in areas where the climate will result in both wet and dry soil surface conditions.

7-12. MATERIALS FOR DUST CONTROL AND/OR SOIL WATERPROOFING

a. **Materials classifications.** A wide selection of materials for dust control and/or soil waterproofing is available to the engineer. No one choice, however, can be singled out as being the most universally acceptable for all problem situations that may be encountered. To simplify the discussion of the various types of materials, they have been grouped into five general classifications as follows: Group I, bituminous materials; Group II, cementing materials; Group III, resinous systems; Group IV, salts; and Group V, miscellaneous materials.

b. Table 7-6. A general summary of the various materials and a guide to their application as either a dust palliative or soil waterproofer is given in table 7-6. This summary is considered to represent the best estimate of the applicability of the materials based on existing information.

c. **Information in table 7-6.** The following information is provided in table 7-6:

Column 1. Identification of the material.

Column 2. The usual form in which the material is supplied.

Column 3. The acceptable method of application. Where a material may be applied either as an admixture or as a surface penetration treatment, the preferred and most generally used method is indicated first.

Column 4. The applicable soil range. The range of soils indicated is that which will normally result in reasonably satisfactory results with the particular material. In some instances, the materials may be used outside of this range, but with recognition that the effectiveness which may be achieved will be decreased. In general, the granular type soils (gravel to coarse sand) may or may not require treatment for dust control or waterproofing depending on the amount of fines present. Fine sands (e.g., dune or windblown sands) will probably require a dust palliative, but will not need to be waterproofed. Soils ranging from silty sands to highly plastic clays may require a dust palliative and/or soil waterproofer.

Columns 5, 6, and 7. Show the primary function of the materials as either a dust palliative or soil waterproofer and, where known, the relative degree of effectiveness which can be expected for the indicated function. The applicability to both the traffic area (column 5) and nontraffic or limited traffic area (column 6) is given under the dust palliative function. The waterproofing function is given for the traffic or limited

TABLE 7-6. *Summary of Soil Stabilizing Materials for Function of Dust Control and/or Soil Waterproofing*

(1) Material	(2) Form of Material	(3) Acceptable Application Methods	(4) Applicable Soil Range	Primary Function, Area of Application, and Degree of Effectiveness †: Dust Palliative (5) Traffic	Dust Palliative (6) Nontraffic or Limited Traffic	(7) Waterproofer (Traffic or Limited Traffic Areas Only)	Quantity Requirements ††: (8) gal per sq yd	(9) lb per sq yd	(10) Minimum Curing Time, Requirements	(11) Remarks
Group I: Bituminous Materials										
Cutback Asphalts										
a. RC-70 to RC-250	Liquid	Admix	Gravel to sand	M	V	M	0.18-0.25	1.5-2.0	12-24 hr	All cutback asphalts will require preheating for penetration or admix application.
		Penetration	Gravel to silty snad	M	V	X	0.25-0.50	2.1-4.0	12-24 hr	
b. MC-70 to MC-250	Liquid	Admix	Sand to silt	M	V	M	0.25-0.55	2.0-4.5	24 hr	
c. MC-30 to MC-250	Liquid	Penetration	Gravel to silty sand	M	V	X	0.25-0.50	2.1-4.0	24 hr	
d. SC-70 to SC-250	Liquid	Admix	Sand to clay of moderate plasticity	M	V	M	0.55-0.72	4.5-6.0	24 hr	
		Penetration	Gravel to silty sand	M	V	X	0.25-0.50	2.1-4.0	24 hr	
Road Tars										
a. RT-3 to RT-6	Liquid	Admix	Gravel to clay of moderate plasticity	V	V	V	0.30-0.50	2.5-4.0	Several days	Same comments as above for cutbacks.
b. RT-1 to RT-6	Liquid	Penetration	Gravel to silty sand	X	X	X	0.25-0.50	2.1-4.0	Several days	
Emulsified Asphalts										
a. SS-1 or SS-1h (Anionic)	Liquid	Admix	Gravel to silty sand	X	X	X	0.10-0.50 (diluted)	0.8-4.0	Several hr	Required water for dilution and requires careful control for proper emulsion break. Dilutions up to 5:1 by water are used.
		Penetration	Gravel to silty sand	X	X	X	0.10-0.50	0.8-4.0	Several hr	

† Relative degree of effectiveness is indicated as follows: S = slightly, M = moderately, V = very, X = Applicable, but effectiveness unknown, Flank = not applicable.

†† For all admixture treatments, the quantities indicated are for a 1-in. depth of treatment and assume a compacted dry density of 100 lb per cu ft.

* Proprietary material.

TABLE 7-6. *(Continued)*

(1) Material	(2) Form of Material	(3) Acceptable Application Methods	(4) Applicable Soil Range	Primary Function, Area of Application, and Degree of Effectiveness † — Dust Palliative (5) Traffic	(6) Nontraffic or Limited Traffic	(7) Waterproofer (Traffic or Limited Traffic Areas Only)	Quantity Requirements †† (8) gal per sq yd	(9) lb per sq yd	(10) Minimum Curing, Time Requirements	(11) Remarks
Special Asphalts a. Peneprime *	Liquid	Penetration	Gravel to clay of moderate plasticity	M	V	M	0.25-0.5	2.1-4.0	4-8 hr	Excellent penetration ability; requires heating for spraying
				Group II: Cementing Material						
Portland Cement	Powder	Admix	All	S	S	S	--	1.5-4.0	12-24 hr	Normally used for strength, but will also provide modest benefits for dust control and waterproofing when used in low quantities as a soil modifer.
Lime (Hydrated)	Powder	Admix	Clays of moderate to high plasticity	S	S	S	--	1.5-4.0	12-24 hr	Same as cement above.
				Group III: Resinous Systems						
Lignin	Liquid or Powder	Admix	Sand to clay of low plasticity	S	S	S	--	4.0-8.0	12-24 hr	Benefits may be only temporary since resin is water soluble.
		Penetration	Sand to silty snad	X	X	X	0.50-1.0	4.0-8.0	2-6 hr	
Concrete Curing Compound (with paraffin base resin)	Liquid	Penetration	Silts to clays	S	M	X	0.1-0.2	1.0-2.0	2 hr	Fairly viscous; requires special spray nozzles; forms thick moderately flexible film on surface when cured; curing depends on temperature and humidity.
				Group IV: Salts						
Sodium Chloride	Granules	Admix	Gravel to silt (with fines present)	S	S	--	--	0.4-0.8	0	All salts are corrosive to metal; subject to leaching; rely on absorption of moisture from air to palliate dust. Brine solution forms surface crust.
Calcium Chloride	Powder or flake	Admix	Gravel to silt (with fines present)	S	S	--	--	0.4-0.8	0	

traffic area only (column 7) since there will not be a general need for maintaining strength in nontraffic areas. If such a requirement exists, the materials indicated as suitable for traffic areas can be considered acceptable for use in nontraffic areas.

