NATIONAL HIGHWAY INSTITUTE SOILS & FOUNDATIONS: CLASSIFICATION OF SOILS/ROCKS

Main Category:	Civil Engineering
Sub Category:	Geotechnical Engineering
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Course Content:	92 pgs
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OFFICIAL COURSE/EXAM

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GEO-116 EXAM PREVIEW

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Exam Preview:

1. Rock is defined as a conglomeration consisting of a wide range of relatively smaller particles derived from a parent rock through mechanical weathering processes that include air and/or water abrasion, freeze-thaw cycles, temperature changes, plant and animal activity and by chemical weathering processes that include oxidation and carbonation.
   a. True
   b. False

2. Intermediate geomaterials (IGMs) are transition materials _____ soils and rocks.
   a. on top of
   b. below
   c. between
   d. adjacent to

3. Classification is the field study-based process of grouping soils with similar engineering characteristics into categories.
   a. True
   b. False

4. The suggested guideline for estimating the in-place apparent density of course-grained soils with an Apparent Density of "Medium Dense" would be a relative density of?
   a. 0-20%
   b. 20-40%
   c. 40-70%
   d. 70-85%

5. The evaluation of the consistency of fine-grained soils with a consistency of "Very Soft" for manual manipulation would result in that the specimen can be pinched in two between the thumb and forefinger; remolded by light finger pressure.
   a. True
   b. False
6. Coarse-grained soils consist of a matrix of either gravel or sand in which more than 50 percent by weight of the soil is retained on the No. ___ sieve.
   a. 50
   b. 75
   c. 100
   d. 200
7. The particle size definition for gravels and sands (after ASTM D 2488) for Coarse Sand would translate to a Grain Size of:
   a. #4 to #10 sieve
   b. #10 to #40 sieve
   c. #40 to #200 sieve
   d. #200 to #300 sieve
8. Fine-grained soils are those having 75 percent or more by weight pass the No. 200 sieve. The so-called fines are either inorganic or organic silts and/or clays.
   a. True
   b. False
9. Colloidal and amorphous organic materials finer than the No. 200 sieve (0.075 mm) are identified and classified in accordance with their drop in plasticity upon oven drying (ASTM D 2487).
   a. True
   b. False
10. The mark of successfully accomplishing a subsurface exploration is the ability to draw a subsurface profile of the project site complete with soil types, rock interfaces, and the relevant design properties. The subsurface profile is a visual display of subsurface conditions as interpreted from all of the methods of explorations and testing described previously. Uncertainties in the development of a subsurface exploration usually indicate the need for additional explorations or testing.
    a. True
    b. False

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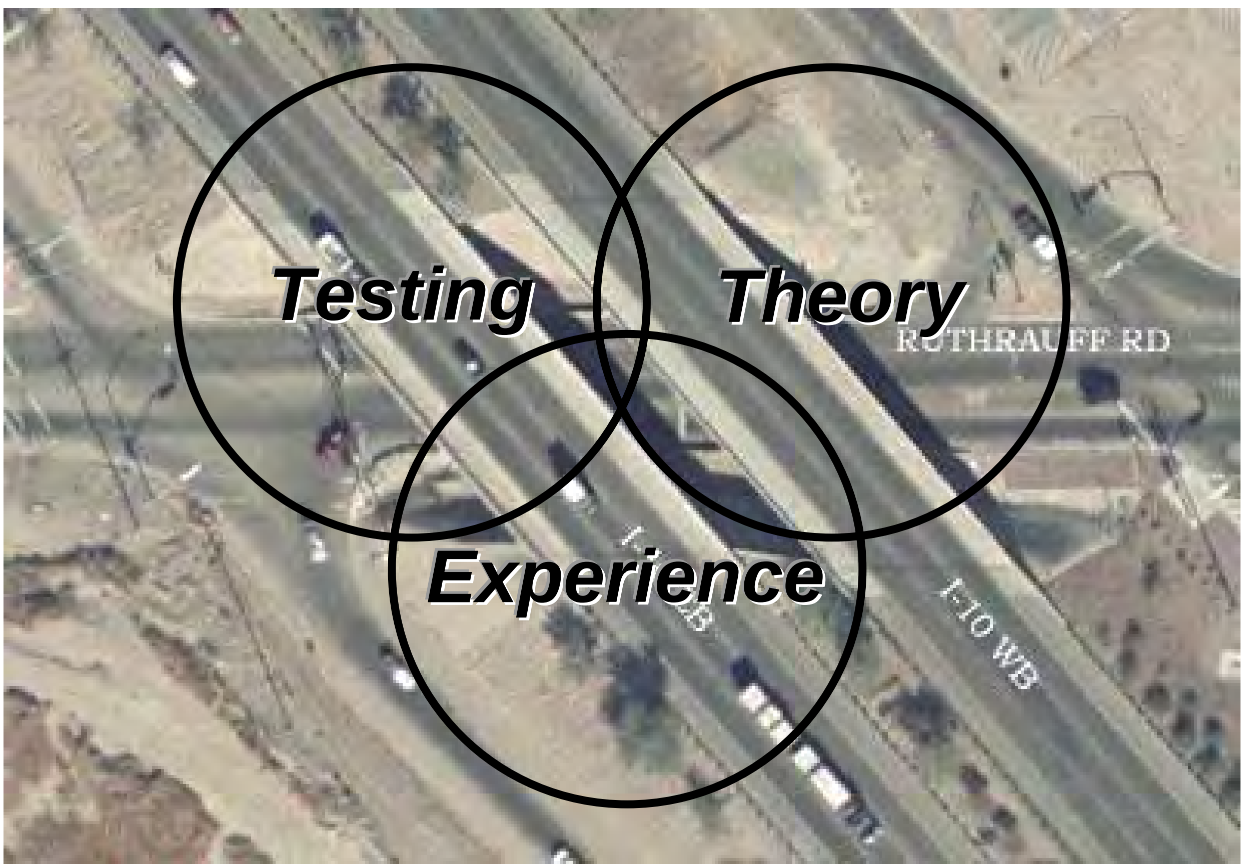
SOILS AND FOUNDATIONS

Reference Manual – Volume I - Ch.4 ENGINEERING DESCRIPTION, CLASSIFICATION AND CHARACTERISTICS OF SOILS AND ROCKS



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This manual is an update of the 3rd Edition prepared by Parsons Brinckerhoff Quade & Douglas, Inc, in 2000. Author: Richard Cheney, PE. The authors of the 1st and 2nd editions prepared by the FHWA in 1982 and 1993, respectively, were Richard Cheney, PE and Ronald Chassie, PE.

16. Abstract

The Reference Manual for Soils and Foundations course is intended for design and construction professionals involved with the selection, design and construction of geotechnical features for surface transportation facilities. The manual is geared towards practitioners who routinely deal with soils and foundations issues but who may have little theoretical background in soil mechanics or foundation engineering. The manual's content follows a project-oriented approach where the geotechnical aspects of a project are traced from preparation of the boring request through design computation of settlement, allowable footing pressure, etc., to the construction of approach embankments and foundations. Appendix A includes an example bridge project where such an approach is demonstrated. Recommendations are presented on how to layout borings efficiently, how to minimize approach embankment settlement, how to design the most cost-effective pier and abutment foundations, and how to transmit design information properly through plans, specifications, and/or contact with the project engineer so that the project can be constructed efficiently.

The objective of this manual is to present recommended methods for the safe, cost-effective design and construction of geotechnical features. Coordination between geotechnical specialists and project team members at all phases of a project is stressed. Readers are encouraged to develop an appreciation of geotechnical activities in all project phases that influence or are influenced by their work.

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Subsurface exploration, testing, slope stability, embankments, cut slopes, shallow foundations, driven piles, drilled shafts, earth retaining structures, construction.	No restrictions.

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PREFACE

This update to the Reference Manual for the Soils and Foundations course was developed to incorporate the guidance available from the FHWA in various recent manuals and Geotechnical Engineering Circulars (GECs). The update has evolved from its first two versions prepared by Richard Cheney and Ronald Chassie in 1982 and 1993, and the third version prepared by Richard Cheney in 2000.

The updated edition of the FHWA Soils and Foundations manual contains an enormous amount of information ranging from methods for theoretically based analyses to “rules of thumb” solutions for a wide range of geotechnical and foundation design and construction issues. It is likely that this manual will be used nationwide for years to come by civil engineering generalists, geotechnical and foundation specialists, and others involved in transportation facilities. That being the case, the authors wish to caution against indiscriminate use of the manual’s guidance and recommendations. The manual should be considered to represent the minimum standard of practice. The user must realize that there is no possible way to cover all the intricate aspects of any given project. Even though the material presented is theoretically correct and represents the current state-of-the-practice, engineering judgment based on local conditions and knowledge must be applied. This is true of most engineering disciplines, but it is especially true in the area of soils and foundation engineering and construction. For example, the theoretical and empirical concepts in the manual relating to the analysis and design of deep foundations apply to piles installed in the glacial tills of the northeast as well as to drilled shafts installed in the cemented soils of the southwest. The most important thing in both applications is that the values for the parameters to be used in the analysis and design be selected by a geotechnical specialist who is intimately familiar with the type of soil in that region and intimately knowledgeable about the regional construction procedures that are required for the proper installation of such foundations in local soils.

General conventions used in the manual

This manual addresses topics ranging from fundamental concepts in soil mechanics to the practical design of various geotechnical features ranging from earthworks (e.g., slopes) to foundations (e.g., spread footings, driven piles, drilled shafts and earth retaining structures). In the literature each of these topics has developed its own identity in terms of the terminology and symbols. Since most of the information presented in this manual appears in other FHWA publications, textbooks and publications, the authors faced a dilemma on the regarding terminology and symbols as well as other issues. Following is a brief discussion on such issues.

- **Pressure versus Stress**

  The terms "pressure" and "stress" both have units of force per unit area (e.g., pounds per square foot). In soil mechanics "pressure" generally refers to an applied load distributed over an area or to the pressure due to the self-weight of the soil mass. "Stress," on the other hand, generally refers to the condition induced at a point within the soil mass by the application of an external load or pressure. For example, "overburden pressure," which is due to the self weight of the soil, induces "geostatic stresses" within the soil mass. Induced stresses cause strains which ultimately result in measurable deformations that may affect the behavior of the structural element that is applying the load or pressure. For example, in the case of a shallow foundation, depending upon the magnitude and direction of the applied loading and the geometry of the footing, the pressure distribution at the base of the footing can be uniform, linearly varying, or non-linearly varying. In order to avoid confusion, the terms "pressure" and "stress" will be used interchangeably in this manual. In cases where the distinction is important, clarification will be provided by use of the terms "applied" or "induced."

- **Symbols**

  Some symbols represent more than one geotechnical parameter. For example, the symbol C_c is commonly used to identify the coefficient of curvature of a grain size distribution curve as well as the compression index derived from consolidation test results. Alternative symbols may be chosen, but then there is a risk of confusion and possible mistakes. To avoid the potential for confusion or mistakes, the Table of Contents contains a list of symbols for each chapter.

- **Units**

  English units are the primary units in this manual. SI units are included in parenthesis in the text, except for equations whose constants have values based on a specific set of units, English or SI. In a few cases, where measurements are conventionally reported in SI units (e.g., aperture sizes in rock mapping), only SI units are reported. English units are used in example problems. Except where the units are related to equipment sizes (e.g., drill rods), all unit conversions are "soft," i.e., approximate. Thus, 10 ft is converted to 3 m rather than 3.05 m. The soft conversion for length in feet is rounded to the nearest 0.5 m. Thus, 15 ft is converted to 4.5 m not 4.57 m.

- **Theoretical Details**

  Since the primary purpose of this manual is to provide a concise treatment of the fundamental concepts in soil mechanics and an introduction to the practical design of various geotechnical features related to highway construction, the details of the theory underlying the methods of analysis have been largely omitted in favor of discussions on the application of those theories to geotechnical problems. Some exceptions to this general approach were made. For example, the concepts of lateral earth pressure and bearing capacity rely too heavily on a basic understanding of the Mohr's circle for stress for a detailed presentation of the Mohr's circle theory to be omitted. However, so as not to encumber the text, the basic theory of the Mohr's circle is presented in Appendix B for the reader's convenience and as an aid for the deeper understanding of the concepts of earth pressure and bearing capacity.

- **Standard Penetration Test (SPT) N-values**

  The SPT is described in Chapter 3 of this manual. The geotechnical engineering literature is replete with correlations based on SPT N-values. Many of the published correlations were developed based on SPT N-values obtained with cathead and drop hammer methods. The SPT N-values used in these correlations do not take in account the effect of equipment features that might influence the actual amount of energy imparted during the SPT. The cathead and drop hammer systems typically deliver energy at an estimated average efficiency of 60%. Today's automatic hammers deliver energy at a significantly higher efficiency (up to 90%). When published correlations based on SPT N-values are presented in this manual, they are noted as N_{60}-values and the measured SPT N-values should be corrected for energy before using the correlations.

  Some researchers developed correction factors for use with their SPT N-value correlations to address the effects of overburden pressure. When published correlations presented in this manual are based upon values corrected for overburden they are noted as $N1_{60}$. Guidelines are provided as to when the N_{60}-values should be corrected for overburden.

- **Allowable Stress Design (ASD) and Load and Resistance Factor Design (LRFD) Methods**

  The design methods to be used in the transportation industry are currently (2006) in a state of transition from ASD to LRFD. The FHWA recognizes this transition and has developed separate comprehensive training courses for this purpose. Regardless of whether the ASD or LRFD is used, it is important to realize that the fundamentals of soil mechanics, such as the determination of the strength and deformation of geomaterials do not change. The only difference between the two methods is the way in which the uncertainties in loads and resistances are accounted for in design. Since this manual is geared towards the fundamental understanding of the behavior of soils and the design of foundations, ASD has been used because at this time most practitioners are familiar with that method of design. However, for those readers who are interested in the nuances of both design methods Appendix C provides a brief discussion on the background and application of the ASD and LRFD methods.

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The authors would like to acknowledge the following events and people that were instrumental in the development of this manual.

- Permission by the FHWA to adapt the August 2000 version of the Soils and Foundations Workshop Manual.

- Provision by the FHWA of the electronic files of the August 2000 manual as well as other FHWA publications.

- The support of Ryan R. Berg of Ryan R. Berg and Associates, Inc. (RRBA) in facilitating the preparation of this manual and coordinating reviews with the key players.

- The support provided by the staff of NCS Consultants, LLC, (NCS) - Wolfgang Fritz, Juan Lopez and Randy Post (listed in alphabetical order of last names). They prepared some graphics, some example problems, reviewed selected data for accuracy with respect to original sources of information, compiled the Table of Contents, performed library searches for reference materials, and checked internal consistency in the numbering of chapter headings, figures, equations and tables.

- Discussions with Jim Scott (URS-Denver) on various topics and his willingness to share reference material are truly appreciated. Dov Leshchinsky of ADAMA Engineering provided copies of the ReSSA and FoSSA programs which were used to generate several figures in the manual as well as presentation slides associated with the course presentation. Robert Bachus of Geosyntec Consultants prepared Appendices D and E. Allen Marr of GeoComp Corporation provided photographs of some laboratory testing equipment. Pat Hannigan of GRL Engineers, Inc. reviewed the driven pile portion of Chapter 9. Shawn Steiner of ConeTec, Inc. and Salvatore Caronna of gINT Software prepared the Cone Penetration Test (CPT) and boring logs, respectively, shown in Chapter 3 and Appendix A. Robert (Bob) Meyers (NMDOT), Ted Buell (HDR-Tucson) and Randy Simpson (URS-Phoenix) provided comments on some sections (particularly Section 8.9).

- Finally, the technical reviews and recommendations provided by Jerry DiMaggio, Silas Nichols, Benjamin Rivers, Richard Cheney (retired) and Justin Henwood of the FHWA, Ryan Berg of RRBA, Robert Bachus of Geosyntec Consultants, Jim Scott of URS, and Barry Christopher of Christopher Consultants, Inc., are gratefully acknowledged.

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A special acknowledgement is due of the efforts of Richard Cheney and Ronald Chassie for their work in the preparation of the previous versions of this manual. It is their work that made this course one of the most popular FHWA courses. Their work in developing this course over the past 25 years is acknowledged.

With respect to this manual, the authors wish to especially acknowledge the in-depth review performed by Jerry DiMaggio and time he spent in direct discussions with the authors and other reviewers. Such discussions led to clarification of some existing guidance in other FHWA manuals as well as the introduction of new guidance in some chapters of this manual.

