BASIC SOIL MECHANICS

WORKBOOK

FAR WEST STATES

UNITED STATES DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

USDA-SCS-PORTLAND, OREG. 1975

SOIL MECHANICS

BY
J. C. Stevenson
and
J. R. Talbot

Engineering and Watershed Planning Unit
Technical Service Center
SOIL CONSERVATION SERVICE
Portland, Oregon

A Work Book
Prepared for Soil Mechanics Training

February 1974
Revised October 1975

This manual contains basic soil mechanics information dealing mainly with engineering classification systems and volume-weight relationships. It was assembled specifically for training purposes and for future reference by trainees. It is intended for use as a basic text for the following courses conducted by the West Employee Development Unit.

- Classification, Identification and Engineering Behavior Characteristics of Soil

- Construction Inspection (sampling and testing portion)

- New Professional Engineers and Geologists Training (NPEG) (Soil Mechanics Portion).

- Qualitative Soil Mechanics and Site Evaluation.

An attempt has been made to take the best procedures from several sources and provide clear, concise information for classifying soils in the engineering systems and making basic soil mechanics calculations. The information has been updated to present accepted criteria and procedures adopted by the Service.

Table of Contents

FIGURES

TABLES

SECTION 1. INTRODUCTION

A. GENERAL

To successfully practice the art of soil engineering, an engineer must have a knowledge of precedents, familiarity with soil mechanics, and a working knowledge of geology.

Soil has been used as a foundation and construction material throughout the history of mankind. By the time the scientific method became recognized as a fruitful approach to the solution of engineering problems, large buildings, bridges, dams, canals, tunnels and roads had been built and many had served their purpose for centuries. From this background of precedents, earthwork and foundation engineering developed primarily as an art steeped in tradition and empirical practices based on earlier successful accomplishments. No man can hope to become an artist in the practice of soil engineering without a rich background of personal experience coupled with a thorough knowledge of the experiences of his contemporaries and predecessors.

The rapidly developing science of soil mechanics provides qualitative and quantitative data on the stress-strain-time characteristics of soils. This knowledge gives the designer a feeling for the behavior of soils, under idealized conditions, that form guides to soil behavior under the more complex conditions in the field. The theories of soil mechanics provide insight into the behavior under simple ideal conditions. These aspects of soil mechanics form a framework that enables the engineer to organize, interpret, and evaluate experience. Each experience encountered by an engineer can be examined and categorized as to the type of material, the loading conditions, the hydraulic conditions, and in relation to theoretical concepts.

The procedures used to calculate bearing capacity, settlement or stability factor of safety of a slope use the framework of soil mechanics to organize experience. In fact, the procedures are valid and justified only to the extent that they have been verified by experience.

The field of soil mechanics enables an engineer to go beyond the limits of his experience. It points the way to new solution of old problems, or to the solution of previously unsolved problems. In this way, it permits the engineer to extrapolate his experience. Of course, such extrapolation involves a measure of uncertainty until the pertinent experience becomes available. Here, soil mechanics forms the guide as to what should be observed to check the design procedures as the work is executed.

Geology is as basic to earthwork design as soil mechanics. Possibly the most significant role of geology is to make the engineer aware of the departures from reality inherent in the simplifying assumptions used in soil mechanics procedures. The geology of a site must be understood to permit making and evaluating the design calculations or predictions. In some instances the geologic structure or the results of geologic processes may completely override all considerations of soil mechanics (jointed clays, etc.)

Every interpretation of the results of a test boring and every interpolation between two borings is an exercise in geology.

B. TYPES OF PROBLEMS ENCOUNTERED IN DESIGN

1. Embankments. Most of the problems encountered in the design of earth or rock embankments may be grouped into four categories:

    a. Determination of the bearing capacity of the foundation.

    b. Determination of the amount of settlement of the various parts of the structure caused by compression of foundation or fill materials.

        (1) Compression caused by loads imposed by the structure.
        (2) Compression caused by alteration of the water table.

    c. Determination of the amount of seepage or water movement through the fill or foundation and the effect of such seepage on the stability of the structure and economic feasibility of the project.

    d. Determination of the stability of earth slopes.

2. Reservoir basins. The majority of the problems encountered in evaluating reservoir basins may be grouped into the following categories:

    a. Determination of the tendency for the basin to leak.

    b. Determination of flow boundaries.

    c. Determination of head conditions, groundwater effects, etc.

    d. Determination of sliding stability of basin walls.

3. Channel systems. The soil engineering problems encountered in the evaluation and design of channel systems normally fit in the following groups:

   a. Determination of the erosion stability of materials occurring in the channel bed and banks.

   b. Determination of the sliding stability of the materials forming the channel bank.

   c. Determination of leakage from channel sections.

   d. Determination of uplift pressures, magnitude of structural loading and bearing capacity for channels to be lined.

4. Other structures. Soil engineering problems associated with the design of drop structures, diversion structures, pumping plants, etc., will generally fall in the following groups:

   a. Determination of the bearing capacity of the foundation.

   b. Determination of the sliding stability of the overall structure.

   c. Determination of the earth-water loads on structural members.

   d. Determination of settlement, both total and differential, of various parts of the structure.

   e. Evaluation of piping problems along the structure-soil interface.

   f. Evaluation of scour potential adjacent to hydraulic structures.

C. INFORMATION REQUIRED AS BASIS FOR DESIGN

To approach the problems encountered in design, certain information about the character of the foundation and the construction materials is essential. The information required for analysis of foundation conditions and the design of the proposed work are:

1. Character and distribution of materials in the foundation to significant depth.

2. The location and seasonal fluctuation of the groundwater table.

3. The surface topography of the site.

4. The geologic structure of the underlying formation.

5. The character, location and available quantities of fill materials.

Other items that will affect the construction operations such as site seismicity, climatic conditions, diversion and control of the stream, magnitude of wave action, time restrictions on construction and the function of the facility must be studied in preparing the design.

D. USE OF SOIL MECHANICS IN CONSTRUCTION

A knowledge of engineering soil behavior is necessary to understand the effect of construction operations on soil masses. The need for this knowledge and experience, along with the exercising of sound engineering judgment by personnel in charge of construction supervision, must be continuously emphasized.

In his book "Earth Dam Projects," J. P. Justin wrote, "An entirely safe and substantial design may be entirely ruined by careless and shoddy execution, and the failure of the structure may be the result. Careful attention to the details of construction is, therefore, fully as important as the preliminary investigation and design."

In earthwork engineering, to achieve maximum economy it is generally necessary to use locally available materials. In doing this the designer assesses the behavioral characteristics of the locally available materials and blends them into an efficient protective design meeting the requirements of site conditions and project objectives.

The earthwork designed must provide a greater tolerance range of earth materials use than that provided by the designer using other construction materials. This is necessary because of the inherent variations in the engineering behavior of earth materials, from deposit to deposit as well as within a given deposit, and because generally earth structures are massive, containing large volumes of materials. Consequently, a much closer relationship between operation of investigation, design, inspection and construction is required for earthwork than is needed for other types of engineered construction.

In general, the following categories of information are needed to insure proper execution of earthwork construction:

1. Knowledge of site conditions, including regional geology, character and distribution of foundation materials, water

table location and fluctuations, character and location of proposed fill materials, and geologic and structural conditions of rock to be encountered.

2. Knowledge of the intent of the designer as well as complete understanding of construction drawings and specifications.

3. Understanding of the engineering behavior of the soils at the site. An understanding of how these soils will respond to construction operations and various types of construction equipment.

4. Understanding of all the earthwork construction control tests and what the results mean with respect to engineering behavior and construction response.

E. NOMENCLATURE, DEFINITIONS AND CONVENTIONS

Insofar as is practicable, the definitions, nomenclature and conventions contained in ASTM D-653, Standard Definitions of Terms and Symbols Relating to Soil Mechanics, prepared by a joint ASCE-ASTM committee, will be followed.

SECTION 2. SOIL CLASSIFICATION AND IDENTIFICATION

A. ENGINEERING SOIL CLASSIFICATION

The fundamental purpose of a soil classification system is to provide a language by means of which one person's knowledge of the general characteristics of a particular soil can be conveyed to another person in a brief and concise manner, without entering into lengthy descriptions and detailed analysis.

A good soil classification system should satisfy the following requirements:

1. It should describe the soil in well-understood terms, which convey an idea of its type and behavior.

2. It should furnish an indication of soil properties and performance.

3. It should be applicable from visual examination - both in a simplied form and, with experience, in a more refined form.

4. It should employ a simple system of notations for graphic abstracts of boring logs on drawings recording boring information.

Basically there are four classifications for describing soils. These are:

1. Geological
2. Textural
3. Pedological
4. Engineering

In engineering work, it is frequently appropriate to use at least two or three, or all of these classifications for describing soils at a given project site, such as (a) the geological classification, which provides a language for stating the geological history and background of the soil; (b) the textural classification which provides names for describing the relative portions of sand, silt and clay size particles in a soil; (c) the pedological classification which reveals the nature of the soil profile as affected by the climatic and other environmental conditions under which it developed and (d) an engineering classification that groups soils on the basis of their characteristics which influence engineering performance.

Great care should be exercised not to read into any classification more information than they are intended to convey. For instance, there is no engineering classification in existence that can be used directly for design values. Testing of individual materials is always necessary to determine the engineering properties of permeability, strength, and compression characteristics.

The development of a soil classification system is accomplished by the empirical correlation of the physical properties of soils with their observed behavior characteristics under such varying conditions as climate, treatment, loading, and construction methods. It is generally found that certain ranges of physical properties may be established within which the soils possessing these properties will react in approximately the same manner to a certain set of imposed conditions.

It is important that, within a given organization, the methods for selecting soil materials for a specific purpose be uniform throughout the organization. The adoption of a standard soil classification system provides a comprehensive means of communication and facilitates the review of reports and designs at all levels within the organization.

Many soil classification systems have been developed; each for the purpose of classifying soils with respect to their suitability for a specific purpose. The "Unified Soil Classification System" developed by Arthur Casagrande and modified by the Bureau of Reclamation and the Corps of Engineers is the classification system used by the Soil Conservation Service for engineering purposes.

The soil classification system developed by the U.S. Bureau of Public Roads and adopted by the American Association of State Highway and Transportation Officials is included because of its use in published soil surveys.

B. UNIFIED SOIL CLASSIFICATION SYSTEM

1. Soil Properties Used in Classification

This system identified soils according to their textural and plasticity qualities and on their groupings with respect to behavior. The system is based on those characteristics of a soil that indicate how it will behave as an engineering construction material.

The soil properties that have been found most useful for this purpose and form the basis of soil identification are:

a. Percentage of gravel, sand and fines (fraction passing No. 200 sieve).

b. Shape of grain-size distribution curve.

c. Plasticity and compressibility characteristics.

These properties are determined by mechanical analysis, liquid limit and plastic limit tests and with experience can be estimated with good accuracy.

2. Definition of Soil Components

Component	Size Range
Boulders	Larger than 12" diameter
Cobbles	Between 3" and 12" diameter
Gravel	Between No. 4 mesh (4.76 mm) and 3" diameter
Sand	Between No. 200 mesh (.074 mm) and No. 4 mesh
Fines*	Smaller than No. 200 mesh

* Fines include silt and clay. A material is called a silt if it is non-plastic or very slightly plastic and exhibits little or no strength when air-dried. Silt fines plot below the A-line, Table 2-1. A material is called a clay if the fines exhibit plasticity within a range of water contents and has considerable strength when air-dried. Clay fines plot above the A-line, Table 2-1.

A comparison of the size of the soil components as defined in the UNIFIED, the AASHTO and the USDA textural classification systems is shown on Figure 2-1.

3. The Plasticity Chart (Figure 2-2)

The plasticity chart is a plotting of the plasticity index versus the liquid limit. The A-line on the plasticity chart is an important empirical boundary.

In general, soils with Atterberg limits that plot above the A-line behave as typical non-organic clays. Soils plotting below the

LABORATORY CRITERIA FOR UNIFIED SOIL CLASSIFICATION SYSTEM

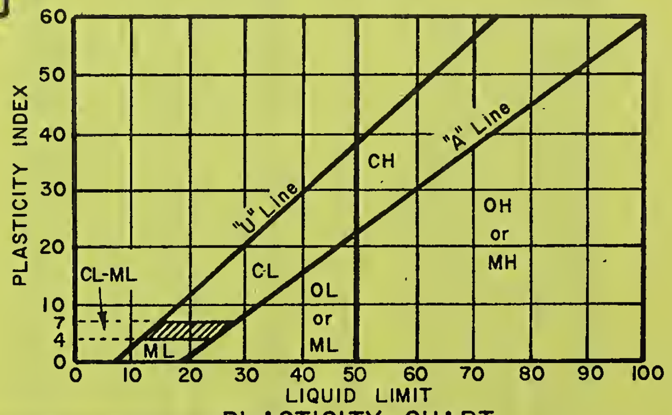
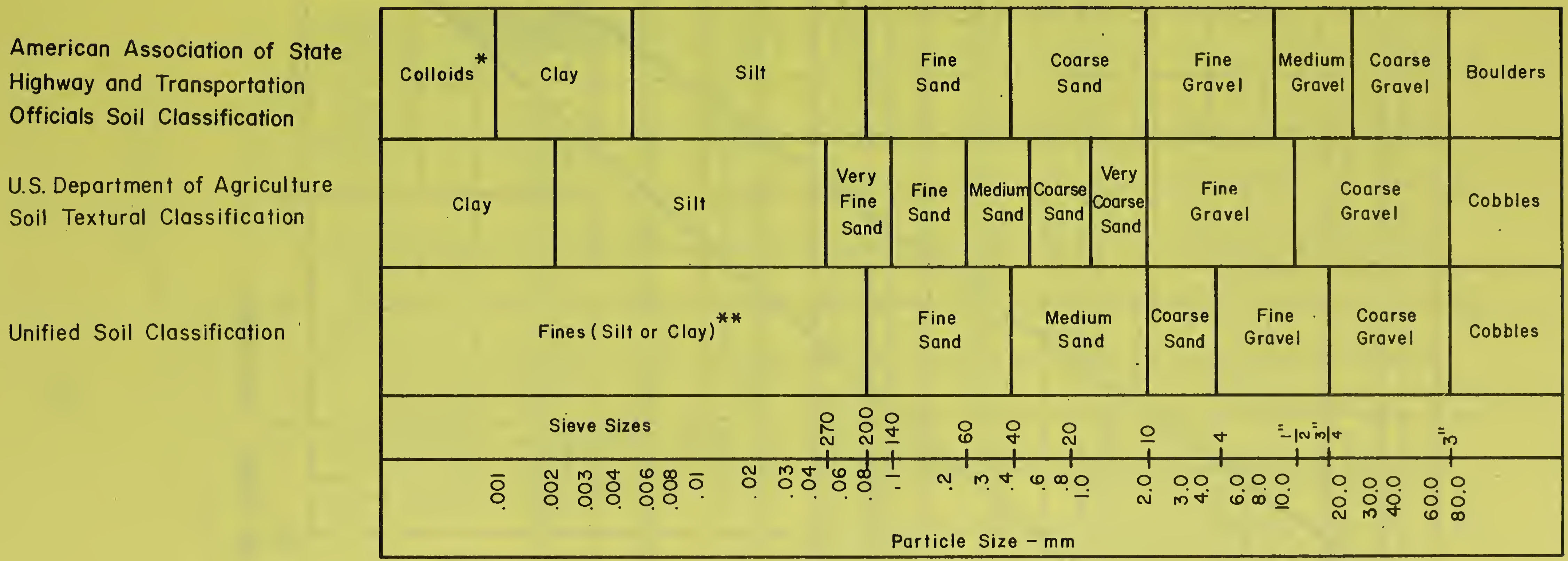
			GROUP SYMBOLS	TYPICAL NAMES	LABORATORY CLASSIFICATION CRITERIA		
Coarse Grained Soils $d_{50} >$ No. 200 Mesh (0.074 mm)	Sands and Gravels	**GRAVELS** Less than half of the coarse fraction passes the No. 4 sieve size.	GW	Well graded gravels, gravel-sand mixtures, little or no fines.	Determine percentages of gravel and sand from grain size curve. Depending on percentage of fines (fraction smaller than No. 200 sieve size) coarse grained soils are classified as follows : Less than 5% GW, GP, SW, SP More than 12% GM, GC, SM, SC 5% to 12% Borderline cases requiring use of dual symbols	$C_u = \frac{D_{60}}{D_{10}}$ Greater than 4 ; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 & 3	
			GP	Poorly graded gravels, gravel-sand mixtures, little or no fines.		Not meeting all graduation requirements for GW	
			GM	Silty gravels, poorly graded, gravel-sand-silt mixtures.		Atterberg limits below "A" line, or PI < 4	Above "A" line with PI between 4 and 7 are borderline cases requiring use of dual symbols.
			GC	Clayey gravels, poorly graded, gravel-sand-clay mixtures.		Atterberg limits above "A" line, PI > 7	
		SANDS More than half of the coarse fraction passes the No. 4 sieve size.	SW	Well graded sands, gravelly sands, little or no fines.		$C_u = \frac{D_{60}}{D_{10}}$ Greater than 6 ; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 & 3	
			SP	Poorly graded sands, gravelly sands, little or no fines.		Not meeting all graduation requirements for SW	
			SM	Silty sands, poorly graded sand-silt mixtures.		Atterberg limits below "A" line, or PI < 4	Above "A" line with PI between 4 and 7 are borderline cases requiring use of dual symbols.
			SC	Clayey sands, poorly graded sand-clay mixtures.		Atterberg limit above "A" line, PI > 7	
Fine Grained Soils $d_{50} <$ No. 200 Mesh (0.074 mm)	Silts and Clays	GO TO PLASTICITY CHART	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands with slight plasticity.	 PLASTICITY CHART		
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.			
			MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.			
			CH	Inorganic clays of high plasticity, fat clays.			
			OL	Organic silts and organic silt-clays of low plasticity. LL<50	Below "A" line and PI and $\frac{\text{L.L. (oven dry soil)}}{\text{L.L. (air dry soil)}} < 0.75$		
			OH	Organic clays of medium to high plasticity. LL>50			
			Pt	Peat and other highly organic soils			

TABLE 2-1 - LABORATORY CRITERIA FOR UNIFIED SOIL CLASSIFICATION SYSTEM

FIGURE 2-1. A COMPARISON OF GRAIN-SIZE LIMITS IN THE 3 CLASSIFICATION SYSTEMS

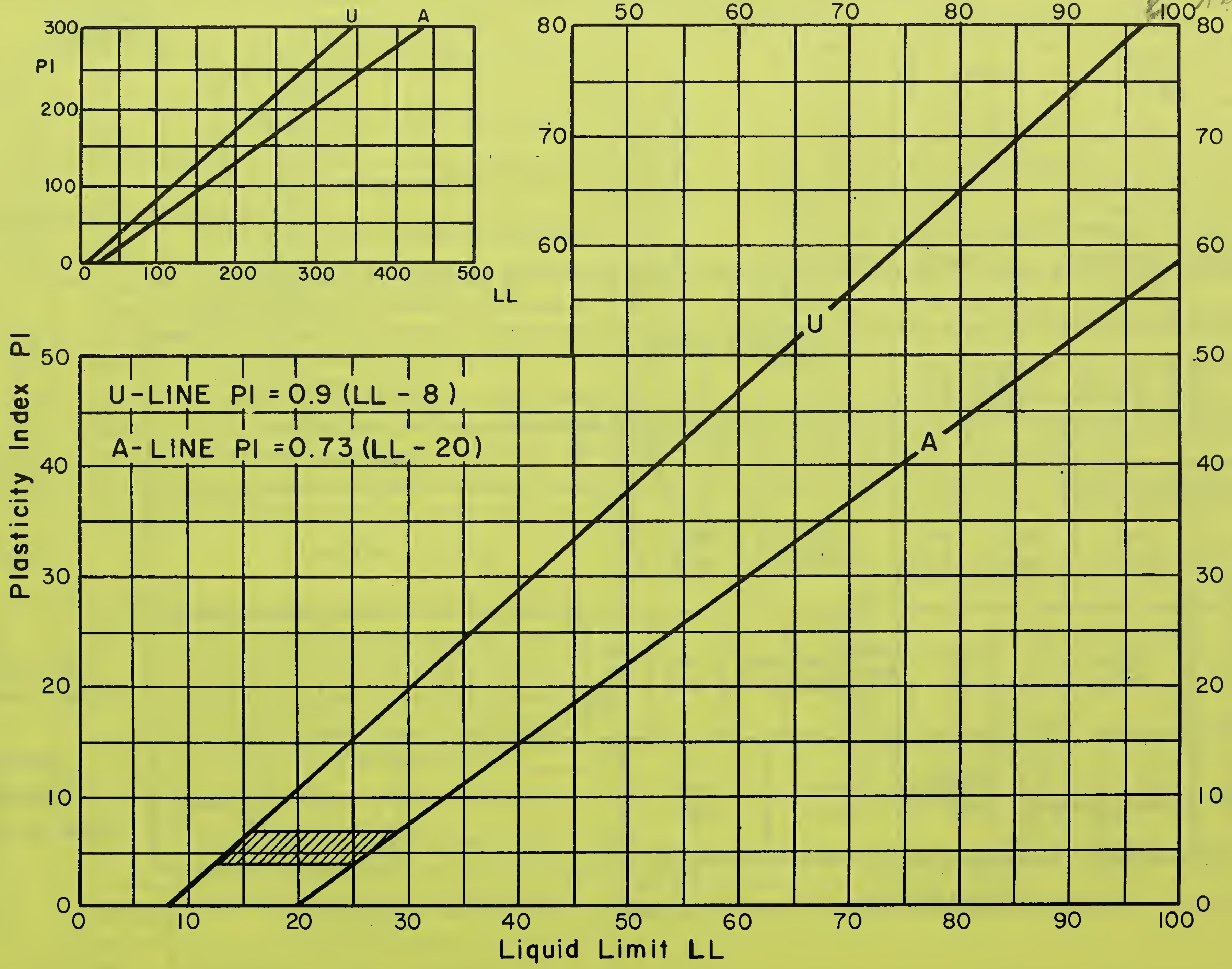


*Colloids included in clay fraction in test reports.

**The LL and PI of "Silt" plot below the "A" line on the plasticity chart, Figure 2-2 and the LL and PI for "Clay" plot above the "A" line

Modified from PCA Soil Primer

FIG. 2-2. PLASTICITY CHART



To date no soils found Above this Line

A-line behave as plastic soils containing organic colloids or as typical inorganic silts. An exception to the above rule is the shaded area above the A-line with PI greater than 4 and less than 7, soils with Atterberg limits plotting in this zone may behave as a clay or as a silt.