Columns 8 and 9. Indicate quantity requirements applicable to the soil range indicated in column 4. The lower quantity of the range shown is generally suitable for the coarser soils, whereas the greater quantity is needed for the finer soils. These quantity requirements are given only as a general guide and, in some cases, effective results may be achieved with lesser or greater amounts than those given in the table.

Column 10. Indicates minimum curing time requirements for the various materials. The curing requirements are specifically pertinent to the function as a dust palliative and/or waterproofer in traffic areas. In nontraffic areas, curing is not usually critical and adequate dust palliation can be expected immediately after compacting an admixture or in the case of penetration materials, immediately after complete absorption of the liquid by the soil surface.

Column 11. Self-explanatory.

7-13. GROUP I, BITUMINOUS MATERIALS

Conventional types of bituminous materials that may be used for dust palliative and soil waterproofing include cutback asphalts, road tars, and emulsified asphalts. A patented commercial cutback asphalt called "Peneprime" has been listed separately in this group, since it has unique penetrating and curing characteristics which are not inherent to conventional cutback asphalts. Asphalts are perhaps the most versatile of soil treatment materials, since they can be applied either by admixing (prepared in place or off site) or by surface penetrations, and are generally effective both as dust palliatives and soil waterproofers. They can be used to treat traffic and nontraffic areas, but have a general disadvantage of requiring fairly long curing times and are particularly sensitive to adverse climatic environment.

a. **Cutback asphalts.** Either RC, MC, or SC types of cutback asphalts of low to medium viscosity grades are applicable for dust palliation or soil waterproofing. Cutbacks, when applied as admixtures to depths of 3 inches or more on a firm subgrade, will produce a fairly durable wearing and waterproof surface. They are less satisfactory when applied by penetration, especially in soils containing a high percentage of fines. The best results for either an admix or penetration application are obtained by preheating the asphalt. (The MC-30 grade may be sprayed without heating if the temperature of the asphalt is 80°F or above.) In applying the cutbacks as admixtures, blending should be accomplished until a uniform mix is achieved. For SC or MC grades, it may be necessary to allow the soil-asphalt mixture to aerate for a period of time before compacting to remove the volatiles. Soils should be fairly dry prior to incorporating cutbacks (2 to 3 percent for granular soils and up to 6 or 8 percent for fine-grained soils depending on the amount of fines). For penetration applications, a slightly moist soil surface may assist penetration.

b. **Road tars.** Road tars of grades RT-3 to RT-6 can be used as admixtures in the same manner as cutbacks, to include use as a waterproofer. Grades RT-1 to RT-6 also can be applied as surface penetration treatments. While the road tars will produce excellent surfaces, curing proceeds very slowly and several days or even weeks may be required to achieve a completely cured layer. Road tars would probably be most acceptable for use on nontraffic areas where curing is less important.

c. **Asphalt emulsions.** For application as either admixtures or penetration treatments, the anionic emulsion SS-1 (for cool weather use) or SS-1h (for warm weather use) can be used. The emulsions are normally diluted with from 1 to 7 parts water by volume prior to use. As a general rule, a 3 to 1 water to emulsified asphalt dilution will be satisfactory for most applications. Emulsions have an advantage over cutbacks in that no preheating of the material is necessary for application as an admixture or penetration treatment. For application by penetration, emulsions are relatively simple to use and should provide good results in most instances.

d. **Peneprime.** Peneprime is a special proprietary asphalt product* which is characteristically similar to an MC-30 grade cutback, but which has certain chemical additives to enhance its soil penetrating characteristics. It is suitable for application by penetration to soils which are relatively impervious to conventional cutbacks and emulsion systems. Silts and moderately plastic clays (to PI of 15) can be treated effectively with Peneprime. The rate of curing of Peneprime is rapid and, if used in traffic areas, they can be trafficked in from 2 to 8 hours after placing. As with other cutbacks, heating to a temperature of 130° to 150°F is necessary to permit uniform spraying with an asphalt distributor.

7-14. GROUP II, CEMENTING MATERIALS

The conventional cementing-type stabilizing materials, portland cement and hydrated lime, are primarily used to improve the strength of weak soils. However, when admixed with soils in relatively low quantities of 2 to 5 percent by dry soil weight, a modified soil is achieved which can be more resistant to dusting and will maintain a higher strength than the untreated soil. The use of portland cement or lime as a treatment for dust palliation or waterproofing is recommended only as a very temporary measure and when other more effective materials are not available. Cement is generally suitable for all soil types, provided reasonably good mixing can be achieved, whereas lime is suitable only to soils containing a high percentage of clay. For maximum results, the water content of the cement- or lime-soil mixture should be at or near optimum for compaction.

*Marketed by the Empire Petroleum Company, Denver, Colorado, with various licensed refineries throughout the U. S.

7-15. GROUP III, RESINOUS SYSTEMS

a. **Lignin.** Lignin is a byproduct of the manufacture of wood pulp. It is usually available in a liquid form, but can be procured in dehydrated or powder form. It is most suitable as a binder additive for granular materials and sands, but can be used in silts and clay soils with limited effectiveness. In the liquid form, it can be sprayed on a soil surface as a temporary measure. Lignin is soluble in water and is readily leached from the soil; thus, repeated applications would be required in areas of periodic rainfall.

b. **Concrete curing compounds.** For treatment of silts and highly plastic clays where penetration cannot be readily accomplished, resin systems which will provide a thin plastic coating on the soil surface may be considered. Such compounds are used to seal the surface of freshly laid concrete during the curing phase. Resinous concrete curing compounds are available from many commercial sources. Because of the thinness of the resulting film, such resins are only moderately effective under traffic and applicable only in traffic areas on extremely firm surfaces. Standard asphalt pressure distributors can be used to place such materials, but the conventional spray nozzles need to be replaced with smaller-opening spray nozzles in order to achieve a uniform distribution at the specified low rate of application of from 0.1 to 0.2 gallon per square yard. Also, special care must be taken to clean the distributor after application of the resin to prevent the resin from setting up in the equipment. This can be done by flushing the equipment with gasoline, naphtha, kerosene, diesel fuel, mineral spirits, or other general solvents.