SI CONVERSION FACTORS				
APPROXIMATE CONVERSIONS FROM SI UNITS				
Symbol	**When You Know**	**Multiply By**	**To Find**	**Symbol**
LENGTH				
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
AREA				
mm^2	square millimeters	0,0015	square inches	in^2
m^2	square meters	10.758	square feet	ft^2
m^2	square meters	1.188	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
VOLUME				
ml	milliliters	0.034	fluid ounces	fl oz
l	liters	0.264	gallons	gal
m^3	cubic meters	35.29	cubic feet	ft^3
m^3	cubic meters	1.295	cubic yards	yd^3
MASS				
g	grams	0.035	ounces	oz
kg	kilograms	2.205	pounds	lb
tonnes	tonnes	1.103	US short tons	tons
TEMPERATURE				
ºC	Celsius	1.8ºC + 32	Fahrenheit	ºF
WEIGHT DENSITY				
kN/m^3	kilonewtons / cubic meter	6.36	Pound force / cubic foot	pcf
	FORCE and PRESSURE or STRESS			
N	newtons	0.225	pound force	lbf
kN	kilonewtons	225	pound force	lbf
kPa	kilopascals	0.145	pound force / square inch	psi
kPa	kilopascals	20.88	pound force / square foot	psf
PERMEABILITY (VELOCITY)				
cm/sec	centimeter/second	1.9685	feet/minute	ft/min

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SOILS AND FOUNDATIONS
VOLUME I

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

LIST OF SYMBOLS

Chapter 1

AASHTO	American Association of State Highway and Transportation Officials
AC	Asphaltic concrete
CMAR	Construction manager at risk
D-B	Design-build
FHWA	Federal Highway Administration
MSE	Mechanically stabilized earth
NHI	National Highway Institute
NMDOT	New Mexico Department of Transportation
PCC	Portland cement concrete
RCC	Reinforced cement concrete
RSS	Reinforced soil slope(s)
USDA	United States Department of Agriculture
USGS	Unites States Geological Survey

Chapter 2

A	Overall contact area
AASHTO	American Association of State Highway and Transportation Officials
B	Width
c	Cohesion
c'	Effective cohesion
D_r	Relative density
D_s, DOSI	Depth of significant influence
e	Void ratio
e_{max}	Maximum void ratio
e_{min}	Minimum void ratio
F_a	Tangential force
F_r	Shearing resistance
G	Specific gravity
G_s	Specific gravity of the solid phase
h	Embankment height
h_w	Depth to water table
K	Value of proportionality constant; coefficient of lateral earth pressure
K_a	Coefficient of active earth pressure
K_o	Coefficient of lateral earth pressure "at rest"
K_p	Coefficient of passive earth pressure
K_w	Coefficient of lateral earth pressure for water
L	Length
LI	Liquidity index
LL	Liquid limit
n	Porosity
P	Load applied
p_a	Active lateral pressure

p_f	Final stress = effective overburden pressure + pressure increment due to external loads
p_h	Lateral stress
PI	Plasticity index
PL	Plastic limit
P_n	Normal force
p_o	Effective overburden pressure
p_p	Passive lateral pressure
p_t	Total overburden pressure
Q	Load
q, q_o	Unit load of embankment
S	Degree of saturation
SI	Shrinking index
SL	Shrinkage limit
t	time
u	Porewater pressure
USCS	Unified Soil Classification System
V	Volume of the total soils mass
V_a	Volume of air phase
V_s	Volume of solid phase
V_v	Volume of total voids
V_w	Volume of water phase
w	Gravimetric water or moisture content
W	Weight of the total soil mass
W_a	Weight of air phase
W_s	Weight of solid phase
W_v	Weight of total voids
W_w	Weight of water phase
z	Depth
z_w	Depth below water table
δ	Angle of interface friction
δ_a	active translation
δ_b	passive translation
Δp	Pressure due to external loads
Δu	Excess pore water pressure
γ	Total unit weight
γ'	Effective unit weight
γ_b	Buoyant unit weight (same as effective unit weight)
γ_d	Dry unit weight
γ_s	Unit weight of the solid phase
γ_{sat}	Saturated unit weight
γ_T or γ_t	Total unit weight
γ_w	Unit weight of water
ϕ	Angle of friction
ϕ'	Effective angle of internal friction

ν	Poisson's ratio
θ	Angle of obliquity
θ_m	Maximum angle of obliquity
σ	Total stress
σ'	Effective stress
σ_n	Normal stress
σ_n'	Effective normal stress
τ	Shearing strength
τ'	Effective shear stress (strength)

Chapter 3

AASHTO	American Association of State Highway and Transportation Officials
AR	Area ratio
ASTM	American Society for Testing and Materials
BPT	Becker (Hammer) penetration test
C_N	Overburden correction factor or stress normalization parameter
CPT	Cone penetration test
CPTu, PCPT	Piezocone penetration test
d	Displacement
DC	Direct current
D_e	Diameter of sampler cutting tip
D_i	Inside diameter of the sampling tube
DMT	Flat plate dilatometer
D_o	Outside diameter of the sampling tube
DOT	Department of Transportation
E_f	Energy efficiency
ER	Energy ratio
F	Force
FEMA	Federal Emergency Management Agency
FHWA	Federal Highway Administration
f_s	Sleeve friction
g	Gravitational constant
GPR	Ground penetrating radar
h	Drop height
i	Inclination
ICR	Inside clearance ratio
ID	Inside Diameter
ISRM	International Society of Rock Mechanics
KE	Kinetic energy
LPT	Large penetration test
m	Mass
N	SPT blows per foot
$N1_{60}$	Overburden-normalized energy-corrected blowcount
N_{60}	Energy-corrected SPT-N value adjusted to 60% efficiency
NAVFAC	Naval Facilities Engineering Command
NCR	No core recovery

N_{meas}	N-Value measured in the field
OD	Outside diameter
PE	Potential energy
PMT	Pressuremeter test
p_o	Vertical effective pressure at the depth where the SPT is performed
PVC	Polyvinyl chloride
q_c	Cone tip resistance
q_t	Tip resistance
R_f	Friction ratio
RQD	Rock quality designation
S_t	Sensitivity
SASW	Spectral analysis of surface waves
SBT	Soil behavioral type
SCPTu	Seismic cone piezocone penetration test
SPT	Standard Penetration Test
SWPPP	Storm Water Pollution Prevention Plan
t_s	Shear wave time
u_m	Pore water pressure
USCS	Unified Soil Classification System
USEPA	United States Environmental Protection Agency
USGS	Unites States Geological Survey
v	Impact velocity
VST	Vane shear test
W	Work

Chapter 4

AASHTO	American Association of State Highway and Transportation Officials
ASTM	American Society for Testing and Materials
C_c	Coefficient of curvature
C_u	Coefficient of uniformity
D_{10}	Diameter of soil particles of which 10% of the soil is finer
D_{30}	Diameter of soil particles of which 30% of the soil is finer
D_{60}	Diameter of soil particles of which 60% of the soil is finer
DOT	Department of Transportation
EGS	Effective grain size
F	Percent passing No. 200 sieve
FHWA	Federal Highway Administration
GI	Group index
GIS	Geographic information system
GSD	Grain size distribution
IGM	Intermediate geomaterial
I_s	Point load index
ISRM	International Society of Rock Mechanics
LI	Liquid index
LL	Liquid limit
N	SPT blows per foot

N_{60}	Energy-corrected SPT-N value adjusted to 60% efficiency
NAVFAC	Naval Facilities Engineering Command
PI	Plasticity index
PL	Plastic limit
RMR	Rock mass rating
RQD	Rock quality designation
SPT	Standard penetration test
U.S.	United States
USCS	Unified Soil Classification System
USDA	United States Department of Agriculture

Chapter 5

A	Activity index
A	Area
AASHTO	American Association of State Highway and Transportation Officials
ASTM	American Society for Testing and Materials
B	Bulk modulus
c	Cohesion
C	Permeability coefficient
C_α	Coefficient of secondary compression
c'	Effective stress cohesion intercept
c'_{cu}	Effective stress cohesion from CU test
CBR	California Bearing Ratio
C_c	Compression index
$C_{c\varepsilon}$	Modified compression index
CD	Consolidated drained triaxial test
CF	Clay fraction
c_h	Coefficient of horizontal consolidation
c_o	Compacted cohesion
CP	Collapse potential
C_r	Recompression index
$C_{r\varepsilon}$	Modified recompression index
c_{sat}	Saturated cohesion
CU	Consolidated undrained triaxial test
c_u	Apparent cohesion
c_v	Coefficient of consolidation
c_v	Coefficient of vertical consolidation
C_α	Secondary compression index
$C_{\alpha\varepsilon}$	Modified secondary compression index
D	Distance between contact points of platens
D	Vane diameter
D_{10}	Diameter of soil particles of which 10% of the soil is finer
D_{60}	Diameter of soil particles of which 60% of the soil is finer
D_e	Equivalent core diameter
D_{max}	Maximum diameter of soil particle
D_{min}	Minimum diameter soil particles

D_r	Relative density
d_s	Equivalent diameter
e	Void ratio
e_{max}	Maximum void ratio
e_{min}	Minimum void ratio
E	Young modulus
E_i	Elastic modulus of intact rock
E_m	Elastic modulus of rock mass
e_{max}	Maximum void ratio
e_{min}	Minimum void ratio
e_o	Initial void ratio
E or E_s	Elastic modulus
FHWA	Federal Highway Administration
g	Gravitational constant
G	Shear modulus
G_S	Specific gravity
H	Height of vane
H	Soil layer thickness
H/D	Height to diameter ratio
H_o	Initial height of specimen
i_B	Angle of taper at the bottom of the vane
IL	Incremental load
I_s	Point load strength index
$I_{s(50)}$	Size-corrected point load strength index
i_T	Angle of taper at the top of the vane
k	Hydraulic conductivity
k_{PLT}	Size correction factor
LI	Liquid index
LIR	Load increment ratio
LL	Liquid limit
LVDT	Linear variable differential transducer
md	Man-days
MPC	Modified Proctor compaction
M_s	Mass of solid component of sample
M_t	Total mass
N	Normal stress
N	SPT blows per foot
$N1_{60}$	Overburden-normalized energy-corrected blowcount
N_{60}	Energy-corrected SPT-N value adjusted to 60% efficiency
NAVFAC	Naval Facilities Engineering Command
NC	Normally consolidated
n_h	Rate of increase of soil modulus with depth
OC	Over consolidated
OCR	Overconsolidation ratio
OMC	Optimum moisture content
P	Breaking load

p_c	Maximum past effective stress
p_c	Preconsolidation pressure
PI	Plasticity index
PL	Plastic limit
p_o	Effective overburden pressure
p_t	Total vertical stress
q_c	Cone tip resistance
q_u	Unconfined compression stress
RC	Relative compaction
RMR	Rock mass rating
RQD	Rock quality designation
S	Degree of saturation
S, S_t	Sensitivity
$s_{collapse}$	Collapse settlement
SL	Shrinkage limit
SPC	Standard Proctor compaction
SPT	Standard penetration test
$s_{r, VST}$	Remolded undrained shear strength (obtained by using VST data)
$s_{t, VST}$	Sensitivity (obtained by using VST data)
s_u	Undrained shear strength
$s_{u, VST}$	Undrained shear strength (obtained by using VST data)
s_u/p_o	Undrained strength ratio
T	Tangential (shear) force
T	Torque (related to VST)
t	Vane edge thickness
t_{100}	Time corresponding to 100% of primary consolidation
T_{max}	Maximum torque (related to VST)
T_{net}	Difference between T_{max} and T_{rod}
T_{rod}	Rod friction (related to VST)
u	Pore water pressure
UC	Unconfined compression test
U.S.	United States
USBR	United States Bureau of Reclamation
USCS	Unified Soil Classification System
UU	Unconsolidated undrained triaxial test
V	Coefficient of variation
V_s	Volume of soil solids
VST	Vane shear test
V_t	Total volume
W	Specimen width
w	Water content
w_n	Natural moisture content
w_{opt}	Optimum moisture content
W_s	Weight of solid component of soil
W_t	Total weight
Z	Depth below ground surface

Δe	Change in void ratio
ΔH_c	Change in height upon wetting
$\Delta\sigma$	Incremental stress
ε	Strain
γ	Unit weight
γ'	Effective unit weight
γ_b	Buoyant unit weight (same as effective unit weight)
$\gamma_{d\ field}$	Field dry unit weight
γ_d or γ_{dry}	Dry unit weight
$\gamma_{d\text{-}max}$	Maximum dry unit weight
γ_s	Unit weight of solid particles in the soil mass
γ_{sat}	Saturated unit weight
γ_t or γ_{tot}	Total unit weight
γ_t	Moist unit weight of compacted soil
γ_w	Unit weight of water
ϕ	Angle of internal friction
ϕ'	Effective friction angle
ϕ	Friction
ϕ'	Peak effective stress friction angle
ϕ'_{cu}	Effective friction angle from CU test
ϕ'_r	Residual effective stress friction angle
μ	Coefficient of friction
ν	Poisson ratio
ρ	Density
ρ_d or ρ_{dry}	Dry mass density
ρ_t or ρ_{tot}	Total mass density
ρ_t	Moist (total) mass density
σ'	Effective normal stress
σ_c	Uniaxial compressive strength
σ_n	Normal stress
σ'_p	Preconsolidation stress
σ_{vo}	Total vertical stress
τ	Shear stress
%C	Percent collapse

Chapter 6

AASHTO	American Association of State Highway and Transportation Officials
b	Unit width
b	Width of slice
c	Cohesion
c	Cohesion component of shear strength
c	Unit cohesion
c'	Effective cohesion
CD	Consolidated drained triaxial test
c_d	Developed cohesion
CU	Consolidated undrained triaxial test
d	Depth factor
D	Depth ratio
F_c	Average factor of safety with respect to cohesion
FHWA	Federal Highway Administration
FS or FOS	Factor of safety
F_ϕ	Average factor of safety with respect to friction angle
h	Depth less than or equal to the depth of saturation
H	Height
H	Height of soil layer in active wedge
h	Slope depth
H	Slope height
H'_w	Height of water within the slope
H_{Fill}	Fill height
h_i	Height of layer at center of slice
H_t	Tension crack height
h_w	Depth from groundwater surface to the centroid point on the circle
H_w	Depth of water outside the slope
H_{zone}	Height of zone
I_N	Interslice normal (horizontal) force
I_S	Interslice shear (vertical) force
K_a	Coefficient of active earth pressure
K_p	Coefficient of passive earth pressure
l	Arc length of slice base
L_s	Radius of circle
L_w	Level arm distance to the center of rotation
N	Normal force component or total normal force
N	Number of reinforcement layers
N'	Effective normal force component
N_{cf}	Critical stability number
N_o	Stability number
N_s	Stability number
P_a	Active force (driving)
p_o	In-situ vertical effective overburden pressure
P_p	Passive force (resisting)
q	Surcharge load

R	Moment arm
R_c	Coverage ratio of the reinforcement
RSS	Reinforced soil slope
S	Frictional force along failure plane
S	Shear strength along failure plane
SPT	Standard penetration test
S_v	Vertical spacing of reinforcement
T	Tangential force component
T_a	Sum of available tensile force per width of reinforcement for all reinforcement layers
$\tan \phi$	Coefficient of friction along failure surface
T_{MAX}	Maximum design tension
$T_{S\text{-}MAX}$	Maximum tensile force
T_{zone}	Maximum reinforced tension required for each zone
U	Pore water force
u	Water pressure on slice base
u	Water uplift pressure against failure surface
UU	Unconsolidated undrained triaxial test
W	Weight of slice
W_i	Partial weight
W_T	Total slice weight
α	Angle between vertical and line drawn from circle center to midpoint of slice base
α_w	Slope of water table from horizontal
γ_{Fill}	Fill soil unit weight
μ'_w	Seepage correction factor
μ_q	Surcharge correction factor
μ_t	Tension crack correction factor
μ_w	Submergence correction factor
σ	The total normal stress against the failure surface slice base due to the weight of soil and water above the failure surface
ΣW_i	Total weight of slice
β	Angle of slope
β	Inclination of the slope
ϕ	Angle of internal friction
ϕ'	Effective angle of internal friction
ϕ_d	Developed angle of internal friction
γ	Unit weight of soil
γ	Unit weight of soil in the active wedge
γ_i	Unit weight of layer i
γ	Effective unit weight
γ_m	Moist unit weight
γ_{sat}	Saturated unit weight
γ_t	Total soil unit weight
γ_w	Unit weight of water

σ'_n	Effective stress between soil grains
τ	Frictional shearing resistance
τ	Total shear strength
τ_d	Developed shear strength

Chapter 7

AASHTO	American Association of State Highway and Transportation Officials
C'	Bearing capacity index
C_c	Compression index
$C_{c\varepsilon}$	Modified compression index
C_r	Mean slope of the rebound laboratory curve
$C_{r\varepsilon}$	Modified recompression index
c_v	Coefficient of consolidation
C_α	Coefficient of secondary consolidation (determined from lab consolidation test)
$C_{\alpha\varepsilon}$	Modified secondary compression index
D_S	Depth of soft soil beneath the toe of the end slope of the embankment
e	Void ratio
e_o	Initial void ratio at p_o
FHWA	Federal Highway Administration
FS_{SQ}	Safety factor against failure by squeezing
H	Height of the fill
H	Thickness of soil layer considered
H_d	Distance to the drainage boundary
h_f	Fill height
H_o	Layer thickness
ID	Inner Diameter
$N1_{60}$	Number of blows per foot corrected for overburden and hammer efficiency
NCHRP	National Cooperative of Highway Research Program
OCR	Over consolidation ratio
p_c	Maximum past effective stress
p_c	Maximum past vertical pressure (preconsolidation)
p_f	Final effective vertical stress at the center of layer n
p_f	Final pressure applied to the foundation subsoil
p_f	Final stress
p_f	Total embanklment pressure
PI	Plasticity index
p_o	Effective overburden pressure
p_o	Existing effective overburden pressure
p_o	Initial effective vertical stress at the center of layer n
RSS	Reinforced soil slope
S	Degree of saturation
S	Settlement
S_c	Settlement due to primary consolidation
SPT N	Number of blows per foot (blow/0.3m)
SPT	Standard penetration test

S_s	Settlement due to secondary compression
S_t	Settlement at time t
s_u	Undrained shear strength of soft soil beneath embankment
$S_{ultimate}$	Settlement at end of primary consolidation
t	Time
$t_{1\ lab}$	Time when secondary compression begins
t_1	Time when approximately 90% of primary compression has occurred
t_{100}	Time for 100%of primary consolidation
$t_{2\ lab}$	Arbitrary time on the curve
t_2	The service life of the structure or any time of interest
t_{90}	Time for 90%of primary consolidation
T_v	Time factor
U	Average degree of consolidation
u_s	Hydrostatic pore water pressure at any depth
u_s	Initial hydrostatic pore water pressure
USACE	United States Army Corps of Engineers
u_{sb}	Hydrostatic pore water pressure at bottom of layer
u_{st}	Hydrostatic pore water pressure at top of layer
u_t	Total pore water pressure at any depth after time t
Z_I	Zone of influence
Δe	Change in void ratio
ΔH	Settlement
Δp	Distributed embankment pressure
Δp	Load increment
Δp	Stress increase
Δp_o	Effective vertical stress increment
Δp_t	Applied vertical stress increment
Δu	Excess pore water pressure at any depth after time t
Δu_i	Initial excess pore water pressure
ε_v	Vertical strain
γ	Unit weight of fill
γ'	Effective unit weight
γ_b	Buoyant unit weight (same as effective unit weight)
γ_f	Fill unit weight
θ	Angle of slope

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CHAPTER 4.0
ENGINEERING DESCRIPTION, CLASSIFICATION AND CHARACTERISTICS OF SOILS AND ROCKS

The geotechnical specialist is usually concerned with the design and construction of some type of geotechnical feature constructed on or out of a geomaterial. For engineering purposes, in the context of this manual, the geomaterial is considered to be primarily rock and soil. A geomaterial intermediate between soil and rock is labeled as an intermediate geomaterial (IGM). These three classes of geomaterials are described as follows:

- **Rock** is a relatively hard, naturally formed solid mass consisting of various minerals and whose formation is due to any number of physical and chemical processes. The rock mass is generally so large and so hard that relatively great effort (e.g., blasting or heavy crushing forces) is required to break it down into smaller particles.

