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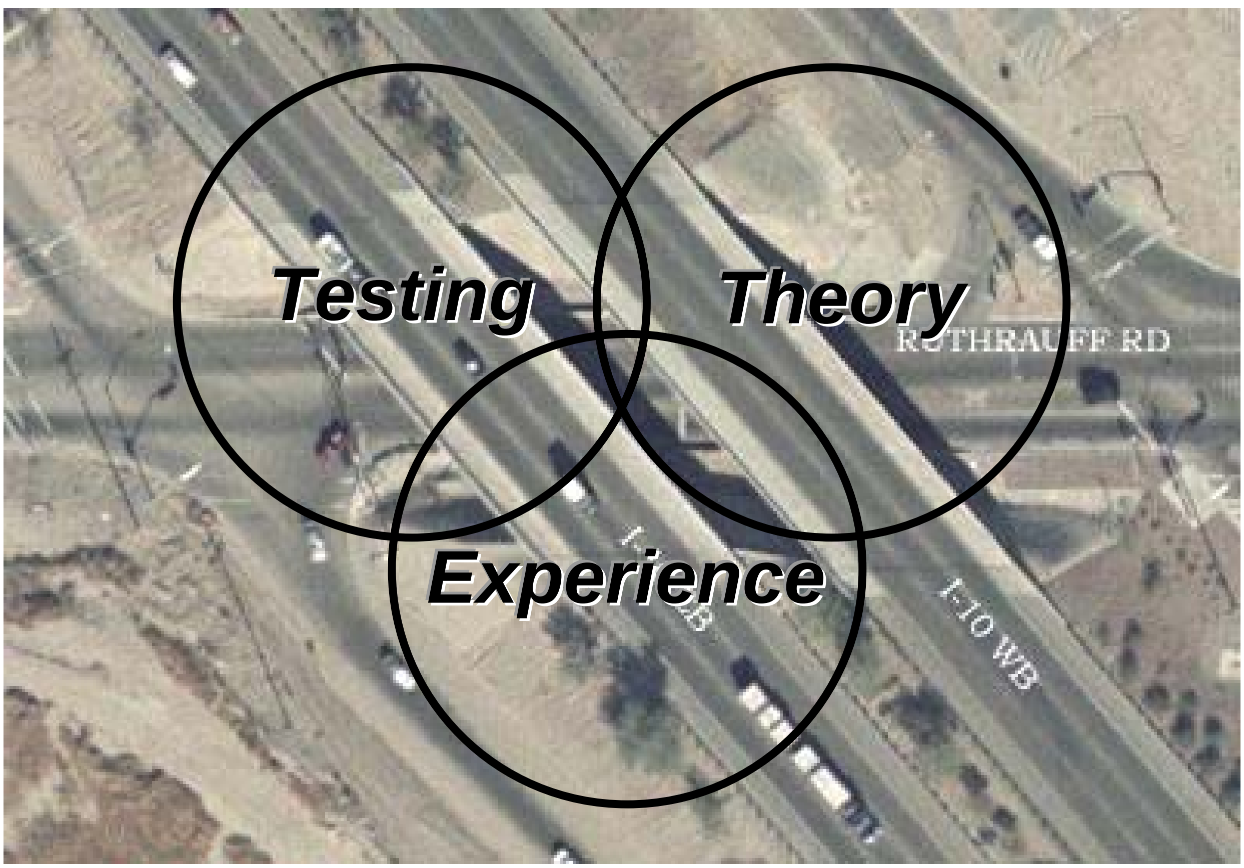
Reference Manual – Volume I



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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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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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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

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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 ϕ	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 1.0
INTRODUCTION

1.1 PURPOSE AND SCOPE

The Soils and Foundations course is sponsored by the National Highway Institute (NHI) to provide practical knowledge in geotechnical and foundation engineering for both civil engineering generalists and geotechnical and foundation specialists. The course is developed around the design and construction aspects of a highway project that includes bridges, earthworks and earth retaining structures. Bridges can range from single span bridges to multi-span bridges as part of a stack interchange. Bridges may be constructed over land, in which case they are known as viaducts, or over water. Examples of transportation facilities that include bridge structures are shown in Figures 1-1 to 1-4. Not all highway projects include bridge structures. Figure 1-5 shows an example of a highway corridor without bridges and in an environmentally sensitive area.

Figure 1-1. Aerial view of a pair of 3-span Interstate 10 (I-10) bridges over a local roadway in Tucson, Arizona.

Figure 1-2. Example of 3-span roadway bridges over another roadway.

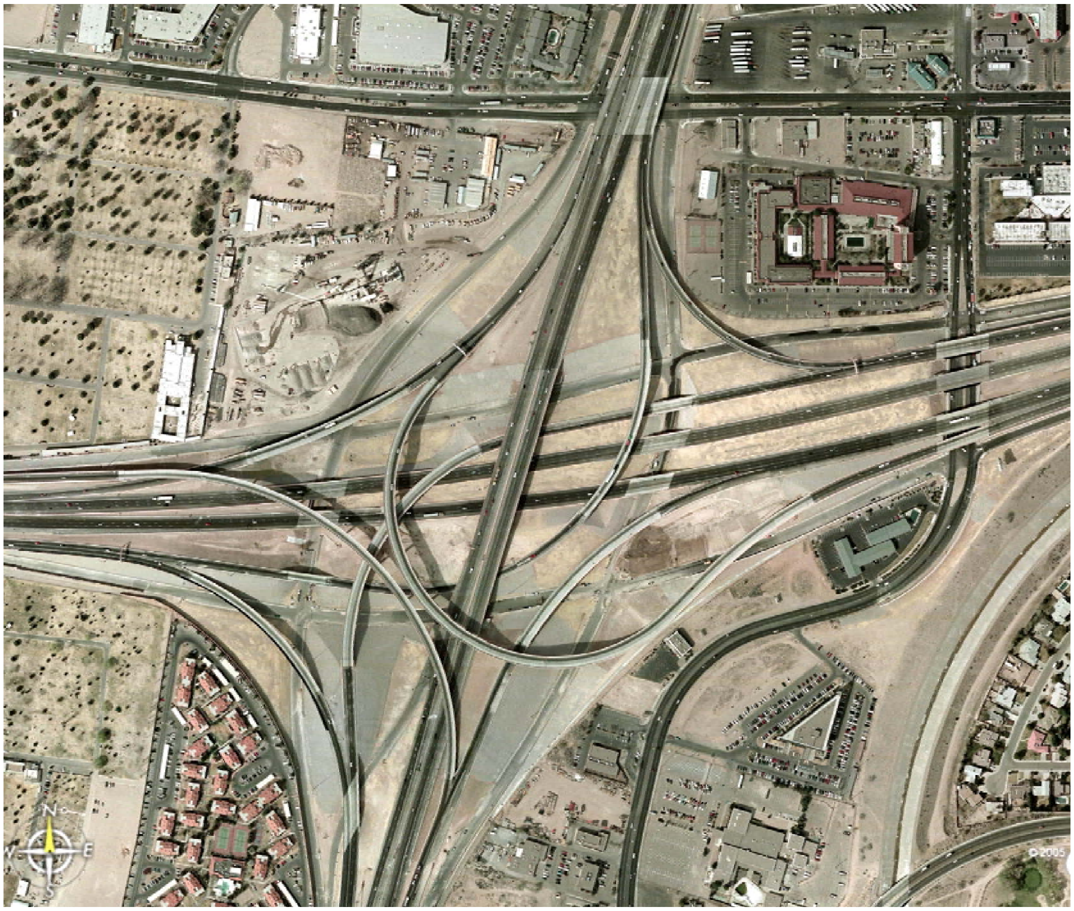

Figure 1-3. The “BIG I” stack interchange at the intersection of I-40 and I-25 in Albuquerque, New Mexico (Photo: Courtesy of Bob Meyers, NMDOT) (Note: A stack interchange is a free-flowing junction between two or more roadways that allows turning in all directions).

Figure 1-4. A major multi-span bridge structure over water (George P. Coleman Bridge over the York River in Yorktown, Virginia).

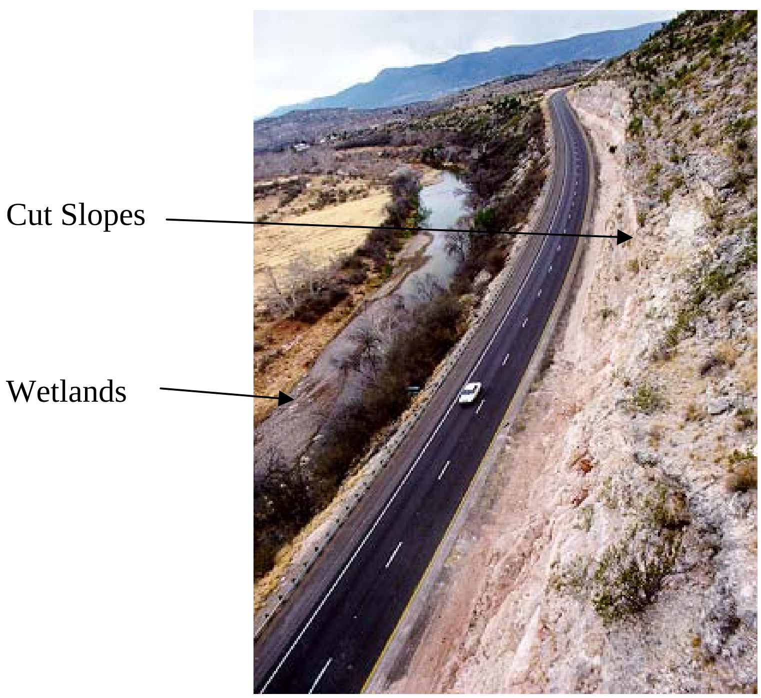

Figure 1-5. Example of a roadway bounded by cut slopes and wetlands in an environmentally sensitive area.

Highway projects can involve a full range of geotechnical engineering assessments and alternatives depending on the complexity of the project. For example, the foundations for the bridge piers and abutments may be shallow foundations, or deep foundations such as driven piles and/or drilled shafts. The approach embankments may be unreinforced slopes or reinforced soil slopes (RSS). Cut slopes may be in rocks and/or soils. Retaining walls may be used at abutments and/or along approaches and may consist of cantilevered walls or mechanically stabilized earth (MSE) walls. The ground under the bridge may be soft and require improvement. Similarly, the transportation corridor may traverse wetlands and special ground improvement measures may be required. Pavements seen in Figures 1-1 to 1-3 and 1-5 may be constructed of asphaltic concrete (AC), Portland cement concrete (PCC) or reinforced cement concrete (RCC) on a variety of subgrade materials[1].

Recognizing the need for consistent guidance for practitioners involved in the planning, design and construction of transportation facilities that include bridges and associated structures, the Federal Highway Administration (FHWA) developed the first version of this manual in 1982. Subsequently, the manual was revised in 1993 and in 2000. The present reference manual, which is the fourth edition, represents a significant update and supersedes earlier editions of the manual. In particular, this manual has been updated to reflect the current standard of geotechnical practice in the planning, design and construction of transportation facilities. As part of this effort, this edition provides guidance consistent with that found in the latest FHWA manuals and courses.

This edition of the manual, like the earlier editions, is geared towards the practicing engineer who routinely deals with soils and foundations problems on highway projects but who may not have a thorough theoretical background of soil mechanics or foundation engineering. The overall goals of this manual are: (i) to explain geotechnical engineering principles, and (ii) to provide sound methods and recommendations related to safe, cost-effective design and construction of geotechnical features. The reader is encouraged to develop an appreciation for the design and construction of geotechnical features in all phases of a project that may influence or could be influenced by his/her work (cost, quality, time, and performance). Coordination among generalists and specialists in all project phases is stressed.

The manual contains an appendix (Appendix A) wherein the geotechnical engineering input to a bridge project is traced from conception (scoping) to completion (post construction) in a serialized illustrative problem that incorporates many of the technical concepts presented in the course. The bridge project used in Appendix A is based on an actual project in the State of New York.

[1] Pavement structures are not addressed in this manual.

1.2 SOILS AND FOUNDATIONS FOR HIGHWAY FACILITIES

Civilization's earliest attempts at construction probably involved soil; however, the understanding of the role of soil as a foundation or building material developed by trial and error. Since the early 20th century, an improved understanding of soil behavior has been achieved by applying the principles of physics, solid mechanics, fluid mechanics, strength of materials, and structural engineering to define soil behavior. The body of knowledge developed by analyzing soil behavior on a theoretically sound basis is called "**soil mechanics**" and its application to solution of actual problems is called "**geotechnical engineering**." Soil is a complex three-phase medium that contains various amounts of water and/or air surrounding the solid particles. It is not a solid mass, i.e., a continuum, as many of the theories of solid mechanics require. Therefore, an entirely theoretical solution of the most commonly encountered soil problems is not practical. The most practical solution to soil problems can be reached by a combination of the sources of information as illustrated in Figure 1-6.

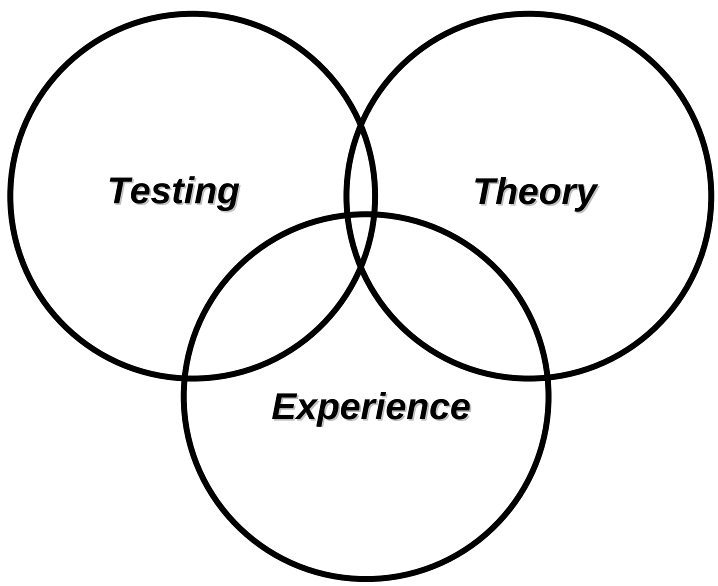

Figure 1-6. Combinations of sources of information required to solve geotechnical engineering issues.

1. **Experience** obtained from previous projects can be developed into the empirical or "rule of thumb" procedures followed by some engineers/specialists today. Often some geotechnical designers rely almost exclusively on experience. The weakness of using this approach exclusively is that experience does not always recognize the factors that cause differences in the engineering properties of soils. What works well at one location may not succeed with the same type of soil at another location because of a change in conditions, such as water content. The current state of the practice

requires the geotechnical specialist to rely on testing and theory in addition to experience or rules of thumb.

2. **Testing** of representative samples of soil in the field and laboratory is required to obtain information on the engineering properties and the characteristics of soils. The results of subsequent engineering analyses will be only as good as the soils data used as input.

3. **Theory** based on principles from various fields of engineering and science tempered by assumptions to fit reality is used to explain or predict the behavior of soils under various conditions.

The engineering analysis of soils is often more complex than the analysis of other construction materials because soil is not a continuum. Therefore, soil typically does not strictly meet the assumptions of the theories of solid mechanics and strength of materials. By contrast, steel and concrete are relatively uniform solids that have predictable properties. For example, the strength of steel is predictable within the elastic range of loading. Even though the strength of steel and concrete may be "ordered," that strength will be essentially constant under a wide range of climatic conditions. Structures can then be built of these materials with a high degree of confidence regarding the material strength.

The engineering properties of the soils, on the other hand, can vary widely over time and space so that their physical properties cannot be defined accurately at all locations for all conditions. Since soils are composed of a mixture of three dissimilar materials - soil solids, liquid fluids (usually water), and gaseous fluids (usually air) - their properties are influenced by the interaction of these three phases in the soil mass. Some of the factors that influence the behavior of soil are:

1. size, shape, and distribution of soil particles,
2. mineralogy,
3. degree of packing of soil particles,
4. amount of water in the soil,
5. climatic conditions, and
6. degree of confinement (i.e., depth).

In short, engineers should understand that the engineering properties of soils can be significantly influenced by many factors.
The success or failure of a geotechnical feature is often decided in the early stages of a project. Geotechnical engineering is a specialized field. Therefore, to assure success of a

project, the input of a qualified and experienced geotechnical specialist should begin at project inception and continue until completion of construction. Geotechnical designs are based upon soil properties that are generally defined from a subsurface exploration and laboratory testing of a very minute physical sampling of the soils. The volume of site soils excavated and exposed during construction is many orders of magnitude greater than that from the subsurface explorations. Thus, a great deal of geotechnical information can and should be gathered during the construction phase of a project to validate or revise the geotechnical design parameters. **A geotechnical design should not be considered complete until construction has been successfully completed.** A geotechnical specialist should also be involved during post-construction activities such as instrumentation monitoring, participating in resolution of contractor disputes and claims activities, and documenting lessons learnt on the project.

Based on the above considerations, early interactions at a project's scoping phase among the geotechnical specialist, other engineers/specialists, the project manager and the contractor will prevent the design of a project element, or even worse the construction of an element, such as alignment or grade, that may require costly foundation treatment later. **It is imperative that good communication and interaction exist among the geotechnical specialist, structural specialist, construction specialist, project manager and contractor <u>throughout</u> the design <u>and</u> the construction process**. Such interactions and involvement are required to insure a cost-effective design and to minimize change orders and contract disputes resulting from design deficiencies and/or misunderstandings during construction. **The importance of communication and interaction is stressed throughout this manual and cannot be overemphasized**.

The flow chart in Figure 1-7 and the six phases identified in Table 1-1 describe the details of geotechnical involvement in a typical project using the design-bid-build (D-B-B) procurement process where the geotechnical specialist interacts with the owner to provide information to the contractor. There are several other procurement processes such as the design-build (D-B) process and the Construction Manager at Risk (CMAR) process. In each such alternative process, the geotechnical specialist is concurrently dealing with both the owners and the contractors, with the geotechnical specialist's direct client being the contractor. Even though the geotechnical involvement is somewhat different in each of these types of procurement processes, it is important to realize that all the items listed in Figure 1-7 as well as in Table 1-1 must be addressed to achieve a successful project.

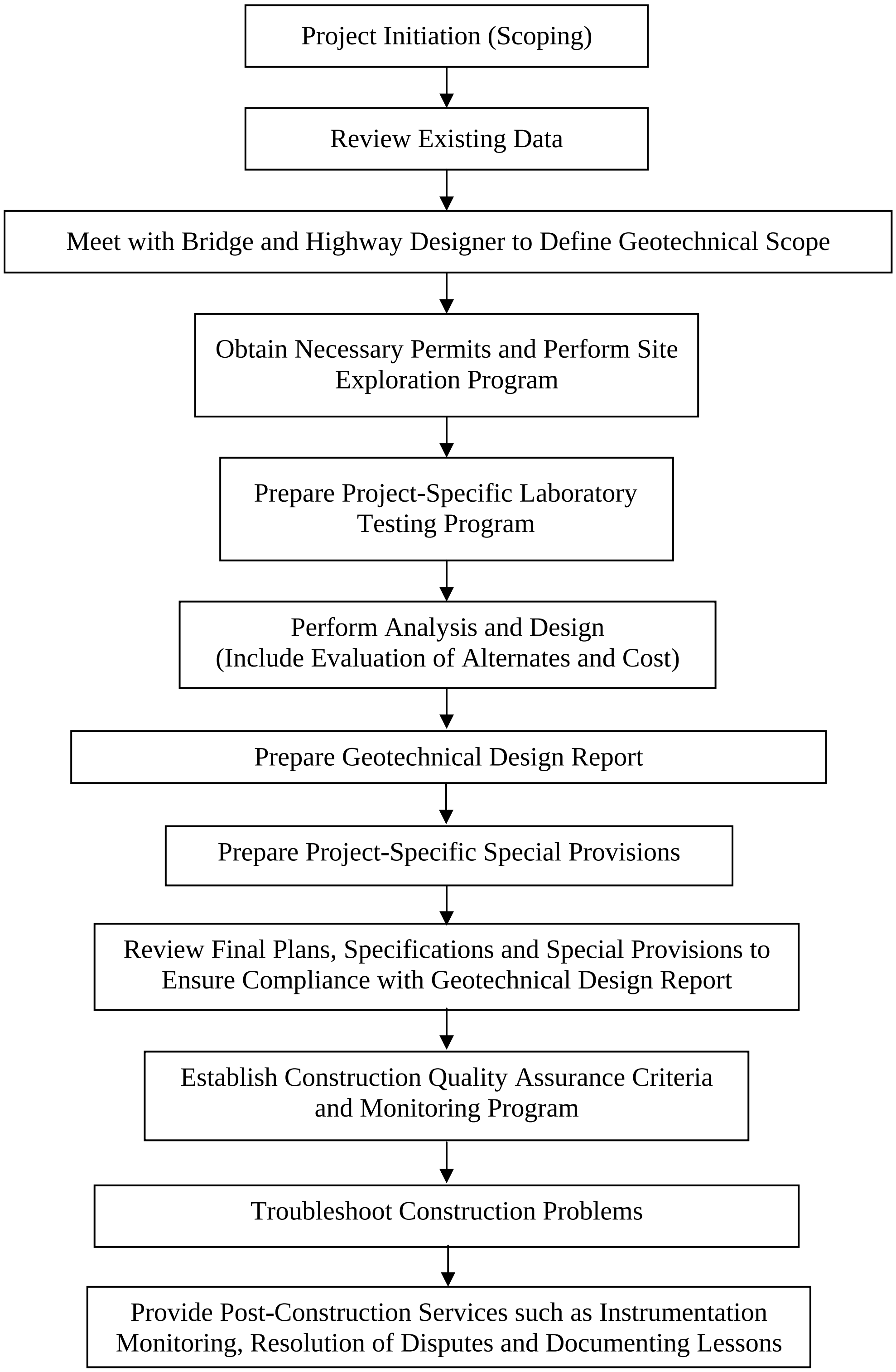

Figure 1-7. Geotechnical activity flow chart for a typical project using design-bid-build procurement process.

Table 1-1
Geotechnical involvement in project phases

Phase	Function
PHASE 1 Planning	1. Study project information, scope and existing data. (a) USGS topographic sheets. (b) USDA soil maps. (c) groundwater bulletins. (d) air photos. 2. Conduct site inspection with project manager. (a) inspect nearby structures for settlement, scour, etc. (b) assess site conditions. 3. Prepare terrain reconnaissance report for planning engineer. Include: (a) anticipated soil, rock and water conditions. (b) major problems or costs that will hinder or preclude construction of the facility. (c) right-of-way required for possible special geotechnical treatment. (d) beneficial shifts in alignment.
PHASE 2 Design Alternatives	1. Assess facility locations with regard to major soil issues. 2. Provide input for specific uses, e.g., soil/rock scour. 3. Implement subsurface exploration and laboratory testing programs after design approval.
PHASE 3 Prepare Detail Plans	1. Review and interpret subsurface information from field and laboratory work. 2. Provide preliminary input to bridge/roadway engineer. 3. Submit report to bridge and roadway engineer summarizing the investigations along with recommendations. Include: (a) coordination with roadway construction. (b) alternate foundation design. (c) subsurface profile. (d) special provisions and specifications.
PHASE 4 Final Design	1. Review final plans 2. Make appropriate adjustments to geotechnical information if necessary
PHASE 5 Construction	1. Provide geotechnical support to the resident engineer during construction. Examples are as follows: (A) *Driven Piles*: (a) submit wave equation analysis to bridge engineer. (b) hammer approval. (c) stress analysis. (d) required blow count. (e) special effects, etc. (B) *Drilled Shafts*: (a) shaft excavation information, e.g., need for casing or slurry. (b) steel placement tolerances. (c) tube placement for integrity testing. (d) concreting requirements. (e) post-installation integrity tests, etc. (C) *Spread footings*: (a) evaluation criteria of stiffness of soils at base of footing excavation, etc. (D) *Retaining Walls*: (a) construction process based on whether wall is top-down or bottom-up construction. (b) backfill compaction requirements, etc. (E) *Slopes/Embankments*: (a) backfill compaction requirements. (b) final grading of a slope, etc. 2. Attend preconstruction meeting with resident engineer and foundation inspector. Explain various important geotechnical issues: (a) general geologic profile. (b) design basis. (c) wave equation analysis for driven piles. (d) end and skin resistance values taking into account strain compatibility for drilled shafts. (e) possible geotechnical problems. 3. Troubleshoot soils-related problems as required. 4. Assist with structural foundation load tests as required.
PHASE 6 Post Construction	1. Review actual pile results versus predicted. Include: (a) blow count for driven piles. (b) installation methods for drilled shafts. (c) length. (d) field problems. (e) load test capacity. 2. Participate in contractor disputes and claims activities.

1.3 ORGANIZATION OF MANUAL

The organization of this manual follows a project-oriented approach whereby a typical bridge project is traced from scoping stage through design computation of settlement, allowable footing pressure, selection of earth retaining structure, to the construction of approach embankments, pile driving or shaft drilling operations, etc. Recommendations are presented on how to layout borings efficiently, how to minimize approach embankment settlement and eliminate the bump at the end of-the bridge, how to design the most cost-effective deep foundation, and how to transmit design information properly to contractors directly through plans, specifications, and special provisions and/or indirectly through contact with the project engineer.

The concepts presented in various chapters are concise and specifically directed at a particular operation in the geotechnical design and construction process. Basic example problems are included in several sections to illustrate how concepts are used and for hands-on knowledge. Continuity between chapters is achieved by sequencing the information in the normal progression of a geotechnical project. In addition, the manual contains an appendix (Appendix A) with the solution to geotechnical issues, in a serialized format, for a highway project involving a bridge and approach embankment over soft ground. In each phase of the fictitious project the geotechnical concepts are developed into specific designs or recommendations for that segment of the problem.

The organization of the manual and a summary of the material presented in each chapter follow.

- Chapter 1 – this chapter (Introduction) presents the purpose and scope of NHI's Soils and Foundation course and provides introductory material about geotechnical activities related to the design and construction aspects of a highway project.

- Chapter 2 (Stress and Strain in Soils) presents basic phase (weight-volume) relationships, effective stress principles, computation of overburden pressures, estimating vertical and horizontal stresses in soils due to external (superimposed) loads on geomaterials.

- Chapter 3 (Subsurface Explorations) presents basic information on subsurface exploration procedures including terrain reconnaissance, subsurface investigation methods, standard penetration test procedures, undisturbed soil sampling, and guidelines for the geotechnical investigation of both roadway and structure sites.

- Chapter 4 (Engineering Description, Classification and Characteristics of Soils and Rocks) discusses the basic engineering characteristics of the main soil and rock groups, and presents procedures for describing and classifying soils and rocks, and for developing a subsurface profile.

- Chapter 5 (Laboratory Testing for Geotechnical Design and Construction) presents several commonly used laboratory tests for soils and rocks including soil classification, basic consolidation and strength testing concepts. This chapter also includes guidelines for laboratory testing on a typical highway project, and a procedure for summarizing and choosing design values from laboratory tests.

- Chapter 6 (Slope Stability) presents the general procedures for the stability analysis of embankments and cut slopes. Basic methods of analysis are shown and explained with emphasis on practical application to highway embankments. Stability charts are presented for a rapid preliminary evaluation of slope stability. Remedial methods are discussed for stability problems.

- Chapter 7 (Approach Roadway Deformations) distinguishes between internal and external settlement within and below embankment fills. Recommendations are provided for select fill and compaction control for soils placed near abutments. Immediate (i.e., short-term) and consolidation (i.e., long-term) settlement, and lateral squeeze are discussed and methods of analysis are presented.

- Chapter 8 (Shallow Foundations) presents the FHWA-recommended foundation design procedure for shallow foundations in soils and rocks. The analysis for both bearing capacity and settlement are discussed and the application of results is presented. Economic considerations of shallow versus deep foundations are discussed.

- Chapter 9 (Deep Foundations) discusses basic concepts in the selection and design of both driven piles and drilled shafts in soils and rocks. Analyses for skin friction and end bearing are addressed for cohesive soil, cohesionless soils and rocks. Foundation installation effects on design are discussed as well as group effects, negative skin friction and deep foundation settlement. The components of pile driving equipment are presented. The use of driving formulae and the wave equation analysis in construction is introduced monitoring. Generic information is presented on the use of load testing. Construction considerations for drilled shafts are also presented.

- Chapter 10 (Earth Retaining Structures) presents the basic lateral earth pressure theories, briefly introduces various wall types, presents a wall classification system, and presents the external stability analysis for a typical fill wall.

- Chapter 11 (Geotechnical Reports) presents outlines for various types of geotechnical reports, discussions on subsurface profiles, guidelines on the use of disclaimers, and suggestions for how to incorporate geotechnical information into contract documents.

1.4 REFERENCES

A detailed list of references is provided in Chapter 12. However, certain primary references were used to develop materials for many sections in this document. In addition, FHWA has either developed or is in the process of developing detailed guidance in the topic areas covered in this document. Most of those documents are reference manuals for geotechnical courses developed for the National Highway Institute. Both the FHWA and other primary references are listed below. The reader is directed to the web site for the FHWA National Geotechnical Team (NGT), http://www.fhwa.dot.gov/engineering/geotech/index.cfm, to obtain information on all geotechnical publications and software that have been developed by FHWA. The NAVFAC manuals and many other public domain manuals can be downloaded from http://www.geotechlinks.com.

1.4.1 Primary FHWA References

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1.4.2 Other Primary References

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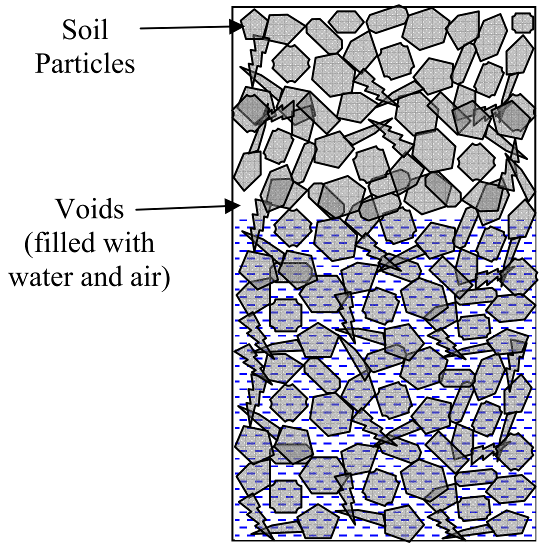
CHAPTER 2.0
STRESS AND STRAIN IN SOILS

Soil mass is generally a three phase system that consists of solid particles, liquid and gas. The liquid and gas phases occupy the voids between the solid particles as shown in Figure 2-1a. For practical purposes, the liquid may be considered to be water (although in some cases the water may contain some dissolved salts or pollutants) and the gas as air. Soil behavior is controlled by the interaction of these three phases. Due to the three phase composition of soils, complex states of stresses and strains may exist in a soil mass. Proper quantification of these states of stress, and their corresponding strains, is a key factor in the design and construction of transportation facilities.

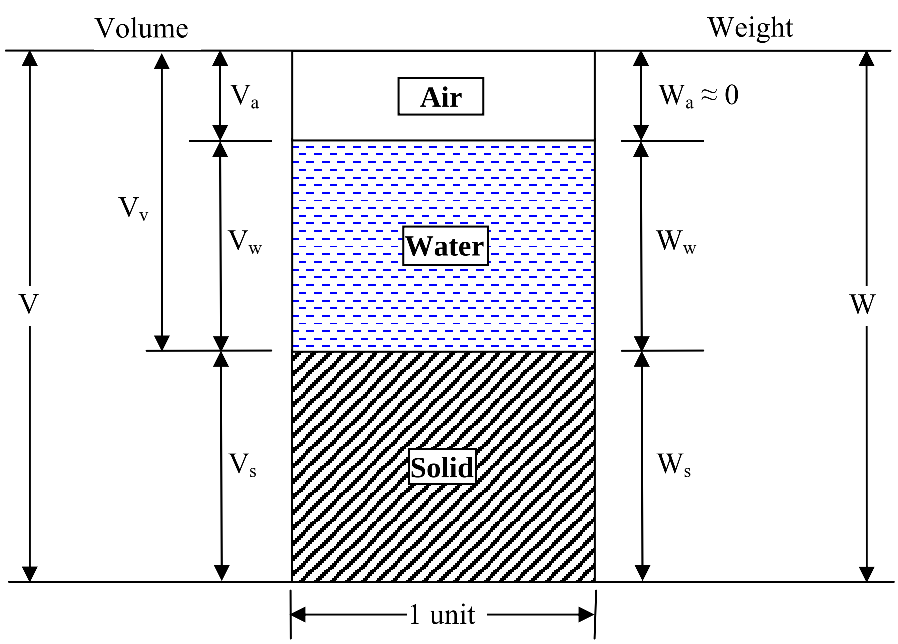
The first step in quantification of the stresses and strains in soils is to characterize the distribution of the three phases of the soil mass and determine their inter-relationships. The inter-relationships of the weights and volumes of the different phases are important since they not only help define the physical make-up of a soil but also help determine the in-situ geostatic stresses, i.e., the states of stress in the soil mass due only to the soil's self-weight. The volumes and weights of the different phases of matter in a soil mass shown in Figure 2-1a can be represented by the block diagram shown in Figure 2-1b. Such a diagram is also known as a phase diagram. A block of unit cross sectional area is considered. The symbols for the volumes and weights of the different phases are shown on the left and right sides of the block, respectively. The symbols for the volumes and weights of the three phases are defined as follows:

V_a, W_a : volume, weight of air phase. For practical purposes, $W_a = 0$.
V_w, W_w: volume, weight of water phase.
V_v, W_v : volume, weight of total voids. For practical purposes, $W_v = W_w$ as $W_a = 0$.
V_s, W_s : volume, weight of solid phase.
V, W : volume, weight of the total soil mass .

Although $W_a = 0$ so that $W_v = W_w$, V_a is generally > 0 and must always be taken into account. Since the relationship between V_a and V_w usually changes with groundwater conditions as well as under imposed loads, it is convenient to designate all the volume not occupied by the solid phase as void space, V_v. Thus, $V_v = V_a + V_w$. Use of the terms illustrated in Figure 2-1b, allows a number of basic phase relationships to be defined and/or derived as discussed next.



(a)



(b)

Figure 2-1. A unit of soil mass and its idealization.

2.1 BASIC WEIGHT-VOLUME RELATIONSHIPS

Various volume change phenomena encountered in geotechnical engineering, e.g., compression, consolidation, collapse, compaction, expansion, etc. can be described by expressing the various volumes illustrated in Figure 2-1b as a function of each other. Similarly, the in-situ stress in a soil mass is a function of depth and the weights of the different soil elements within that depth. This in-situ stress, also known as overburden stress (see Section 2.3), can be computed by expressing the various weights illustrated in Figure 2-1b as a function of each other. This section describes the basic inter-relationships among the various quantities shown in Figure 2-1b.

2.1.1 Volume Ratios

A parameter used to express of the volume of the voids in a given soil mass can be obtained from the ratio of the volume of voids, V_v, to the total volume, V. This ratio is referred to as **porosity, n,** and is expressed as a percentage as follows:

$$n = \frac{V_V}{V} x100 \qquad 2\text{-}1$$

Obviously, the porosity can never be greater than 100%. As a soil mass is compressed, the volume of voids, V_v, and the total volume, V, decrease. Thus, the value of the porosity changes. Since both the numerator and denominator in Equation 2-1 change at the same time, it is difficult to quantify soil compression, e.g., settlement or consolidation, as a function of porosity. Therefore, in soil mechanics the volume of voids, V_v, is expressed in relation to a quantity, such as the volume of solids, V_s, that remains unchanging during consolidation or compression. This is done by the introduction of a quantity known as **void ratio, e,** which is expressed in decimal form as follows:

$$e = \frac{V_V}{V_S} \qquad 2\text{-}2$$

Unlike the porosity, the void ratio can have values greater than 1. That would mean that the soil has more void volume than solids volume, which would suggest that the soil is "loose" or "soft." Therefore, in general, the smaller the value of the void ratio, the denser the soil. As a practicality, for a given type of coarse-grained soil, such as sand, there is a minimum and maximum void ratio. These values can be used to evaluate the **relative density, D_r (%),** of that soil at any intermediate void ratio as follows:

$$D_r = \frac{(e_{max} - e)}{(e_{max} - e_{min})} x100 \quad \text{2-2a}$$

At e = e_{max} the soil is as loose as it can get and the relative density equals zero. At e = e_{min} the soil is as dense as it can get and the relative density equals 100%. Relative density and void ratio are particularly useful index properties since they are general indicators of the relative strength and compressibility of the soil sample, i.e., high relative densities and low void ratios generally indicate strong or incompressible soils; low relative densities and high void ratios may indicate weak or compressible soils.

While the expressions for porosity and void ratio indicate the relative volume of voids, they do not indicate how much of the void space, V_v, is occupied by air or water. In the case of a saturated soil, all the voids (i.e., soil pore spaces) are filled with water, $V_v = V_w$. While this condition is true for many soils below the ground water table or below standing bodies of water such as rivers, lakes, or oceans, and for some fine-grained soils above the ground water table due to capillary action, the condition of most soils above the ground water table is better represented by consideration of all three phases where voids are occupied by both air and water. To express the amount of void space occupied by water as a percentage of the total volume of voids, the term **degree of saturation, S,** is used as follows:

$$S = \frac{V_W}{V_V} x100 \quad \text{2-3}$$

Obviously, the degree of saturation can never be greater than 100%. When S = 100%, all the void space is filled with water and the soil is considered to be **saturated**. When S = 0%, there is no water in the voids and the soil is considered to be **dry**.

2.1.2 Weight Ratios

While the expressions of the distribution of voids in terms of volumes are convenient for theoretical expressions, it is difficult to measure these volumes accurately on a routine basis. Therefore, in soil mechanics it is convenient to express the void space in gravimetric, i.e., weight, terms. Since, for practical purposes, the weight of air, W_a, is zero, a measure of the void space in a soil mass occupied by water can be obtained through an index property known as the ***gravimetric*** **water or moisture content, w,** expressed as a percentage as follows:

$$w = \frac{W_w}{W_s} x100 \qquad 2\text{-}4$$

The word "gravimetric" denotes the use of weight as the basis of the ratio to compute water content as opposed to volume, which is often used in hydrology and the environmental sciences to express water content. Since water content is understood to be a weight ratio in geotechnical engineering practice, the word "gravimetric" is generally omitted. Obviously, the water content can be greater than 100%. This occurs when the weight of the water in the soil mass is greater than the weight of the solids. In such cases the void ratio of the soil is generally greater than 1 since there must be enough void volume available for the water so that its weight is greater than the weight of the solids. However, even if the water content is greater than 100%, the degree of saturation may not be 100% because the water content is a weight ratio while saturation is a volume ratio.

For a given amount of soil, the total weight of soil, W, is equal to $W_s + W_w$, since the weight of air, W_a, is practically zero. The water content, w, can be easily measured by oven-drying a given quantity of soil to a high enough temperature so that the amount of water evaporates and only the solids remain. By measuring the weight of a soil sample before and after it ahs been oven dried, both W and W_s, can be determined. The water content, w, can be determined as follows since $W_a = 0$:

$$w = \frac{W - W_s}{W_s} = \frac{W_w}{W_s} x100 \qquad 2\text{-}4a$$

Most soil moisture is released at a temperature between 220 and 230°F (105 and 110°C). Therefore, to compare reported water contents on an equal basis between various soils and projects, this range of temperature is considered to be a standard range.

2.1.3 Weight-Volume Ratios (Unit Weights) and Specific Gravity

The simplest relationship between the weight and volume of a soil mass (refer to Figure 2-1b) is known as the **total unit weight, γ_t,** and is expressed as follows:

$$\gamma_t = \frac{W}{V} = \frac{W_w + W_s}{V} \qquad 2\text{-}5$$

The total unit weight of a soil mass is a useful quantity for computations of vertical in-situ stresses. For a constant volume of soil, the total unit weight can vary since it does not account for the distribution of the three phases in the soil mass. Therefore the value of the total unit weight for a given soil can vary from its maximum value when all of the voids are filled with water (S=100%) to its minimum value when there is no water in the voids (S=0%). The former value is called the **saturated unit weight, γ_{sat}**; the latter value is referred to as the **dry unit weight**, γ_d. In terms of the basic quantities shown in Figure 2-1b and with reference to Equation 2-5, when $W_w = 0$ the **dry unit weight, γ_d,** can be expressed as follows:

$$\gamma_d = \frac{W_S}{V} \qquad \text{2-6}$$

For computations involving soils below the water table, the buoyant unit weight is frequently used where:

$$\gamma_b = \gamma_{sat} - \gamma_w \qquad \text{2-7}$$

where, γ_w equals the unit weight of water and is defined as follows:

$$\gamma_w = \frac{W_w}{V_w} \qquad \text{2-8}$$

In the geotechnical literature, the buoyant unit weight, γ_b, is also known as the effective unit weight, γ', or submerged unit weight, γ_{sub}. Unless there is a high concentration of dissolved salts, e.g., in sea water, the unit weight of water, γ_w, can be reasonably assumed to be 62.4 lb/ft^3 (9.81 kN/m^3).

To compare the properties of various soils, it is often instructive and preferable to index the various weights and volumes to unchanging quantities, which are the volume of solids, V_s, and the weight of solids, W_s. A ratio of W_s to V_s, is known as the unit weight of the solid phase, γ_s, and is expressed as follows:

$$\gamma_S = \frac{W_S}{V_S} \qquad \text{2-9}$$

The unit weight of the solid phase, γ_s, should not be confused with the dry unit weight of the soil mass, γ_d, which is defined in Equation 2-6 as the total unit weight of the soil mass when there is no water in the voids, i.e., at S = 0%. The distinction between γ_s and γ_d is very subtle, but it is very important and should not be overlooked. For example, for a solid piece of rock (i.e., no voids) the total unit weight is γ_s while the total unit weight of a soil whose voids are dry is γ_d. In geotechnical engineering, γ_d is more commonly of interest than γ_s.

Since the value of γ_w is reasonably well known, the unit weight of solids, γ_s, can be expressed in terms of γ_w. The concept of **Specific Gravity, G,** is used to achieve this goal. In physics textbooks, G is defined as the ratio between the mass density of a substance and the mass density of some reference substance. Since unit weight is equal to mass density times the gravitational constant, G can also be expressed as the ratio between the unit weight of a substance and the unit weight of some reference substance. In the case of soils, the most convenient reference substance is water since it is one of the three phases of the soil and its unit weight is reasonably constant. Using this logic, the **specific gravity of the soil solids**, G_s, can be expressed as follows:

$$G_S = \frac{\gamma_S}{\gamma_w} \qquad 2\text{-}10$$

The **bulk specific gravity of a soil** is equal to γ_t / γ_w. The "bulk specific gravity" is not the same as G_s and is not very useful in practice since the γ_t of a soil can change easily with changes in void ratio and/or degree of saturation. Therefore, the bulk specific gravity is almost never used in geotechnical engineering computations.

The value of G_s can be determined in the laboratory, but it can usually be estimated with sufficient accuracy for various types of soil solids. For routine computations, the value of G_s for sands composed primarily of quartz particles may be taken as 2.65. Tests on a large number of clay soils indicate that the value of G_s for clays usually ranges from 2.5 to 2.9 with an average value of 2.7.

2.1.4 Determination and Use of Basic Weight-Volume Relations

The five relationships, n, e, w, γ_t and G_s, represent the basic weight-volume properties of soils and are used in the classification of soils and for the development of other soil properties. These properties and how they are obtained and applied in geotechnical engineering are summarized in Table 2-1. A summary of commonly used weight-volume (unit weight) relations that incorporate these terms is presented in Table 2-2.

Table 2-1
Summary of index properties and their application

Property	Symbol	Units[1]	How Obtained (AASHTO/ASTM)	Comments and Direct Applications
Porosity	n	Dim	From weight-volume relations	Defines relative volume of voids to total volume of soil
Void Ratio	e	Dim	From weight-volume relations	Volume change computations
Moisture Content	w	Dim	By measurement (T 265/ D 4959)	Classification and in weight-volume relations
Total unit weight [2]	γ_t	FL^{-3}	By measurement or from weight-volume relations	Classification and for pressure computations
Specific Gravity	G_s	Dim	By measurement (T 100/D 854)	Volume computations

NOTES:
1 F=Force or weight; L = Length; Dim = Dimensionless. Although by definition, moisture content is a dimensionless decimal (ratio of weight of water to weight of solids) and used as such in most geotechnical computations, it is commonly reported in percent by multiplying the decimal by 100.
2 Total unit weight for the same soil can vary from "saturated" (S=100%) to "dry" (S=0%).

Table 2-2
Weight-volume relations (after Das, 1990)

Unit-Weight Relationship	Dry Unit Weight (No Water)	Saturated Unit Weight (No Air)
$\gamma_t = \frac{(1+w)G_s\gamma_w}{1+e}$ $\gamma_t = \frac{(G_s+Se)\gamma_w}{1+e}$ $\gamma_t = \frac{(1+w)G_s\gamma_w}{1+\frac{wG_s}{S}}$ $\gamma_t = G_s\gamma_w(1-n)(1+w)$	$\gamma_d = \frac{\gamma_t}{1+w}$ $\gamma_d = \frac{G_s\gamma_w}{1+e}$ $\gamma_d = G_s\gamma_w(1-n)$ $\gamma_d = \frac{G_s\gamma_w}{1+\frac{wG_s}{S}}$ $\gamma_d = \frac{eS\gamma_w}{(1+e)w}$ $\gamma_d = \gamma_{sat} - n\gamma_w$ $\gamma_d = \gamma_{sat} - \left(\frac{e}{1+e}\right)\gamma_w$	$\gamma_{sat} = \frac{(G_s+e)\gamma_w}{1+e}$ $\gamma_{sat} = [(1-n)G_s+n]\gamma_w$ $\gamma_{sat} = \left(\frac{1+w}{1+wG_s}\right)G_s\gamma_w$ $\gamma_{sat} = \left(\frac{e}{w}\right)\left(\frac{1+w}{1+e}\right)\gamma_w$ $\gamma_{sat} = \gamma_d + n\gamma_w$ $\gamma_{sat} = \gamma_d + \left(\frac{e}{1+e}\right)\gamma_w$
In above relations, γ_w refers to the unit weight of water, 62.4 pcf (=9.81 kN/m^3).		