The U-line on the plasticity chart represents the upper limit of the LL and PI plots for soils. At this date, (1973) no soils have been tested that plot above this line.

The plasticity chart is very useful in classifying soils for engineering purposes. The soil groupings of the unified soil classification system are shown on a plasticity chart in Table 2-1 and on Figure 2-3.

Much useful information about the behavior of a soil can be inferred from the plasticity chart.

If a mineral is ground up and the Atterberg limits of the various grain size fractions determined, the points representing such test results plot along a straight line roughly parallel to the A-line. Depending on the mineralogical compositions of the grains, the plot may be above or below the A-line.

The points representing the limits of soils from a geologically well defined sedimentary deposit will plot along a line roughly parallel to the A-line.

4. <u>Summary of the Unified Classification System</u>

a. Designates soils as fine or coarse according to median size. Coarse soil if median size is larger than 0.074 mm (No. 200 mesh). Fine grained if median size is smaller than 0.074 mm (No. 200 mesh).

b. Coarse grained soils are classified on the basis of:

   (1) Grain size and distribution
   (2) Quantity of fines
   (3) Character of fines

c. Fine grained soils are classified on basis of:

   (1) Liquid limit - High if LL is greater than 50
   Low if LL is less than 50

   (2) Plasticity index - Clay if above A-line, silt if below A-line.

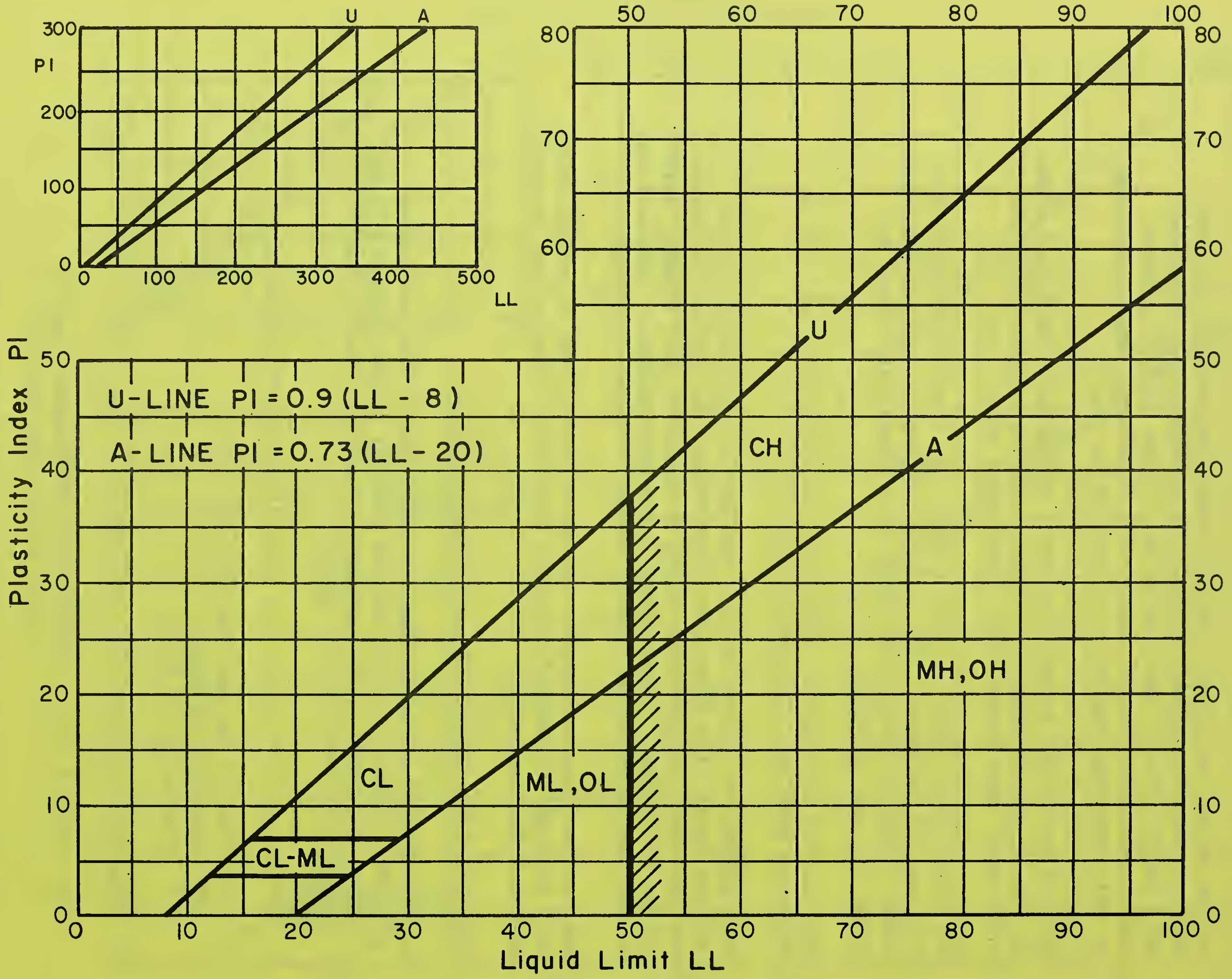


FIG. 2-3. UNIFIED SOIL CLASSIFICATION GROUP POSITIONS ON PLASTICITY CHART

(3) Grading is of minor importance.

d. Soil groups and group symbols of the Unified Classification System.

Basic Symbols	Modifying Symbols
G - gravel	W - well graded
S - sand	P - poorly graded
	C - with clay fines
	M - with silt fines
C - clay	L - low liquid limit
M - silt	H - high liquid limit
O - organic	
Pt - peat	

Each soil is classified and identified with a verbal description and a group symbol consisting of two of the above letters. The letters may be considered as initials of the name of the most typical soil in the group.

Coarse grained soils are sub-divided into:

(1) Gravel and gravelly soils; symbol G

(2) Sands and sandy soils; symbol S

The gravels and the sands are each divided into four groups:

(a) Well graded, fairly clean material; symbol W, in combinations GW and SW.

(b) Poorly graded, fairly clean material; symbol P, in combinations GP and SP.

(c) Coarse material with clay fines, symbol C, in combinations GC and SC.

(d) Coarse materials with silt fines, symbol M, in combinations GM and SM.

Fine grained soils are subdivided into:

(1) The inorganic silty and very fine sandy soils; symbol M, used for fine-grained non-plastic or slightly plastic soils.

(2) The inorganic clays; symbol C.

(3) The organic silts and clays; symbol O.

Each of these types of fine-grained soils is grouped according to its liquid limit into:

(a) Fine-grained soils having liquid limits less than 50; symbol L, in combinations ML, CL and OL.

(b) Fine-grained soils having liquid limits greater than 50: symbol H, in combinations MH, CH and OH.

Highly organic soils, usually fibrous, such as peat and swamp soils having high compressibility, are not subdivided and are placed in one group; symbol Pt.

The sequence of the group symbols need not be memorized but the meaning of the symbols and the sequence of the major divisions, that is, G-S-L-H should be learned.

When a material does not clearly fall into one group, boundary classification such as GW-SW or CL-ML should be used.

5. Method of Classification (Laboratory Test Data)

a. For all soils.

Step No. 1. Classify as coarse grained or fine grained soil.
Coarse grained if the median size is larger than 0.074 mm (#200 sieve).
Fine grained if the median size is less than 0.074 mm (#200 sieve).

b. For fine grained soils.

Step No. 2. Enter plasticity chart with LL-PI data.

Classify as silt if LL-PI data plot below A-line (Table 2-1) or PI is less than 4.
Classify as clay if LL-PI data plot above A-line with PI greater than 7.
Dual classification symbols should be used if LL-PI data plot above A-line with the PI between 4 and 7.

c. For coarse grained soils.

(1) Step No. 2. Classify as sand or gravel.

Sand if 50% or more of coarse fraction is sand size (#4 to #200 sieve).
Gravel if 50% or more of coarse fraction is gravel size (#4 to 3" sieve).

(2) Step No. 3. Classify as clean or with fines.

Clean if material contains less than 5% non-plastic fines that do not interfere with the free draining properties. In areas subject to frost action, the material should not contain more than 3% of particles smaller than 0.02 mm in size to classify as clean. A material containing less than 5% plastic fines should be classed as a borderline soil with dual symbols.

With fines if material contains more than 12% fines.

Borderline case with dual symbols if materials contain between 5-12% fines.

(3) For clean sand and gravels

Step No. 4. Compute the coefficient of uniformity C_u (D_{60} size divided by D_{10} size) and the coefficient of curvature C_c (D_{30} size squared divided by the product of the D_{10} size and the D_{60} size). Classify soil as well graded or poorly graded.

Sand is well graded if C_u is greater than 6 and C_c is greater than 1 and less than 3; poorly graded otherwise. Gravel is well graded if C_u is greater than 4 and C_c is greater than 1 and less than 3; poorly graded otherwise.

(4) For sands or gravels with fines.

Step No. 4. Determine character of fines.

Coarse grained with silt fines if LL-PI data plot below A-line (Table 2-1) or PI is less than 4.
Coarse grained with clay fines if LL-PI data plot above A-line with PI greater than 7.

Examples illustrating the use of the Unified Soil Classification System are on pages 2-12 through 2-21. The examples cover a wide range of soil materials. The procedure used in classifying each soil is listed in the remarks space on the soil classification report sheets for each example.

LABORATORY NO.

MATERIALS TESTING REPORT	**U. S. DEPARTMENT of AGRICULTURE SOIL CONSERVATION SERVICE**	**SOIL CLASSIFICATION**

PROJECT and STATE Typical Sample

SAMPLE LOCATION Pit 1

FIELD SAMPLE NO. 2.1

DEPTH 1.0 to 4.5 ft.

GEOLOGIC ORIGIN

TYPE OF SAMPLE Bag

TESTED AT Portland, Oregon

APPROVED BY JCS

DATE 12-67

SYMBOL MH

DESCRIPTION Light gray, sandy clayey silt A-7-5 (28)

GRAIN SIZE DISTRIBUTION

C_c	
C_u	
D_{MAX}	2.0
D_{85}	0.06
D_{60}	0.012
D_{50}	0.005
D_{30}	0.001
D_{15}	
D_{10}	

UNDISTURBED CONDITION

DRY UNIT WEIGHT		MOISTURE
g/cc	pcf	%

SOLUBLE SALTS	SHRINKAGE LIMIT
2.4 %	22.0 %

ATTERBERG LIMITS

NATURAL MOISTURE		AIR DRY		OVEN DRY	
LL	PI	LL	PI	LL	PI
		59	27		

SPECIFIC GRAVITY (G_s)

(-) #4	(+) #4
2.74	

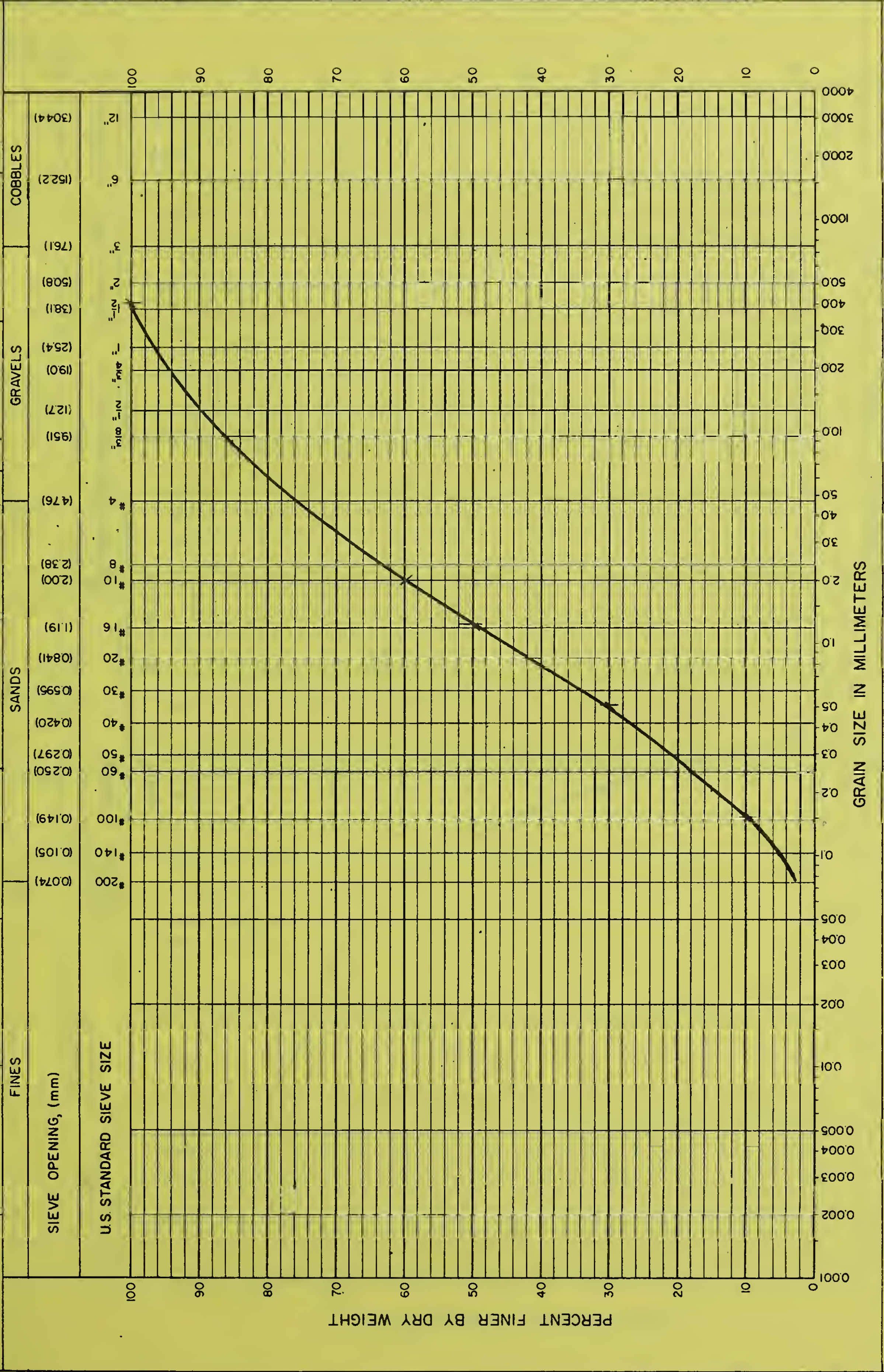
REMARKS: Step 1 - Median size less than 0.074 mm, therefore, fine grained.
Step 2 - Determine group symbols from LL-PI plot on plasticity chart.
Class MH because LL-PI plot below A line. LL greater than 50.

MATERIALS TESTING REPORT	U. S. DEPARTMENT of AGRICULTURE SOIL CONSERVATION SERVICE	SOIL CLASSIFICATION

PROJECT and STATE: Typical Sample
SAMPLE LOCATION: Channel Sta. 37+40
FIELD SAMPLE NO.: 2.2
DEPTH: 4.0 to 11.7 ft.
GEOLOGIC ORIGIN:
TYPE OF SAMPLE: Bag
TESTED AT: Portland, Oregon
APPROVED BY: JCS
DATE: 12-67
SYMBOL: SP
DESCRIPTION: Poorly graded, gravelly sand A-1-b (o)

GRAIN SIZE DISTRIBUTION

D_{10}	0.15
D_{15}	0.20
D_{30}	0.50
D_{50}	1.30
D_{60}	2.0
D_{85}	9.0
$D_{MAX.}$	4.0
C_u	13.3
C_c	0.8



UNDISTURBED CONDITION

DRY UNIT WEIGHT		MOISTURE
g/cc	pcf	%

SHRINKAGE LIMIT	SOLUBLE SALTS
%	%

ATTERBERG LIMITS

NATURAL MOISTURE		AIR DRY		OVEN DRY	
LL	PI	LL	PI	LL	PI
		NP			

SPECIFIC GRAVITY (G_s)

(-) #4	(+) #4
2.66	

REMARKS: Step 1 - Coarse grained, median size greater than 0.074 mm. Step 2 - Clean, less than 5% finer than #200. Step 3 - Sand because 72% of coarse fraction is sand. Step 4 - Determine C_u & C_c, C_u = 13.3, C_c = 0.8. Classify as SP because C_c less than 1.0.

LABORATORY NO.

MATERIALS TESTING REPORT	**U. S. DEPARTMENT of AGRICULTURE SOIL CONSERVATION SERVICE**	**SOIL CLASSIFICATION**

PROJECT and STATE: Typical Sample

SAMPLE LOCATION: Sta. 37+50 East Bank Channel

FIELD SAMPLE NO.: 2.3

DEPTH: 12.0 to 13.7 ft.

GEOLOGIC ORIGIN:

TYPE OF SAMPLE: Bag

TESTED AT: Portland, Oregon

APPROVED BY: JCS

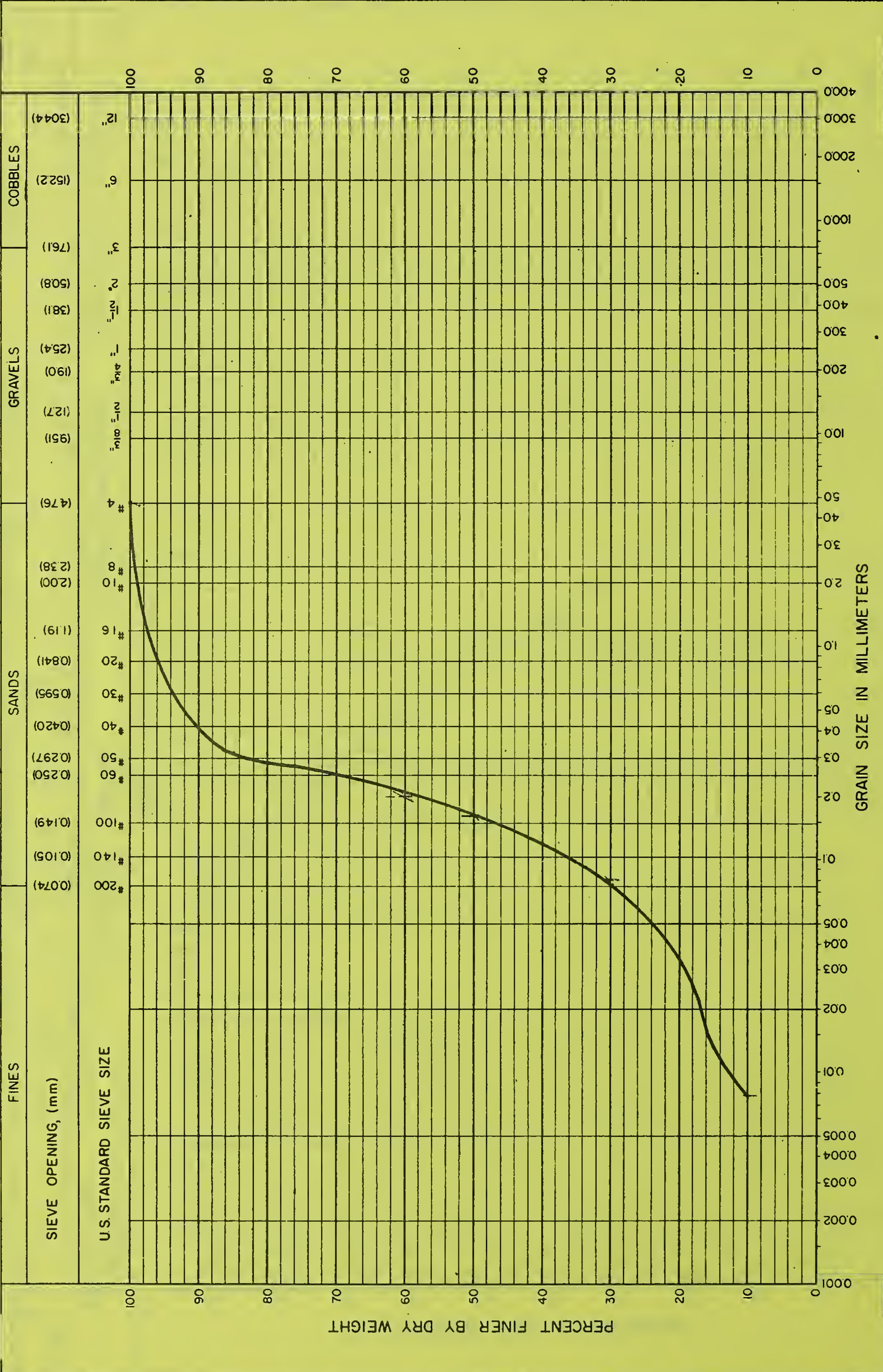
DATE: 12-67

SYMBOL: SM

DESCRIPTION: Light brownish gray, silty sand A-2-4 (o)

GRAIN SIZE DISTRIBUTION

C_c	
C_u	
$D_{MAX.}$	4.76
D_{85}	0.32
D_{60}	0.2
D_{50}	0.16
D_{30}	0.08
D_{15}	0.014
D_{10}	0.008



UNDISTURBED CONDITION

DRY UNIT WEIGHT	g/cc pcf
MOISTURE	%

SHRINKAGE LIMIT	%
SOLUBLE SALTS	%

ATTERBERG LIMITS

	LL	PI
OVEN DRY		
AIR DRY		NP
NATURAL MOISTURE		

SPECIFIC GRAVITY (G_s)

(-) #4	(+) #4
2.79	

REMARKS: Step 1 - Coarse grained, median size greater than 0.074 mm.
Step 2 - With fines, 30% passes #200. Step 3 - Sand, contains no gravel.
Step 4 - LL-PI plot below A-line, class SM.

LABORATORY NO. ______

MATERIALS TESTING REPORT

U. S. DEPARTMENT of AGRICULTURE
SOIL CONSERVATION SERVICE

SOIL CLASSIFICATION

PROJECT and STATE: Typical Sample

SAMPLE LOCATION: ℄ Sta. 3+73

FIELD SAMPLE NO.: 2.4

DEPTH: 0.5 to 1.7 ft.

GEOLOGIC ORIGIN:

TYPE OF SAMPLE: Cube (Undist.)