- **Soil** is defined as a conglomeration consisting of a wide range of relatively smaller particles derived from a parent rock through mechanical weathering processes that include air and/or water abrasion, freeze-thaw cycles, temperature changes, plant and animal activity and by chemical weathering processes that include oxidation and carbonation. The soil mass may contain air, water, and/or organic materials derived from decay of vegetation, etc. The density or consistency of the soil mass can range from very dense or hard to loose or very soft.

- **Intermediate geomaterials** (IGMs) are transition materials between soils and rocks. The distinction of IGMs from soils or rocks for geotechnical engineering purposes is made purely on the basis of strength of the geomaterials. Discussions and special design considerations of IGMs are beyond the scope of this document.

The following three terms are often used by geotechnical specialists to describe a geomaterial: **identification**, **description** and **classification**. For soils, these terms have the following meaning:

- **Identification** is the process of determining which components exist in a particular soil sample, i.e., gravel, sand, silt, clay, etc.

- **Description** is the process of estimating the relative percentage of each component to prepare a word picture of the sample (ASTM D 2488). Identification and description are accomplished primarily by both a visual examination and the feel of the sample, particularly when water is added to the sample. Description is usually performed in the field and may be reevaluated by experienced personnel in the laboratory.

- **Classification** is the laboratory-based process of grouping soils with similar engineering characteristics into categories. For example, the Unified Soil Classification System, USCS, (ASTM D 2487), which is the most commonly used system in geotechnical work, is based on grain size, gradation, and plasticity. The AASHTO system (M 145), which is commonly used for highway projects, groups soils into categories having similar load carrying capacity and service characteristics for pavement subgrade design.

It may be noted from the above definitions that the description of a geomaterial necessarily includes its identification. Therefore, as used in this document, the term "description" is meant to include "identification."

The important distinction between classification and description is that standard AASHTO or ASTM laboratory tests must be performed to determine the classification. It is often unnecessary to perform the laboratory tests to classify every sample. Instead soil technicians are trained to identify and describe soil samples to an accuracy that is acceptable for design and construction purposes. ASTM D 2488 is used for guidance in such visual and tactile identification and description procedures. These visual/tactile methods provide the basis for a preliminary classification of the soil according to the USCS and AASHTO system.

During progression of a boring, the field personnel should describe only the soils encountered. Group symbols associated with classification should not be used in the field. It is important to send the soil samples to a laboratory for accurate visual description and classification by a laboratory technician experienced in soils work, as this assessment will provide the basis for later testing and soil profile development. Classification tests can be performed in the laboratory on representative samples to verify the description and assign appropriate group symbols based on a soil classification system (e.g., USCS). If possible, the moisture content of every sample should be determined since it is potentially a good indicator of performance. The test to determine the moisture content is simple and inexpensive to perform.

4.01 Primary References

The primary references for this Chapter are as follows:

ASTM (2006). *Annual Book of ASTM Standards – Sections 4.02, 4.08, 4.09 and 4.13.* ASTM International, West Conshohocken, PA.

AASHTO (2006). *Standard Specifications for Transportation Materials and Methods of Sampling and Testing,* Parts I and II, American Association of State Highway and Transportation Officials, Washington, D.C.

FHWA (2002a). *Geotechnical Engineering Circular 5 (GEC5) - Evaluation of Soil and Rock Properties*. Report No FHWA-IF-02-034. Authors: Sabatini, P.J, Bachus, R.C, Mayne, P.W., Schneider, J.A., Zettler, T.E., Federal Highway Administration, U.S. Department of Transportation.

4.1 SOIL DESCRIPTION

Soil description/identification is the systematic naming of individual soils in both written and spoken forms (ASTM D 2488, AASHTO M 145). Soil classification is the grouping of soils with similar engineering properties into a category by using the results of laboratory-based index tests, e.g., group name and symbol (ASTM D 2487, AASHTO M 145). It is important to distinguish between a visual description of a soil and its classification in order to minimize potential conflicts between general visual evaluations of soil samples in the field and more precise laboratory evaluations supported by index tests.

The soil's description should include as a **minimum**:

- Apparent consistency (e.g., soft, firm, etc. for fine-grained soils) or density adjective (e.g., loose, dense, etc. for coarse-grained soils);
- Water content condition adjective (e.g., dry, moist, wet);
- Color description (e.g., brown, gray, etc.);
- Main soil type name, often presented in all capital letters (e.g. SAND, CLAY);
- Descriptive adjective for main soil type (e.g., fine, medium, coarse, well-rounded, angular, etc. for coarse-grained soils; organic, inorganic, compressible, laminated, etc., for fine-grained soils);
- Particle-size distribution adjective for gravel and sand (e.g., uniform, well-graded,

gap-graded);

- Plasticity adjective (e.g., high, low) and soil texture (e.g., rough, smooth, slick, waxy, etc.) for inorganic and organic silts or clays;
- Descriptive term for minor type(s) of soil (with, some, trace, etc.);
- Minor soil type name with "y" added if the fine-grained minor component is less than 30 percent but greater than 12 percent or the coarse-grained minor component is 30 percent or more (e.g., silty for fine grained minor soil type, sandy for coarse-grained minor soil type);
- Descriptive adjective "with" if the fine-grained minor soil type is 5 to 12 percent (e.g., with clay) or if the coarse-grained minor soil type is less than 30 percent but 15 percent or more (e.g., with gravel). Note: some practices use the descriptive adjectives "some" and "trace" for minor components;
- Inclusions (e.g., concretions, cementation);
- Geological name (e.g., Holocene, Eocene, Pleistocene, Cretaceous), if known, in parenthesis or in notes column.

The various elements of the soil description are generally stated in the order given above. For example, a soil description might be presented as follows:

Fine-grained soils: Soft, wet, gray, high plasticity CLAY, with f. Sand; (Alluvium)

Coarse-grained soils: Dense, moist, brown, silty m-f SAND, with f. Gravel to c. Sand; (Alluvium)

When minor changes occur within the same soil layer (e.g., a change in apparent density), the boring log should indicate a description of the change, such as "same, except very dense."

4.1.1 Consistency and Apparent Density

The consistency of fine-grained soils and apparent density of coarse-grained soils can be estimated from the energy-corrected SPT N-value, N_{60}. The consistency of clays and silts varies from very soft to firm to stiff to hard. The apparent density of coarse-grained soil ranges from very loose to dense to very dense. Suggested guidelines for estimating the in-place apparent density or consistency of soils are given in Tables 4-1 and 4-2, respectively.

Table 4-1
Evaluation of the apparent density of coarse-grained soils (after Peck, *et al.*, 1974)

N_{60}	Apparent Density	Relative Density, %
0 – 4	Very loose	0 – 20
>4 - 10	Loose	20 – 40
>10 - 30	Medium dense	40 – 70
>30 - 50	Dense	70 – 85
>50	Very Dense	85 – 100
The above guidance may be misleading in gravelly soils.		

Table 4-2
Evaluation of the consistency of fine-grained soils (after Peck, *et al.*, 1974)

N_{60}	Consistency	Unconfined Compressive Strength, q_u, ksf (kPa)	Results of Manual Manipulation
<2	Very soft	< 0.5 (<25)	Specimen (height = twice the diameter) sags under its own weight; extrudes between fingers when squeezed.
2 - 4	Soft	0.5 – 1 (25 – 50)	Specimen can be pinched in two between the thumb and forefinger; remolded by light finger pressure.
4 - 8	Medium stiff	1 – 2 (50 – 100)	Can be imprinted easily with fingers; remolded by strong finger pressure.
8 - 15	Stiff	2 – 4 (100 – 200)	Can be imprinted with considerable pressure from fingers or indented by thumbnail.
15 - 30	Very stiff	4 – 8 (200 – 400)	Can barely be imprinted by pressure from fingers or indented by thumbnail.
>30	Hard	> 8 >400	Cannot be imprinted by fingers or difficult to indent by thumbnail.
Note that N_{60}-values should not be used to determine the design strength of fine grained soils.			

The apparent density or consistency of the soil formation can vary from these empirical correlations for a variety of reasons. Judgment remains an important part of the visual identification process. Field index tests (e.g., smear test, dried strength test, thread test) which will be described in the next section are suggested as aids in estimating the consistency of fine grained soils.

In some cases the sampler may pass from one layer into another of markedly different properties; for example, from a dense sand into a soft clay. In attempting to identify apparent density, an assessment should be made as to what part of the blow count corresponds to each layer since the sampler begins to reflect the presence of the lower layer before it actually reaches it.

4.1.2 Water Content (Moisture)

The relative amount of water present in the soil sample should be described by an adjective such as dry, moist, or wet as indicated in Table 4-3.

Table 4-3
Adjectives to describe water content of soils (ASTM D 2488)

Description	Conditions
Dry	No sign of water and soil dry to touch
Moist	Signs of water and soil is relatively dry to touch
Wet	Signs of water and soil definitely wet to touch; granular soil exhibits some free water when densified

4.1.3 Color

The color must be described when the sample is first retrieved in the field at the as-sampled water content since the color may change with changes in the water content. Primary colors should be used (brown, gray, black, green, white, yellow, red). Soils with different shades or tints of basic colors are described by using two basic colors; e.g., gray-green. Some agencies may require use of the Munsell color system (USDA, 1993). When the soil is marked with spots of color, the term "mottled" can be applied. Soils with a homogeneous texture but having color patterns that change and are not considered mottled can be described as "streaked."

4.1.4 Type of Soil

The constituent parts of a given soil type are defined on the basis of texture in accordance with particle-size designators separating the soil into coarse-grained, fine-grained, and highly organic designations. Soil with more than 50 percent by weight of the particles larger than the U.S. Standard No. 200 sieve (0.075 mm) is designated coarse-grained. Soil (inorganic and organic) with 50 percent or more by weight of the particles finer than the No. 200 sieve (0.075 mm) is designated fine-grained. Soil primarily consisting of less than 50 percent by volume of organic matter, dark in color, and with an organic odor is designated as organic soil. Soil with organic content more than 50 percent is designated as peat. The soil type designations used by FHWA follow ASTM D 2487; i.e., gravel, sand, silt, clay, organic silt, organic clay, and peat.

4.1.4.1 Coarse-Grained Soils (Gravel and Sand)

Coarse-grained soils consist of a matrix of either gravel or sand in which more than 50 percent by weight of the soil is retained on the No. 200 sieve (0.075 mm). Coarse-grained soils may contain fine-grained soil, i.e., soils passing the No. 200 sieve (0.075 mm), but the percent by weight of the fine-grained portion is less than 50 percent. The gravel and sand components are defined on the basis of particle size as indicated in Table 4-4. The particle-size distribution is identified as well graded or poorly graded. Well graded coarse-grained soil contains a good representation of all particle sizes from largest to smallest, with ≤12 percent fines. Poorly graded coarse-grained soil is uniformly graded, i.e., most of the coarse-grained particles are about the same size, with ≤ 12 percent fines. Gap graded coarse grained soil can be either a well graded or poorly graded soil lacking one or more intermediate sizes within the range of the gradation.

Gravels and sands may be described by adding particle-size distribution adjectives in front of the soil type in accordance with the criteria given in Table 4-5. Based on correlation with laboratory tests, the following simple field identification tests can be used as an aid in identifying granular soils.

Table 4-4
Particle size definition for gravels and sands (after ASTM D 2488)

Component	Grain Size	Determination
Boulders*	12" + (300 mm +)	Measurable
Cobbles*	3" to 12" (300 mm to 75 mm)	Measurable
Gravel		
Coarse	¾" – 3" (19 mm to 75 mm)	Measurable
Fine	¾" to #4 sieve (¾" to 0.187") (19 mm to 4.75 mm)	Measurable
Sand		
Coarse	#4 to #10 sieve (0.19" to 0.079") (4.75 mm – 2.00 mm)	Measurable and visible to the eye
Medium	#10 to #40 sieve (0.079" to 0.017") (2.00 mm – 0.425 mm)	Measurable and visible to the eye
Fine	#40 to #200 sieve (0.017" to 0.003") (0.425 mm- 0.075 mm)	Measurable but barely discernible to the eye
*Boulders and cobbles are not considered soil or part of the soil's classification or description, except under miscellaneous description; i.e., with cobbles at about 5 percent (volume).		

Table 4-5
Adjectives for describing size distribution for sands and gravels (after ASTM D 2488)

Particle-Size Adjective	Abbreviation	Size Requirement
Coarse	c.	< 30% m-f sand or < 12% f. gravel
Coarse to medium	c-m	< 12% f. sand
Medium to fine	m-f	< 12% c. sand and > 30% m. sand
Fine	f.	< 30% m. sand or < 12% c. gravel
Coarse to fine	c-f	> 12% of each size[1]

[1] 12% and 30% criteria can be modified depending on fines content. The key is the shape of the particle-size distribution curve. If the curve is relatively straight or dished down, and coarse sand is present, use c-f, also use m-f sand if a moderate amount of m. sand is present. If one has any doubts, determine the above percentages based on the amount of sand or gravel present.

Feel and Smear Tests: A pinch of soil is handled lightly between the thumb and fingers to obtain an impression of the grittiness (i.e., roughness) or softness (smoothness) of the constituent particles. Thereafter, a pinch of soil is smeared with considerable pressure between the thumb and forefinger to determine the degrees of grittiness (roughness), or the softness (smoothness) of the soil. The following guidelines may be used:

- Coarse- to medium-grained sand typically exhibits a very gritty feel and smear.
- Coarse- to fine-grained sand has less gritty feel, but exhibits a very gritty smear.
- Medium- to fine-grained sand exhibits a less gritty feel and smear that becomes softer (smoother) and less gritty with an increase in the fine sand fraction.
- Fine-grained sand exhibits a relatively soft feel and a much less gritty smear than the coarser sand components.
- Silt components less than about 10 percent of the total weight can be identified by a slight discoloration of the fingers after smear of a moist sample. Increasing silt increases discoloration and softens the smear.

Sedimentation Test: A small sample of soil is shaken in a test tube filled with water and allowed to settle. The time required for the particles to fall a distance of 4-inches (100 mm) is about 1/2 minute for particle sizes coarser than silt. About 50 minutes would be required for particles of 0.0002 in (0.005 mm) or smaller (often defined as "clay size") to settle out.

For sands and gravels containing more than 5 percent fines, the type of inorganic fines (silt or clay) can be identified by performing a shaking/dilatancy test. See fine-grained soils section.

Visual Characteristics: Sand and gravel particles can be readily identified visually, but silt particles are generally indistinguishable to the eye. With an increasing silt component, individual sand grains become obscured, and when silt exceeds about 12 percent, the silt almost entirely masks the sand component from visual separation. Note that gray fine-grained sand visually appears to contain more silt than the actual silt content.

4.1.4.2 Fine-Grained Soils

Fine-grained soils are those having 50 percent or more by weight pass the No. 200 sieve. The so-called fines are either inorganic or organic silts and/or clays. To describe fine-grained soils, plasticity adjectives and soil-type adjectives should be used to further define the soil's plasticity and texture. The following simple field identification tests can be used to estimate the degree of plasticity of fine-grained soils.

Shaking (Dilatancy) Test (Holtz and Kovacs, 1981). Water is dropped or sprayed on a portion of a fine-grained soil sample mixed and held in the palm of the hand until it shows a wet surface appearance when shaken or bounced lightly in the hand or a sticky nature when touched. The test involves lightly squeezing the wetted soil sample between the thumb and forefinger and releasing it alternatively to observe its reaction and the speed of the response. Soils that are predominantly silty (nonplastic to low plasticity) will show a dull dry surface upon squeezing and a glassy wet surface immediately upon release of the pressure. This phenomenon becomes less and less pronounced in soils with increasing plasticity and decreasing dilatancy,

Dry Strength Test (Holtz and Kovacs, 1981). A relatively undisturbed portion of the sample is allowed to dry out and a fragment of the dried soil is pressed between the fingers. Fragments which cannot be crumbled or broken are characteristic of clays with high plasticity. Fragments which can be disintegrated with gentle finger pressure are characteristic of silty materials of low plasticity. Thus, in generally, fine-grained materials with relatively high dry strength are clays of high plasticity and those with relatively little dry strength are predominantly silts.

Thread Test (After Burmister, 1970). Moisture is added to or worked out of a small ball (about 1.5 in (40 mm) diameter) of fine grained soil and the ball kneaded until its consistency approaches medium stiff to stiff (compressive strength of about 2,100 psf (100 kPa)). This condition is observed when the material just starts to break or crumble. A thread is then rolled out between the palm of one hand and the fingers of the other to the smallest diameter possible before disintegration of the sample occurs. The smaller the thread achieved, the higher the plasticity of the soil. Fine-grained soils of high plasticity will have threads smaller than 0.03 in (3/4 mm) in diameter. Soils with low plasticity will have threads larger than 0.12 in (3 mm) in diameter.

Smear Test (FHWA, 2002b). A fragment of soil smeared between the thumb and forefinger or drawn across the thumbnail will, by the smoothness and sheen of the smear surface, indicate the plasticity of the soil. A soil of low plasticity will exhibit a rough textured, dull smear while a soil of high plasticity will exhibit a slick, waxy smear surface.

Table 4-6 identifies field methods to approximate the plasticity range for the dry strength, thread, and smear tests.

Table 4-6
Field methods to describe plasticity (FHWA, 2002b)

Plasticity Range	Adjective	Dry Strength	Smear Test	Thread Smallest Diameter, in (mm)
0	Nonplastic	none - crumbles into powder with mere pressure	gritty or rough	ball cracks
1 - 10	low plasticity	low - crumbles into powder with some finger pressure	rough to smooth	1/4 – 1/8 (6 to 3)
>10 - 20	medium plasticity	medium - breaks into pieces or crumbles with considerable finger pressure	smooth and dull	1/16 (1.5)
>20 - 40	high plasticity	high - cannot be broken with finger pressure; spec. will break into pieces between thumb and a hard surface	Shiny	0.03 (0.75)
>40	very plastic	very high - can't be broken between thumb and a hard surface	very shiny and waxy	0.02 (0.5)

4.1.4.3 Highly Organic Soils

Colloidal and amorphous organic materials finer than the No. 200 sieve (0.075 mm) are identified and classified in accordance with their drop in plasticity upon oven drying (ASTM D 2487). Further identification markers are:

1. dark gray and black and sometimes dark brown colors, although not all dark colored soils are organic;
2. most organic soils will oxidize when exposed to air and change from a dark gray/black color to a lighter brown; i.e., the exposed surface is brownish, but when the sample is pulled apart the freshly exposed surface is dark gray/black;
3. fresh organic soils usually have a characteristic odor that can be recognized,

particularly when the soil is heated;

4. compared to inorganic soils, less effort is typically required to pull the material apart and a friable break is usually formed with a fine granular or silty texture and appearance;
5. workability of organic soils at the plastic limit is weaker and spongier than an equivalent inorganic soil;
6. the smear, although generally smooth, is usually duller and appears more silty than an equivalent inorganic soil's; and
7. the organic content of organic soils can also be determined by the combustion test method (AASHTO T 267, ASTM D 2974).