7-16. GROUP IV, SALTS

Salts, particularly sodium chloride and calcium chloride, have been used with varying degrees of success as a soil treatment material. Calcium chloride is deliquescent and, if the relative humidity is about 30 percent or greater, will absorb and hold moisture from the air. Its primary function is as a dust palliative, (not a soil waterproofer) and its usefulness generally will be limited to nontraffic areas. Sodium chloride achieves some dust control by virtue of retaining moisture and also by some cementing due to salt crystallization. Both calcium chloride and sodium chloride are soluble in water and are readily leached from the soil surface; thus, frequent applications will usually be required. Salts may also be applied by surface penetration in the form of a concentrated solution or brine. Frequent applications of a salt solution can ultimately build up a thin crusted surface which will be fairly hard and free of dust. Salt applied as a solution also is susceptible to the leaching of rainfall. Salts are highly corrosive to metal and should never be used where they may result in damage to aircraft.

7-17. GROUP V, MISCELLANEOUS MATERIALS

a. **Water.** As a very temporary measure for allaying dust, a soil surface can be sprinkled with water. As long as the ground surface remains moist or damp, dust can be controlled to a slight degree with water. Depending on the soil and climate, frequent treatment may be required. Excessive quantities should not be applied to clay soil surfaces, since a muddy or slippery surface may result when wet and a more severe dust condition after drying.

b. **Various oils.** Waste oil, bunker oil, crude oil, or other types of oils which may be available can be used as temporary dust palliatives. Such materials will be mostly applicable to nontraffic areas of airfields. Periodic treatment or multiple application by spraying may be necessary. After several treatments, a packed oily-soil crust is usually developed which will have good resistance to abrasion by traffic, and which can be moderately resistant to water. Good penetration can be expected in the more permeable soils; but clayey soils or tightly knit surface may resist treatment. For such surfaces, it may be desirable to lightly scarify the surface, apply about 0.5 gallon per square yard, and lightly compact.

c. **Turf.** Although not indicated in table 7-6, turfs or grasses are extremely effective for preventing dust. Although time will generally not permit the growth of turf at most airfield complexes, every advantage of existing ground vegetation should be taken in planning an airfield. Unnecessary stripping of natural grasses during construction should be avoided wherever possible.

7-18. APPLICATION CRITERIA

a. **Traffic areas.** The traffic area or main elements of an airfield include the runway, parallel and lateral taxiways, and parking aprons. In the proposed use of a soil treatment in the traffic areas, application will be required over the entire design area of the main elements.

b. **Nontraffic areas.** The nontraffic areas refer to those areas adjacent to the main elements which are not normally designed to carry traffic, or at best will support limited traffic. These areas include shoulders and overruns. The primary purpose for treatment of nontraffic areas is to control dust, and the extent to which this is done depends on the width of the treated area which is provided adjacent to the traffic areas. This width varies with the type of aircraft involved. This area is based on the consideration that treatment will be provided to a width adjacent to all main elements equal to the wing overhang of the critical using aircraft when the aircraft is positioned with its wheel at the extreme edge of the traffic areas. Treatment to such a width will assure virtually complete dust control under the most severe dust generating condition. In many cases, adequate but not necessarily complete dust control may be achieved by treatment to widths of about one-half of the width used to establish the area values. Where such control is acceptable, the total area of treatment (and the material requirements) will be reduced by a factor of one-half.

c. **Treatment depth requirements.** Requirements for depth of treatment apply exclusively to admixture applications. For any proposed admixture application, a minimum treatment depth of 3 inches is recommended. A 3-inch depth generally will be adequate for all nontraffic areas; for traffic areas, a depth ranging from 3 to 6 inches should be considered, depending on the ability of the soil surface to resist rutting. This is a function of the soil strength.

d. **Soil strength requirements.** A major factor influencing the applicability of either a surface penetration or admixture treatment is the bearing capacity of the existing or prepared ground surface. This is particularly significant to treatment of the traffic areas. Specific strength criteria by which the performance of various soil treatment materials can be predicted for different levels of usage by different aircraft are not presently available. As a general rule, the effectiveness of a surface penetration treatment will be destroyed rapidly when surface rutting exceeds 1½ to 2 inches. Similarly, the value of an admixture treatment is lost when the depth of rutting approaches the depth of the treated layer. In the absence of specific strength criteria, considerable judgment will need to be exercised by the engineer. As a very rough guide, rutting will be within tolerable limits if the soil strength is two or three times greater than the minimum strength requirements for operations on unsurfaced soils.

SELF TEST

Note: The followng exercises comprise a self test. The figures following each question refer to a paragraph containing information related to the question. Write your answer in the space below the question. When you have finished answering all the questions for this lesson, compare your answers with those given for this lesson in the back of this booklet. Do not send in your solutions to these review exercises.

1. Soils are most frequently stabilized by either mechanical or chemical processes. Give the three primary reasons for stabilizing a soil. (7-1)

2. To achieve the best results from compaction, a soil should be at or near its optimum moisture content. If the water content of a soil is above optimum, what should you do? (7-2b)

3. If the moisture content of a soil is less than optimum, it should be scarified and water added. With the following information, compute the number of gallons of water that should be added. (7-2c(2))

dry unit weight	120 lb/cu ft
in-place moisture content	7%
optimum moisture content	10%
lift thickness	4 inches
lift area	6000 sq ft
5 percent allowance for evaporation	

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4. By use of table 7-1, determine the amount of water that must be added to attain optimum moisture content of a soil under the following conditions. (7-2c(2), table 7-1)

dry unit weight	95 lb/cu ft
in-place moisture content	4%
optimum moisture content	8%
lift thickness	6 inches
lift area	1000 sq yd
use no allowance for evaporation	

5. If it has been determined that 2,500 gallons of water must be added to attain optimum moisture content in an area of 1000 square yards with a lift thickness of 6 inches, how many gallons would be required if 0.2 inch of rain fell immediately after the in-place moisture content was determined? (7-2c(2), table 7-2)

Special situation for exercises 6 through 10. Two materials are to be blended, material B, the in-place soil, and material A from a nearby borrow source. The mechanical analysis, liquid limit, and plasticity index of each plus the specifications are given in table 7-7. Note figures 7-3 and 7-4.

TABLE 7-7. *For Use with Exercises 6 Through 10*

Mechanical analysis			
Sieve designation	Percent passing, by weight		
	Borrow "A"	Existing "B"	Specified
1½-inch	100	100	100
1-inch	90	75	70-100
¾-inch	85	50	60-90
⅜-inch	80	45	45-75
No. 4	75	40	30-60
No. 10	45	15	20-50
No. 40	40	10	10-30
No. 200	30	5	5-15
Plasticity characteristics			
	Percent by weight		
Liquid limit	30	15	Not more than 28.
Plasticity index	6.5	0	Not more than 6.