TESTED AT: Portland, Oregon

APPROVED BY: JCS

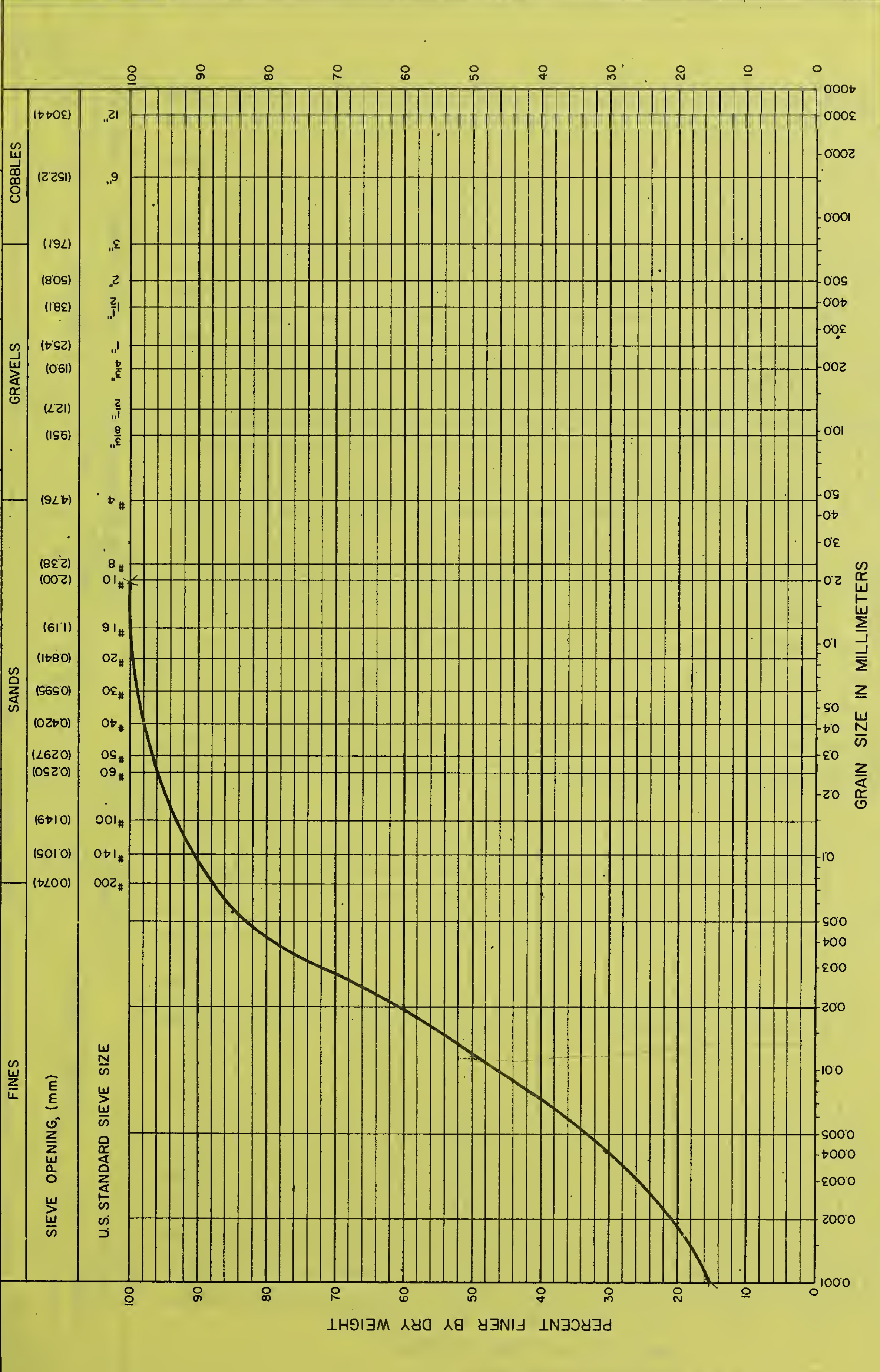
DATE: 12-67

SYMBOL: MH

DESCRIPTION: Slightly organic, dark brown, sandy, clayey silt A-7-5(28

GRAIN SIZE DISTRIBUTION

D_{10}	D_{15}	D_{30}	D_{50}	D_{60}	D_{85}	$D_{MAX.}$	C_u	C_c
	0.001	0.004	0.012	0.02	0.055	2.00		



UNDISTURBED CONDITION

MOISTURE: 59 %

DRY UNIT WEIGHT: g/cc ______ pcf 73.0

SHRINKAGE LIMIT: 23.0 %

SOLUBLE SALTS: 0.5 %

ATTERBERG LIMITS

NATURAL MOISTURE		AIR DRY		OVEN DRY	
LL	PI	LL	PI	LL	PI
63	29	59	27	52	22

SPECIFIC GRAVITY (G_S)

(-) #4	(+) #4
2.71	

REMARKS:
Step 1 - Fine grained because median size less than 0.074 mm.
Step 2 - LL-PI plot below A-line, LL greater than 50, MH or OH.
Step 3 - LL_{OD}/LL_{AD} = 0.88 greater than 0.75, class MH.

FILE CODE ENG-22

LABORATORY NO

MATERIALS TESTING REPORT | **U. S. DEPARTMENT of AGRICULTURE SOIL CONSERVATION SERVICE**

SOIL CLASSIFICATION

PROJECT and STATE: Typical Sample

SAMPLE LOCATION: Sta. 110+25

FIELD SAMPLE NO.: 2.5

DEPTH: 0.5 to 13.2

GEOLOGIC ORIGIN:

TYPE OF SAMPLE: Bag

TESTED AT: Portland, Oregon

APPROVED BY: JCS

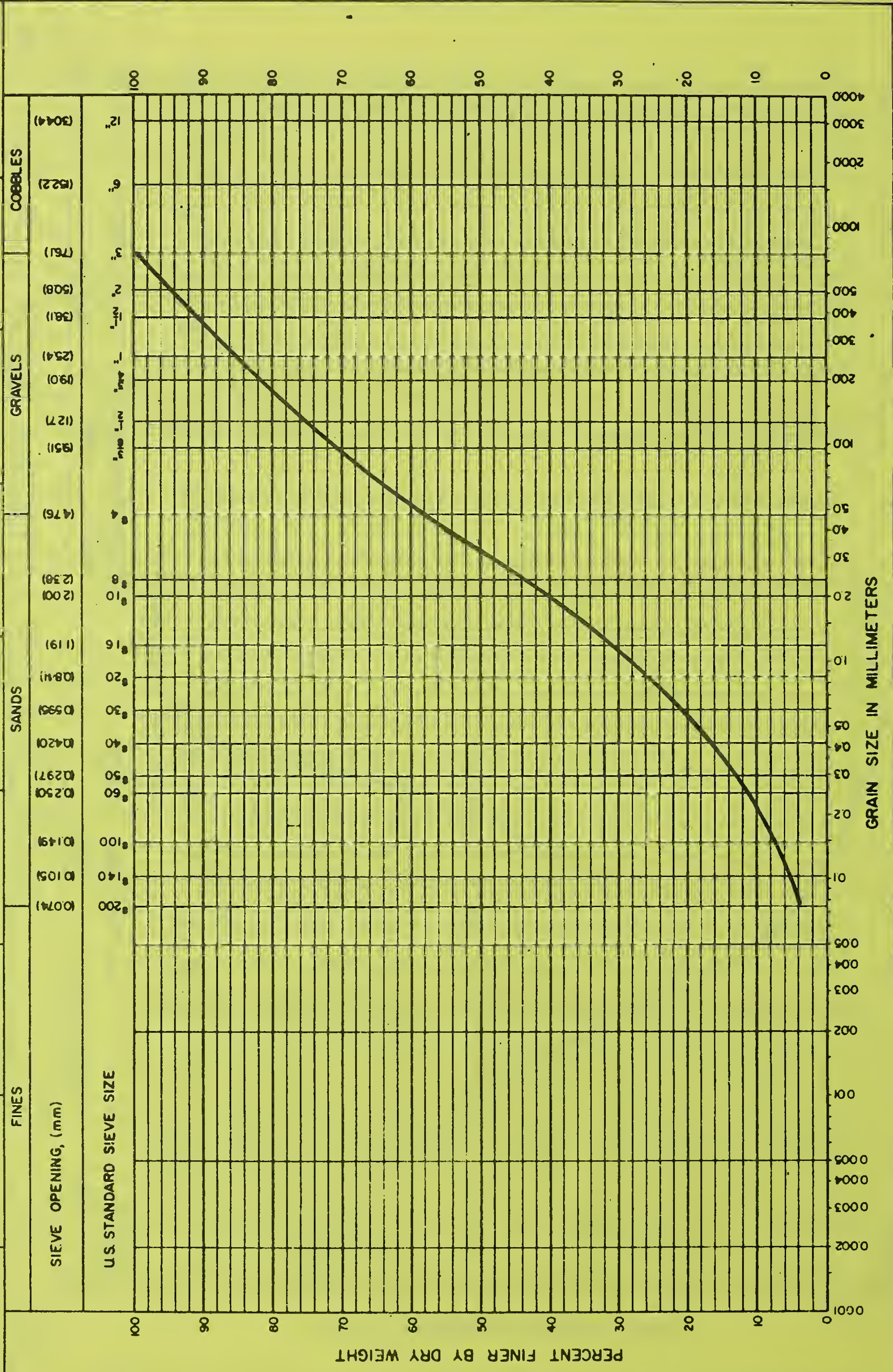
DATE: 12-68 (Rev.)

SYMBOL: SW

DESCRIPTION: Well graded, gravelly sand A-1-a (o)

GRAIN SIZE DISTRIBUTION

D_{10}	0.20
D_{15}	.36
D_{30}	1.2
D_{50}	3.0
D_{60}	5.0
D_{85}	26.0
D_{MAX}	75.0
C_u	25.0
C_c	1.4



UNDISTURBED CONDITION

MOISTURE	DRY UNIT WEIGHT
%	g/cc pcf

SHRINKAGE LIMIT: %

SOLUBLE SALTS: %

ATTERBERG LIMITS

NATURAL MOISTURE		AIR DRY		OVEN DRY	
LL	PI	LL	PI	LL	PI
		NP			

SPECIFIC GRAVITY (G_s)

(-) #4	(+) #4
2.66	2.60

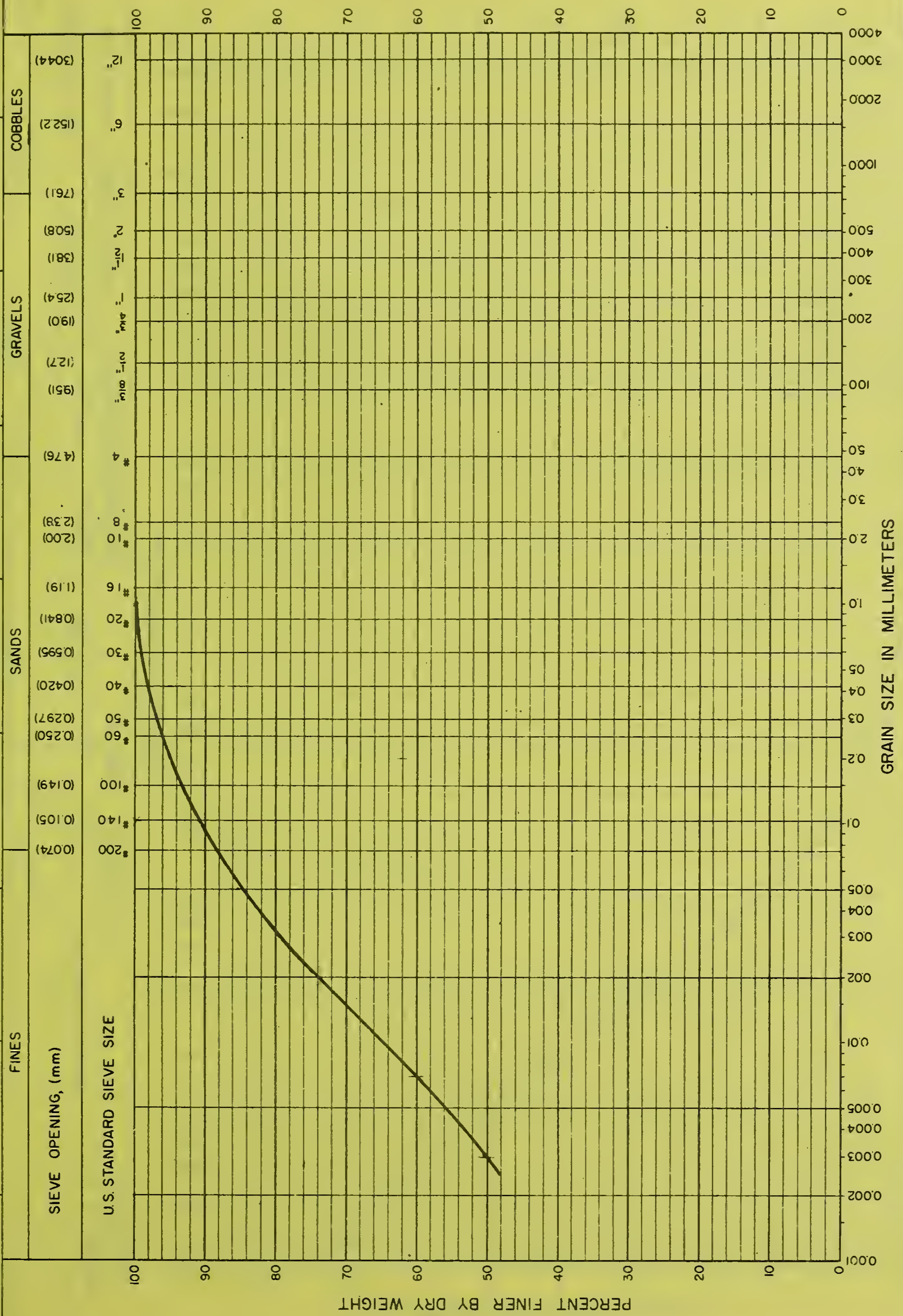
REMARKS: Step 1 - Coarse grained, median size greater than 0.074.
Step 2 - Clean, less than 5% passes #200. Step 3 - Sand, 55% coarse fraction sand size.
Step 4 - Determine C_u & C_c, C_u = 25.0, C = 1.4, class SW.

MATERIALS TESTING REPORT	U. S. DEPARTMENT of AGRICULTURE SOIL CONSERVATION SERVICE	SOIL CLASSIFICATION

PROJECT and STATE Typical Sample			SAMPLE LOCATION Sta. 4+42
FIELD SAMPLE NO. 2.6	DEPTH 47.0 to 49.0	GEOLOGIC ORIGIN	
TYPE OF SAMPLE Denison	TESTED AT Portland, Oregon	APPROVED BY JCS	DATE 12-67
SYMBOL CH	DESCRIPTION Dark gray, highly plastic clay A-7-6 (59)		

GRAIN SIZE DISTRIBUTION

C_c	
C_u	
$D_{MAX.}$	1.0
D_{85}	0.05
D_{60}	0.007
D_{50}	0.003
D_{30}	
D_{15}	
D_{10}	



UNDISTURBED CONDITION

MOISTURE	DRY UNIT WEIGHT g/cc	pcf
63 %		82.5

SHRINKAGE LIMIT 8.4 %

SOLUBLE SALTS %

ATTERBERG LIMITS

NATURAL MOISTURE		AIR DRY		OVEN DRY	
LL	PI	LL	PI	LL	PI
82	63	80	62	77	60

SPECIFIC GRAVITY (G_s)

(-) #4	(+) #4
2.83	

REMARKS: Step 1 - Fine grained, median size less than 0.074 mm.
Step 2 - LL-PI plot above A-line, LL greater than 50, class CH.

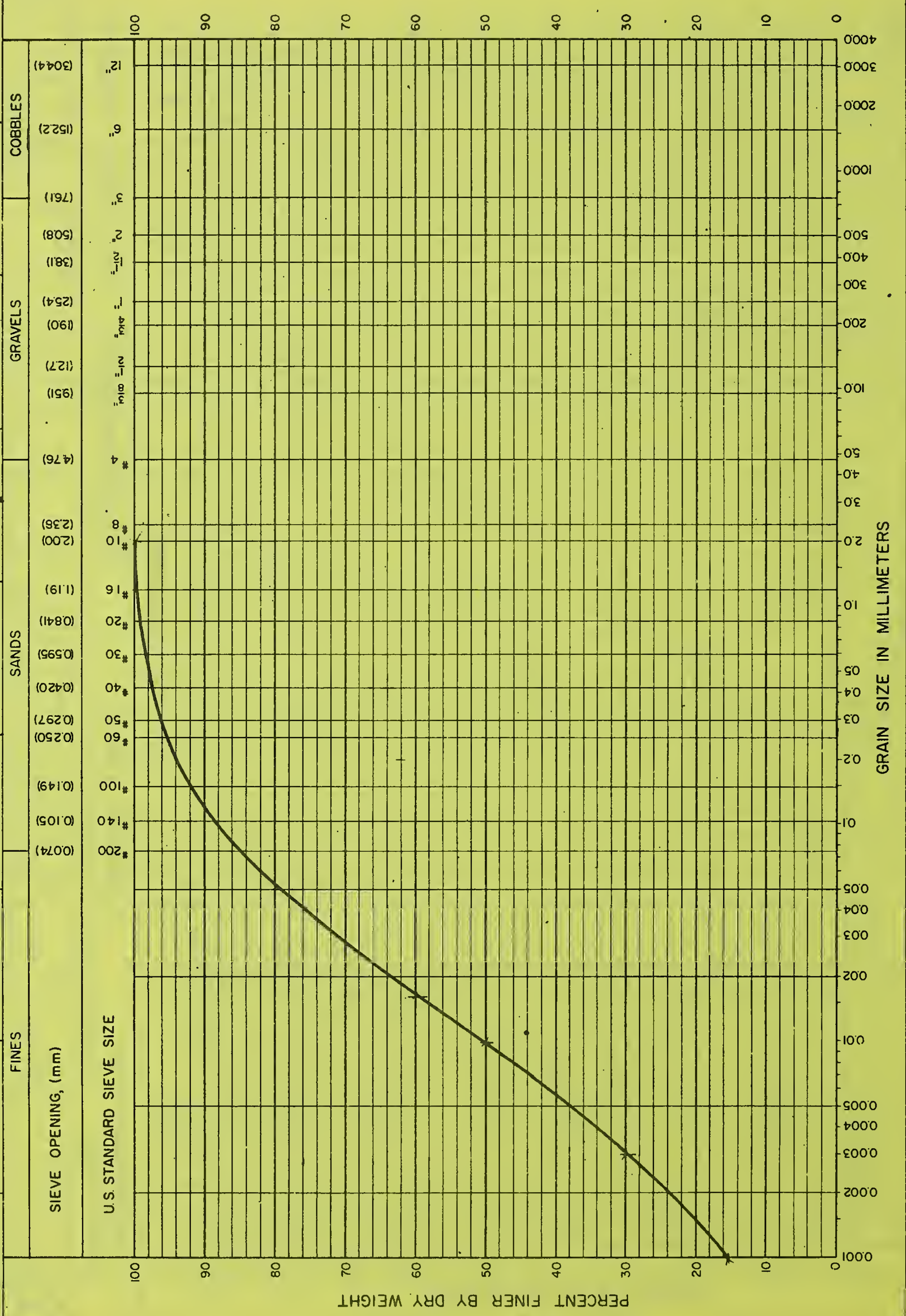
LABORATORY NO. ____

MATERIALS TESTING REPORT	U. S. DEPARTMENT of AGRICULTURE SOIL CONSERVATION SERVICE	SOIL CLASSIFICATION

PROJECT and STATE Typical Sample			SAMPLE LOCATION Sta. 44+20
FIELD SAMPLE NO. 2.7	DEPTH 22.0 to 27.0	GEOLOGIC ORIGIN	
TYPE OF SAMPLE Jar	TESTED AT Portland, Oregon	APPROVED BY JCS	DATE 12-67
SYMBOL CL	DESCRIPTION Yellowish brown, sandy silty clay A-6 (9)		

GRAIN SIZE DISTRIBUTION

$D_{MAX.}$	2.0
D_{85}	0.8
D_{60}	0.16
D_{50}	0.01
D_{30}	0.003
D_{15}	0.001
D_{10}	
C_u	
C_c	



UNDISTURBED CONDITION

MOISTURE	DRY UNIT WEIGHT	
%	g/cc	pcf

SHRINKAGE LIMIT 9.5 %

SOLUBLE SALTS %

ATTERBERG LIMITS

	NATURAL MOISTURE		AIR DRY		OVEN DRY	
	LL	PI	LL	PI	LL	PI
			26	14		

SPECIFIC GRAVITY (G_S)

(+) #4	(-) #4

REMARKS: Step 1 - Fine grained, median size less than 0.074 mm.
Step 2 - LL-PI plot above A-line, LL less than 50, class CL

LABORATORY NO.