2.1.5 Size of Grains in the Solid Phase

As indicated in Figure 2-1a, the solid phase is composed of soil grains. One of the major factors that affect the behavior of the soil mass is the size of the grains. The size of the grains may range from the coarsest (e.g., boulders, which can be 12- or more inches [300 mm] in diameter) to the finest (e.g., colloids, which can be smaller than 0.0002–inches [0.005 mm]). Since soil particles come in a variety of different shapes, the size of the grains is defined in terms of an effective grain diameter. The distribution of grain sizes in a soil mass is determined by shaking air-dried material through a stack of sieves having decreasing opening sizes. Table 2-3 shows U.S. standard sieve sizes and associated opening sizes. **Sieves with opening size 0.25 in (6.35 mm) or less are identified by a sieve number which corresponds to the approximate number of square openings per linear inch of the sieve (ASTM E 11).**

To determine the grain size distribution, the soil is sieved through a stack of sieves with each successive screen in the stack from top to bottom having a smaller (approximately half of the upper sieve) opening to capture progressively smaller particles. Figure 2-2 shows a selection of some sieves and starting from right to left soil particles retained on each sieve, except for the powdery particles shown on the far left, which are those that passed through the last sieve on the stack. The amount retained on each sieve is collected, dried and weighed to determine the amount of material passing that sieve size as a percentage of the total sample being sieved. Since electro-static forces impede the passage of finer-grained particles through sieves, testing of such particles is accomplished by suspending the chemically dispersed particles in a water column and measuring the change in specific gravity of the liquid as the particles fall from suspension. The change in specific gravity is related to the fall velocities of specific particle sizes in the liquid. This part of the test is commonly referred to as a hydrometer analysis. Because of the strong influence of electro-chemical forces on their behavior, colloidal sized particles may remain in suspension indefinitely (particles with sizes from 10^{-3} mm to 10^{-6} mm are termed "**colloidal**.") Sample grain size distribution curves are shown in Figure 2-3. The nomenclature associated with various grain sizes (cobble, gravel, sand, silt or clay) is also shown in Figure 2-3. Particles having sizes larger than the No. 200 sieve (0.075 mm) are termed "**coarse-grained**" while those with sizes finer than the No. 200 sieve are termed "**fine-grained**."

The results of the sieve and hydrometer tests are represented graphically on a grain size distribution curve or gradation curve. As shown in Figure 2-3, an arithmetic scale is used on the ordinate (Y-axis) to plot the percent finer by weight and a logarithmic scale is used on the abscissa (X-axis) for plotting particle (grain) size, which is typically expressed in millimeters.

Table 2-3
U.S. standard sieve sizes and corresponding opening dimension

U.S. Standard Sieve No.[1]	Sieve Opening (in)	Sieve Opening (mm)	Comment (Based on the Unified Soil Classification System (USCS) discussed in Chapter 4)
3	0.2500	6.35	
4	0.1870	4.75	• Breakpoint between fine gravels and coarse sands • Soil passing this sieve is used for compaction test
6	0.1320	3.35	
8	0.0937	2.36	
10	0.0787	2.00	• Breakpoint between coarse and medium sands
12	0.0661	1.70	
16	0.0469	1.18	
20	0.0331	0.850	
30	0.0234	0.600	
40	0.0165	0.425	• Breakpoint between medium and fine sands • Soil passing this sieve is used for Atterberg limits
50	0.0117	0.300	
60	0.0098	0.250	
70	0.0083	0.212	
100	0.0059	0.150	
140	0.0041	0.106	
200	0.0029	0.075	• Breakpoint between fine sand and silt or clay
270	0.0021	0.053	
400	0.0015	0.038	

Note:

1. The sieve opening sizes for various sieve numbers listed above are based on Table 1 from ASTM E 11. Sieves with opening size greater than No. 3 are identified by their opening size. Some of these sieves are as follows:

4.0 in	(101.6 mm)	1-1/2 in	(38.1 mm)	½ in	(12.7 mm)
3.0 in	(76.1 mm)*	1-1/4 in	(32.0 mm)	3/8 in	(9.5 mm)
2-1/2 in	(64.0 mm)	1.0 in	(25.4 mm)	5/16 in	(8.0 mm)
2.0 in	(50.8 mm)	¾ in	(19.0 mm)**		
1-3/4 in	(45.3 mm)	5/8 in	(16.0 mm)		

* The 3 in (76.1 mm) sieve size differentiates between cobbles and coarse gravels. **The ¾ in (19 mm) sieve differentiates between coarse and fine gravels.

Figure 2-2. Example of laboratory sieves for mechanical analysis for grain size distributions. Shown (right to left) are sieve nos. 3/8-in (9.5-mm), No. 10 (2.0-mm), No. 40 (0.425 mm) and No. 200 (0.075 mm). Example soil particle sizes shown at the bottom of the photo include (right to left): medium gravel, fine gravel, medium-coarse sand, silt, and clay (kaolin) (FHWA, 2002b).

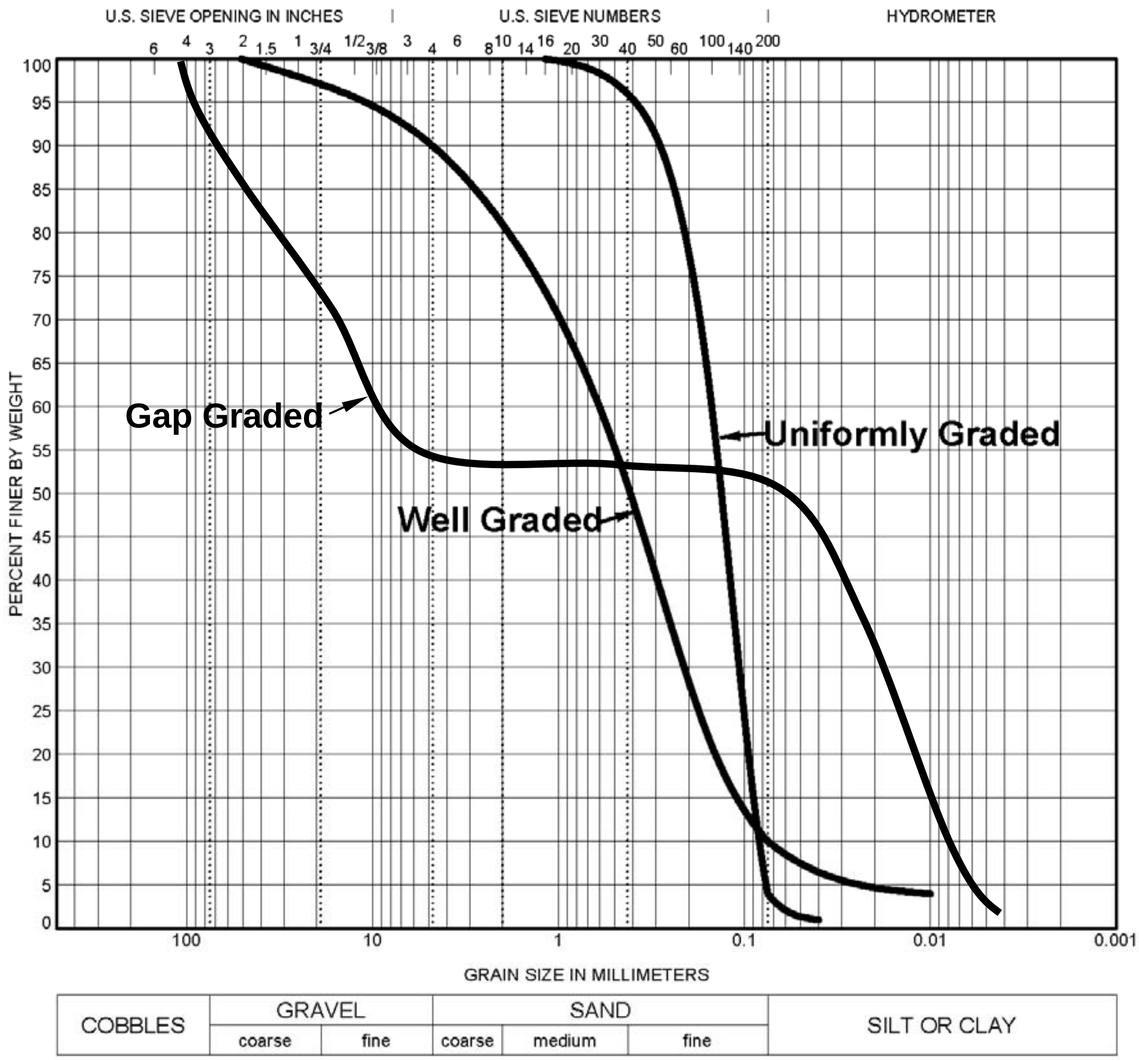


Figure 2-3. Sample grain size distribution curves.

The logarithmic scale permits a wide range of particle sizes to be shown on a single plot. More importantly it extends the scale, thus giving all the grains sizes an approximately equal amount of separation on the X-axis. For example, a grain-size range of 4.75 mm (No.4 sieve) to 0.075 mm (No. 200 sieve) when plotted on an arithmetic scale, will have the 0.075 mm (No. 200 sieve), 0.105 mm (No. 140 sieve), and 0.150 mm (No. 100) particle size plot very close to each other. The logarithmic scale permits separation of grain sizes that makes it easier to compare the grain size distribution of various soils.

The shape of the grain size distribution curve is somewhat indicative of the particle size distribution as shown in Figure 2-3. For example,

- A smooth curve covering a wide range of sizes represents a *well-graded* or *non-uniform* soil.

- A vertical or near vertical slope over a relatively narrow range of particle sizes indicates that the soil consists predominantly of the particle sizes within that range of particle sizes. A soil consisting of particles having only a few sizes is called a *poorly-graded* or *uniform* soil.

- A curve that contains a horizontal or nearly horizontal portion indicates that the soil is deficient in the grain sizes in the region of the horizontal slope. Such a soil is called a *gap-graded* soil.

Well-graded soils are generally produced by bulk transport processes (e.g., glacial till). Poorly graded soils are usually sorted by the transporting medium e.g., beach sands by water; loess by wind. Gap-graded soils are also generally sorted by water, but certain sizes were not transported.

2.1.6 Shape of Grains in Solid Phase

The shape of individual grains in a soil mass plays an important role in the engineering characteristics (strength and stability) of the soil. Two general shapes are normally recognized, bulky and platy.

2.1.6.1 Bulky Shape

Cobbles, gravel, sand and some silt particles cover a large range of sizes as shown in Figure 2-2; however, they are all bulky in shape. The term bulky is confined to particles that are relatively large in all three dimensions, as contrasted to platy particles, in which one dimension is small as compared to the other two, see Figure 2-4. The bulky shape has five subdivisions listed in descending order of desirability for construction

- *Angular* particles are those that have been freshly broken up and are characterized by jagged projections, sharp ridges, and flat surfaces. Angular gravels and sands are generally the best materials for construction because of their interlocking characteristics. Such particles are seldom found in nature, however, because physical and chemical weathering processes usually wear off the sharp ridges in a relatively short period time. Angular material is usually produced artificially, by crushing.

- *Subangular* particles are those that have been weathered to the extent that the sharper points and ridges have been worn off.

- *Subrounded* particles are those that have been weathered to a further degree than subangular particles. They are still somewhat irregular in shape but have no sharp corners and few flat areas. Materials with this shape are frequently found in stream beds. If composed of hard, durable particles, subrounded material is adequate for most construction needs.

- *Rounded* particles are those on which all projections have been removed, with few irregularities in shape remaining. The particles resemble spheres and are of varying sizes. Rounded particles are usually found in or near stream beds or beaches.

- *Well rounded* particles are rounded particles in which the few remaining irregularities have been removed. Like rounded particles, well rounded particles are also usually found in or near stream beds or beaches.

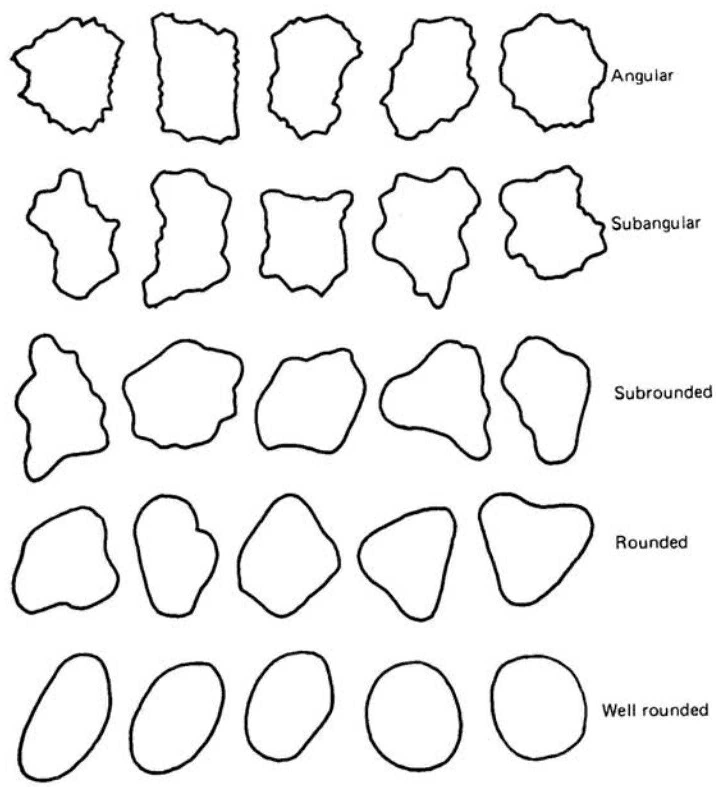


Figure 2-4. Terminology used to describe shape of coarse-grained soils (Mitchell, 1976).

2.1.6.2 Platy Shape

Platy, or flaky, particles are those that have flat, plate like grains. Clay and some silts are common examples. Because of their shape, flaky particles have a greater surface area than bulky particles, assuming that the weights and volumes of the two are the same. For example, 1 gram of bentonite (commercial name for montmorillonite clay) has a surface area of approximately 950 yd^2 (800 m^2) compared to a surface area of approximately 0.035 yd^2 (0.03 m^2) of 1 gram of sand. Because of their mineralogical composition and greater specific surface area, most flaky particles also have a greater affinity for water than bulky particles. Due to the high affinity of such soils for water, the physical states of such fine-grained soils change with the amount of water in these soils. The effect of water on the physical states of fine grained soils is discussed next.

2.1.7 Effect of Water on Physical States of Soils

For practical purposes, the two most dominant phases are the solid phase and the water phase. It is intuitive that as the water content increases, the contacts between the particles comprising the solid phase will be "lubricated." If the solid phase is comprised of coarse particles, e.g. coarse sand or gravels, then water will start flowing between the particles of the solid phase. If the solid phase is comprised of fine-grained particles, e.g., clay or silt, then water cannot flow as freely as in the coarse-grained solid phase because pore spaces are smaller and solids react with water. However, as the water content increases even the fine-grained solid phase will conduct water and under certain conditions the solid phase itself will start deforming like a viscous fluid, e.g., like a milk shake or a lava flow. The mechanical transformation of the fine-grained soils from a solid phase into a viscous phase is a very important concept in geotechnical engineering since it is directly related to the load carrying capacity of soils. It is obvious that the load carrying capacity of a solid is greater than that of water. Since water is contained in the void space, the effect of water on the physical states of fine-grained soils is important. Some of the basic index properties related to the effect of water are described next.

The physical and mechanical behavior of fine-grained-soils is linked to four distinct states: solid, semi-solid, plastic and viscous liquid in order of increasing water content. Consider a soil initially in a viscous liquid state that is allowed to dry uniformly. This state is shown as Point A in Figure 2-5, which shows a plot of total volume versus water content. As the soil dries, its water content reduces and consequently so does its total volume as the solid particles move closer to each other. As the water content reduces, the soil can no longer flow like a viscous liquid. Let us identify this state by Point B in Figure 2-5. The water content at Point B is known as the "Liquid Limit" in geotechnical engineering and is denoted by LL.

As the water content continues to reduce due to drying, there is a range of water content at which the soil can be molded into any desired shape without rupture. In this range of water content, the soil is considered to be "plastic."

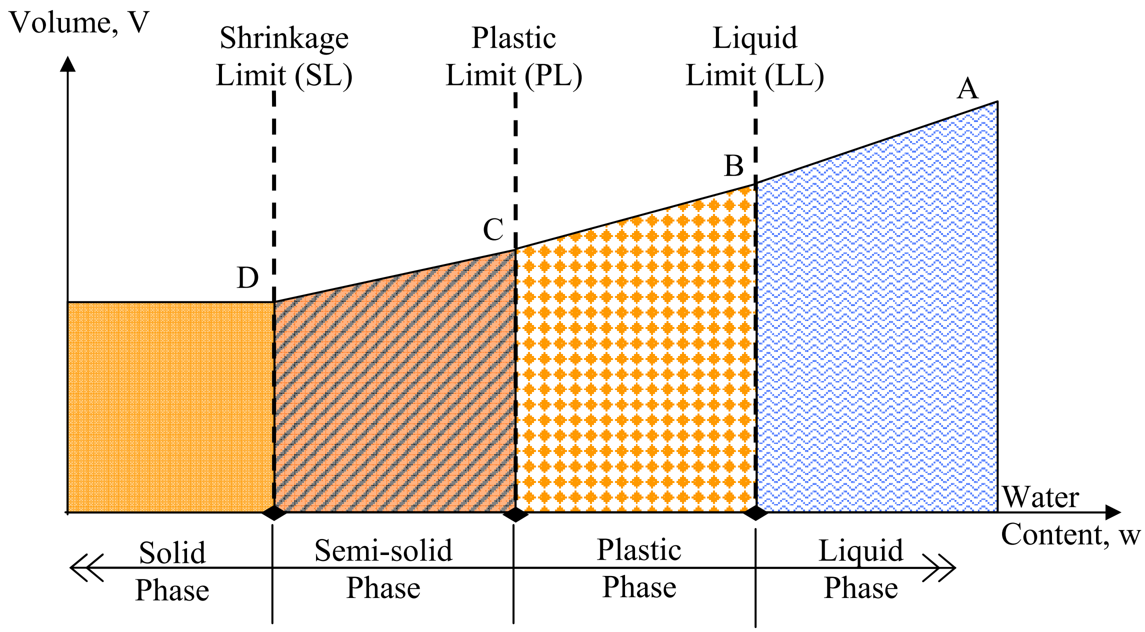


Figure 2-5. Conceptual changes in soil phases as a function of water content.

If the soil is allowed to dry beyond the plastic state, the soil cannot be molded into any shape without showing cracks, i.e., signs of rupture. The soil is then in a semi-solid state. The water content at which cracks start appearing when the soil is molded is known as the "Plastic Limit." This moisture content is shown at Point C in Figure 2-5 and is denoted by PL. The difference in water content between the Liquid Limit and Plastic Limit, is known as the **Plasticity Index**, PI, and is expressed as follows:

$$PI = LL - PL \tag{2-11}$$

Since PI is the difference between the LL and PL, it denotes the range in water content over which the soil acts as a plastic material as shown in Figure 2-5.

As the soil continues to dry, it will be reduced to its basic solid phase. The water content at which the soil changes from a semi-solid state to a solid state is called the **Shrinkage Limit**, SL. No significant change in volume will occur with additional drying below the shrinkage limit. The shrinkage limit is useful for the determination of the swelling and shrinkage characteristics of soils.

The liquid limit, plastic limit and shrinkage limit are called Atterberg limits after A. Atterberg (1911), the Swedish soil scientist who first proposed them for agricultural applications.

For foundation design, engineers are most interested in the load carrying capacity, i.e., strength, of the soil and its associated deformation. The soil has virtually no strength at the LL, while at water contents lower than the PL (and certainly below the SL) the soil may have considerable strength. Correspondingly, soil strength increases and soil deformation decreases as the water content of the soil reduces from the LL to the SL. Since the Atterberg limits are determined for a soil that is remolded, a connection needs to be made between these limits and the in-situ moisture content, w, of the soil for the limits to be useful in practical applications in foundation design. One way to quantify this connection is through the **Liquidity Index**, LI, that is given by:

$$LI = \frac{w - PL}{PI} \qquad \text{2-12}$$

The liquidity index is the ratio of the difference between the soil's in-situ water content and plastic limit to the soil's plasticity index. The various phases shown in Figure 2-5 and anticipated deformation behavior can now be conveniently expressed in terms of LI as shown in Table 2-4.

Table 2-4
Concept of soil phase, soil strength and soil deformation based on Liquidity Index

Liquidity Index, LI	Soil Phase	Soil Strength (Soil Deformation)
$LI \geq 1$	Liquid	Low strength (Soil deforms like a viscous fluid)
$0 < LI < 1$	Plastic	Intermediate strength • at w ≈ LL, the soil is considered soft and very compressible • at w ≈ PL, the soil is considered stiff (Soil deforms like a plastic material)
$LI \leq 0$	Semi-solid to Solid	High strength (Soil deforms as a brittle material, i.e., sudden, fracture of material)

Another valuable tool in assessing the characteristics of a fine-grained soil is to compare the LL and PI of various soils. Each fine-grained soil has a relatively unique value of LL and PI. A plot of PI versus LL is known as the **Plasticity Chart** (see Figure 2-6). Arthur Casagrande, who developed the concept of the Plasticity Chart, had noted the following during the First Pan American Conference on Soil Mechanics and Foundation Engineering (Casagrande, 1959).

> *I consider it essential that an experienced soils engineer should be able to judge the position of soils, from his territory, on a plasticity chart merely on the basis of his visual and manual examination of the soils. And more than that, the plasticity chart should be for him like a map of the world. At least for certain areas of the chart, that are significant for his activities, he should be well familiar. The position of soils within these areas should quickly convey to him a picture of the significant engineering properties that he should expect.*

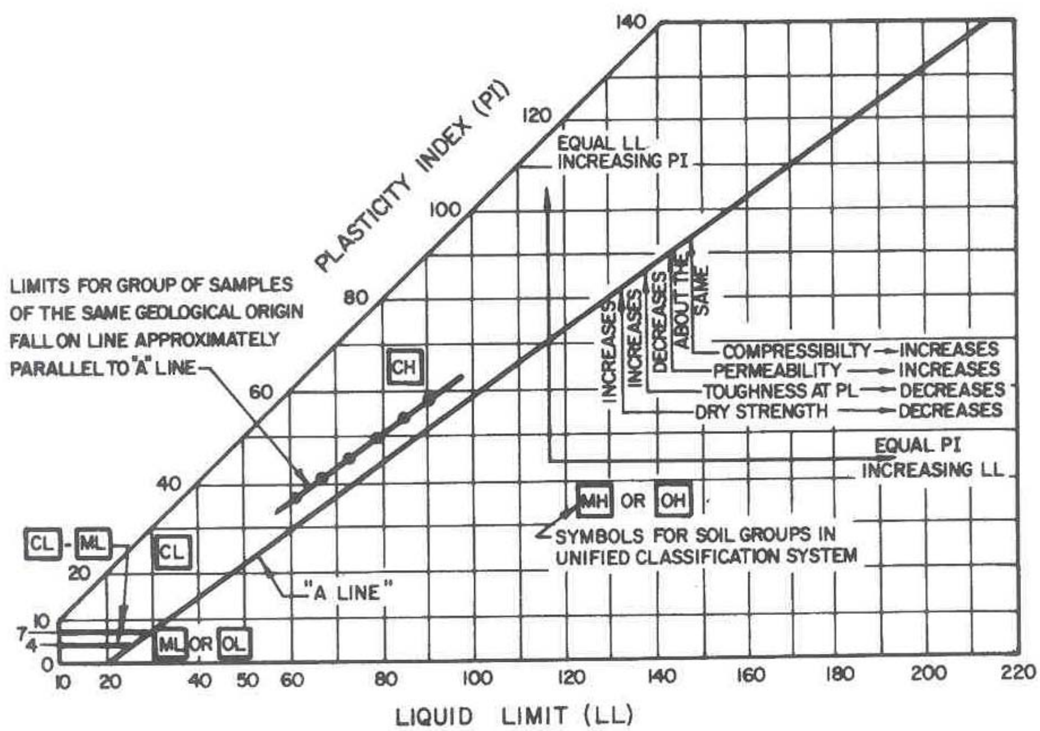


Figure 2-6. Plasticity chart and significance of Atterberg Limits (NAVFAC, 1986a).

Casagrande proposed the inclusion of the A-line on the plasticity chart as a boundary between clay (above the A-line) and silt (below the A-line) to help assess the engineering characteristics of fine-grained soils. Once PI and LL are determined for a fine-grained soil at a specific site, a point can be plotted on the plasticity chart that will allow the engineer to develop a feel for the general engineering characteristics of that particular soil. The plasticity chart also permits the engineer to compare different soils across the project site and even between different project sites. (The symbols for soil groups such as CL and CH are discussed later in this manual.) The plasticity chart, including the laboratory determination of the various limits (LL, PL and SL), are discussed further in Chapters 4 and 5. Additional useful terms such as "Activity Ratio" that relate the PI to clay fraction are also introduced in Chapter 5.

2.2 PRINCIPLE OF EFFECTIVE STRESS

The contacts between soil grains are effective in resisting applied stresses in a soil mass. Under an applied load, the total stress in a saturated soil sample is composed of the intergranular stress and the pore water pressure. When pore water drains from a soil, the contact between the soil grains increases, which increases the level of intergranular stress. This intergranular contact stress is called the **effective stress**. **The effective stress, p_o, within a soil mass is the difference between the total stress, p_t, and pore water pressure, u**. The **principle of effective stress** is a fundamental aspect of geotechnical engineering and is written as follows:

$$p_o = p_t - u \qquad \text{2-13}$$

In general, soil deposits below the ground water table will be considered saturated and the ambient pore water pressure at any depth may be computed by multiplying the unit weight of water, γ_w, by the height of water above that depth. The total stress at that depth may be found by multiplying the total unit weight of the soil by the depth. The effective stress is the total stress minus the pore water pressure. This concept is used to construct the profile of pressure in the ground as a function of depth and is discussed next.

2.3 OVERBURDEN PRESSURE

Soils existing at a distance below ground are affected by the weight of the soil above that depth. The influence of this weight, known generally as **overburden**, causes a state of stress to exist, which is unique at that depth, for that soil. This state of stress is commonly referred to as the **overburden** or **in-situ** or **geostatic state of stress**. When a soil sample is removed from the ground, as during the field exploration phase of a project, that in-situ state of stress is relieved as all confinement of the sample has been removed. In laboratory testing, it is important to reestablish the in-situ stress conditions and to study changes in soil properties when additional stresses representing the expected design loading are applied. The stresses to be used during laboratory testing of soil samples are estimated from either the total or effective overburden pressure. The engineer's first task is determining the total and effective overburden pressure variation with depth. This relatively simple task involves estimating the average total unit weight for each soil layer in the soil profile, and determining the depth of the water table. Unit weight may be reasonably well estimated from tests on undisturbed samples or from standard penetration N-values and visual soil identification. The water table depth, which is typically recorded on boring logs, can be used to compute the hydrostatic pore water pressure at any depth. The **total overburden pressure, p_t,** is found by multiplying the total unit weight of each soil layer by the corresponding layer thickness and continuously summing the results with depth. The **effective overburden pressure, p_o,** at any depth is determined by accumulating the weights of all layers above that depth with consideration of the water level conditions at the site as follows:

<u>Soils above the water table</u>

- Multiply the total unit weight by the thickness of each respective soil layer above the desired depth, i.e., $p_o = p_t$.

<u>Soils below the ground water table</u>

- Compute pore water pressure u as $z_w \gamma_w$ where z_w is the depth below ground water table and γ_w is the unit weight of water

- To obtain effective overburden pressure, p_o, subtract pore water pressure, u, from p_t

- For soils below the ground water table, p_t is generally assumed to be equal to p_{sat}

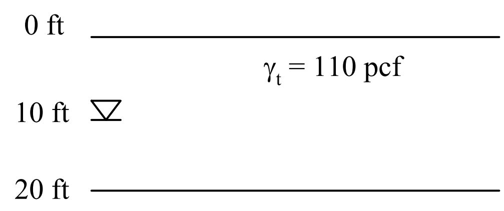
Alternatively, the following approach can be used:

- Reduce the total unit weights of soils below the ground water table by the unit weight of water (62.4 pcf (9.8 kN/m^3)), i.e., use effective unit weights, γ', and multiply by the thickness of each respective soil layer between the water table and the desired depth below the ground water table, i.e., $p_o = (\gamma_t - \gamma_w)$ (depth), or γ' (depth).

In the geotechnical literature, the effective unit weight, γ', is also known as the buoyant unit weight or submerged unit weight and symbols, γ_b or γ_{sub}, respectively are used.

An example is solved in Figure 2-7.

Example 2-1: Find p_o at 20 ft below ground in a sand deposit with a total unit weight of 110 pcf and the water table 10 ft below ground. Assume $\gamma_t = \gamma_{sat}$. Plot p_t and p_o versus depth from 0 ft – 20 ft.



Solution:

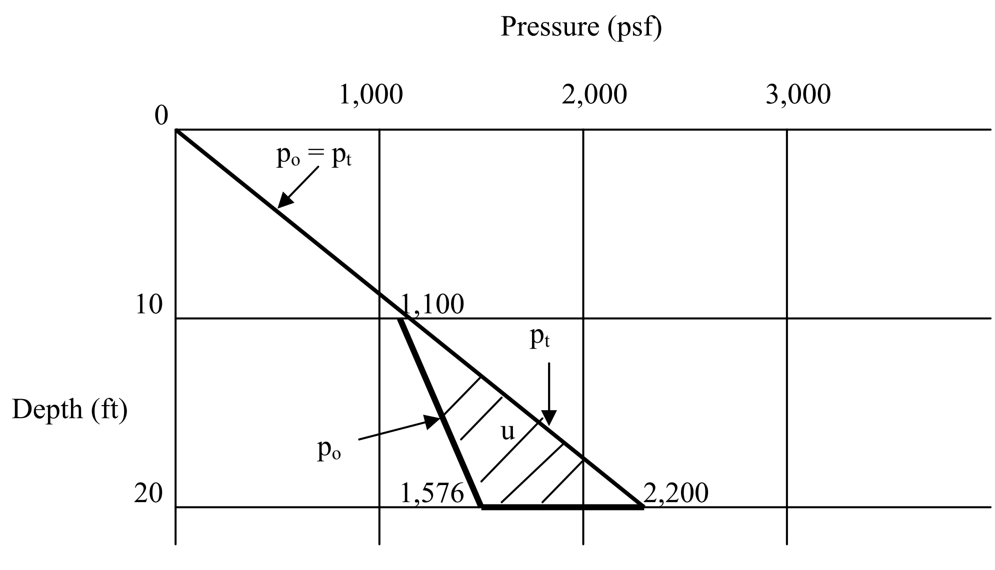
From Equation 2-13, $p_o = p_t - u$

p_t @ 10 ft = p_o @ 10 ft = 10 ft × 110 pcf = 1,100 psf

p_t @ 20 ft = p_t @ 10 ft + (10 ft × 110 pcf) = 2,200 psf

u @ 20 ft = 10 ft × 62.4 pcf = 624 psf

p_o @ 20 ft = p_t @ 20 ft - u @ 20 ft = 2,200 psf – 624 psf = 1,576 psf



Pressure Diagram

A plot of effective overburden pressure versus depth is called a "p_o – diagram" and is used throughout all aspects of geotechnical testing and analysis.

Figure 2-7. Example calculation of a p_o-diagram.

2.4 VERTICAL STRESS DISTRIBUTION IN SOIL DUE TO EXTERNAL LOADINGS

When a load is applied to the soil surface, it increases the vertical and lateral stresses within the soil mass. The increased stresses are greatest directly under the loaded area and dissipate within the soil mass as a function of distance away from the loaded area. This is commonly called spatial attenuation of applied loads. A schematic of the vertical stress distribution with depth along the centerline under an embankment of height, h, constructed with a soil having total unit weight, γ_t, is shown in Figure 2-8.

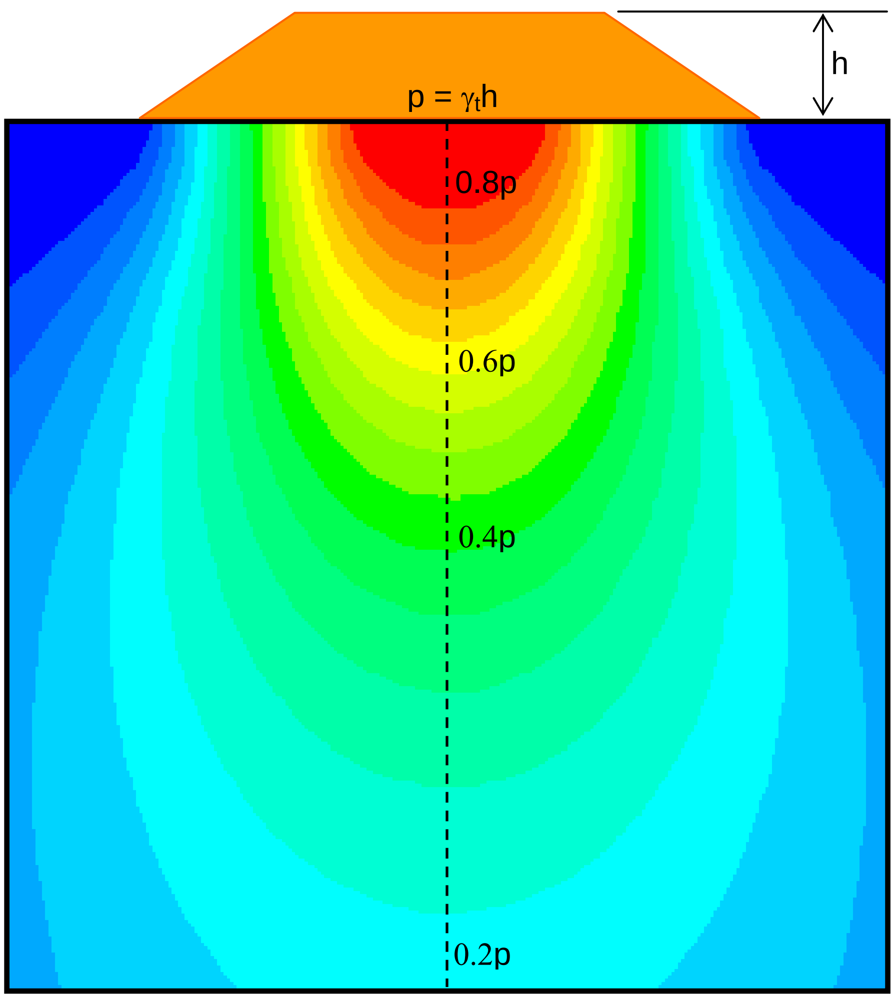


Figure 2-8. Schematic of vertical stress distribution under embankment loading. Graphic generated by FoSSA (2003) program.

(Note: Version 1.0 of FoSSA program is licensed to FHWA. See Appendix E for a brief overview of the FoSSA program).

Estimation of vertical stresses at any point in a soil mass due to external loadings are of great significance in the prediction of volume change of soils (e.g., settlement) under buildings, bridges, embankments and many other structures. The computation of the total vertical stress change induced by an external loading will depend on the configuration of the external loads. Common examples of the external loads are as follows:

- Uniform strip loads such as the load on a long wall footing of sufficient width,

- Uniformly loaded square, rectangular or circular footings such as column footings of buildings, pier footings, footings for water tanks, mats, etc., and

- Triangular and/or trapezoidal strip loads such as the loads of long earth embankments.

The theory of elasticity is often used to compute the stresses induced within a soil mass by external loadings. The most widely used elastic formulae were first developed by Boussinesq (1885) for point loads acting at the surface of a semi-infinite elastic half-space. These formulae, often known as **Boussinesq solutions**, can be integrated to give stresses below external loadings acting on a finite area. The basic assumptions in these formulae are (a) the stress is proportional to strain, (b) the soil is homogeneous (i.e., the properties are constant throughout the soil mass), and (c) the soil is isotropic (i.e., the properties are the same in all directions through a point). Westergaard (1938) modified the Boussinesq solutions by assuming that the semi-infinite elastic half-space is interspersed with infinitely thin but perfectly rigid layers that allow vertical movement but no lateral movement. In reality, a soil mass never fulfills the assumptions of either of these idealized solutions. Nevertheless, these elastic solutions, with appropriate modifications and judgment, have been found to yield acceptable approximate estimates of stresses in the soil mass and are widely used in geotechnical engineering practice. The Boussinesq solutions are generally used in most situations, even those where layered soils are encountered provided the thickness of the layers is on the order of a few feet or more. On the other hand, the Westergaard solutions are usually used for varved clays where the predominant soil mass is clay interspersed with thin layers of sand whose thickness is on the order of inches.

The derivations of the equations for various common loadings cited above are tedious. They are omitted in this manual so that the reader can concentrate on the use of published solutions, generally in the form of charts. The following sections contain the chart solutions for some of the loadings most commonly encountered in practice. Caution in the use of these charts is advised since they all pertain to **stress increments** at very well-defined points within the soil mass due to the applied pressures indicated. **The total stress acting at a point of interest is equal to the stress increment at that point due to the newly applied**

load plus existing stresses at that point due to the geostatic stress and stresses due to other external loads applied previously.

2.4.1 Uniformly Loaded Continuous (Strip) and Square Footings

A loaded area is considered to be infinitely long when its length, L, to width, B, ratio is greater than or equal to 10, i.e., $L/B \geq 10$. The load on such an area is commonly known as a strip load. Figure 2-9 presents vertical pressure isobars under strip and square footings based on Boussinesq's theory. An **isobar** is a line that connects all points of equal stress increment below the ground surface. In other words, an isobar is a stress increment contour.

Each isobar represents a fraction of the stress applied at the surface and delineates the zone of influence of the footing such that the area contained within two adjacent isobars experiences stresses greater than the lower isobar and less than the upper isobar. Since these isobars form closed figures that resemble the form of a bulb, they are also termed **bulbs of pressure** or simply the **pressure bulbs**. The pressure bulb concept gives the user a feel for the spread of the stresses through a soil mass.

According to linear elastic theory, the size of the pressure bulb is proportional to the size of the loaded area. This is a key concept in geotechnical engineering that is used to evaluate the **depth of significant influence, DOSI**, denoted by D_S of an applied surface load. The depth D_S is a finite depth below which there are no significant strains in the soil mass due to the loads imposed at the surface. Typically, strains are not significant once the stresses have attenuated to a value of 10 to 15% of those at the surface. For example, Figure 2-9a shows that for "infinitely long" strip footings, D_S = 4 to 6B, while for square footings, Figure 2-9b shows that D_S = 1.5 to 2B. The depths corresponding to this 10 to 15% criterion can be used to determine the minimum depth of field exploration for proposed strip or square footings to ensure that the anticipated significant depth is explored.

It may be seen from Figure 2-9 that the effect of the vertical stresses extends laterally beyond the width of the loaded area, B. This observation is very useful in assessing the influence of one loaded area on the other. Alternatively, this observation can be used to determine an adequate spacing between adjacent loaded areas. It also indicates that the effect of construction activities may be felt beyond a specific site. Such effects should be evaluated before construction so that mitigation measures can be taken to avoid legal implications.

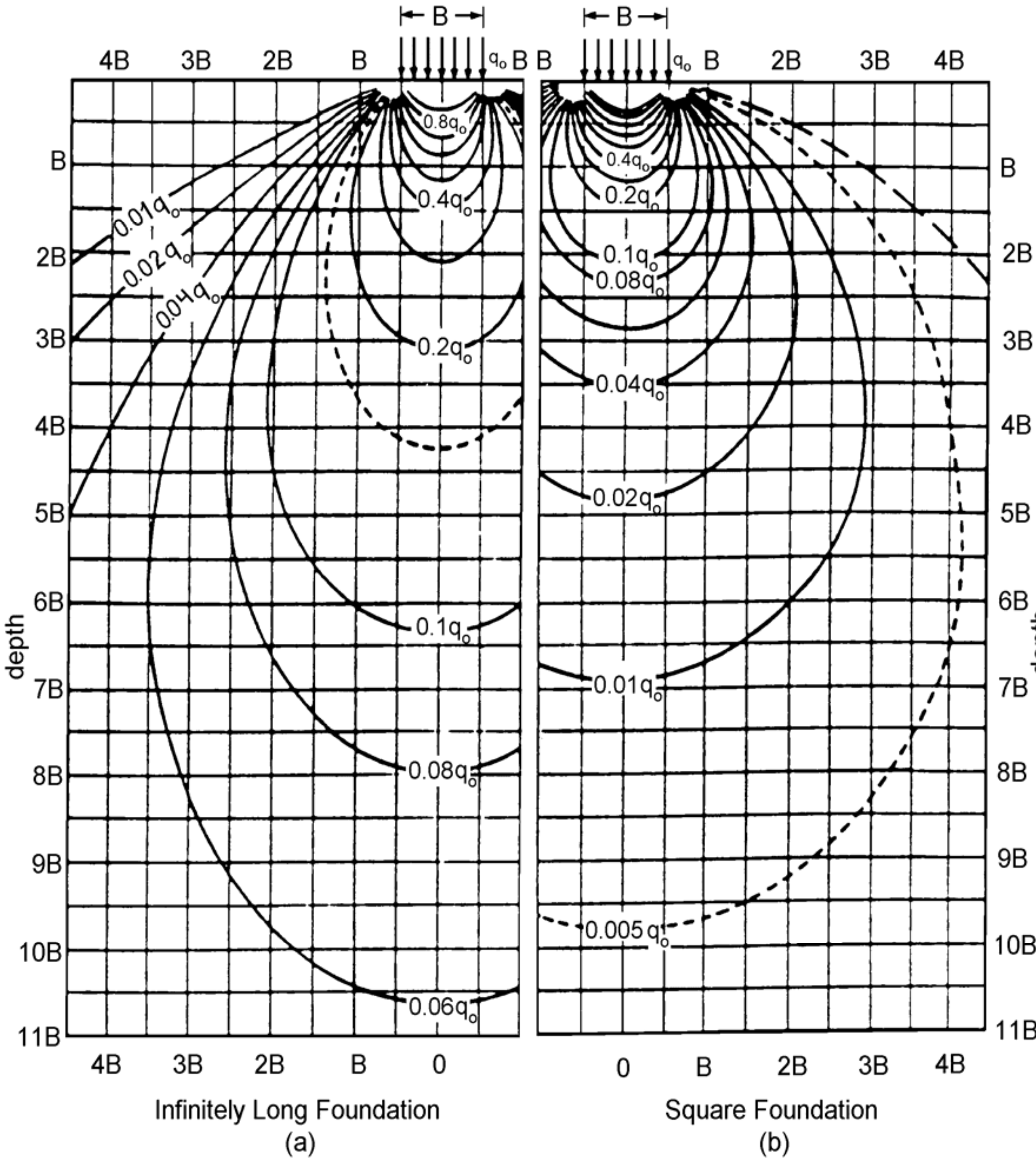


Figure 2-9. Vertical stress contours (isobars) based on Boussinesq's theory for continuous and square footings (modified after Sowers, 1979; AASHTO, 2002).

2.4.2 Approximate (2:1) Stress Distribution Concept

As an approximation to the exact solution given by the Boussinesq charts, the total load at the surface of the soil mass may be distributed over an area of the same shape as the loaded area on the surface, but with dimensions that increase with depth at a rate of one horizontal unit for every two vertical units. This is illustrated in Figure 2-10, which shows a rectangular area of dimensions B x L at the surface. At a depth, z, the total load is assumed to be uniformly distributed over an area (B+z) by (L+z). Since the stress is distributed at the rate of 2:1 (vertical:horizontal), this approximation method it is commonly known as the "**2:1 stress distribution**" method.

The relationship between the approximate distribution of stress determined by this method and the exact distribution is illustrated in Figure 2-10. In this figure, the vertical stress distribution at a depth B below a uniformly loaded square area of width B is shown along a horizontal line that passes beneath the center of the area and extends beyond the edges of the loaded area. Also shown is the approximate uniform distribution at depth B determined by the 2:1 stress distribution method described above. The discrepancy between the two methods decreases as the ratio of the depth considered to the size of the loaded area increases (Perloff and Baron, 1976).

2.5 REPRESENTATION OF IMPOSED PRESSURES ON THE p_o DIAGRAM

The pressure distributions computed by using the charts in Section 2.4, can be shown superimposed on the p_o diagram as shown in Figure 2-7. As discussed in the previous sections, an applied pressure at the surface causes stress increments within the soil mass that decrease with depth due to spatial attenuation. This is shown in Figure 2-11 where Δp is plotted with respect to the p_o line that represents the existing geostatic stress distribution. As can been seen in Figure 2-11, Δp approaches the p_o line, which indicates that at a sufficient depth the effect of the externally imposed loads reduces significantly. In other words, this means that most of the strain due to the increased stress from the applied load will be experienced at relatively shallow depths below the load. As noted earlier, this depth is known as the **depth of significant influence (DOSI)**, D_S. Also, as indicated previously, D_S depends on the load and load configuration as demonstrated by the pressure distribution charts in Section 2.4. Figure 2-11 also shows that that the **final stress, p_f, in the soil mass at any depth is equal to $p_o + \Delta p$**.

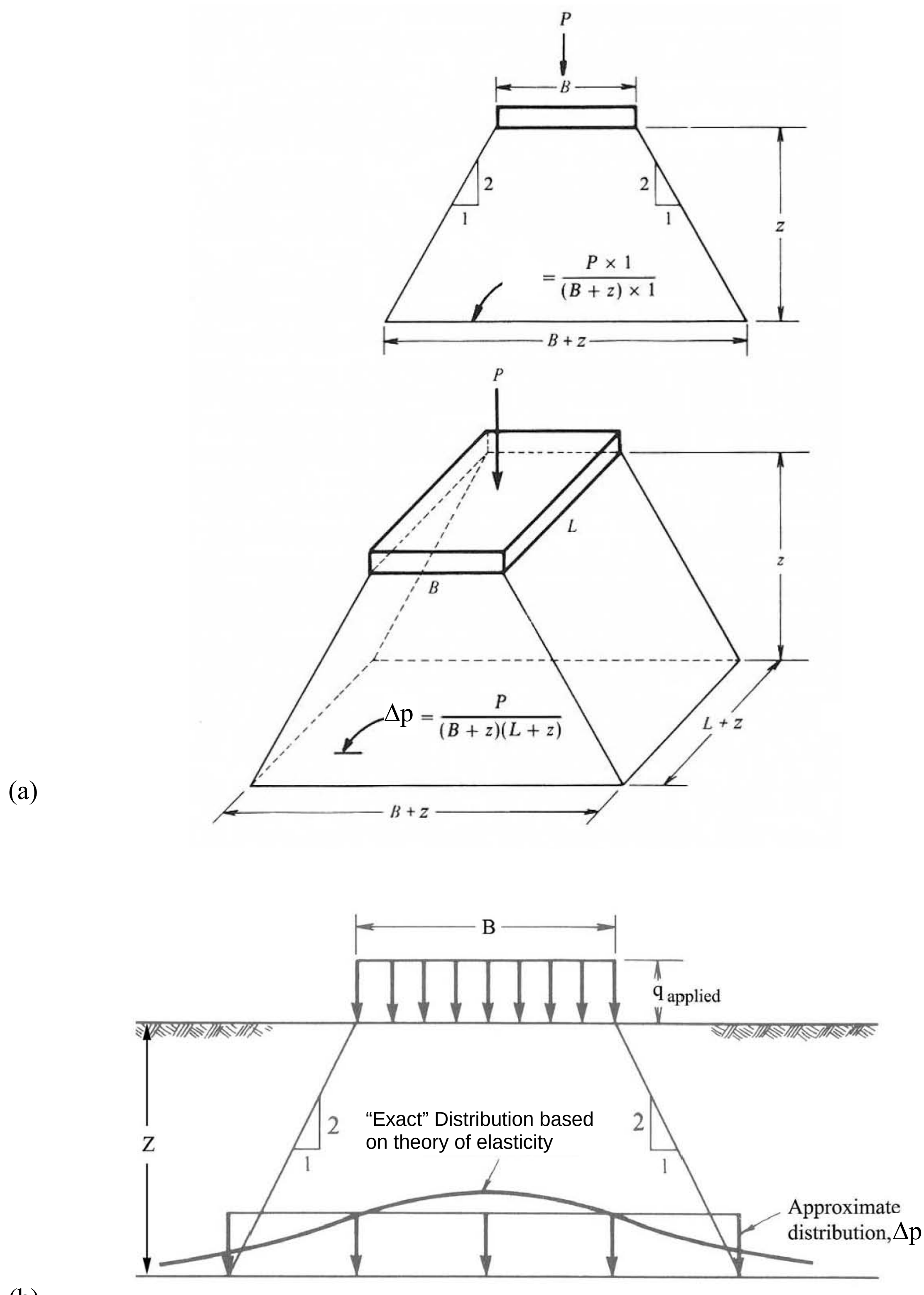


Figure 2-10. Distribution of vertical stress by the 2:1 method (after Perloff and Baron, 1976).