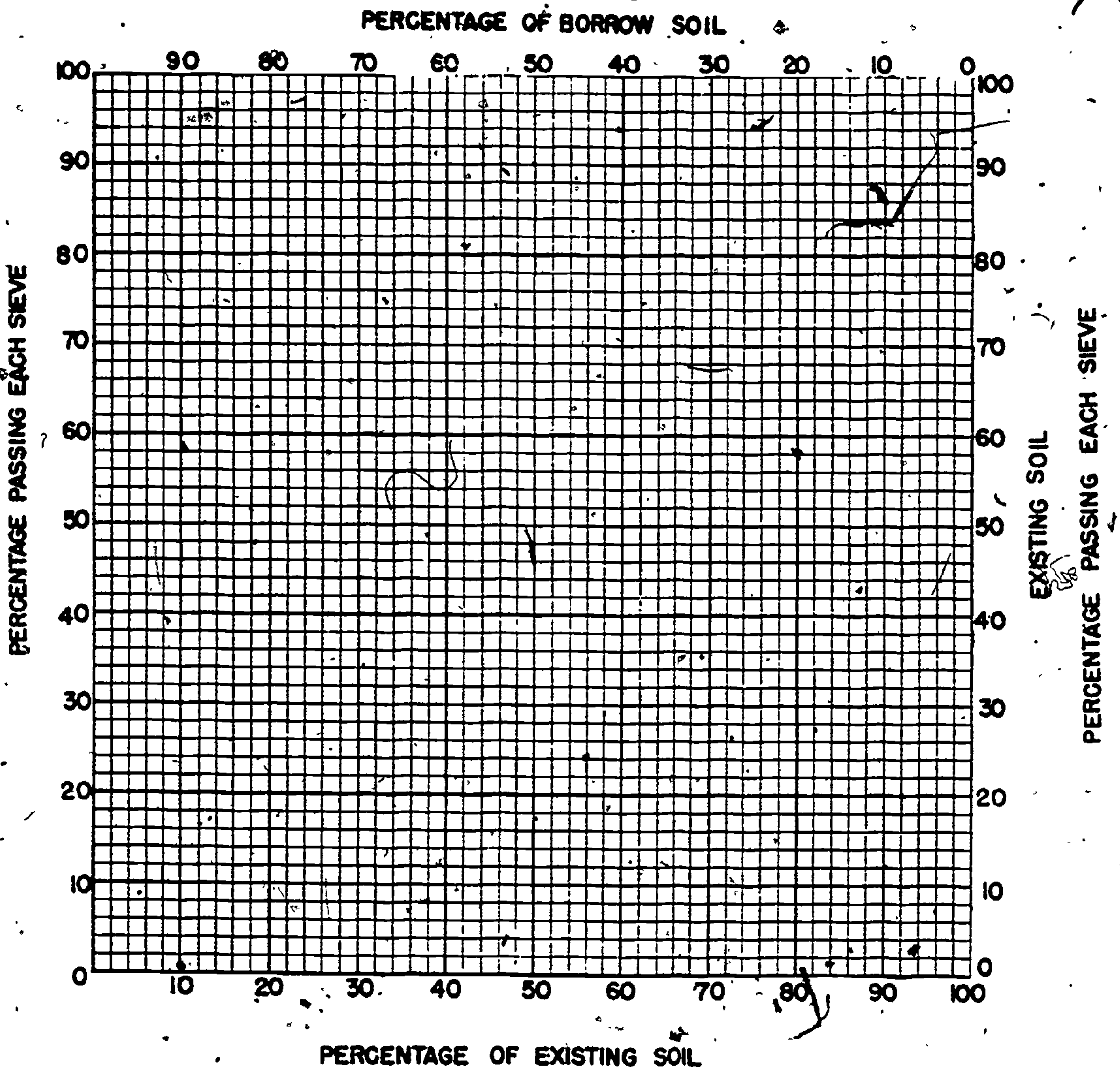

Figure 7-3. For use with table 7-6 and exercises 6 through 8.

6. What is the maximum percentage of borrow soil the mixture can contain and still meet the gradation requirements for No. 4 sieve? (7-3f(4), fig 7-1c)

7. What is the maximum percentage of borrow soil that the mixture can contain and still meet the gradation requirements? (7-3f(4), fig 7-1d)

8. What is the maximum percentage of existing soil the mixture can contain and still meet the gradation requirements? (7-3f(5), fig 7-1d)

9. What is the plasticity index (approximate) of a mixture of 30% borrow and 70% in-place soil? (7-3g, fig 7-2)

10. What is the liquid limit (approximate) of a mixture of 30% borrow and 70% in-place soil? (7-3g, fig 7-2)

11. The two general application techniques common to all soil stabilization processes are (1) admix application (2) surface penetration. When stabilization for strength improvement is required, what must be done? (7-4a)

12. A soil treated with portland cement will result in a mass generally referred to as soil-cement. Portland cement's effectiveness as a stabilizer will rapidly diminish as the plasticity index of the soil exceeds what point value? (7-6a)

13. Lime stabilization is the process of stabilizing a soil in which the additive is lime. Hydrated lime is most applicable for stabilization of which type of soils? (7-7b(1))

14. Under what circumstances does quicklime have an advantage over hydrated lime in the stabilization of soils? (7-7b(2))

15. It has been perviously stated in this lesson that there are three primary reasons for soil stabilization. Which of these reasons are well satisfied by the use of bituminous materials? ~~(7-8b)~~