MATERIALS TESTING REPORT	U. S. DEPARTMENT of AGRICULTURE SOIL CONSERVATION SERVICE	SOIL CLASSIFICATION

PROJECT and STATE Typical Sample	SAMPLE LOCATION Sta. 32+70

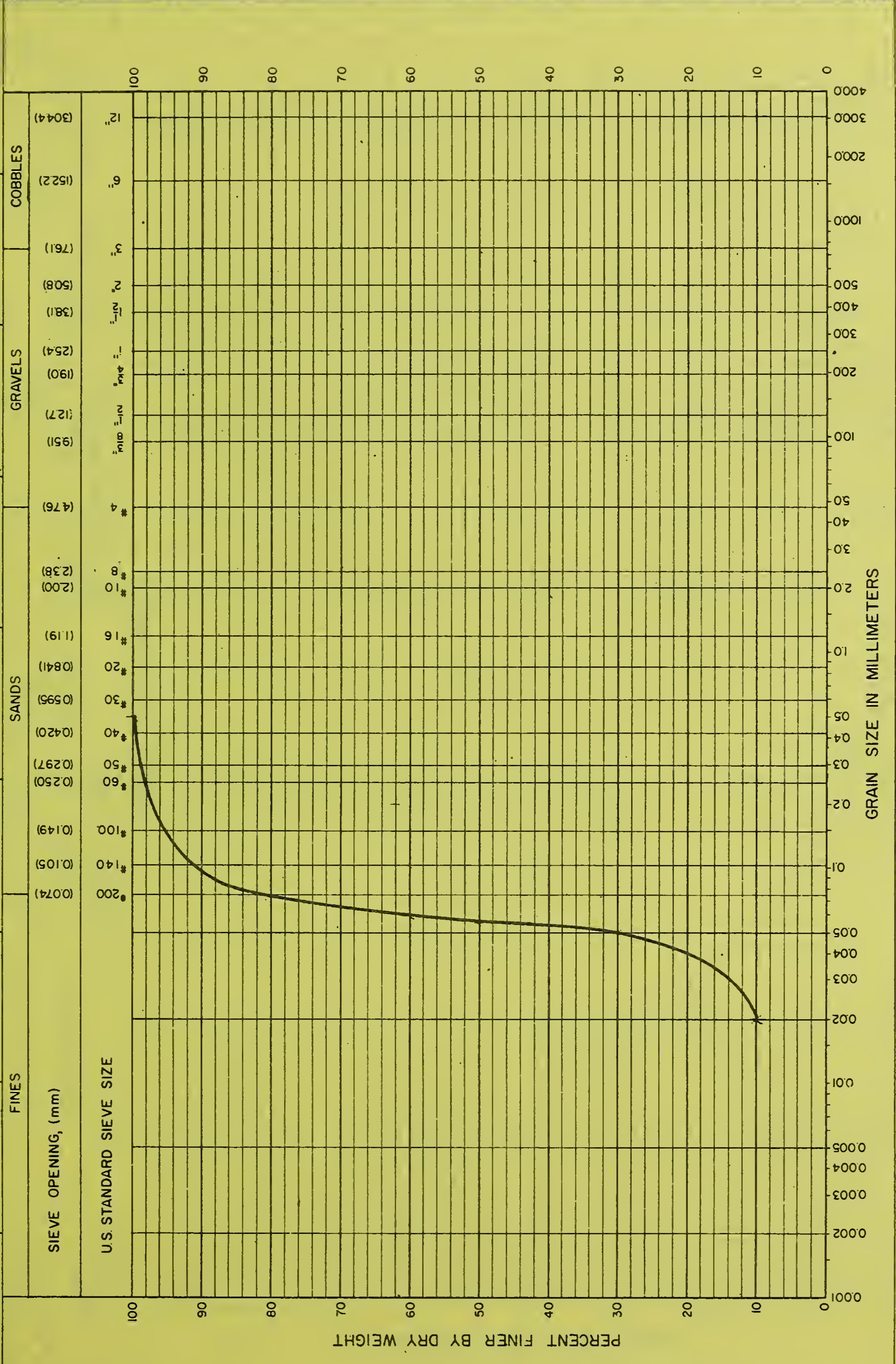
FIELD SAMPLE NO. 2.8	DEPTH 3.0 to 7.4 ft	GEOLOGIC ORIGIN

TYPE OF SAMPLE Bag	TESTED AT Portland, Oregon	APPROVED BY JCS	DATE 12-67

SYMBOL ML	DESCRIPTION Gray brown, sandy silt A-4 (o)

GRAIN SIZE DISTRIBUTION

D_{10}	0.02
D_{15}	0.04
D_{30}	0.05
D_{50}	0.055
D_{60}	0.06
D_{85}	0.08
D_{MAX}	0.5
C_u	
C_c	



UNDISTURBED CONDITION

DRY UNIT WEIGHT		MOISTURE
g/cc	pcf	%

SHRINKAGE LIMIT	SOLUBLE SALTS
%	%

ATTERBERG LIMITS

NATURAL MOISTURE		AIR DRY		OVEN DRY	
LL	PI	LL	PI	LL	PI
		NP			

SPECIFIC GRAVITY (G_s)

(-) #4	(+) #4
2.72	

REMARKS: Step 1 - Fine grained, median size less than 0.074 mm.
Step 2 - LL-PI plot below A-line, class ML

LABORATORY NO.

MATERIALS TESTING REPORT

U. S. DEPARTMENT of AGRICULTURE
SOIL CONSERVATION SERVICE

SOIL CLASSIFICATION

PROJECT and STATE: Typical Sample

SAMPLE LOCATION: Sta. 3+30

FIELD SAMPLE NO.: 2.9

DEPTH: 8.0 to 8.7 ft.

GEOLOGIC ORIGIN:

TYPE OF SAMPLE: Undisturbed Cube

TESTED AT: Portland, Oregon

APPROVED BY: JCS

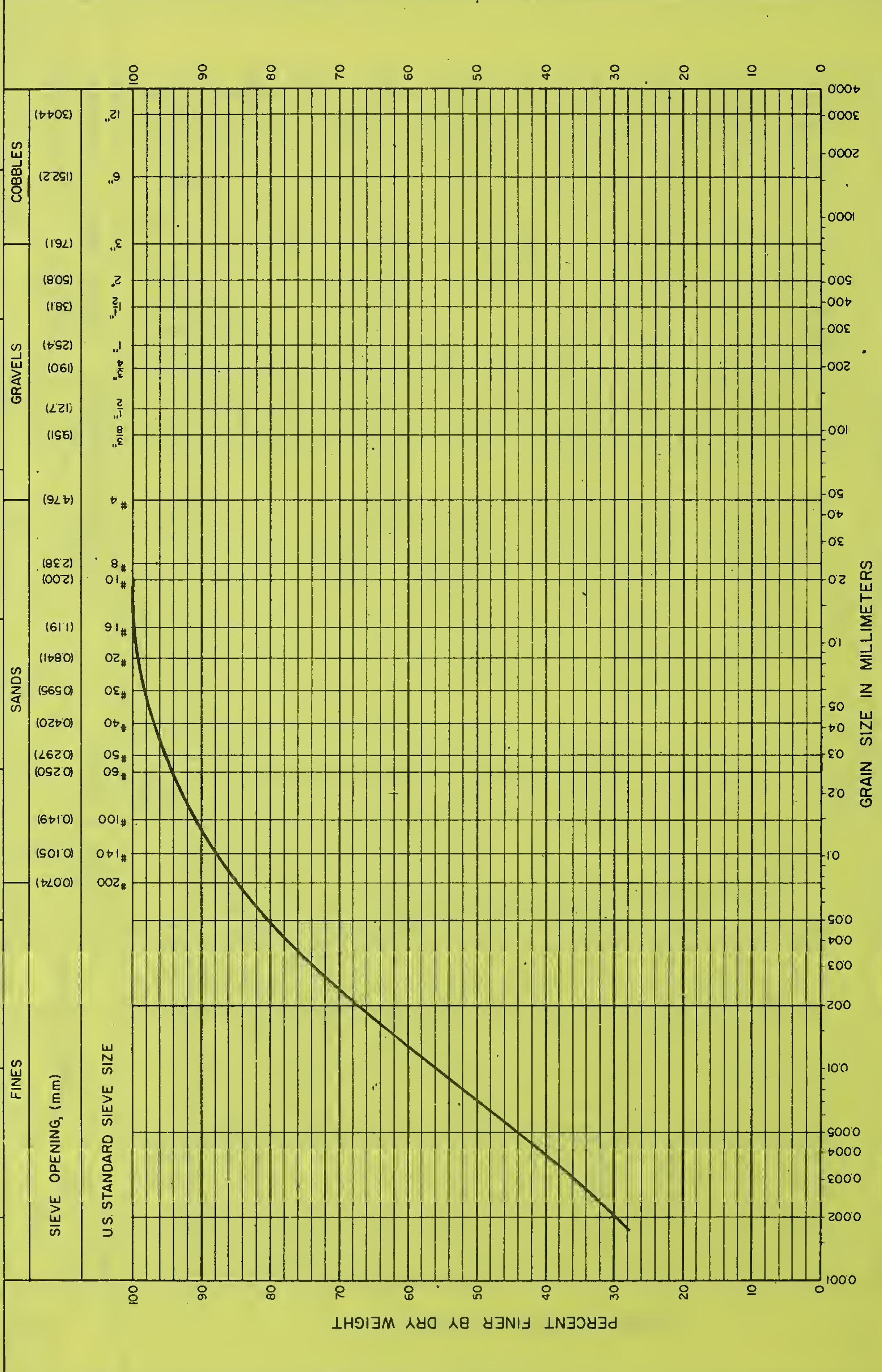
DATE: 12-67

SYMBOL: OL

DESCRIPTION: Dark gray, organic clayey silt A-7-5 (19)

GRAIN SIZE DISTRIBUTION

C_c	
C_u	
$D_{MAX.}$	2.0
D_{85}	0.08
D_{60}	0.013
D_{50}	0.007
D_{30}	0.002
D_{15}	
D_{10}	



UNDISTURBED CONDITION

MOISTURE	DRY UNIT WEIGHT
56.5 %	g/cc 57.7 pcf

SHRINKAGE LIMIT %

SOLUBLE SALTS %

ATTERBERG LIMITS

NATURAL MOISTURE		AIR DRY		OVEN DRY	
LL	PI	LL	PI	LL	PI
54	23	50	20	36	11

SPECIFIC GRAVITY (G_s)

(-) #4	(+) #4
2.44	

REMARKS:
Step 1 - Fine grained, median size less than 0.074 mm.
Step 2 - LL-PI plot below A-line, OL or ML.
Step 3 - LL_{OD}/LL_{AD} = 0.66, less than 0.7, class OL

9-64

LABORATORY NO.

MATERIALS TESTING REPORT	U. S. DEPARTMENT of AGRICULTURE SOIL CONSERVATION SERVICE	SOIL CLASSIFICATION

PROJECT and STATE	SAMPLE LOCATION
Typical Sample	Sta 8+05

FIELD SAMPLE NO.	DEPTH	GEOLOGIC ORIGIN
2.10	3.7 to 5.9 ft	

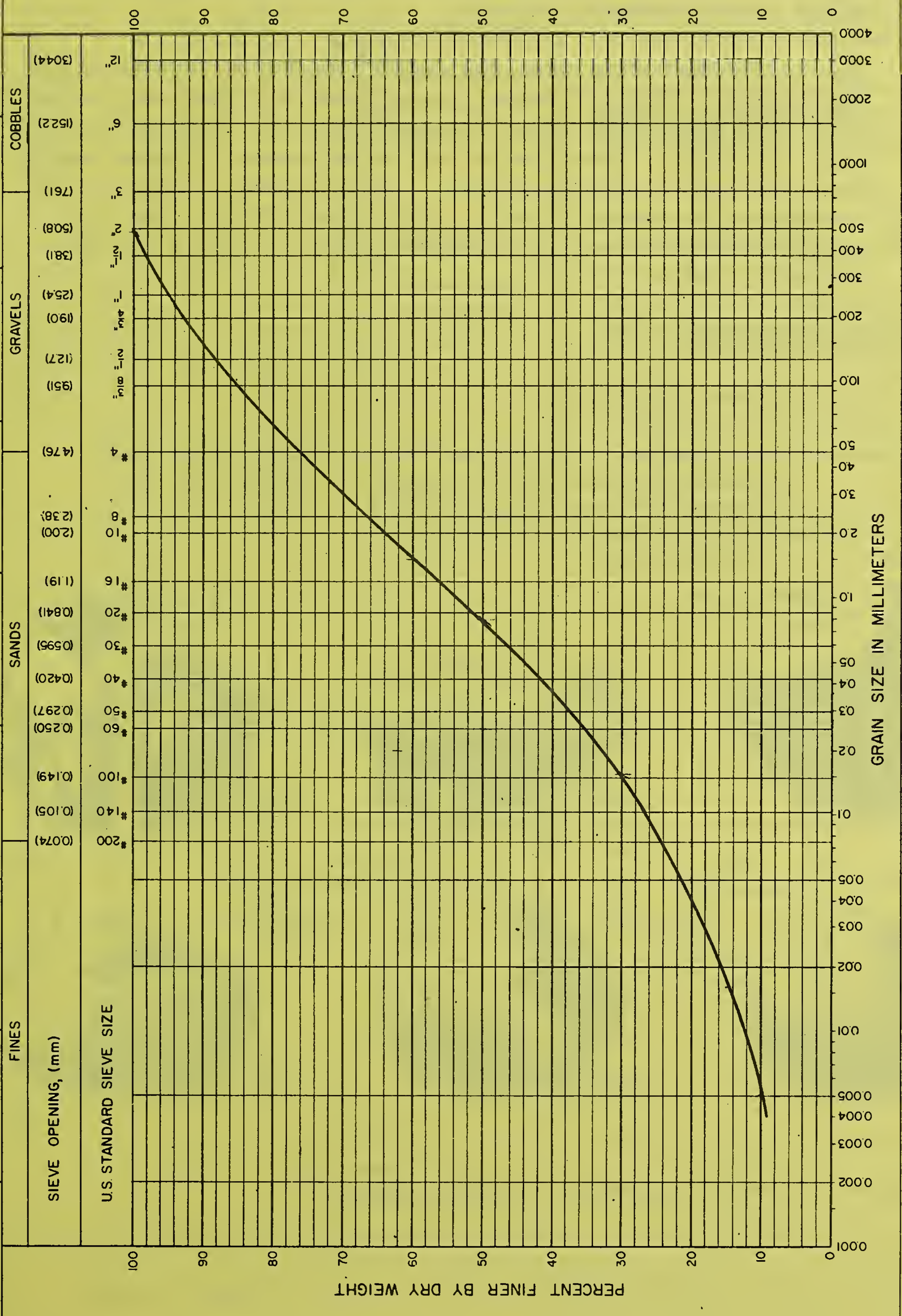
TYPE OF SAMPLE	TESTED AT	APPROVED BY	DATE
Bag	Portland, Oregon	JCS	12-67

SYMBOL SC

DESCRIPTION Reddish brown, gravelly, clayey sand A-2-6 (o)

GRAIN SIZE DISTRIBUTION

C_c	
C_u	
D_{MAX}	50.0
D_{85}	10.0
D_{60}	1.50
D_{50}	0.80
D_{30}	0.15
D_{15}	0.016
D_{10}	0.005



UNDISTURBED CONDITION			
MOISTURE		DRY UNIT WEIGHT	
%		g/cc	pcf

SHRINKAGE LIMIT	SOLUBLE SALTS
%	1.2 %

ATTERBERG LIMITS					
NATURAL MOISTURE		AIR DRY		OVEN DRY	
LL	PI	LL	PI	LL	PI
		28	11		

SPECIFIC GRAVITY (G_s)	
(-) #4	(+) #4

REMARKS: Step 1 - Median size is greater than 0.074 mm, coarse grained. Step 2 - 66% coarse fraction is sand size, sand. Step 3 - 24% fines; go to plasticity chart with LL-PI, plots above A-line, clay fines. Step 4 - Classify as sand with clay fines, SC.

6. Field Identification of Soils

a. General

The unified soil classification system is so devised that it is possible, with experience, to classify most soils correctly on the basis of field identification methods alone. The easiest way to learn field identification is under the guidance of an experienced man. While learning the procedures, a man should systematically compare the numerical test results for typical soils in each group with the "feel" of the materials while performing the field identification methods.

The following tests were developed largely by Professor A. Casagrande, Graduate School of Engineering, Harvard University, and have been widely adopted for use in identification of soils. These tests can be performed without equipment and are simple in nature. Do not make a decision on the basis of a single test. Use all applicable tests, then identify the soil. Identification criteria are presented in Table 2-2.

b. Test Methods

(1) Visual Inspection

Grain Shape. Observe and classify the sand and gravel particles as to degree of angularity or roundness. (See Figure 2-4.)

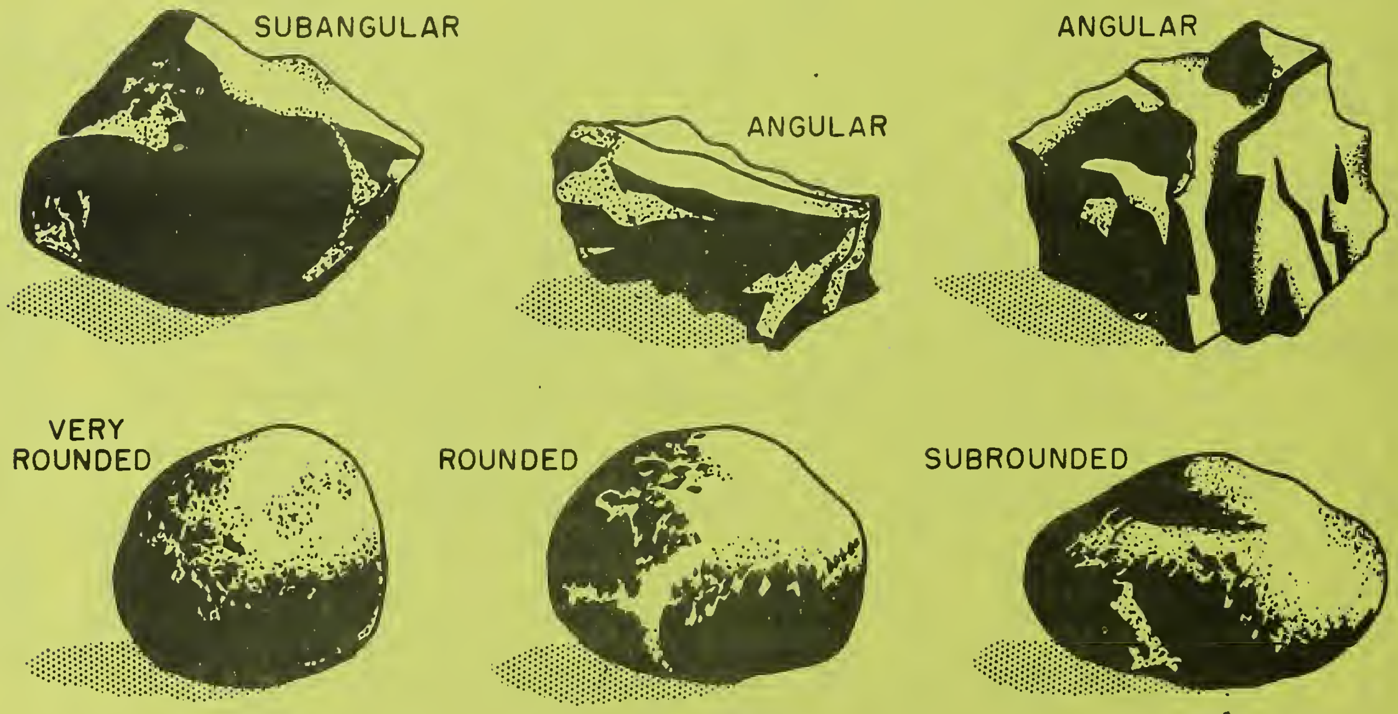


FIG. 2-4. GRAIN SHAPE

TABLE 2-2.

UNIFIED SOIL CLASSIFICATION

FIELD IDENTIFICATION

FIELD IDENTIFICATION PROCEDURES

<table>
<tr><td rowspan="8">COARSE GRAINED SOILS
More than half of material (by weight) is of individual grains visible to the naked eye.</td><td rowspan="15">No. 200 sieve size is about the smallest particle visible to the naked eye.</td><td rowspan="4">GRAVEL AND GRAVELLY SOILS
More than half of Coarse Fraction (by weight) is larger than $\frac{1}{4}$ inch size.</td><td rowspan="8">For visual classification the $\frac{1}{4}$ inch size may be used as equivalent to the No.4 sieve size</td><td rowspan="2">CLEAN GRAVELS
Will not leave a dirt stain on a wet palm.</td><td colspan="5">Wide range in grain sizes and substantial amounts of all intermediate partical sizes.</td><td>GW</td></tr>
<tr><td colspan="5">Predominately one size or a range of sizes with some intermediate sizes missing.</td><td>GP</td></tr>
<tr><td rowspan="2">DIRTY GRAVELS
Will leave a dirt stain on a wet palm.</td><td colspan="5">Nonplastic fines or fines with low plasticity (for identification of fines see characteristics of ML below.)</td><td>GM</td></tr>
<tr><td colspan="5">Plastic fines (for identification of fines see characteristics of CL or CH below)</td><td>GC</td></tr>
<tr><td rowspan="4">SAND AND SANDY SOILS
More than half of Coarse Fraction (by weight) is smaller than $\frac{1}{4}$ inch size</td><td rowspan="2">CLEAN SANDS
Will not leave a dirt stain on a wet palm.</td><td colspan="5">Wide range in grain size and substantial amounts of all intermediate partical sizes.</td><td>SW</td></tr>
<tr><td colspan="5">Predominantly one size or a range of sizes with some intermediate sizes missing</td><td>SP</td></tr>
<tr><td rowspan="2">DIRTY SANDS
Will leave a dirt stain on a wet palm.</td><td colspan="5">Nonplastic fines or fines with low plasticity (for identification of fines see characteristics of ML below).</td><td>SM</td></tr>
<tr><td colspan="5">Plastic fines (for identification of fines see characteristics of CL or CH below)</td><td>SC</td></tr>
<tr><td rowspan="7">FINE GRAINED SOILS
More than half of material (by weight) is of individual grains not visible to the naked eye.</td><td rowspan="4">SILTS AND CLAYS
(Low Liquid Limit)</td><td rowspan="7">See Identification Procedures</td><td>ODOR</td><td>DRY STRENGTH</td><td>DILATANCY (SHAKE) REACTION</td><td>TOUGHNESS</td><td>RIBBON (NEAR THE PL)</td><td>SHINE (NEAR THE PL)</td><td></td></tr>
<tr><td></td><td>Slight</td><td>Rapid</td><td>Low to None</td><td>None</td><td>Dull</td><td>ML</td></tr>
<tr><td></td><td>Medium to High</td><td>Medium to Slow</td><td>Medium</td><td>Weak</td><td>Slight to Shiny</td><td>CL</td></tr>
<tr><td>Pronounced Organic</td><td>Medium</td><td>Slow to None</td><td>Low (Spongy)</td><td>None</td><td>Dull to Slight</td><td>OL</td></tr>
<tr><td rowspan="3">SILTS AND CLAYS
(High Liquid Limit)</td><td></td><td>Medium</td><td>Very Slow to None</td><td>Medium to High</td><td>Weak to Strong</td><td>Slight</td><td>MH</td></tr>
<tr><td></td><td>Very High</td><td>None</td><td>High</td><td>Strong</td><td>Shiny</td><td>CH</td></tr>
<tr><td>Pronounced Organic</td><td>High</td><td>None</td><td>Low to Medium (Spongy)</td><td>Weak</td><td>Dull to Slight</td><td>OH</td></tr>
<tr><td colspan="5">HIGHLY ORGANIC SOILS</td><td colspan="5">Readily identified by color, odor, spongy feel and frequently by fibrous texture</td><td>Pt</td></tr>
</table>

Grain Size and Gradation. Sand and gravel sizes are readily identified by visual inspection. Individual grains below the smallest sand size cannot be seen by the naked eye and must be identified by other tests.