A chart such as that shown in Figure 2-11 is even more useful when the soil stratigraphy is plotted on it. Then the stress levels in various layers will be clearly identified, which can help the engineer determine depth of borings to collect subsurface information within DOSI as well as perform proper analysis.

Example 2-2 illustrates these concepts by providing calculations of p_f with depth due to stress increments from a strip load and presenting the results of the calculations on a p_o-diagram.

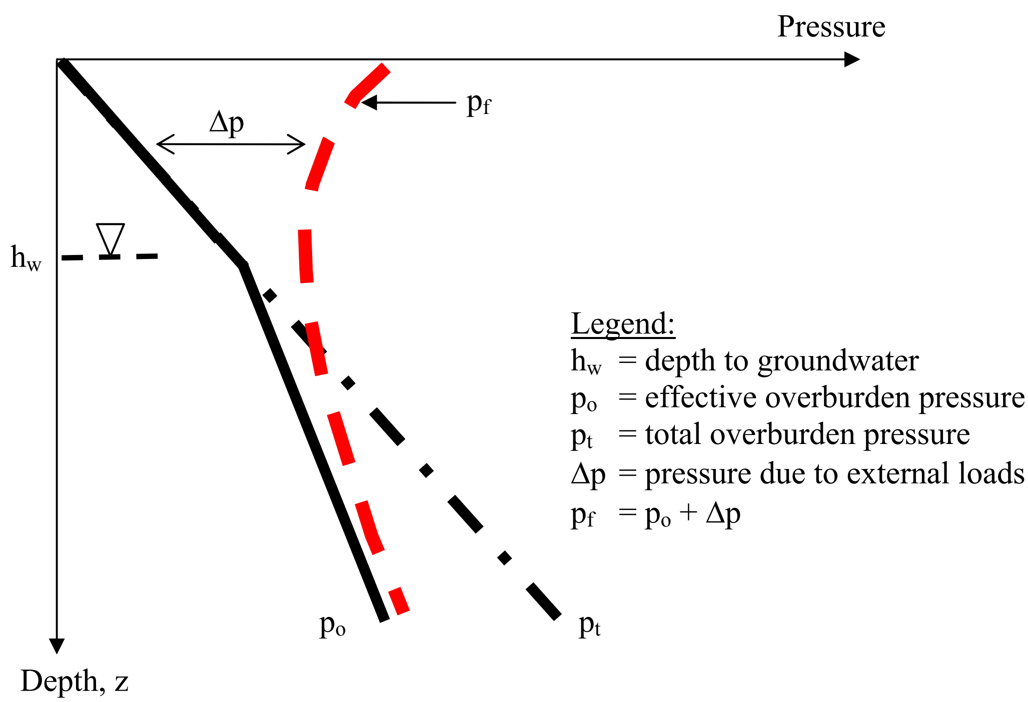
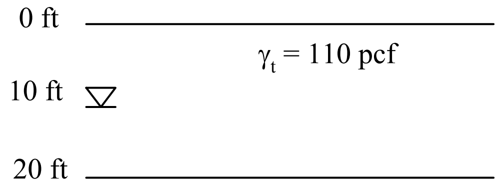


Figure 2-11. Combined plot of overburden pressures (total and effective) and pressure due to imposed loads.

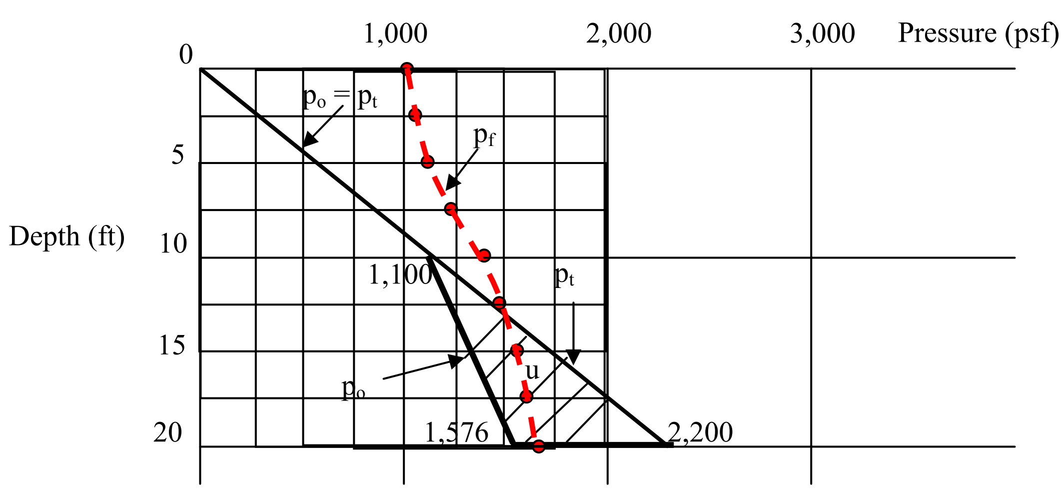
Example 2-2: For the Example 2-1 shown in Figure 2-7, assume that a 5 ft wide strip footing with a loading intensity of 1,000 psf is located on the ground surface. Compute the stress increments, Δp, under the centerline of the footing and plot them on the p_o diagram shown in Figure 2-7 down to a depth of 20 ft.



Solution:

For the strip footing, use the left chart in Figure 2-9. As per the terminology of the chart in Figure 2-9, B = 5 ft and q_o= 1,000 psf. Compile a table of stresses for various depths and plot as follows:

Depth z, ft	z/B	Isobar Value, x	Stress, Δp = x(q_o), psf	p_o, psf	p_f= p_o+Δp psf
2.5	0.5	0.80	800	(110)(2.5)=275	1,075
5.0	1.0	0.55	550	(110)(5.0)=550	1,100
7.5	1.5	0.40	400	(110)(7.5)=825	1,225
10.0	2.0	0.32	320	(110)(10.0)=1,100	1,420
12.5	2.5	0.25	250	1,100+(12.5-10.0)(110-62.4)=1,219	1,469
15.0	3.0	0.20	200	1,100+(15.0-10.0)(110-62.4)=1,338	1,538
17.5	3.5	0.18	180	1,100+(17.5-10.0)(110-62.4)=1,457	1,637
20.0	4.0	0.16	160	1,100+(20.0-10.0)(110-62.4)=1,576	1,736



Pressure Diagram

Figure 2-12. Example calculation of p_f with stress increments from strip load on p_o-diagram.

2.6 LOAD-DEFORMATION PROCESS IN SOILS

When subjected to static and/or dynamic loads, soils deform mainly because of a change in void volume rather than through deformation of the soil solids. When the void volume decreases the soil is said to compress, consolidate, collapse or compact. There is an important distinction between these four mechanisms although conceptually they appear to be the same since each pertains to a reduction in volume.

- ***Compression***: Compression is defined as a relatively rapid decrease in void volume that partially saturated (unsaturated) soils undergo as air is expelled from the voids during loading.

- ***Consolidation:*** Consolidation is generally defined as a time-dependent decrease in void volume that saturated and near-saturated soils undergo as water is expelled from the voids during loading. The conceptual process of consolidation is discussed in Section 2.6.1.

- ***Collapse:*** Collapse is primarily related to soil structure and its response to an increase in water content that results in a rapid decrease in void volume. Collapse-susceptible soils characteristically have dry densities less than approximately 100 pcf (16 kN/m^3) that suggest high void ratios. Their structure is like a honeycomb with fine-grained "bridges" connecting coarser-grained particles. When dry, these soils are able to sustain externally applied loads with very little deformation. However, upon being wetted they tend to undergo a rapid decrease in void volume as the fine-grained "bridges" lose strength and the entire structure collapses. The magnitude of the potential collapse increases with increasing load. One of the important things to note is that full saturation (S=100%) is not required for these types of soils to collapse. Often collapse occurs at a degree of saturation of 50 to 70%. Collapse-susceptible soils are very common in the southwest and midwest of the United States and in many other parts of the world.

- ***Compaction***: Compaction is the name given to the compression that takes place generally under an impact-type loading (e.g., modified and standard Proctor), a static loading (e.g., rubber-tired or steel drum rollers) or kneading-type loading (e.g., sheepsfoot roller). Most commonly, the compaction processes are deliberate and intended to achieve a dense packing of soil particles. Regardless of the type of loading, the moisture content of the soil being compacted is far enough below the saturation moisture content that the compaction mechanism is considered to be related to compression (i.e., expulsion of air) rather than consolidation (i.e., expulsion of water) from the voids. Typically, the desired moisture content in the case of compaction is slightly above or below the PL. If the moisture content of the soil being compacted gets to be too close to the saturation moisture content then "pumping" will occur, i.e. water in addition to air will be forced out of the soil.

These distinctions in load-deformation processes should be kept in mind during the discussions that follow in subsequent sections of this chapter.

Finally, in contrast to the above processes that involve void volume decrease, there are conditions under which soils may actually increase in volume. When the void volume increases under static and/or dynamic load, the soil is said to **dilate**. **Dilation** can occur in either saturated or partially saturated soils. It is a function of the initial void ratio, confinement stress, and the magnitude and direction of the loading/unloading imposed on the soil. **Expansion**, on the other hand, is generally considered to be due to the presence of expansive clay minerals, such as montmorillonite (commercially known as "bentonite"), in the soil and the response of these minerals to the introduction of water into the void spaces. The physico-chemical properties of expansive clay minerals cause inter-particle repulsions to take place in the presence of water so that even under considerable externally applied loads these soils will undergo an increase in void volume that leads to swelling. A variation of the expansion is **heave** which can occur due to various factors such as frost action or reduction in overburden pressure due to excavation.

2.6.1 Time Dependent Load-Deformation (Consolidation) Process

Deformation of a saturated soil is more complicated than that of a dry soil since water, which fills the voids, must be squeezed out of the soil before readjustment of the soil grains can occur. The more permeable a soil is, the faster the deformation under load will occur. However, when the load on a saturated soil is quickly increased, the increase is initially carried by the pore water resulting in the buildup of an **excess pore water pressure**, **Δu**. **Excess pore water pressure is water pressure greater than the hydrostatic pressure.** As drainage of the water takes place more and more load is gradually transferred from the pore water to the soil grains until the excess pore water pressure has dissipated completely and the soil grains readjust to a denser configuration under the applied load. This time dependent process is called **consolidation** and results in a decreased void ratio and greater unit weight relative to conditions before the load was applied. To illustrate this concept, one-dimensional (vertical) drainage of the water will be considered here. The process is analogous to loading a spring-supported piston in a cylinder filled with water. The spring-piston analogy is shown schematically in Figure 2-13 and is briefly discussed below.

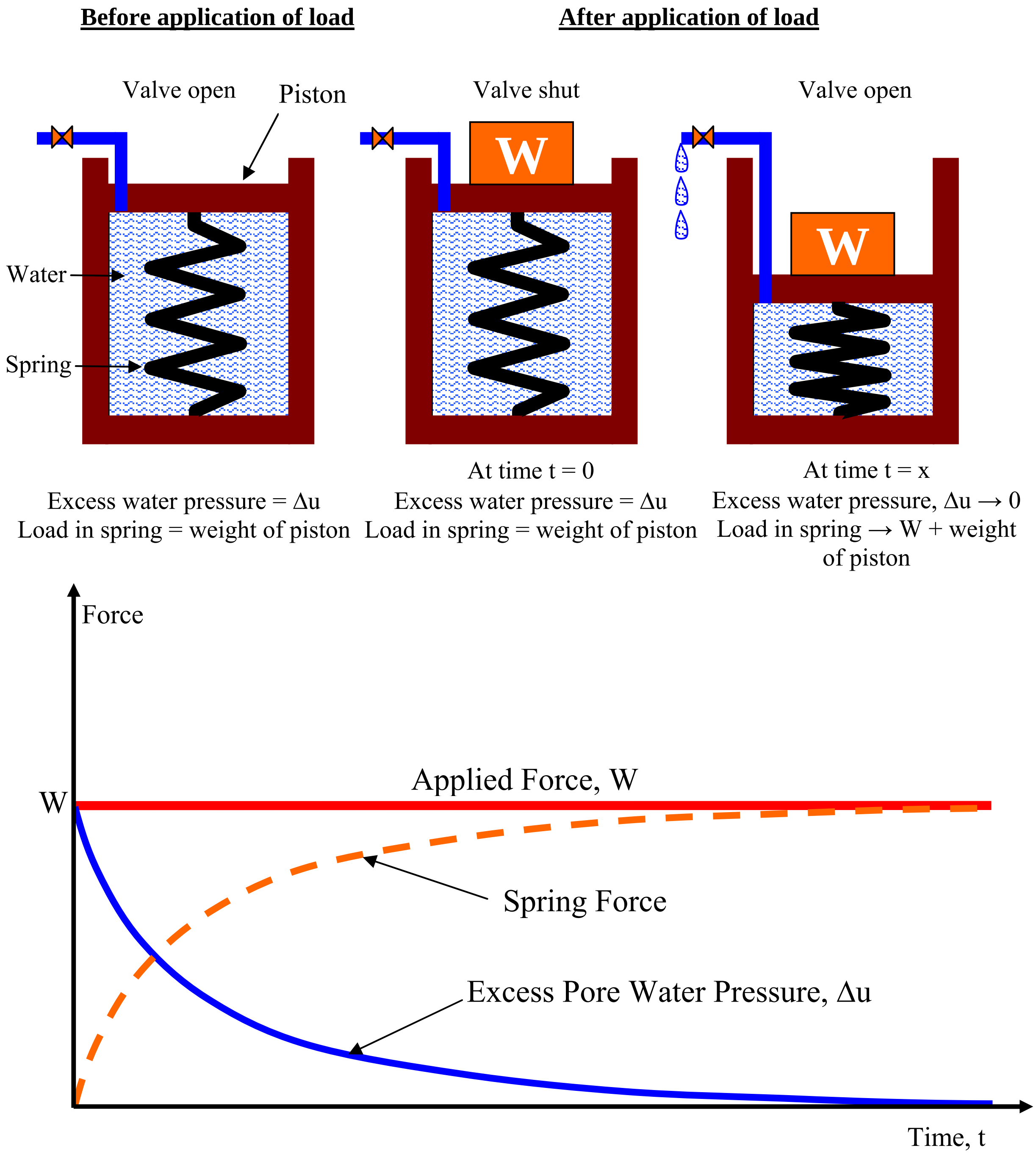


Figure 2-13. Spring-piston analogy for the consolidation process in fine-grained soils.

In the spring-piston model, the spring represents the solid phase of the soil and the water below the piston is the pore water under saturated condition in the soil mass. Before a new load, W, is applied to the piston, the system is assumed to be in equilibrium, i.e., the drainage valve is open and there is no excess pore water pressure, $\Delta u = 0$. The spring alone is carrying any previously applied loads, such as the weight of the piston itself. The drainage valve is closed just before the new load is applied. If the valve is completely shut-off and the piston is leak-proof, then, there is no chance for water to escape. Such a condition represents a clay-water system in which the clay is very impermeable so that there is significant resistance to drainage of water in any direction. When the new load, W, is placed on the piston (this is called the initial or "time = 0" condition), the total applied pressure immediately below the piston, p_t, which equals the load, W, divided by the area of the piston, is immediately transferred to the water. Since the drainage valve is closed and water is virtually incompressible, the water pressure increases to a value equal to the total applied pressure, i.e., the excess water pressure $\Delta u = p_t$.

At "time = 0," the spring does not carry any of the applied load W. The excess water pressure is analogous to the pore water pressure that would be developed in a clay-water system under externally applied loads, e.g., loads due to construction of an embankment on soft saturated clay. If the valve is now opened, the water will drain to relieve the excess pressure in it. With the escape of the water, a part of the pressure carried by the water is transferred to the spring where it induces a stress increase analogous to an effective increase in the inter-particle stresses, p_o, in a soil mass. The transfer of pressure from the water to the spring occurs over a period of time as shown on the bottom part of Figure 2-13, however, at any time during the process, the increased stress in the spring, p_o, plus the excess pressure in the water, Δu, must equal the applied pressure, p_t. This transfer of pressure from the water to the spring goes on until the flow stops. At that time all of the applied pressure, p_t, will be carried by the spring, p_o, and none by the water, i.e., $\Delta u = 0$, and the system will have come into equilibrium under the applied load. The time required to attain equilibrium depends on the avenue provided to the water to escape, i.e., the longest drainage path the water has to take to leave the system. In Figure 2-13 the longest drainage path is the length of the cylinder. Obviously, the system would drain quicker if there were another standpipe-type drain at the bottom of the cylinder.

Regardless of the number of avenues provided for drainage, the rate of excess water pressure drop generally decreases with time as shown in the lower half of Figure 2-13. After the spring water system attains an equilibrium condition under the imposed load, the compression of the piston is analogous to the settlement of the clay-water system under an externally applied load. This process is called **consolidation**.

2.6.2 Comparison of Drainage Rates between Coarse-Grained and Fine-Grained Soils

Figure 2-14 shows a comparison of excess pore water pressure dissipation in coarse-grained and fine-grained soils. The relatively large pore spaces in coarse-grained soils permit the water to drain quicker in comparison to fine-grained soils. This leads to a quick transfer of applied loads to the soil solids with an associated decrease in void space. This quick load transfer results in a displacement that is commonly termed "rapid" in contrast to the "long-term" displacement that is associated with the consolidation process in fine-grained soils.

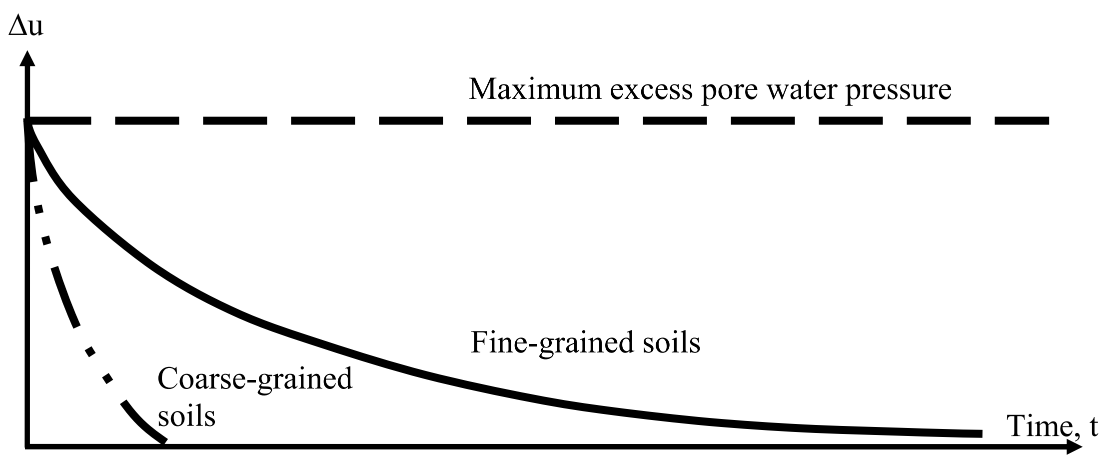


Figure 2-14. Comparison of excess pore water pressure dissipation in coarse-grained and fine-grained soils.

2.7 LATERAL STRESSES IN FOUNDATION SOILS

In most cases, the vertical stress at any depth in a soil mass due to its self weight is the summation of the simple products of the unit weight of each soil layer and its corresponding thickness down to the depth of interest. This vertical stress was denoted by p_t and the effective component of this pressure was denoted by p_o. Due a variety of factors, including depositional patterns, the lateral stress, p_h, in a soil mass is usually not the same as the vertical stress, p_o. Since the vertical stress is known with reasonable certainty for practical purposes, the lateral stress can be assumed to be a certain percentage of the vertical stress and can be expressed as follows:

$$p_h = K\, p_o \qquad \text{2-14}$$

For an elastic solid, the value of the proportionality constant, K, can be expressed in terms of Poisson's ratio, ν, as follows:

$$K = \frac{\nu}{1-\nu} \qquad 2\text{-}15$$

Poisson's ratio, ν, is defined as a ratio of lateral to vertical strains. The value of Poisson's ratio is a function of the type of material, e.g., ν is practically zero for cork (hence its suitability as a bottle stopper), for concrete ν is between 0.1 and 0.2, and for steel ν is between 0.27 and 0.30. A theoretical upper limit of Poisson's ratio is 0.5 (rubber comes close to this limiting value). In the case of soils, ν will have a different value depending upon the type of soil and its moisture condition. For example, for free-draining soils a reasonable value of ν would be in the range of 0.25 to 0.35, while for very soft saturated clays under rapid loading conditions the value of ν would be close to 0.5. Thus, for free-draining soils, the value of K based on elasticity theory will range from 33% to 54% corresponding to ν=0.25 and ν=0.35, respectively, while for soft clays the value of K ≈ 1 since $\nu \approx 0.5$.

Even though a soil mass is not an elastic body, the point to be noted here is that at any point within the soil mass both vertical and horizontal (or lateral) stresses exist. When external forces are imposed on a soil mass, they will result in an increase in vertical stresses as discussed in Sections 2.5 and 2.6. Equation 2-14 indicates that an increase in vertical stresses will in turn lead to an increase in lateral stresses. While the increase in vertical stresses is important in assessing vertical settlements, change in lateral stresses may affect the load acting, for example, against piles supporting a bridge abutment, see Figure 2-15. In this figure, it can be seen that the increase in vertical stress imposed by the embankment leads to an increase in the lateral stress in the ground that causes lateral deformation ("squeeze") of the soft soil. As the soft soil spreads laterally it will have an effect on foundations. Therefore, it is important to evaluate the increase in lateral stresses due to vertical loadings.

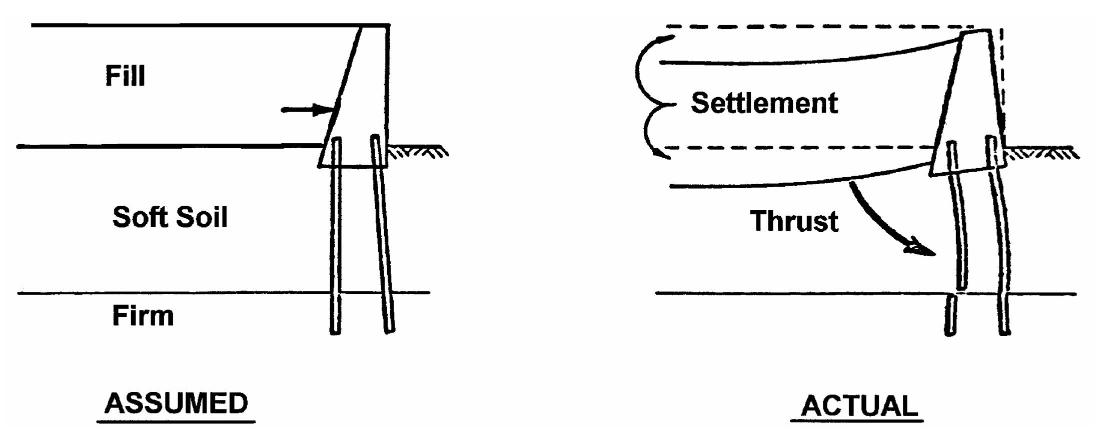


Figure 2-15. Schematic of effect of lateral stresses.

A two-dimensional (2-D) representation of the lateral stresses transverse to an embankment centerline is shown in Figure 2-16. This schematic was developed with a soft layer of soil under the embankment. It can be seen that significant lateral stresses are generated in the soil below the embankment load. Note that the vertical stresses due to the embankment can cause zones of tensile stresses to develop near the toes of the embankment as shown by the hatched zones in Figure 2-16. This means that tensile cracks are likely to develop near the toes of the embankments for this particular case. This knowledge can help the geotechnical specialist to select proper ground improvement measures rationally and to develop and implement an instrumentation program. The key point to understand based on the schematics shown in Figures 2-15 and 2-16 is that lateral deformations can be three-dimensional and can affect a number of facilities such as buried utilities, embankment slopes and bridge foundations. Lateral deformations can also affect off-site structures very easily leading to potential legal actions. The three-dimensional (3-D) lateral deformations coupled with vertical deformations due to vertical stresses can create a complex state of deformation that needs to be carefully considered in the design of geotechnical features.

Similar to the estimation of vertical stresses, the theory of linear elasticity yields equations for lateral stress distribution. However, in these equations Poisson's ratio is assumed to be a constant. Hence, the use of chart solutions in these cases is not as simple as for the vertical stress case since complicated equations have to be evaluated (Poulos and Davis, 1974). One can prepare spreadsheet solutions based on the equations or use commercially available computer programs that have already programmed the equations. Program FoSSA (2003) by ADAMA Engineering (Version 1.0 was licensed to FHWA) is an example of a program capable of computing the vertical and lateral stresses due to surface loading, including embankment and multiple footings. Figures 2-10 and 2-16 were generated using the FoSSA program.

2.7.1 Effect of Shear Strength of Soils on Lateral Pressures

Up to now the stresses in soils have been explained by using unit weights and the theory of elasticity. Elastic theory, when suitably modified to reflect observed phenomena in soils, provides a tool to obtain a reasonable first approximation to a solution for many problems in geotechnical engineering. However, elastic theory does not recognize the role of shear strength of soil in the development of lateral pressures. For example, soils have an ability to stand vertically or at a certain slope. The reason for this observed ability is that soil has shear strength and to some degree can support itself. This shear strength may come from friction and/or cohesion between the soil particles. It is intuitive that these components of shear strength should also somehow affect the lateral pressures in soils computed by use of the theory of elasticity. The shear strength of soils and its representation for analytical purposes is discussed in the Section 2.8 followed in Section 2.9 by a demonstration of how the shear strength parameters can be used to express lateral pressures. Readers are referred to Lambe and Whitman (1979) or Holtz and Kovacs (1981) for detailed discussions.

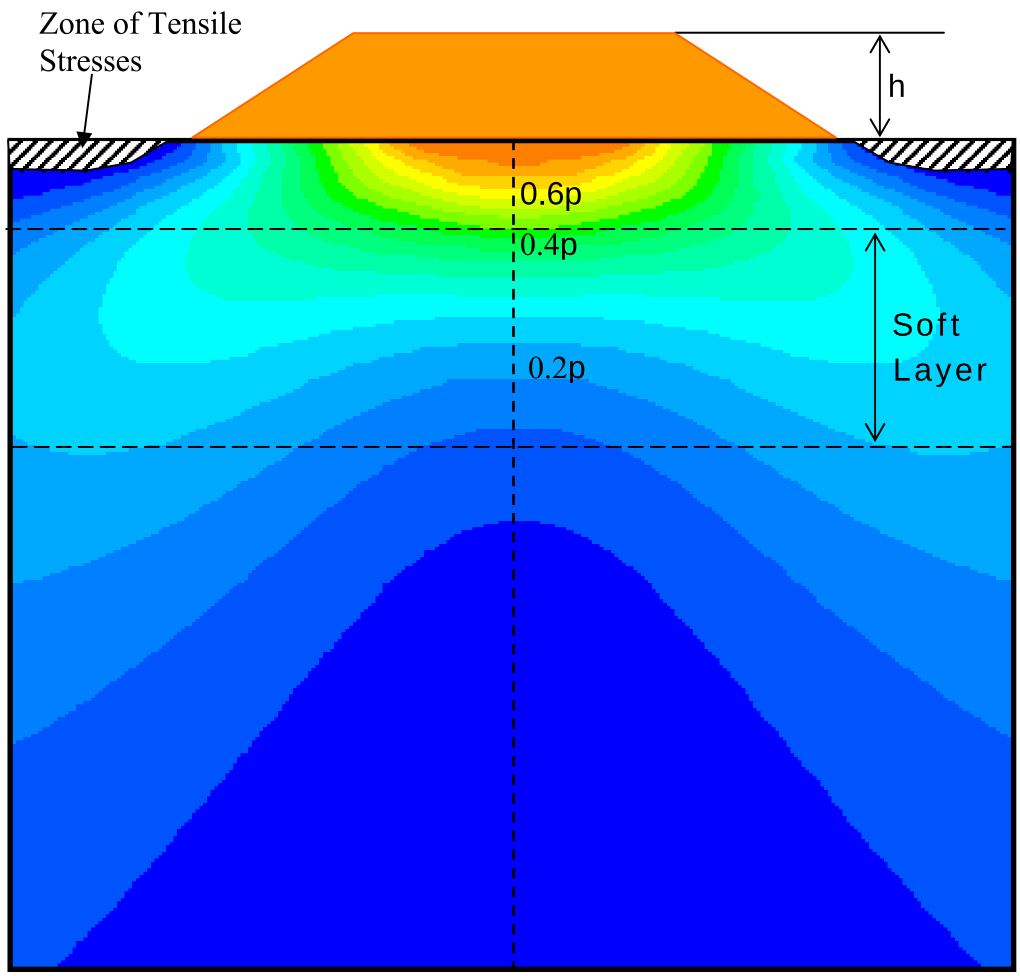


Figure 2-16. Schematic of vertical stress distribution under embankment loading. Graphic generated by FoSSA (2003) program.

(Note: Version 1.0 of FoSSA program is licensed to FHWA. See Appendix E for a brief overview of the FoSSA program).

2.8 STRENGTH OF SOILS TO RESIST IMPOSED STRESSES

If the imposed stress in a soil mass is increased until the deformations (movements) become unacceptably large, a "failure" is considered to have taken place. In this case, the strength of the soil is considered to be insufficient to withstand the applied stress.

The strength of geologic materials is a variable property that is dependent on many factors, including material properties, magnitude and direction of the applied forces and their rate of application, drainage conditions of the mass, and the magnitude of confining pressure. Unlike steel whose strength is usually discussed in terms of either tension or compression and concrete whose strength is generally discussed in terms of compressive strength only, the strength of soil is generally discussed in terms of shear strength. Typical geotechnical failures occur when the shear stresses induced by applied loads exceed the soil's shear strength somewhere within the soil mass.

2.8.1 Basic Concept of Shearing Resistance and Shearing Strength

The basic concept of shearing resistance and shearing strength can be understood by first studying the principle of friction between solid bodies. Consider a prismatic block B resting on a plane surface XY as shown in Figure 2-17. The block B is subjected to two forces:

- A normal force, P_n, that acts perpendicular to the plane XY, and
- A tangential force, F_a, that acts parallel to the plane XY.

Assume that the normal force, P_n, is constant and that the tangential force, F_a, is gradually increased. At small values of F_a, the block B will not move since the applied force, F_a, will be balanced by an equal and opposite force, F_r, on the plane of contact XY. The resisting force, F_r, is developed as a result of surface roughness on the bottom of the block B and the plane surface XY. The angle, θ, formed by the resultant R of the two forces F_r and P_n with the normal to the plane XY is known as the **angle of obliquity**.

If the applied horizontal force, F_a, is gradually increased, the resisting force, F_r, will likewise increase, always being equal in magnitude and opposite in direction to the applied force. When the force F_a reaches a value that increases the angle of obliquity to a certain maximum value θ_m, the block B will start sliding along the plane. Recall that during this entire process the normal force, P_n, remains constant. The following terminology can now be developed:

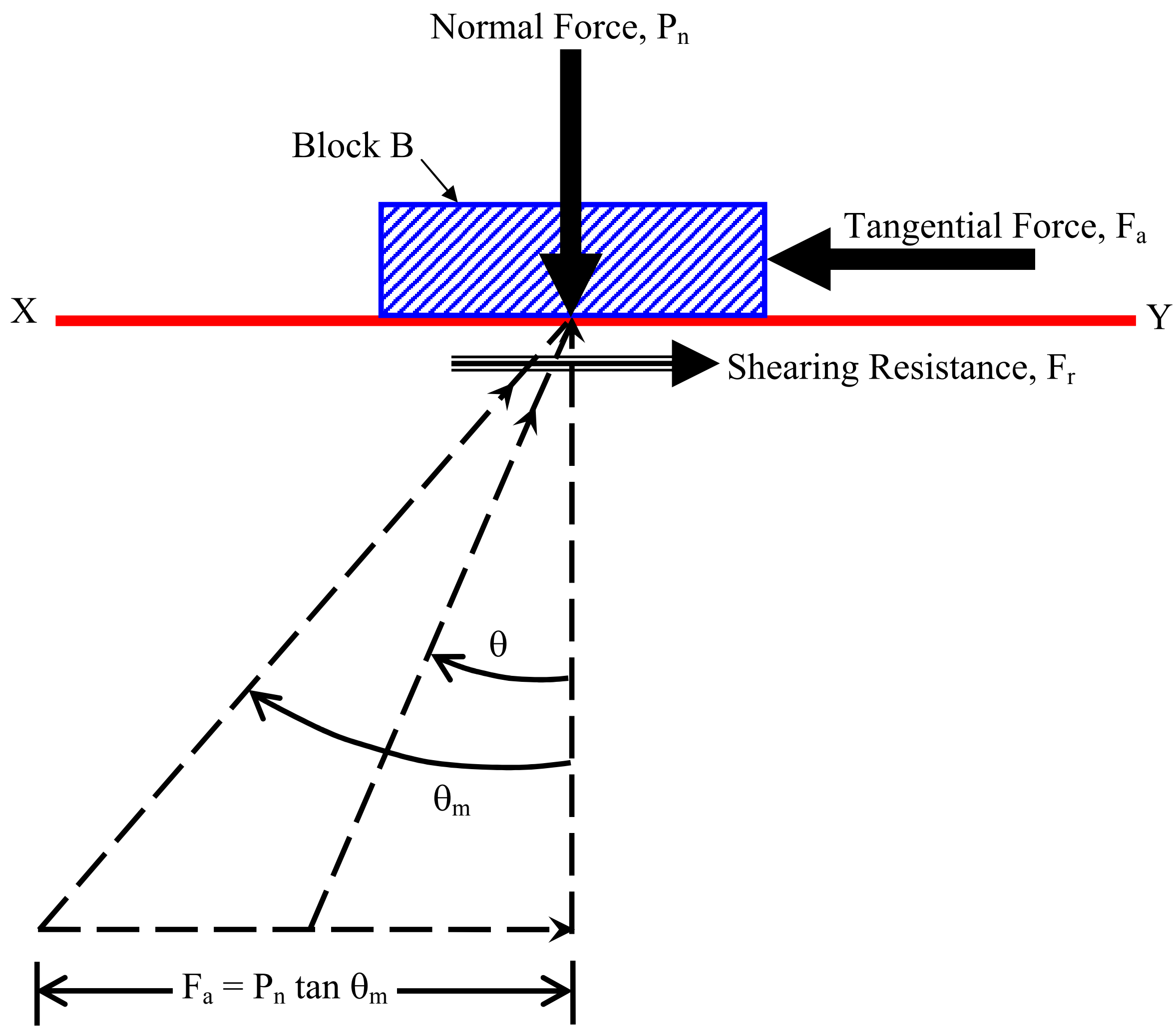


Figure 2-17. Basic concept of shearing resistance and strength (after Murthy, 1989).

- If the block B and the plane surface XY are made of the same material, the angle θ_m is equal to ϕ, which is termed the **angle of friction** of the material. The value tan ϕ is called the **coefficient of friction**.

- If the block B and the plane surface XY are made of dissimilar materials, the angle θ_m is equal to δ, which is termed the **angle of interface friction** between the bottom of the block and the plane surface XY. The value tan δ is called the **coefficient of interface friction**.

- The applied horizontal force, F_a, on the block B is a shearing force and the developed force is called **frictional resistance** or **shearing resistance**. The maximum frictional or shearing resistance that the materials are capable of developing on the interface is $(F_a)_{max}$.

If the same experiment is conducted with a greater normal force, P_n, the maximum frictional or shearing resistance $(F_a)_{max}$, will be correspondingly greater. A series of such experiments would show that for the case where the block and surface are made of the same material, the maximum frictional or shearing resistance is approximately proportional to the normal load P_n as follows:

$$(F_a)_{max} = P_n \tan \phi \tag{2-16}$$

If A is the overall contact area of the block B on the plane surface XY, the relationship in Equation 2-16 may be written as follows to obtain stresses on surface XY:

$$\frac{(F_a)_{max}}{A} = \left(\frac{P_n}{A}\right)\tan\phi \tag{2-17}$$

or

$$\tau = \sigma_n \tan\phi \tag{2-18}$$

The term σ_n is called the **normal stress** and the term τ is called the **shear strength**. A graphical representation of Equation 2-18 is shown in Figure 2-18a. In reality, the relationship is curved, but since most geotechnical problems involve a relatively narrow range of pressures, the relationship is assumed to be linear as represented by Equation 2-18 over that range.

The concept of frictional resistance explained above applies to soils that possess only the frictional component of shear strength, i.e., generally coarse-grained granular soils. But soils that are not purely frictional exhibit an additional strength component due to some kind of internal electro-chemical bonding between the particles. This bonding between the particles is typically found in fine-grained soils and is termed **cohesion, c**. Simplistically, the shear strength, τ, of such soils is expressed by two additive components as follows and can be graphically represented as shown in Figure 2-18(b):

$$\tau = c + \sigma_n \tan\phi \tag{2-19}$$

Again, in reality, the relationship is curved. But, as noted above, since most geotechnical problems involve a relatively narrow range of pressures, the relationship is assumed to be linear as represented by Equation 2-19 over that range.

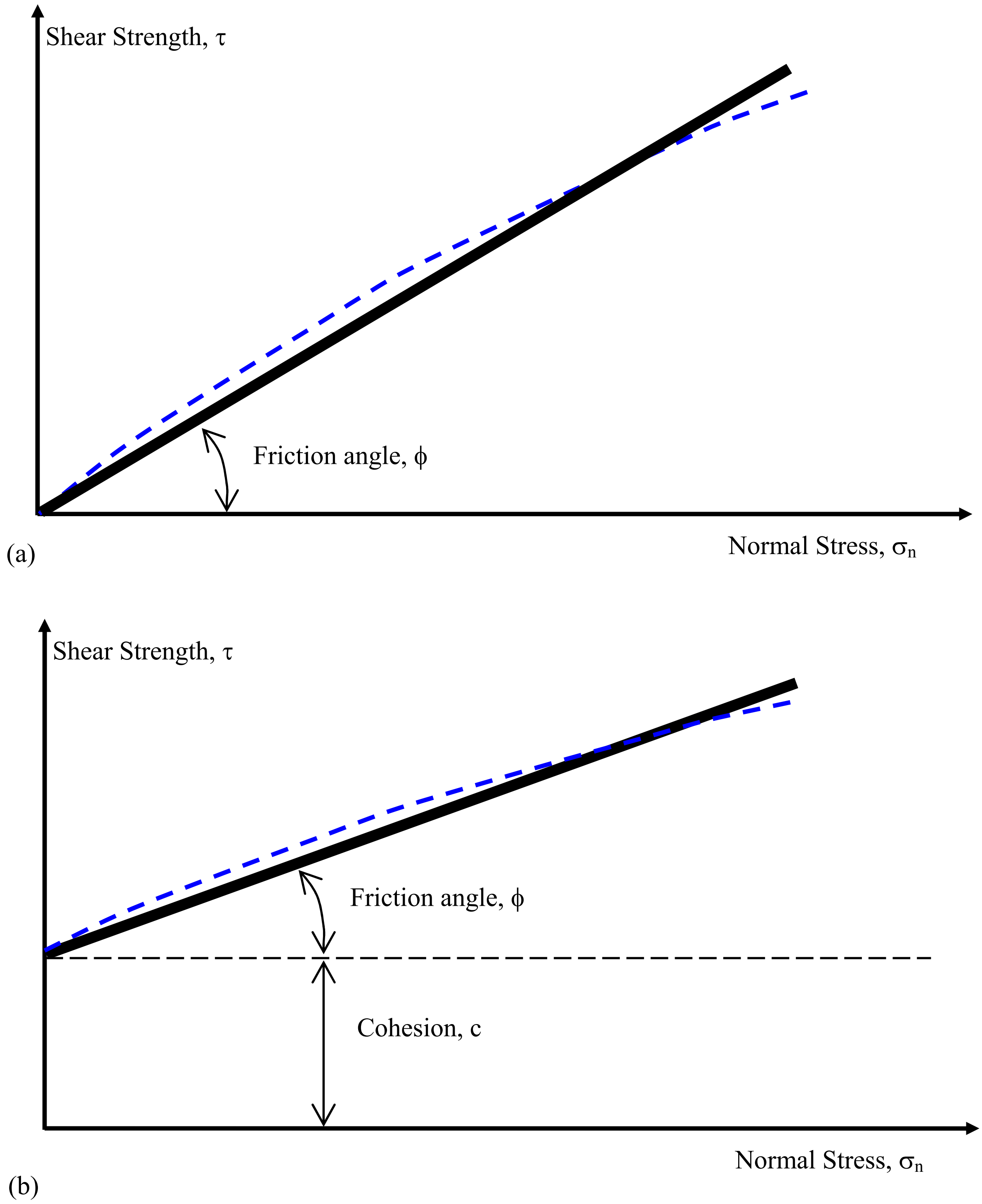


Figure 2-18. Graphical representation of shearing strength.

Equation 2-19 was first proposed by French engineer Coulomb and is used to express shear strength of soils. When plotted on arithmetic axes the resulting straight line is conventionally known as the Mohr-Coulomb (M-C) failure envelope. “Mohr” is included in “Mohr-Coulomb” because Equation 2-19 can also be derived based on concept of Mohr’s circle. The development of the Mohr-Coulomb failure envelope based on the application of Mohr’s circle is presented in Appendix B.

As indicated previously, the deformation of soils occurs under effective stresses. In terms of effective stresses, Equation 2-19 can be re-written as follows:

$$\tau' = c' + (\sigma_n - u) \tan \phi' = c' + \sigma' \tan \phi' \qquad 2\text{-}20$$

where c' = effective cohesion, σ' is the effective normal stress and ϕ' is the effective friction angle. Further discussion on the cohesion and friction angle is presented in Chapter 4.

In geotechnical engineering, the normal stresses are commonly expressed using the overburden pressure concept introduced in Section 2.3. In terms of overburden pressure, the term σ_n in above equations is the same as p_t and the term σ' is the same as p_o. Thus, Equations 2-19 and 2-20 can be expressed in terms of overburden stresses as follows:

$$\tau = c + p_t \tan\phi \qquad 2\text{-}21$$

$$\tau' = c' + (p_t - u) \tan \phi' = c' + p_o \tan \phi' \qquad 2\text{-}22$$

Since this manual relates to geotechnical engineering, Equations 2-21 and 2-22 will be used to express the M-C failure envelope. The physical meaning of the M-C failure envelope shown in Figure 2-18(a) and Figure 2-18(b) may be explained as follows:

- Every point on the M-C failure envelope represents a combination of normal and shear stress that results in failure of the soil, i.e., the Mohr failure envelope essentially defines the strength of the soil. In other words, **any point along the M-C envelope defines the limiting state of stress for equilibrium**.

- If the state of stress is represented by a point below the M-C failure envelope then the soil will be stable for that state of stress.

- States of stress beyond the M-C failure envelope cannot exist since failure would have occurred before that point could be reached.

2.9 STRENGTH OF SOILS RELATED TO LATERAL EARTH PRESSURES

The concept of shear strength described in the previous section can now be used to understand the phenomenon of lateral earth pressure in a soil mass, which is related to problems of slope stability and earth retention. From a theoretical viewpoint, problems in these three areas (earth pressures, slope stability, and retaining structures) fall into a class of problems involving plasticity theory and are best solved by some form of equilibrium solution. Many geotechnical engineering text books (e.g., Lambe and Whitman, 1979; Holtz and Kovacs, 1981) deal with these solutions extensively. From a practical viewpoint, values of earth pressure are needed either directly or indirectly to determine:

a) If an unrestrained slope is stable and
b) If not, what kind of retaining structure will be required to stabilize the slope.

The simplest consideration of earth pressure theory starts with the assessment of the vertical geostatic effective stress, p_o, at some depth in the ground (effective overburden pressure) as considered in Section 2.3. The lateral geostatic effective stress, p_h, at this depth is given in general terms by Equation 2-14 where, for an ideally elastic solid, the value of the lateral earth pressure coefficient, K, is given by Equation 2-15. However, the behavior of real soils under loads is not always ideally elastic. To simplify the discussion of this topic, consider only dry coarse-grained cohesionless soils. The geostatic effective stress condition on a soil element at any depth, z, is shown in Figure 2-19a. Since the ground is "at-rest" without any external disturbance, this condition is commonly referred to as the "**at-rest**" condition with zero deformation. The coefficient of lateral earth pressure for this condition is labeled $\mathbf{K_o}$.

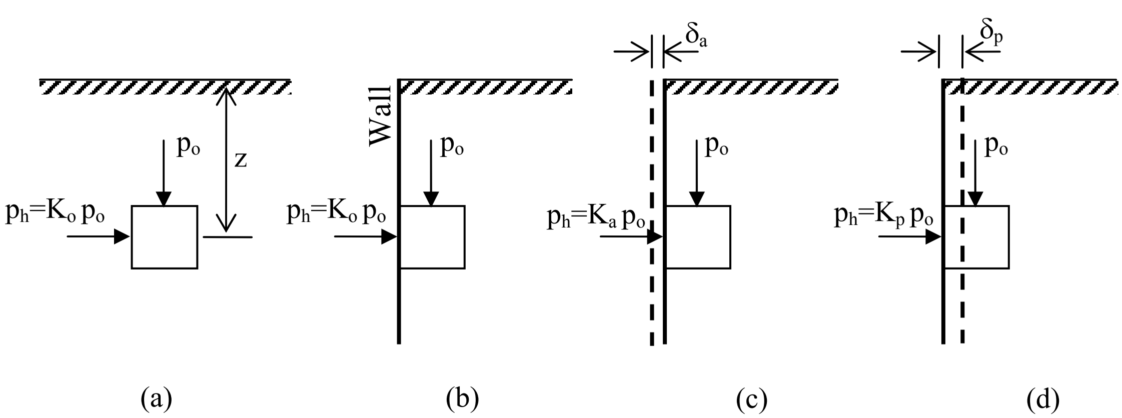


Figure 2-19. Stress states on a soil element subjected only to body stresses: (a) In-situ geostatic effective vertical and horizontal stresses, (b) Insertion of hypothetical infinitely rigid, infinitely thin frictionless wall and removal of soil to left of wall, (c) Active condition of wall movement away from retained soil, (d) Passive condition of wall movement into retained soil.