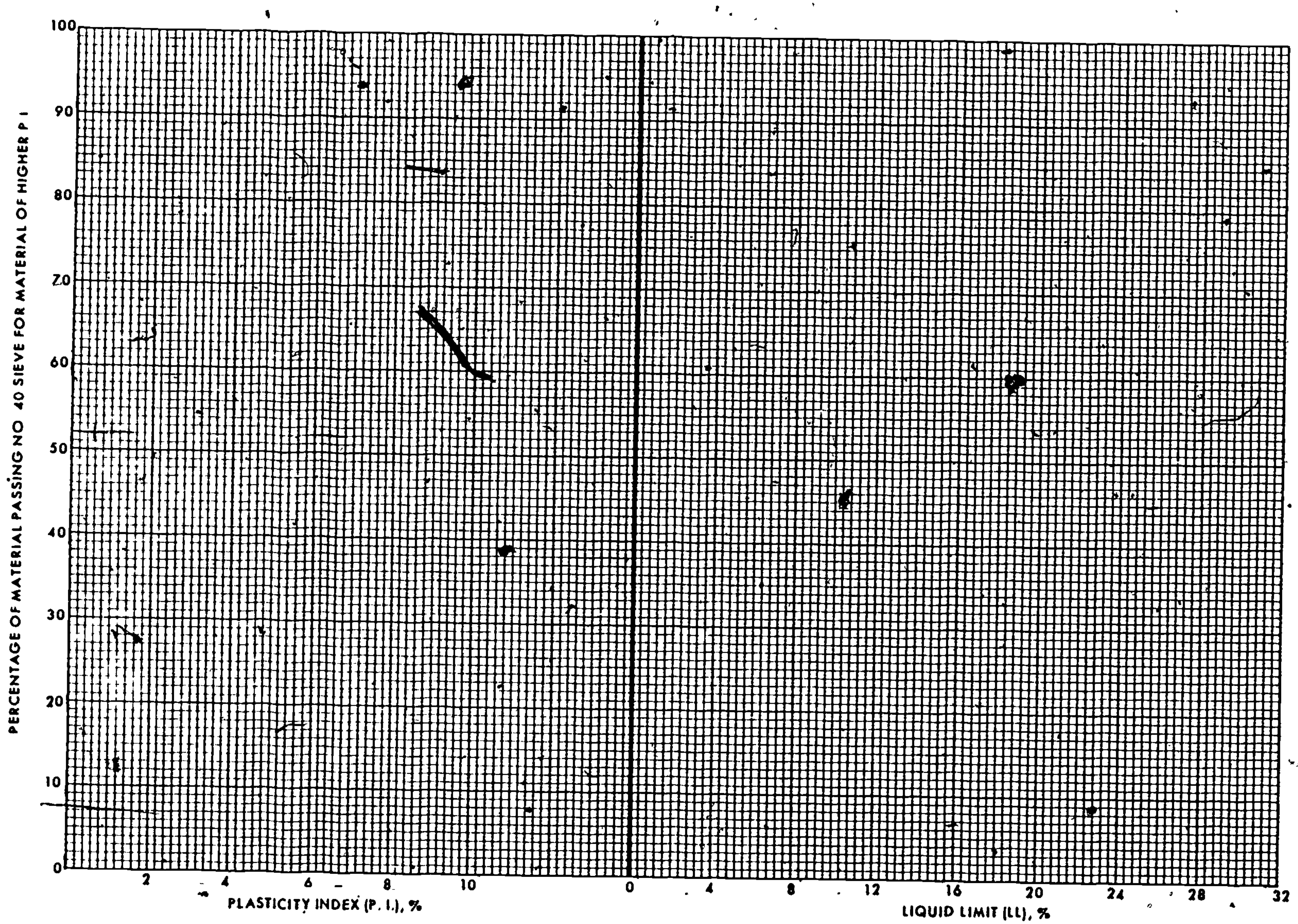

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Figure 7-4. For use with exercises 9 and 10.

16. In those circumstances where bituminous materials are effective in strength improvement of soils, how is the most effecitve soil-bituminous mixture determined? (7-8d)

17. Before using chemical stabilization, one must know how much stabilizer is needed. How many pounds of cement are needed to treat a 4,000-square foot area of soil (90 lb/cu ft dry density) to a depth of 6 inches with a 3 percent treatment? (7-9a)

18. Before using chemical stabilization, one must know how much stabilizer is needed. How many pounds of MC-70 are needed to treat a 5,000-square foot area of soil (90 lb/cu ft dry density) to a depth of 6 inches with a 6 percent treatment? (7-9b)

19. Give a definition of the term "dust" and an indication of the size of dust particles. (7-10a(1))

20. The objective of waterproofing a soil surface is to preserve its in-place or as constructed strength during wet weather operations. What types of soils are most susceptible to changes of water content? (7-11a(1))

21. Referring to table 7-6, what material would you select as a soil waterproofer if curing time was not critical? (7-13b, table 7-6)

22. What is the most common method of applying peneprime as a waterproofer? (7-13d, table 7-6)

23. Give a brief statement concerning the use of calcium chloride as a soil stabilizer. (7-16)

24. Explain the recommended method of application of waste oil, crude oil, etc. as dust palliatives, to a clayey soil. (7-17b)

25. What is one of the principal means of determining depths to which admixture treatments should be accomplished? (7-18c)

LESSON 8

CONSTRUCTION METHODS AND PRACTICES

CREDIT HOURS ------------------------ 1

TEXT ASSIGNMENT ------------------- Attached memorandum.

MATERIAL REQUIRED ----------------- None.

LESSON OBJECTIVE ------------------ Upon completion of this lesson you should be able to accomplish the following in the indicated topic areas:

1. **Reconnaissance and Site Selection.** Carry out reconnaissance and site selection in a logical and orderly sequence utilizing maps, aerial photographs, intelligence reports, etc. as available.

2. **Planning and Construction.** Plan and supervise the construction of subgrade, subbase, and base courses keeping in mind urgency of completion, economy of manhours and materials, availability and utilization of equipment, and utility of the finished product.

ATTACHED MEMORANDUM

8-1. RECONNAISSANCE

a. **Sources of information.** Frequently, information pertinent to the areas or sites selected for reconnaissance may be obtained from various sources such as: military intelligence reports; strategic and technical reports prepared by the Office, Chief of Engineers; national intelligence surveys (NIS); maps (geographic, geologic, topographic, soil, vegetation, weather, road, etc.); G2 and air force periodic intelligence reports; and aerial photographs.

b. **Reconnaissance requirements.** Where a large area is involved, examination of a mosaic of airphotos offers the best means of identifying possible airfield sites. Even air reconnaissance cannot provide the broad view and opportunity for study and restudy of the topography, drainage, and vegetation possible through the use of photograph mosaics. Airphoto coverage should provide the overlap of adjacent photographs (60 percent or more) needed for stereoscopic examination. When sites have been tentatively selected, large-scale photographic coverage should be obtained to permit careful study of topography, vegetation, drainage, and cultural features. Whenever possible, photoplotting equipment should be employed to provide close contour interval coverage of airfield sites. This will permit determination of the quantities of earthwork involved.

c. **Air reconnaissance.** Air reconnaissance of possible airfield sites selected from study of the airphoto coverage furnishes an opportunity to visualize the construction problems, camouflage possibilities, and extent of surface access routes. An air reconnaissance party generally will consist of a pilot and an engineer observer. The pilot should be familiar with operational characteristics of the aircraft to be used at the proposed airfield so that he can accurately assess such flying problems as approaches, physical obstructions, and mental hazards. The engineer observer preferably should be the engineer officer responsible for the ground reconnaissance and therefore qualified to assess design and construction problems.

d. **Ground reconnaissance.** It is emphasized that answers to most questions relative to site selection will be found on the ground. Ground reconnaissance is required to provide definitive information regarding: soil types, moisture conditions, and existing strength; possible sources of higher quality or select materials suitable for use as strengthening layers; clearing requirements; microgeometry of the natural surface; evidence of flooding or high water from nearby surface streams or rivers; ground-water conditions; evaluation of existing airfield facilities; condition of access roads (including bridges, culverts, etc.) and other modes of surface transportation.