To estimate the gradation of coarse-grained soils, spread a representative sample out on a flat surface and observe the distribution or uniformity of grain sizes. Observe the proportion of fines and subject the fine-grained fraction to all tests described for fine-grained soils.

For gradation of fine-grained soils, shake the sample in a jar of water and allow it to settle out. Approximate gradation is indicated by the separation of the particles in the jar from top to bottom. Silt remains in suspension for at least one minute; clay, one hour or more.

(2) Shaking Test (Dilatancy)

The shaking test aids in the identification of fine-grained soils. After removing particles larger than No. 40 sieve size, prepare a pat of moist soil with a volume of about one-half cubic inch. Add enough water if necessary to make the soil soft but not sticky.

Place the pat in the open palm of one hand and shake horizontally, striking vigorously against the other hand several times (Figure 2.5). A positive reaction

Method of shaking

FIG. 2-5. SHAKING TEST

consists of the appearance of water on the surface of the pat which changes to a livery consistency and becomes glossy. When the sample is squeezed between the fingers, the water and gloss disappear from the surface, the pat stiffens, and finally it cracks or crumbles. Shake the broken pieces until they become livery and flow together again. Distinguish between slow, medium, and rapid reaction to the shaking test.

Rapid Reaction indicates a lack of plasticity, such as is the case with a typical inorganic silt, a rock flour, or a very fine sand.

Slow Reaction indicates a slightly plastic silt or silty clay.

No Reaction indicates a clay or a peaty (organic) material.

(3) Breaking Test (Dry Strength)

The breaking test may be used to determine the dry strength of a soil, which is a measure of its cohesiveness.

After removing particles larger than No. 40 sieve size, mold a pat of soil to the consistency of putty, adding water if necessary. Allow the pat to dry completely by oven, sun, or air drying, and then test its strength by breaking and crumbling between the fingers (Figure 2-6). This strength is a measure of the character and quantity of the colloidal fraction contained in the soil. The dry strength increases with increasing plasticity.

Method of crumbling soil
between fingers

FIG. 2-6. BREAKING TEST

Slight dry strength indicates an inorganic silt, a rock flour, or a silty sand. However, the sand feels gritty when the sample is powdered. The dried soil pat can be powdered with slight finger pressure.

Medium dry strength indicates a low to medium plastic inorganic clay. Considerable finger pressure is required to powder the sample.

High dry strength indicates a highly plastic, inorganic clay. The dried sample may be broken but cannot be powdered by finger pressure.

Note: Cohesion or high dry strength may be furnished by some cementing material such as calcium carbonate or iron oxide. For example, non-plastic lime rock or coral may develop high dry strength.

(4) Plasticity Tests

Plasticity is the physical property of a fine-grained soil which allows it to be kneaded into a putty-like consistency at the proper moisture content.

To test plasticity of a soil, prepare a moist pat and remove the coarse particles. Start rolling it out with the palm of the hand on a flat surface into approximately 1/8 inch diameter threads (See Figure 2-7. Fold the threads into a lump, roll it out again, and repeat the process until moisture loss causes the thread to crumble just when the 1/8-inch diameter is reached. The moisture content at this stage is called the plastic limit. Note the toughness of the threads as the plastic limit is reached and test to see if the crumbled pieces can be lumped together again.

A. Method of rolling thread.

B. Thread of soil above plastic limit.

C. Crumbling thread as plastic limit is reached.

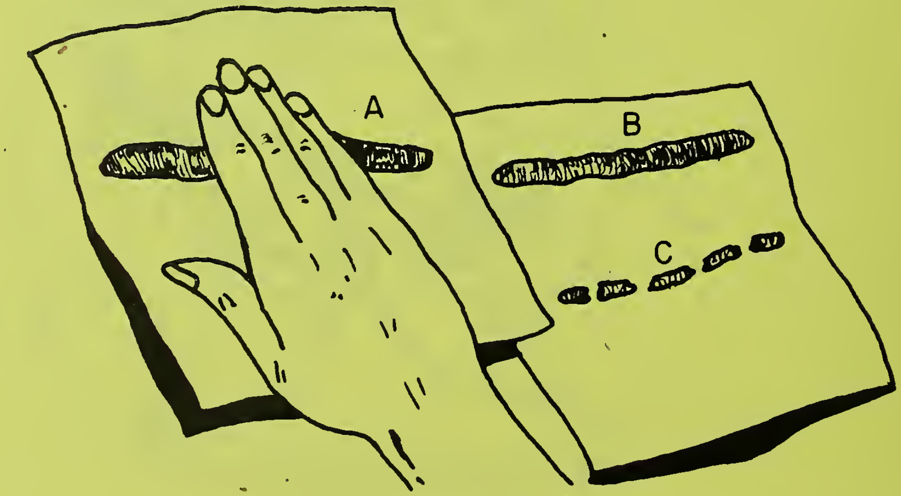


FIG. 2-7. PLASTICITY TEST

Any soil that can be readily rolled into a thread without crumbling is plastic.

High plasticity clay forms a tough thread which can be remolded into a lump below the plastic limit and deformed under high finger pressure without crumbling.

Medium plasticity soil forms a medium tough thread, but the lump crumbles soon after the plastic limit is reached.

Low plasticity soil forms a weak thread that cannot be lumped together below the plastic limit. Plastic soils containing organic material or much mica form threads which are very soft and spongy.

(5) Odor Test

Freshly sampled organic soils usually have a distinctive odor which aids in their identification. The odor can be made more apparent by heating a wet sample.

(6) Acid Test

Drop a little hydrochloric acid on a piece of soil. A fizzing reaction indicates calcium carbonate.

(7) Shine Test

Rub a dry or slightly moist sample with the finger nail or a knife blade. A shiny surface indicates a highly plastic clay; a dull surface indicates a silt or clay of low plasticity.

7. Description of Undisturbed Soil

The identity of a soil as determined by the laboratory and field tests should be supplemented by a description of the soil in its undisturbed condition. The description should include remarks concerning the following characteristics:

a. Color

The color of a soil in both the moist and dry condition should be described as accurately as possible by use of standard color designations. Acceptable soil color descriptions are: white, gray, black, brownish-black, reddish-gray, brownish-gray, orange, red brown, yellowish-brown, olive-brown, yellow, olive, blue,green, etc.,; these terms should be further modified by the adjectives light, medium, dark, or vivid. Use of the Munsell soil color charts and plates is suggested when more precise soil color descriptions are desired.

Mottles or streaks should be indicated and their description should follow the main color designation of the soil; for example, "stiff clay, plastic when moist, friable when dry; medium blue, mottled with brown, when moist; light green, mottled with brown when dry."

b. Texture

The term "texture" refers to the distinctive appearance and "feel" of the soil, which are direct indications of the fineness and uniformity of the soil. Texture should be described by a standard adjective as listed below:

Sharp - Typical of gravelly or sandy soils.
Gritty - Typical of coarse silts or sandy silts and clay.
Floury - Typical of fine silts.
Smooth - Typical of clays or fine silty clays.

c. Consistency

The term "consistency" refers to the degree of adhesion between soil particles or the resistance to deformation or rupture under applied pressure. Consistency of the moist soil in both the undisturbed and remolded states should be described by standard adjectives as listed below:

(1) Cohesionless soils (sands and silts)

Loose - Poorly graded, lacks binder or cementing agent, not well compacted.

Dense - Well graded, well compacted, may contain binder or cementing agent.

(2) Cohesive Soils (clays)

Very Soft - Easily penetrated several inches by fist..

Soft - Easily penetrated several inches by thumb.

Medium - Can be penetrated by thumb with moderate effort.

Stiff - Readily indented by thumb but penetrated only with a great effort.

Very Stiff - Readily indented by thumbnail.

Hard - Indented with difficulty by thumbnail.

Additional adjectives used in connection with the terms listed above in describing consistency of cohesive soils are: sticky, plastic, friable, brittle, jointed, stratified, varved.

d. Moisture Content.

The apparent moisture content of the soil should be described by such terms as dry, moist, wet, saturated, etc.

e. Cobbles and Boulders

The amount of each of the following sizes over 3 inches in diameter is of great importance in the selection of sources of and the use of embankment material.

3" to 6" - this size is generally left in the material used in an embankment. The amount of this size is necessary to evaluate the suitability of the proposed fill material.

Larger than 6" size - this size of material is generally excluded from the impervious zones of an embankment. The amount of this size must be known to determine waste volumes and its possible use in other zones of the proposed work.

C. AASHTO SOIL CLASSIFICATION SYSTEM

1. Soil Properties Used in Classification

This system identifies soils according to their textural and plasticity qualities and on their groupings with respect to behavior. This system is based on those characteristics of a soil that indicate how it will behave as an engineering construction material.

The soil properties that have been found most useful for this purpose and form the basis of the AASHTO soil classification system are:

a. Percentage of gravel, coarse sand, fine sand, and fines (fraction passing No. 200 sieve).

b. Plasticity and compressibility characteristics.

c. Group index.

These properties are determined by mechanical analysis, liquid limit and plastic limit tests and with experience can be estimated with good accuracy.

2. Definition of Soil Components

Component	Size Range
Boulders	Larger than 3" diameter
Gravel	Between No. 10 mesh and 3"
Coarse Sand	Between No. 40 mesh and No. 10 mesh
Fine Sand	Between No. 200 mesh and No. 40 mesh
Fines*	Smaller than No. 200 mesh

* Fines are classed as silty if the plastic index is equal to or less than 10 and clayey if it is equal to or greater than 11.

3. Summary of the AASHTO Classification System

a. Designates soils as fine or coarse according to percent passing the No. 200 mesh. Coarse soil, if less than 35% passes the No. 200 mesh. Fine soil, if more than 35%, passes the No. 200 mesh.

b. Coarse grain soils are further classified on the basis of:

(1) Grain size and distribution.

(2) Quantity of fines.

(3) Character of fines.

(4) Group index.

c. Fine grained soils are classified on the basis of:

(1) Liquid limit - high if LL is greater than 40
- low if LL is less than 40.

(2) Plasticity index - clay if greater than 10.
- silt if 10 or less.

(3) Group index.

d. Soil groups and group symbols of the AASHTO classification system:

(1) Granular materials - containing 35 percent or less passing the No. 200 sieve.

(a) Group A-1. The typical material of this group is a well graded mixture of stone fragments or gravel, coarse sand, fine sand, and a nonplastic or feebly plastic soil binder. This group includes stone fragments, gravel, coarse sand, volcanic cinders, etc. without soil binder.

1 Subgroup A-1-a includes those materials consisting predominantly of stone fragments or gravel, either with or without a well graded binder of fine material.

2 Subgroup A-1-b includes those materials consisting predominantly of coarse sand either with or without a well graded soil binder.

(b) Group A-3. The typical material of this group is fine beach sand or fine desert blow sand without silty or clay fines or with a very small amount of nonplastic silt. The group may include stream-deposited mixtures of poorly graded fine sand and limited amounts of coarse sand and gravel.

(c) Group A-2. This group includes a wide variety of "granular" materials which are mixtures of the materials falling in Groups A-1 and A-3 and the silt-clay materials of Groups A-4, A-5, A-6, and A-7. It includes all materials containing 35% or less passing the No. 200 sieve which cannot be classified as A-1 or A-3 because of fines content or plasticity or both.

1 Subgroups A-2-4 and A-2-5 include various granular materials containing 35% or less passing the No. 200 sieve and with a minus No. 40 portion having the characteristics of the A-4 and A-5 groups. These groups include such materials as gravel and coarse sand with silt contents or plasticity indexes in excess of the limitations of Group A-1, and fine sand with nonplastic silt content in excess of the limitations of Group A-3.

2 Subgroups A-2-6 and A-2-7 include materials similar to those described under Subgroups A-2-4 and A-2-5, except that the fine portion contains plastic clay having the characteristics of the A-6 and A-7 group.

(2) Silt-clay materials containing more than 35 percent passing the No. 200 sieve.

(a) Group A-4. The typical material of this group is a nonplastic or moderately plastic silty soil usually having 75% or more passing the No. 200 sieve. The group also includes mixtures of fine silty soil and up to 65% of sand and gravel retained on No. 200 sieve.

(b) Group A-5. The typical material of this group is similar to that described under Group A-4, except that it is usually of diatomaceous or micaceous character and may be highly elastic, as indicated by the high liquid limit.

(c) Group A-6. The typical material of this group is a plastic clay soil usually having 75% or more passing the No. 200 sieve. The group also includes mixtures of fine clayey soil and up to 64% of sand and gravel retained on the No. 200 sieve. Materials of this group usually have high volume change between wet and dry states.

(d) Group A-7. The typical material of this group is similar to that described under Group A-6, except that it has the high liquid limits characteristic of the A-5 group, and may be elastic as well as subject to high volume change.

  1 Subgroup A-7-5 includes those materials with moderate plasticity indexes in relation to liquid limit and which may be highly elastic as well as subject to considerable volume change.

  2 Subgroup A-7-6 includes those materials with high plasticity indexes in relation to liquid limit and which are subject to extremely high volume change.

(e) Group A-8. These soils are highly organic peats or mucks. Classification of these materials is based on visual inspection, and is not dependent on the percentage passing the No. 200 sieve, liquid limit, or plasticity index. The material is primarily composed of partially decayed organic matter, generally has a fibrous texture, dark brown or black color, and a very noticeable odor of decay.

4. Method of Classification (Laboratory Test Data)

a. For all soils.

(1) Step No. 1. Determine the group index from the formula:

$$GI = (F-35)[0.2 + 0.005(LL-40)] + 0.01(F-15)(PI-10)$$

in which

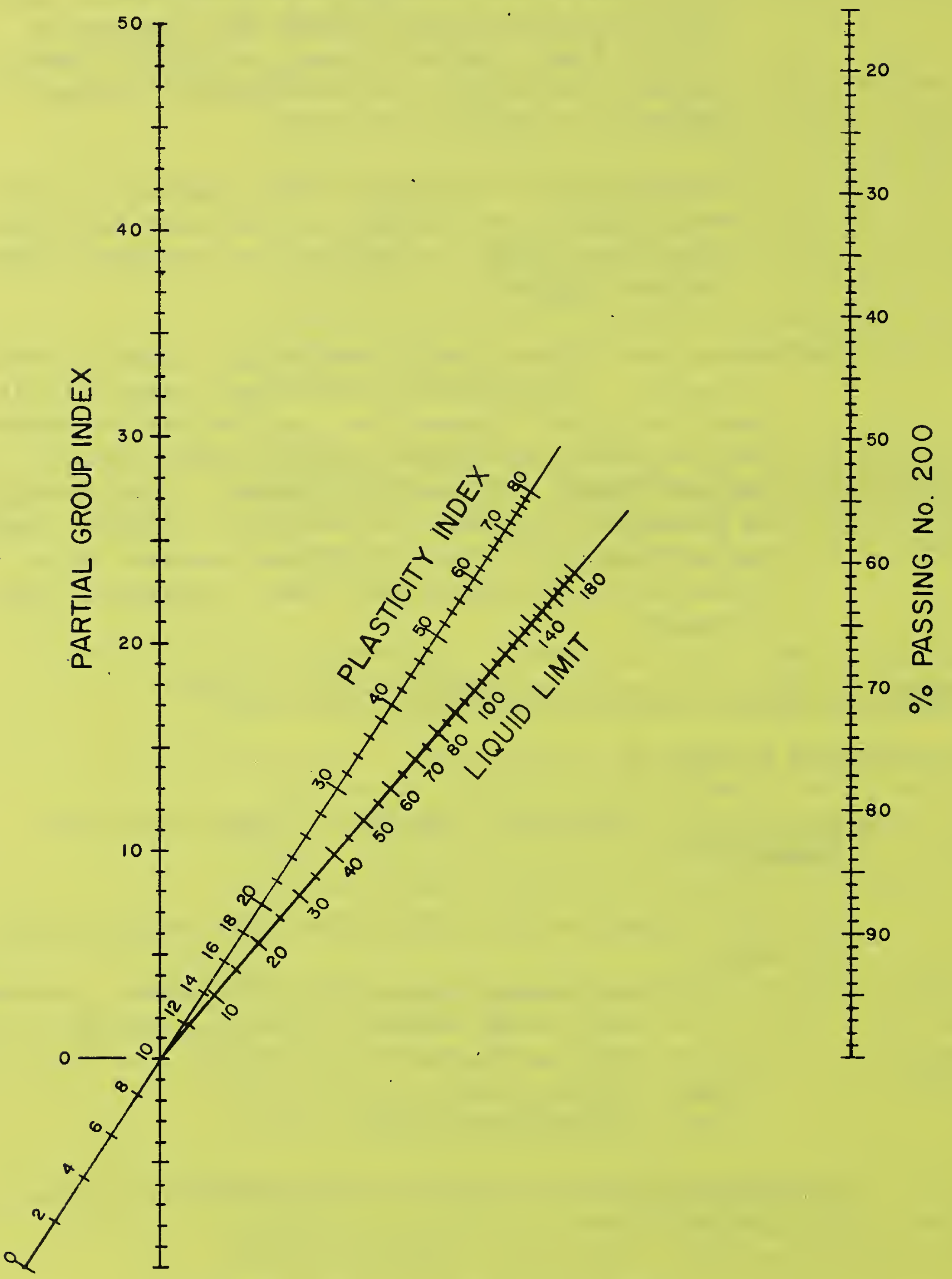
F = percentage passing No. 200 sieve, expressed as a whole number. This percentage is based only on the material passing the 3-in. sieve.
LL = liquid limit
PI = plasticity index

The group index may also be determined by the use of Figure 2-8.

Group Index (GI) = $(F-35)\left[0.2+0.005(LL-40)\right]+0.01(F-15)(PI-10)$
where F = % Passing No. 200 sieve, LL = Liquid Limit, and
PI = Plasticity Index.

When working with A-2-6 and A-2-7 subgroups the Partial Group Index (PGI) is determined from the PI only.

When the combined Partial Group Indexes are negative, the Group Index should be reported as zero.



Example:

82 % Passing No. 200 sieve
LL = 38
PI = 21

Then:

PGI = 8.9 for LL
PGI = 7.4 for PI
GI = 16

FIG. 2-8. GROUP INDEX CHART

(a) If calculated group index is negative, report group index as zero.

(b) Report group index to the nearest whole number.

(c) In calculating group index of A-2-6 and A-2-7 soils, use only PI portion of formula or of Figure 2-8.

(2) Step No. 2. Classify as coarse grained or fine grained soil.

Coarse grained if less than 35% passing the No. 200 mesh. Fine grained if more than 35% passing the No. 200 mesh.

b. For fine grained soils.

Step No. 3. Determine soil group from Table 2-3 and Figure 2-9.

c. For coarse grained soils.

(1) Step No. 3. Determine percentages passing the No. 10, 40, and 200 mesh and determine soil groups in the coarse grained portion of Table 2-3.

(2) Step No. 4. Determine liquid limit and plasticity index and determine soil group using Table 2-3 and Figure 2-9.

Example problems on page 2-12 to 2-21 have AASHTO class shown.

The position of the soils groups in the AASHTO classification system on a plasticity chart are shown in Figure 2-10.

AASHTO CLASSIFICATION OF SOILS AND SOIL-AGGREGATE MIXTURES

General Classification	Granular Materials (35 % or less passing No. 200)							Silt-Clay Materials (More than 35% passing No. 200)			
Group Classification	A-1			A-2							A-7
	A-1-a	A-1-b	A-3	A-2-4	A-2-5	A-2-6	A-2-7	A-4	A-5	A-6	A-7-5 A-7-6
Sieve Analysis, Percent passing:											
No. 10	50 max.	—	—	—	—	—	—	—	—	—	—
No. 40	30 max.	50 max.	51 min.	—	—	—	—	—	—	—	—
No. 200	15 max.	25 max.	10 max.	35 max.	35 max.	35 max.	35 max.	36 min.	36 min.	36 min.	36 min.
Characteristics of Fraction passing No. 40:											
Liquid limit	—		—	40 max.	41 min.	40 max.	41 min.	40 max.	41 min.	40 max.	41 min.
Plasticity index	6 max.		N.P.	10 max.	10 max.	11 min.	11 min.	10 max.	10 max.	11 min.	11 min.[a]
Usual types of Significant Constituent Materials	Stone Fragments, Gravel and Sand		Fine Sand	Silty or Clayey Gravel and Sand				Silty Soils		Clayey Soils	
General Rating as Subgrade	Excellent to Good					Fair to Poor					

[a] Plasticity index of A-7-5 subgroup is equal to or less than LL minus 30. Plasticity index of A-7-6 subgroup is greater than LL minus 30.