To relate to the lateral earth pressures acting on retaining structures, assume that a hypothetical, infinitely thin, infinitely rigid "wall" is inserted into the soil without changing the "at rest" stress condition in the soil. For the sake of discussion, assume that the hypothetical wall maintains the "at rest" stress condition in the soil to the right of the wall when the soil to the left of the wall is removed. This condition is shown in Figure 2-19b. Now suppose that the "at rest" condition is removed by allowing the hypothetical vertical wall to move slightly to the left, i.e., away from the soil element as shown in Figure 2-19c. In this condition, the vertical stress would remain unchanged. However, since the soil is cohesionless and cannot stand vertically on its own, it actively follows the wall. In this event, the horizontal stress decreases, which implies that the lateral earth pressure coefficient is less than K_o since the vertical stress remains unchanged. When this occurs the soil is said to be in the "**active**" state. The lateral earth pressure coefficient at this condition is called the "**coefficient of active earth pressure," K_a**, and its value at failure is expressed in terms of effective friction angle, ϕ', as follows:

$$K_a = \frac{1 - \sin\phi'}{1 + \sin\phi'} \tag{2-23}$$

Returning to the condition shown in Figure 2-19b, now suppose that the "at rest" condition is removed by moving the hypothetical vertical wall to the right, i.e., into the soil element as shown in Figure 2-19d. Again, the vertical stress would remain unchanged. However, the soil behind the wall passively resists the tendency for it to move, i.e., the horizontal stress would increase, which implies that the lateral earth pressure coefficient would become greater than K_o since the vertical stress remains unchanged. When this occurs the soil is said to be in the "**passive**" state. The lateral earth pressure coefficient at this condition is called the "**coefficient of passive earth pressure," K_p**, and its value at failure is expressed in terms of effective friction angle, ϕ', as follows:

$$K_p = \frac{1 + \sin\phi'}{1 - \sin\phi'} \tag{2-24}$$

When failure occurs during either of the two processes described above, **"Rankine" failure zones** form within the soil mass. The details of how the failure zones develop are described in most geotechnical engineering textbooks and will not be treated here. The so-called "Rankine" failure zones and their angles from the horizontal are shown in Figure 2-20.

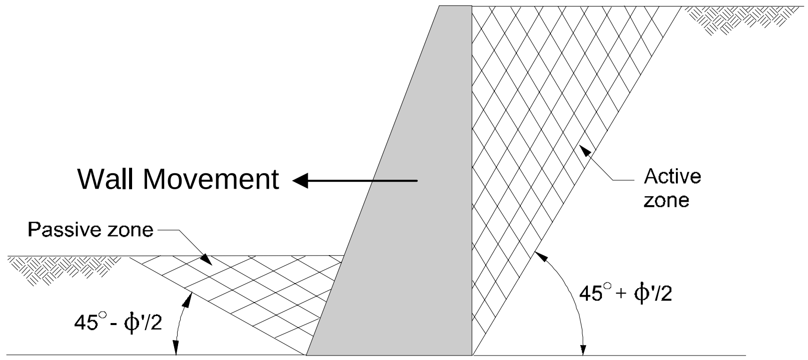


Figure 2-20. Development of Rankine active and passive failure zones for a smooth retaining wall.

2.9.1 Distribution of Lateral Earth and Water Pressures

The earth pressure coefficients, K_a and K_p, can be substituted into Equation 2-14 to obtain equations for active and passive lateral earth pressures, respectively as follows:

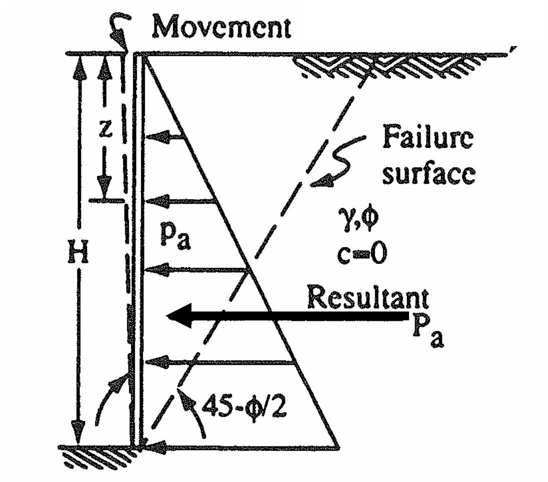
$$p_a = K_a \, p_o \qquad \text{2-25a}$$

$$p_p = K_p \, p_o \qquad \text{2-25b}$$

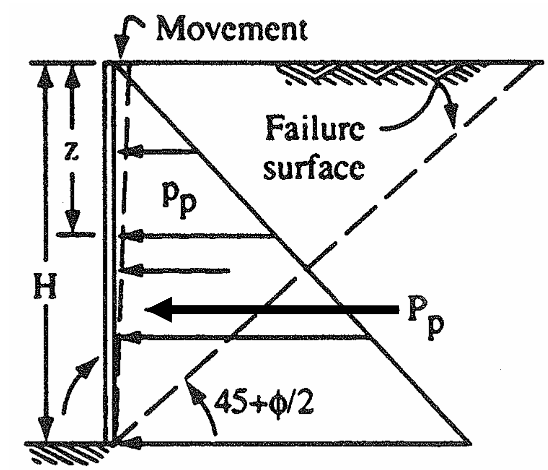
It can be seen from Equations 2-25a and 2-25b that the lateral pressures p_a and p_p are a certain fraction of the vertical <u>effective</u> overburden pressure p_o. Thus, active and passive lateral earth pressures are effective pressures and their distribution will be same as that for p_o. The overburden pressure increases in proportion to the unit weight and is typically triangular for a given geomaterial. The general distribution of the active and passive pressures along with the configuration of active and passive failure surfaces is shown in Figure 2-21a and 2-21b, respectively.

In cases where ground water exists, the lateral pressure due to the water at any depth below the ground water level is equal to the hydrostatic pressure at that point since the friction angle of water is zero and use of either Equation 2-23 or 2-24 leads to a coefficient of lateral pressure for water, K_w equal to 1.0. The computation of the vertical water pressure was demonstrated previously in Example 2-1. Since K_w=1, the same computation applies for the lateral pressure as well. The lateral earth pressure is computed by using the vertical <u>effective</u> overburden pressure p_o at any depth and applying Equations 2-25a and 2-25b. The lateral earth pressure is added to the hydrostatic water pressure to obtain the total lateral pressure acting on the wall at any point below the ground water level. For a typical soil friction angle of 30 degrees, K_a = 1/3. Since K_w = 1, it can be seen that the **lateral pressure due to water**

is approximately 3 times that due the active lateral earth pressure. A general case for the distribution of combined active lateral earth pressure and lateral water pressure is shown in Figure 2-22. As will be discussed in Chapter 10 (Earth Retaining Structures), this disparity in lateral pressures has serious consequences when the stability of walls is considered and is the reason why drainage behind walls is so important.



(a)



(b)

Active pressure at depth z: $p_a = K_a \gamma z$ Passive pressure at depth z: $p_p = K_p \gamma z$

Active force within depth z: $P_a = K_a \gamma z^2/2$ Passive force within depth z: $P_p = K_p \gamma z^2/2$

Figure 2-21. Failure surfaces, pressure distribution and forces (a) active case, (b) passive case.

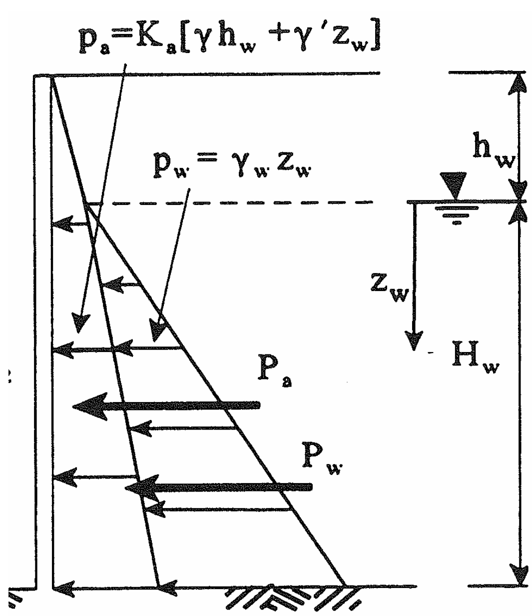


Figure 2-22. General distribution of combined active earth pressure and water pressure.

2.9.2 Deformations Associated with Lateral Pressures

The active and passive pressures are predicated on the development of a certain amount of lateral deformation in the soil. The magnitudes of these lateral deformations and their effect on the development of earth pressures at failure are discussed in Chapter 10 (Earth Retaining Structures).

2.10 UNSATURATED SOIL MECHANICS

As discussed in this Chapter, soil is three phase system that consists of solid particles, liquid and gas. Classical soil mechanics concentrates primarily on the behavior of saturated or dry soils, i.e., a two phase system. For soils in a saturated state, the principle of effective stress is invoked to quantify stress and strain in the soil mass. For soils in a dry state, pore water pressure does not exist and the total stress and effective stress are the same. In reality, all the pore space in soil within the depth of significant influence of geotechnical features is rarely occupied by liquid or gas alone. This is particularly true for soils above the ground water table and soils that are mechanically compacted as in the case of earthworks. In such soils the degree of saturation is generally intermediate between 0% (dry soil) and 100% (saturated soil). Under these conditions, negative pore pressures, i.e., suction, may exist within the soil mass depending upon the type of soil and its grain size distribution. An example of the presence of negative pore pressures is the capillary rise often encountered above the water table. Such negative pore pressures affect all aspects of soil behavior ranging from volume change and shear strength to seepage. Consequently, unsaturated soil behavior impacts a broad array of engineering issues ranging from foundation design and performance to flow through earth embankments and the engineering of facilities on or in expansive, collapsible and compacted soils (ASCE 1993, 1997).

To date the tendency in engineering practice has often been to apply a total stress approach where the effects of negative pore pressures are not properly simulated. In the last couple of decades significant progress has been made to model such negative pore pressures and that field of study is often called "unsaturated soil mechanics." Discussion of the engineering behavior of unsaturated soils is beyond the scope of this manual. At this stage, it is important simply to realize that advanced studies beyond those discussed in this manual may be required on projects where unsaturated state can significantly affect the engineering behavior of soils. The interested readers are directed to the work by Fredlund and Rahardjo (1993), who provide a comprehensive treatment of unsaturated soils.

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CHAPTER 3.0
SUBSURFACE EXPLORATIONS

To perform properly, a structure must interact favorably with the soil on which it rests. The modern geotechnical specialist, who often must build in areas that were considered too poor to build upon a few years ago, must be well versed in the fundamentals of soil mechanics. This knowledge will be used in the design of structural foundations and earthworks to answer the following questions. Will settlements be excessive? Can the structure tolerate settlements? Will the proposed foundation type perform better than another type? Can the foundation soils safely support the imposed embankment or footing loads? Will the proposed cut or fill slopes have adequate stability? Are the foundation and earthwork designs cost-effective?

The engineer should have adequate knowledge of the subsurface conditions at a site before attempting to answer these questions. A site- and project-specific subsurface model must be developed for the cost-effective engineering design of a facility. Figure 3-1 shows a flow chart that identifies a recommended process for developing a subsurface model for engineering design. The investment of a few tens of thousands of dollars in a systematic approach as outlined in Figure 3-1 could result in design and construction savings of hundreds of thousands of dollars by preventing costly failures or overly conservative designs.

The process shown in Figure 3-1 is logical and is generally followed on many projects. In many cases, however, old "rules-of-thumb" and "status quo" approaches can result in an unconscious "by-passing" of critical steps. In particular, selection of the correct tests to determine the relevant engineering properties, the interpretation of the results of those tests, and summarization of data are often poorly performed. Rigorous attention to the rational process in Figure 3-1 is required to assure efficient and thorough exploration and testing programs, especially since many projects are fragmented to the extent that drilling, testing, and design are performed by different parties. This document provides guidance on all the items presented in Figure 3-1. The three major steps in the flow chart in Figure 3-1 and the applicable chapters in this document are as follows:

Step 1: Subsurface Exploration and Field Testing (this Chapter)

Step 2: Laboratory Testing and Test Interpretation (Chapters 4 and 5)

Step 3: Engineering Design (Chapters 6 to 10 and Appendix A)

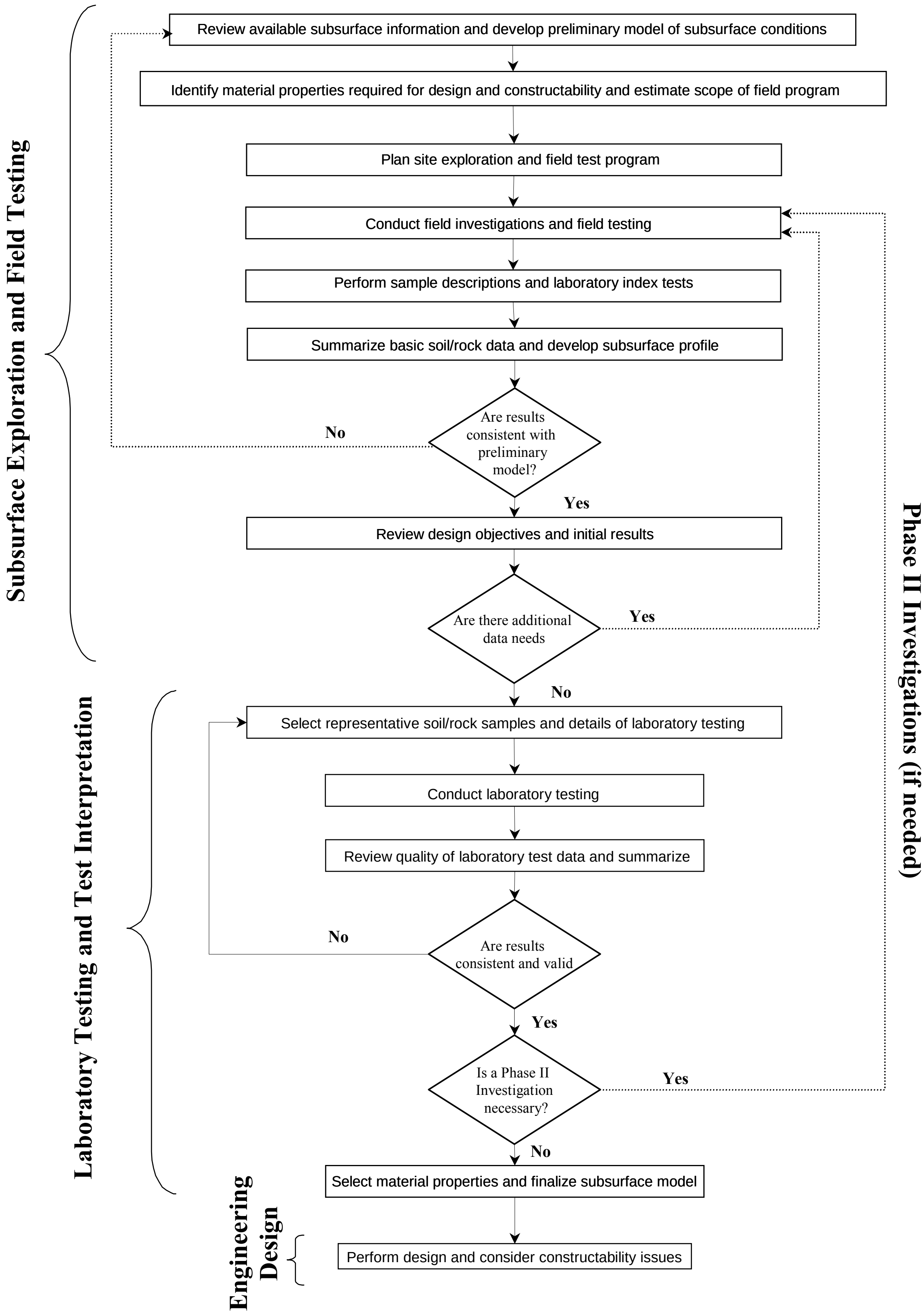


Figure 3-1. Recommended process for developing subsurface model for engineering design (FHWA, 2002a).

3.01 Primary References

The primary references for this Chapter as well as Chapters 4 and 5 are as follows:

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.

FHWA (2002b). *Subsurface Investigations (Geotechnical Site Characterization)*. Report No. FHWA NHI-01-031, Authors: Mayne, P. W., Christopher, B. R., and DeJong, J., Federal Highway Administration, U.S. Department of Transportation.

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.

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

3.1 PREPARING FOR SUBSURFACE EXPLORATION

The initial step in any highway project must include consideration of the soil or rock on which the highway embankment and structures are to be supported. The extent of the site exploration will depend on many factors, not the least of which will be the project scheduling, general subsurface conditions, and the nature of the loads to be supported. In any event, certain basic steps should be followed before exploration equipment is mobilized to the project site. The first step in the exploration is to collect and analyze all existing data. A review of available information prior to the field reconnaissance will help establish what to look for at the site. In the Eighth Rankine Lecture, Glossop (1968) stated the following truism regarding site exploration: "**If you do not know what you should be looking for in a site investigation, you are not likely to find much of value.**" For a highway project, basic sources of geotechnical information should be reviewed to determine landform boundaries and to provide a basis for outlining the project subsurface exploration program. Those sources and functional uses are identified in Table 3-1.

Table 3-1
Sources of historical site data (after FHWA, 2002a)

Source	Functional Use	Location	Examples
Utility Maps	• Identifies buried utility locations • Identifies access restrictions • Prevents damage to utilities	Local agencies/utility companies	Power line identification prior to an intrusive exploration prevents extensive power outage, expensive repairs, and bodily harm
Aerial Photographs	• Identifies manmade structures • Identifies potential borrow source areas • Provides geologic and hydrological information which can be used as a basis for site reconnaissance • Track site changes over time	Local Soil Conservation Office, United States Geological Survey (USGS), local library, local & national aerial survey companies	Evaluating a series of aerial photographs may show an area on site which was filled during the time period reviewed
Topographic Maps	• Provides good index map of site area • Allows for estimation of site topography • Identifies physical features in the site area • Can be used to assess access restrictions	USGS, State Geological Survey	Engineer identifies access areas/restrictions, identifies areas of potential slope instability; and can estimate cut/fill capacity before visiting the site
Existing Subsurface Exploration Report	• May provide information on nearby soil/rock type; strength parameters; hydrogeological issues; foundation types previously used; environmental concerns	USGS, United States Environmental Protection Agency (USEPA), State/local agencies, developers, etc.	A five year old report for a nearby roadway widening project provides geologic, hydrogeologic, and geotechnical information for the area, reducing the scope of the exploration
Geologic Reports and Maps	• Provides information on nearby soil/rock type and characteristics; hydrogeological issues, environmental concerns	USGS and State Geological Survey	A twenty year old report on regional geology identifies earth fissure rock types (including fracture and orientation data) and groundwater flow patterns
Water/Brine Well Logs	• Provide stratigraphy of the site and/or regional area • Varied quality from state to state • Groundwater levels	State Geological Survey/Natural Resources, Department of water resources	A boring log of a water supply well two miles from the site area shows site stratigraphy facilitating evaluations of required depth of exploration

Table 3-1 (Continued)
Sources of historical site data (after FHWA, 2002a)

Source	Functional Use	Location	Examples
Flood Insurance Maps	• Identifies 100 and 500 yr floodplains near water bodies • Caution against construction in a floodplain • Provide information for evaluation of scour potential	Federal Emergency Management Agency (FEMA), USGS, state/local agencies	Prior to exploration, the flood map shows that the site is in a 100 yr floodplain and the proposed structure is moved to a new location
Soil Survey	• Identifies site soil types • Permeability of site soils • Climatic and geologic information	Local Soil Conservation Service	The local soil survey provides information on near-surface soils to facilitate preliminary borrow source evaluation
Sanborn Fire Insurance Maps	• Useful in urban areas • Maps for many cities are continuous for over 100 years. • Identifies building locations and type • Identifies business type at a location (e.g., chemical plant) • May highlight potential environmental problems at an urban site	State library/Sanborn Company (www.sanborncompany.com)	A 1929 Sanborn map of St. Louis shows that a lead smelter was on site for 10 years. This information prevents an exploration in a contaminated area.

A necessary part involved in review of existing data is to identify the major geologic processes that occurred at the project site because this will permit the geotechnical specialist to develop an understanding of how the local soil and rock formations may have developed. The soil formation process and consideration of landforms in designs of geotechnical features is discussed briefly followed by discussions of subsurface exploration programs.

3.1.1 Soil Formations and Landforms

Soils are a result of the weathering of rocks. In general, rocks are classified as igneous, sedimentary, and metamorphic. Igneous rocks are products of melts (magma) generated an unknown distance below the earth's surface. Sedimentary rocks are cemented and/or compressed materials derived from pre-existing sediments deposited in layers by water or by air. A metamorphic rock is any rock that originates by a process of change from what it was previously. Any former igneous, sedimentary, or metamorphic rock can be metamorphosed (changed) into a new metamorphic rock by an increase in temperature and/or pressure and/or by reaction with surrounding hot fluids and gases. Regardless of the type of rock, most weathering takes place near the ground surface. Rock weathering can occur due to mechanical (physical) and/or chemical processes as follows:

- Mechanical or physical process refers to the process whereby the intact rock breaks into smaller fragments. Physical weathering may be caused by expansion resulting from unloading (e.g., exfoliation or spalling off of the exterior surface of the exposed rock), abrasion, temperature changes (e.g., freeze/thaw), erosion by wind or rain, crystal growth (e.g., ice and other crystals such as salt crystals that form as the result of the capillary action of water containing salts in solution), and organic activity (e.g., forces exerted by growing plants and roots in voids and crevasses of rock).

- Chemical process refers to the process whereby the minerals in the rock are altered into new compounds. Chemical weathering is usually preceded by hydration and hydrolysis and may be caused by, oxidation (e.g., chemical reaction with rainwater), solution (e.g., dissolution of limestone) and/or leaching (e.g., dissolution of the cementing agent in the rock). Chemical weathering commonly occurs by fluids seeping into the fractures caused by mechanical (physical) weathering processes. These fluids are chiefly acids created as rainwater dissolves carbon dioxide from the atmosphere and more carbon dioxide and organic acids from the soil. Most chemical weathering processes result in an increase in volume (that causes an increase in stress within the rock mass), lower density materials (e.g., soils), smaller particle sizes (e.g., clay sizes), and more stable minerals (that may decrease the rate of chemical weathering).

The combined effects of the mechanical and chemical weathering processes vary considerably with climate and the mineralogy of the parent rock. The chemical reactions proceed most rapidly and completely in humid tropics and subtropics and least effectively in cold or arid climates (Goodman, 1993). Thus, in the Arctic regions and deserts, the mechanical processes of physical weathering act virtually alone to gradually breakup the rock into a fractured or rubbled mass whereas, in the tropics, the two weathering processes work together rapidly first to break up the rock and then to alter newly exposed rock surfaces during a project's life.

Once the intact rock is broken into fragments, the rate of weathering depends on the particle size and the climate. In general, small particles weather at a faster rate than large ones due to their larger surface area. The weathering processes can result in particle sizes that are not distinguishable by the naked eye (e.g., colloidal particle size) and can be identified only by equipment such as electron microscopes. Based on particle size, the principal terms used by civil engineers to describe soils are gravel, sand, silt and clay. These terms were discussed in Chapter 2 as a function of the particle sizes they represent and some of their physical characteristics. For example, silt and clay particles are finer than the No. 200 sieve (0.075 mm) and exhibit varying properties in presence of water.

Soils created by a particular geologic process assume characteristic topographic features, called **landforms**, which can be readily identified by the geotechnical specialist. A landform contains soils with generally similar engineering properties and typically extends irregularly over wide areas of a project alignment. Early identification of landforms can be used to optimize the subsurface exploration program. Landforms may be described according to the method of formation as **residual soil landforms** or **transported soil landforms.** Soils commonly associated with these two types of landforms are briefly described as follows:

3.1.1.1 Residual Soils

A residual soil landform is one that was formed in its present location through weathering of the parent (or bed) rock. Residual soils tend to be characterized by angular to subangular particles, mineralogy similar to parent rock, and the presence of large angular fragments within the overall soil mass. Because residual soil weathers from parent bedrock, its profile with depth represents a history of the weathering process. Figure 3-2 shows a typical weathering profile for metamorphic and igneous rocks. In Figure 3-2, the weathering profile is divided into three zones: *residual soil*, *weathered rock*, and *unweathered rock*. Deere and Patton (1971) present 12 other weathering-profile classification systems proposed by workers from around the world. Regardless of the weathering-profile classification, the following are some of the properties for such profiles:

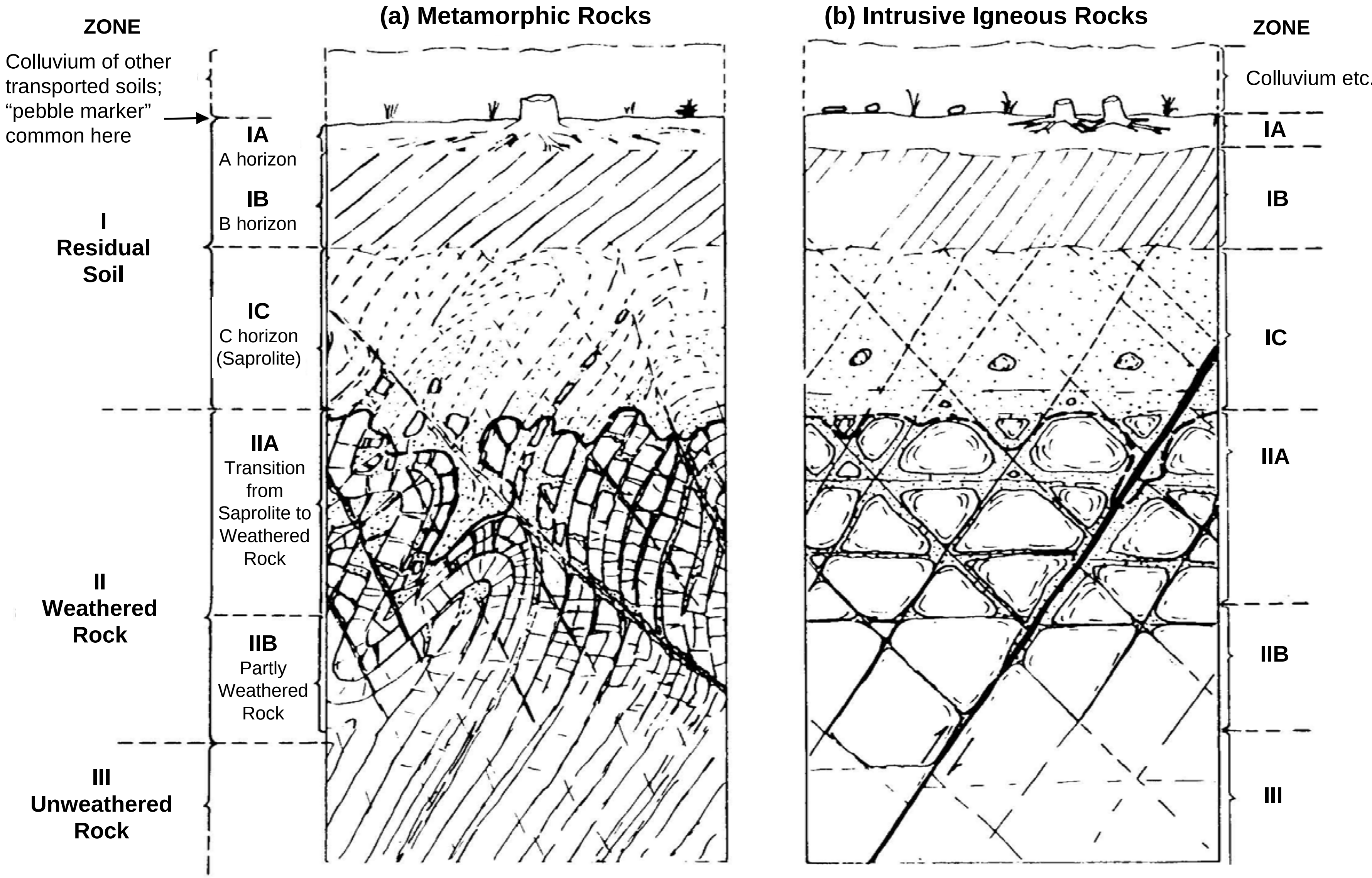


Figure 3-2. Typical weathering profile for metamorphic and igneous rocks (Deere and Patton, 1971).

- The permeability and shear strength gradually change with depth. These two parameters control both the amount of rainfall infiltration and the location of the shear surface when external loads are applied on or in these soils.
- Soil profile thickness and properties depend upon parent bedrock, discontinuities, topography, and climate. Because these factors vary horizontally, the profile can vary significantly over relatively short horizontal distances.
- Deep profiles form in tropical regions where weathering agents are especially strong and advanced stages of chemical weathering form cemented soils called *laterites*. The technical literature often refers to residual soils as tropical soils.
- The material in the transitory zone between residual soil and unweathered rock is called *saprolite*. Saprolites are generally unsaturated, weakly bonded and heterogeneous soils with relict joint systems (Lambe, 1996). Saprolites have widely varying void ratios and widely varying mineralogy and shear strength (Vaughn, *et al.*, 1988).

3.1.1.2 Transported Soils

A transported soil is one that was formed from rock weathering at one location and transported by some exterior agent to another location. The transporting agent may be water (principal agent), a glacier, wind, and/or gravity. Often the deposits of transported soils are given names indicative of the mode of transportation causing the deposit, e.g., alluvial deposits, glacial till, etc. Transported soils are characterized by subrounded to rounded particles and a wide variety of particle sizes. Table 3-2 summarizes commonly encountered landforms composed of transported soils, their primary formational process, and their engineering significance.

3.1.1.3 Area Concept for Explorations Based on Landforms

Knowledge of the landforms and the engineering properties of the soils and rocks enables the designer to determine the most economical location for a highway alignment and grade, to evaluate design problems for each type of soil or rock, and to determine sources of granular borrow material. Once a landform is identified, the geotechnical specialist can develop an **"area" concept** for exploration. In this concept, the lateral extents of landforms in the immediate vicinity of the footprint of the proposed facility are first identified. Then, a limited exploration program (e.g., geologic mapping, geophysical work and some preliminary cone penetration tests and borings) is implemented at strategic locations such that the general characteristics of the landforms are identified. The exploration program can then be refined to obtain specific information on soil types of interest with respect to the footprint of the proposed facility and the anticipated loadings.

Table 3-2
Common landforms of transported soils and their engineering significance

Agent	Landform	Formational Process and General Engineering Significance for Study
Water	Flood Plain	• Formed in valleys that are nearly flat and near the high water level of streams. At flood stage the valley is essentially a "flood plain" that is susceptible to widespread shallow flooding. • Generally poor construction site with fine-grained soils and water problems. Potential scour area. Spread footing design below ground will probably require undercut, low foundation pressure and scour protection. Pile foundations probable. Additional shallow explorations required along footing length to determine buried meandering channels. Historic high water levels should be used in design.
	Coastal Plain	• Formed similar to flood plain but in coastal areas. • Consider spread footings for moderate loads except for high water areas. Potential scour area. Soil "set-up" possible for friction piles (see Chapter 9).
	Terraces	• Formed when a stream or water body cuts into a previously deposited sediment or as the stream bed is lowered over geological periods due to normal erosion or to crustal deformations. Terraces are also known as *bajadas*. • Consider spread footings for lightly loaded foundations.
	Lakebed (Lacustrine, Varves)	• Formed by sedimentation in lake (fresh water) environments. *Varves* are a particular type of lake deposit formed during glacial periods from seasonal ice melting, which temporarily increased the runoff velocity so that precipitated sand layers alternate with layers of precipitates such as silt or silt-clay made at low velocities. • Suitable only for spread footings to support light loads and even then settlement may be expected. Pile foundation probable and often deep. Obtain undisturbed tube samples for laboratory testing. Consider drilling with "mud" rather than casing. Long-term water observations necessary to determine static water level due to impervious soil. Potential scour area.
	Deltas	• Formed by sediments precipitated at the mouths of rivers or streams into bays, oceans, or lakes. • The use of spread footings must be carefully studied as poor soils often underlie deltaic sands and gravels. The parent material is capable of sustaining high spread footing loads. Piles may be required to penetrate delta material and poor soil. Use casing of adequate size to obtain undisturbed samples of poor soil. Potential scour area.
	Alluvial Fans. Filled Valleys (Basin Deposits)	• Formed similar to delta deposits, but typically found in arid areas where mountain stream runoff flows into wide valleys or on to the plains at the mouths of streams. In arid climates, alluvial fans can become cemented by salts left in the ground by evaporating water or by dropping groundwater. Cemented soils can be loose to dense (e.g., *caliche*) or open-graded (*collapsible*). • Consider spread footings for low to moderate loads except at lower elevation of alluvial fans where high water table is possible. In case of collapsible soils, either treat the soils such that collapse potential is mitigated or use deep foundations to bypass such soils. If the caliche is firm to hard, spread foundations can be used.

Table 3-2 (continued)
Common landforms of transported soils and their engineering significance

Agent	Landform	Formation and General Engineering Significance for Study
Ice (Glacier) and meltwater associated with ice	Moraines (Terminal, lateral)	• Formed by soil deposits pushed into ridges around the periphery of a glacier. *Terminal moraines* are ridges of material scraped or bulldozed to the front of a glacier; *lateral moraines* develop along the sides of a glacier. The moraine may not be a single nicely rolled ridge, but rather a highly serrated, above ground level, earth mass. • Advisable to use spread footings for all foundation loads. Piles should not be used due to very difficult driving and boulders. Core all rock to 10 ft (3.0 m) in case boulders are encountered.
	Glacial Till (ground moraine)	• Glacial Till (also termed ground moraine or simply till) is the deposit of ice-suspended material through the bottom of the glacier. As glaciers melted, materials suspended in the ice precipitated onto the underlying soil or rock to form glacial till. Till deposits are characterized by all sizes of particles with no obvious arrangement. Much of northern US has glacial till. • Where till is unsorted, dense and contains considerable sand and gravel, it is advisable to use spread footings for all foundation loads. Piles should not be used due to difficult driving conditions and boulders. Core all rock encountered to depth of 10 ft (3.0 m) as large boulders may be encountered. Long-term water observations necessary to determine static water level due to soil density.
	Outwash	• Sediments precipitated from glacial melts in the discontinuities between ridges in moraines. Small lakes may temporarily form in depressions behind ridges, producing lacustrine (fresh water) sediments. • Spread footing normally used to support moderate to heavy foundation loads. Piles, if required, will be short. Use large diameter sample spoon to permit representative sample to be obtained as average particle size may jam 1-⅜ in (35.3 mm) sample spoon. Standard penetration test may be erratically high due to large particle sizes. Commercial value as sand and gravel sources since the material often contains very little amount of fines, i.e., particle size less than No. 200 (0.075 mm) sieve.
	Eskers	• Eskers are deposits (usually as ridges) formed by precipitation of water-suspended material flowing in ice tunnels. • Advisable to use spread footings for all loads as soil contains much gravel and is dense. Piles not recommended. Large diameter sample spoon recommended as above for outwash. Commercial value as sand and gravel sources since the material often contains very little amount of fines, i.e., particle size less than No. 200 (0.075 mm) sieve.
	Drumlins	• Drumlins are isolated mounds of glacial debris varying from about 35 (10 m) ft to 230 ft (70 m) high and 650 ft (200 m) to 2600 ft (800 m) long. Most drumlins are of the order of 100 ft (30 m) or less in height and 1000 ft (300 m) or less in length. They often occur in groups called drumlin fields (several). • Suitable for spread footing design with moderate to heavy loads. Piles seldom used due to dense coarse nature of subsoil. Commercial value as sand and gravel sources since the material often contains very little amount of fines, i.e., particle size less than No. 200 (0.075 mm) sieve.

Table 3-2 (continued)
Common landforms of transported soils and their engineering significance

Agent	Landform	Formation and General Engineering Significance for Study
Wind (Aeolian)	Loess	• Formed by wind blowing silt and clay with the deposit held together by a montmorillonite binder. Generally derived from glacial outwash in the US. Low density (often less than 90 pcf (14 kN/m^3)), low wet strength (i.e., collapsible upon water ingress), has the ability to stand on vertical cuts due to cementing agents between particles. • Consider spread footings for low to moderate loads. Heavy loads should be pile supported with the bearing resistance obtained below the loess deposit. Accurate ground water level determination important.
	Sand Dune	• Formed by wind action blowing the sand. Transport occurs mainly along the ground until an obstruction is met, whereupon a dune (or mound) forms. Later winds may demolish the dune and redeposit the material at a new location further downwind. Dune sands tend to be well rounded from abrasion. • Consider spread footings for small foundations not subject to vibratory loading. Heavy structural loads should be supported on friction piles.
Gravity	Colluvium	• Formed by physical and chemical weathering of bedrock. The fragmented particles, given sufficient topographic relief, tend to move down slopes under gravitational forces and accumulate as distinctive deposits along the lower portions of slopes, in topographic depressions, and especially at the base of cliffs. • The characteristics of colluvial materials vary according to the characteristics of the bedrock sources and the climate under which the weathering and transport occur. From an engineering viewpoint, colluvium is weakly stratified and consists of a heterogeneous mixture of soil and rock fragments ranging in size from clay particles to rock more than 3 ft (1 m) in diameter. Because they are found along the lowest portions of valley sides, such deposits frequently need to be partially excavated to allow passage of transportation facilities. The resulting cut slopes are commonly unstable and require constant monitoring and maintenance. Colluvial soils are prone to creep (slow movement with time) and landslides are common in such soils.
	Talus (Scree)	• Talus is colluvium composed of predominantly large fragments. Talus fragments can be huge boulders tens of feet across; however, a lower size limit has not been well defined. With time, the coarse fragments may degrade or finer materials may be added by wind or water transport so that these deposits slowly become infilled with a matrix of fine-grained materials. The degree of infilling of these talus deposits may vary horizontally and vertically. • Rock-supported talus is often inherently unstable and may be hazardous to even walk across. Furthermore, the open structure is porous. Talus deposits are not suitable for engineering structures. Talus deposits could be used to make riprap.

The area concept is a powerful tool, particularly for linear highway facilities, as it streamlines the subsurface exploration program costs and provides the planning engineer with useful data during the design and construction phases of a project. It also permits early identification of the type and extent of problem soils to be encountered during construction and therefore allows construction costs to be estimated more accurately.

3.2 FIELD RECONNAISSANCE

Application of the area concept requires the use of proper subsurface exploration equipment and techniques. In particular the use of wide area exploration techniques such as remote sensing and geophysical techniques can provide insight of general subsurface conditions in the project area economically. An adequate site exploration can be accomplished only under the direction of a geotechnical specialist who knows the general limitations of the exploration equipment as well as the general demands of the project. A field reconnaissance, preferably with the bridge designer, roadway designer, and project manager, is recommended to assess subsurface conditions prior to establishing a subsurface exploration.

The field reconnaissance should include:

1. Inspection of nearby structures to determine their performance with the particular foundation type utilized. If settlement is suspected, the original structural plans should be reviewed and the structure surveyed by using the original benchmark.

2. For water crossings, inspection of structural footings and the stream banks up and down stream for evidence of scour. Take careful note of the stream bed material. Often large boulders exposed in the stream but not encountered in the borings, are an indication of potential subsurface obstructions to pile installation.

3. Recording the location, type, and depth of any existing structures or abandoned foundations that may infringe on the new highway facility.

4. Relating site conditions to the proposed boring operations. Record potential problems with utilities (overhead and underground), site access, private property, or obstructions.

Figure 3-3 is an example of a field reconnaissance form used to record data pertinent to the site. Upon completion of the site inspection, the geotechnical specialist should prepare a terrain reconnaissance report in which the general suitability of the site is assessed. The report should:

Field Reconnaissance Report
_______ Department of Transportation

Project No: ____________________ County ____________________ Sta. No. ____________________

Reported By: ______________________________ Date ______________________________

1. Staking of Line
   - Well Staked
   - Poorly Staked (We can work)
   - Request Division to Restake

2. Bench Marks
   In Place: Yes _______ No _________
   Distance from bridge - m (ft) ____________________

3. Property Owners
   Granted Permission: Yes _________ No _________
   Remarks on Back ____________________

4. Utilities
   Will Drillers Encounter Underground or
   Overhead Utilities? Yes __________ No __________
   Maybe _________ At Which Holes? ______________
   What Type? ____________________
   Who to See for Definite Location ________________
   ____
   ____

5. Geologic Formation

6. Surface Soils
   Sand ________ Clay ________ Sandy Clay ________
   Muck _______ Silt ________ Other ____________

7. General Site Description
   Topography
   Level _______ Rolling ________ Hillside _______
   Valley ______ Swamp ________ Gullied _______
   Groundcover
   Cleared _____ Farmed _____ Buildings ________
   Heavy Woods _________ Light Woods _________
   Other ______________________________
   Remarks on Back ____________________

8. Bridge Site
   Replacing ______________________________
   Widening ______________________________
   Relocation ______________________________
   Check Appropriate Equipment
   - Truck Mounted Drill Rig/Cone Rig
   - Track Mounted Drill Rig/Cone Rig
   - Failing 1500
   - Truck Mounted Skid Rig
   - Skid Rig
   - Rock Coring Rig
   - Wash Boring Equipment
   - Water Wagon
   - Pump
   - Hose ___________ m (ft)

8. Bridge Site - Continued
   Cut Section - m (ft) ____________________
   Fill Section - m (ft) ____________________
   If Stream Crossing:
   Will Pontoons Be Necessary? ____________________
   Can Pontoons Be Placed in Water Easily?
   ____
   Can Cable Be Stretched Across Stream?
   How Long? ____________________
   Is Outboard Motorboat Necessary? ________________
   Current:
   Swift ___________ Moderate ______ Slow ____________
   Describe Streambanks scour.
   If Present Bridge Nearby:
   Type of Foundation ____________________
   Any Problems Evident in Old Bridge Including
   Scour ______________________________
   (describe on back)
   Is Water Nearby for Wet Drilling - m (ft) ____________
   Are Abandoned Foundations in Proposed
   Alignment? ______________________________

9. Ground Water Table
   Close to Surface - m (ft) ____________________
   nearby Wells - Depth - m (ft) ____________________
   Intermediate Depth - m (ft) ____________________
   Artesian head - m (ft) ____________________

10. Rock
    Boulders Over Area? Yes _________ No __________
    Definite Outcrop? Yes ___________ No __________
    (show sketch on back)
    What kind? ______________________________

11. Special Equipment Necessary

12. Remarks on Access
    (Describe any Problems on Access)

13. Debris and Sanitary Dumps
    Stations ______________________________
    Remarks ______________________________
    ____
    ____
    ____

Reference: Modified from 1978 AASHTO Foundation Investigation Manual

Figure 3-3. Typical field reconnaissance form.

1. Flag major potential problems, which may preclude construction.
2. Recommend beneficial shifts in location.
3. Present a general discussion of expected subsurface conditions.
4. Present cost estimate for extraordinary geotechnical treatments.
5. Prepare an estimate of subsurface exploration quantities, costs, and time.

This information should be transmitted to all the groups involved with the project such as the project manager, roadway designer, and bridge designer.

3.3 SUBSURFACE EXPLORATION PROGRAM

The procedures employed in any subsurface exploration program are dependent on a variety of factors that vary from site to site. The project design objectives and the expected subsurface conditions have the major influence on the subsurface explorations. Highway projects necessarily involve both earthwork and structural foundations. Typical boring programs for highways on new alignments are established such that basic information is first gathered along the entire highway alignment and subsequent detailed borings are taken as required at the locations of structures or in problem earthwork areas as disclosed by the initial basic program. Subsurface explorations for widening or improvements of existing highways generally are done in one stage as the location is predetermined.

Consideration should be given, particularly for large or complex projects, to performing geologic mapping and geophysical explorations after the field reconnaissance and prior to any invasive explorations such as borings. Geologic mapping and geophysical explorations can be quick and provide a much larger coverage of the project area as compared to invasive explorations. The information from field reconnaissance, geologic mapping and geophysical explorations can then be used to setup the conventional subsurface exploration and testing program. Geophysical explorations are discussed in Section 3.15.

After the field reconnaissance and geophysical explorations are completed, "invasive" explorations using drilled borings and in-situ tests must be performed to obtain in-situ properties and physical samples for identification and testing. The sampling techniques and tools are discussed in Section 3.4 and other sections of this chapter. The objectives for such explorations are as follows:

1. Determine stratigraphy.
   a. physical description and extent of each stratum.
   b. thickness and elevation of top and bottom of each stratum.

2. For fine-grained soils (each stratum) determine:
   a. natural moisture contents.
   b. Atterberg limits.
   c. stiffness.
   d. presence of organic materials.
   e. evidence of desiccation or previous soil disturbance, shearing, or slickensides.
   f. swelling characteristics.
   g. unconfined compressive strength - typically estimated from Standard Penetration Tests or Cone Penetration Tests.
   h. shear strength.
   i. compressibility.

3. For coarse-grained soils (each stratum) determine:
   a. in-situ density (average and range) typically determined from Standard Penetration Tests or Cone Penetration Tests.
   b. grain-size distributions (gradation).
   c. presence of organic materials.

4. Determine depth to ground water (for each aquifer if more than one is present).
   a. piezometric surface over site area: existing, past, and probable range in future (observe at several times).
   b. perched water table.

5. Determine depth to bedrock.
   a. depth over entire site.
   b. type of rock.
   c. extent and character of weathering.
   d. joints, including distribution, spacing, whether open or closed, and joint infilling.
   e. faults.
   f. solution effects in limestone or other soluble rocks.
   g. core recovery and soundness (RQD).
   h. hardness and strength.

3.4 SAMPLING TECHNIQUES AND TOOLS

The purpose of this section is to provide information on various in-situ testing methods that are currently used to establish site-specific soil and rock properties for design and construction. The execution of a conventional subsurface exploration and testing program usually includes rotary drilling, Standard Penetration Testing (SPT), disturbed and undisturbed sample recovery, and laboratory testing. Although procedures for these commonly performed activities are described in AASHTO and ASTM standards and are well known to most geo-professionals, important testing details are sometimes overlooked that can result in data having marginal quality. This section discusses the importance of carefully selecting and properly conducting the appropriate field and/or laboratory testing method.

In-situ testing methods are increasingly being used on transportation projects, however testing procedures and test limitations are not as well understood as those of the more conventional methods of subsurface exploration and testing such as the use of drilled borings. In this chapter, procedures for various in-situ and laboratory testing methods are presented as they relate to obtaining high quality data for the evaluation of engineering properties of soils and rocks for transportation projects. Information on equipment calibration, measured test parameters, quality control, and the range of ground conditions that apply to each test is also presented.

Several in-situ tests define the geostratigraphy and provide direct measurements of soil properties and geotechnical parameters. The common in-situ tests include: Standard Penetration Test (SPT), Cone Penetration Test (CPT), Piezocone Test (CPTu), Flat Plate Dilatometer Test (DMT), Pressuremeter Test (PMT), and Vane Shear Test (VST). Although the load is applied differently in each test, the purpose of each test is to measure the corresponding response of the soil in an attempt to evaluate the soil's engineering properties, such as strength and/or stiffness. Figure 3-4 depicts these various devices and a graphical representation of how load is applied.

Some state DOTs perform these tests by using agency-owned equipment. In many cases however, the agency may retain an outside contractor for these services either directly or as part of an overall project development package. Several technical reports and manuals are available that describe these test methods. A brief list of these references is provided in Table 3-3. Agencies that perform or contract for these testing services are encouraged to obtain the references identified in Table 3-3. In this manual, only the SPT and the CPT tests will be discussed since they are the most commonly used.