Fine-grained soils, where the organic content appears to be less than 50 percent of the volume (about 22 percent by weight), should be described as soils with organic material or as organic soils such as clay with organic material or organic clays etc. If the soil appears to have an organic content greater than 50 percent by volume it should be described as peat. The engineering behavior of soils below and above the 50 percent dividing line is entirely different. It is therefore critical that the organic content of soils be determined both in the field and in the laboratory (AASHTO T 267, ASTM D 2974). Simple field or visual laboratory identification of soils as organic or peat is neither advisable nor acceptable.

It is very important not to confuse topsoil with organic soils or peat. Topsoil is the relatively thin layer of soil found on the surface composed of partially decomposed organic materials, such as leaves, grass, small roots etc. Topsoil contains many nutrients that sustain plant and insect life and should not be used to construct geotechnical features or to support engineered structures.

4.1.4.4 Minor Soil Type(s)

Two or more soil types may be present in many soil formations,. When the percentage of the fine-grained minor soil type is less than 30 percent but greater than 12 percent, or the total sample or the coarse-grained minor component is 30 percent or more of the total sample, the minor soil type is indicated by adding a "y" to its name (e.g., f. gravelly, c-f. sandy, silty, clayey). Note the gradation adjectives are given for granular soils, while the plasticity adjective is omitted for the fine-grained soils.

When the percentage of the fine-grained minor soil type is 5 to 12 percent or for the coarse-grained minor soil type is less than 30 percent but 15 percent or more of the total sample, the minor soil type is indicated by adding the descriptive adjective "with" to the group name (i.e., with clay, with silt, with sand, with gravel, and/or with cobbles).

Some local practices also use the descriptive adjectives “some” and “trace” for minor components as follows:

- "trace" when the percentage is between 1 and 12 percent of the total sample; or
- "some" when the percentage is greater than 12 percent and less than 30 percent of the total sample.

4.1.4.5 Inclusions

Additional inclusions or characteristics of the sample can be described by using "with" and the descriptions described above. For example:

- with petroleum odor
- with organic matter
- with foreign matter (roots, brick, etc.)
- with shell fragments
- with mica
- with parting(s), seam(s), etc. of (give soil’s complete description)

4.1.4.6 Other Descriptors

Depending on local conditions, the soils may be described based on reaction to HCl acid, and type and degree of cementation. ASTM D 2488 provides guidance for such descriptors.

4.1.4.7 Layered Soils

Soils of different types can be found in repeating layers of various thickness. It is important that all such formations and their thicknesses are noted. Each layer is described as if it is a non-layered soil by using the sequence for soil descriptions discussed above. The thickness and shape of layers and the geological type of layering are noted according to the descriptive terms presented in Table 4-7. The thickness designation is given in parentheses before the type of layer or at the end of each description, whichever is more appropriate.

Examples of descriptions for layered soils are:

- Medium stiff, moist to wet 0.2 to 0.75 in (5 to 20 mm) interbedded seams and layers of gray, medium plastic, silty CLAY and lt. gray, low plasticity SILT; (Alluvium).

- Soft moist to wet varved layers of gray-brown, high plasticity CLAY (0.2 to 0.75-in (5 to 20 mm)) and nonplastic SILT, trace f. sand (0.4 to 0.6 in (10 to 15 mm)); (Alluvium).

Table 4-7
Descriptive terms for layered soils (NAVFAC, 1986a)

Type of Layer	Thickness	Occurrence
Parting	< 1/16" (< 1.5 mm)	
Seam	1/16 to ½" (1.5 mm to 12 mm)	
Layer	½" to 12" (12 mm to 300 mm)	
Stratum	> 12" (>300 mm)	
Pocket		Small erratic deposit
Lens		Lenticular deposit
Varved (also layered)		Alternating seams or layers of silt and/or clay and sometimes fine sand
Occasional		One or less per 12" (300 mm) of thickness or laboratory sample inspected
Frequent		More than one per 12" (300 mm) of thickness or laboratory

4.1.4.8 Geological Name

The soil description should include the geotechnical specialist's assessment of the origin of the soil unit and the geologic name, if known. This information is generally placed in parentheses or brackets at the end of the soil description or in the field notes column of the boring log. Some examples include:

a. *Washington, D.C.*-Cretaceous Age Material with SPT N-values between 30 and 100: Very hard gray-blue silty CLAY (CH), moist **[Potomac Group Formation]**

b. *Newport News, VA*-Miocene Age Marine Deposit with SPT N-values around 10 to 15: Stiff green sandy CLAY (CL) with shell fragments, calcareous **[Yorktown Formation].**

c. *Tucson, AZ* – Holocene Age Alluvial Deposit with SPT N-values around 35: Cemented clayey SAND (SC), dry [**Pantano Formation].**

4.2 SOIL CLASSIFICATION

As previously indicated, final identification with classification is best performed in the laboratory. This process will lead to more consistent final boring logs and avoid conflicts with field descriptions. The Unified Soil Classification System (USCS) group name and symbol (in parenthesis) appropriate for the soil type in accordance with AASHTO M 145 (or ASTM D 3282) or ASTM D 2487 is the most commonly used system in geotechnical work and is covered in this section. For classification of highway subgrade material, the AASHTO classification system (see Section 4.2.2) is used. The AASHTO classification system is also based on grain size and plasticity.

4.2.1 Unified Soil Classification System (USCS)

The Unified Soil Classification System (ASTM D 2487) groups soils with similar engineering properties into categories base on grain size, gradation and plasticity. Table 4-8 provides a simplification of the group breakdown based on percent passing No. 200 sieve (0.075 mm) and Table 4-9 provides an outline of the complete laboratory classification method. The procedures, along with charts and tables, for classifying coarse-grained and fine-grained soils follow.

Table 4-8
Basic USCS soil designations based on percent passing No. 200 sieve (0.075 mm) (after ASTM D 2487; Holtz and Kovacs, 1981)

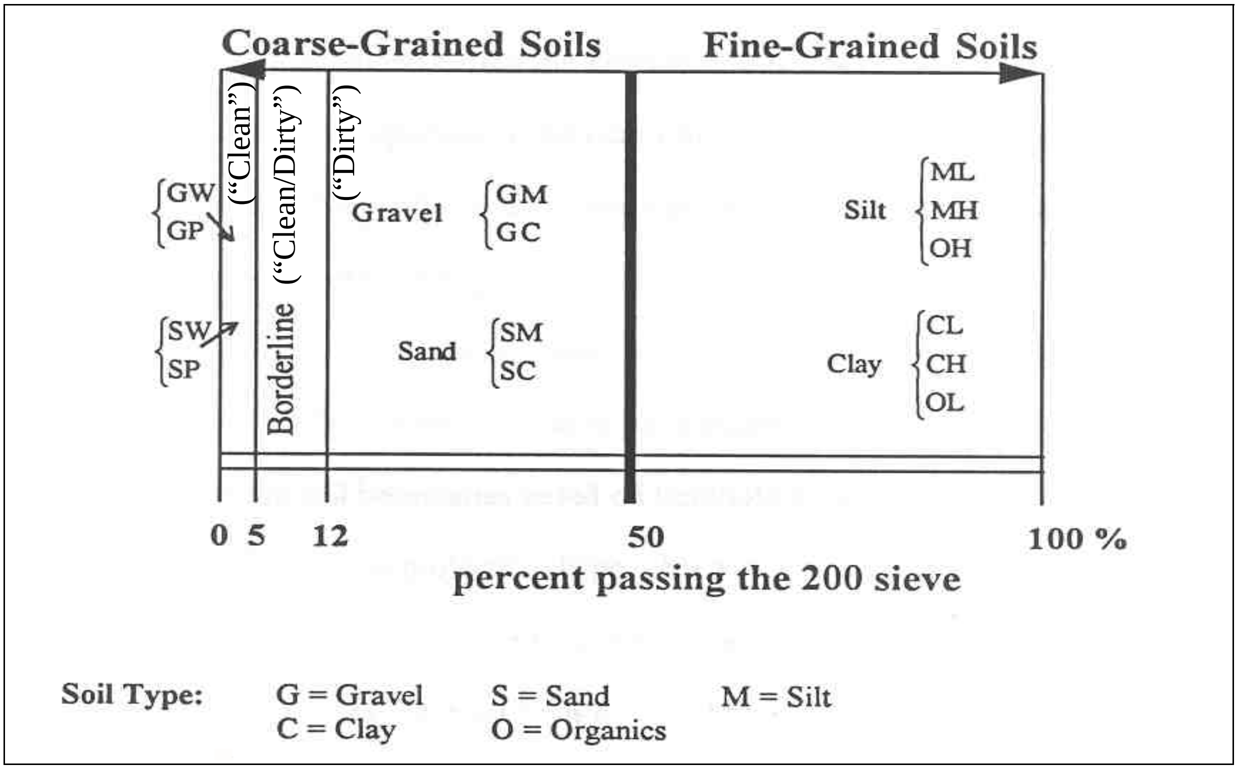


Table 4-9
Soil classification chart (laboratory method) (after ASTM D 2487)

Criteria for Assigning Group Symbols and Group Names Using Laboratory Tests[a]			Soil Classification	
COARSE-GRAINED SOILS (Sands and Gravels) - more than 50% retained on No. 200 (0.075 mm) sieve **FINE-GRAINED (Silts and Clays) -** 50% or more passes the No. 200 (0.075 mm) sieve			**Group Symbol**	**Group Name[b]**
GRAVELS More than 50% of coarse Fraction retained on No. 4 Sieve	CLEAN GRAVELS < 5% fines	$C_u \geq 4$ and $1 \leq C_c \leq 3$[e]	GW	Well-graded gravel[f]
		$C_u < 4$ and/or $1 > C_c > 3$[e]	GP	Poorly-graded gravel[f]
	GRAVELS WITH FINES > 12% of fines[c]	Fines classify as ML or MH	GM	Silty gravel[f,g,h]
		Fines classify as CL or CH	GC	Clayey gravel[f,g,h]
SANDS 50% or more of coarse fraction passes No. 4 Sieve	CLEAN SANDS < 5% fines[d]	$C_u \geq 6$ and $1 \leq C_c \leq 3$[e]	SW	Well-graded Sand[i]
		$C_u < 6$ and/or $1 > C_c > 3$[e]	SP	Poorly-graded sand[i]
	SANDS WITH FINES > 12% fines[d]	Fines classify as ML or MH	SM	Silty sand[g,h,i]
		Fines classify as CL or CH	SC	Clayey sand[g,h,i]
SILTS AND CLAYS Liquid limit less than 50	Inorganic	PI > 7 and plots on or above "A" line[j]	CL	Lean clay[k,l,m]
		PI < 4 or plots below "A" line[j]	ML	Silt[k,l,m]
	Organic	$\frac{\text{Liquid limit - overdried}}{\text{Liquid limit - not dried}} < 0.75$	OL	Organic clay[k,l,m,n]
				Organic silt[k,l,m,o]
SILTS AND CLAYS Liquid limit 50 or more	Inorganic	PI plots on or above "A" line	CH	Fat clay[k,l,m]
		PI plots below "A" line	MH	Elastic silt[k,l,m]
	Organic	$\frac{\text{Liquid limit - oven dried}}{\text{Liquid limit - not dried}} < 0.75$	OH	Organic clay[k,l,m,p]
				Organic silt[k,l,m,q]
Highly fibrous organic soils	Primary organic matter, dark in color, and organic odor		Pt	Peat

Table 4-9 (Continued)
Soil classification chart (laboratory method) (after ASTM D 2487)

NOTES:

a Based on the material passing the 3 in (75 mm) sieve.

b If field sample contained cobbles and/or boulders, add "with cobbles and/or boulders" to group name.

c Gravels with 5 to 12% fines require dual symbols:
- GW-GM, well-graded gravel with silt
- GW-GC, well-graded gravel with clay
- GP-GM, poorly graded gravel with silt
- GP-GC, poorly graded gravel with clay

d Sands with 5 to 12% fines require dual symbols:
- SW-SM, well-graded sand with silt
- SW-SC, well-graded sand with clay
- SP-SM, poorly graded sand with silt
- SP-SC, poorly graded sand with clay

e $C_u = \frac{D_{60}}{D_{10}} \quad C_c = \frac{(D_{30})^2}{(D_{10})(D_{60})}$

[C_u: Uniformity Coefficient; C_c: Coefficient of Curvature]

f If soil contains ≥ 15% sand, add "with sand" to group name.

g If fines classify as CL-ML, use dual symbol GC-GM, SC-SM.

h If fines are organic, add "with organic fines" to group name.

i If soil contains ≥ 15% gravel, add "with gravel" to group name.

j If the liquid limit and plasticity index plot in hatched area on plasticity chart, soil is a CL-ML, silty clay.

k If soil contains 15 to 29% plus No. 200 (0.075 mm), add "with sand" or "with gravel," whichever is predominant.

l If soil contains ≥ 30% plus No. 200 (0.075mm), predominantly sand, add "sandy" to group name.

m If soil contains ≥ 30% plus No. 200 (0.075 mm), predominantly gravel, add "gravelly" to group name.

n PI ≥ 4 and plots on or above "A" line.

o PI < 4 or plots below "A" line.

p PI plots on or above "A" line.

q PI plots below "A" line.

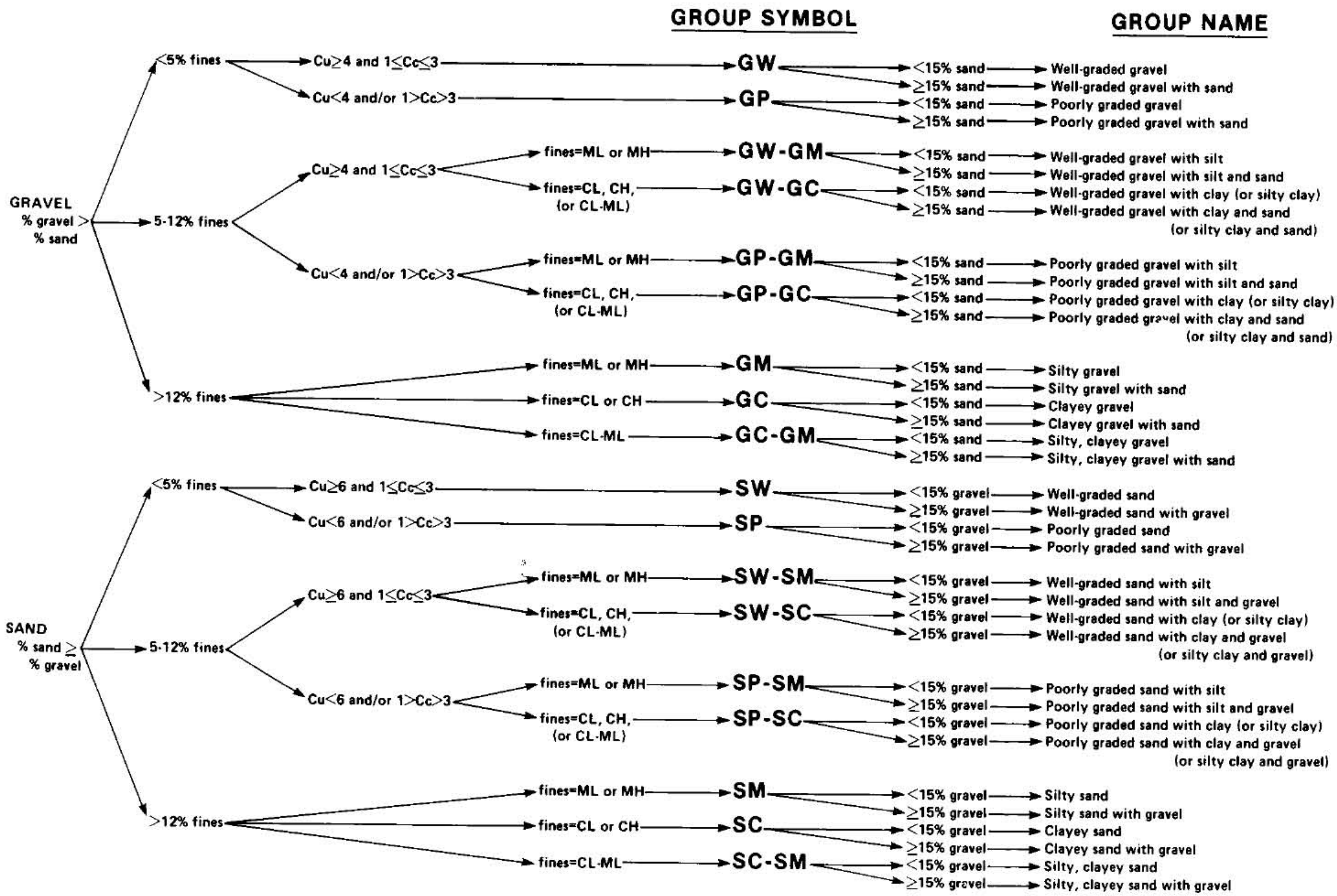


Figure 4-1: Flow chart to determine the group symbol and group name for coarse-grained soils (ASTM D 2487).

4.2.1.1 Classification of Coarse-Grained Soils

Coarse-grained soils are defined as those in which 50 percent or more by weight are retained on the No. 200 sieve (0.075 mm). The flow chart to determine the group symbol and group name for coarse-grained soils is given in Figure 4-1. This figure is identical to Figure 3 in ASTM D 2487 except for the recommendation to capitalize the primary soil type; e.g., GRAVEL.