TABLE 2-3

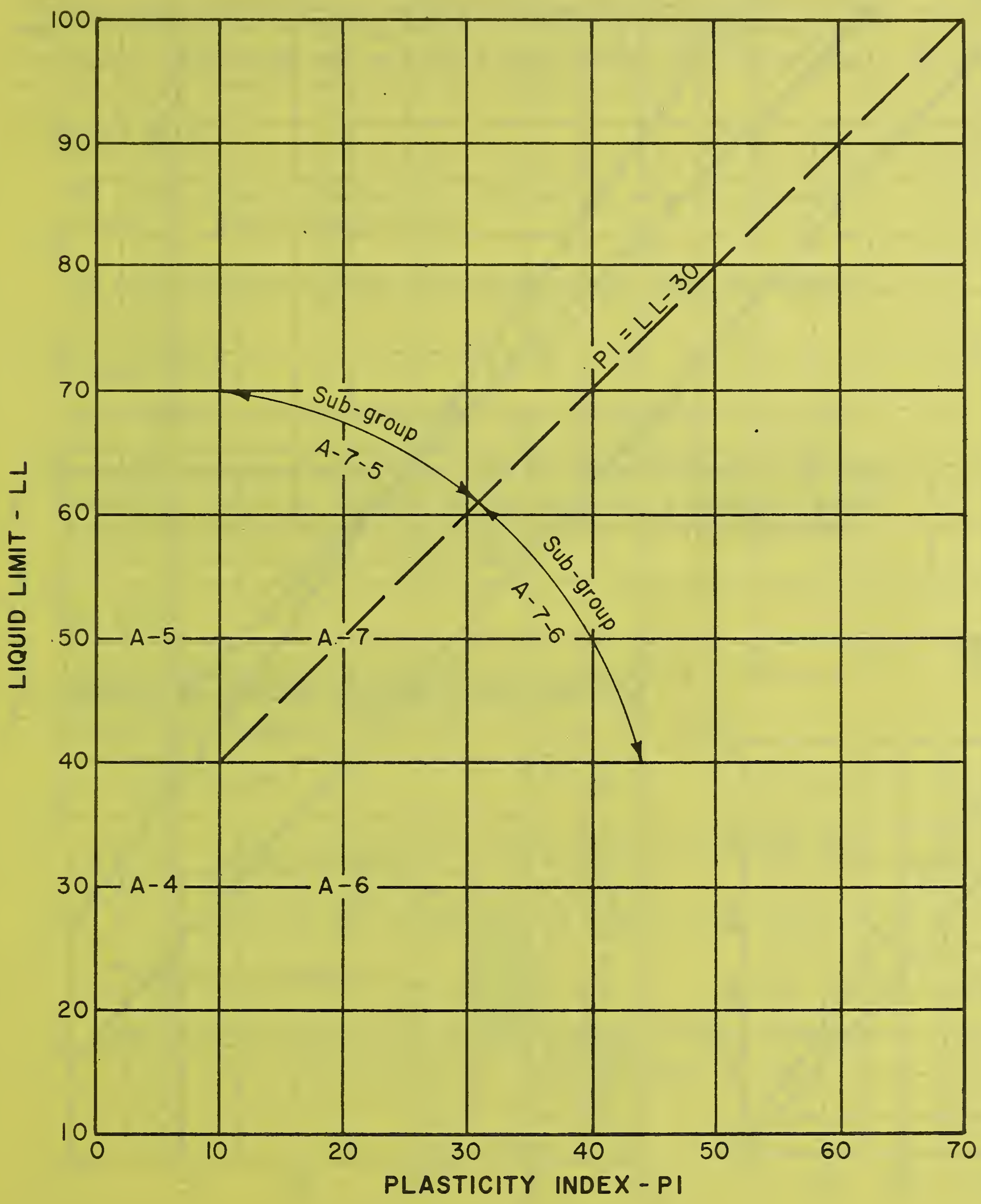


FIG. 2-9. Liquid limit and plasticity index ranges for the A-4, A-5, A-6 and A-7 Subgrade Groups (AASHTO System)

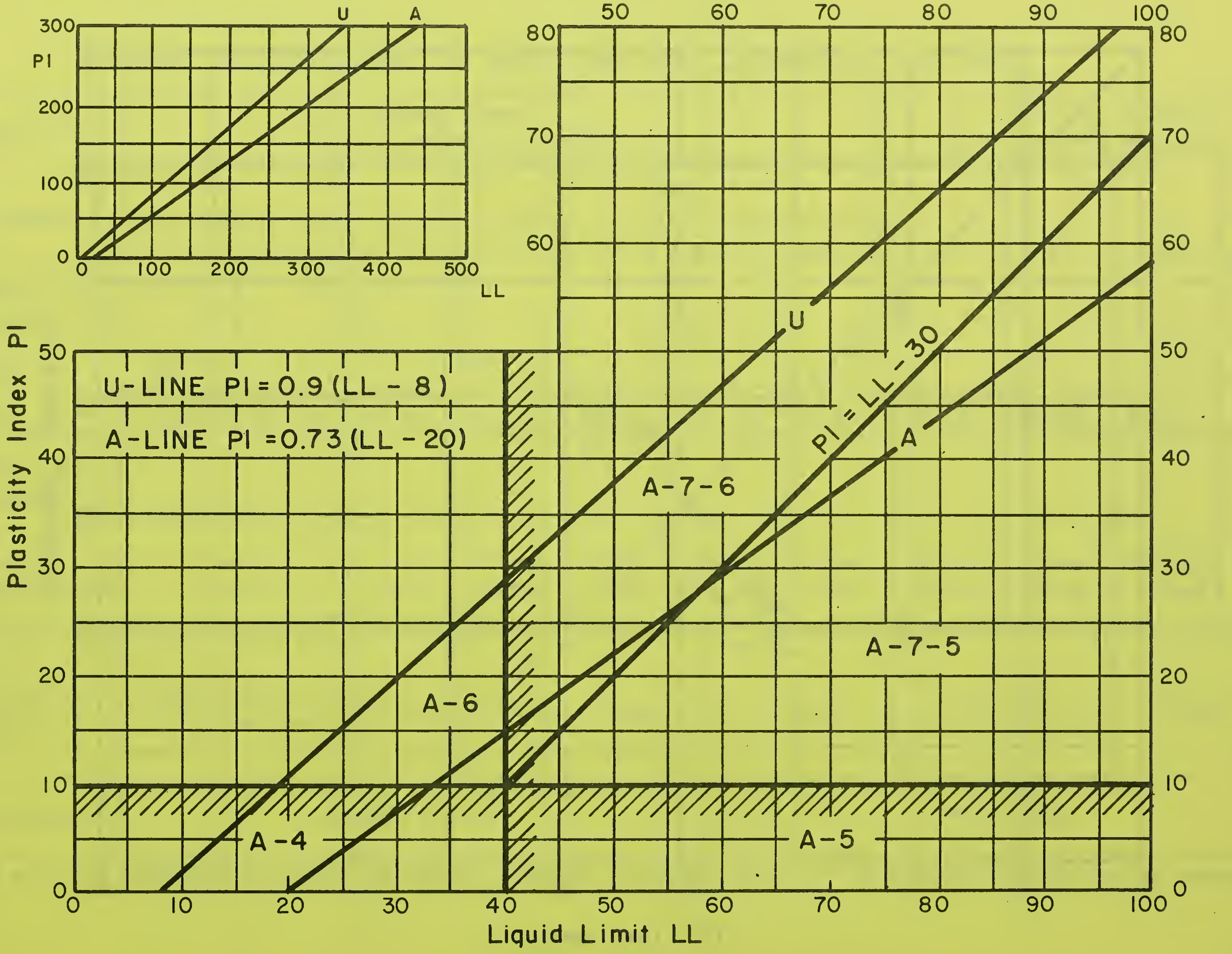


FIG. 2-10. AASHTO SOIL CLASSIFICATION GROUP POSITIONS ON PLASTICITY CHART

SECTION 3. THE SOIL-AIR-WATER SYSTEM

A. SOIL MASS AS A MULTIPHASE SYSTEM

The soil mass consists of a framework of solid particles of varying size and shape enclosing voids of varying size between the particles. The voids may be filled with water, air, or partly with water and partly with air (see Figure 3-1).

1. Solid Phase

The solid phase includes all soil particles with included organic or other solid matter.

The soil particles range from boulders to fines as defined in Section 2.

2. Liquid Phase

For purposes of soil mechanics, the liquid phase is considered to be water, however under field conditions it may include dissolved salts, oil, gas or other liquids. In soil mechanics, water is defined as the material evaporated when a soil sample is dried to constant weight at a temperature of 105 to 110 degrees centigrade.

3. Gas Phase

In soil mechanics, the gas phase is considered to be air.

4. Weight Relationships (see Fig. 3-1 and Fig. 3-2)

a. Nomenclature

W_s = Weight of the solid phase of the soil (oven dry).
W_w = Weight of water.
W_a = Weight of gas (assumed to be zero).
W = Total weight of the mass $= W_s + W_w + W_a = W_s + W_w$

b. Moisture Content = w

The ratio of the weight of water to the weight of solids in the soil mass expressed as a percentage.

$$w = \frac{W_w}{W_s} (100)$$

c. Specific Gravity

(1) Specific gravity of solids = G_s

Ratio of the weight in air of a given volume of soil solids at a stated temperature to the weight in air of an equal volume of distilled water at a stated temperature.

$$G_s = \frac{\gamma_s}{\gamma_o}$$

where γ_s = unit weight of solids
γ_o = unit weight of water at 4° C.

(2) Apparent specific gravity = G_a or G

Ratio of the weight in air of a given volume of the impermeable material (that is the solid matter including its impermeable voids) at a stated temperature to the weight in air of an equal volume of distilled water at a stated temperature.

$$G_a = G = \frac{\gamma_{s+v}}{\gamma_o}$$

where γ_{s+v} = unit weight of solids plus impermeable voids

(3) Bulk specific gravity (specific mass gravity) = G_m

Ratio of the weight in air of a given volume of permeable material (including both permeable and impermeable voids normal to the material) at a stated temperature to the weight in air of an equal volume of distilled water at a stated temperature.

$$G_m = \frac{\gamma_t}{\gamma_o}$$

where γ_t = unit weight of soil mass

5. Volume Relations (see Fig. 3-1 and Fig. 3-2)

a. Nomenclature

V_s = Volume of solid phase of soil
V_w = Volume of water
V_a = Volume of gas
V_v = Volume of voids = $V_w + V_a$

V = Total volume of soil mass

$V = V_s + V_a + V_w = V_s + V_v$

b. Porosity = n

The ratio of the volume of voids to the total volume of the mass, expressed as a percentage.

$$n = \frac{V_v}{V} (100) = \frac{e}{1+e} (100)$$

c. Void ratio = e

The ratio of the volume of voids to the volume of solids in the soil mass.

$$e = \frac{V_v}{V_s} = \frac{n}{100-n}$$

d. Degree of saturation = S

The ratio of the volume of water to the volume of voids, expressed as a percentage.

$$S = \frac{V_w}{V_v} (100)$$

6. Volume and Weight Relationships

a. Total unit weight = γ_t

The ratio of the total weight to the total volume

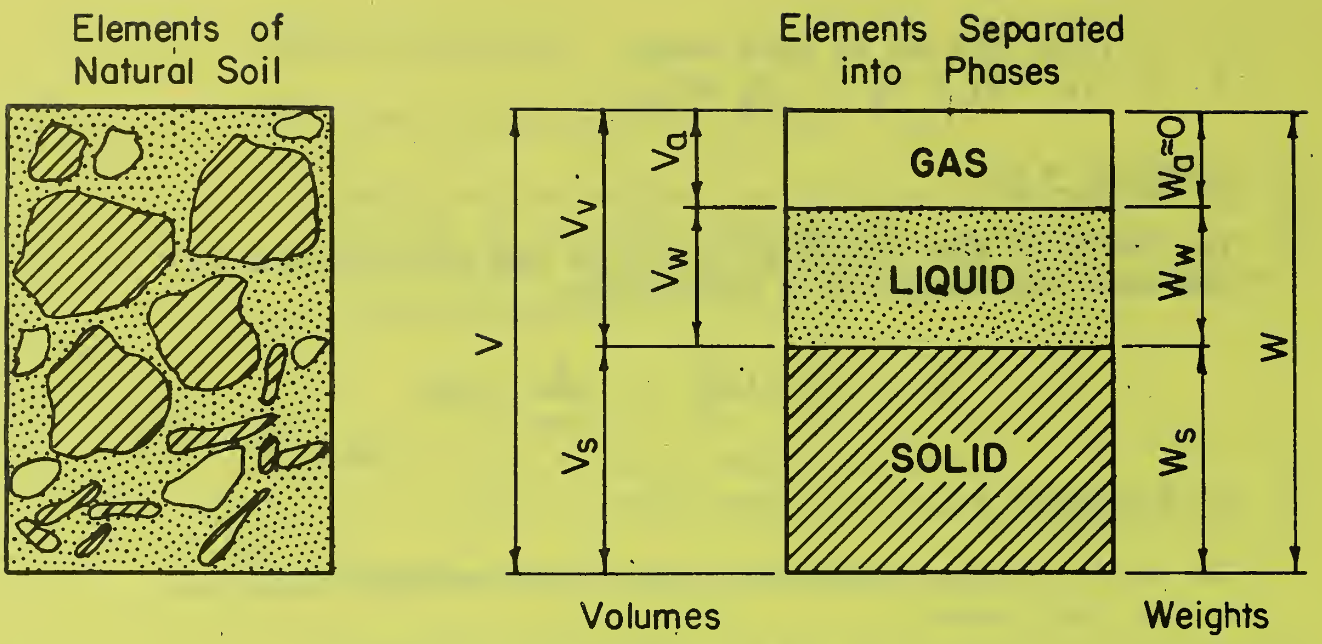
$$\gamma_t = \frac{W}{V} = \frac{G_s + S_e}{1+e} \gamma_w = \frac{1+w}{1+e} G_s \gamma_w$$

b. Unit weight of water = γ_w = 62.4 pcf.

c. Dry unit weight = γ_d

The ratio of the weight of solids to the total volume.

$$\gamma_d = \frac{W_s}{V} = \frac{G_s}{1+e} \gamma_w = \frac{G_s \gamma_w}{1+\frac{wG_s}{S}} \quad \frac{\gamma_t}{1+w}$$



VOLUME

Porosity, $n = \frac{V_v}{V}(100)$

Void Ratio, $e = \frac{V_v}{V_s}$

Degree of Saturation $S = \frac{V_w}{V_v}(100)$

$n = \frac{e}{1+e}(100)$; $e = \frac{n}{100-n}$

WEIGHTS

Water Content, $w = \frac{W_w}{W_s}(100)$

SPECIFIC GRAVITY

Mass, $G_m = \frac{\gamma_t}{\gamma_o}$ γ_o = Unit weight of water at 4°C

Apparent, $G_a = \frac{\gamma_{s+v}}{\gamma_o}$

Solids, $G_s = \frac{\gamma_s}{\gamma_o}$

UNIT WEIGHT

Total, $\gamma_t = \frac{W}{V} = \frac{G_s + Se}{1+e}\gamma_w = \frac{1+w}{1+e}G_s\gamma_w$

Solids, $\gamma_s = \frac{W_s}{V_s}$

Water, $\gamma_w = 62.4 \text{ pcf}$

Dry $\gamma_d = \frac{W_s}{V} \equiv \frac{G_s}{1+e}\gamma_w = \frac{G_s\gamma_w}{1+\frac{wG_s}{S}} = \frac{\gamma_t}{1+w}$

Submerged, (Buoyant) $\gamma_b = \frac{G_s - 1 - e(1-S)}{1+e}\gamma_w$

Submerged, Saturated Soil $\gamma_b = \frac{G_s - 1}{1+e}\gamma_w$; $\gamma_b = \gamma_{sat.} - \gamma_w$

$G_s w = Se$

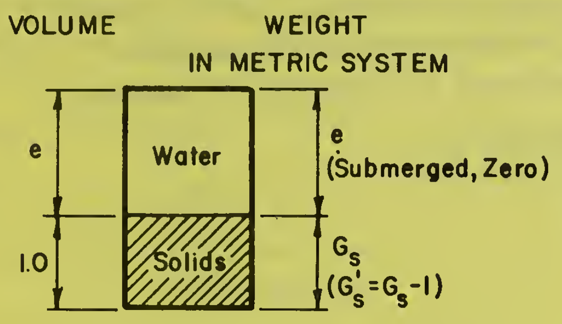
RELATIONSHIP AMONG SOIL PHASES

FIGURE 3-1

a

Given : e , G_s

ASSUME $V_s = 1$



FULLY SATURATED SOIL

$$w = \frac{e}{G_s}$$

$$G_m = \frac{G_s + e}{1 + e}$$

$$\gamma_m = G_m \gamma_w = \frac{G_s + e}{1 + e} \gamma_w$$

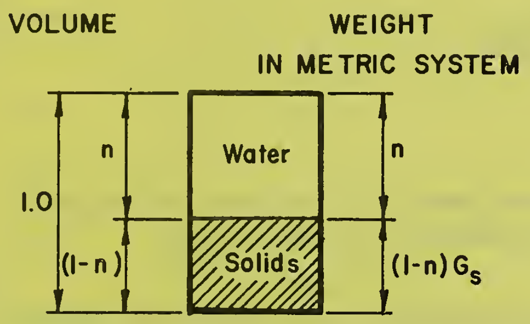
$$\gamma_b = \gamma_m - \gamma_w$$

$$\gamma_d = \frac{G_s}{1 + e} \gamma_w$$

b

Given : n , G_s

ASSUME $V = 1$



FULLY SATURATED SOIL

$$w = \frac{n}{(1-n)G_s}$$

$$G_m = n + (1-n) G_s$$

$$\gamma_m = G_m \gamma_w = \left[n + (1-n) G_s \right] \gamma_w$$

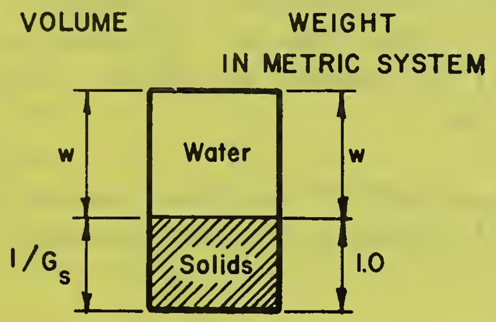
$$\gamma_b = \gamma_m - \gamma_w$$

$$\gamma_d = (1-n) G_s \gamma_w$$

c

Given : w, G_s

ASSUME $W_s = 1$



FULLY SATURATED SOIL

$$e = \frac{w}{1/G_s} = wG_s$$

$$G_m = \frac{1 + w}{w + 1/G_s} = \frac{(1+w) G_s}{w G_s + 1}$$

$$\gamma_m = G_m \gamma_w = \frac{(1+w) G_s}{w G_s + 1} \gamma_w$$

$$G'_m = G_m - 1 = \frac{G_s - 1}{w G_s + 1}$$

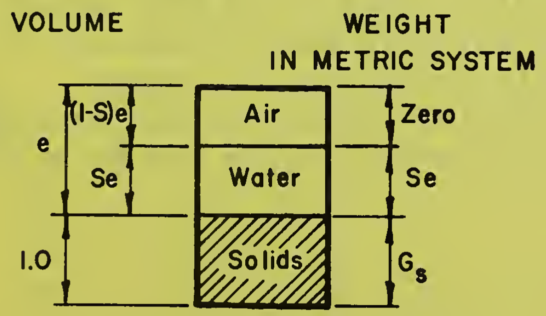
$$\gamma_b = \gamma_m - \gamma_w$$

$$\gamma_d = \frac{1}{w + 1/G_s} \gamma_w = \frac{G_s}{w G_s + 1} \gamma_w$$

d

Given : e, S, Gs

ASSUME $V_s = 1$



PARTIALLY SATURATED SOIL

$$w = \frac{Se}{G_s}$$

$$G_m = \frac{G_s + Se}{1 + e}$$

$$\gamma_m = \frac{G_s + Se}{1 + e} \gamma_w$$

$$\gamma_b = \frac{G_s - 1 - e(1-S)}{1 + e} \gamma_w$$

$$\gamma_d = \frac{G_s}{1 + e} \gamma_w$$

FIG. 3-2 - WEIGHT-VOLUME RELATIONSHIPS

d. Submerged or buoyant unit weight = γ_b

The buoyant unit weight is the effective weight when a soil mass is submerged in a body of water.

$$\gamma_b = \frac{G_s-1-e(1-S)}{1+e} \gamma_w$$

or when $S = 100\%$

$$\gamma_b = \frac{G_s-1}{1+e} \gamma_w$$

and $\gamma_b = \gamma_{sat} - \gamma_w$

B. CONSISTENCY AND MECHANICAL PROPERTIES

1. Gradation

The distribution of the various particle sizes throughout a mass of soil is known as the grading of the soil.

The gradation of a soil is determined by a process termed mechanical analysis. This process includes the sieve analysis for larger particles and the sedimentation or hydrometer analysis for smaller particles.

The knowledge of the grain size distribution alone does not allow an accurate estimate of such properties as permeability, strength, and cohesion. Supplemental information including density and consistency is needed to permit better estimation of these soil properties.

The effects of gradation on soil behavior can best be illustrated by discussing some typical grain size distribution curves.

The three curves shown on Figure 3-3 represent specimens each having the same range of particle sizes but different gradations.

Curve A represents a specimen with an even distribution of particle sizes. Such a material is said to be well graded. A material with this gradation may have good resistance to erosion or scour, may be readily compacted to a dense condition, may develop high shearing resistance and bearing capacity, and have relatively low permeability and be slightly compressible.

LABORATORY NO.