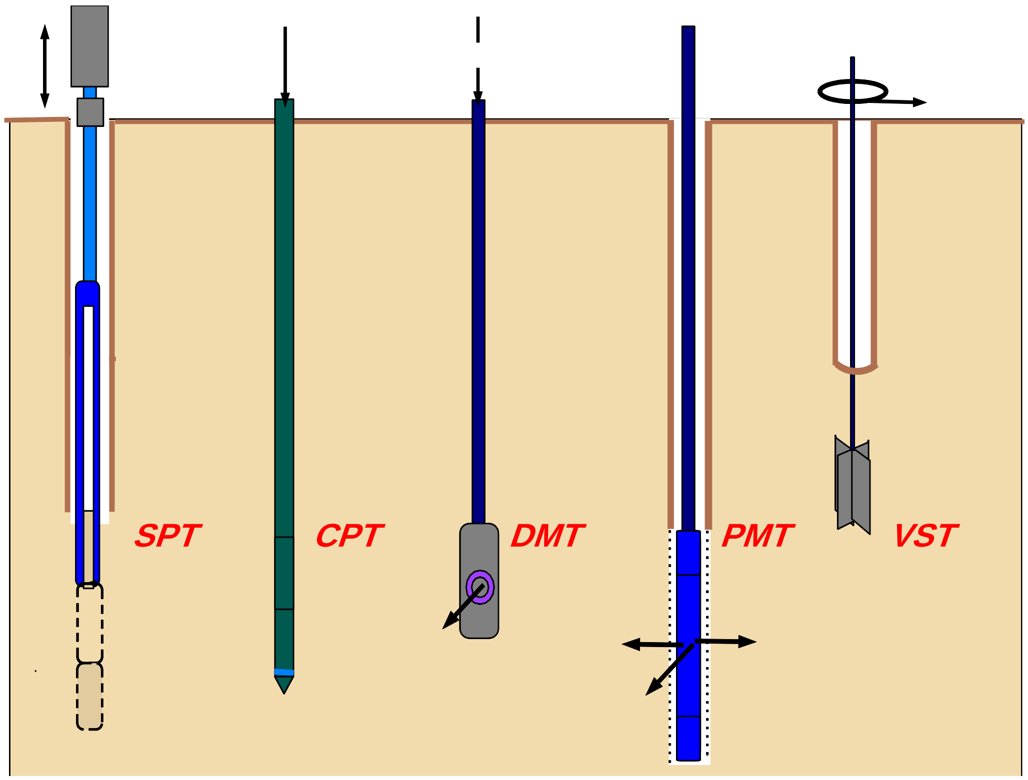


Figure 3-4. Common in-situ tests for geotechnical site characterization of soils (FHWA, 2002b).

Boreholes are required for conducting the SPT and normal versions of the PMT and VST. Therefore a drill rig and crew are required for the performance of these tests. Boreholes are not required for the CPT, CPTu, and DMT; therefore these tests are called "direct-push" technologies. Although boreholes are not required for these tests, special mobilized equipment and data acquisition systems are required. Specialized versions of the PMT (i.e., full-displacement type) and VST can be conducted without boreholes. In such cases either standard drill rigs or mobile hydraulic systems (cone trucks) are used to push the probes to the required test depths. Obviously direct push test methods are not suitable in soil profiles that contain boulders, hard cemented layers and bedrock. For such profiles, borehole methods prevail as the testing device may be advanced through the hard layers by coring or non coring techniques. An advantage of direct-push soundings is that cuttings or spoil are not generated, however, this advantage is offset by a significant disadvantage, i.e., no soil sample is retrieved for classification or subsequent laboratory testing. Another advantage of the CPT and CPTu tests is that they provide a continuous record of soil response through the entire depth of the direct push. The other tests are performed at discrete intervals so that the soil's response is measure at specific depths only. In addition, important layers can be missed with any of the discrete interval test methods.

Table 3-3
Reference publications on in-situ testing (FHWA, 2002b)

Test Method	AASHTO/ ASTM Designation	Reference
SPT	AASHTO T206 ASTM D 1586	FHWA (2002b). *Subsurface Investigations (Geotechnical Site Characterization)*. Report No. FHWA NHI-01-031, Authors: Mayne, P. W., Christopher, B. R., and DeJong, J., Federal Highway Administration, U.S. Department of Transportation.
CPT, CPTu, SCPTu	ASTM D 3341, D5778	FHWA (1992a). *The Cone Penetrometer Test*. Report No. FHWA NHI-91-043, Authors: Riaund J-L and Miran J., Federal Highway Administration, U.S. Department of Transportation. Lunne, T., Robertson, P.K., and Powell, J.J.M. (1997) *Cone Penetration Testing in Geotechnical Practice,* E & F Spon.
DMT	Suggested ASTM Method	FHWA (1992b). *The Flat Dilatometer Test*. Report No. FHWA NHI-91-044, Authors: Riaund J-L and Miran J., Federal Highway Administration, U.S. Department of Transportation.
PMT	ASTM D 4719	FHWA (1989a). *The Pressuremeter Test for Highway Applications*. Report No. FHWA IP-89-008, Authors: Briaud J-L, Federal Highway Administration, U.S. Department of Transportation. Clarke, B.G. (1995) *Pressuremeters in Geotechnical Design*, Blackie Academic & Professional.
VST	ASTM D 2573	ASTM (1988). *Vane Shear Strength Testing in Soils: Field and Laboratory Studies*, American Society for Testing and Materials, Committee D-18 on Soil and Rock for Engineering Purposes, Philadelphia, PA.

3.5 BORING METHODS

Geotechnical borings are a critical component of any subsurface exploration program. They are performed to satisfy several objectives including those listed below.

- identify the subsurface distribution of materials with distinctive properties, including the presence and thickness of distinct layers;
- retrieve samples of each layer for laboratory tests to determine engineering properties;
- determine depth to groundwater; and
- provide access for the introduction of in-situ testing devices.

There are many types of equipment used in current practice for advancing a soil or rock boring. Typical types of soil borings are listed in Table 3-4(a), rock coring methods in Table 3-4(b), and other exploratory techniques in Table 3-4(c). Detailed information on soil and rock boring procedures can be found in AASHTO (1988), FHWA (2002b), and ASTM D 4700. A brief description of typical soil boring methods is provided below (Day, 1999).

3.5.1 Auger Borings

An auger is an apparatus with a helical shaft that can be manually or mechanically advanced to bore a hole into soil. Large and small diameter augers are shown in Figure 3-5. The practice of advancing a borehole with a mechanical auger consists of rotating the auger while applying a downward pressure on the auger to penetrate soil and possibly weak or weathered rock. The auger may be continuous, where the helix extends along the entire length of the shaft, or discontinuous when the auger helix is at the bottom of the drill stem.

- *Discontinuous or single flight auger borings and bucket auger borings.* There are basically two types of discontinuous augers: discontinuous flight augers and bucket augers. Commonly available discontinuous flight augers have diameters ranging from 0.25 to 3 ft (0.075 to 1 m) and bucket augers have diameters ranging from 1 to 8 ft (0.3 to 2.5 m). For discontinuous flight auger borings, the auger is periodically removed from the hole and the soil lodged in the grooves of the flight auger is removed. When a bucket auger is used, it too is periodically removed from the hole and the soil in the bucket removed. A casing is generally not used for discontinuous flight and bucket auger borings. Therefore, these methods are not recommended for boreholes deeper than 35 ft (10 m), or where the hole may cave-in during the excavation of loose or soft soils, or when the boring is below the groundwater table. In firm stiff clays, discontinuous auger borings can be performed to depths in excess of 35 ft (10 m).

- *<u>Continuous flight auger borings.</u>* As the name implies, continuous flight augers have the auger flights continuous along the entire length of the auger. As shown in Figure 3-6a, there are two types of continuous flight augers: **solid stem** and **hollow stem**. For both of these type augers the drill cuttings are returned to the ground surface via the auger flights. The <u>solid stem</u> auger must be removed from the borehole to allow access to the hole for insertion of sampling or testing devices. Because the auger must be periodically removed from the borehole, a solid stem auger is not appropriate in sands and soft soils or in soil deposits where groundwater is close to the surface. A <u>hollow-stem</u> auger has a circular hollow core that allows for sampling through the center of the auger. As shown in Figure 3-6c, hollow-stem augers come in a variety of diameters. The hollow-stem auger acts like a casing and allows for sampling in loose or soft soils or when the excavation is below the ground water table. A plug (Figure 3-6d) is necessary when hollow stem augers are advanced to prevent cuttings from migrating through the hollow stem. The plug is removed to permit SPT sampling. In loose sands and soft clays extending below the water table, drilling fluids are often used to minimize and mitigate disturbance effects and keep the hole open. The components of the hollow stem auger system are shown in Figure 3-6b.

(a)

(b)

Figure 3-5. (a) Large diameter auger, (b) Small diameter continuous flight auger.

Table 3-4(a)
Soil and soft rock boring methods (FHWA, 2002a)

Method	Procedure	Applications	Limitations / Remarks
Auger boring (ASTM D 1452)	Dry hole drilled with hand or power auger; samples recovered from auger flights	In soil and soft rock; to identify geologic units and water content above water table	Soil and rock stratification destroyed; sample mixed with water below the water table
Hollow-stem auger boring	Hole advanced by hollow-stem auger; soil sampled below auger as in auger boring above	Typically used in soils that would require casing to maintain an open hole for sampling	Sample limited by larger gravel; maintaining hydrostatic balance in hole below water table is difficult
Wash-type boring	Light chopping and strong jetting of soil; cuttings removed by circulating fluid and discharged into settling tub	Soft to stiff cohesive materials and fine to coarse granular soils	Coarse material tends to settle to bottom of hole; should not be used in boreholes above water table where undisturbed samples are desired.
Becker Hammer Penetration Test (BPT)	Hole advanced using double acting diesel hammer to drive a 6.6-in (168 mm) double-walled casing into the ground. Several sizes are available.	Typically used in soils with gravel and cobbles; casing is driven open-ended if sampling of materials is desired	Skin friction of casing difficult to account for; unsure as to the repeatability of test
Bucket Auger boring	A 2 to 4 ft (0.6 to 1.2 m) diameter drilling bucket with cutting teeth is rotated and advanced. At the completion of each advancement, the bucket is retrieved from the boring and soil is emptied on the ground.	Most soils above water table; can dig harder soils than above types and can penetrate soils with cobbles and boulders if equipped with a rock bucket	Not applicable in running sands; used for obtaining large volumes of disturbed samples and where it is necessary to enter a boring to make observations

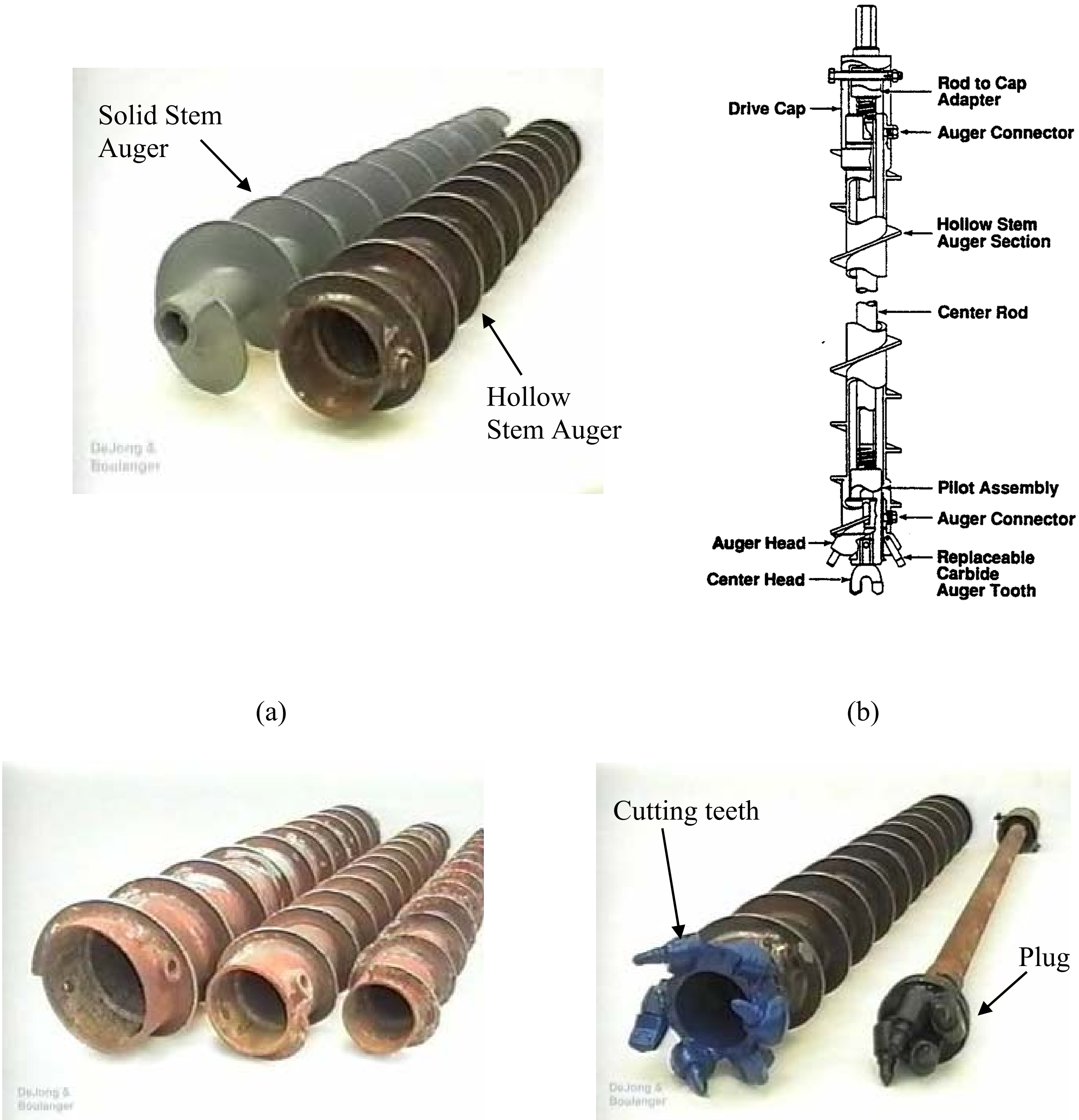
Table 3-4(b)
Rock core drilling methods (FHWA, 2002a)[1]

Method	Procedure	Type of sample	Applications	Limitations / Remarks
Rotary coring of rock (ASTM D 2113; AASHTO T 225)	Outer tube with diamond (or tungsten carbide) bit on lower end rotated to cut annular hole in rock; core protected by stationary inner tube; cuttings flushed upward by drill fluid	Rock cylinder 1 in to 4 in (25 to 100 mm) in diameter and as long as 10 ft (3 m), depending on rock soundness. Standard coring size is 2-1/8 in (54 mm) diameter.	To obtain continuous core in sound rock (percent of core recovered depends on fractures, rock variability, equipment, and driller skill)	Core lost in fracture or variable rock; blockage prevents drilling in badly fractured rock; dip of bedding and joint evident but not strike
Rotary coring of rock, wire line	Same as ASTM D 2113, but core and stationary inner tube retrieved from outer core barrel by lifting device or "overshot" suspended on thin cable (wire line) through special large-diameter drill rods and outer core barrel	Rock cylinder 1-1/8 in to 3-3/8 in (28 to 85 mm) wide and 5 ft to 10 ft (1.5 to 3 m) long	To recover core better in fractured rock which has less tendency for caving during core removal; to obtain much faster cycle of core recovery and resumption of drilling in deep holes	Core lost in fracture or variable rock; blockage prevents drilling in badly fractured rock; dip of bedding and joint evident but not strike
Rotary coring of swelling clay, soft rock	Similar to rotary coring of rock; swelling core retained by third inner plastic liner	Soil cylinder 1-1/8 in to 3-3/8 in (28 to 85 mm) wide and 2 ft to 5 ft (0.6 m to 1.5 m) long encased in plastic tube	In soils and soft rocks that swell or disintegrate rapidly in air (protected by plastic tube)	Sample smaller; equipment more complex than other soil sampling techniques

(1) See Section 3.6.4 for additional discussion on types of core barrels (i.e., single-, double-, or triple-tube).

Table 3-4(c)
Other exploratory techniques (FHWA, 2002a)

Method	Procedure	Type of sample	Applications	Limitations / Remarks
Borehole camera	Inside of core hole viewed by circular photograph or scan	No sample, but a visual representation of the material	To examine stratification, fractures, and cavities in hole walls	Best above water table or when hole can be stabilized by clear water
Pits and Trenches	Pit or trench excavated to expose soils and rocks	Chunks cut from walls of trench; size not limited	To determine structure of complex formations; to obtain samples of thin critical seams such as failure surface	Moving excavation equipment to site, stabilizing excavation walls, and controlling groundwater may be difficult; useful in obtaining depth to shallow rock and for obtaining undisturbed samples on pit/trench sidewalls; pits need to be backfilled
Rotary or cable tool well drill	Toothed cutter rotated or chisel bit pounded and churned	Pulverized	To penetrate boulders, coarse gravel; to identify hardness from drilling rates	Identification of soils or rocks difficult
Percussive Method (jack hammer or air track)	Impact drill used; cuttings removed by compressed air	Rock dust	To locate rock, soft seams, or cavities in sound rock	Drill becomes plugged by wet soil



(a) (b)

(c) (d)

Figure 3-6. (a) Solid stem auger and hollow stem auger, (FHWA, 2002b) (b) Hollow stem auger components (ASTM D 4700), (c) Sizes of hollow stem auger flights (FHWA, 2002b), (d) Outer and inner assembly of hollow stem auger (FHWA, 2002b).

3.5.2 Wash-type Borings

Wash-type borings use circulating drilling fluid (e.g., water or mud) to remove cuttings from the borehole, Figure 3-7. Cuttings are created by the chopping, twisting, and jetting action of the drill bit that breaks the soil or rock into small fragments. Tri-cone bits are often used in dense soil or soft rock. If bentonite or a polymeric drilling mud cannot be used to maintain an open borehole, casings are often used to prevent cave-in of the borehole. The use of casing will require a significant amount of additional time and effort but will result in a protected borehole. When drilling mud is used during subsurface boring, it will be difficult to classify the soil from the auger cuttings because of contamination with the mud. Also, the outside of samples may become coated with drilling mud.

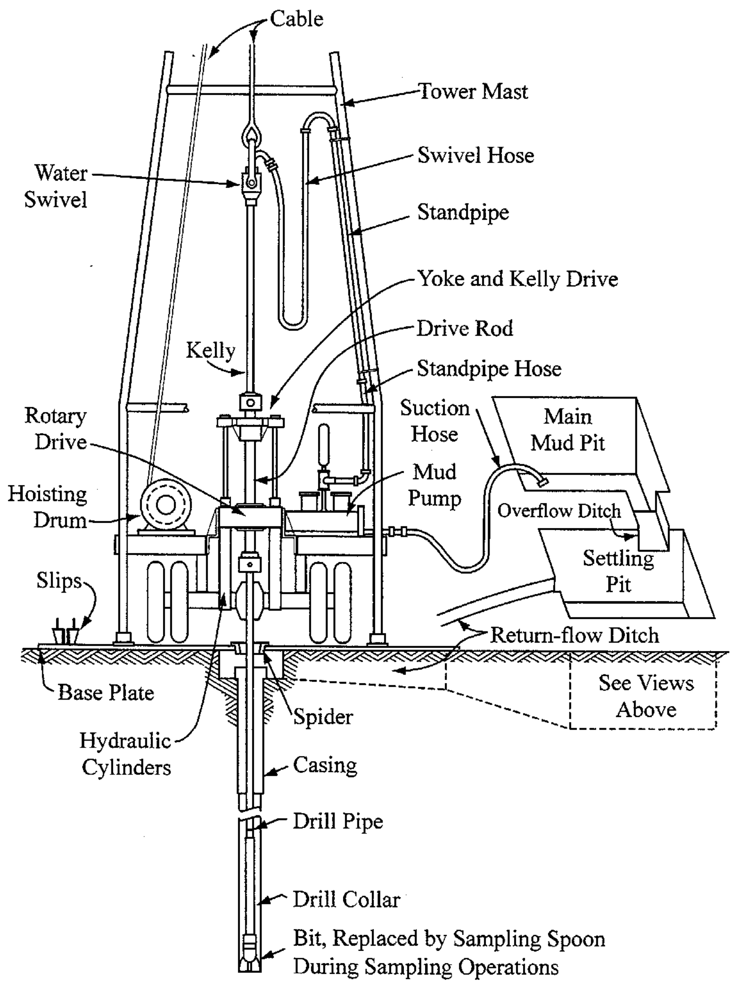
The properties of the drilling fluid and the quantity of water pumped through the drill bit will determine the size of particles that can be removed from the boring with the circulating fluid. In formations containing gravels, cobbles, or larger particles, coarse material may be left at the bottom of the boring. In these instances, cleaning the bottom of the boring with a larger diameter sampler (such as the 3 in (75 mm) OD split barrel sampler) may be needed to obtain a representative sample of the formation.

3.5.3 Coring in Rocks

The previously described methods are typically used for soil exploration. The following methods are primarily used for rock exploration.

- *Rotary coring*. This type of coring equipment is most commonly used for rock exploration when an intact core of the rock is desired. Power rotation of the drilling bit is accompanied with introduction of a circulating fluid to remove cuttings from the hole. The drilling bits are specifically designed to core rock, and inner/outer tubes or casings are used to capture the intact core. Table 3-4(b) lists various types of rotary coring techniques for rock, although many of these techniques are also applicable to dense or stiff soil.

- *Percussion drilling*. This type of drilling equipment is often used to penetrate hard rock for subsurface exploration or for the purpose of drilling wells. The drill bit works much like a jackhammer, rising and falling to break up and crush the rock material. Air is commonly used to clean the hole and transport the cuttings to the ground surface. Table 3-4(c) includes a description on the use of the percussion drilling techniques.



(a)

(b)

(c)

Figure 3-7. (a) Schematic of drilling rig for rotary wash methods (after Hvorslev, 1948), (b) Typical drilling configuration, (c) Settling basin (mud tank).

3.6 SAMPLING METHODS

3.6.1 Disturbed Sampling of Soil

Disturbed sampling of soil provides a means to evaluate stratigraphy by visual examination and to obtain soil specimens for laboratory index testing. Disturbed samples are usually collected using split-barrel samplers (Figure 3-8; AASHTO T 206, ASTM D 1586), although several other techniques are available for disturbed sample collection in boreholes (see Table 3-5(a) and 3-5(b)). Shallow disturbed samples can also be obtained by using hand augers and test pits. Direct push methods, such as GeoProbe sampling, can be used to obtain continuous disturbed samples but these methods have limitations in sampling depth similar to those of solid stem and bucket augers (i.e., generally good for depths less than 33 feet (10 meter) unless in firm to stiff clays). Samples obtained via disturbed sampling methods are generally used for index property testing in the laboratory. They should not be used to prepare specimens for consolidation and strength tests.

Figure 3-8. Split barrel sampler.

3.6.2 Undisturbed Sampling of Soil

Undisturbed soil samples are required for performing laboratory strength and consolidation tests on cohesive soils having consistencies ranging from soft to stiff. High-quality samples for such tests are particularly important for approach embankments and for structural foundations and wall systems that may stress compressible strata. In reality, it is impossible to retrieve truly undisturbed samples since changes in the state of stress in the sample occur upon sampling and removal of the sample from depth. The goal of high-quality undisturbed sampling is to minimize the potential for: (1) alteration of the soil structure; (2) changes in moisture content or void ratio; and (3) changes in chemical composition of the soil. Due to cost and ease of use, the thin-walled Shelby tube (Figure 3-9) is the most commonly used sampler for obtaining relatively undisturbed samples of soft to stiff fine-grained soils.

Table 3-5(a)
Common samplers to retrieve disturbed soil samples (modified after NAVFAC, 1986a)

Sampler	Typical Dimensions	Soils that Give Best Results	Method of Penetration	Cause of Low Recovery	Remarks
Split Barrel	Standard is 2 in (50 mm) outside diameter (OD) and 1-3/8 in (35 mm) inside diameter (ID)	All soils finer than gravel size particles that allow sampler to be driven; gravels invalidate drive data; A soil retainer may be required in granular soils.	140 lb (64 kg) hammer driven	Gravel may block sampler	A SPT is performed using a standard penetrometer and hammer (see text); samples are extremely disturbed
Continuous helical-flight auger	Diameters range 3 in to 16 in (75 to 400 mm; penetrations to depths exceeding 50 ft (15 m)	Most soils above water table; will not penetrate hard soils or those containing cobbles or boulders	Rotation	Hard soils, cobbles, boulders	Method of determining soil profile, bag samples can be obtained; log and sample depths must account for lag time between penetration of bit and arrival of sample at surface, to minimize errors in estimated sample depths

Table 3-5(b)
Common samplers to retrieve disturbed soil samples (modified after NAVFAC, 1986a)

Sampler	Typical Dimensions	Soils that Give Best Results	Method of Penetration	Cause of Low Recovery	Remarks
Disc auger	Up to 3.5 ft (1 m) diameter; usually has maximum penetration depth of 25 ft (8 m)	Most soils above water table; will not penetrate hard soils or those containing cobbles or boulders	Rotation	Hard soils, cobbles, boulders	Method of determining soil profile, bag samples can be obtained; log and sample depths must account for lag time between penetration of bit and arrival of sample at surface, to minimize errors in estimated sample depths
Bucket auger	Up to 4 ft (1.2 m) diameter common; larger sizes available; with extensions, depth over 80 ft (25 m) are possible	Most soils above water table; can penetrate harder soils than above types and can penetrate soils with cobbles and boulders if equipped with a rock bucket	Rotation	Soil too hard to penetrate	Several bucket types available, including those with ripper teeth and chopping tools; progress is slow when extensions are used
Test boring of large samples, Large Penetration Test (LPT)	2 in to 3 in (50 to 75 mm) ID and 2.5 in to 3.5 in (63 mm to 89 mm) OD samplers (examples, Converse sampler, California Sampler)	In sandy to gravelly soils	Up to 350 lb (160 kg) 350 lb hammer driven	Large gravel, cobbles, and boulders may block sampler	Sample is intact but very disturbed; A resistance can be recorded during penetration, but is not equivalent to the SPT N-value and is more variable due to no standard equipment and methods

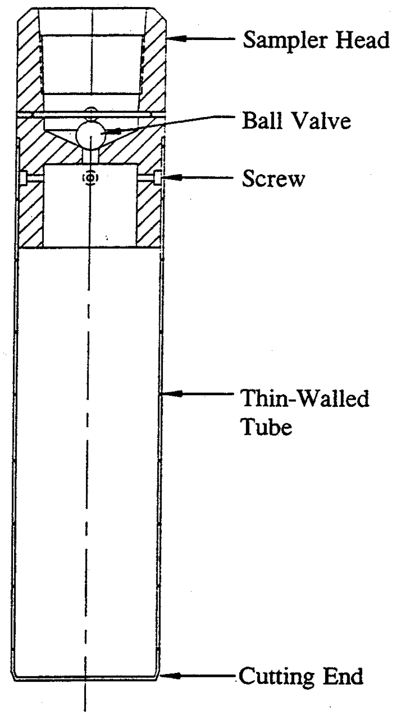


Figure 3-9. Schematic of thin-walled (Shelby) tube (after ASTM D 4700) and photo of tube with end caps (FHWA, 2002b).

Thin walled Shelby tube sampling is discussed in Section 3.5.3. Depending upon the in-situ condition of the fine-grained soil (e.g., stiffness and whether significant granular material is in the soil matrix), alternative sampling devices may be used to obtain nominally undisturbed soil samples. These alternative samplers include:

- Stationary piston sampler (Figure 3-10);
- Denison sampler (Figure 3-11);
- Pitcher samplers (Figure 3-12);
- Hydraulic piston sampler (Osterberg Sampler).

Summary information on these samplers is provided in Table 3-6 and detailed procedures for these sampling techniques are provided in FHWA (1997, 2002b). Although not common for typical transportation-related projects, a variety of special samplers are available to obtain samples of soil and soft rocks. These specialty samplers include the retractable plug sampler, the Sherbrooke sampler, and the Laval sampler.

Table 3-6
Nominally undisturbed soil samplers (modified after NAVFAC, 1986a)

Sampler	Typical Dimensions	Soils that Give Best Results	Method of Penetration	Cause of Disturbance or Low Recovery	Remarks
Shelby tube (ASTM D 1587; AASHTO T 207)	3 in (76 mm) OD and 2-7/8 in (73 mm) ID most common; available from 2 in to 5 in (50 to 127 mm) OD; 30 in (760 mm) sampler length standard	Cohesive fine-grained or soft soils; gravelly and very stiff soils will crimp tube	Pressing with relatively rapid, smooth stroke; can be carefully hammer driven but this will induce additional disturbance	Erratic pressure applied during sampling, hammering, gravel particles, crimping of tube edge, improper soil types for sampler, pressing tube greater than 80% of tube length	Simplest device for undisturbed samples; boring should be clean before sampler is lowered; little waste area in sampler; not suitable for hard, dense or gravelly soils
Stationary piston	3 in (76 mm) OD most common; available from 2 in to 5 in (50 to 127 mm) OD; 30 in (760 mm) sampler length standard	Soft to medium clays and fine silts; not for sandy soils	Pressing with continuous, steady stroke	Erratic pressure during sampling, allowing piston rod to move during press, improper soil types for sampler	Piston at end of sampler prevents entry of fluid and contaminating material requires heavy drill rig with hydraulic drill head; samples generally less disturbed compared with Shelby tube; not suitable for hard, dense, or gravelly soil
Hydraulic piston (Osterberg)	3 in (76 mm) OD is most common; available from 2 in to 4 in (50 to 100 mm) OD; 36 in (910 mm) sampler length standard	Silts and clays, some sandy soils	Hydraulic or compressed air pressure	Inadequate clamping of drill rods, erratic pressure	Needs only standard drill rods; requires adequate hydraulic or air capacity to activate sampler; samples generally less disturbed compared with Shelby tube; not suitable for hard, dense, or gravelly soil
Denison	3.5 in to 7 in (89 to 177 mm) OD, producing samples 2-3/8 in 6.3 in (60 to 160 mm); 24 in (610mm) sampler length	Stiff to hard clay, silt, and sands with some cementation, soft rock	Rotation and hydraulic pressure	Improper operation of sampler; poor drilling procedures	Inner tube face projects beyond outer tube, which rotates; amount of projection can be adjusted; generally takes good samples; not suitable for loose sands and soft clays
Pitcher sampler	4 in (100 mm) OD; uses 3 in (76-mm) diameter Shelby tubes; sample length 24 in (610 mm)	Same as Denison	Same as Denison	Same as Denison	Differs from Denison in that inner tube projection is spring controlled; often ineffective in cohesionless soils
Foil Sampler	Continuous samples 2 in (50 mm) wide and as long as 65 ft (20 m)	Fine grained soils including soft sensitive clays, silts, and varved clays	Pushed into the ground with steady stroke; Pauses occur to add segments to sample barrel	Samplers should not be used in soils containing fragments or shells	Samples surrounded by thin strips of stainless steel, stored above cutter, to prevent contact of soil with tube as it is forced into soil

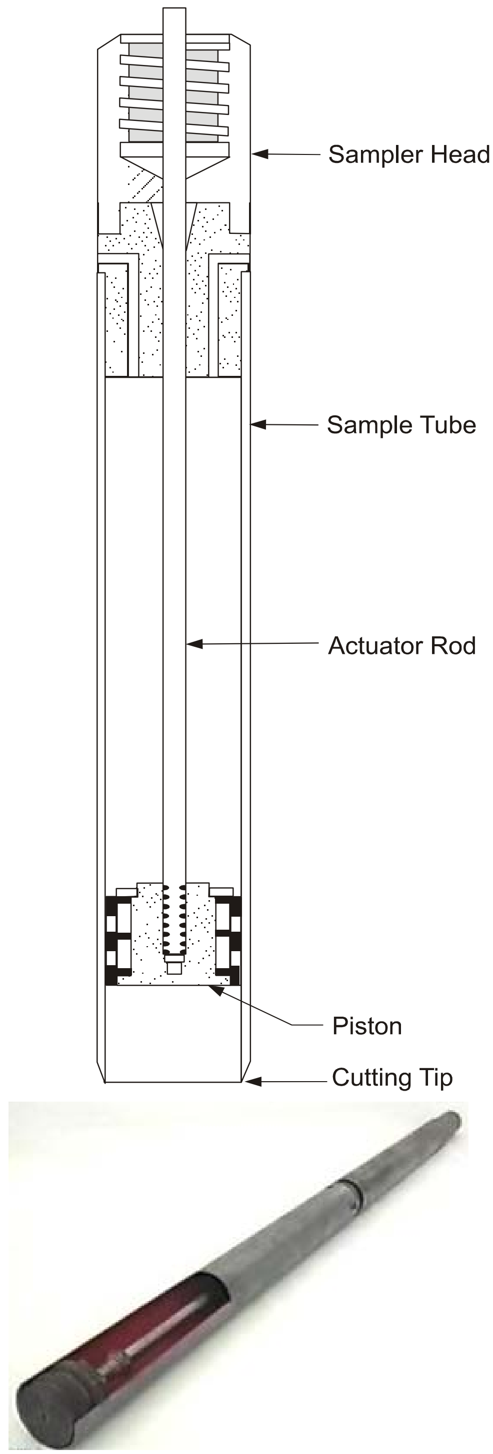


Figure 3-10. Stationary piston sampler schematic (after ASTM D 4700) and photo (FHWA, 2002b).

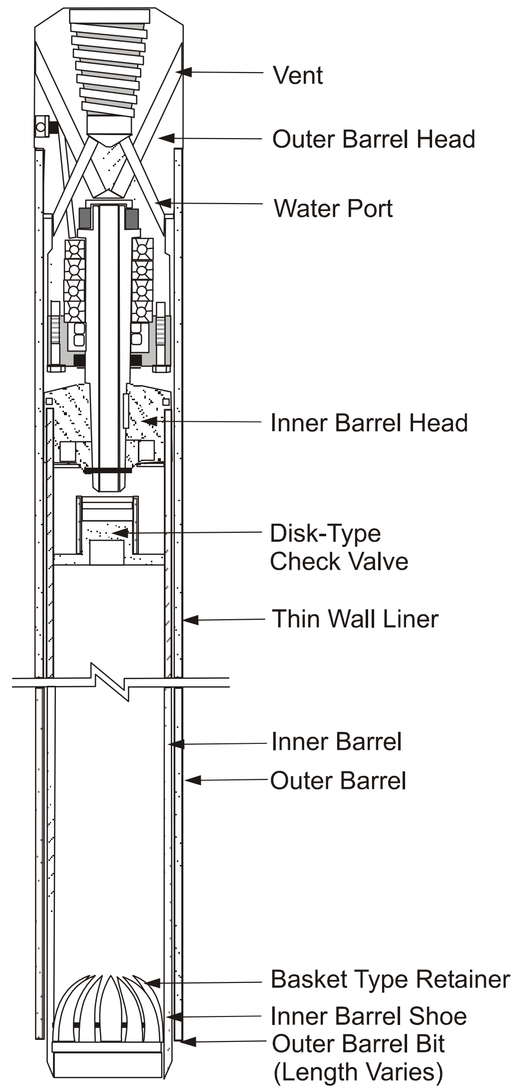


Figure 3-11. Denison sampler (FHWA, 1997).

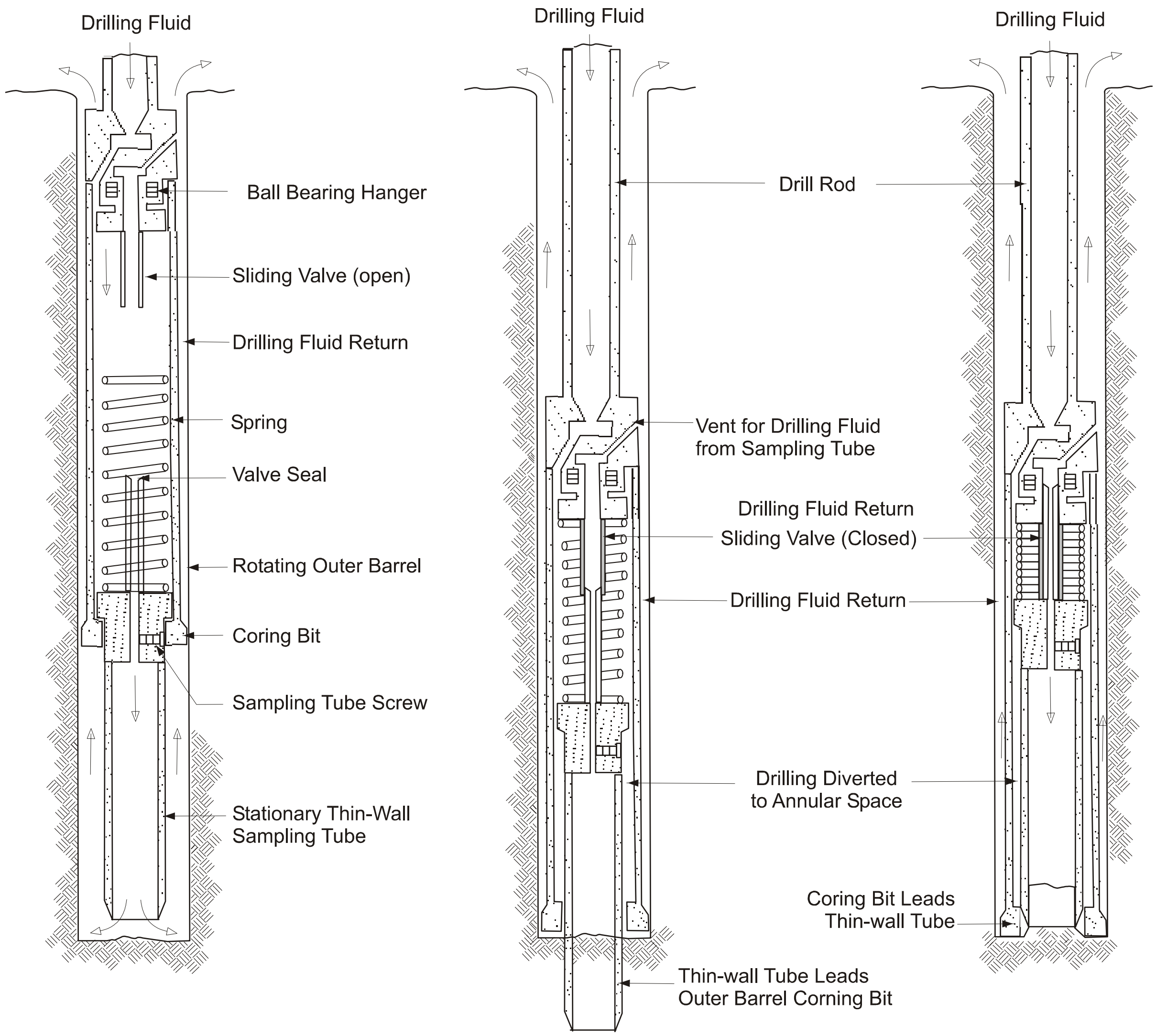


Figure 3-12. Pitcher sampler (FHWA, 1997, 2002b).

When dealing with relatively shallow soils that are very stiff, brittle, partially cemented, or that contain coarse gravel or stones, the best method to obtain large relatively undisturbed samples is by block sampling. Block sampling involves isolating a soil column, encasing it in paraffin wax, and covering it with an open-ended box or tube (usually about 12 in (300 mm) square). The bottom is cut, sealed and covered, and the sample is transported to the laboratory. This sampling technique is generally difficult to implement at depths greater than approximately 10 ft (3 m).

3.6.3 Thin-Walled (Shelby) Tube Sampling

The importance of appropriate sampling practice using Shelby tubes cannot be over-emphasized. Poor sampling practices, exposure to extreme temperatures, and careless handling of samples can cause sample disturbance that may result in misleading test results that can lead to uneconomical or unsafe designs.

- Geometry of a Thin-Walled Tube: The area ratio (AR) and the inside clearance ratio (ICR) are parameters that are used to evaluate the disturbance potential for different types of soil samplers. These parameters are defined as follows:

$$AR = \frac{D_o^2 - D_i^2}{D_i^2} \text{ x 100 percent} \qquad \text{3-1a}$$

$$ICR = \frac{D_i - D_e}{D_e} \qquad \text{3-1b}$$

  where D_e = diameter at the sampler cutting tip, D_i = inside diameter of the sampling tube, and D_o = outside diameter of the sampling tube. For a sample to be considered undisturbed, the ICR should be approximately 1 percent and the AR should be 10 percent or less. Using a tube with this ICR value minimizes the friction buildup between the soil sample and the sampler during the advancement of the sampler. Using a tube with an AR value less than 10 percent enables the sampler to cut into the soil with minimal displacement of the soil. Thin-walled tubes (e.g., Shelby tubes) are typically manufactured to meet these specifications, but a thicker walled tube with an ICR of zero is commonly used in the Gulf states (e.g., Texas, Louisiana) to sample very stiff overconsolidated clays. The use of the thicker walled tube minimizes buckling of the sampler in the stiff deposits, and the ICR of zero minimizes sample expansion within the tube. Additional information on suitable geometry for thin-walled tubes is provided in ASTM D 1587.

- Sample Tube Inspection and Storage: Tubes received from the manufacturer should be inspected to assure that no damage has occurred to the ends of the tubes. Plastic end caps, which will later be used to facilitate securing of the sample, should be placed on the ends of the tube at this time.

- Cleaning Borehole Prior to Sampling: Depending upon the methods used, drilling and sampling procedures will cause some disturbance in the vicinity of the advancing face of a borehole. This is especially the case if a sample is overpushed, if casing is advanced ahead of the borehole, or during continuous sampling operations. It is recommended that a borehole be advanced and cleaned to two to three diameters below the bottom of the previous sample to minimize disturbance. Additionally, after advancement of the borehole, caving may occur at the bottom of the hole. Thus, the bottom of the borehole should be cleaned out thoroughly before the sampling device is advanced. Improper cleaning will lead to severe disturbance of the upper material (accumulated settled material), and possibly disturbance of the entire sample. Cleaning is usually performed by washing materials out of the hole. It should be ensured that the jet holes are not directed downward, for this will erode soft or granular materials to an unknown depth. All settled material should be removed to the edge of the casing. In deep or wide borings, special cleaning augers may be used to decrease time for cleaning and produce a cleaner hole.

- Tube Advancement and Retrieval: Tubes should be advanced without rotation in a smooth and relatively rapid manner. The length of the sampler advancement should be limited to 24 in (600 mm) for a 30 in (760 mm) long tube to minimize friction along the wall of the sampler and allow for loose material in the hole. The amount of recovery should be compared to the advanced length of the sampler to assess whether material has been lost, the sample has swelled, or some caved material has been collected at the top of the tube. The possible presence of caved material should be noted at the top of the tube so that no laboratory moisture content or performance tests are performed on that material. After advancing to the target depth, the drill rod should be rotated one full turn to shear off the bottom of the sample. Prior to shearing, a waiting period of 5 to 15 minutes is recommended for tubes in soft soils to permit the sample to reach equilibrium inside the tube and prevent the sample from falling out the bottom of the tube during retrieval. This waiting period may be reduced for stiffer soils.

- Preparation for Shipment: Upon removal of the sample from the borehole, the ends should be capped with the plastic end caps and the tube should be labeled. The label should be written directly on the tube with a permanent marking pen, and include: (1) tube and boring identification number; (2) sample depth; (3) top and bottom of sample; (4) length of recovery; (5) sampling date; (6) job name and/or number; and (7) sample description. Tube samples that are intended for laboratory performance testing (i.e., strength, consolidation, hydraulic conductivity) should never be extruded from the tube in the field and stored in alternative containers. Samples should be extruded only in the laboratory under controlled conditions. After a thin-walled tube sample has been taken, slough or cuttings from the upper end of the tube should be removed by use of a cleanout tool. The length of sample recovered should be measured and the soil classified for the log. About 1 in (25 mm) of material at the bottom end of the tube should be removed and the cuttings placed in a properly labeled storage jar. Both ends of the tube should then be sealed with at least a 1 in (25 mm) thick layer of microcrystalline (non-shrinking) wax after a plastic disk has been placed to protect the ends of the sample (Figure 3-13a). The use of relatively low temperature wax will minimize shrinkage and potential moisture migration within the sample. The remaining void above the top of the sample should be filled with moist sand. Plastic end caps should then be placed over both ends of the tube and electrician's tape wrapped over the joint between the collar of the cap and the tube and over the screw holes. The capped ends of the tubes are then dipped in molten wax. Alternatively, O-ring packers can be inserted into the sample ends and then sealed (Figure 3-13b). This method of sealing the sample may be preferable as it is cleaner and more rapid than waxing. In both cases, the sample must be sealed to ensure proper preservation of the sample. The tube should be kept vertical, with the top of the sample in the upright position. If the sample needs to be inverted for purposes such as sealing, care should be taken to ensure the sample does not slide within the tube. Samples must be stored upright in a protected environment to prevent freezing, desiccation, and alteration of the moisture content (ASTM D 4220).

  Shipment: Sample tubes must to be packed **upright** in accordance with guidelines provided in ASTM D 4220, or in an equivalent sample box. Tubes should be isolated from other sample tubes, and fit snugly in the case to protect against vibration or shock. The cushioning material between the samples should be at least 1 in (25 mm) thick, and the cushioning on the container floor should be at least 2 in (50 mm) thick. The samples should not be exposed to extreme heat or cold. If possible, the geotechnical specialist should deliver the samples to the laboratory or use a special delivery service provider who offers shipping of fragile items (e.g., FedEx) to ship samples. The use of a chain of custody form for sample traceability records is encouraged.

(a) 

(b) 

Figure 3-13. Shelby tube sealing methods, (a) Microcrystalline Wax, (b) O-Ring packer (FHWA, 2002b).

3.6.4 Undisturbed Sampling of Rock (Rock Coring)

When equipment for rock coring is being considered, the dimensions, type of core barrel, type of coring bit, and drilling fluid are important variables. The minimum depth of rock coring should be determined based on the local geology of the site and the type of structure to be constructed. Coring should also be performed to a depth that assures that refusal is not encountered on a boulder. A brief description of issues related to rock coring is provided in this document. Additional information on drilling rigs, methods of circulating drill cuttings (i.e., fluid or air), hole diameters, and casings is provided in ASTM D 2113.

3.6.4.1 Core Barrels

Four different types of core barrels are described in ASTM D 2113 including:

1) Single Tube - Figure 3-14(a);
2) Rigid Double Tube - Figure 3-14(b);
3) Swivel Double Tube - Figure 3-14(c); and
4) Triple Tube.