- **The shape of the grain-size distribution (GSD) curve or "gradation curve" as it is frequently called, is one of the more important aspects in a soil classification system for coarse-grained soils.** The shape of the gradation curve can be characterized by a pair of "shape" parameters called the coefficient of uniformity, C_u, and the coefficient of curvature, C_c, to which numerical values may be assigned. By assigning numerical values to such shape parameters it becomes possible to compare grain-size distribution curves for different soils without having to plot them on the same diagram. In order to define shape parameters certain characteristic particle sizes must be identified that are common to all soils. Since the openings of a sieve are square, particles of many different shapes are able to pass through a sieve of given size even though the abscissa on the gradation curve is expressed in terms of particle "diameter," which implies a spherical-shaped particle. Therefore, the "diameter" shown on the gradation curve is an effective diameter so that the characteristic particle sizes that must be identified to define the shape parameters are in reality effective grain sizes (EGS).

  A useful EGS for the characterizing the shape of the gradation curve is the grain size for which 10 percent of the soil by weight is finer. This EGS is labeled D_{10}. This size is convenient because Hazen (1911) found that the ease with which water flows through a soil is a function of the D_{10}. In other words, Hazen found that the sizes smaller than the D_{10} affected the permeability more than the remaining 90 percent of the sizes. Therefore, the D_{10} is a logical choice as a characteristic particle size. Other convenient sizes were found to be the D_{30} and the D_{60}, which pertain to the grain size for which thirty and sixty percent, respectively, of the soil by weight is finer. These EGSs are used as follows in the Unified Soil Classification System (USCS) for the classification of coarse grained soils.

- **Slope of the gradation curve**: The shape of the curve could be defined relative to an arbitrary slope of a portion of the gradation curve. Since one EGS has already been identified as the D_{10}, the slope of the gradation curve could be described by identifying another convenient point (EGS) that is "higher" on the curve. Hazen selected this other convenient size as the D_{60} that indicates the particle size for which 60 percent of the soil by weight is finer. The slope between the D_{60} and the D_{10} can then be related to the degree of uniformity of the sample through a parameter called the "Coefficient of Uniformity" or the "Uniformity Coefficient," C_u, which is expressed as follows:

$$C_u = \frac{D_{60}}{D_{10}} \qquad \text{4-1}$$

- **Curvature of the gradation curve**: The second "shape" parameter is used to evaluate the curvature of the gradation curve between the two arbitrary points, D_{60} and D_{10}. A third EGS, D_{30}, that indicates the particle size for which 30 percent of the soil by weight is finer, is chosen for this purpose. The curvature of the slope between the D_{60} and the D_{10} can then be related to the three EGS' through a parameter called the "Coefficient of Curvature" or the "Coefficient of Concavity" or the "Coefficient of Gradation," C_c, which is expressed as follows:

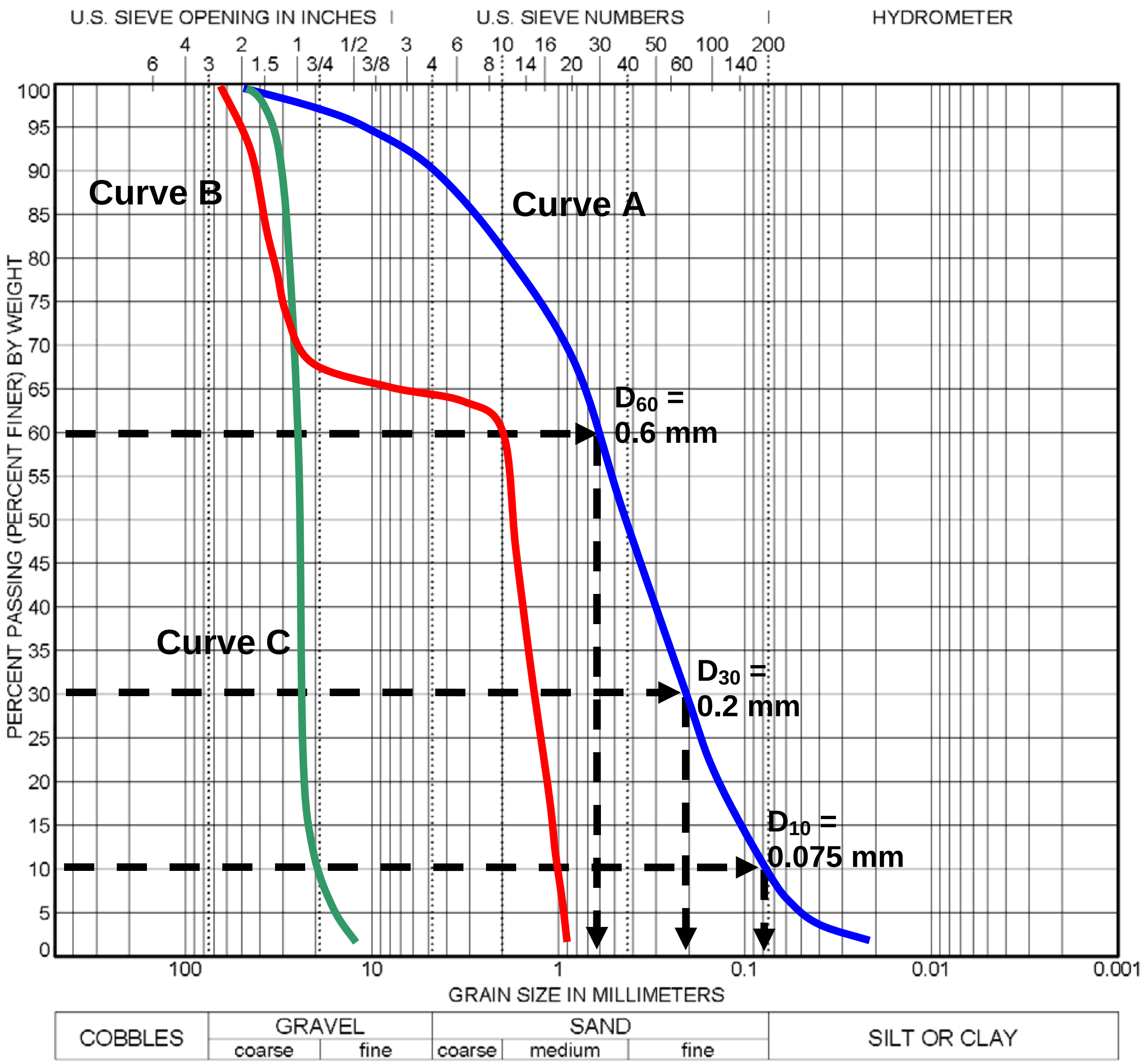
$$C_c = \frac{D_{30}^2}{D_{60} \text{ x } D_{10}} \qquad \text{4-2}$$

By use of the two "shape" parameters, C_u and C_c, the uniformity of the coarse-grained soil (gravel and sand) can now be classified as well-graded (non-uniform), poorly graded (uniform), or gap graded (uniform or non-uniform). Table 4-10 presents criteria for such classifications.

Table 4-10
Gradation based on C_u and C_c parameters

Gradation	**Gravels**	**Sands**
Well-graded	$C_u \geq 4$ and $1 < C_c < 3$	$C_u \geq 6$ and $1 < C_c < 3$
Poorly graded	$C_u < 4$ and $1 < C_c < 3$	$C_u < 6$ and $1 < C_c < 3$
Gap graded*	C_c not between 1 and 3	C_c not between 1 and 3
*Gap-graded soils may be well-graded or poorly graded. In addition to the C_c value it is recommended that the shape of the GSD be the basis for definition of gap-graded.		

C_u and C_c are statistical parameters and provide good initial guidance. However, **the plot of the GSD curve must always be reviewed in conjunction with the values of C_u and C_c to avoid incorrect classification.** Examples of the importance of reviewing the GSD curves are presented in Figure 4-2 and discussed subsequently.



Curve	D_{10} (mm)	D_{30} (mm)	D_{60} (mm)	C_u	C_c	Gradation
A	0.075	0.2	0.6	8.0	0.9	Well graded (1)
B	1	1.5	2	2.0	1.12	Poorly graded - Gap graded (2)
C	19	25	27	1.4	1.2	Poorly graded
(1) Soil does not meet C_u and C_c criteria for well-graded soil but GSD curve clearly indicates a well-graded soil						
(2) The C_u and C_c parameters indicate a uniform (or poorly) graded material, but the GSD curve clearly indicates a gap-graded soil.						

Note: For clarity only the D_{10}, D_{30}, and D_{60} sizes for Curve A are shown on the figure.

Figure 4-2. Evaluation of type of gradation for coarse-grained soils.

Discussion of Figure 4-2: Curve A in Figure 4-2 has C_u = 8 and C_c = 0.9. The soil represented by Curve A would not meet the criteria listed in Table 4-10 for well-graded soil, but yet an examination of the GSD curve shows that the soil is well-graded. Examination of the GSD curve is even more critical for the case of gap graded soils because the largest particle size evaluated by parameters C_u and C_c is D_{60} while the gap grading may occur at a size larger than D_{60} size as shown for a 2/3:1/3 proportion of gravel: sand mix represented by Curve B in Figure 4-2. Based on the criteria in Table 4-10, the soil represented by Curve B would be classified as a uniform or poorly graded soil which would be an incorrect classification. Such incorrect classifications can and do occur on construction sites where the contractor may (a) simply mix two stockpiles of uniformly graded soils leftover from a previous project. (b) use multiple sand and gravel pits to obtain borrow soils, and/or (c) mix soils from two different seams or layers of poorly graded material in the same gravel pit. Figure 4-2 is an illustration on the importance of evaluating the shape of the GSD curve in addition to the statistical parameters C_u and C_c. Practical aspects of the engineering characteristics of granular soils are discussed in Section 4.4.

4.2.1.2 Classification of Fine-Grained Soils

Fine-grained soils, or “fines,” are those in which 50 percent or more by weight pass the No. 200 (0.075 mm) sieve, The classification of fine-grained soils is accomplished by use of the plasticity chart (Figure 4-3). For fine-grained organic soils, Table 4-11 may be used. Inorganic silts and clays are those that do not meet the organic criteria as given in Table 4-11. The flow charts to determine the group symbol and group name for fine-grained soils are given in Figure 4-4a and 4-4b. These figures are identical to Figures 1a and 1b in ASTM D 2487 except that they are modified to show the soil type capitalized; e.g., CLAY. Dual symbols are used to classify organic silts and clays whose liquid limit and PI plot above the "A"-line, for example, CL-OL instead of OL and CH-OH instead of OH. To describe the fine-grained soil types more fully, plasticity adjectives and soil types used as adjectives should be used to further define the soil type's texture, plasticity, and location on the plasticity chart (see Table 4-12). Examples using Table 4-11 are given in Table 4-12. An example description of fine-grained soils is as follows:

Soft, wet, gray, high plasticity CLAY, with f. Sand; Fat CLAY (CH); (Alluvium)

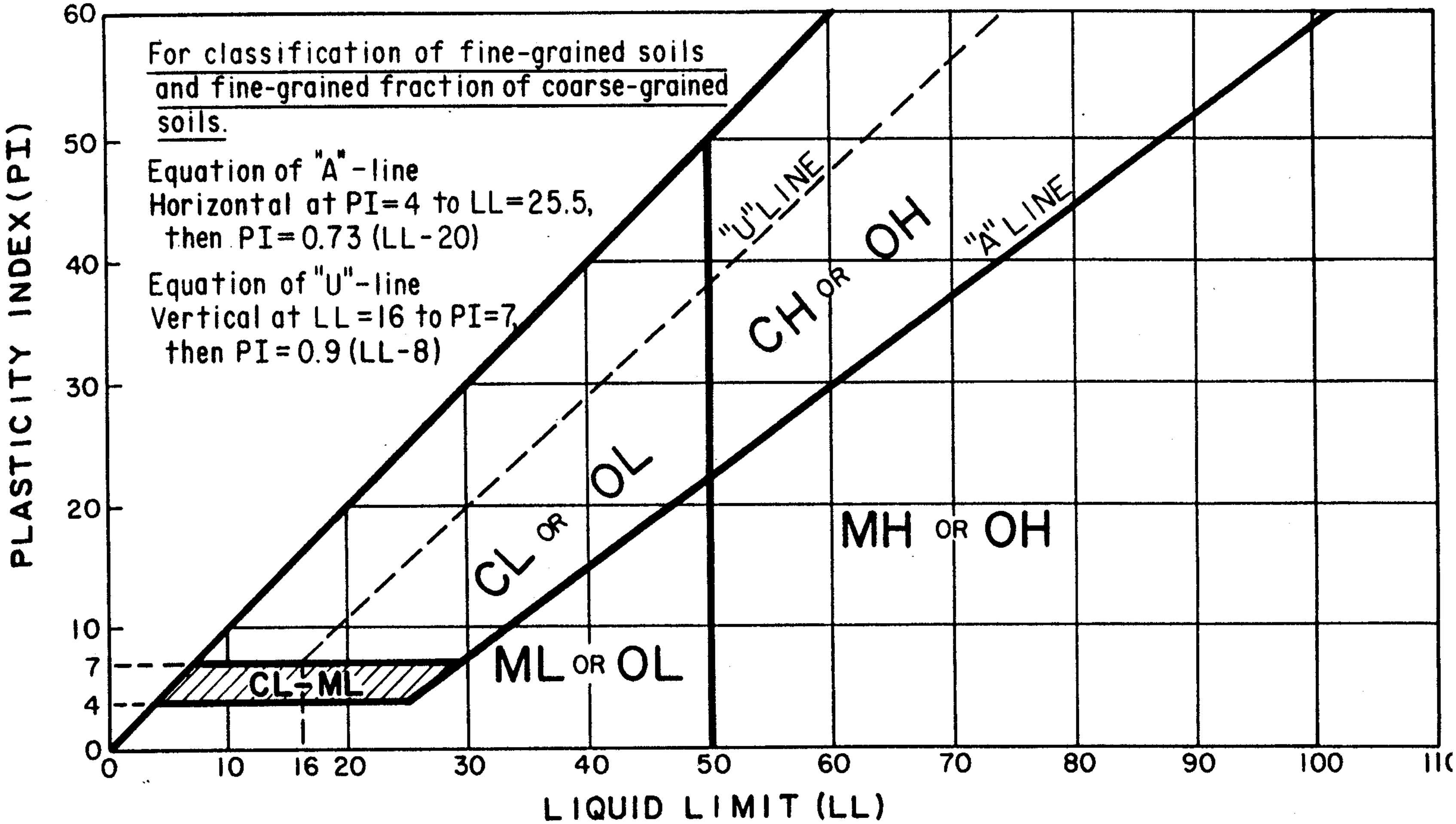


Figure 4-3. Plasticity chart for Unified Soil Classification System (ASTM D 2487).

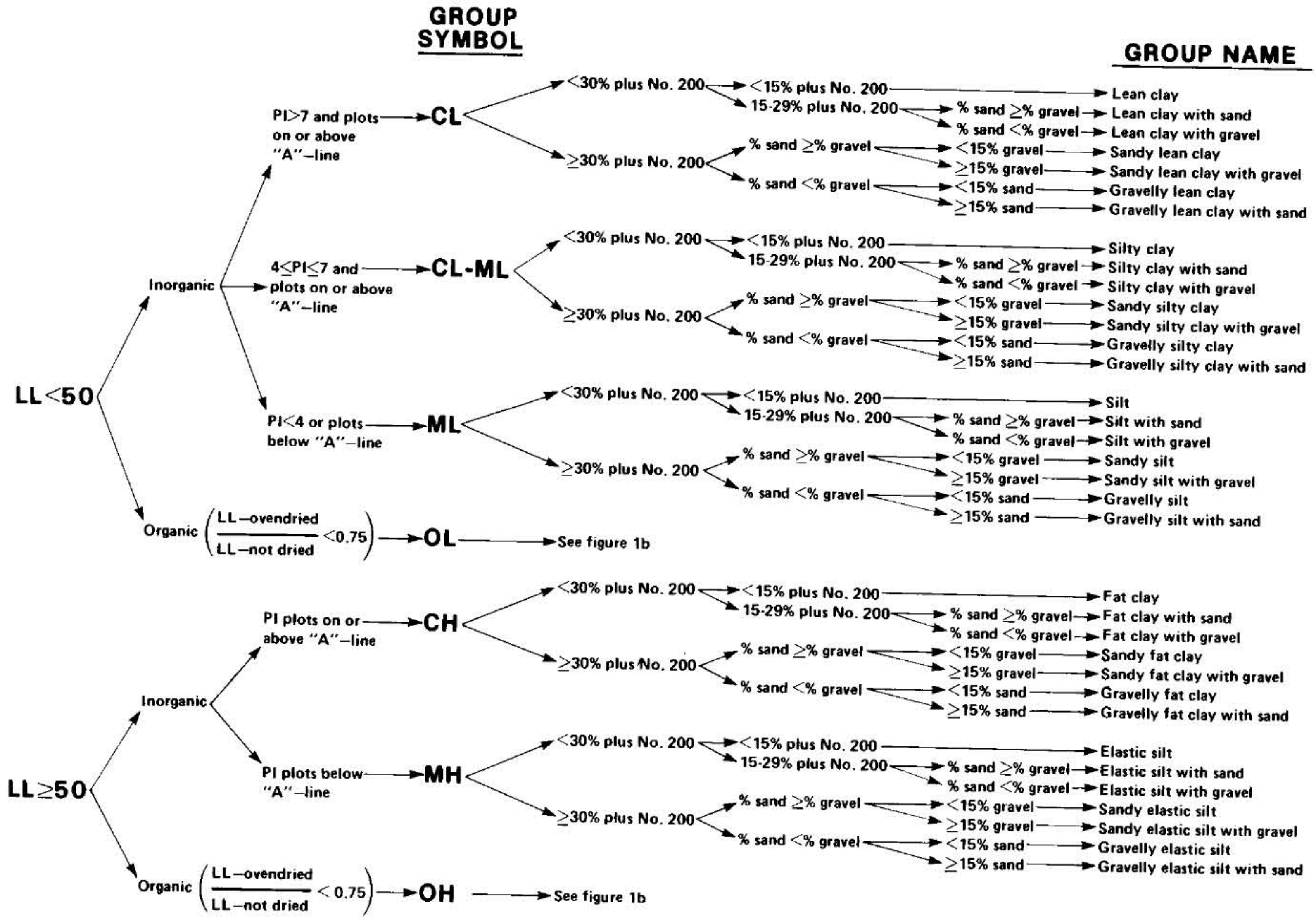


Figure 4-4a. Flow chart to determine the group symbol and group name for fine-grained soils (ASTM D 2487).