MATERIALS TESTING REPORT	**U. S. DEPARTMENT of AGRICULTURE SOIL CONSERVATION SERVICE**	**SOIL CLASSIFICATION**

PROJECT and STATE *Sample Curves*	SAMPLE LOCATION

FIELD SAMPLE NO.	DEPTH	GEOLOGIC ORIGIN

TYPE OF SAMPLE	TESTED AT	APPROVED BY	DATE

SYMBOL	DESCRIPTION

GRAIN SIZE DISTRIBUTION

D$_{10}$

D$_{15}$

D$_{30}$

D$_{50}$

D$_{60}$

D$_{85}$

D$_{MAX.}$

C$_u$

C$_c$

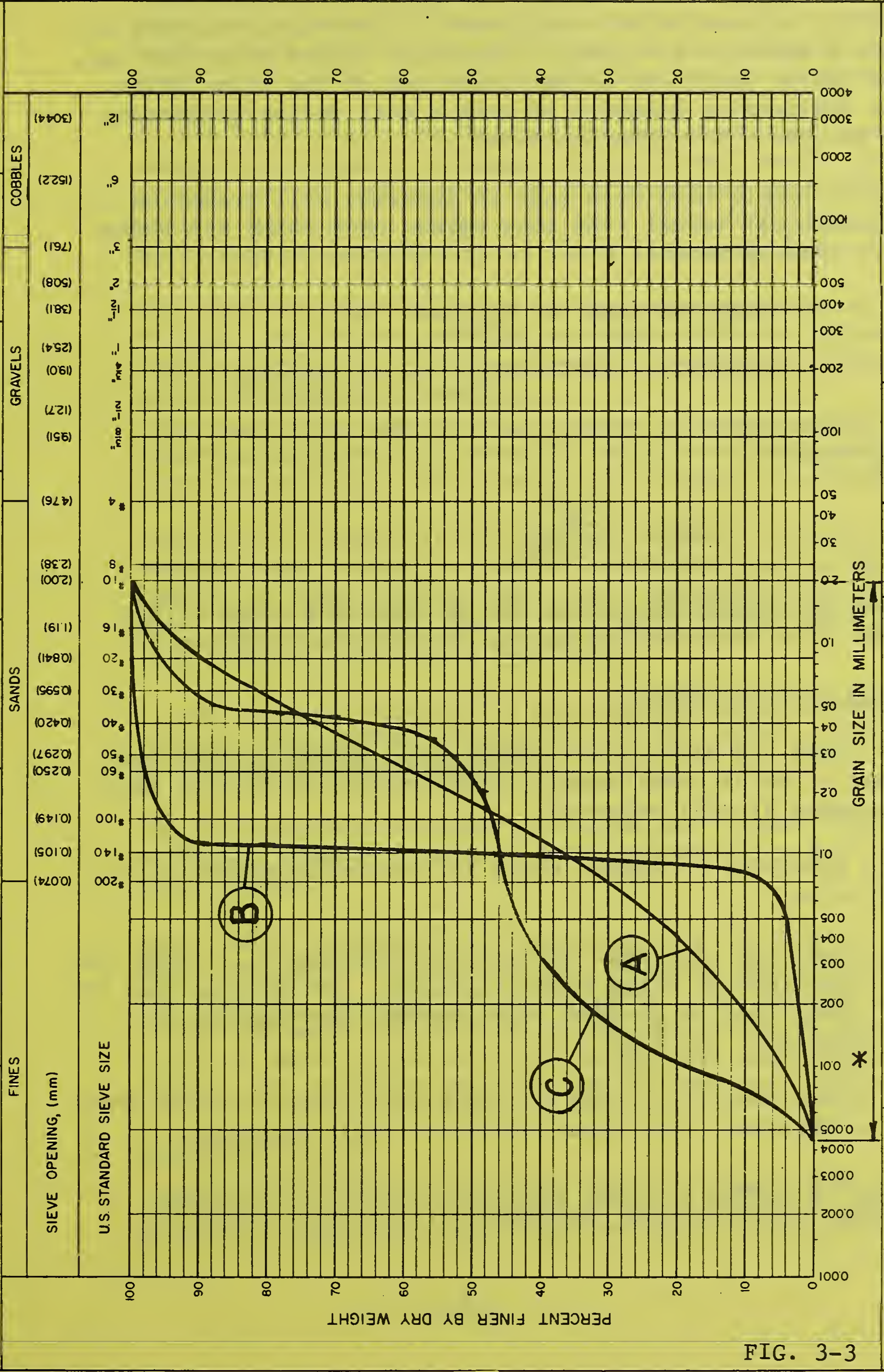


FIG. 3-3

UNDISTURBED CONDITION

DRY UNIT WEIGHT g/cc pcf

MOISTURE %

SHRINKAGE LIMIT %

SOLUBLE SALTS %

ATTERBERG LIMITS

NATURAL MOISTURE		AIR DRY		OVEN DRY	
LL	PI	LL	PI	LL	PI

SPECIFIC GRAVITY (G$_s$)

(-) #4	(+) #4

REMARKS: * All three soils have the same range of grain sizes

Curve B represents a material that is primarily composed of particles of one size. This material is considered to be poorly graded. Such a material contains insufficient fines to fill the voids between the larger particles and consequently will have an open porous structure even with heavy compaction. As a result it will probably be more easily displaced under load, be highly permeable, and have less supporting power and lower shearing strength.

Curve C is said to be skip-graded. The characteristics of such a material are usually dominated by the finer faction, the coarse particles being inclusions in the fine matrix. The engineering behavior of such a soil defies prediction on the basis of gradation alone.

Many attempts have been made to describe soil gradation in mathematical terms. The most common used terms are those developed by Hazen:

Effective size $= D_{10}$ and

Uniformity coefficient, $C_u = D_{60}/D_{10}$

In common usage in recent years is the coefficient of curvature

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

These relationships define the position and shape of the grain size distribution curve, not the soil particles.

2. Grain Size and Shape

The shape and size of individual soil particles have been found to be of considerable importance in explaining many phenomena of soil behavior.

With reference to shape, there are two major groups of soil particles:

a. Those that are more or less equidimensional. These are described as being bulky in shape. The bulky grains are classified as rounded, sub-rounded, sub-angular or angular. See Fig. 2-4.

b. Those that are platey or scale-like, many of which have a thickness less than 1/100th of their diameter.

This difference in shape reflects a difference in the crystal structure and characteristics of the mineral of which the particle is composed. Primary minerals have no plane of marked weakness in any one direction. Consequently, particles of the primary minerals are generally bulky in shape. Secondary minerals have planes of weakness in at least one direction and are subject to what is known as basal cleavage. As a result, particles of the secondary minerals are of platey shape.

The mechanical processes of rock weathering normally does not break up or grind the hard equidimensional bulky mineral grains much smaller than 0.01 mm. As a consequence most sand, gravel and coarse silt size particles are of this shape.

On the other hand, the friable flake-shaped particles of secondary minerals, although initially very small, are readily ground, broken and chemically disintegrated into still smaller particles. Hence, the very fine fractions of natural soils consist principally of flake-like particles.

The size and shape of bulky grains have an important bearing on that part of the shearing strength which is influenced by internal friction. The more angular particles offering the higher frictional strengths.

The bulky grained soil normally has greater permeability, is less compressible, and is easier to compact to a high density.

The state of compaction of natural deposits of granular soils depends upon the size of their constituent particles. A deposit of fine sand may exist in a relatively loose condition while a gravel deposit is nearly always found to be in a comparatively dense condition.

It was originally proved by Atterberg that only particles of colloid size exhibit plasticity and only then if they were plate-like instead of bulky.

The flat, platey or scaley particles have lower frictional strength, greater cohesion, lower permeability and are more compressible.

Again it is emphasized that gradation, grain size, and shape when used alone to estimate soil behavior can give misleading results. However, an experienced soil engineer may obtain valuable indications of the properties of a soil from study of these characteristics along with the Atterberg limits, the in-place density, and the geologic origin.

3. Relative Density

Deposits of granular soils are found in nature in varying states of compaction from loose to dense. Generally, the engineering properties of these materials are largely controlled by their degree of compaction.

A dense soil consisting of well rounded sand particles essentially of one size may have a higher shear strength than a loose soil composed of a well graded mixture of angular sand and gravel particles.

When subjected to a static load, even relatively loose granular soils are virtually non-compressible. If vibrations or shear stresses are applied to a loose granular soil, substantial volume changes will occur. The reduction in volume of such soils can result in excessive differential movement and much damage to a structure founded on the material. This type of volume change generally occurs almost instantaneously and is much affected by the location and intensity of the applied dynamic load. These soils when under the water table and subjected to vibratory loads or shear stresses may have a considerable build up of excess pore pressure. In such cases the effective stresses may approach zero and liquifaction can occur.

The relative density of granular soils is determined by the following formulae:

In terms of the void ratios, e

$$D_d = \frac{e_{max} - e}{e_{max} - e_{min}} (100)$$

where D_d = relative density, expressed as a percentage

e_{max} = void ratio in loosest state

e_{min} = void ratio in densest state

e = void ratio at state of compaction at which the relative density is to be expressed.

In terms of dry density

$$D_d = \frac{\frac{1}{\gamma_{min}} - \frac{1}{\gamma}}{\frac{1}{\gamma_{min}} - \frac{1}{\gamma_{max}}} (100)$$

where γ_{min} = minimum dry unit weight of soil

γ_{max} = maximum dry unit weight of soil

γ = dry unit weight of soil at which the the relative density is to be determined

Methods of determining the maximum and minimum void ratios and dry densities are discussed in Section 4.

4. Soil Consistency

a. General

The physical properties of fine grained soils differ greatly at different water contents. A given clay may act almost like a liquid, it may behave as a plastic solid, as a semi-solid or as a solid with limited elastic properties.

Consistency is defined as the degree of resistance of a fine-grained soil to flow or to deformation in general.

If the water content of a clay slurry is reduced by slow desiccation, the clay passes from a liquid state through a plastic state and finally into a solid state. The water contents at which different clays pass from one of these states into another are very different. Therefore, the water contents at these transitions can be used for identification and comparison of different clays. The transition from one state to another does not occur abruptly as soon as some critical water content is reached. It occurs gradually over a fairly large range of water contents. Consequently, all attempts to establish criteria for these boundaries have resulted in setting somewhat arbitrary limits.

b. Atterberg Limits

The consistency limits set up by Atterberg are the most widely applied in soil mechanics. The Atterberg limit tests are discussed in Section 4.

Consider a clay soil that is mixed with sufficient water to exhibit the properties of a true liquid. In this state the degree of saturation is much greater than 100%.

If this soil is allowed to dry out without disturbance, a point will be reached where the degree of saturation with respect to the volume of the soil-air-water mass is 100%.

When this point is reached, the mass begins to behave as a viscous liquid. The moisture content at this change of state is defined as the flocculation limit. This limit has little practical significance. In the viscous liquid state, the soil mass will flow slowly under its own weight. At this point with further drying out, capillary forces are set up as the water starts to retreat into the pore spaces of the soil-air-water mass. These forces exert compressive forces on the soil grains and forces them closer together causing a decrease in volume of the mass. This is illustrated by Figure 3-4. The rate of drying out and the rate of volume change are such that the degree of saturation of the mass remains at 100%.

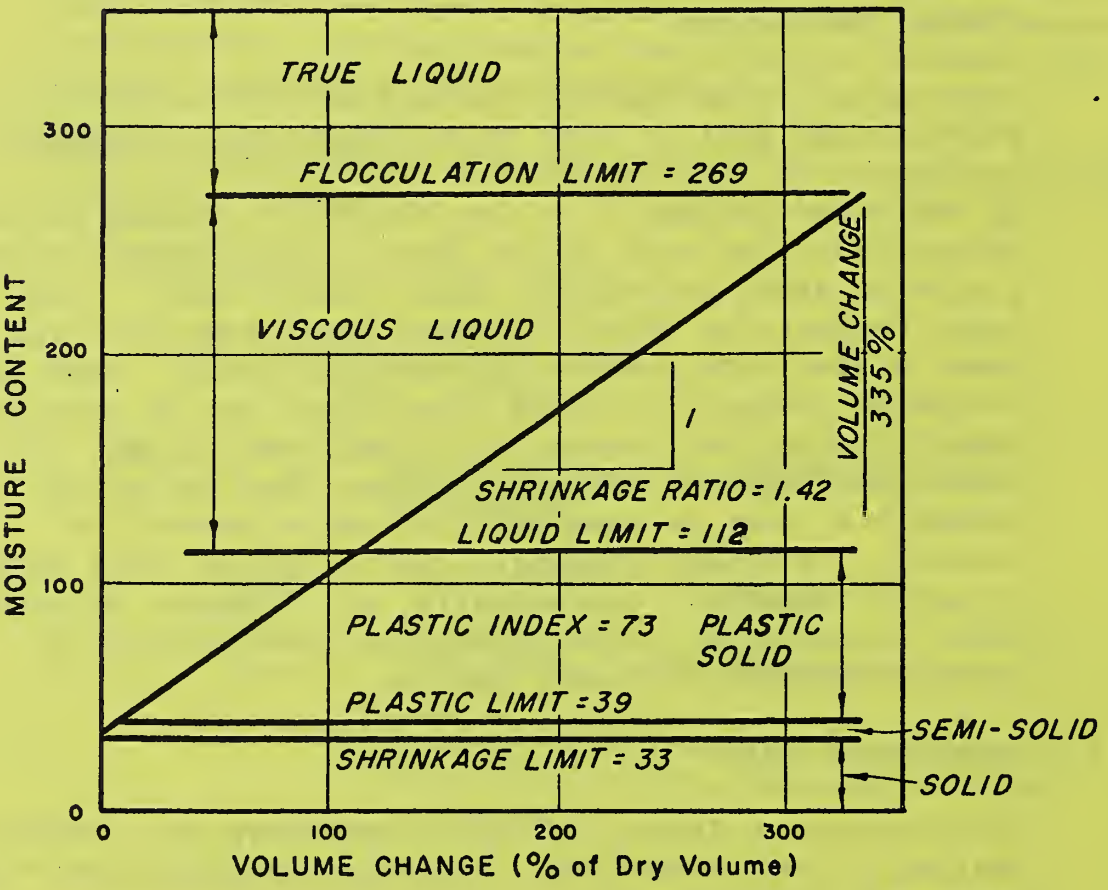


FIG. 3-4 - SHRINKAGE DIAGRAM FOR A COHESIVE SOIL

REFERENCE: ENGINEERING PROPERTIES OF SOIL, HOGENTOGLER, MC GRAW - HILL, 1937

As this drying out and shrinkage continues, a point is reached where the properties of the soil-air-water mass change from those of a viscous liquid to those of a plastic solid. The moisture content at this point is defined as the liquid limit. With additional drying out, the capillary forces become larger and force the soil particles closer together. With the change in volume, the degree of saturation is maintained at 100% and the percent moisture versus percent volume change plot traces the line shown on Fig. 3-4.

As the drying out and shrinkage continues, a point is reached where the properties of the soil-air-water mass change to those of a semi-solid. The moisture content at this point is defined as the plastic limit. The range of moisture contents between the liquid limit and the plastic limit is called the plasticity index. This index represents the range of moisture through which the soil behaves as a plastic solid. That is, the soil can be permanently deformed under load without rupture.

As additional drying occurs, a point is reached where the capillary forces cannot force the soil particles closer together. At this point no further decrease in volume occurs with additional drying out. The moisture content at this point is defined as the shrinkage limit. This is the lowest point where the degree of saturation is 100% in the drying out process.

Below the shrinkage limit, a cohesive soil has the properties of a solid and develops its maximum bearing capacity and strength. A soil containing relatively small quantities of inactive colloidal material may be weak and friable in this state. A soil containing chemically active colloids may develop very high strength and hardness.

The ratio of the volume change of soil to its loss in moisture is called the shrinkage ratio. This ratio is equal to the bulk specific gravity of the soil.

Nonplastic soils undergo the same general volume change pattern with drying out but they do not go through the plastic state or become a solid. The total volume change of nonplastic soils is very low compared to highly plastic soils.

c. Consistency of Undisturbed Soils

Fine grained soils exist in a wide range of states of consistency under natural conditions. The consistency of undisturbed soils is dependent on the mode of deposition and on other factors that have occurred in the geologic history of the deposit such as dessication, glacial loading, weathering and unloading of thick deposits of soil by erosion, etc.

The following nomenclature describing the consistency of undisturbed soils is widely used. This nomenclature is based on the undrained, unconfined compressive strength (q_u).

Consistency	q_u (tons/sq.ft.)
Very soft	less than 0.25
Soft	0.25 - 0.5
Medium	0.5 - 1.0
Stiff	1.0 - 2.0
Very stiff	2.0 - 4.0
Hard	more than 4.0

The consistency of undisturbed soils based on simple qualitative estimates is described in item 7c of Section 2.

d. Sensitivity of cohesive soils

When remolded at the natural moisture content, most cohesive soils exhibit a considerable loss in strength and stiffness.

Present theory is that this loss in strength is probably due to the reorientation of soil particles into less favorable positions in the soil mass and to a reduction in bonding forces between particles.

Some clays when allowed to set after remolding without a change in moisture condition will regain some of the strength lost, this process is termed thixotropy.

The sensitivity of cohesive soils is represented by the ratio of the undrained, unconfined compressive strength before remolding to the undrained, unconfined compressive strength after remolding.

$$S_t = \frac{q_u \text{ (undisturbed soil)}}{q_u \text{ (remolded soil)}}$$

where S_t is the sensitivity.

The following table is used to evaluate the sensitivity of soils.

Classification	S_t (sensitivity)
Insensitive	less than 2
Moderately sensitive	2-4
Sensitive	4-8
Very sensitive	8-16
Slightly quick	16-32
Medium quick	32-64
Quick	greater than 64

The variation in natural moisture content of cohesive soils is an effective indication of the homogeneity of the deposit. Cohesive soils with a natural moisture content near the plastic limit will generally be relatively stiff while those near the liquid limit may be soft or if stiff, very sensitive to remolding.

The liquidity index as proposed by Terzaghi is used to relate the natural moisture content to the Atterberg Limits.

$$L_I = \frac{w - PL}{LL - PL}$$

where L_I = liquidity index
w = natural moisture content
PL = plastic limit
LL = liquid limit

It is generally possible to relate the liquidity index of soils to soil properties such as shear strength. This permits qualitative evaluation of the more complex properties after the natural moisture content and Atterberg Limits have been determined.

C. PHYSIO-CHEMICAL RELATIONSHIPS

To understand the behavior characteristics of fine grained soils, it is necessary to consider the chemical effects and properties as well as the physical properties of the soil particles.

The first step in understanding clay behavior is to become acquainted with the clay minerals.

Since the first x-ray diffraction studies were made on clays in 1925 much research into the factors controlling the behavior of clays has been done.

This research has established that argillaceous (clay) materials are composed essentially of extremely small particles (of the order of one or two microns or less), of one or more members of a small group of minerals that have come to be known as the clay minerals.

Chemically, the clay minerals are hydrous silicates of aluminum or iron and magnesium. Some of them also contain alkalies or alkaline earths as essential components.

The clay minerals are dominantly crystalline, i.e., the atoms composing them are arranged in definite geometric patterns.

Most of the clay minerals have sheet or layered structures, some of the clay minerals have elongate tubular or fibrous structures. Thus agrillaceous materials can be considered as being essentially made up of extremely small particles, each one of which is either a book of sheet like units or a bundle of tubes or fibers.

Individual soils or clays may contain more than one kind of book like units or mixtures of books and bundles of tubes or fibers.

1. Structure and Composition of Clay Minerals

   There are two fundamental building blocks for clay mineral structures.

   a. The silica tetrahedral unit (Fig. 3-5a)

      In this unit four oxygens or hydroxyls having the configuration of a tetrahedron, enclose a silicon atom.

      The tetrahedra are combined in a sheet structure (Fig. 3-5b) so that the oxygens of the bases of all the tetrahedra are in a common plane and each oxygen belongs to two tetrahedra.

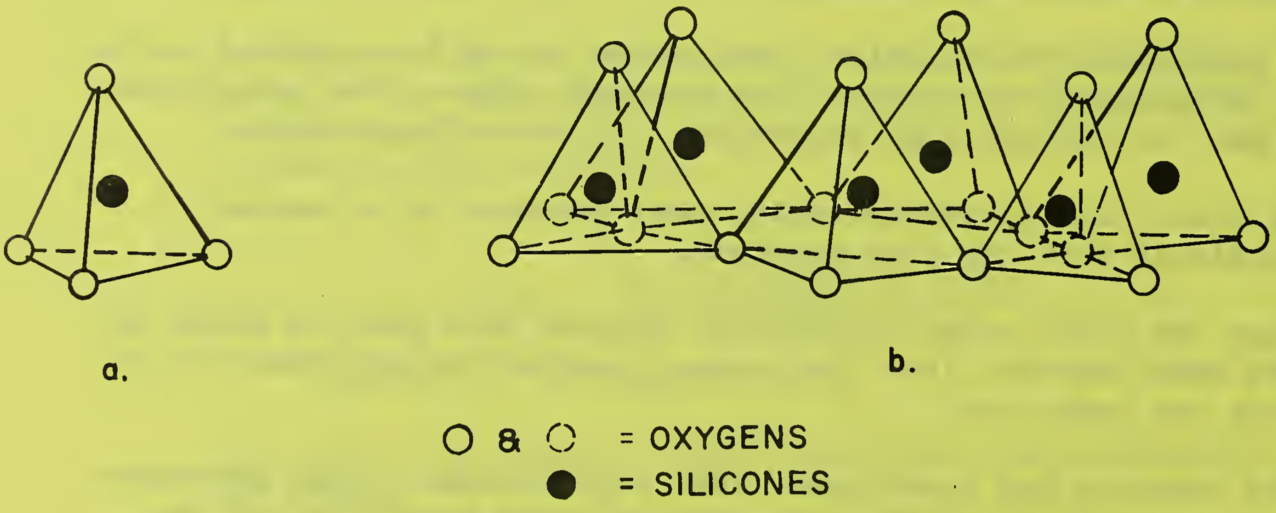


FIG. 3-5 - SILICA TETRAHEDRAL UNIT

The silica tetrahedral sheet alone may be viewed as a layer of silicon atoms between a layer of oxygens and a layer of hydroxyls (tips of the tetrahedra).

b. The octahedral unit (Fig. 3-6)

In this unit an aluminum, magnesium, or iron unit is enclosed in six hydroxyls having the configuration of an octahedron.

The octahedral units are joined together into a sheet structure (Fig. 3-6b) which may be viewed as two layers of packed hydroxyls with the aluminum, iron or magnesium cation between the sheets in octahedral coordination.

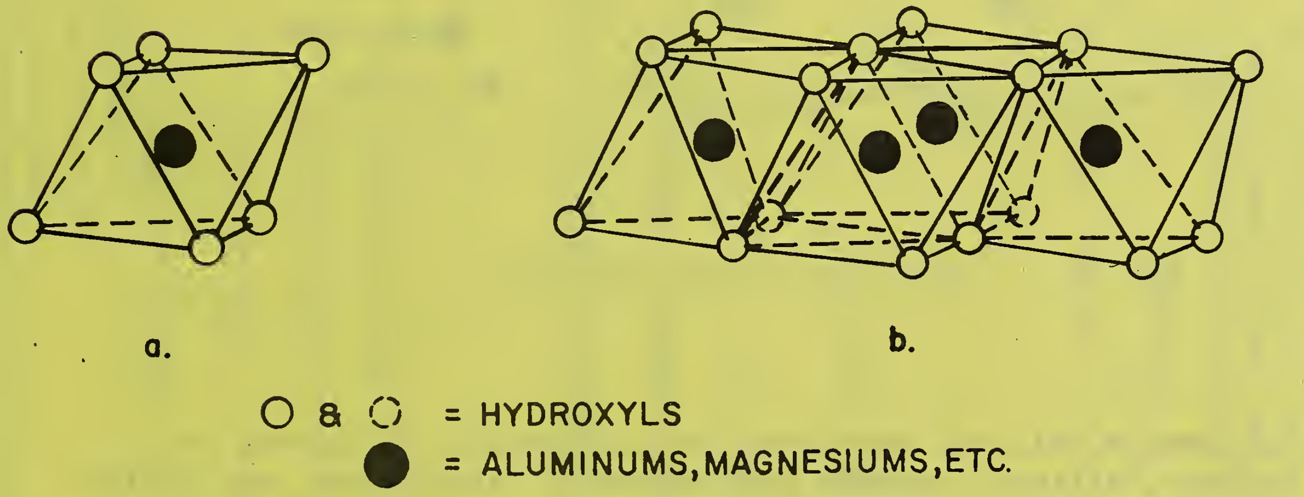


FIG. 3-6 - OCTAHEDRAL UNIT

2. <u>Shapes of Clay Minerals</u>

The shapes of some of the common clay minerals are:

Kaolinite	sheets (often regular hexagonal sheets
Halloysite	tubes of curled sheets
Montmorillonite . .	flaky sheets
Nontronite	laths
Illite	irregular flakey sheets
Attapulgite	narrow fibers
Chlorite	sheets
Vermiculite	sheets

3. Kaolinite

The kaolinite structural unit is an alumina octahedral layer with a parallel superimposed silica tetrahedral layer intergrown in such a way that the tips of the silica sheet and one of the layers of the octahedral unit form a common sheet.