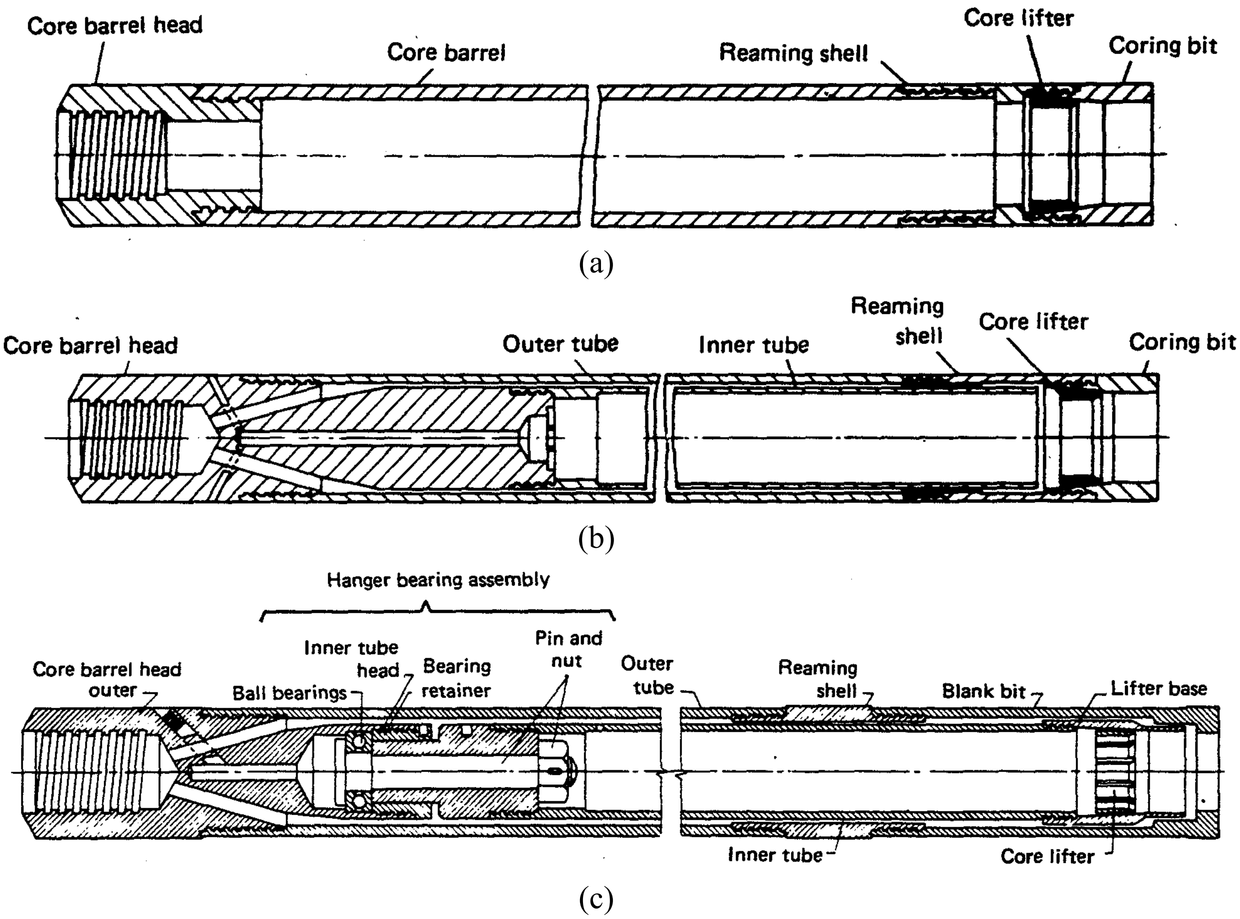


Figure 3-14. (a) Single, (b) and (c) Double tube rock core barrels (FHWA, 1997).

Since the double core barrel isolates the rock from the drilling fluid stream to yield better recovery, it is the minimum standard of core barrel that should be used in practice when an intact core is required for testing. Figure 3-15 shows the outer and inner assembly for a double-tube core barrel. The inner tube of a swivel-type core barrel does not rotate during drilling, which results in less disturbance and better recovery in weak and fractured rock. Rigid type double tube core barrels should not be used where core recovery is a concern. Triple tube swivel-type core barrels will produce better recovery and less core breakage than a double tube barrel.

Most rock coring today is done by use of the wireline method, which was introduced in the 1960s. In this method, an inner tube containing the core is detached from the core barrel assembly when the core barrel is full or a blockage occurs. The tube and core contained in it are pulled to the surface by wire dropped down the string of drill rods. A latch or "overshot assembly," which snaps on to the top of the inner tube, is used for this purpose. The inner tube is then rapidly hoisted to surface within the string of drill rods. After the core is removed, the inner tube is dropped down into the outer core barrel and drilling resumes. Thus, the core is retrieved without having to pull all of the rods and production rates, particularly for deep cores, are therefore greater than those for conventional techniques.

Table 3-7 lists the available core sizes. The standard size rock core, NX, has a diameter of 2-1/8 in (54 mm). Generally larger core sizes will lead to less mechanical breakage and yield greater recovery, but the associated cost for drilling will be much higher. Since the size of the core will affect the percent recovery, the core barrel size should be clearly recorded on the log. Additionally, the core barrel length can increase recovery in fractured and weathered rock zones. In these zones a core barrel length of 5 ft (1.5 m) is recommended. Core barrel lengths should not be greater than 10 ft (3 m) under any conditions because of the potential for damage to the long cores.

(a)

(b)

Figure 3-15. Double tube core barrel. (a) Outer barrel assembly (b) Inner barrel assembly (FHWA, 2002b).

3.6.4.2 Coring Bits

The coring bit is the bottommost component of the core barrel assembly. It is the grinding action of this component that cuts the core from the rock mass. The following three basic categories of bits are in use: diamond, carbide and sawtooth (Figure 3-16).

Coring bits are generally selected by the driller and are often approved by the geotechnical specialist. Bit selection should be based on a general knowledge of drill bit performance for the expected formations and the proposed drilling fluid.

Table 3-7
Dimensions of core sizes (FHWA, 1997)

Size	Diameter of Core mm (in)	Diameter of Borehole mm (in)
EX,EXM	21.5 (0.846)	37.7 (1.484)
EWD3	21.2 (0.835)	37.7 (1.484)
AX	30.1 (1.185)	48.0 (1.890)
AWD4, AWD3	28.9 (1.138)	48.0 (1.890)
AWM	30.1 (1.185)	48.0 (1.890)
AQ Wireline, AV	27.1 (1.067)	48.0 (1.890)
BX	42.0 (1.654)	59.9 (2.358)
BWD4, BWD3	41.0 (1.614)	59.9 (2.358)
BXB Wireline, BWC3	36.4 (1.433)	59.9 (2.358)
BQ Wireline, BV	36.4 (1.433)	59.9 (2.358)
NX	54.7 (2.154)	75.7 (2.980)
NWD4,NWD3	52.3 (2.059)	75.7 (2.980)
NXB Wireline, NWC3	47.6 (1.874)	75.7 (2.980)
NQ Wireline, NV	47.6 (1.874)	75.7 (2.980)
HWD4,HXB Wireline,	61.1 (2.406)	92.7 (3.650)
HQ Wireline	63.5 (2.500)	96.3 (3.791)
CP, PQ Wireline	85.0 (3.346)	122.6 (4.827)

Figure 3-16. Coring bits: Diamond (top left), Carbide (top right), and Sawtooth (bottom center) (FHWA, 2002b).

Diamond coring bits, such as surface set or impregnated-diamond types, are the most versatile since they can produce high-quality cores in soft to extremely hard rock materials (see Figure 3-10, top left). Compared to other types, diamond bits in general permit more rapid coring and, as noted by Hvorslev (1948), exert lower torsional stresses on the core. Lower torsional stresses permit the retrieval of longer cores and cores of smaller diameter. The wide variation in the hardness, abrasiveness, and degree of fracturing encountered in rock has led to the design of bits to meet specific conditions known to exist or expected to be encountered at given sites. Thus, wide variations in the quality, size, and spacing of diamonds, in the composition of the metal matrix, in the face contour, and in the type and number of waterways are found in bits of this type. Similarly, the diamond content and the composition of the metal matrix of impregnated bits are varied to meet differing rock conditions.

Carbide bits use tungsten carbide in lieu of diamonds. There are of several types of carbide bits. The standard type carbide bit is shown in Figure 3-16, top right. Bits of this type are used to core soft to medium hard rock. They are less expensive than diamond bits. However, the rate of drilling is slower than with diamond bits.

Sawtooth bits consist of teeth cut into the bottom of the bit (see Figure 3-10, bottom center). The teeth are faced and tipped with a hard metal alloy such as tungsten carbide to provide water resistance and thereby to increase the life of the bit. Although these bits are less expensive than diamond bits, they do not provide as high a rate of coring and do not have a salvage value. The saw tooth bit is used primarily to core overburden and very soft rock.

An important feature in all bits is the type of waterways provided in the bits for the passage of drilling fluid. Bits are available with so-called “conventional” waterways, which are passages cut on the interior face of the bit, or with bottom discharge waterways, which are internal passages that discharge at the bottom face of the bit behind a metal skirt separating the core from the discharge fluid. Bottom discharge bits should be used when soft rock or rock having soil-filled joints is cored to prevent erosion of the core by the drilling fluid before the core enters the core barrel.

Bit selection is based on the anticipated rock formation as well as the expected drilling fluid. Diamond bits are applicable in all rock types. They permit greater rates of coring than other types of bits. Carbide bits are less expensive than diamond bits and can be used in soft to medium-hard rock. Sawtooth bits are the least expensive of the three, however they have no salvage value. They lead to slower coring and are typically used only in soft rock.

3.6.4.3 Drilling Fluid

In many instances, clear water is used as the drilling fluid in rock coring. If drilling mud is required to stabilize collapsing holes or to seal zones when there is loss of drill water, the geotechnical specialist should be notified to confirm that the type of drilling mud is acceptable. Drilling mud will clog open joints and fractures, which adversely affects permeability measurements and piezometer installations. Drilling fluid should be contained in a settling basin to remove drill cuttings and to allow recirculation of the fluid. Generally, drilling fluids can be discharged onto the ground surface. However, special precautions or handling may be required if the material is contaminated with oil or other substances. Such fluids may require disposal off site. Water flow over the ground surface should be avoided as much as possible. Local environmental agencies should be contacted for permits because some drilling fluids may have adverse effects on local surface and subsurface environments. Certain local agencies may also require implementation of a Storm Water Pollution Prevention Plan (SWPPP).

3.6.5 Observations During Rock Core Drilling

3.6.5.1 Drilling Rate/Time

The drilling rate should be monitored and recorded on the boring log in the units of minutes per 1 ft (0.3 m). Only time spent advancing the boring should be used to determine the drilling rate.

3.6.5.2 Core Photographs

Cores in the split core barrel should be photographed immediately upon removal from the borehole. A label should be included in the photograph to identify the borehole, the depth interval and the number of the core run. It may be desirable to get a "close-up" of interesting features in the core. Wetting the surface of the recovered core by using a spray bottle and/or sponge prior to photographing will often enhance the color contrasts of the core.

A tape measure or ruler should be placed across the top or bottom edge of the core box to provide a scale in the photograph. The tape or ruler should be at least 3 ft (1 m) long, and it should have relatively large, high contrast markings to be visible in the photograph.

A color bar chart is often desirable in the photograph to provide indications of the effects of variation in film age, film processing, and the ambient light source. The photographer should strive to maintain uniform light conditions from day to day, and those lighting conditions should be compatible with the type of film selected for the project. The use of a digital camera is a convenient way to circumvent some of the problems associated with the use of film cameras for photographing rock cores.

3.6.5.3 Rock Classification

The rock type and its inherent discontinuities, joints, seams, and other facets should be documented. See Chapter 4 for a discussion of rock description and classification.

3.6.5.4 Recovery

The core recovery is the length of rock core recovered from a core run. The recovery ratio is the ratio of the length of core recovered to the total length of the core drilled on a given run, expressed as either a fraction or a percentage. Core length should be measured along the core centerline. When the recovery is less than the length of the core run, the non-recovered section should be assumed to be at the end of the run unless there is reason to suspect otherwise (e.g., weathered zone, drop of rods, plugging during drilling, loss of fluid, and rolled or re-cut pieces of core). Non-recovery should be marked as NCR (no core recovery) on the boring log, and entries should not be made for bedding, fracturing, or weathering in that interval.

Recoveries greater than 100 percent may occur if core that was not recovered during a run is subsequently recovered in the next run. Recoveries greater than 100 percent should be recorded and adjustments to data should not be made in the field.

3.6.5.5 Rock Quality Designation (RQD)

The RQD is a quantitative measure that represents a modified core recovery percentage. By definition the RQD is the sum of the lengths of all pieces of sound core over 4 in (100 mm) long divided by the length of the core run (Deere, 1963). The correct procedure for measuring RQD is illustrated in Figure 3-17. The RQD is an index of rock quality. Problematic rock that is highly weathered, soft, fractured, sheared, and jointed typically yields lower RQD values than more intact rock. Thus, RQD is simply a measurement of the percentage of "good" rock recovered from an interval of a borehole.

It should be noted that the original definition of RQD reported by Deere (1963) was based on measurements made on NX-size core. Experience in recent years reported by Deere and Deere (1989) indicates that cores with diameters both slightly larger and smaller than NX may be used for computing RQD. The wire line cores using NQ, HQ, and PQ are also considered acceptable. Use of RQD for the smaller BQ and BX sizes is discouraged because of a greater potential for core breakage and loss that would result in a smaller value of RQD.

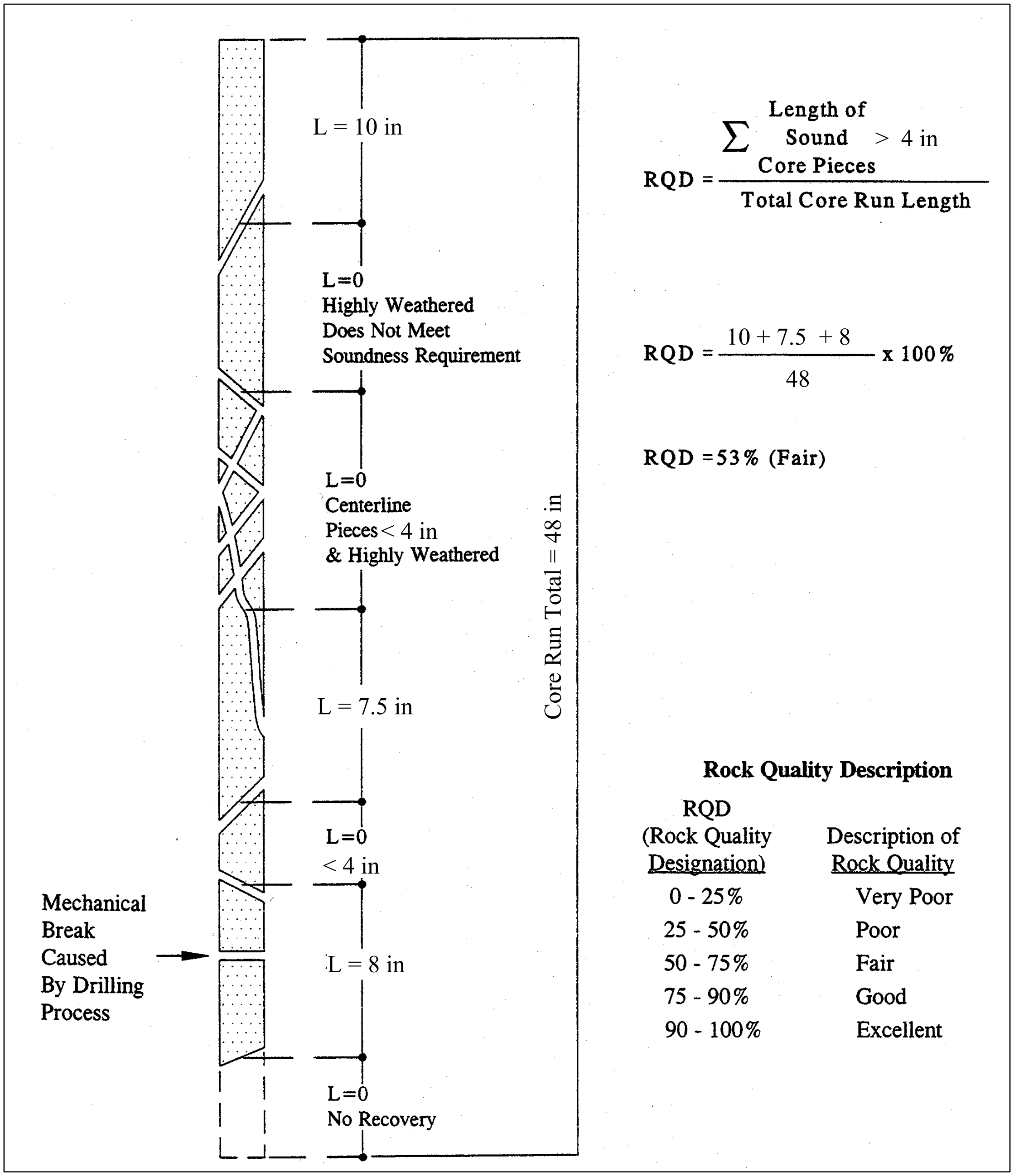


Figure 3-17. Modified core recovery as an index of rock mass quality (FHWA, 1997).

Length Measurements of Core Pieces

The same piece of core could be measured three ways: along the centerline, from tip to tip, or along the fully circular barrel section (Figure 3-18). The recommended procedure is to measure the core length along the centerline. This method is advocated by the International Society for Rock Mechanics (ISRM), Commission on Standardization of Laboratory and Field Tests (ISRM, 1981). The centerline measurement is preferred because: (1) it results in a standardized RQD not dependent on the core diameter, and (2) it avoids unduly penalizing the rock quality for cases where fractures run parallel to the borehole and are cut by a second set of fractures.

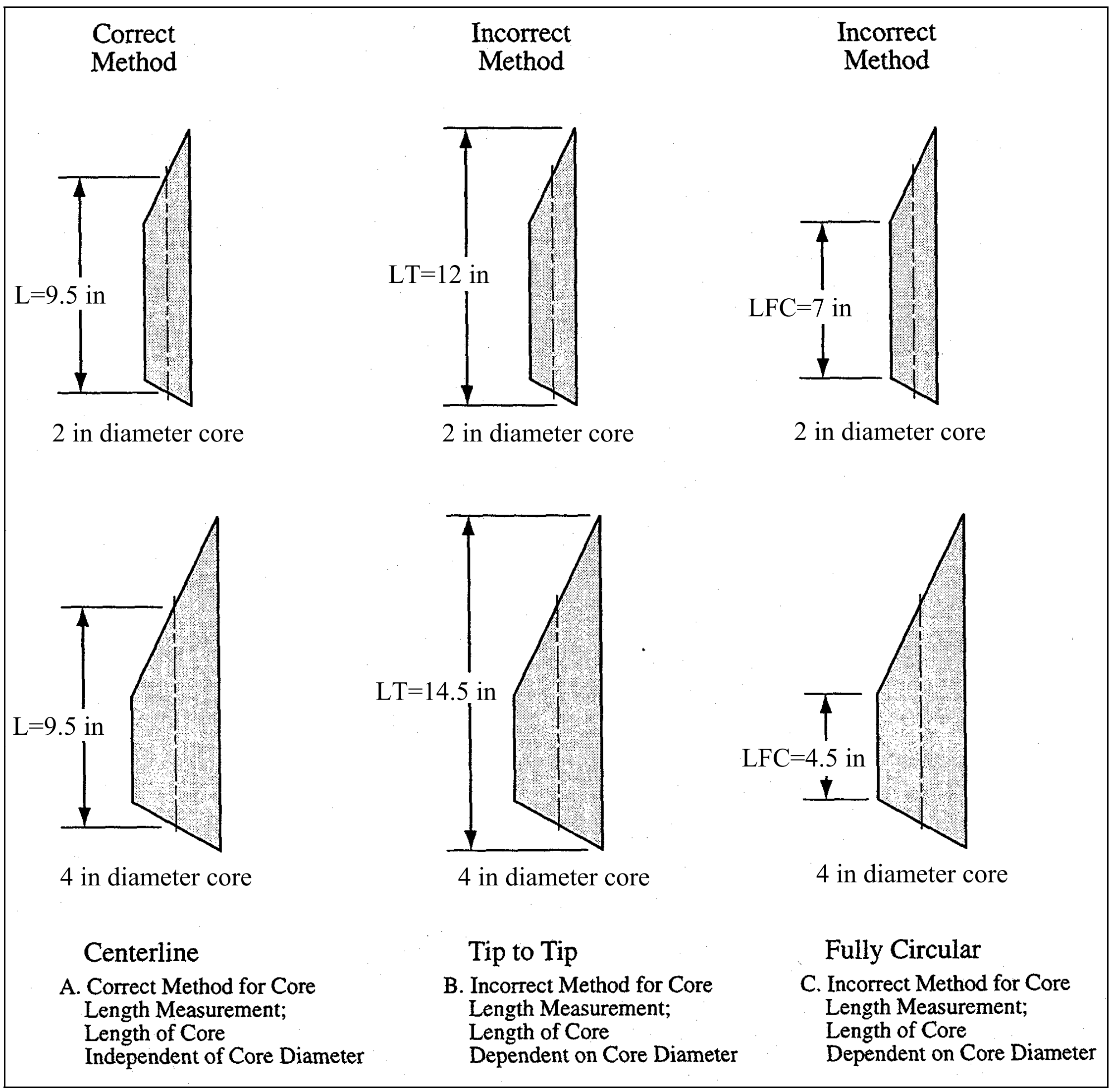


Figure 3-18. Length measurements for core RQD determination (FHWA, 1997).

Assessment of Soundness

Pieces of core which are not "hard and sound" should not be counted for the RQD even though they possess the requisite 4 in (100 mm) length. The purpose of the soundness requirement is to downgrade the rock quality where the rock has been altered and weakened either by agents of surface weathering or by hydrothermal activity. Obviously, in many instances a judgment decision must be made as to whether or not the degree of chemical alteration is sufficient to reject the core piece.

One commonly used procedure is not to count a piece of core if there is any doubt about its meeting the soundness requirement as evidenced by discolored or bleached grains, heavy staining, pitting, or weak grain boundaries. This procedure may unduly penalize the rock quality, but it errs on the side of conservatism. A second procedure that is occasionally used includes the altered rock within the RQD summed percentage, but indicates by means of an asterisk (RQD*) that the soundness requirements have not been met. The advantage of this method is that the RQD* will provide some indication of the rock quality with respect to the degree of fracturing, while also noting its lack of soundness.

Core breaks caused by the drilling process should be fitted together and counted as one piece. Drilling breaks are usually evidenced by rough fresh surfaces. For schistose and laminated rocks, it is often difficult to discern the difference between natural breaks and drilling breaks. When in doubt about a break, it should be considered as natural in order to be conservative in the calculation of RQD for most uses. Obviously, this practice would not be conservative when the RQD is used as part of a ripping or dredging estimate.

3.6.5.6 Drilling Fluid Recovery

The loss of drilling fluid during the advancement of a boring can be indicative of the presence of open joints, fracture zones or voids in the rock mass being drilled. Therefore, the volumes of fluid losses and the intervals over which they occur should be recorded. For example, "no fluid loss" means that no fluid was lost except through spillage and filling the hole. "Partial fluid loss" means that a return was achieved, but the amount of return was significantly less than the amount being pumped in. "Complete water loss" means that no fluid returned to the surface during the pumping operation. A combination of opinions from the field personnel and the driller on this matter will result in the best estimate.

3.6.5.7 Core Handling and Labeling

Rock cores from geotechnical explorations should be stored in structurally sound core boxes made of wood or corrugated waxed cardboard (Figure 3-19). Wooden boxes should be provided with hinged lids, with the hinges on the upper side of the box and a latch to secure the lid in a closed position.

Cores should be handled carefully during transfer from barrel to box to preserve mating across fractures and fracture-filling materials. Breaks in core that occur during or after the core is transferred to the core box should be refitted and marked with three short parallel lines across the fracture trace to indicate a mechanical break. Breaks made to fit the core into the core box and breaks made to examine an inner core surface should be marked as such. These deliberate breaks should be avoided unless absolutely necessary. Cores should be placed in the boxes from left to right, top to bottom. When the upper compartment of the box is filled, the next lower (or adjoining) compartment should be filled beginning at the left-hand side, and so on the same way until the box is filled. The depths of the top and bottom of the core and each noticeable gap in the formation should be marked by a clearly labeled wooden spacer block.

If there is less than 100 percent core recovery for a run, a cardboard tube spacer of the same length as the core loss should be placed in the core box either at the depth of core loss, if known, or at the bottom of the run. The depth of core loss, if known, or length of core loss should be marked on the spacer with a black permanent marker. The core box labels should be completed using an indelible black marking pen. An example of recommended core box markings is shown in Figure 3-19. The core box lid should have identical markings both inside and out, and both exterior ends of the box should be marked as shown in Figure 3-19. For angled borings, depths marked on core boxes and boring logs should be those measured along the axis of the boring. The angle and orientation of the boring should be noted on the core box and the boring log.

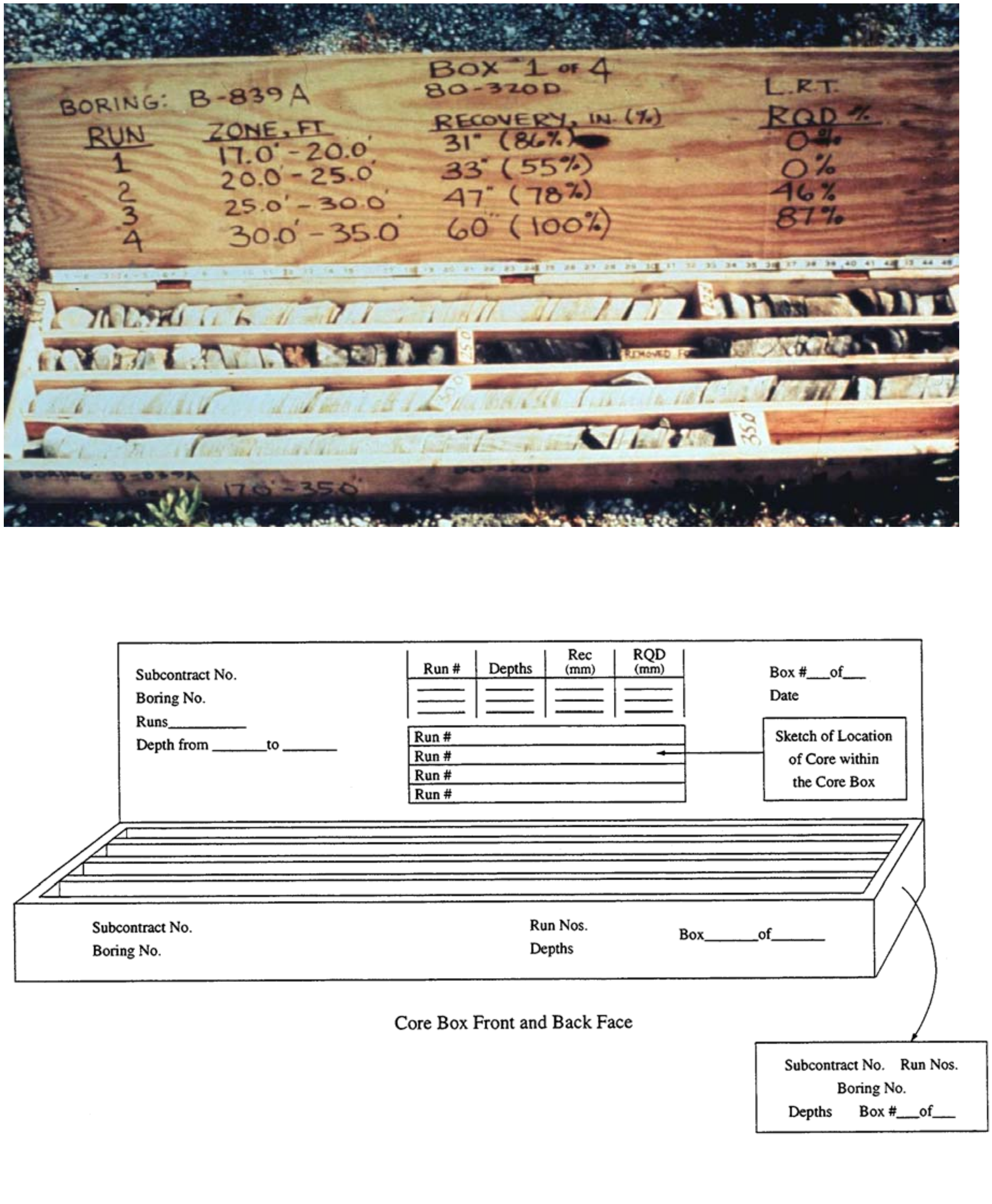


Figure 3-19. Core box for storage of recovered rock and labeling.

3.6.5.8 Care and Preservation of Rock Samples

A detailed discussion of sample preservation and transportation is presented in ASTM D 5079. Four levels of sample protection are identified as follows:

a) routine care,
b) special care,
c) soil-like care, and
d) critical care.

Routine care in placing rock core in core boxes will be used for most geotechnical explorations. ASTM D 5079 suggests enclosing the core in a loose-fitting polyethylene sleeve prior to placing the core in the core box.

Special care is considered appropriate if the moisture state of the rock core (especially shale, claystone and siltstone) and the corresponding properties of the core may be affected by exposure. Special care can also be applied if it is important to maintain fluids other than water in the sample. Critical care is needed to protect samples against shock and vibration or variations in temperature, or both. For soil-like care, samples should be treated as indicated in ASTM D 4220.

3.6.6 Geologic Mapping

Geologic mapping is the systematic collection of local, detailed geologic data, and, for engineering purposes, is used to characterize and document the condition of a rock mass or outcrop. The data derived from geologic mapping are a portion of the data required for the design of a cut slope or for the stabilization of an existing slope. Geologic mapping can often provide more extensive and less costly information than drilling. Soil and soil-like materials, although occasionally mapped, are not considered in this section. For a detailed discussion of geologic mapping, the reader is referred to the FHWA manual on rock slopes (FHWA, 1998a).

3.7 STANDARD PENETRATION TEST (SPT)

The standard penetration test (SPT) is performed during the advancement of a soil boring to obtain a disturbed drive sample (split barrel type) of the soil being penetrated and an approximate measure of its dynamic resistance. The test was introduced by the Raymond Pile Company in 1902 and remains today as the most common in-situ test performed worldwide. The procedures for the SPT are detailed in ASTM D 1586 and AASHTO T 206. A summary of the important features of the test follows.

The SPT involves the driving of a hollow thick-walled tube into the ground and measuring the number of blows to advance the split-barrel sampler having standard dimensions of 2 in (50 mm) outside diameter (OD) and 1-3/8 in (35 mm) inside diameter (ID) a vertical distance of 1 ft (300 mm), see Figure 3-20. A 140 pound (63.5 kilogram) hammer is repeatedly dropped from a height of 30 in (0.76 m) to achieve three successive 6 in (150 mm) increments of penetration. The first recorded increment is considered as a “seating” penetration, while the number of blows to advance the second and third increments are summed to give the N-value ("blow count") or SPT-resistance (reported in blows per foot (0.3 m)).

The SPT can be halted when a total of 100 blows have been counted or if the number of blows exceeds 50 in any given 6 in (150 mm) increment, or if the sampler fails to advance during 10 consecutive blows. SPT refusal is defined by penetration resistances exceeding 100 blows per 2 in (50 mm), although ASTM D 1586 has re-defined this limit at 50 blows per 1 in (25 mm). If bedrock, or an obstacle such as a boulder, is encountered, the boring may be advanced further by using diamond core drilling or non-core rotary methods (ASTM D 2113; AASHTO T 225) at the discretion of the geotechnical specialist. In certain cases, this SPT criterion may be utilized to define the top of bedrock within a particular geologic setting where boulders are not of concern or not of great impact on the project requirements. The advantages and disadvantages of the SPT are listed in Table 3-8.

Table 3-8
Advantages and disadvantages of the Standard Penetration Test (SPT)

Advantages	Disadvantages
• Obtain both a sample and an N-value • Simple and rugged • Suitable in many soil types • Can be performed in weak rocks • Readily available throughout the U.S.	• Disturbed sample (index tests only) • N-value is a crude number for analysis • Not applicable in soft clays & loose silts • High variability and uncertainty • Unreliable in gravelly soils

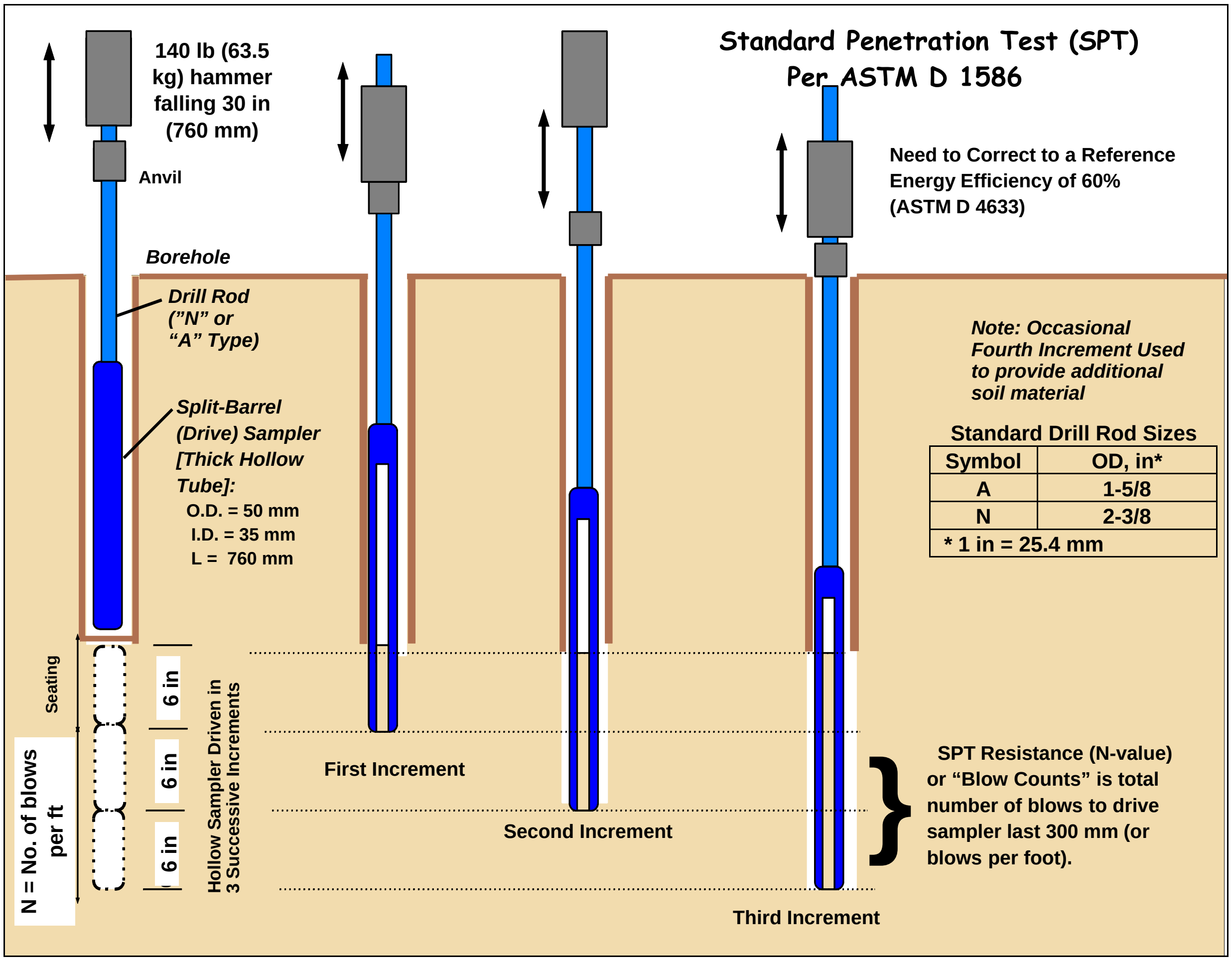


Figure 3-20. Sequence of driving split-barrel sampler during the Standard Penetration Test (modified after FHWA, 2002b).

The SPT is conducted at the bottom of a soil boring that has been advanced by use of either flight augers or rotary wash drilling methods. The borehole can be cased or uncased. At regular depth intervals, the drilling process is interrupted to perform the SPT. Generally, at depths shallower than 10 ft (3 m) the SPT is performed continuously or at intervals of 2.5 ft (0.75 m). Below a depth of 10 ft (3 m) the SPT is generally performed at intervals of 5 ft (1.5 m) to the planned end of the boring or refusal. If the borehole extends below the groundwater table, the head of water in the borehole must be maintained at or above the ambient groundwater level to avoid inflow of water and borehole instability.

Liners may be placed inside the split-barrel sampler with the same inside diameter as the cutting shoe, see Figure 3-21a. This allows samples to remain intact during transport to the laboratory. The liners may be arranged in a set of 1-inch (25 mm) high rings in which case "ring" samples of pre-determined height may be obtained. In U.S. practice, it is normal to omit the liner. The resistance of the sampler to driving is altered depending upon whether or not a liner is used (Skempton 1986, Kulhawy and Mayne, 1990). Therefore, **when the liners are used, their use should be clearly mentioned in the boring logs.**

Steel or plastic sampler "catchers" are often required to keep samples of clean granular soils in the split-barrel sampler. Figure 3-21 shows a variety of catchers. They are inserted inside the sampler between the cutting shoe and the sample barrel to help retain loose or flowing materials. These catchers permit the soil to enter the sampler during driving but upon withdrawal they close and thereby retain the sample. **Use of sample catchers should be noted on the boring log.**

(a)

(b)

Figure 3-21: (a) Stainless steel and brass liners, (b) Sample catchers (FHWA, 2002b).

3.7.1 Energy Efficiency of Hammers

In current U.S. practice, three types of drop hammers and four types of drill rods are used in performing the SPT. Drop hammer types are typically donut, safety, and automatic (see Figure 3-22). Typical drill rod sizes are N or A (see Figure 3-20 for sizes). The test results are highly dependent upon the type of equipment used and the experience of the operator performing the test. One of the more important factors for obtaining useful data from the test is the energy efficiency of the system. The theoretical energy of a free-fall system with the specified mass and drop height is 350 ft-lb (48 kg-m), but the actual energy is less due to a number of factors including frictional losses and eccentric loading that are specific to the hammer drop. The energy efficiency of the rotating cathead and rope system commonly used in the past depends on numerous factors including: type of hammer, number of rope turns, conditions of the sheaves and rotating cathead (e.g., lubricated, rusted, bent, new, old), age of the rope, actual drop height, vertical plumbness, weather and moisture conditions (e.g., wet, dry, freezing), and other variables (see for example Skempton, 1986). In the recent past the trend has been towards the use of automated systems for lifting and dropping the mass in order to minimize these factors. Automated systems provide more reliable and more reproducible results than the rotating cathead and rope system used in the past.

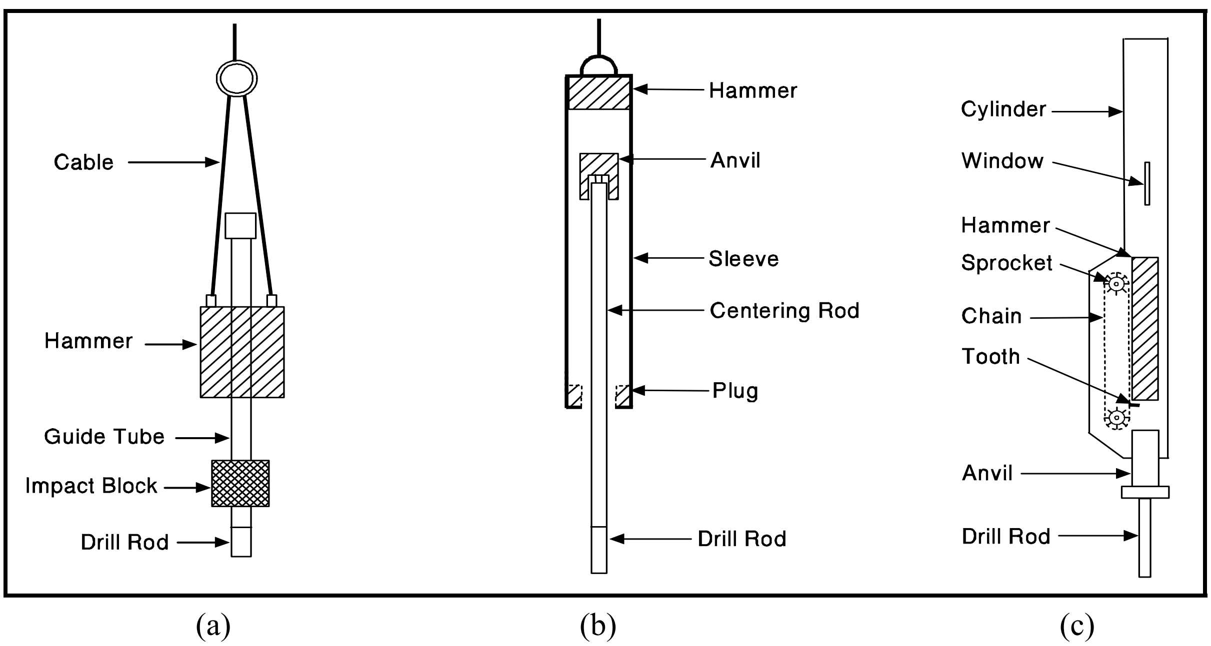


Figure 3-22. SPT hammer types, (a) Donut, (b) Safety, and (c) Automatic (FHWA, 2006a).

A calibration of energy efficiency for a specific drill rig and operator is recommended by ASTM D 4633. Instrumented strain gages and accelerometer measurements are used for these calibrations in an attempt to standardize the energy levels. The standard of practice for energy efficiency varies from about 35% to 85% with cathead systems using donut or safety hammers. The average for cathead systems in the United States is approximately 60%. The newer automatic trip-hammers can deliver between 80 to 100% efficiency, depending upon the type of commercial system being used.

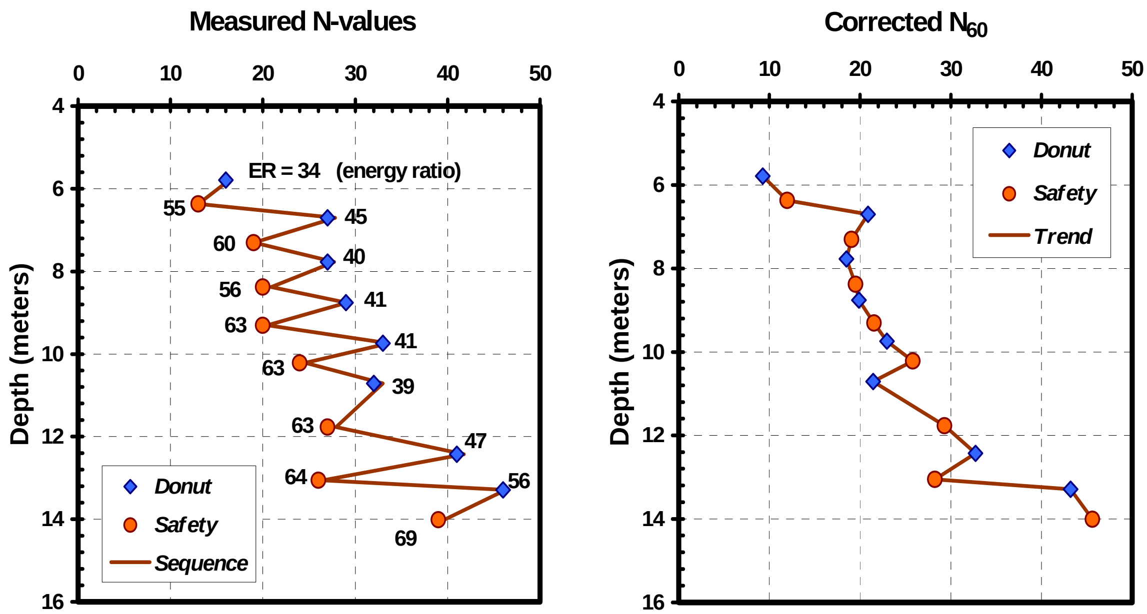
If energy efficiency (E_f) is measured, then the energy-corrected SPT N-value adjusted to 60% efficiency (N_{60}) is given by:

$$N_{60} = (E_f/60)\, N_{meas} \qquad 3\text{-}2$$

where N_{meas} is the N-value measured in the field during the test. N-values measured in the field should be corrected to N_{60} for all soils, if possible. The relative magnitudes of corrections for energy efficiency, sampler lining, rod lengths, and borehole diameter are given by Skempton (1986) and Kulhawy and Mayne (1990), but only as general guidelines. Theoretically it is mandatory to measure E_f to get the proper correction to N_{60}. In absence of data, AASHTO (2004 with 2006 Interims) recommends $E_f = 60$ for rope and cathead systems, i.e., donut and safety hammers and $E_f = 80$ for automatic hammer systems.

The efficiency may be obtained by comparing either the work done (W = F.d = force times displacement) or the kinetic energy ($KE = \frac{1}{2}mv^2$) with the potential energy of the system ($PE = mgh$), where m = mass, v = impact velocity, $g = 32.2\ ft/s^2 = 9.8\ m/s^2$= gravitational constant, and h = drop height. Thus, the energy ratio (ER) is defined as W/PE, or ER = KE/PE. **It is important to note that geotechnical foundation practice and engineering usage based on SPT correlations have been developed on the basis of the standard-of-practice, corresponding to an average ER ≈ 60 %. Thus, it is recommended to adjust measured N-values (N_{meas}) to N_{60} values.**

Figure 3-23 exemplifies the need for correcting measured N-values to a reference energy level where the successive SPTs were conducted by alternating the use of donut and safety hammers in the same borehole. The energy ratios were measured for each test and gave 34 < ER < 56 for the donut hammer (average = 45%) and 55 < ER < 69 for the safety hammer (average = 60%) at this site. The individual trends for the measured N-values from donut and safety hammers are quite apparent in Figure 3-23(a), whereas a consistent profile is obtained in Figure 3-23(b) once the data have been corrected to ER = 60%.



(a) (b)

Figure 3-23. SPT N-values from (a) Uncorrected data, and (b) Corrected to 60% efficiency (Data modified after Robertson and Campanella, 1983).

3.7.2 Effect of Overburden Stress on N-values

Since N-values of similar materials increase with increasing effective overburden stress, the corrected blow count (N_{60}) is often normalized to 1-atmosphere (1 tsf or about 100 kPa) effective overburden stress by using overburden normalization schemes. The energy-corrected blow count normalized for overburden is referred to as $N1_{60}$, and is equal to:

$$N1_{60}=C_N N_{60} \qquad 3\text{-}3$$

where C_N is the overburden correction factor (or stress normalization parameter) calculated as (Peck, *et al.*, 1974):

C_N = $[0.77 \log_{10}(20/p_o)]$, and $C_N < 2.0$

p_o = vertical effective pressure at the depth where the SPT test is performed (tsf)

N_{60} = SPT blow count corrected for hammer efficiency (blows/ft) – refer to Equation 3-2.

Note that the constants in Equation 3-3 are unit dependent therefore the units of p_o must be tsf. Figure 3-24 presents the overburden correction factor as a function of vertical effective stress.