8-2. PRELIMINARY SOILS EXPORATION

a. **General.** Prior to preparation of drawings and specifications for a paving project, the engineering division conducts a detailed subgrade investigation program. This program usually consists of sufficient exploratory test pits or borings in the area of the proposed construction to determine the general suitability of the subgrade soils. If it is evident that borrow will be necessary for construction of an acceptable subgrade, the investigation includes the proposed borrow areas. Sufficient samples are obtained from the exploratory test pits or borings, and laboratory tests are conducted for the design of the flexible pavement. The general characteristics of each material are determined and the soil types classified in accordance with the Unified Soil Classification System.

b. **Tests.** The types and number of tests conducted during the subgrade investigation prior to design will depend upon the characteristics and locations of the subgrade soils and the soils in the borrow areas. The tests performed will generally include: classification (washed mechanical analysis, liquid limit, plastic limit, volumetric and linear shrinkage), in-place moisture content, in-place density, compaction, and California Bearing Ratio. Although performed primarily for design purposes, the results should be available for field use when construction begins.

8-3. PRELIMINARY SURVEY

The preliminary survey is a detailed study of a location tentatively selected on the basis of reconnaissance, survey information, and recommendations. It consists basically of running a traverse along a proposed route, recording topography, and plotting results. In the case of roads, it may be necessary to run several preliminary surveys if the reconnaissance indicated more than one route as being feasible. For roads, the route centerline is established, stationed, and profiled; horizontal and vertical control points (reference points) are set; and cross sections are taken to enable rough calculations of the earthwork involved. (Sometimes cross sections may be taken during the reconnaissance survey if the conditions warrant.) If the best available route has not already been chosen, it should be selected at this time. The airfield survey consists of establishing controls or references, noting terrain features, measuring glide angle clearance, making soil profiles, and investigating drainage patterns and approaches. The final centerline will be accurately established during this survey.

8-4. SITE SELECTION

a. **Importance.** In the theater of operations, for many airfields the need will be critical, and completion of planning and construction within tactical time limitations will be difficult at best. Since no amount of clever design can possibly compensate for a bad construction site, it is emphasized that site selection must be made with the utmost care. The engineer officer must visualize site requirements relative to engineering feasibility and adequacy for aircraft operations and tactical suitability, each in its proper perspective. He also must establish priority ratings for alternate sites where construction can be accomplished by available forces within the time available. In certain situations, even the "best" site in a particular area will not be acceptable because it cannot be developed in time. The engineer officer must report such situations to those responsible for operational planning so that adjustments can be made to gain more construction time or to substitute other resources for the air mission.

b. **General site requirements.** In general, all construction sites must be: sufficient in size for intended mission; accessible to logistical supply and services; feasible from an engineering standpoint; adequate from an aircraft operations viewpoint; and suitable from a tactical viewpoint.

c. **Procedures.** Procedures for site selection are covered generally, no detailed discussion is presented herein. Planning for rapid airfield construction often demands unusual dependence on airphoto interpretation and study. By estimating construction effort, it is possible to quickly eliminate most geographic areas within the tactical boundaries where the severity of vegetation, topography, and soil conditions (alone or in combination) preclude completion of construction within acceptable time limits. The engineer can then concentrate his efforts and limit ground reconnaissance to those areas that can be developed with the forces available by the prescribed completion date.

8-5. CONSTRUCTION OF SUBGRADE

a. **General.** The design of a flexible pavement is primarily based on the supporting or bearing capacity of the subgrade soil. The same is also basically true for rigid pavement, although to a much lesser degree. Compaction generally produces substantial increases in the bearing capacity of the subgrade soils and develops a more uniform foundation upon which both the subbase and base course soils, for the flexible pavements, are constructed. Both research and experience have proven that it is necessary to compact all subgrade soils underlying road or airfield pavements to the highest densities practical to prevent detrimental settlement under the traffic of heavy wheel loads. Therefore, the specified field density must be obtained if flexible and rigid pavements are to give satisfactory service. However, it should be borne in mind that at high moisture contents, soils can be overcompacted to the extent where pore pressure can develop and failure occur before the pavement is placed. This of course should be avoided and when observed, should be corrected.

b. **Construction procedures.** The construction equipment should be in such number, type, and condition as to cover the job adequately and permit completion of the project within the time limit required. The area is excavated to the depth required by the plans and/or to satisfactory material. Where the volume of fill required exceeds the volume of cut available, or where cut material is unsatisfactory for fill, material will be excavated from a satisfactory borrow area and used for backfill. Excavation should be planned, and borrow areas located to hold the haul distance to a minimum. Adequate drainage should be maintained at all times. Cutting or filling to temporary or improper elevations should be held at a minimum as this may impound rainwater and seriously impede progress of the work.

c. **Moisture content.** Studies have shown that cohesive soils, compacted well on the wet side of optimum, as sometimes necessary for highly swelling soils, have low strengths throughout their life as subgrades in pavements because drying usually does not occur. Also, cohesive soils compacted well on the dry side of optimum have low strengths when subjected to future saturation. It is generally the policy to compact cohesive soils, except those having high swell, slightly on the dry side of optimum as determined in the laboratory compaction test in order to obtain maximum strengths (for a given density) under future saturation. Occasionally, soils will be found where the field optimum differs from the laboratory optimum. Where this can be established, the field optimum should govern. Swelling soils are compacted at the moisture content which will, for the required density, result in the least amount of swell on future saturation. This moisture content is usually higher than the laboratory or field optimum. Cohesionless sands and gravels compact best when either air-dry or almost saturated. Since the air-dry condition can rarely be obtained, they are usually compacted in as wet a condition as possible.

d. **Lift thickness.** Studies have shown that the density within a lift after compaction is highest at the surface of the lift and decreases with depth in the lift. The lift thicknesses mentioned previously are about the maximum that can be used with the respective pieces of equipment, and still obtain an average density throughout the lift that will be within the specification requirements. Since the density within the lift is highest at the top and lower at the bottom, it can be seen that an increase in density can be obtained by decreasing the lift thickness. Also, it can be seen that lifts that are thicker than specified are detrimental from the standpoint of obtaining the specified average compaction. Lift thickness is usually specified in terms of compacted depth, and loose depth varies for different materials, but is generally in the order of 1.25 times the compacted depth.