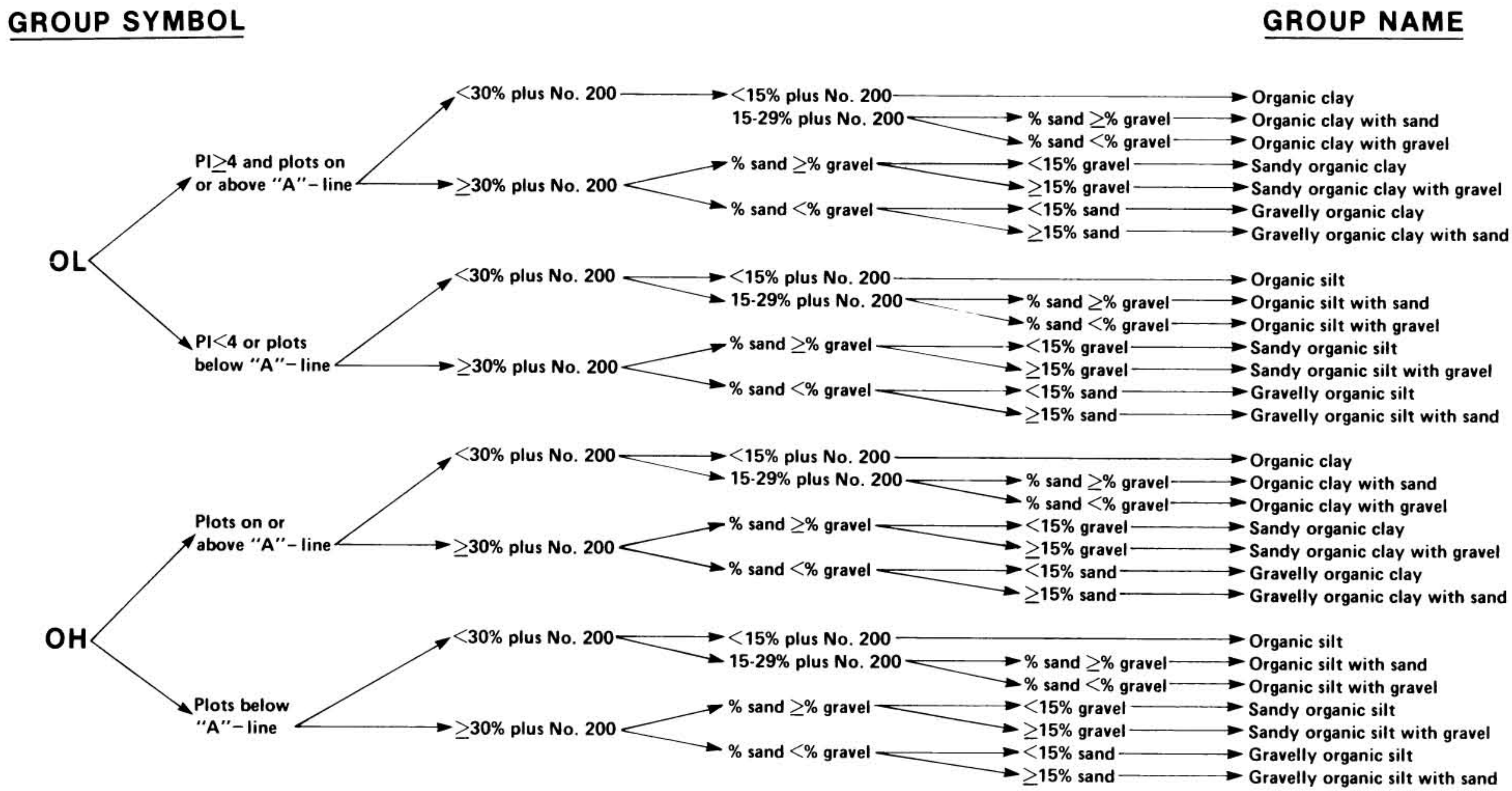


Figure 4-4b. Flow chart to determine the group symbol and group name for organic soils (ASTM D 2487).

Table 4-11
Soil plasticity descriptors (based on Figures 4-3, 4-4a and 4-4b)

Plasticity Index Range	Plasticity Adjective	Adjective for Soil Type, Texture, and Plasticity Chart Location		
		ML & MH (Silt)	CL & CH (Clay)	OL & OH (Organic Silt or Clay)[1]
0	nonplastic	-	-	ORGANIC SILT
1 - 10	low plasticity	-	silty	ORGANIC SILT
>10 - 20	medium plasticity	Clayey	silty to no adj.	ORGANIC clayey SILT
>20 - 40	high plasticity	Clayey	-	ORGANIC silty CLAY
>40	very plastic	Clayey	-	ORGANIC CLAY
Soil type is the same for above or below the "A"-line; the dual group symbol (CL-OL or CH-OH) identifies the soil types above the "A"-line.				

Table 4-12
Examples of description of fine-grained soils (based on Figures 4-3, 4-4a and 4-4b)

Group Symbol	PI	Group Name	Complete Description For Main Soil Type (Fine-Grained Soil)
CL	9	lean CLAY	low plasticity silty CLAY
ML	7	SILT	low plasticity SILT
ML	15	SILT	medium plastic clayey SILT
MH	21	elastic SILT	high plasticity clayey SILT
CH	25	fat CLAY	high plasticity silty CLAY or high plasticity CLAY, depending on smear test (for silty relatively dull and not shiny or just CLAY for shiny, waxy)
OL	8	ORGANIC SILT	low plasticity ORGANIC SILT
OL	19	ORGANIC SILT	medium plastic ORGANIC clayey SILT
CH	>40	fat CLAY	very plastic CLAY

4.2.2 AASHTO Soil Classification System

The AASHTO soil classification system is shown in Table 4-13. The AASHTO classification system is useful in determining the relative quality of the soil material for use in earthwork structures, particularly embankments, subgrades, subbases and bases.

According to this system, soil is classified into seven major groups, A-1 through A-7. Soils classified under groups A-1, A-2 and A-3 are granular materials where 35% or less of the particles pass through the No. 200 sieve (0.075 mm). Soils where more than 35% pass the No. 200 sieve (0.075 mm) are classified under groups A-4, A-5, A-6 and A-7. Soils where more than 35% pass the No. 200 sieve (0.075 mm) are mostly silt and clay-size materials. The classification procedure is shown in Table 4-13. The classification system is based on the following criteria:

i *Grain Size*: The grain size terminology for this classification system is as follows:

- **Gravel**: fraction passing the 3 in (75 mm) sieve and retained on the No. 10 (2 mm) sieve.
- **Sand**: fraction passing the No. 10 (2 mm) sieve and retained on the No. 200 (0.075 mm) sieve
- **Silt and clay**: fraction passing the No. 200 (0.075 mm) sieve

ii *Plasticity*: The term *silty* and *clayey* are used as follows:

- **Silty**: use when the fine fractions of the soil have a plasticity index of 10 or less.
- **Clayey**: use when the fine fractions have a plasticity index of 11 or more.

iii. If cobbles and boulders (size larger than 3 in (75 mm)) are encountered they are excluded from the portion of the soil sample on which the classification is made. However, the percentage of material is recorded.

To evaluate the quality of a soil as a highway subgrade material, a number called the *group index* (GI) is also incorporated along with the groups and subgroups of the soil. The group index is written in parenthesis after the group or subgroup designation. The group index is given by Equation 4-3 where F is the percent passing the No. 200 (0.075 mm) sieve, LL is the liquid limit, and PI is the plasticity index.

$$GI = (F-35)[0.2+0.005(LL-40)] + 0.01(F-15)(PI-10) \qquad 4\text{-}3$$

Table 4-13
AASHTO soil classification system based on AASHTO M 145 (or ASTM D 3282)

GENERAL CLASSIFICATION	**GRANULAR MATERIALS (35 percent or less of total sample passing No. 200 sieve (0.075 mm)**							**SILT-CLAY MATERIALS (More than 35 percent of total sample passing No. 200 sieve (0.075 mm)**			
GROUP CLASSIFICATION	A-1		A-3	A-2				A-4	A-5	A-6	A-7
	A-1-a	A-1-b		A-2-4	A-2-5	A-2-6	A-2-7				A-7-5, A-7-6
Sieve analysis, percent passing:											
No. 10 (2 mm)	50 max.										
No. 40 (0.425 mm)	30 max.	50 max.	51 min.								
No. 200 (0.075 mm)	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 (0.425 mm)											
Liquid limit				40 max.	41 min.	40 max.	41 min.	40 max.	41 min.	40 max.	41 min.
Plasticity index	6 max.		NP	10 max.	10 max.	11 min.	11 min.	10 max.	10 max.	11 min.	11 min.*
Usual significant constituent materials	Stone fragments, gravel and sand		Fine sand	Silty or clayey gravel and sand				Silty soils		Clayey soils	
Group Index**	0		0	0		4 max.		8 max.	12 max.	16 max.	20 max.

Classification procedure:
With required test data available, proceed from left to right on chart; correct group will be found by process of elimination. The first group from left into which the test data will fit is the correct classification.

*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 (see Fig 4-5).

**See group index formula (Eq. 4-3). Group index should be shown in parentheses after group symbol as: A-2-6(3), A-4(5), A-7-5(17), etc.

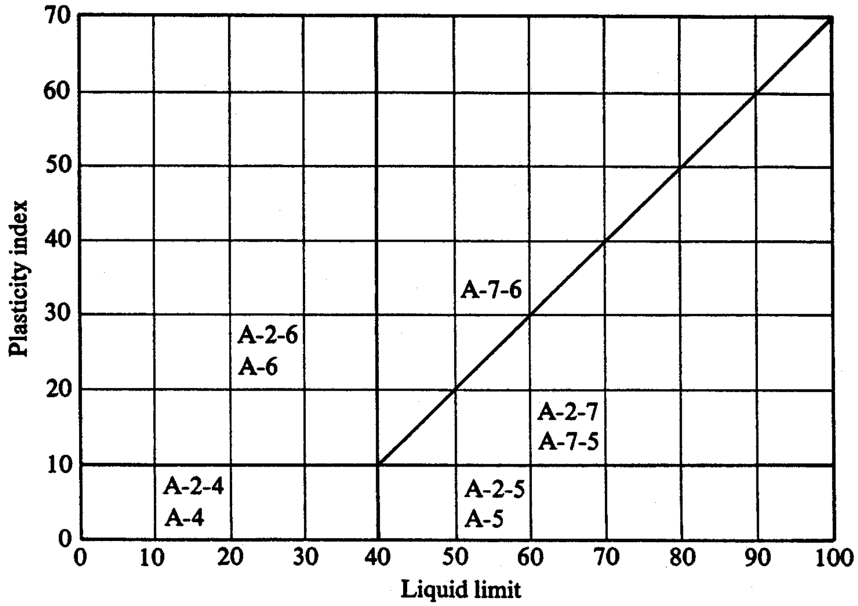


Figure 4-5. Range of liquid limit and plasticity index for soils in groups A-2, A-4, A-5, A-6 and A-7 per AASHTO M 145 (or ASTM D 3282).

The first term of Equation 4-3 is the partial group index determined from the liquid limit. The second term is the partial group index determined from the plasticity index. Following are some rules for determining group index:

- If Equation 4-3 yields a negative value for GI, it is taken as zero.
- The group index calculated from Equation 4-3 is rounded off to the nearest whole number, e.g., GI=3.4 is rounded off to 3; GI=3.5 is rounded off to 4.
- There is no upper limit for the group index.
- The group index of soils belonging to groups A-1-a, A-1-b, A-2-4, A-2-5, and A-3 will always be zero.
- When the group index for soils belonging to groups A-2-6 and A-2-7 is calculated, the partial group index for PI should be used, or

$$GI=0.01(F-15)(PI-10) \qquad 4\text{-}4$$

In general, the quality of performance of a soil as a subgrade material is inversely proportional to the group index.

A comparison of the USCS and AASHTO system is shown in Figures 4-6 and 4-7.

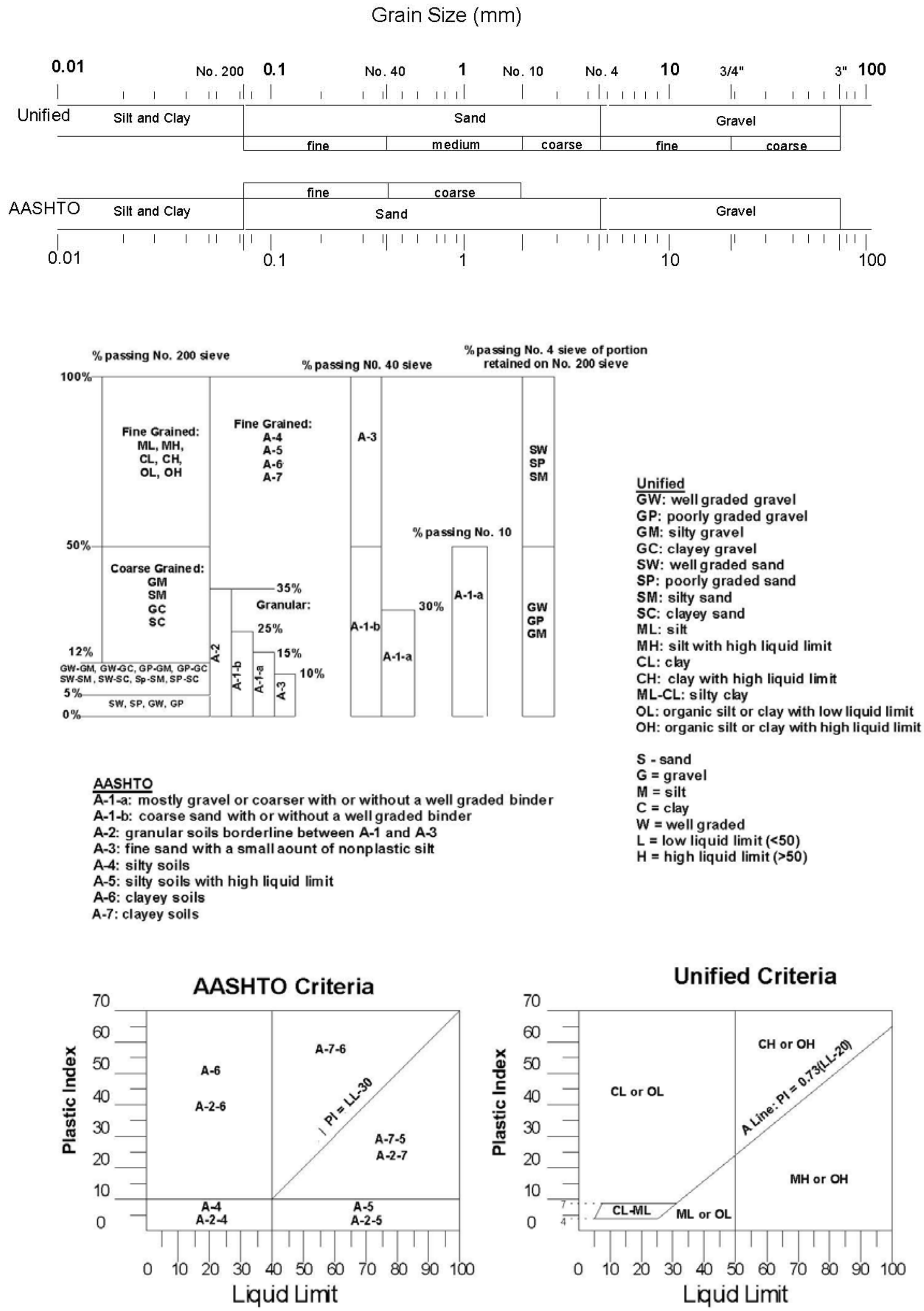


Figure 4-6. Comparison of the USCS with the AASHTO soil classification system (after Utah DOT – Pavement Design and Management Manual, 2005).

Soil Group in Unified System	Comparable Soil Groups in AASHTO System		
	Most Probable	Possible	Possible but Improbable
GW	A-1-a	—	A-2-4, A-2-5, A-2-6, A-2-7
GP	A-1-a	A-1-b	A-3, A-2-4, A-2-5, A-2-6, A-2-7
GM	A-1-b, A-2-4, A-2-5, A-2-7	A-2-6	A-4, A-5, A-6, A-7-5, A-7-6, A-1-a
GC	A-2-6, A-2-7	A-2-4, A-6	A-4, A-7-6, A-7-5
SW	A-1-b	A-1-a	A-3, A-2-4, A-2-5, A-2-6, A-2-7
SP	A-3, A-1-b	A-1-a	A-2-4, A-2-5, A-2-6, A-2-7
SM	A-1-b, A-2-4, A-2-5, A-2-7	A-2-6, A-4, A-5	A-6, A-7-5, A-7-6, A-1-a
SC	A-2-6, A-2-7	A-2-4, A-6, A-4, A-7-6	A-7-5
ML	A-4, A-5	A-6, A-7-5	—
CL	A-6, A-7-6	A-4	—
OL	A-4, A-5	A-6, A-7-5, A-7-6	—
MH	A-7-5, A-5	—	A-7-6
CH	A-7-6	A-7-5	—
OH	A-7-5, A-5	—	A-7-6
Pt	—	—	—

Soil Group in AASHTO System	Comparable Soil Groups in Unified System		
	Most Probable	Possible	Possible but Improbable
A-1-a	GW, GP	SW, SP	GM, SM
A-1-b	SW, SP, GM, SM	GP	—
A-3	SP	—	SW, GP
A-2-4	GM, SM	GC, SC	GW, GP SW, SP
A-2-5	GM, SM	—	GW, GP, SW, SP
A-2-6	GC, SC	GM, SM	GW, GP SW, SP
A-2-7	GM, GC, SM, SC	—	GW, GP, SW, SP
A-4	ML, OL	CL, SM, SC	GM, GC
A-5	OH, MH, ML, OL	—	SM, GM
A-6	CL	ML, OL, SC	GC, GM, SM
A-7-5	OH, MH	ML, OL, CH	GM, SM, GC, SC
A-7-6	CH, CL	ML, OL, SC	OH, MH, GC, GM, SM

Figure 4-7. Comparison of soil groups in the USCS with the AASHTO Soil Classification Systems (Holtz and Kovacs, 1981).

4.3 ENGINEERING CHARACTERISTICS OF SOILS

The major engineering characteristics of the main soil groups discussed in the previous section as related to foundation design are summarized as follows. A discussion on the practical aspects of the engineering characteristics is presented for granular and fine-grained soils following these summaries.