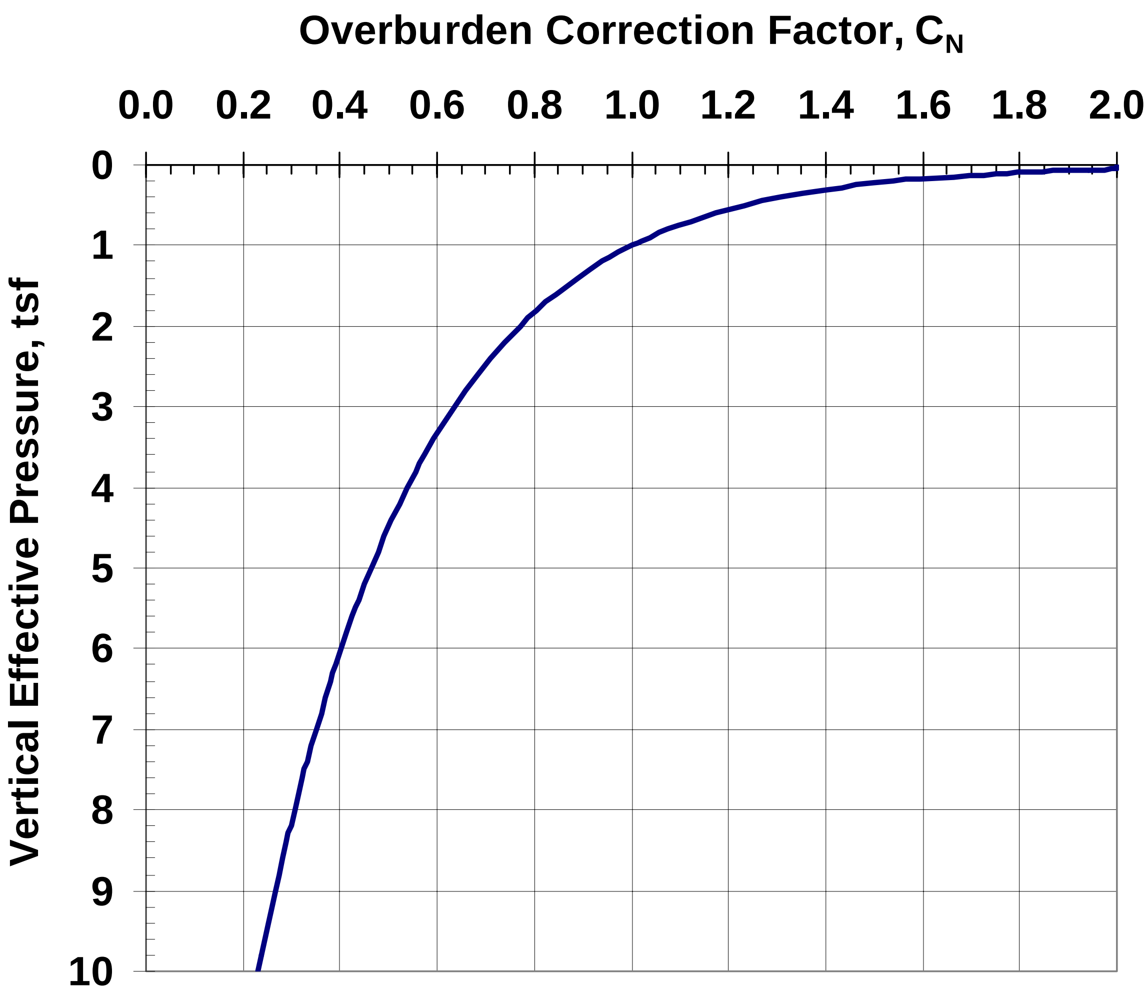


Figure 3-24. Variation of overburden correction factor, C_N, as a function of vertical effective stress.

Caution should be exercised in applying the overburden correction factor to indurated cemented soils, e.g., hard caliche soils encountered in the desert southwest. In such soils, the overburden pressure may not be a direct function of the depth of the soil. Therefore, the overburden correction is not recommended for such soils since it may lead to overly conservative designs.

3.7.3 Correlation of SPT N-Values with Basic Soil Characteristics

SPT N-values are an indication of the relative density of cohesionless soils and the consistency of cohesive soil. Table 3-9 shows N-value ranges correlated to the relative density of sands and the consistency of fine-grained soils. It is emphasized that these correlations are unreliable for gravels, silts and clays and should serve only as crude estimates for these materials.

Table 3-9
Soil properties correlated with Standard Penetration Test values (after Peck, *et al.*, 1974)

Sands (Reliable)		Silts and Clays (Unreliable)	
N_{60}	Relative Density	N_{60}	Consistency
0-4	Very loose	Below 2	Very soft
5-10	Loose	2-4	Soft
11-30	Medium Dense	5-8	Medium
31-50	Dense	9-15	Stiff
Over 50	Very dense	16-30	Very stiff
		Over 30	Hard

3.7.3.1 Applicability of SPTs in Gravelly Soils

The SPT can be performed in a wide variety of soil types as well as weak rocks, however the SPT is not particularly useful in the characterization of gravelly soils. Since the split-spoon inside diameter is 1-⅜ in (35 mm), gravel sizes larger than 1-⅜ in (35 mm) will not enter the spoon. Therefore, soil descriptions may not reflect actual gravel content of the deposit. Also, gravel pieces may plug the end of the spoon and cause the SPT blow count to be erroneously large. Thus, the SPT in such cases produces refusal blow counts (i.e., > 50 blows per 1 in (25 mm)) that are misleading and lead to unconservative designs. In this case, "Large Penetration Tests" (LPTs), such as the Becker Penetration Test (BPT), are more suitable. The LPTs consist of driving a pipe (casing) larger than the standard split spoon sampler into the ground with a pile-driving hammer. While the pipe is being driven, the driving resistance or blow count/ft of penetration is recorded. Unlike the SPT, the LPT blow count is non-standardized and is a function of the drill rod size, pipe (sampler) size, hammer type, and hammer efficiency. Careful energy calibrations are required to correlate the LPT blow counts to SPT N-values However, this effort may be worthwhile considering that the results of SPT in gravelly soils are unreliable and misleading. Daniel, *et al.* (2003) present methods for evaluating LPT blow counts.

Since the gravel content cannot be measured by the SPT, it is recommended that consideration be given to obtaining bulk samples by drilling large diameter borings with augers similar to the one shown in Figure 3-5a. The bulk samples obtained from such borings will also help evaluate whether the soil deposit is indeed a gravel deposit or gravels are larger particles floating in a softer soil matrix. The bulk samples will also permit an accurate determination of the Unified Soil Classification System (USCS) designation (see Chapter 4), which will be useful from design as well as constructability considerations.

3.7.4 SPT Test Errors

Although the procedures for conducting the SPT test have been standardized, several errors can creep into the test. The most common errors are:

1. Effect of overburden pressure. Soils of the same density will give smaller blow counts near the ground surface. The overburden stress normalization parameter (C_N) can be used to correct for this factor.

2. Variations in the 30 in (770 mm) free fall of the drive weight. The drop height is often gauged by eye with the older rotating cathead and rope system Newer hammer systems automatically release the weight at a height of 30 inches. The energy correction factor accounts for this factor.

3. Interference with the free fall of the drive weight by the guides or the hoist rope required in the rotating cathead and rope system. Newer automatic hammer systems eliminate rope interference. The energy correction factor accounts for this factor.

4. Use of a drive shoe that is damaged or worn from too many "refusal" blow counts ($N_{meas} \geq 100$ blows/foot).

5. Failure to seat the sampler properly on undisturbed material in the bottom of the boring.

6. Inadequate cleaning of loosened material (slough) from the bottom of the boring.

7. Failure to maintain sufficient hydrostatic pressure in the borehole during drilling below the groundwater table. Unbalanced hydrostatic pressures between the borehole drill water and the groundwater table can cause the test zone to become "quick." This can happen when a continuous-flight auger is used with the end plugged and with a water level in the hollow stem below that in the hole.

8. Effect of gravel size as discussed in Section 3.7.3.1.

9. Samples retrieved from dilatant soils (fine sands, sandy silts) that exhibit unusually high blow counts should be examined in the field to determine if the sampler drive shoe is plugged. Poor sample recovery is usually an indication of plugging.

10. Careless work on the part of the drill crew.

The use of qualified and experienced drillers cannot be overemphasized. Agencies that maintain their own drilling personnel and equipment generally achieve much more reliable and consistent results than those that routinely let boring contracts to the lowest bidder.

Soil type, density, and overburden pressure are the most significant factors affecting SPT N-values (assuming good workmanship and equipment). Table 3-10 lists factors affecting the SPT and SPT results.

Regardless of the impressive list of shortcomings, the SPT is not likely to be abandoned for several reasons:

1. The test is very economical in terms of cost per unit of information.

2. The test results provides soil samples, which can be tested for index properties and visually examined.

3. Long service life of the enormous amount of equipment in use.

4. The accumulation of a large SPT database that is continually expanding.

5. The results of the SPT have been correlated with a number of soil properties to provide estimates of the values of those properties. The estimated values are often used for preliminary designs in lieu of values obtained from tests run specifically to determine those properties.

6. The fact that other methods can be readily used to supplement the SPT when the borings indicate more refinement in sample/data collection.

Table 3-10
Factors affecting the SPT and SPT results (after Kulhawy and Mayne, 1990)

Cause	Effects	Influence on SPT N-value
Inadequate cleaning of hole	SPT is performed in loose slough. Therefore soil may become trapped in sampler and may be compressed as sampler is driven, reducing recovery	Increases
Failure to maintain adequate head of water in borehole when test is performed below groundwater level	Bottom of borehole may become "quick"	Decreases
Careless measure of hammer drop	Hammer energy varies (generally variations cluster on low side)	Increases
Hammer weight inaccurate	Hammer energy varies (driller supplies weight; variations of 5 - 7 percent are common)	Increases or decreases
Hammer strikes drill rod collar eccentrically	Hammer energy reduced	Increases
Lack of hammer free fall because of ungreased sheaves, new stiff rope on weight, more than two turns on cathead, incomplete release of rope each drop	Hammer energy reduced	Increases
Sampler driven above bottom of casing	Sampler driven in disturbed, artificially densified soil	Increases greatly
Careless counting of hammer blows	Inaccurate results	Increases or decreases
Use of non-standard sampler	Correlations with SPT sampler invalid	Increases or decreases
Coarse gravel or cobbles in soil	Sampler becomes clogged or impeded	Increases
Use of bent drill rods	Inhibited transfer of energy of sampler	Increases

3.8 LOG OF BOREHOLE INFORMATION ("BORING LOGS")

The importance of accurate field notes and good logging of boreholes cannot be overemphasized. The logger must realize that a good field description must be recorded. The field-boring log is the major portion of the factual data used in the analysis of foundation conditions.

The boring log is a record that should contain all of the information obtained from a boring whether or not it may seem important at the time of drilling. It is important to record the maximum amount of information accurately. This record is the "field" boring log, as opposed to the "finished" boring log used in the preparation of the geotechnical data report. The finished log is drawn from the data presented in the field log supplemented by the results of visual identifications of samples and classification tests made in the laboratory. A typical boring log form is shown on Figures 3-25. The form presented in Figures 3-25 can be used for recording ***field*** data as well.

3.8.1 Boring Log Format

A wide variety of boring log forms are used by various agencies. The specific log to be used for a given type of boring will depend on local practice. The log in Figures 3-25 is just one example of a log used by geotechnical specialists. For detailed information on boring logs, the reader can refer to FHWA (1997, 2002b). The boring log shown in Figures 3-25 is used in this document simply to present the reader with an idea of the basic information that should be included in a boring log. Specific projects will likely require more detailed logs. Often separate logs are used for logging information from borings in soils and rocks unlike the log shown in Figures 3-25, which combines this information.

3.8.2 Duties of the Logger

The technical background and experience of the person who logs the field information will vary by organization. Some organizations will have a geotechnical engineer, an engineering geologist, a geologist, or a trained technician to accompany the drill crew, while others may train the drill crew foreman to log the borehole. In order to obtain the maximum amount of accurate data, the logger should work closely with the driller and be alert for changes in materials and operations while drilling is being performed. The logger is generally responsible for recording the following basic information on the field boring log:

REGION 3	**SUBSURFACE EXPLORATION LOG**	HOLE BAF-2
COUNTY Orange		LINE Baseline
PROJECT Interstate 0		STA. 92+00
DATE START 5/4/92	HAMMER FALL-CASING 18"	OFFSET 50' Lt
DATE FINISH 5/6/92	HAMMER FALL-SAMPLER 30"	SURF. ELEV. 996.2
CASING O.D. 2½" I.D. 2¼"	WEIGHT OF HAMMER-CASING 300 LBS.	
SAMPLER O.D. 2" I.D. 1-3/8"	WEIGHT OF HAMMER-SAMPLER 140 LBS.	
RIG TYPE Acker B-40		
CORE BARREL Double Tube		

TIME	4:00 pm	8:00 am	
DATE	5/4/92	5/6/92	
DEPTH TO WATER	10'	10'	

DEPTH BELOW SURFACE	BLOWS ON CASING	SAMPLE NO.	BLOWS ON SAMPLER 0/0.5	BLOWS ON SAMPLER 0.5/1.0	BLOWS ON SAMPLER 1.0/1.5	BLOWS ON SAMPLER 1.5/2.0	RECOVER (in)	DESCRIPTION OF SOIL AND ROCK	MOIST CONT %
0	6	J1	1	2	2		5		
	19								
	27	J2	1	3	3		10		
	35								
	21	J3	2	5	6		17		
	30	J4	7	9	12		15		
	22								
	25	J5	8	7	15		18	GR. FINE TO COARSE SAND	
	24							MOIST NON PLASTIC	
10	28	J6	14	20	20		16		
	27	J7	15	18	19				
	36								
	34	J8	13	16	17		17		
	37								
	39	J8	15	10	3		18	15'	
	31								
	40								
	46								
	46								
20	45								
	41	J10	2	4	4		15		
	42								
	56								
	52								
	58							GR. SILTY CLAY	
	50	J11	2	3	3		14	MOIST - PLASTIC	
	56								
	52								
	49								
30	58								
	52	J12	1	2	3		18		
	56								
	61								
	63								
	65								

THE SUBSURFACE INFORMATION SHOWN HERE WAS OBTAINED FOR STATE DESIGN AND ESTIMATE PURPOSES. IT IS MADE AVAILABLE TO AUTHORIZED USERS ONLY THAT THEY MAY HAVE ACCESS TO THE SAME INFORMATION AVAILABLE TO THE STATE. IT IS PRESENTED IN GOOD FAITH, BUT IS NOT INTENDED AS A SUBSTITUTE FOR INVESTIGATION, INTERPRETATION OR JUDGMENT OF SUCH AUTHORIZED USERS.

CONTRACTOR ACME Drilling, Inc. SM ___________

DRILL RIG OPERATOR Klinedinst
SOIL & ROCK DESCRIP. Chassie
REGIONAL SOILS ENGR. Cheney
SHEET 1 OF 2
STRUCTURE NAME/NO. Apple Freeway #2

HOLE BAF-2

CREATED BY gINT SOFTWARE www.gintsoftware.com

Figure 3-25a. Example subsurface exploration log (0 – 35 ft depth).

REGION 3
COUNTY Orange
PROJECT Interstate 0
DATE START 5/4/92
DATE FINISH 5/6/92
CASING O.D. 2½" I.D. 2¼"
SAMPLER O.D. 2" I.D. 1-3/8"
RIG TYPE Acker B-40
CORE BARREL Double Tube

SUBSURFACE EXPLORATION LOG

HAMMER FALL-CASING 18"
HAMMER FALL-SAMPLER 30"
WEIGHT OF HAMMER-CASING 300 LBS.
WEIGHT OF HAMMER-SAMPLER 140 LBS.

HOLE BAF-2
LINE Baseline
STA. 92+00
OFFSET 50' Lt
SURF. ELEV. 996.2

TIME	4:00 pm	8:00 am	
DATE	5/4/92	5/6/92	
DEPTH TO WATER	10'	10'	

DEPTH BELOW SURFACE	BLOWS ON CASING	SAMPLE NO.	BLOWS ON SAMPLER 0/0.5	0.5/1.0	1.0/1.5	1.5/2.0	RECOVER (in)	DESCRIPTION OF SOIL AND ROCK	MOIST CONT %
	62	J13	1	2	4		16	GR. SILTY CLAY MOIST - PLASTIC	
	63								
	72								
	71							39'	
40	89								
	75		30	20	35				
	81								
	104								
	120							GR SANDY GRAVEL MOIST - NON PLASTIC (Cored Boulder 42.5' to 45' recovered 12"-7 pieces: used boulder buster)	
	140								
	110								
	136								
	166								
	191								
50	482							TOP OF ROCK 50'	
								HARD UNWEATHERED BASALT Run 1 50 - 55' RQD = 80% 55'	
								HARD UNWEATHERED BASALT Run 2 55 - 60' RQD = 75% 60'	
60								END OF BORING 60'	
70									

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CONTRACTOR ACME Drilling, Inc. SM

DRILL RIG OPERATOR Klinedinst
SOIL & ROCK DESCRIP. Chassie
REGIONAL SOILS ENGR. Cheney
SHEET 2 OF 2
STRUCTURE NAME/NO. Apple Freeway #2
HOLE BAF-2

Figure 3-25b. Example subsurface exploration log (35 - 60 ft depth).

1. General description of each soil and rock stratum, and the depth to the top and bottom of each stratum. As noted before, the log demonstrated in Figure 3-25 is intended to be a field log. On the final log, the description of the soil should be much more detailed and follow a specified soil classification system. Soils in the geotechnical engineering community are most often classified according to the Unified Soil Classification System (USCS). For example, the soils between depths 15 ft to 39 ft have been simply described in the field as "GR. SILTY CLAY, MOIST-PLASTIC." The full classification as per USCS on the final log may read as follows (the detailed description and classification of soils and the USCS are discussed in Chapter 4):

   Soft, wet, gray, high plasticity CLAY, with Silt; Fat CLAY (CH); (Alluvium)

2. The depth to groundwater at the time it is first encountered and afterwards at the end of each day, at completion of boring, and, if possible, at least 24 hours after completion of the boring.

3. The depth at which each sample is taken, the type of sample taken, its number, and any loss of samples taken during extraction from the hole.

4. The depths at which field tests are made and the results of the test.

5. Information generally required by the log format, such as:
   - Boring number and location.
   - Date of start and finish of the hole.
   - Name of driller (and of logger, if applicable).
   - Elevation at top of hole.
   - Depth of hole and reason for termination.
   - Diameter of any casing used.
   - Size of hammer and free fall used on casing (if driven).
   - Blows per foot to advance casing (if driven).
   - Description and size of sampler.
   - Size of drive hammer and free fall used on sampler in dynamic field tests.
   - Blow count for each 6 in (150 mm) to drive sampler. (Sampler should be driven three 6 in (150 mm) increments or to a total of 100 blows).
   - Type of drilling rig used.

- Type and size of core barrel used.
- Length of time to drill each core run or foot of core run.
- Length of each core run and amount of core per run.
- Recovery of sample in inches and RQD of rock core.
- Project identification.

6. Notes regarding any other pertinent information and remarks on miscellaneous conditions encountered, such as:
   - Depth of observed groundwater, elapsed time from completion of drilling, conditions under which observations were made, and comparison with the elevation noted during reconnaissance (if any).
   - Artesian water pressure.
   - Obstructions encountered.
   - Difficulties in drilling (caving, coring boulders, surging or rise of sands in casing, caverns, etc.).
   - Loss of circulating water and addition of extra drilling water.
   - Drilling mud and casing as needed and why.
   - Odor of recovered sample.
   - Sampler plugged.
   - Poor recovery.

7. Any other information the collection of which may be required by agency policy (e.g., names and associations of visitors to the site, etc.).

3.9 CONE PENETRATION TESTING (CPT)

The history of field cone penetrometers began with a design by the Netherlands Department of Public Works in 1930. This "Dutch" cone penetrometer was a mechanical operation using a manometer to read loads. Paired sets of inner and outer rods are pushed into the ground in 8 in (200 mm) intervals. In 1948, electric cones permitted continuous measurements to be taken downhole. In 1965, the addition of sleeve friction measurement devices allowed an indirect means for identifying soil types. Later, in 1974, the electric cone was combined with a piezoprobe to form the first piezocone penetrometer. Most recently, additional sensors have been added to form specialized devices such as the resistivity cone, acoustic cone, seismic cone, vibrocone, cone pressuremeter, and lateral stress cone.

The cone penetration test (CPT) was first introduced in the U.S. in 1965. Since that time, the CPT has developed into one of the most popular in-situ testing methods because it is fast, economical, and provides continuous profiling of the geostratigraphy and allows for continuous in-situ evaluation of soil properties. Depending upon equipment capability as well as soil conditions, 330 to 1150 ft (100 to 350 m) of penetration testing may be completed in one day.

As shown in Figure 3-26, the CPT involves the hydraulic push of an instrumented steel probe into the soil at a constant rate to obtain continuous vertical profiles of stresses and/or other measurements. No borehole, cuttings, or spoil are produced by this test. Testing is conducted in accordance with ASTM D 5778.

The CPT can be used in very soft clays to dense sands. It is not suitable for use in highly indurated or cemented soils or in soils containing significant amounts of gravel and boulders. The advantages and disadvantages of the CPT are listed in Table 3-11.

Table 3-11
Advantages and disadvantages of the Cone Penetration Test (CPT) (FHWA, 2002b)

Advantages	Disadvantages
• Fast and continuous profiling • Economical and productive • Results not operator-dependent • Strong theoretical basis in interpretation • Particularly suitable for soft soils	• High capital investment • Requires skilled operator to run • Electronic drift, noise, and calibration • No soil samples are obtained • Unsuitable for gravel/boulder deposits*
*Note: Except where special rigs are provided and/or additional drilling support is available.	

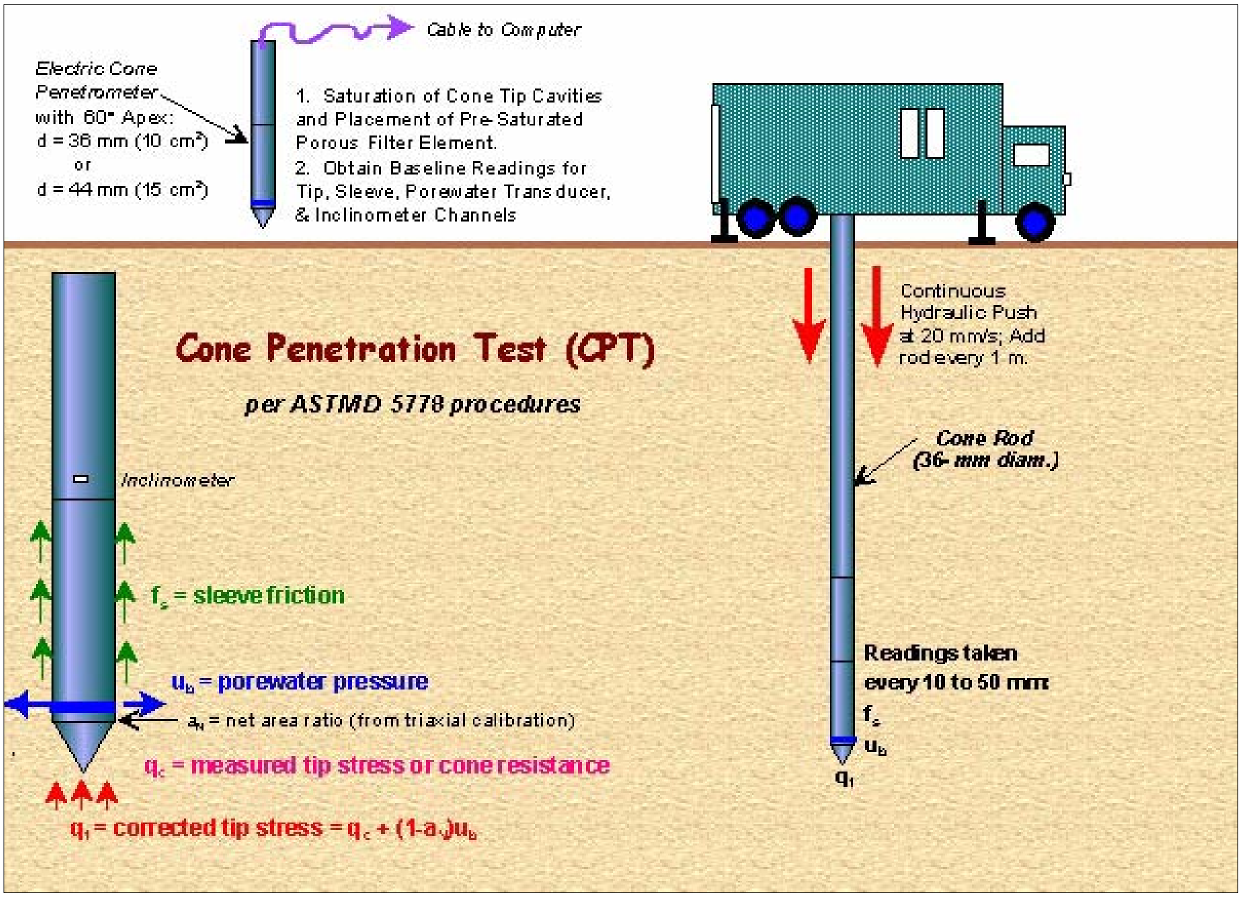


Figure 3-26. Procedures and components of the Cone Penetration Test (FHWA, 2002b).

Although the test provides continuous logging of the in-situ response of the soil, which can lead to more accurate and reliable analyses, no soil samples are available for laboratory testing. For that reason the CPT provides an excellent complement to the more conventional soil test boring with SPT measurements and subsequent laboratory testing on retrieved samples.

3.9.1 Equipment Description and Operation

Electronic cones are now the dominant cone type used in cone penetration testing. Therefore, mechanical cones are not discussed in this document. Electronic cones may be further divided into three primary types: (a) the standard friction cone (CPT), (b) the piezo-cone (PCPT or more commonly CPTu), and (c) the seismic cone piezo-cone (SCPTu). Each of these cones is briefly described here. To assist in following the brief descriptions, the standard terminology regarding the cone penetrometer is shown in Figure 3-27.

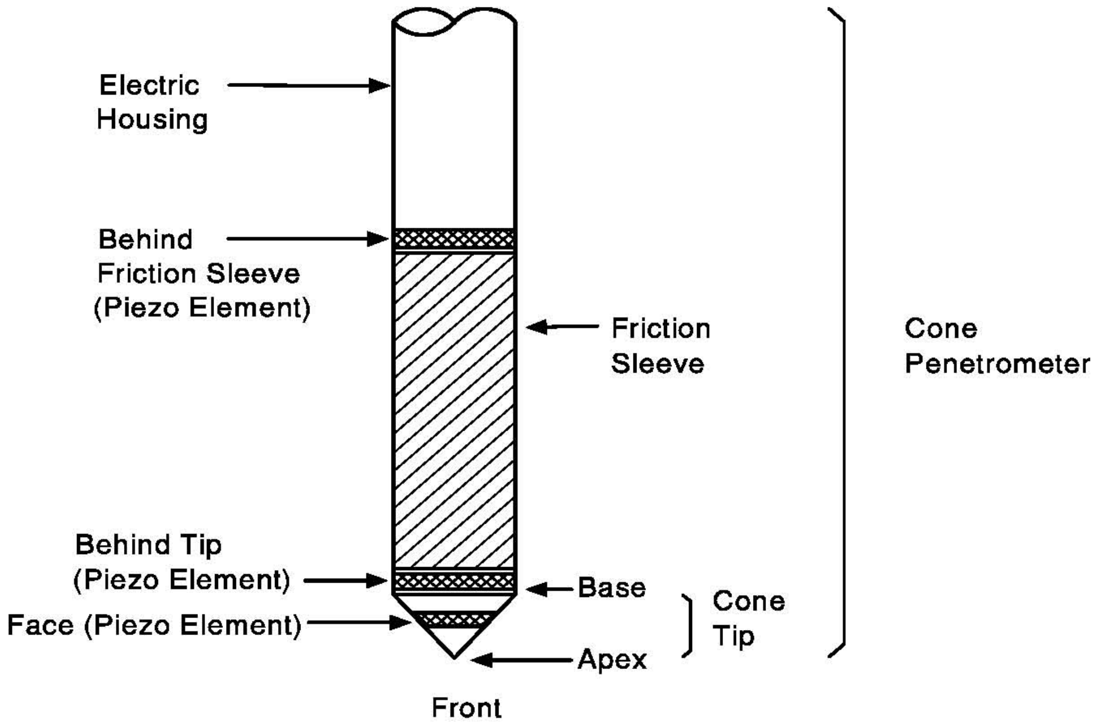


Figure 3-27. Cone penetrometer terminology (from Robertson and Campanella, 1989).

3.9.2 The Standard Cone Penetration Test (CPT)

The equipment necessary for performing a standard CPT includes a penetrometer, cone rod or drill rod, an electrical cable, a data acquisition system, and a hydraulic actuator attached to equipment that has sufficient reaction mass to advance the penetrometer. The equipment that provides the reaction mass can be a conventional drilling rig, however, a dedicated CPT truck commonly weighing 20 to 25 tons (200 to 250 kN) is more commonly used.

A standard cone penetrometer is a 1.4 in (35.7 mm) diameter cylindrical probe with a 60^o conical apex at the tip. The tip has a projected area of 1.6 in^2 (10 cm^2). The surface area of the sleeve above the cone is 23.3 in^2 (150 cm^2). More robust penetrometers are available with a 1.7 in (44 mm) diameter body, a 2.3 in^2 (15 cm^2) projected tip area, and a 31 to 35 in^2 (200 to 225 cm^2) sleeve surface area. A penetrometer having a projected cone area of 2.3 in^2 (15 cm^2) will generally provide the same response as one having a projected cone area of 1.6 in^2 (10 cm^2). The "size" of a cone is defined by its projected tip area, e.g. a 1.6 in^2 (10 cm^2) cone or a 2.3 in^2 (15 cm^2) cone. Figure 3-28 shows a number of different cone penetrometers and piezocones.

Figure 3-28. Cone and piezocone penetrometers (note the quarter for scale) (FHWA, 2002b).

A section of standard cone rod is typically 3.3 ft (1 m) in length with a 1.4 in (35.7 mm) outer diameter and a 0.9 in (22 mm) inner diameter. Alternatively, the penetrometer can be pushed with standard AW drill rod (1¾ in (44.4 mm) OD; 1¼ in (31.8mm) ID) or EW drill rod (1⅜ in (34.9 mm) OD; 15/16 in (23.8 mm) ID).

The cone cable runs through the hollow cone/drill rods and attaches to an electronic data acquisition system at the ground surface. The data acquisition system generally consists of an analog signal conditioner, an analog to digital (A-D) converter, and a computer processor. Current data acquisition systems are attached to one or two computer monitors so the operator and engineer can observe data recorded during the sounding in real time. Real time monitoring allows for decisions to be made in the field with respect to the sounding. This is helpful if auxiliary tests, such as a pore pressure dissipation test, are to be performed in certain soil layers, or if the test is to be terminated once a certain layer is encountered. Printers can be attached to the computer processor to obtain a real-time printout of the data. Printed data are a good backup in case an unforeseen incident causes the computer to crash resulting in the loss of the electronically stored data. Data are typically recorded every ¾ to 2 in (20 to 50 mm) of vertical penetration.

The test procedure for the standard cone penetration test and the nature of the data acquired during the test are described in Sections 3.9.5 and 3.9.6, respectively.

3.9.3 The Piezo-cone Penetration Test (CPTu)

The piezo-cone (CPTu) is essentially the same as the standard electronic friction cone except that it includes porous filter piezo-elements that may be located at the cone tip, on the cone face, behind the cone tip, or behind the friction sleeve. These porous filter elements are used to measure pore water pressure during penetration. Saturation of the porous element and cavity is essential to obtain reliable pore water pressure measurements.

3.9.4 The Seismic Piezocone Penetration Test (SCPTu)

For the seismic piezocone test (SCPTu), a geophone is located approximately 1.6 feet (500 millimeters) uphole from the cone tip. The geophone detects shear waves generated at the ground surface at intervals of approximately 3 or 5 ft (1 or 1.5 m), corresponding to successive rod additions. If necessary, adjustments should be made if AW or EW rods are used to advance the cone since they typically come in longer lengths.

3.9.5 Test Procedures

The test procedure for the CPT consists of hydraulically pushing the cone at a rate of 0.8 in/s (20 mm/s) in accordance with ASTM D 5778 by using either a standard drill rig or a specialized cone truck as the reaction mass (see Figure 3-29). The advance of the probe requires the successive addition of rods at approximately 3 or 5 ft (1 or 1.5 m) intervals. Readings of tip resistance (q_t), sleeve friction (f_s), inclination (i), and pore water pressure (u_m) are taken at least every 2 in (50 mm) (i.e., at approximately 2.5-sec intervals). For the seismic cone test, shear wave arrival times (t_s) are typically recorded at rod breaks corresponding to 3 or 5 ft (1 or 1.5 m) intervals.

Figure 3-29. Cone penetration testing from cone truck.

3.9.6 CPT Profiles

The results of the individual channels of a piezocone penetration test are plotted with depth, as illustrated in a typical plot shown in Figure 3-30. Since soil samples are not obtained with the CPT, an indirect assessment of Soil Behavioral Type (SBT) is inferred by an examination of the readings. The numbers can be processed for use in empirical chart classification systems, or the raw readings can be easily interpreted for soil strata changes. A simplified soil classification chart for a standard electric friction cone is presented in Figure 3-31. The sleeve friction, often expressed in terms of a friction ratio $R_f = f_s/q_t$, also is a general indicator of soil type. For example, in sands, usually $0.5\% < R_f < 1.5\ \%$; and in clays, normally $3\% < R_f < 10\%$. In the lower half of the Figure 3-31, the center column shows an approximate relationship between the SPT N-value and the cone tip resistance, q_c. The SPT N-value obtained from this relationship should be considered to be equivalent to $N1_{60}$.

3.9.7 CPT Profile Interpretation

The CPT sounding shown in Figure 3-30 was taken in the immediate vicinity of the boring recorded in the boring log shown in Figures 3-25. These two logs permit an interesting comparison to be made of the SPT and CPT procedures. In the sounding shown in Figure 3-30, a clayey and sandy stratum (clay, clayey silt, silt, silty sand and sandy silt) occurs from the ground surface to a depth of 10 ft (3 m). These strata are underlain by a thick layer of sand and sandy silt to depth of approximately 20 ft (6 m), which in turn is underlain by a clay layer extending down to a depth of approximately 45 ft (14 m). Finally, a dense gravelly sand layer is encountered, which the cone penetrometer could not penetrate. The SPT tests could however be performed in this dense layer since it was possible to drill into this layer.

Figure 3-30 is a good example that demonstrates the advantage of continuous sounding compared to the samples obtained at discrete intervals using SPT procedures. For example, depending on the sampling interval in the SPT test, the silt layer within the clay layer may not have been undetected. Even if it had been detected, it would not have been possible to estimate its thickness accurately between the locations of the SPT samples because the SPT samples are commonly retrieved at 5 ft (1.5 m) intervals. The implications of this shortcoming can be significant in design. For example, silt consolidates faster than clay. If the designer is not aware of the silt layer, then he/she might design a wick drain surcharge system that, based on a clay layer 25 ft (7.5 m) thick, will take longer to consolidate than what might actually be the case. In fact, the CPT profile in Figure 3-30 shows that only the 15 ft (4.5 m) portion of the clay layer below the silt layer has excess pore pressures, which suggests that it will be the primary source of consolidation settlements.

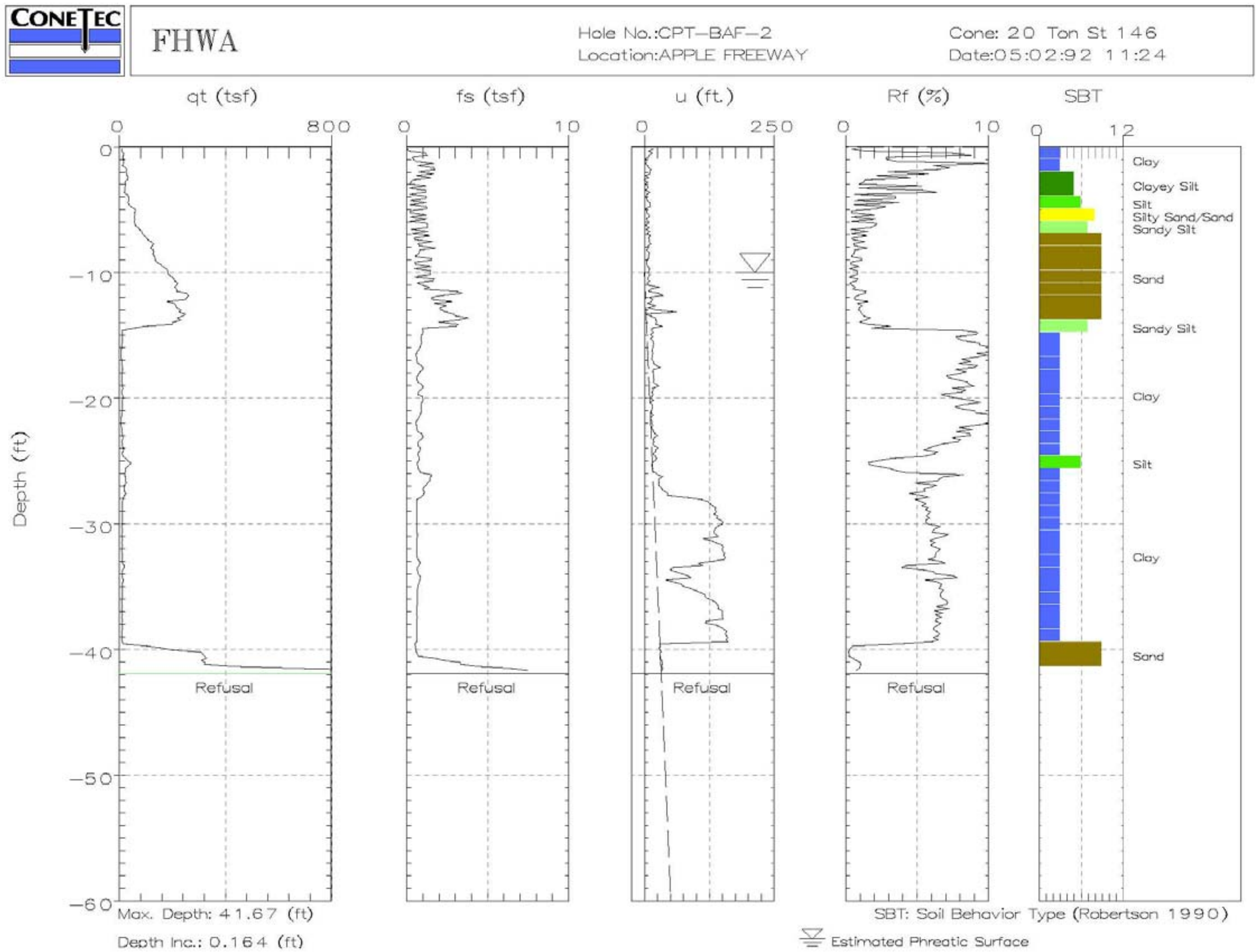
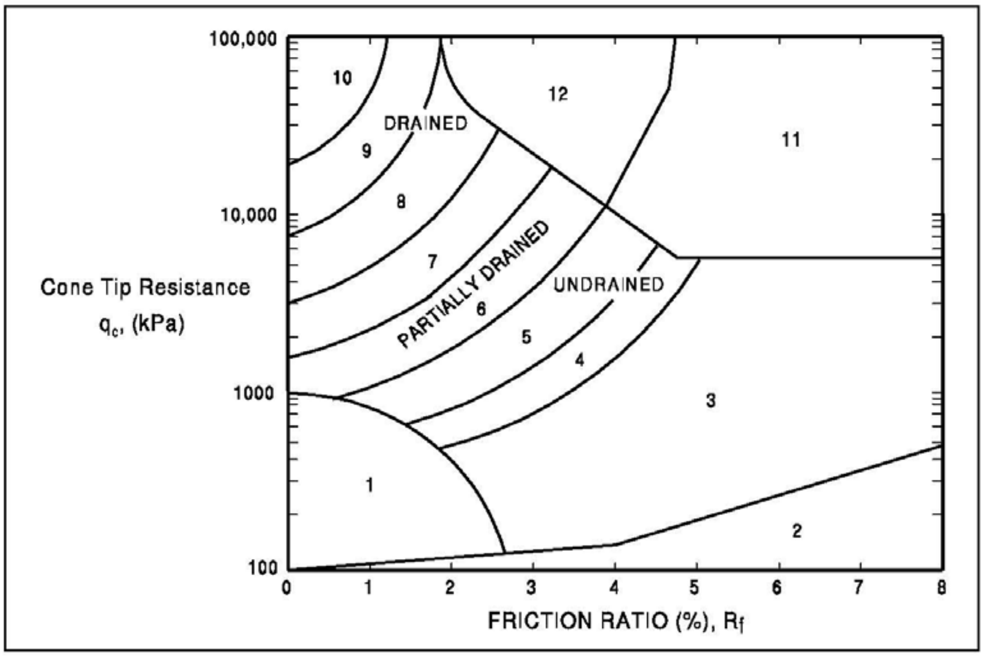


Figure 3-30. Piezo-cone results for Apple Freeway Bridge.

It is important for the reader to understand that CPT procedures do not allow retrieval of physical samples that can be tested in the laboratory to characterize various phenomena such as consolidation and shear strength. Thus, it is most beneficial to use the CPT with another method, such as the boring technique used in the SPT, that allows the retrieval of physical samples for laboratory testing. Performing the CPT before sampling in borings will permit identification of the specific depths where disturbed and undisturbed physical samples should be obtained for laboratory tests.



Zone	q_c/N	Soil Behavior Type
1)	2	sensitive fine grained
2)	1	organic material
3)	1	clay
4)	1.5	silty clay to clay
5)	2	clayey silt to silty clay
6)	2.5	sandy silt to clayey silt
7)	3	silty sand to sandy silt
8)	4	sand to silty sand
9)	5	sand
10)	6	gravelly sand to sand
11)	1	very stiff fine grained
12)	2	sand to clayey sand

Figure 3-31. A commonly used simplified soil classification chart for standard electronic friction cone (after Robertson, *et al.*, 1986).

3.10 DILATOMETER TEST (DMT)

The dilatometer is an in-situ testing device that was developed in Italy in the early 1970s and first introduced in the U.S. in 1979. Like the cone penetrometer, the dilatometer is generally hydraulically pushed into the ground although it may also be driven. When the dilatometer can be pushed into the ground with tests conducted at 8 in (200 mm) increments, 100 to 130 ft (30 to 40 m) of soundings may be completed in a day. The primary utilization of the dilatometer test (DMT) in pile foundation design is the delineation of subsurface stratigraphy and interpreted soil properties. However, it would appear that the CPT/CPTu is generally better suited to this task than the DMT. The DMT may be a potentially useful test for the design of piles subjected to lateral loads. Design methods in this area show promise, but are still in the development stage. For design of axially loaded piles, the DMT has limited direct value. A picture of the DMT equipment is presented in Figure 3-32.

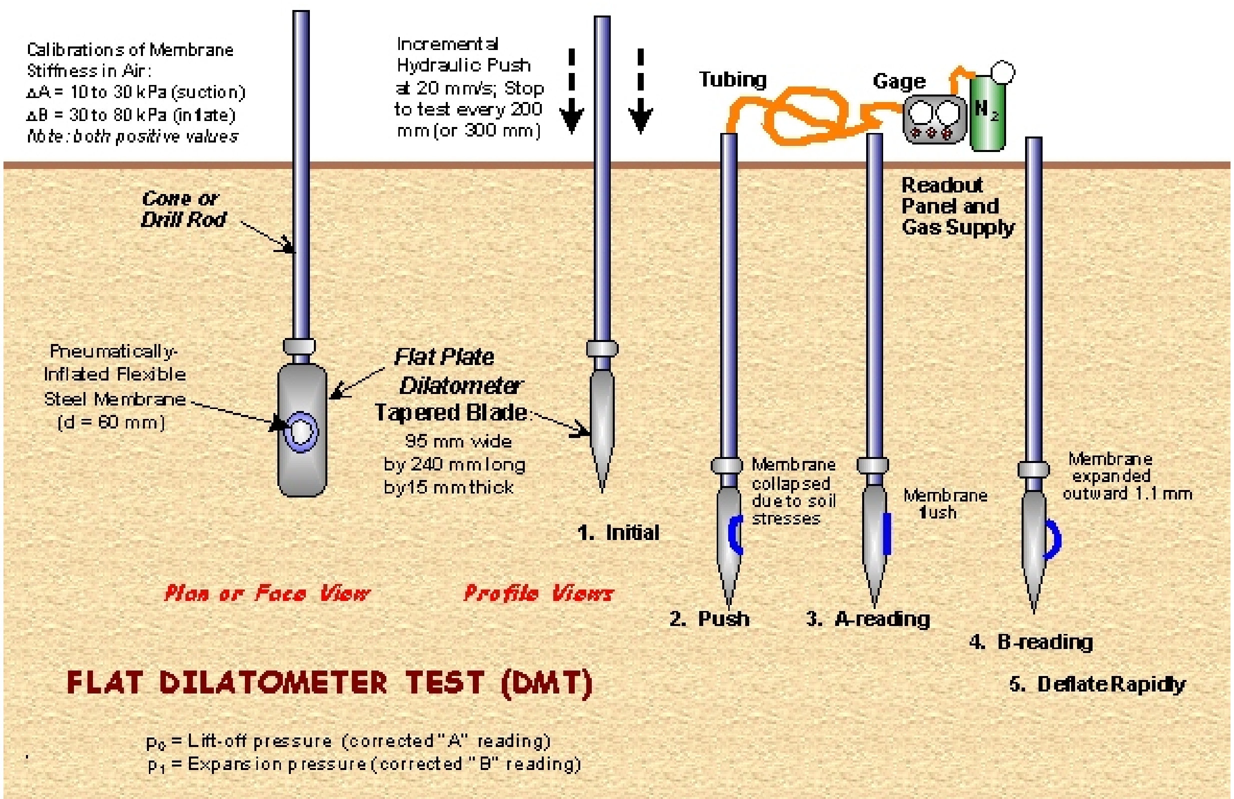


Figure 3-32. Dilatometer test equipment and procedure (FHWA 2002b).

3.11 PRESSUREMETER TEST (PMT)

The pressuremeter is an in-situ device used to evaluate soil and rock properties. The pressuremeter has been used in Europe for many years and was introduced into the U.S. in the mid 1970s. The pressuremeter imparts lateral pressures to the soil, and the soil shear strength and compressibility are determined by interpretation of a pressure-volume relationship. The pressuremeter test (PMT) allows a determination of the load-deformation characteristics of soil in axisymmetric conditions. Deposits such as soft clays, fissured clays, sands, gravels and soft rock can be tested with pressuremeters. A pressuremeter test produces information on the elastic modulus of the soil as well as the at rest horizontal earth pressure, the creep pressure, and the soil limit pressure. A schematic of the pressuremeter test is presented in Figure 3-33.