e. **Compaction.** All rollers that will be used in compaction should be inspected well in advance of the start of compaction operations to determine that they meet all specification requirements and that they are in good working order. Also, a plan for applying uniform coverages should be worked out with the construction unit because uniform density cannot be obtained unless uniform coverages are applied. If possible, the rollers should not turn on the section being compacted as the turning action tears the surface. Also, coverage is not uniform on the turns. Where turning is necessary, the radius of the turns should be as large as possible and extra passes should be made on the turns so that the turning area receives the necessary number of passes.

f. **Control tests.** The primary tests that are made for the control of the construction of the subgrade are moisture and density determinations. Moisture tests are made to determine if the soil has the proper moisture content for rolling. Both moisture and density tests are made after rolling to determine if the material was placed at the proper moisture and if the required density was obtained. Since the optimum moisture and the required density vary with soil type, it is necessary to make compaction tests for each soil type prior to the start of operations and sufficient classification tests during operations (washed mechanical analysis and Atterberg limits) to identify the soil type so that the density can be compared to the proper laboratory compaction curve. Also, in certain cases, CBR tests are made to determine if the proper strengths are being obtained. Control tests are made to furnish a basis for the judgment of the construction foreman and the inspector during operations and to insure that the completed subgrade will comply with specification requirements. To be effective, therefore, the results of the control tests must be reported while the operation is in progress. Data reported after completion of a subgrade operation and after the subgrade has been buried under the base or subbase, are of no value in controlling construction. Normal procedures are for the inspector to notify the project laboratory that tests are needed. The project laboratory performs the tests and reports the results directly to the inspector as soon as they are completed.

g. **Record tests.** Record tests are those which represent the as-built conditions. Generally, the tests which are made for control purposes can be used for record purposes provided proper notes are made on the test sheets. For example, if the tests made following compaction of a lift indicate that the moisture was within the desired range and that the density was equal to the specification requirements, these tests could be considered as record tests, and should be so noted. If, however, the density did not meet the specification requirements, additional testing would be necessary following rerolling and the earlier group would not be the record tests for that area.

h. **Maintenance.** After the last lift of the subgrade has been finished, checked to final planned grade, and accepted, one must maintain this finished subgrade until base course operations have covered it. He is still liable for all maintenance and repairs; and it is his responsibility, and to his advantage, to see that base course operations are "ready and waiting" upon acceptance of the subgrade.

8-6. CONSTRUCTION OF SUBBASE

a. General. The subbase material is a selected, intermediate quality material placed between the subgrade and base so that the flexible pavement design thickness may be achieved as economically as possible. The load distribution function of this material requires that it be constructed according to specification requirements so that full benefit of its strength may be obtained. Compaction requirements must be enforced.

b. Material. Since the primary reason for using a subbase is to reduce the overall cost of the pavement, a material should be used that can be obtained locally, or from borrow areas near the job, and at a cost somewhat less than that of the regular base course material. It may very from a light clay to a coarse gravel, but should always be capable of developing a load bearing capacity greater than that of the proposed subgrade. The material in most local borrow areas is nonuniform and frequently is stratified. Hence care should be exercised to see that it is mixed as thoroughly as possible during excavation and that it does not become segregated during placement.

c. Construction procedure. The construction method used for the subbase should vary with the material type. A fine-grained material will generally be processed, placed and rolled in accordance with the procedures outlined in paragraph 8-5 above. The coarser subbase material will be handled as outlined in paragraph 8-7 below. In general, the same requirements apply, such as uniform distribution before compaction, layer thickness after compaction, and uniform mixing.

8-7. CONSTRUCTION OF BASE COURSE

a. General. The base course in modern flexible pavements must be a processed aggregate that can be so compacted that it will not consolidate appreciably under heavy traffic. In no case should the density be less than that specified in the contract specifications.

b. Source of material. The specifications generally require one to submit samples of the proposed base material for approval at least 30 days prior to use. Although it is not the usual policy of the Corps of Engineers to designate a material source, an investigation of material sources should be carried out during the design stage to establish definitely that materials meeting the design criteria are available. During the preliminary investigation, or before approval of the aggregate source is given, sufficient test holes should be made to determine the quality of the source and the amount of material available. Often one may propose the use of an unknown source. It will be his responsibility to submit sufficient samples to permit determination of the quality of materials from this source. These samples should be submitted well in advance of the use of the material.

c. Method of base course construction (where plant mixing is not required), roads and airfields for light traffic.

(1) **General.** The following described method may be used where the base course material is produced as a single material and does not require an admixture. The base course material is hauled in approved pneumatic-tired vehicles, and dumped on the subgrade. These loads are dumped at a predetermined spacing to provide the required compacted thickness of base course. Control of dumping will eliminate excessive movement of material, and the section will be brought to uniform grade with the least amount of effort.

(2) **Windrowing.** The material dumped on the subgrade will be uniformly bladed into windrows. Moisture is added to the material as it is being moved into the windrows by the water truck, which follows immediately behind the blade. The addition of water reduces segregation and also provides moisture for compaction. Several passes will be required to move the material into windrows and two blades greatly speed this operation. As soon as the windrowing is completed, an inspection should be made to determine the uniformity of the material, and samples should be taken for testing in the project laboratory.

(3) **Compacting.** The material should be bladed to a uniform section in light lifts. The blade is followed by water truck when additional moisture is needed and by the com-

paction equipment of the steel-wheel, vibratory, or sheepsfoot type. Rigid control of addition of the moisture and rolling should be maintained at all times. The weight of the rollers must be adequate to obtain the desired compaction, and the number of water trucks and rollers must be sufficient to achieve the desired results. The blading, watering, and rolling operation continues until a uniform section is obtained. By maintaining control of the dumping of the material, windrowing, and lifts, the section will be uniform when the last lift is bladed. A few passes of very light blading will bring the section to a uniform grade. The pnuematic-tired rollers should continue rolling during this operation, or a heavy pneumatic-tired roller may be used at this time. This could also serve as tests of proof-rolling if desired. Usually when the section is brought to grade, the blade is carrying coarse aggregate without fines. This coarse aggregate should be evenly distributed over the section and rear-wheeled (rolled with rear wheels of three-wheel roller) into the base course by a 10-ton steel-wheeled roller.

d. **Finished surface.**

(1) **General.** The nature of the finished base course surface is one of the determining factors in the smoothness of the surface of the finished pavement. In particular, it must be recognized that if the finished base does not conform to the specified grade when tested with a straightedge, the finished pavement probably will not do so. The surface of the base should be reasonably smooth and aggregate faces should be showing in the surface and conform to specified requirements.