4.3.1 Engineering Characteristics of Coarse-Grained Soils (Sands and Gravels)

- Generally very good foundation material for supporting structures and roads.
- Generally very good embankment material.
- Generally the best backfill material for retaining walls.
- Might settle under vibratory loads or blasts.
- Dewatering may be difficult in open-graded gravels due to high permeability.
- Generally not frost susceptible.

4.3.2 Engineering Characteristics of Fine-Grained Soils (Inorganic Clays)

- Generally possess low shear strength.
- Plastic and compressible.
- Can lose part of shear strength upon wetting.
- Can lose part of shear strength upon disturbance.
- Can shrink upon drying and expand upon wetting.
- Generally very poor material for backfill.
- Generally poor material for embankments.
- Can be practically impervious.
- Clay slopes are prone to landslides.

4.3.3 Engineering Characteristics of Fine-Grained Soils (Inorganic Silts)

- Relatively low shear strength.
- High capillarity and frost susceptibility.
- Relatively low permeability.
- Frost heaving susceptibility
- Difficult to compact.

4.3.4 Engineering Characteristics of Organic Soils

The term organic designates those soils, other than topsoil, that contain an appreciable amount of vegetative matter and occasionally animal organisms in various states of decomposition. Any soil containing a sufficient amount of organic matter to influence its engineering properties is called an organic soil. The organic matter is objectionable for three main reasons:

1. Reduces load carrying capacity of soil.
2. Increases compressibility considerably.
3. Frequently contains toxic gasses that are released during the excavation process.

Generally organic soils, whether peat, organic clays, organic silts, or even organic sands, are not used as construction materials.

4.4 PRACTICAL ASPECTS OF ENGINEERING CHARACTERISTICS OF COARSE-GRAINED SOILS

Grain size distribution is the single most important element in the design of structures on, in, or composed of granular soils. As discussed in Chapter 2, grain size distribution is determined by sieving a dried soil sample of known weight through a nest of U.S. Standard sieves with decreasing mesh opening sizes. Figures 2-3 and 4-2 presented sample grain size distribution curves, also known as gradation curves, and introduced the terminology "well graded," "poorly graded," and "gap graded."

Much can be learned about a soil's behavior from the shape and location of the curve. For instance, the "well graded" curve shown in Figure 4-2 represents a non-uniform soil with a wide range of particle sizes that are evenly distributed. Densification of a well-graded soil causes the smaller particles to move into the voids between the larger particles. As the voids in the soil are reduced, the density and strength of the soil increase. Specifications for select structural fill should contain required ranges of different particle sizes so that a dense, non-compressible backfill can be achieved with reasonable compactive effort. For example, the well-graded soil represented by Curve A shown in Figure 4-2 could be specified by providing the gradation limits listed in Table 4-14.

As shown by Curve C in Figure 4-2, a poorly graded or uniform soil is composed of a narrow range of particle sizes. When compaction is attempted, inadequate distribution of particle sizes prevents reduction of the volume of voids by infilling with smaller particles. Such uniform soils should be avoided as select fill material. However, uniform soils do have an important use as drainage materials. The relatively large and permanent void spaces act as conduits to carry water. Obviously, the larger the average particle size the larger the void space. The "French drain" is an example of the engineering use of a coarse uniform soil. Table 4-15 presents a typical specification for drainage materials having a narrow band of particle sizes. For material specifications related to drain material, it is important to specify that gap-graded materials shall not be acceptable. This is because gap-graded materials have variable permeabilities that may cause malfunction of the drain with associated damage to the geotechnical feature associated with the drain.

Table 4-14
Example gradation limits of well-graded granular material
(see Curve A in Figure 4-2)

Sieve Size	Percent Passing by Weight
2″ (50.8 mm)	100
#10 (2 mm)	75-90
#40 (0.425 mm)	40-60
#200 (0.075 mm)	0 – 15

Table 4-15
Example gradation limits of drainage materials
(see Curve C in Figure 4-2)

Sieve Size	Percent Passing by Weight
2″ (50.8 mm)	100
1 ½ ″ (37.5 mm)	90-100
¾ ″ (19 mm)	0-15

4.5 PRACTICAL ASPECTS OF ENGINEERING CHARACTERISITICS OF FINE-GRAINED SOILS

As indicated in Chapter 2, the plasticity index (PI) is the difference between the liquid limit (LL) and the plastic limit (PL). The PI represents the range of water content over which the soil remains plastic. In general, the greater the PI, the greater the amount of clay particles present and the more plastic the soil. The more plastic a soil, the more likely it will be to have the following characteristics:

1. Be more compressible.
2. Have greater potential to shrink upon drying and/or swell upon wetting.
3. Be less permeable.

In addition to the PI, the Liquidity Index (LI) is a useful indicator of the engineering characteristics of fine-grained soils. Table 2-4 in Chapter 2 identifies the strength and deformation characteristics of fine-grained soils in terms of the LI.

4.6 DESCRIPTION OF ROCK

When providing rock descriptions, geotechnical specialists should use technically correct geological terms. Local terms in common use may be acceptable if they help describe distinctive characteristics. Rock cores should be logged when wet for consistency of color description and greater visibility of rock features such as hairline fractures. The guidelines presented in the ISRM (1981), should be reviewed for additional information regarding logging procedures for core drilling.

The rock's lithologic description should include as a minimum the following items:

- Rock type
- Color
- Grain size and shape
- Texture (stratification/foliation)
- Mineral composition
- Weathering and alteration
- Strength
- Other relevant notes

The various elements of the rock's description should be stated in the order listed above, for example:

"Limestone, light gray, very fine-grained, thin-bedded, unweathered, strong"

The rock description should include identification of discontinuities and fractures. The description should also include a drawing of the naturally occurring fractures and mechanical breaks.

4.6.1 Rock Type

Rocks are classified according to their origin into three major divisions: igneous, sedimentary, and metamorphic (see Table 4-16). These three groups are subdivided into types according to mineral and chemical composition, texture, and internal structure. For some projects a library of hand samples and photographs representing lithologic rock types present in the project area should be maintained.

Table 4-16
Rock groups and types (FHWA, 1997)

Igneous		
Intrusive (Coarse Grained)	Extrusive (Fine Grained)	Pyroclastic
Granite Syenite Diorite Diabase Gabbro Peridotite Pegmatite	Rhyolite Trachyte Andesite Basalt	Obsidian Pumice Tuff
Sedimentary		
Clastic (Sediment)	Chemically Formed	Organic Remains
Shale Mudstone Claystone Siltstone Sandstone Conglomerate Limestone, oolitic	Limestone Dolomite Gypsum Halite	Chalk Coquina Lignite Coal

Metamorphic	
Foliated	Non-foliated
Slate Phyllite Schist Gneiss	Quartzite Amphibolite Marble Hornfel

4.6.2 Color

Colors should be consistent with a Munsell Color Chart (USDA, 1993) and recorded for both wet and dry conditions as appropriate.

4.6.3 Grain Size and Shape

The grain size description should be classified according to the terms presented in Table 4-17. Table 4-18 is used to classify the shape of the grains. The grain size descriptions are consistent with those used in the USCS for soil particles.

Table 4-17
Terms to describe grain size (typically for sedimentary rocks)

Description	Grain Size (mm)	Characteristic of Individual Grains
Very coarse grained	#4 (> 4.75)	Can be easily distinguished by eye
Coarse grained	#10 to #4 (2.00 -4.75)	Can be easily distinguished by eye
Medium grained	#40 to #10 (0.425 -2.00)	Can be distinguished by eye
Fine grained	#200 to #40 (0.075-0.425)	Can be distinguished by eye with difficulty
Very fine grained	< #200 (< 0.075)	Cannot be distinguished by unaided eye

Table 4-18
Terms to describe grain shape (for sedimentary rocks)

Description	Characteristic
Angular	Showing very little evidence of wear. Grain edges and corners are sharp. Secondary corners are numerous and sharp.
Subangular	Showing some evidence of wear. Grain edges and corners are slightly rounded off. Secondary corners are slightly less numerous and slightly less sharp than in angular grains.
Subrounded	Showing considerable wear. Grain edges and corners are rounded to smooth curves. Secondary corners are reduced greatly in number and highly rounded.
Rounded	Showing extreme wear. Grain edges and corners are smoothed off to broad curves. Secondary corners are few in number and rounded.
Well-rounded	Completely worn. Grain edges or corners are not present. No secondary edges or corners are present.

4.6.4 Stratification/Foliation

Significant non-fracture structural features should be described. The thickness should be described by using the terms in Table 4-19. The orientation of the bedding/foliation should be measured from the horizontal or with respect to the core axis.

Table 4-19. Terms to describe stratum thickness

Descriptive Term	Stratum Thickness in (mm)*
Very Thickly bedded	(> 1 m)
Thickly bedded	(0.5 to 1.0 m)
Thinly bedded	(50 mm to 500 mm)
Very Thinly bedded	(10 mm to 50 mm)
Laminated	(2.5 mm to 10 mm)
Thinly Laminated	(< 2.5 mm)
* Conventionally measured in m or mm. (1 m = 3.28 ft; 25.4 mm = 1 in)	

4.6.5 Mineral Composition

The mineral composition should be identified by a geologist based on experience and the use of appropriate references. The most abundant mineral should be listed first, followed by minerals in decreasing order of abundance. For some common rock types, the mineral composition need not be specified (e.g. dolomite, limestone).

4.6.6 Weathering and Alteration

Weathering and alteration is due to the weathering processes discussed in Chapter 3, e.g., physical, chemical and thermal mechanisms. Terms and abbreviations used to describe weathering and alteration are presented in Table 4-20.

4.6.7 Strength

The point load test described in Chapter 5 is recommended for the measurement of sample strength. The point-load index, I_s, obtained from the point load test should be converted to uniaxial compressive strength. Categories and terminology for describing rock strength based on the uniaxial compressive strength are presented in Table 4-21. Table 4-21 also presents guidelines for common qualitative assessments of strength that can be performed with the aid of a geologist's hammer and a pocket knife while the geotechnical specialist is mapping or doing primary logging of core at the drill rig site. The field estimates should be confirmed where appropriate by comparison with selected laboratory tests.

Table 4-20
Terms to describe rock weathering and alteration (ISRM, 1981)

Grade (Term)	Description
I (Fresh)	Rock shows no discoloration, loss of strength, or other effects of weathering/alteration
II (Slightly Weathered/Altered)	Rock is slightly discolored, but not noticeably lower in strength than fresh rock
III (Moderately Weathered/Altered)	Rock is discolored and noticeably weakened, but less than half is decomposed; a minimum 2 in (50 mm) diameter sample cannot be broken readily by hand across the rock fabric
IV (Highly Weathered/Altered)	More than half of the rock is decomposed; rock is weathered so that a minimum 2 in (50 mm) diameter sample can be broken readily by hand across the rock fabric
V (Completely Weathered/Altered)	Original minerals of rock have been almost entirely decomposed to secondary minerals even though the original fabric may be intact; material can be granulated by hand
VI (Residual Soil)	Original minerals of rock have been entirely decomposed to secondary minerals, and original rock fabric is not apparent; material can be easily broke by hand

Table 4-21
Terms to describe rock strength (ISRM, 1981)

Grade (Description)	Field Identification	Approximate Range of Uniaxial Compressive Strength, psi (kPa)
R0 (Extremely Weak Rock)	Can be indented by thumbnail	35 (250) - 150 (1,000)
R1 (Very Weak Rock)	Can be peeled by pocket knife	150 (1,000) - 725 (5,000)
R2 (Weak Rock)	Can be peeled with difficulty by pocket knife	725 (5,000) - 3,500 (25,000)
R3 (Medium Strong Rock)	Can be indented 3/16 in (5 mm) with sharp end of pick	3,500 (25,000) - 7,000 (50,000)
R4 (Strong Rock)	Requires one blow of geologist's hammer to fracture	7,000 (50,000) - 15,000 (100,000)
R5 (Very Strong Rock)	Requires many blows of geologist's hammer to fracture	15,000 (100,000) - 36,000 (250,000)
R6 (Extremely Strong Rock)	Can only be chipped with blows of geologist's hammer	> 36,000 (>250,000)

4.6.8 Hardness

Hardness is commonly assessed by the scratch test. Descriptions and abbreviations used to describe rock hardness are presented in Table 4-22.

Table 4-22
Terms to describe rock hardness (FHWA, 2002b)

Description (Abbr)	Characteristic
Soft (S)	Reserved for plastic material alone.
Friable (F)	Easily crumbled by hand, pulverized or reduced to powder.
Low Hardness (LH)	Can be gouged deeply or carved with a pocket knife.
Moderately Hard (MH)	Can be readily scratched by a knife blade; scratch leaves a heavy trace of dust and scratch is readily visible after the powder has been blown away.
Hard (H)	Can be scratched with difficulty; scratch produces little powder and is often faintly visible; traces of the knife steel may be visible.
Very Hard (VH)	Cannot be scratched with pocket knife. Leave knife steel marks on surface.

4.6.9 Rock Discontinuity

Discontinuity is the general term for any mechanical break in a rock mass that has zero or low tensile strength. Discontinuity is the collective term used for most types of joints, weak bedding planes, weak schistosity planes, weakness zones, and faults. The spacing between discontinuities is defined as the perpendicular distance between adjacent discontinuities. The spacing should be measured perpendicular to the planes in the set. Table 4-23 presents guidelines to describe discontinuity spacing.

Discontinuities should be described as closed, open, or filled. **Aperture** is the term used to describe the perpendicular distance separating the adjacent rock walls of an open discontinuity in which the intervening space is air- or water-filled. **Width** is the term used to describe the distance separating the adjacent rock walls of filled discontinuities. The terms presented in Table 4-24 should be used to describe apertures. Terms such as "wide," "narrow" and "tight" are used to describe the width of discontinuities such as thickness of veins, fault gouge filling, or joints openings. Guidelines for use of such terms are presented in Tables 4-23 and 4-24.

Table 4-23. Terms to describe discontinuities (after ISRM, 1981)

Discontinuity Type		
F	-	Fault
J	-	Joint
Sh	-	Shear
Fo	-	Foliation
V	-	Vein
B	-	Bedding

Amount of Infilling		
Su	-	Surface Stain
Sp	-	Spotty
Pa	-	Partially Filled
Fi	-	Filled
No	-	None

Discontinuity Spacing (m)*		
EW	-	Extremely Wide (>6)
VW	-	Very Wide (2-6)
W	-	Wide (0.6-2)
M	-	Moderate (0.2-0.6)
C	-	Close (0.06-0.2)
VC	-	Very Close (0.02-0.06)
EC	-	Extremely close (<0.02)

Discontinuity Width (mm)*		
W	-	Wide (12.5-5.0)
MW	-	Moderately Wide (2.5-12.5)
N	-	Narrow (1.25-2.5)
VN	-	Very Narrow (<1.25)
T	-	Tight (~ 0)

Surface Shape of Joint		
Wa	-	Wavy
Pl	-	Planar
St	-	Stepped
Ir	-	Irregular

Type of Infilling		
Cl	-	Clay
Ca	-	Calcite
Ch	-	Chlorite
Fe	-	Iron Oxide
Gy	-	Gypsum/Talc
H	-	Healed
No	-	None
Py	-	Pyrite
Qz	-	Quartz
Sd	-	Sand

Roughness of Surface		
Slk	-	Slickensided (surface has smooth, glassy finish with visual evidence of striations)
S	-	Smooth (surface appears smooth and feels so to the touch)
SR	-	Slightly Rough (asperities on the discontinuity surface are distinguishable and can felt)
R	-	Rough (some ridges and side-angle steps are evident; asperities are clearly visible, and discontinuity surface feels very abrasive)
V R	-	Very Rough (near-vertical steps and ridges occur on the discontinuity surface

* Conventionally measured in m or mm. (1 m = 3.28 ft; 1 in = 25.4 mm)

Table 4-24. Terms to classify discontinuities based on aperture size (ISRM, 1981)

Aperture (mm)*	Description	
<0.1 0.1 - 0.25 0.25 – 0.5	Very tight Tight Partly open	"Closed Features"
0.5 - 2.5 2.5 - 10 > 10	Open Moderately open Wide	"Gapped Features"
1-100 100-1000 >1 m	Very wide Extremely wide Cavernous	"Open Features"

* Conventionally measured in mm, cm or m. (1 m = 3.28 ft; 1 in = 25.4 mm)

For faults or shears that are not thick enough to be represented on the boring log, the measured thickness is recorded numerically in millimeters.

Discontinuities are further characterized by the surface shape of the joint and the roughness of its surface in addition to the fill material separating the adjacent rock walls of the discontinuities. Filling is characterized by its type, amount, width (i.e., perpendicular distance between adjacent rock walls) and strength. If non-cohesive fillings are identified, then the filling should be identified qualitatively, e.g., fine sand. Refer to Table 4-23 for guidelines to characterize these features.

4.6.10 Fracture Description

Naturally occurring fractures are numbered and described by using the same terminology that is used for discontinuities. The number of naturally occurring fractures observed in each 1 ft (0.5 m) of core should be recorded as the fracture frequency. Mechanical breaks, thought to have occurred during drilling, are not counted. The following criteria can be used to identify natural breaks:

1. A rough brittle surface with fresh cleavage planes in individual rock minerals suggests an artificial fracture.

2. A generally smooth or somewhat weathered surface with soft coating or infilling materials, such as talc, gypsum, chlorite, mica, or calcite indicates a natural discontinuity.

3. In rocks showing foliation, cleavage or bedding it may be difficult to distinguish between natural discontinuities and artificial fractures when the discontinuities are parallel with the incipient weakness planes. If drilling has been carried out carefully then the questionable breaks should be counted as natural features to be on the conservative side.

4. Depending upon the drilling equipment, part of the length of core being drilled may occasionally rotate with the inner barrels in such a way that grinding of the surfaces of discontinuities and fractures occurs. In weak rock types it may be very difficult to decide if the resulting rounded surfaces represent natural or artificial features. When in doubt, conservatively assume that they are natural.

The fracture description can be strongly time dependent and moisture content dependent in the case of certain varieties of shales and mudstones that have relatively weakly developed diagenetic bonds. A diagenetic bond is the bond that is formed in a deposited sediment by chemical and physical processes during its conversion to rock. A frequent problem is "discing," in which an initially intact core separates into discs on incipient planes. The process generally becomes noticeable perhaps within a few minutes of core recovery. This phenomenon is experienced in several different forms:

1. Stress relief cracking and swelling by the initially rapid release of strain energy in cores recovered from areas of high stress, especially in the case of shaley rocks.