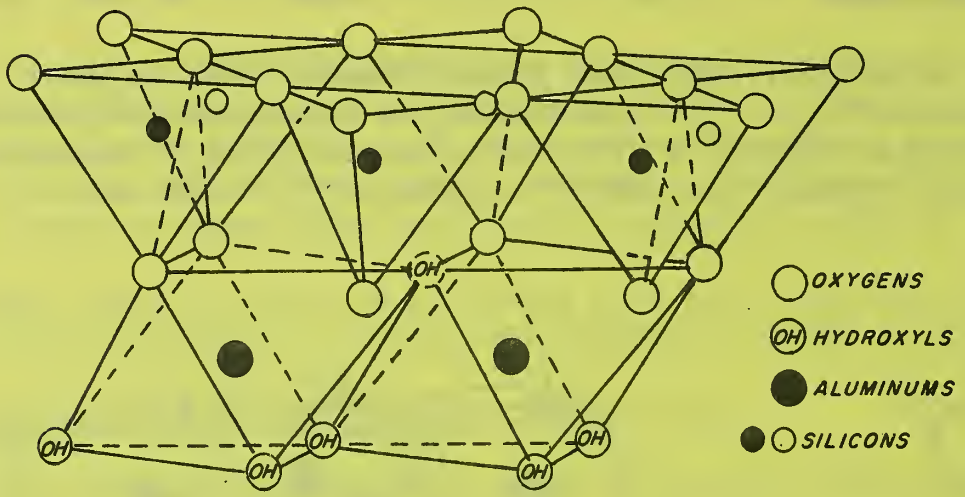


FIG. 3-7 - BASIC KAOLINITE STRUCTURE

The kaolin unit may be viewed as a succession of layers of oxygens, silions, oxygens and hydroxyls, aluminiums, and hydroxyls. This unit is about 7A thick, and extends indefinitely in the other two directions, i.e. the flat dimension of the sheet. The kaolinite mineral is a stacking of such 7A thick sheets, the structure is like that of a book with each leaf 7A thick. Successive 7A thick layers are held together with hydrogen bonds. The mineral cleaves fairly easily along the plane surface of the 7A units.

Table 3-1 shows the average dimensions of the kaolinite mineral.

Normally there is no substitution of one cation for another within the kaolinite structure so that the mineral has the fixed formula

$$(OH)_8 Al_4 Si_4 O_{10}$$

In some clays, the kaolinite particles are composed of 7A thick units regularly stacked one above the others. In other clays, the stacking is somewhat random.

TABLE 3-1 DIMENSIONS OF TYPICAL CLAY PLATELETS

	Characteristic Ratios of Dimensions	Approximate Range of Actual Dimensions in Angstroms		Specific Surface (Sq. Meters /gr.)
		Length and Breadth	Thickness	
Montmorillonite	100 x 100 x 1	1,000 to 5,000	10 to 50	800
Illite	20 x 20 x 1	1,000 to 5,000	50 to 500	80
Kaolinite	10 x 10 x 1	1,000 to 20,000 (2u)	100 to 1000	10

TABLE 3-2 CLASSIFICATION OF CLAY MINERALS

Clay Mineral Group	Chemical Formula	Mineral Sub-Group
Kaolinite	$(OH)_8\ Al_4\ Si_4\ O_{10}$	Kaolinite Dickite Nacrite
Montmorillonite	$(OH)_4\ Al_4\ Si_8\ O_{20} \cdot nH_2O$	Swelling Montmorillonite Non-Swelling Montmorillonite Talc Nontronite
Illite (Hydrous Mica)	$(OH)_4\ Ky(Al_4 \cdot Fe_4 \cdot Mg_4 \cdot Mg_6)\ (Si_{8-y} \cdot Al_y)\ O_{20}$	Illite
Halloysite*	$(OH)_8\ Al_4\ Si_4\ O_{10}$ $(OH)_8\ Al_4\ Si_4\ O_{10} \cdot 4H_2O$	Halloysite

---

* Halloysite occurs in two forms having the compositions shown. The first of these is more stable, and the second slowly changes to the first by dehydration. Particles of halloysite clay minerals are fibrous or lath-shaped, instead of having the more usual flake shape.

4. Halloysite

Structurally, the halloysite mineral is similar to kaolinite as it is composed of sheets about 7A thick, each sheet being made up of one silica tetrahedral layer and one alumina octahedral layer. Halloysite differs from kaolinite in that successive 7A units are more randomly stacked one above the other and a single molecular layer of water may enter between the 7A units. With a molecular layer of water between each sheet, the mineral has the composition,

$$(OH)_8 \ Al_4 \ Si_4 \ O_{10} \ 4H_2O$$

Without the water layer, the composition is the same as kaolinite.

A further structural factor of halloysite revealed by electron micrographs is that the mineral occurs in elongated units that appear to be tubes, as if the sheet layers had rolled up.

5. Montmorillonite

This mineral is made of sheet-like units, each unit being composed of two silica tetrahedral sheets and one octahedral sheet. The octahedral sheet is between the silica sheets with the tips of the tetrahedra pointed toward the octahedral sheet.

The tips of each tetrahedral sheet and a hydroxyl layer of the octahedral sheet are intergrown to form a single layer.

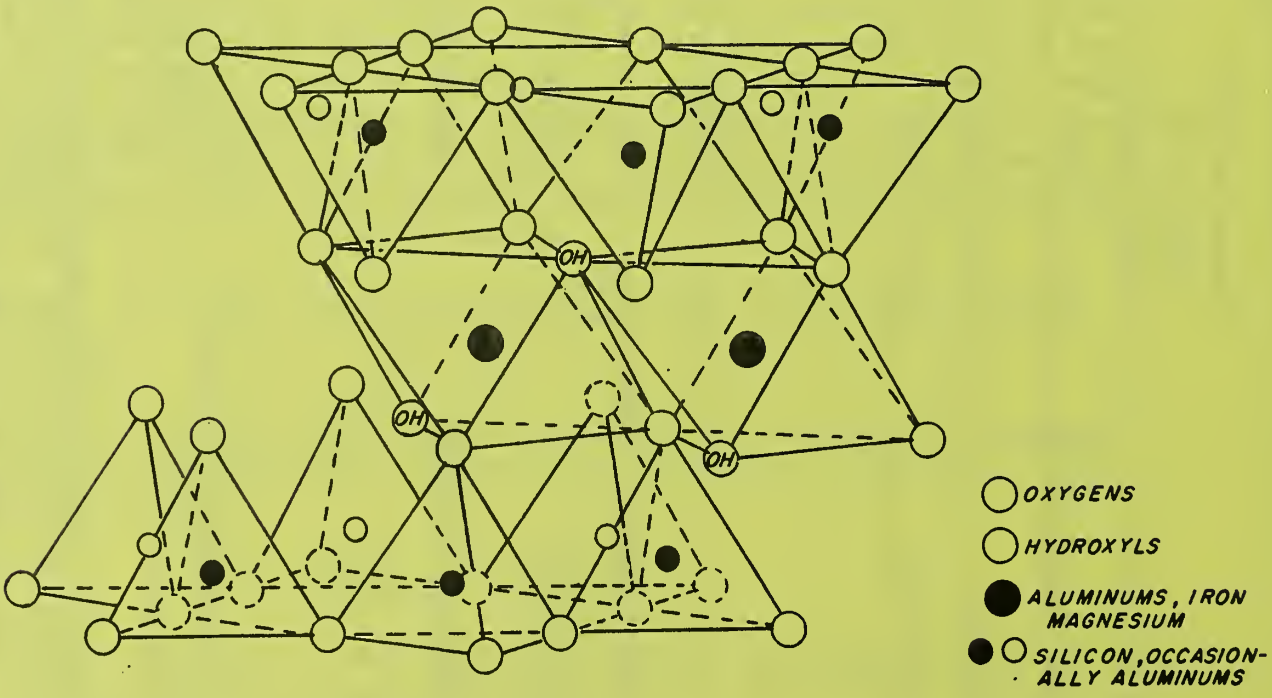


FIG. 3-8 - BASIC MONTMORILLONITE STRUCTURE

The montmorillonite structure can be viewed as a sheet structure with each sheet composed of successive layers of oxygens, silicons, oxygens and hydroxyls, aluminum (or irons or magnesium), oxygens and hydroxyls, silicons and oxygens.

The thickness of each sheet is about 9.5A and the dimensions in the other two directions are indefinite.

The octahedral units may be aluminum, iron or magnesium, or a combination of these elements.

If aluminum is the sole occupant of the octahedral positions, only two-thirds of the possible positions are filled.

If magnesium is the sole occupant about all possible positions are filled. Also, a small amount (normally less than 15%) of aluminum may replace silicon in the tetrahedral layers.

Therefore, it is apparent that there is considerable possible element replacement within the structure of the montmorillonites. Of great importance is the fact that there are always replacements within the structures, and they always produce a net positive charge deficiency.

This deficiency is balanced by absorbed cations which are held on the outside of the sheets. These cations in general are readily exchangeable. While it is difficult to write a general formula, common compositions of montmorillonite are:

$$(OH)_4\ Si_8\ (Al_{3.34}\ Mg_{66})\ O_{20}$$

$$(OH)_4\ (Si_{7.34}\ Al_{66})\ Fe_4\ O_{20}$$

$$(OH)_4\ (Si_{7.34}\ Al_{66})\ Mg_6\ O_{20}$$

Special mineral names have been applied to varieties of montmorillonite with distinctive chemical compositions. Thus

> Saponite is used when magnesium is about the sole octahedral cation.
>
> Nontronite is used when iron fills these positions in the octahedron.

In the montmorillonite mineral the 9.5A thick sheets are stacked one above the other like leaves of a book. There is very little bonding between successive sheets and water may enter between the sheets causing the mineral to swell. Also about 80% of the absorbed balancing cations are between these sheets. The water layer that can exist between sheets may be about any thickness depending somewhat on the nature of the absorbed cations and the water available.

If sufficient water is available, the mineral can split into individual unit layers 9.5A thick.

The average range of sizes of the montmorillonites are shown in Table 3-1.

6. Illite

The structure of illite is similar to that of montmorillonite except that there is always a substantial (20%∓) replacement of silicons by aluminums in the tetrahedral layers and potassiums are between the layers serving to balance the charges resulting from the replacement of the silicon with aluminum and to tie the sheet units together. The structure does not swell with the introduction of water between successive sheets.

The population of the octahedral positions can vary from one illite to another as can the amount of aluminum replacing silicon and the balancing potassiums.

Illite is similar to the micas in structure and composition but differ from them mainly in containing less potassium, slightly higher silicon to alumina molecular ratios and in being less well ordered.

A general formula is

$$(OH)_4\ Ky(Si_{8-y}\ Al_y)\ (Al_4 \cdot Mg_6 \cdot Mg_4 \cdot Fe_4)\ O_{20}$$

D. SOIL STRUCTURE

The mechanical properties of the so-called "cohesive soils" have been dealt with as a subject of scientific research for more than 30 years.

In the early 1920's the general opinion concerning cohesion was that it was due to some undefined action of a hypothetical amorphous clay substance.

The first major contribution toward our present knowledge of clay behavior came in 1923 when Assar Hadding proved by x-ray analyses that clays had mainly a crystalline nature.

In 1925, Karl Terzaohi published a remarkably modern discussion of the structure and bonds in cohesive soils. He drew attention to the adhesion between particles and held that the cohesion in clays was due to this interparticle adhesion. He suggested that clay particles stick together at the points of contact with forces sufficiently strong for the building of a honeycomb structure. This permits comparatively large amounts of water to be enclosed within voids built up of aggregates of minerals glued to each other by the adhesion forces. Thus each cell in the honeycomb structure was supposed to be made up of numerous single mineral grains.

In 1926, V. M. Goldschmidt published his views and the results of his research on clays. He performed a series of experiments with mixtures of clay minerals and various liquids such as water, benzene, carbon tetrachloride, liquid sulphur dioxide and ammonia. In these experiments, he found that the clay minerals exhibited plasticity only when mixed with a liquid of polar nature (i.e. opposite electrical charges on each end of the molecule). He concluded that the clay properties were dependent on both the crystal chemistry of the mineral phase and the atomic structure of the liquid phase. His theory was that the clay properties were due to crystalline minerals surrounded by a film of absorbed water molecules and that the water molecules stuck to each other and to the minerals because of their dipole moments. His work led him to the opinion that the flaky minerals in highly sensitive clays are arranged in unstable cardhouse structures (Figure 3-9). Surplus water was encased in the space between a few mineral flakes leaning on each other and the difference in clays of high and low sensitivity was due to a denser arrangement of the minerals in the clays of low sensitivity.

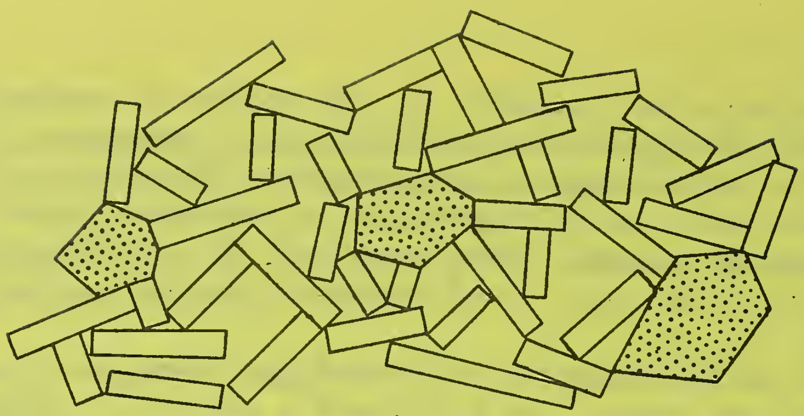
In 1932, Arthur Casagrande presented a theory of honeycomb structure in sensitive soils very similar to Terzaghi's theory.

In 1953, T. Wm. Lambe published a paper indicating that for undisturbed marine clays an open structure similar to Goldschmidt's cardhouse exists, whereas for fresh water clays the structure was somewhat denser and for remolded clays, a high degree of parallelism with a short range order between flakes would exist. (Figure 3-9). The work of Rosenquist and Bjerrum in 1955 and 1956 support the views of Lambe.

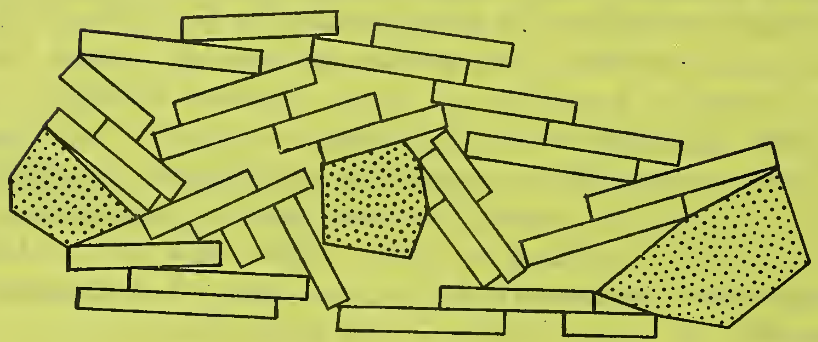
In 1957, T. K. Tan presented the view that a clay mineral network dominated by contacts between a corner of one mineral and the plane of another, would best describe the structure of clays. This is a refinement of the Goldschmidt-Lambe hypothesis.

The Work of Rosenquist et al of the Norwegian Geotechnical Institute in 1958 and 1959 with the electron microscope has conclusively proved the validity of the cardhouse concept of clay structure.

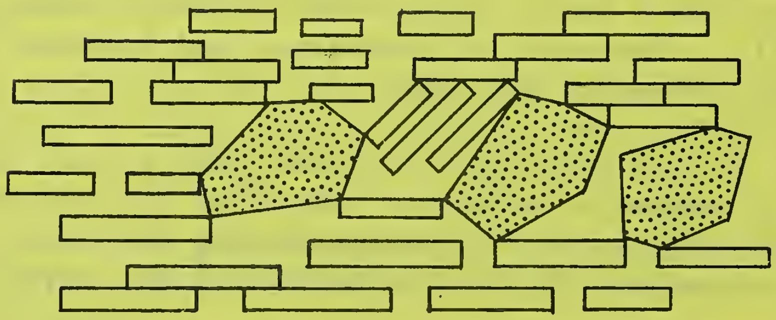
At present there are five stable units of soil structure. These are shown on Figure 3-10.



UNDISTURBED SALT WATER DEPOSITS



UNDISTURBED FRESH WATER DEPOSITS



REMOLDED

CARDHOUSE STRUCTURE OF CLAY

REFERENCE: GOLDSCHMIDT & LAMBE

FIG. 3-9

Figure 3-10a illustrates the only stable structural unit for clean coarse grained soils. In this unit the strength that results in the stability of these types of soils is mainly because of friction. (The so-called "macro-dilatency", which is caused by the lifting of one mineral grain on the one side of the shear plane when the shear movement takes place.) This strength is proportional to the effective normal stress.

Figure 3-10b illustrates the disperse colloidal structure. This dispersed or oriented system is the structure associated with the condition of the minimum free energy in a mass of colloidal particles. A soil in this structural condition is acted on by forces of attraction or by forces of repulsion and by external forces.

This type of soil structure is about the least stable of all the colloidal structures. Distinctive features of dispersed soil structure are the approximately parallel position of adjacent particles and the almost complete absence of contact between particles.

In these clays it is the liquid rather than the solid phase that is continuous. The strength and behavior of such clays depend largely on characteristics of the colloid system.

Figure 3-10c and 3-10d show the flocculated types of "card-house" structure. These types of soil structure are characterized by edge-to-surface contact of the plate like colloid particles. The two factors contributing to this arrangement are:

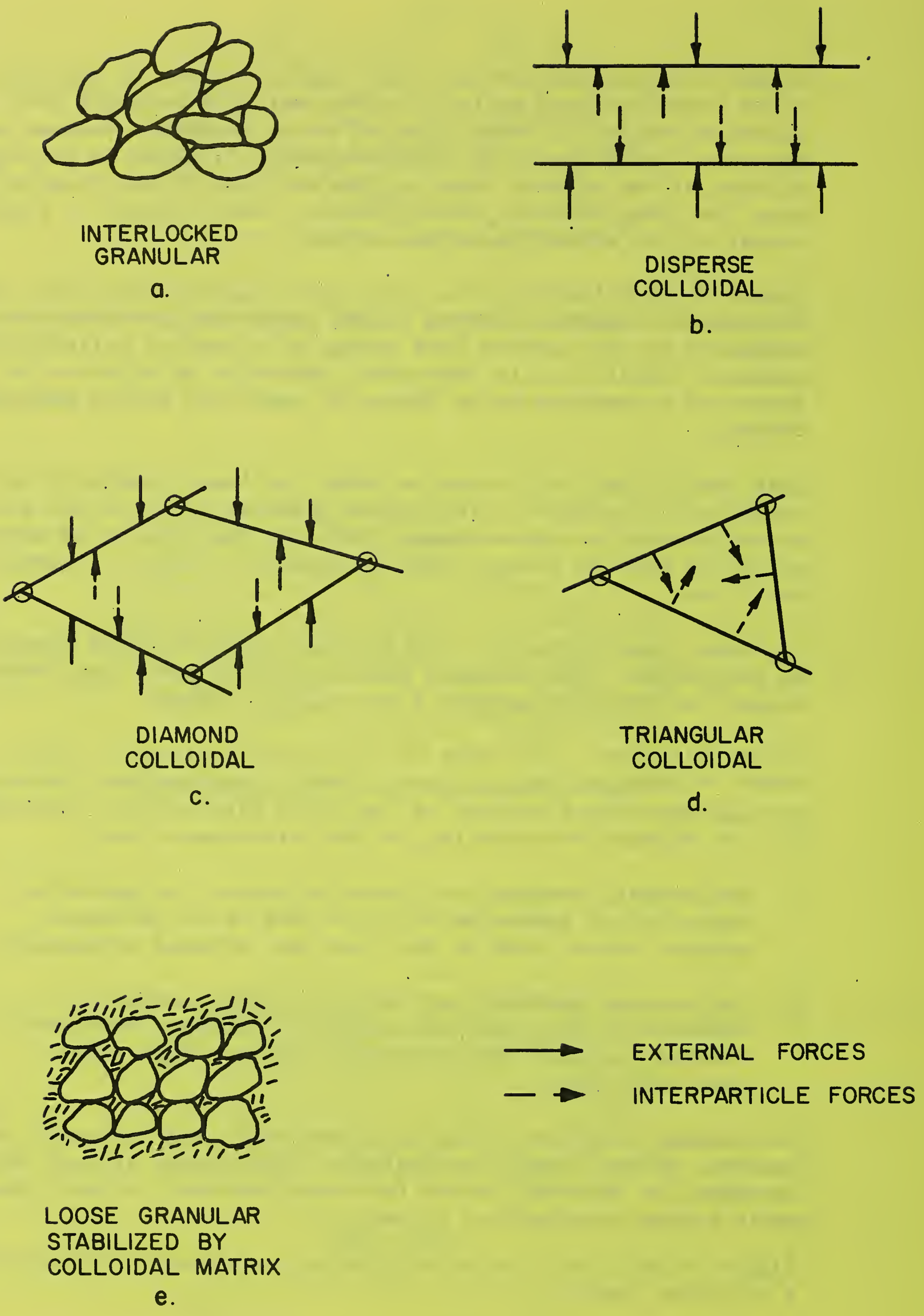
1. The overall tendency of a mass of plate-like particles deposited at random would be to take up an irregular pattern rather than to fall into an oriented arrangement.

2. The natural tendency for edge-to-surface contact is enhanced by electrostatic attraction between negatively charged surfaces and positively charged edges under some conditions.

The diamond structure tends to expand under the influence of internal forces (expansive system). This system is much more dependent on external forces for stability than is the triangular system (contractive system).

Figure 3-10e shows the loose granular structure stabilized by a colloidal matrix.

In this system the clay matrix acts as a void filler and thus resists the tendency of the larger grains to achieve an interlocking particle arrangement.



Reference:
Trollope and Chan

FIVE STABLE UNITS OF SOIL STRUCTURE

FIGURE 3-10