(2) **Smoothness test.** The surface of each layer of the base course should not show any deviation in excess of 3/8 inch when tested with a 10-foot straightedge applied parallel with and at right angles to the center line of the paved area. Any deviation in excess of this amount should be corrected by the contractor or construction agency by removing material to the total depth of the lift and replacing with new material, and compacting as specified above.

(3) **Slush-rolling.** The ultimate purpose of slush-rolling (rolling with enough water to produce a slushy surface) is merely to smooth and tighten the surface rather than to secure additional compaction of the base course. Slush-rolling should be permitted on a cured base course only. If the surface is generally satisfactory but there are some relatively large areas which require slushing, do not slush the entire area of base course; confine this treatment to the rough areas alone. Slushing makes fines and brings more fines to the top. An excessive layer of fines is structurally weak and will not produce adequate bond. In general it may be stated that **slush-rolling should not be used on a high type base course material** and should be employed only on other types of base courses when required by the specifications.

(4) **Wet-rolling.** All base courses require a final finish of the surface; however, the high-type base courses, with their improved requirements for gradation, liquid limit, and plasticity index, should obtain such a finish automatically after the final compaction and/or proof-rolling. For less critical base courses, or where deemed necessary by the construction agency, wet rolling as well as slush-rolling may be used to obtain a final finish of the surface. Each method has its strong points and in some cases the particular job may require a combination of the two.

e. **Edges of base course.**

(1) **With side forms.** After removing the forms, the space between the compacted aggregate and the earth backing shall be filled with approved material, in such a quantity, and to the height, that will compact to the final thickness of the course being constructed and allow at least a 1-foot width of the shoulder to be rolled and compacted together with the base course material.

(2) **Without side forms.** Approved material should be placed along the edges of the base courses in such quantity as will compact to the thickness of the course being constructed, or, when the course is being constructed in two or more layers, to the thickness of each layer of the course; allowing in each operation at least a 1-foot width of the shoulder to be rolled and compacted

simultaneously with the rolling and compacting of each layer of the base course.

f. Curing and maintenance. The base course should be adequately cured before priming and paving are attempted. If it is too wet, it will not take the prime properly. Also, if there is too much moisture in the base, this moisture tends to come out, particularly in hot weather, and the prime may strip from the base under construction traffic. Rain tends to strip prime from a base that was too wet when primed. Heavy rains may strip a properly primed base to some degree, but the tendency is not nearly so great as on improperly cured bases. It is not possible to designate exactly the lowest acceptable moisture content for the upper portion of the base course prior to priming as this depends on too many variable factors, but in general it should not exceed one half of the optimum rolling moisture. On the other hand, it is undesirable for the base to dry out completely as cracks may develop and a heavy rain will then cause swelling and loss of density. Therefore, depending upon condition of the base, length of time before priming operations begin, and weather conditions, it may be necessary to maintain the base by light sprinkling.

g. Tests.

(1) **General.** The engineering division laboratory will have performed the acceptance tests on the finished base material during the initial operations of the crushing and screening plant. The project laboratory, however, tests the base throughout the construction period. Tests of the base material must be made, therefore, on samples from (a) the crushing and screening plant to assure production of a material that meets the grading requirements of the specifications, (b) the loosely spread base course prior to rolling to assure uniformity of the gradation and for moisture control, and (c) the base course during and after compaction for control of the construction methods and to obtain moisture and density data on the compacted base course for conformance with specification requirements. The tests performed by the project laboratory during construction of a base course are: gradation of material, soil (Atterberg limits) tests on that portion of material passing the No. 40 sieve, modified AASHO compaction, moisture content, field density, and in some cases, field in-place CBR.

8-8. AVAILABLE EQUIPMENT

a. General. Numerous types of equipment are used in subgrade construction. Some types work well in one material but are entirely inadequate in others. It may prove necessary to use several types of equipment in conjunction with each other on one project. The equipment must be capable of handling the material needed for an entire depth of lift in one operation. The following list of equipment, with pertinent comments, may be of help to the engineer in determining whether the construction forces have the right amount and types for the material to be handled on his particular job.

b. Pulverizing, mixing, and blending equipment, pulvimixers and rotatillers. These specialized pieces of equipment are well adapted to pulverizing and blending materials, and for the uniform incorporation of moisture. Construction forces should have them or be able to get them. The equipment available is: tandem gang disks, gang plows, spring-tooth harrow, and heavy duty cultivator.

c. Watering equipment. The pressure-tank water truck gives a more uniform flow of water, although the gravity-type tank is in most general use. The controls for the spray bars should be in the cab. Spray bars should not leak. To insure uniform water coverage the truck should be moving at uniform speed from the time immediately before the sprays are turned on until after the sprays are turned off.

d. Compaction equipment. The following compaction equipment is available: heavy pneumatic-tired rollers, sheepsfoot roller, pneumatic roller, steel-wheeled roller, and impact and vibratory rollers.

SELF TEST

Note: The following exercises comprise a self test. The figures following each question refer to a paragraph containing information related to the question. Write your answer in the space below the question. When you have finished answering all the questions for this lesson, compare your answers with those given for this lesson in the back of this booklet. Do not send in your solutions to these review exercises.

1. A reconnaissance is required for tentative construction sites. When a large area is involved, what is the best means of reconnaissance? (8-1b)

2. Before the drawings and specifications are made for a project a subgrade investigation is made. How is the general suitability of the subgrade soil determined? (8-2a)

3. The preliminary survey is a more detailed study of a tentatively selected location for construction. Of what does it normally consist? (8-3)

4. Planning often demands unusual dependence on airphoto interpretation and study. What is a quick way to eliminate most tentatively selected sites? (8-4c)

5. Subgrade soils generally should be compacted to the highest density practical. Under what particular condition would this not be true? (8-5a)

6. The loose lift depth must be controlled in order to attain uniform compaction of a lift from top to bottom. What is generally considered to be the relationship of the thickness of the loose lift to the thickness of the compacted lift? (8-5d)

7. When a subbase course is used, it is placed between the subgrade and the base course. While the material used may vary from light clay to coarse gravel, there is a specific criteria concerning its load bearing capacity. What is it? (8-6b)

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8. After compacting, the base course must be brought to grade. What is done with the coarse aggregate retained on the blade? (8-7c(3))

9. The base course should be primed before paving. What is the rule of thumb for the lowest acceptable moisture content for the upper portion of the base course prior to priming? (8-7f)

10. What is the most important factor in obtaining an even spread of water when using a water truck? (8-8c)