2. Dehydration cracking experienced in the weaker mudstones and shales that may reduce RQD values from 100 percent to 0 percent in a matter of minutes. The initial integrity might possibly have been due to negative pore water pressure.

3. Slaking and cracking experienced by some of the weaker mudstones and shales when they are subjected to wetting and drying.

Any of these forms of "discing" may make logging of fracture frequency unreliable. Whenever such conditions are anticipated, core should be logged by a geotechnical specialist as it is being recovered and at subsequent intervals until the phenomenon is predictable.

4.6.11 Rock Mass Classification

In determining the rock strength for transportation facilities constructed in, on, or of rock, it is most important to account for the presence of discontinuities, such as joints, faults or bedding planes. Therefore, for most conditions, the **rock mass** strength properties, rather than the intact rock properties must be determined for use in design. The rock mass is the in-situ, fractured rock that will almost always have significantly lower strength than the intact rock because of discontinuities that divide the rock mass into blocks. Therefore, the strength of the rock mass will depend on such factors as the shear strength of the surfaces of the blocks, the spacing and continuous length of the discontinuities and their alignment relative to the direction of loading. These factors were identified in the previous sections. Using these factors, Bieniawski (1989) proposed a method for estimating rock mass properties from an index that characterizes the overall properties of the rock mass quality. This index is known as the **rock mass rating (RMR)**. Originally developed for tunnel support design, the RMR has been adopted by AASHTO (2004 with 2006 Interims) because the RMR is determined from readily measurable parameters. Table 4-25 identifies the following five measurable parameters and assigns relative ratings to each parameter:

Table 4-25
Geomechanics classification of rock masses (AASHTO 2004 with 2006 Interims)

	PARAMETER		RANGES OF VALUES						
1	Strength of intact rock material	Point load strength index	>1,200 psi	600 to 1,200 psi	300 to 600 psi	150 to 300 psi	For this low range – uniaxial compressive test is preferred		
		Uniaxial compressive strength	>30,000 psi	15,000 to 30,000 psi	7,500 to 15,000 psi	3,600 to 7,500 psi	1,500 to 3,600 psi	500 to 1,500 psi	150 to 500 psi
	Relative Rating		15	12	7	4	2	1	0
2	Drill core quality RQD		90% to 100%	75% to 90%	50% to 75%	25% to 50%	<25%		
	Relative Rating		20	17	13	8	3		
3	Spacing of joints		>10 ft	3 to 10 ft	1 to 3 ft	2 in. to 1 foot	<2 in.		
	Relative Rating		30	25	20	10	5		
4	Condition of joints		• Very rough surfaces • Not continuous • No separation • Hard joint wall rock	• Slightly rough surfaces • Separation <0.05” • Hard joint wall rock	• Slightly rough surfaces • Separation <0.05” • Soft joint wall rock	• Slickensided surfaces - or - • Gouge <0.2 in thick – or- • Joints open 0.05-0.2” • Continuous joints	• Soft gouge >0.2” thick - or - • Joints open >0.2” • Continuous joints		
	Relative Rating		25	20	12	6	0		
5	Ground water conditions (use one of the three evaluation criteria as appropriate to the method of exploration)	Inflow per 30 ft tunnel length	None	<400 gallons/hr	400 to 2,000 gallons/hr	>2,000 gallons/hr			
		Ratio= joint water pressure/ major principal stress	0	0.0 to 0.2	0.2 to 0.5	>0.5			
		General Conditions	Completely Dry	Moist only (interstitial water)	Water under moderate pressure	Severe water problems			
	Relative Rating		10	7	4	0			

Note: 1 psi = 6.895 kPa; 1 in = 25.4 mm

1. Strength of intact rock material.
2. Drill core quality as expressed by RQD.
3. Spacing of joints.
4. Condition of joints.
5. Ground water conditions.

The RMR is determined as the sum of the five relative ratings. The RMR should be adjusted in accordance with the criteria in Table 4-26. **The rock classification should be determined in accordance with Table 4-27 where RMR refers to the adjusted value**.

Table 4-26
Geomechanics rating adjustment for joint orientations
(after AASHTO 2004 with 2006 Interims)

Orientations of joints		Very favorable	Favorable	Fair	Unfavorable	Very Unfavorable
Ratings	Tunnels	0	-2	-5	-10	-12
	Foundations	0	-2	-7	-15	-25
	Slopes	0	-5	-25	-50	-60

Table 4-27
Geomechanics rock mass classes determined from total ratings
(AASHTO 2004 with 2006 Interims)

RMR (Note 1)	100 to 81	80 to 61	60 to 41	40 to 21	<20
Class No.	I	II	III	IV	V
Description	Very good rock	Good rock	Fair rock	Poor rock	Very poor rock
Note 1:RMR is adjusted for structural application and rock joint orientation as per Table 4-26 prior to evaluating the Class No.					

4.7 SUBSURFACE PROFILE DEVELOPMENT

The mark of successfully accomplishing a subsurface exploration is the ability to draw a subsurface profile of the project site complete with soil types, rock interfaces, and the relevant design properties. The subsurface profile is a visual display of subsurface conditions as interpreted from all of the methods of explorations and testing described previously. Uncertainties in the development of a subsurface exploration usually indicate the need for additional explorations or testing. Because of the diverse nature of the geologic processes that contribute to soil formation, actual subsurface profiles can be extremely varied both vertically and horizontally, and can differ significantly from interpreted profiles developed from boring logs. Therefore, subsurface profiles developed from boring logs should contain some indication that the delineation between strata do not necessarily suggest that distinct boundaries exist between the strata or that the interpolations of strata thickness between borings are necessarily correct. The main purpose of subsurface profiles is to provide a starting point for design and not necessarily to present an accurate description of subsurface conditions.

In the optimum situation, the subsurface profile is developed in stages. First, a rough profile is established from the driller's logs by the geotechnical specialist. The object is to discover any obvious gaps or question marks while the drill crew is still at the site so that additional work can be performed immediately. Once a crew has left the site, a delay of months may occur before their schedule permits them to reoccupy the site, not to mention the additional cost to remobilize/demobilize. The drilling inspector or crew chief should be required to call the project geotechnical specialist when the last scheduled boring has begun to request instructions for any supplemental borings.

When all borings are completed and laboratory visuals and moisture content data received, the initial subsurface profile should be revised. Estimated soil layer boundaries and accurate soil descriptions should be established for soil deposits. Estimated bedrock interfaces should be identified. Most importantly, the depth to perched or regional groundwater should be indicated. The over-complication of the profile by noting minute variations between adjacent soil samples can be avoided by:

1. Reviewing the geologic history of the site, e.g., if the soil map denotes a lakebed deposit overlying a glacial till deposit, do not subdivide the lakebed deposit because adjacent samples have differing amounts of silt and clay. Realize before breaking down the soil profile that probably only two layers exist and variations are to be expected within each. Important variations such as the average thickness of silt and clay varves can be noted adjacent to the visual description of the layer.

2. Remembering that the soil samples examined are only a minute portion of the soil underlying the site and must be considered in relation to adjacent samples as well as adjacent borings.

A few simple rules should be followed at this stage to interpret the available data properly:

1. Review the USDA Soil Survey map for the county and determine major surface and near-surface deposits that can be expected at the site.

2. Examine the subsurface log containing SPT results and the laboratory visual descriptions with accompanying moisture contents.

3. Review representative soil samples to check laboratory identifications and to calibrate your interpretations with those of the laboratory technicians who performed the visual description.

4. Establish rational mechanics for drawing the soil profile. For example:

    a. Use a vertical scale of 1 in equals 10 ft or 20 ft; generally, any smaller scale tends to compress data visually and prevent proper interpretation.

    b. Use a horizontal scale equal to the vertical scale, if possible, to simulate actual relationships. However, the total length should be kept within 36 inches (920 millimeter) to permit review in a single glance.

When the subsurface layer boundaries and descriptions have been established, determine the extent and details of laboratory testing. Do not casually read the driller's log and randomly select certain samples for testing. Plan the test program intelligently from the subsurface profile and for the proposed feature. Identify major soil deposits and assign appropriate tests for the design project under investigation.

The final subsurface profile is the geotechnical specialist's best interpretation of all available subsurface data. The final subsurface profile should include the following:

- interpreted boundaries of soil and rock
- the average physical properties of the soil layers, e.g., unit weight, shear strength, etc.
- a visual description of each layer including USCS symbols for soil classification
- location of the ground water level, and

* notations for special items such as boulders, artesian pressure, etc.

If the inclusion of all of the information listed above clutters the subsurface profile, then complementary tables containing some of that information should be developed to accompany the profile. Figures 4-8 and 4-9 show a typical boring location plan and an interpreted subsurface profile. Note that **the interpreted boundaries of rock and groundwater profiles are for internal agency use. Such interpretations should not be presented in bid documents.** Another example of boring location plan and subsurface profile is presented in Chapter 11 (Geotechnical Reports).

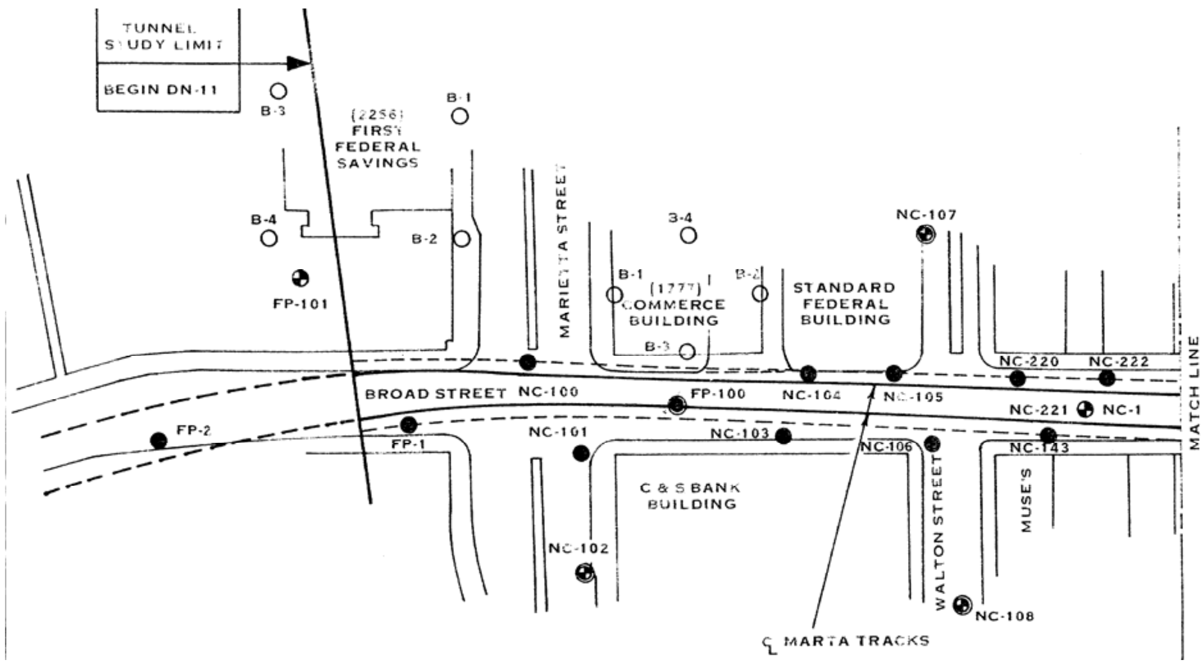


Figure 4-8. Example boring location plan (FHWA, 2002a).

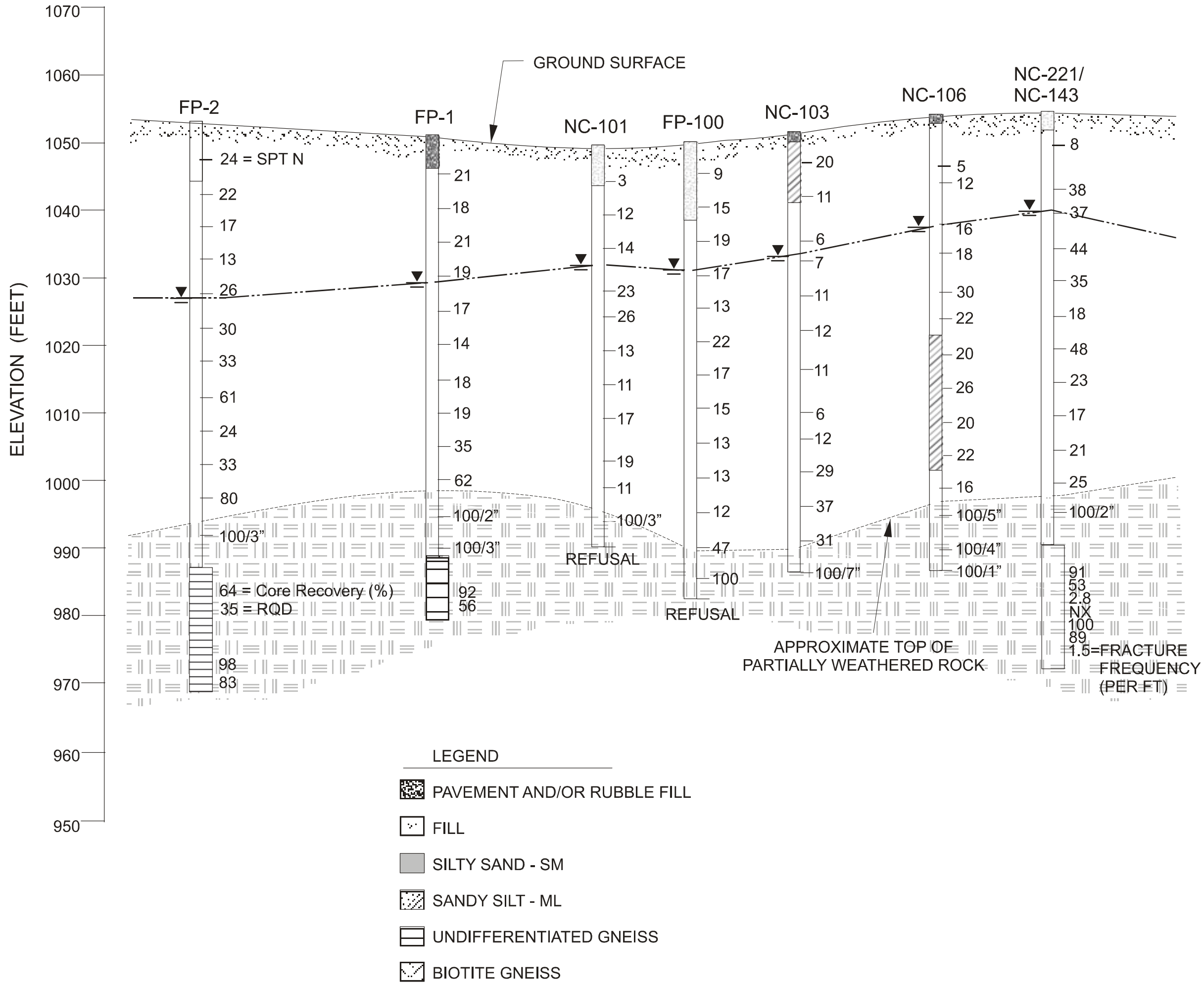


Figure 4-9. Example interpreted subsurface profile (FHWA, 2002a).

4.7.1 Use of Historical Data in Development of Subsurface Profile

Data from historical boring logs from the area can be used to supplement data provided by the current boring logs in developing a subsurface profile, however, such historical logs need to be reviewed carefully well in advance of drilling activities to ensure that the data are accurate. In some cases, boring log locations are referenced to the center alignment of a roadway without the location of the borehole having been actually surveyed. It is imperative to ensure that a consistent coordinate system is used to establish the correct relative location of all borings. Since borings would have likely been performed over an extended period of time or for different contracts along a roadway alignment (i.e., project centerlines are commonly changed during project development), it is possible that coordinate systems will not be consistent. Simply stated, if a historical boring cannot be located confidently on a site plan, then the boring has limited usefulness for establishing stratigraphy. Also, it is likely that different drill rigs with different operators and different energy efficiencies were used in the collection of SPT data on historical boring logs. This factor must also be recognized when an attempt is made to correlate engineering properties to SPT blow count values. However, the geotechnical specialist should realize that while there may be potential limitations in the use of historical borings, it is necessary to review these borings relative to the design under consideration. As an example, a historical boring may indicate a thick layer of very soft clay as evidenced by the description “weight of rod/weight of hammer” in the SPT recording box of the log at a large number of test depths. While shear strength and consolidation properties cannot be reliably estimated based on SPT blow count values, the historical boring may provide useful information concerning the depth to a firm stratum.

Most DOTs have collected large amounts of subsurface data from previous investigations within their states. Unfortunately, much of these data are archived with related project data once the project has been completed, and thus may not be readily available or accessible for use during future projects. Additionally, the subsurface data may not be fully utilized if the locations of the borings are not identified properly or if the plan drawing of the project site is not maintained with the boring logs. To overcome this problem, many DOTs currently use longitude and latitude to identify the boring locations, in lieu of or in conjunction with the conventional positioning format that uses station and offset. Unfortunately, the vast majority of the historical subsurface boring information is available only on paper. Therefore, a considerable amount of work is required to convert that data into electronic form before it can be fully appreciated and used to establish an electronic database of the subsurface information.

Several DOTs have recently commenced using electronic boring records for their projects. Not only does the use of electronic boring records provide a redundancy to compliment the paper copy, but it also preserves data in a way that has the potential for automated electronic data management. One method of electronic data management increasingly used by DOTs involves the use of a centralized electronic database in conjunction with Geographic Information System (GIS) techniques to locate and identify borings on a plan. In its most simplistic form, the electronically stored data are managed and assessed visually by using GIS software, where each boring location is identified on a plan map. An appropriately developed database and GIS can be used to great advantage by the DOT. Specifically, in addition to the previously mentioned advantages of having electronic data records compliment paper logs, it is possible to:

1. catalog borings that were conducted previously;
2. inventory data regarding specific problematic formations across the state; and

3. develop cross sections that depict subsurface conditions across a site or within a region.

This type of application of electronic boring records and data base accessibility can facilitate the development of subsequent subsurface investigations that are appropriately focused and that optimize the utility of existing data.