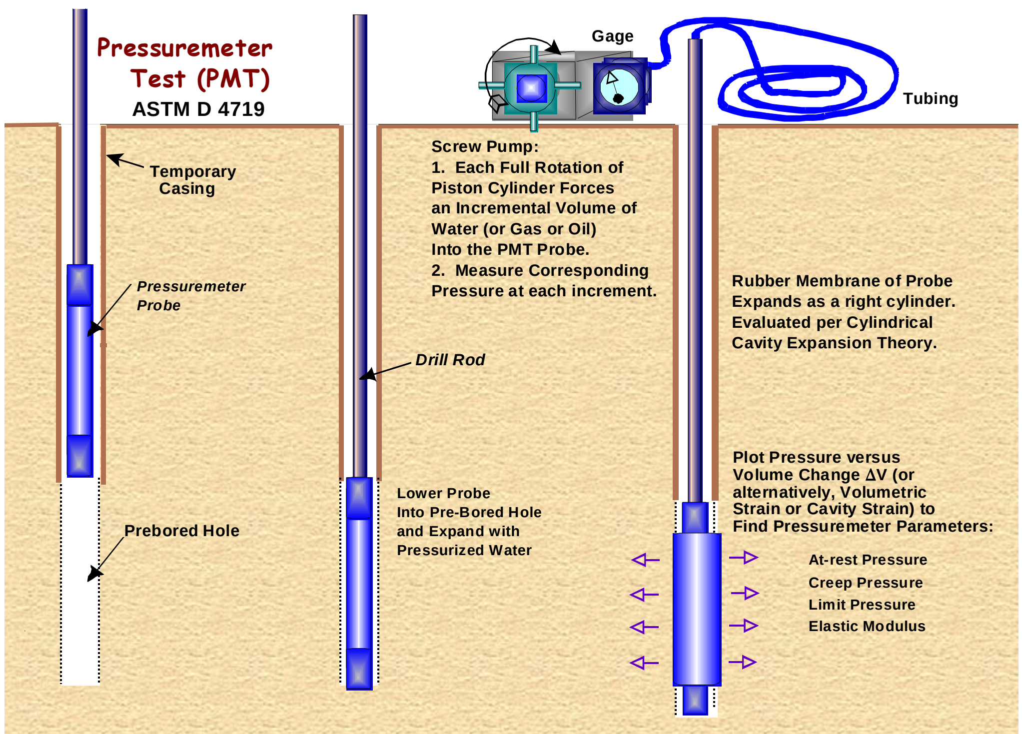


Figure 3-33. Pressuremeter test schematic (FHWA, 2002b).

The utilization of test results is based upon semi-empirical correlations from a large number of tests and observations on actual structures. For piles subjected to lateral loads, the pressuremeter test is a useful design tool and can be used for determination of p-y curves. For design of vertically loaded piles, the pressuremeter test has limited value. Pile design procedures using pressuremeter data have been developed and may be found in FHWA (1989a). Details on test procedures may be found in ASTM D 4719.

3.12 VANE SHEAR TEST (VST)

The vane shear test is an in-situ test for determining the undrained shear strength of soft to medium clays. Figure 3-34 is a schematic drawing of the essential components and test procedure. The test consists of forcing a four-bladed vane into undisturbed soil and rotating it until the soil shears. Two shear strengths are usually recorded, the peak shearing strength and the remolded shearing strength. These measurements are used to determine the **sensitivity of clay**, which is defined as the ratio of the peak undrained shearing strength to the remolded undrained shearing strength. Sensitivity, S_t, allows analysis of the soil resistance to be overcome during pile driving in clays which is useful for pile driveability analyses. It is necessary to measure skin friction along the steel connector rods which must be subtracted to determine the actual shear strength. The VST generally provides the most accurate undrained shear strength values for clays with undrained shear strengths less than 1 ksf (50 kPa). The test procedure has been standardized in AASHTO T 223-74 and ASTM D 2573.

It should be noted that the sensitivity of a clay determined from a vane shear test provides insight into the set-up potential of the clay deposit. However, the sensitivity value is a qualitative and not a quantitative indicator of soil set-up. Classification of clayey soils based on sensitivity values is presented in Table 3-12.

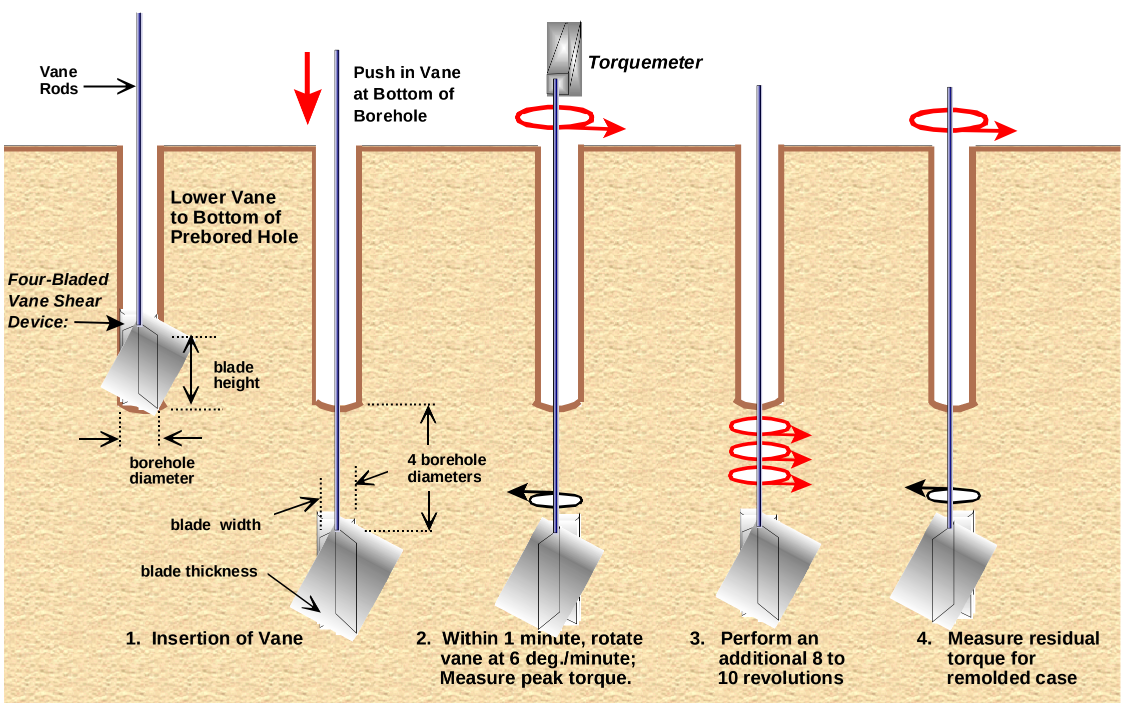


Figure 3-34. Vane shear test equipment and procedure (after FHWA, 2002b).

Table 3-12
Classification of Sensitivity Values (Mitchell, 1976)

Classification	**Sensitivity, S_t**
Insensitive	~ 1.0
Slightly sensitive clays	1 – 2
Medium sensitive clays	2 – 4
Very sensitive clays	4 – 8
Slightly quick clays	8 – 16
Medium quick clays	16 – 32
Very quick clays	32 – 64
Extra quick clays	> 64

3.13 GROUNDWATER MEASUREMENTS

Observations of the groundwater level and pore water pressure are an important part of all geotechnical explorations. The identification of groundwater conditions should receive the same level of care given to soil descriptions and samples. Measurements of water entry during drilling and measurements of the groundwater level at least once following drilling should be considered a minimum effort to obtain water level data, unless alternate methods, such as installation of observation wells, are defined by the geotechnical specialist. Detailed information regarding groundwater observations can be obtained from ASTM D 4750 and ASTM D 5092.

3.13.1 Information on Existing Wells

Many states require the drillers of water wells to file logs of the wells they have drilled. These are good sources of information of the materials encountered and water levels recorded during well installation. The well owners, both public and private, may have records of the water levels after installation, which may provide extensive information on fluctuations of the water level. This information may be available at state agencies regulating the drilling and installation of water wells, such as the Department of Transportation, the Department of Natural Resources, State Geologist, Hydrology Department, Department of Environmental Quality, and Division of Water Resources.

3.13.2 Open Borings

The water level in open borings should be measured after any prolonged interruption in drilling, at the completion of each boring, and at least 12 hours (preferably 24 hours) after completion of drilling. Additional water level measurements should be made at the completion of the field exploration and at other times designated by the engineer. The date and time of each observation should be recorded.

If the borehole has caved, the depth to the collapsed region should be recorded on the boring record as the collapse may have been caused by groundwater conditions. The elevations of the caved depths of certain borings may be consistent with groundwater table elevations at the site. This consistency may become apparent once the subsurface profile is constructed (see Chapters 4 and 11).

Drilling mud obscures observations of the groundwater level owing to filter cake action and the greater specific gravity of the drilling mud compared to that of the water. If drilling fluids are used to advance borings, the drill crew should be instructed to bail the hole prior to making groundwater observations.

3.13.3 Observation Wells

The observation well, also referred to as a piezometer, is the fundamental means for measuring water head in an aquifer and for evaluating the performance of dewatering systems. In theory, a "piezometer" measures the water pressure in a confined aquifer or at a specific horizon of the geologic profile, while an "observation well" measures the level of a water table in an aquifer (Powers, 1992). In practice, however, the two terms are often used interchangeably to describe any device for determining static water head.

The term "observation well" is applied to any well or drilled hole used for the purpose of long-term studies of groundwater levels and pressures. Existing wells and bore holes in which casing is left in place are often used to observe groundwater levels. These, however, are not considered to be as satisfactory as wells constructed specifically for the purpose of measuring groundwater conditions. The latter may consist of a standpipe installed in a previously drilled exploratory hole or a hole drilled solely for use as an observation well.

Details of typical observation well installations are shown in Figure 3-35. The simplest type of observation well is formed by a small-diameter polyvinyl chloride (PVC) pipe set in an open hole. The bottom of the pipe is slotted and capped, and the annular space around the slotted pipe is backfilled with clean sand. The area above the sand is sealed with bentonite, and the remaining annulus is filled with grout, concrete, or soil cuttings. A surface seal, which is sloped away from the pipe, is commonly formed with concrete in order to prevent the entrance of surface water. The top of the pipe should also be capped to prevent the entrance of foreign material; there should be a small vent hole in the top of the removable cap. In some localities, regulatory agencies may stipulate the manner for installation and closure of observation wells.

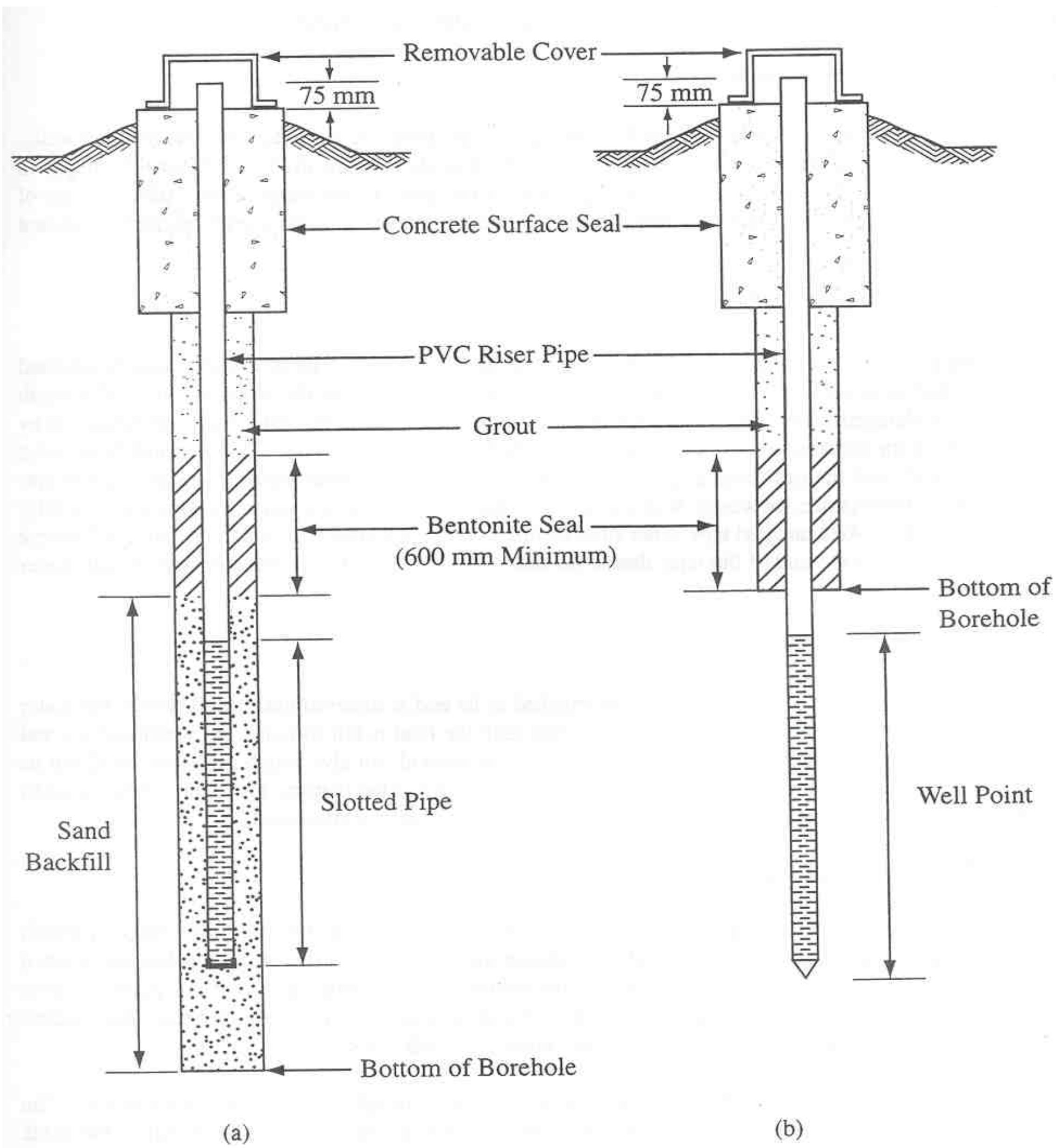


Figure 3-35. Representative details of observation well installations. (a) Drilled-in-place stand-pipe piezometer, (b) Driven well point (FHWA, 1997).

Driven or pushed-in well points are another common type of observation well for use in granular soil formations and very soft clay (Figure 3-35b). The well is formed by a stainless steel or brass well point threaded to a galvanized steel pipe. In granular soils, an open boring or rotary wash boring is advanced to a point several inches above the measurement depth and the well point is driven to the desired depth. A seal is commonly required in the boring above the well point with a surface seal at the ground surface. Note that observation wells may require development (see ASTM D 5092) to minimize the effects of installation, drilling fluids, etc. Minimum pipe diameters should allow introduction of a bailer or other pumping apparatus to remove fine-grained materials in the well to improve the response time.

Local jurisdictions may impose specific requirements on "permanent" observation wells, including closure and reporting of the location and construction that must be considered in the planning and installation. Licensed drillers and special fees may be required.

Piezometers are available in a number of designs. Commonly used piezometers are of the pneumatic and the vibrating wire type. Interested readers are directed to Dunnicliff (1988) and FHWA (1997) for a detailed discussion of the various types of piezometers.

3.13.4 Water Level Measurements

A number of devices have been developed for sensing or measuring the water level in observation wells. Following is a brief presentation of three methods that are commonly used to measure the depth to groundwater. In general, common practice is to measure the depth to the water surface by using the top of the casing as a reference, with the reference point at a common orientation (often north) marked or notched on the well casing.

3.13.4.1 Chalked Tape

In this method a short section at the lower end of a metal tape is chalked. The tape with a weight attached to its end is then lowered until the chalked section has passed slightly below the water surface. The depth to the water is determined by subtracting the depth of penetration of the line into water, as measured by the water line in the chalked section, from the total depth from the top of casing. This is probably the most accurate method, and the accuracy is useful in pump tests where very small drawdowns are significant. The method is cumbersome, however, when a series of rapid readings is taken since the tape must be fully removed each time. An enameled tape is not suitable unless it is roughened with sandpaper so it will accept chalk. The weight on the end of the tape should be small in volume so it does not displace enough water to create an error in the water level.

3.13.4.2 Tape with a Float

In this method, a tape with a flat-bottomed float attached to its end is lowered until the float hits the water surface and the tape goes slack. The tape is then lifted until the float is felt to touch the water surface and the tape is just taut; the depth to the water surface is then measured. With practice this method can give rough measurements, but its accuracy is poor. A refinement is to mount a heavy whistle, open at the bottom, on a tape. When it sinks in the water, the whistle will give an audible beep as the air within it is displaced.

3.13.4.3 Electric Water-Level Indicator

This battery operated indicator consists of a weighted electric probe attached to the lower end of a length of electrical cable that is marked at intervals to indicate the depth. When the probe reaches the water a circuit is completed. This condition is registered by a meter mounted on the cable reel. Various manufacturers produce the instrument, utilizing a neon lamp, a horn, or an ammeter as the signaling device. The electric indicator has the advantage that it may be used in extremely small holes.

The instrument should be ruggedly built, since some degree of rough handling can be expected. The distance markings must be securely fastened to the cable. Some models are available in which the cable itself is manufactured as a measuring tape. The sensing probe should be shielded to prevent shorting out against metal risers. When the water is highly conductive, erratic readings can develop in the moist air above the actual water level. Sometimes careful attention to the intensity of the neon lamp or the pitch of the horn will enable the reader to distinguish the true level. A sensitivity adjustment on the instrument can be useful. If oil or iron sludge has accumulated in the observation well, the electric probe will give unreliable readings.

3.13.4.4 Data Loggers

When timed and frequent water level measurements are required, as for a pump test or slug test, data loggers are useful. Data loggers are in the form of an electric transducer near the bottom of the well that senses changes in water level as changes in pressure. A data acquisition system is used to acquire and store the readings. A data logger can eliminate the need for onsite technicians on night shifts during extended field permeability testing. A further significant saving is in the technician's time back in the office. The preferred models of the data logger not only record the water level readings but permit the data to be downloaded into a personal computer and, with appropriate software, to be quickly reduced and plotted. These devices are also extremely useful for cases where measurement of artesian pressures is required or where data for tidal corrections during field permeability tests are necessary.

3.14 GUIDELINES FOR MINIMUM SUBSURFACE EXPLORATION

In regard to the scope of the subsurface exploration program for a structure or a geotechnical feature, one must consider the small cost of the borings in relation to the foundation costs. The knowledge gained from a thorough subsurface exploration program will allow the geotechnical specialist to evaluate various candidate foundation schemes and provide recommendations for those that can be built most efficiently and economically on the project site. Without an adequate subsurface exploration program, the result is generally an extremely conservative foundation recommendation.

Planning a subsurface exploration program should include determining the location and depth of borings, test pits, or other procedures to be used and establishing the methods of soil sampling and testing to be employed. Usually, the extent of the work is estimated based on available geological studies, earlier explorations, or records of existing structures. The number of borings and their locations in a site area will depend on the proposed structure, design parameters, access issues, geologic constraints, and expected stratigraphy.

Although no rigid rules apply universally to geotechnical explorations, certain general principles are usually followed in practice. Recommended guidelines for the minimum number of exploration points and their spacing are provided in Table 3-13. This table was developed based on a number of FHWA documents. This table should be used only as a first step in estimating the minimum number of borings for a particular design, as actual boring spacings will be dependent upon the project type and geologic environment. In all cases, it is recommended that the depth of the exploration should be such that the depth of significant influence (DOSI) is explored. For a given configuration of loading, the DOSI may exceed the minimum guidelines in Table 3-13 in which case the depth of exploration should be increased accordingly. Some other general guidance in addition to that in Table 3-13 is as follows:

- In areas underlain by heterogeneous soil deposits and/or rock formations, it will probably be necessary to exceed the minimum guidelines presented in Table 3-13 to capture variations in soil and/or rock type and to assess consistency across the site.

Table 3-13
Guidelines for minimum number of exploration points and depth of exploration (modified after FHWA, 2002a)

Application	Minimum Number of Exploration Points and Location of Exploration Points	Minimum Depth of Exploration
Retaining walls	(1) A minimum of one exploration point for each retaining wall. (2) For retaining walls more than 100 ft (30 m) in length, exploration points spaced every 100 to 200 ft (30 to 60 m) with locations alternating from in front of the wall to behind the wall. (3) For anchored walls, additional exploration points in the anchorage zone spaced at 100 to 200 ft (30 to 60 m). (4) For soil-nail walls, additional exploration points at a distance of 1.0 to 1.5 times the height of the wall behind the wall spaced at 100 to 200 ft (30 to 60 m).	(1) Investigate to a depth below bottom of wall between 1 and 2 times the wall height or a minimum of 10 ft (3 m) into bedrock. (2) Exploration depth should be great enough to fully penetrate soft highly compressible soils (e.g. peat, organic silt, soft fine grained soils) into competent material of suitable bearing capacity (e.g., stiff to hard cohesive soil, compact dense cohesionless soil, or bedrock).
Embankment Foundations	(1) A minimum of one exploration point every 200 ft (60 m) (erratic conditions) to 400 ft (120 m) (uniform conditions) of embankment length along the centerline of the embankment. (2) At critical locations, (e.g., maximum embankment heights, maximum depths of soft strata) a minimum of three exploration points in the transverse direction to define the existing subsurface conditions for stability analyses. (3) For bridge approach embankments, at least one exploration point at abutment locations.	(1) Exploration depth should be, at a minimum, equal to twice the embankment height unless a hard stratum is encountered above this depth. (2) If soft strata are encountered extending to a depth greater than twice the embankment height, the exploration depth should be great enough to fully penetrate the soft strata into competent material (e.g., stiff to hard cohesive soil, compact to dense cohesionless soil, or bedrock).
Cut Slopes	(1) A minimum of one exploration point every 200 ft (60 m) (erratic conditions) to 400 ft (120 m) (uniform conditions) of slope length. (2) At critical locations (e.g., maximum cut depths, maximum depths of soft strata) a minimum of three exploration points in the transverse direction to define the existing subsurface conditions for stability analyses. (3) For cut slopes in rock, perform geologic mapping along the length of the cut slope.	(1) Exploration depth should be, at a minimum, 15 ft (4.5 m) below the minimum elevation of the cut unless a hard stratum is encountered below the minimum elevation of the cut. (2) Exploration depth should be great enough to fully penetrate through soft strata into competent material (e.g., stiff to hard cohesive soil, compact to dense cohesionless soil, or bedrock). (3) In locations where the base of cut is below ground-water level, increase depth of exploration as needed to determine the depth of underlying pervious strata.

Table 3-13 (Continued)
Guidelines for minimum number of exploration points and depth of exploration (after FHWA, 2002a)

Application	Minimum Number of Exploration Points and Location of Exploration Points	Minimum Depth of Exploration
Shallow Foundations	(1) For substructure (e.g., piers or abutments) widths less than or equal to 100 ft (30 m), a minimum of one exploration point per substructure. (2) For substructure widths greater than 100 ft (30 m), a minimum of two exploration points per substructure. (3) Additional exploration points should be provided if erratic subsurface conditions or sloping rock surfaces are encountered.	Depth of exploration should be: (1) great enough to fully penetrate unsuitable foundation soils (e.g., peat, organic silt, soft fine grained soils) into competent material of suitable bearing capacity (e.g. stiff to hard cohesive soil, compact to dense cohesionless soil or bedrock); and (2) **at least to a depth where stress increase due to estimated footing load is less than 10% of the applied stress at the base of the footing**; and (3) in terms of the width of the footing: at least 2 times for axisymmetric case and 4 times for strip footing (interpolate for intermediate cases); and (4) if bedrock is encountered before the depth required by item (2) above is achieved, exploration depth should be great enough to penetrate a minimum of 10 ft (3 m) into the bedrock, but rock exploration should be sufficient to characterize compressibility of infill material of near-horizontal to horizontal discontinuities.
Deep Foundations	(1) For substructure (e.g., bridge piers or abutments) widths less than or equal to 100 ft (30 m), a minimum of one exploration point per substructure. (2) For substructure widths greater than 100 ft (30 m), a minimum of two exploration points per substructure. (3) Additional exploration points should be provided if erratic subsurface conditions are encountered. (4) Due to large expense associated with construction of rock-socketed shafts, conditions should be confirmed at each shaft location.	(1) In soil, depth of exploration should extend below the anticipated pile or shaft tip elevation a minimum of 20 ft (6 m), or a minimum of two times the maximum pile group dimension, whichever is deeper. All borings should extend through unsuitable strata such as unconsolidated fill, peat, highly organic materials, soft fine-grained soils, and loose coarse-grained soils to reach hard or dense materials. (2) For piles bearing on rock, a minimum of 10 ft (3 m) of rock core shall be obtained at each exploration point location to verify that the boring has not terminated on a boulder. (3) For shafts supported on or extending into rock, a minimum of 10 ft (3 m) of rock core, or a length of rock core equal to at least three times the shaft diameter for isolated shafts or two times the maximum shaft group dimension, whichever is greater, shall be extended below the anticipated shaft tip elevation to determine the physical characteristics of rock within the zone of foundation influence.

- In the case of embankments, the guidance provided in Table 3-13 for the depth of exploration is the minimum recommended for any transportation facility, i.e., 2 times the height of the embankment. The minimum guidance may not be the same as the DOSI, which is a function of the geometry (crest width, height, configuration of side slopes, and the base width) of the embankment. For the same height of embankment, the DOSI increases as the base width of an embankment increases, and may vary from 4 to 6 times the height of the embankment. An example of this is shown in Figures 2-8 in Chapter 2, which shows that the DOSI can extend to depths much deeper than 2 times the height of the embankment. This may be particularly critical in cases where there are soft soils in the subsurface. Thus, for such situations, the geotechnical specialist should use tools such as the FoSSA program or published elastic solutions (Poulos and Davis, 1974) to determine the depth of exploration.

- For situations where large-diameter rock-socketed shafts will be used or where drilled shafts are being installed in karstic formations, it may be necessary to advance a boring at the location of each shaft.

- In a laterally homogeneous area, drilling or advancing a large number of borings may be redundant, since each sample tested would exhibit similar strength and compressibility properties.

- In all cases, it is necessary to understand how the design and construction of the geotechnical feature will affect the soil and/or rock mass in order to optimize the exploration.

During exploration, each exploration point (e.g., drill hole or CPT sounding) should be designated by a unique identification number to prevent duplication during subsequent explorations. For example, it is not unusual to find projects where borehole numbering was done by only single numbers so that the same designations were used during one or more subsequent explorations. A suggested method to avoid duplication is to designate that all bridge holes begin with the letter "B," followed by the initials of the highway or river being crossed and finally a sequential number. For example, the first boring for a structure on Apple Freeway would be designated DH-BAF-1, where the DH means a "drill hole" where SPTs were performed as opposed to CPT-BAF-1, where the CPT refers to the first CPT sounding performed for a bridge structure on Apple Freeway.

The geotechnical specialist should plot the proposed boring locations on a site topographic map prior to initiation of drilling. Notes taken during the site visit should be reviewed before boring locations are selected so that site access restrictions can be considered. Boring locations should never be selected arbitrarily or randomly. Alternate boring locations should be considered and a contingency plan should be developed in case a boring needs to be relocated due to access restrictions or unexpected geologic conditions (e.g., locate a replacement boring within a maximum of 15 ft (4.5 m) from the location of a boring that could not be drilled at a particular location). Field personnel unfamiliar with the objectives and rationale behind the planning of the site exploration should maintain contact with the person in responsible charge of the field exploration during field activities and discuss issues such as the relocation of a boring with that person. Arbitrary or random boring selection will increase the chances of boring relocation, confusion, and wasted time in the field. Final boring locations should be surveyed and recorded as part of the permanent project record. Elevations and northing and easting should be provided for each boring.

3.14.1 Recommendations for Sampling Depth Intervals in Soils

It is difficult to establish a prescriptive drilling, sampling, and testing protocol that is applicable to all sites. To be most effective the geotechnical specialist should:

(1) apply conventional guidelines with project-specific requirements and constraints;

(2) recognize the advantages and limitations of the available sampling devices and in-situ testing methods.

Some general recommendations for minimum sampling depth intervals are as follows:

- For preliminary screening, disturbed samples might be taken continuously in the upper 10 ft (3 m), at 5 ft (1.5 m) intervals up to 100 ft (30 m), and possibly every 10 ft (3 m) at depths greater than 100 ft (30 m).
- Disturbed samples should be taken at every abrupt change in stratum as indicated by a noticeable change in the drilling pressure.
- Where footings are to be placed on natural soil, continuous spoon samples are recommended for a depth equal to 15 ft (4.5 m) or 1.5 times the width of the footing, whichever is greater, as measured below the anticipated footing base elevation.

- For characterization and assessment of design properties in fine-grained soils, a minimum of one undisturbed sample should be taken in each stratum, with additional samples taken at 10- to 20-ft (3 to 6 m) intervals with depth.

- Undisturbed Shelby tube samples should be obtained at 5 ft (1.5 m) intervals in at least one boring in cohesive soils. For cohesive deposits greater than 30 ft (9 m) in depth, the tube sample interval can be increased to 10 ft (3 m). Undisturbed samples may not need to be taken in each boring if the deposit is relatively homogeneous within closely spaced borings.

These minimum guidelines and intervals may need to be increased depending upon the project requirements and site geologic conditions. The sampling interval may need to be increased when soil/rock conditions change frequently with depth; however, these changes need to be considered in the context of the design. Therefore, ongoing communication between field personnel and the office/design engineer is absolutely essential. Once the site stratigraphy has been established, it may not be necessary to sample every time there is a change in stratigraphy if the changes have no impact on design. For example, it may not be necessary to sample alternating layers of coarse-grained deposits where settlement is of concern, and for designs concerned with bearing capacity. Similarly, although samples below the anticipated extent of the area influenced by the load may be reduced, samples should be obtained in case the type of foundation changes between preliminary and final design.

The sampling interval will vary between individual projects and regional geologies. If soils are anticipated to be difficult to sample or trim in the laboratory due to defects (e.g., bent tubes, improper handling, etc.), the frequency of sample collection should be greater than average to offset the number of samples that may be unusable in the laboratory for performance property evaluation (e.g., shear strength). When borings are widely spaced, it may be appropriate to retrieve undisturbed samples in each boring. For closely spaced borings or in deposits of lateral uniformity, undisturbed samples may be needed only in select borings. If a thin clay seam is encountered during drilling and not sampled, the boring may need to be offset and re-drilled to obtain a sample.

It is often quite helpful to combine in-situ soundings with conventional disturbed/undisturbed sampling. For example, by performing CPT or CPTu soundings prior to conventional drilling and sampling, it may be possible to target representative and/or critical areas where samples can be obtained later. This concept was illustrated previously by the discussion in Section 3.9.7 and Figure 3-30. The use of precursor soundings may reduce some of the potential drilling redundancy in heterogeneous environments. Geophysical methods can also be used to provide useful information on conditions between and even beyond boring locations.

3.14.2 Recommendations for Sampling Depth Intervals in Rocks

For explorations for slopes and foundations within rock, it is important to consider structural geology in addition to the information obtained as part of a rock-coring program. For example, the orientation and characteristics of a clay-filled discontinuity are critical since they can be used to judge whether a rock slope will be stable or unstable or whether a structural foundation will undergo minor or significant settlement. A detailed structural geologic assessment may provide enough information to limit the scope of a rock-coring program significantly or even preclude such a program. For example, drilling and coring may not be required where applied loads are significantly less than the bearing capacity of the rock, where there is no possibility of sliding instability in a rock slope, or where there are extensive rock outcrops from which information can be obtained to establish the subsurface conditions confidently for design and constructability assessments (Wyllie, 1999).

3.14.3 Recommendations for Water Level Monitoring in Borings

The water level in each boring should be observed and the depth below the top of hole recorded on the drill log with the date and time of the reading for each of the following situations:

a) Water seepage or artesian pressure encountered during drilling. Artesian pressure may be measured by extending drill casing above the ground until flow stops. Report the pressure as the number of ft (m) of head above ground.

b) Water level at the end of each day and at completion of boring.

c) Water level 24 hours (minimum) after hole completion. Long term readings may require installation of a perforated plastic tube before abandoning the hole.

A false indication of water level may be obtained when water is used in drilling and adequate time is not permitted after the boring is completed for the water level to stabilize. In low permeability soils, such as clays, more than one week may be required to obtain accurate readings. Proper safety precautions should be taken if a hole is allowed to remain open for such an extended period of time.

3.15 GEOPHYSICAL TESTS

As indicated in Section 3.3, geophysical testing can be used as part of the initial site exploration to provide supplementary information to data collected by other means (i.e., borings, test pits, geologic surveys, etc.). Geophysical testing can be used for establishing stratification of subsurface materials, the profile of the top of bedrock, the depth to groundwater, the boundaries of various types of soil deposits, the rippability of hard soil and rock, and the presence and depth of voids, buried pipes, and existing foundations. Data from geophysical testing should always be correlated with information from the direct methods of exploration discussed previously.

3.15.1 Types of Geophysical Tests

There are a number of different types of geophysical in-situ tests that can be used to obtain stratigraphic information from which engineering properties can be estimated. Table 3-14 provides a summary of the various geophysical methods that are currently available in U.S. practice. Further information on the procedures used for these methods is provided in FHWA (2003). Additional general discussion regarding the major test methods listed in Table 3-14 is presented below, with particular emphasis on potential applications to highway engineering.

1. **Seismic Methods**: These methods are becoming increasingly popular for geotechnical engineering practice because they have the potential to provide data regarding the compression and shear wave velocities of the subsurface materials. The shear wave velocity is directly related to small-strain material stiffness, which, in turn, is often correlated to compressive strength and soil/rock type. These techniques are often used for assessing the vertical stiffness profile in a soil deposit and for assessing the location at depth of the interface between soil and rock. Seismic refraction method involves measurement of time of arrival of the initial ground motion generated by the energy source while the seismic reflection method involves measurement of the energy arrival after the initial ground motion.

2. **Electrical Methods**: These methods are usually used to locate voids or locally distinct materials. With regards to highway applications, these procedures may be used to assess the potential for karst activity along a planned transportation corridor, or for locating large underground voids and/or specific underground anomalies such as storage drums and/or tanks. Electrical methods provide qualitative information only and are usually part of a two- or three-phased exploration program.

Table 3-14
Geophysical testing techniques (modified after FHWA, 2002a)

Method	Basic Field Procedures	Applications	Limitations
SEISMIC METHODS			
Seismic Refraction	Impact load is applied to the ground surface. Seismic energy refracts off soil/rock layer interfaces and the *time of arrival* is recorded on the ground surface using several dozen geophones positioned along a line or performing repeated events using a single geophone.	• depth to bedrock • depth to water table • thickness and relative stiffness soil/rock layers	• does not work if stiffness decreases with depth or if soft layer underlies stiff layer • works best when sharp stiffness discontinuity is present
Spectral-Analysis-of-Surface-Waves (SASW)	Impact load is applied to the ground surface. Surface waves propagate along ground surface and are recorded on the ground surface with two geophones positioned along a line.	• depth to bedrock • measurement of shear wave velocity • thickness and stiffness of surface pavement layer • qualitative indicator of cracking in pavement	• resolution decreases significantly with increasing depth • accurate interpretation may require a significant amount of expertise • interpretation is difficult if a stiff layer overlies a soft layer and soft layer properties are desired
ELECTRICAL METHODS			
DC Resistivity	DC current is applied to the ground by electrodes. Voltages are measured at different points on the ground surface with other electrodes positioned along a line.	• depth to water table • inorganic groundwater contamination • groundwater salinity • soil layer thickness • delineation of certain vertical features (e.g., sinkholes, contamination plumes, waste trenches)	• slow; must install electrodes directly in the ground • resolution decreases significantly with increasing depth • resolution is difficult in highly heterogeneous deposits
Electromagnetics	Electrical coils are held over the ground. Current passing through the coils induces a magnetic field in the ground, which is measured with receiver coils.	• groundwater salinity • inorganic groundwater contamination • detection of buried metal objects • delineation of certain vertical features (e.g., sinkholes, contamination plumes, waste trenches)	• extra effort is required to characterize depth of target • resolution decreases significantly with increasing depth
Ground Penetrating Radar (GPR)	Electromagnetic energy is pulsed into the ground. This energy reflects off boundaries between different soil layers and is measured at the ground surface.	• depth to water table • identification of buried objects • thickness of pavement layers • void detection	• not effective below the water table or in clay • depth of penetration is limited to about 30 ft (10 m)

Table 3-14
Geophysical testing techniques (modified after FHWA, 2002a)

Method	Basic Field Procedures	Applications	Limitations
GRAVITY AND MAGNETIC METHODS			
Gravity	The Earth's gravitational field is measured at the ground surface.	• identification of large subsurface voids • identification of large objects possessing unusually high or low densities	• results are non-unique (i.e. more than one subsurface condition can give the same result) • primarily, large-scale reconnaissance tool; applications in highway engineering are limited
Magnetics	The Earth's magnetic field is measured at the ground surface.	• identification of ferrous materials • identification of soil/rock containing large amounts of magnetic minerals	• results are non-unique (i.e. more than one subsurface condition can give the same results) • primarily a large-scale reconnaissance tool; applications in highway engineering are limited
NEAR-SURFACE NUCLEAR METHODS			
Neutron Moisture Content	Instrument is placed on the ground surface and neutrons are emitted into the ground. Energy of returning neutrons is related to the moisture content in the ground (hydrogen atoms decrease the energy of the neutrons detected at the sensor).	• estimate of water content in compacted soil • estimate of asphalt content in asphalt concrete • can be quantitative if properly calibrated to site conditions	• limited exploration depth (a few inches) • possible health and safety hazard if operators not properly trained • will detect hydrogen ion (i.e. gas, clay) in non-water bearing stratum
Gamma Density	Instrument is placed on the ground surface and gamma radiation is emitted into the ground. Returning gamma energy is a function of material density (denser materials absorb more gamma energy so less is detected at the sensor)	• estimate of density of soil or asphalt concrete	• limited exploration depth (less than one foot); • exploration depth further limited to a few inches if ground cannot be penetrated • possible health and safety hazard if operators not properly trained

Table 3-14 (continued)
Geophysical testing techniques (modified after FHWA, 2002a)

Method	Basic Field Procedures	Applications	Limitations
BOREHOLE METHODS			
Crosshole/ Downhole	Energy sources and geophones are placed in boreholes and/or on ground surface; interval travel times are converted into seismic wave velocity as a function of depth in the borehole.	• measurement of wave velocities for seismic site response analysis • depth to water table • correlation of lithologic units with surface seismic • identification of thin layers at depth	• requires one or more boreholes and significant support field equipment
Suspension Logger	Field instrument is placed in a fluid-filled borehole and used to measure P-(compression) and S-(shear) wave velocities in surrounding soil or rock.	• measurement of wave velocities for seismic site response analysis • correlation of lithologic units with surface seismic • identification of thin layers at depth	• requires borehole and significant support field equipment, which is expensive • borehole must be fluid-filled
Electrical Logging	Field instrument is placed in a borehole. Electrical fields are directly applied or electromagnetically induced into surrounding soil or rock and electrical resistivity is measured.	• estimate of soil/rock permeability or porosity • identification of inorganic contaminant plumes or saltwater intrusion • identification of thin layers at depth	• requires borehole and significant support field equipment, which is expensive • generally cannot operate in a cased borehole • may require fluid-filled borehole • results may be dependent upon drilling mud salinity
Nuclear Logging	Field instrument is placed in a borehole. Surrounding soil or rock is irradiated with neutrons particles and/or gamma energy. Energy and neutrons returning to the instrument are measured and related to rock density, porosity and pore fluid type.	• estimate of soil/rock type, density, porosity, and pore fluid density • identification of thin layers at depth	• requires borehole and significant support field equipment, which is expensive • possible health and safety hazard if operators are not properly trained
Lithology Logging	Field instrument is placed in a borehole; naturally occurring electrical fields and radiation levels are related to soil or rock type.	• classification of soil or rock type • identification of thin layers at depth	• requires borehole and significant support field equipment, which is expensive • may require fluid-filled borehole • results are dependent upon site-specific conditions and/or borehole fluid salinity

3. **Gravity and Magnetic Methods**: These methods are similar to electrical methods, except that they rely on correlations between the potential gravitational and/or magnetic influence of voids and subsurface anomalies and measured differences in the earth's micro-gravitational and/or magnetic fields, rather than on changes in electrical fields. These methods provide measurements at specific points unlike seismic and electrical methods that provide measurements over large areas.

4. **Near-surface nuclear methods**: These methods have been used for several years for compaction control of fills in the field. Through careful calibration, it is possible to assess the moisture content and density of compacted soils reliably. These methods have been widely adopted as reliable quantitative methods.

5. **Borehole Methods**: Downhole geophysical methods provide reliable indications of a wide range of soil properties. For example, downhole/crosshole methods provide reliable measures of shear wave velocity. As indicated previously, shear wave velocity is directly related to small-strain stiffness and is correlated to strength and soil/rock type. Although downhole logging methods have seen little use in highway construction, they have been the mainstay for deep geologic characterization in oil exploration. The principal advantage of downhole logging is the ability to obtain several different geophysical tests/indicators by "stringing" these tools together in a deep borehole.

3.15.2 Advantages and Disadvantages of Geophysical Tests

As with the other methods of exploration, geophysical testing offers some advantages and some disadvantages that should be considered before these techniques are recommended for a specific application. These are summarized as follows:

3.15.2.1 Advantages of Geophysical Tests

1. Many geophysical tests are non-invasive. Therefore such tests offer significant benefits in cases where conventional drilling, testing, and sampling are difficult (e.g., deposits of gravel, talus deposits, etc.) or where potentially contaminated soils may occur in the subsurface.

2. In general, geophysical testing can cover a relatively large geographical area thereby providing the opportunity to characterize large areas with relatively few tests. Geophysical testing is particularly well-suited to projects that have large longitudinal extent such as new highway construction.

3. Geophysical measurements are used to assess the properties of soil and rock at very small strains, typically on the order of 0.001 percent, thereby providing information on truly elastic properties.

4. For the purpose of obtaining information on the subsurface, geophysical methods are relatively inexpensive considering the large area over which they provide information.

3.15.2.2 Disadvantages of Geophysical Tests

1. Most methods work best for situations in which there is a large difference in the property being measured between adjacent subsurface units. In seismic methods, it is difficult to develop good stratigraphic profiling if the general stratigraphy consists of hard material overlying soft material.

2. Each geophysical method has limitations that may be associated with equipment, signal noise, unfavorable site and subsurface conditions, and processing constraints.

3. Results can be non-unique and are generally interpreted qualitatively. Therefore useful results can be obtained only through analyses performed by a geotechnical specialist experienced with the particular testing method.
4. Specialized and more electronically sophisticated equipment is required as compared to the more conventional subsurface exploration tools.

3.15.3 Examples of Uses of Geophysical Tests

The following are a few examples where geophysical testing could be used on highway projects to compliment conventional exploration.

1. <u>Highly Variable Subsurface Conditions</u>: In several geologic settings, the subsurface conditions along a transportation corridor may be expected to be variable. This variability could be from underlying karst development above limestone; alluvial deposits, including buried terrace gravels, across a wide floodplain; buried boulders in a talus slope, etc. For these cases, conventional exploration techniques may be very difficult and if "refusal" is encountered at one depth, there is a strong likelihood that different materials could underlie the region. Development of a preliminary subsurface characterization profile by using geophysical testing could prove advantageous in designing future focused explorations.

2. Regional Studies: Along a transportation corridor it may be necessary to assess the depth to (and through) rippable rock or highly cemented caliche. Alternative alignments may or may not be possible, but the cost implications may be significant. Therefore, it is important to obtain a profile related to rock/soil stiffness. Geophysical testing is a logical consideration for this application as a precursor to invasive explorations.

3. Settlement Sensitive Structures: The prior two examples related to cases where the geophysical testing served as the front-end of a multi-phase project. In the case where a settlement-sensitive structure is to be founded on deposits of sands, knowledge of the in-situ modulus of the sand deposit is critical. After the characteristics of the site are assessed, it may be helpful to quantify the deformation modulus by the use of geophysical testing at the specific foundation site.

These examples demonstrate that geophysical testing can play a potentially important role in the subsurface characterization of soils and rocks. As with the other investigative "tools" described in this document, the particular selection of the appropriate technology is very much a function of the site conditions and the goals of the characterization program.

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 <u>not</u> 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)

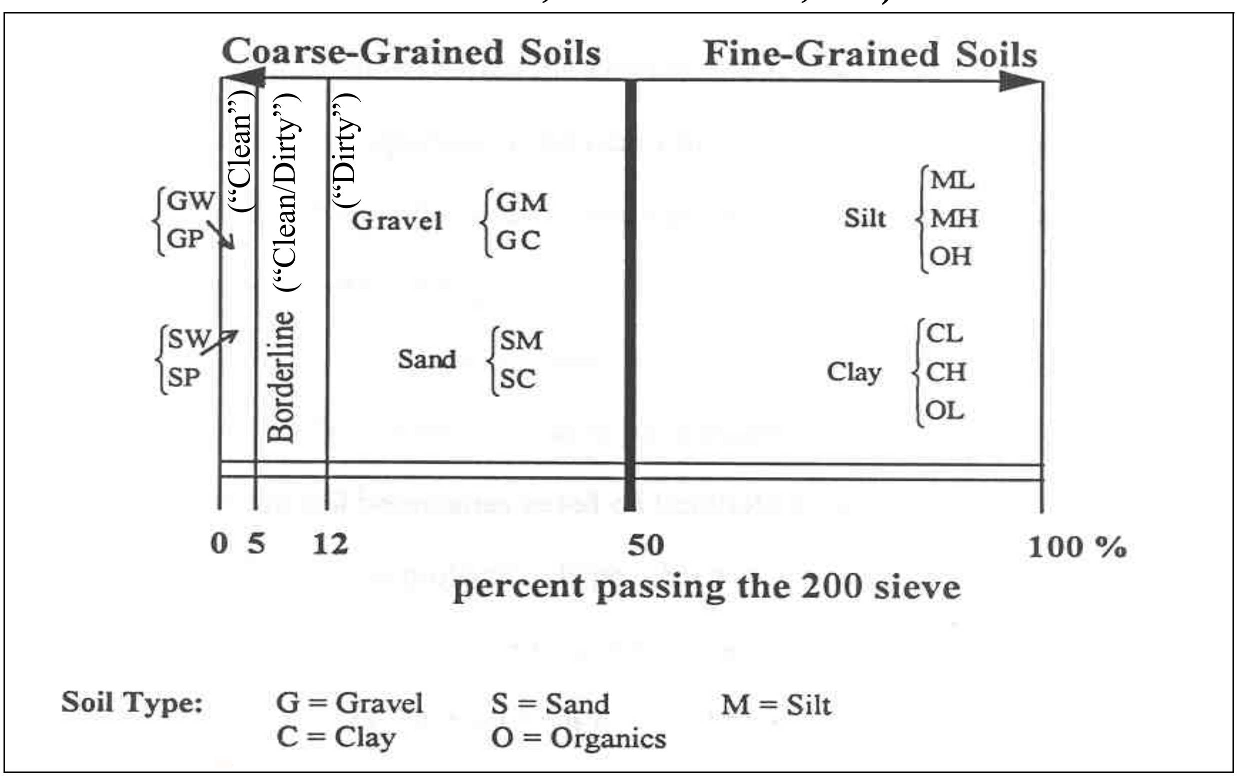


Table 4-9
Soil classification chart (laboratory method) (after ASTM D 2487)

Criteria for Assigning Group Symbols and Group Names Using Laboratory Tests[a] **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			Soil Classification	
			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.

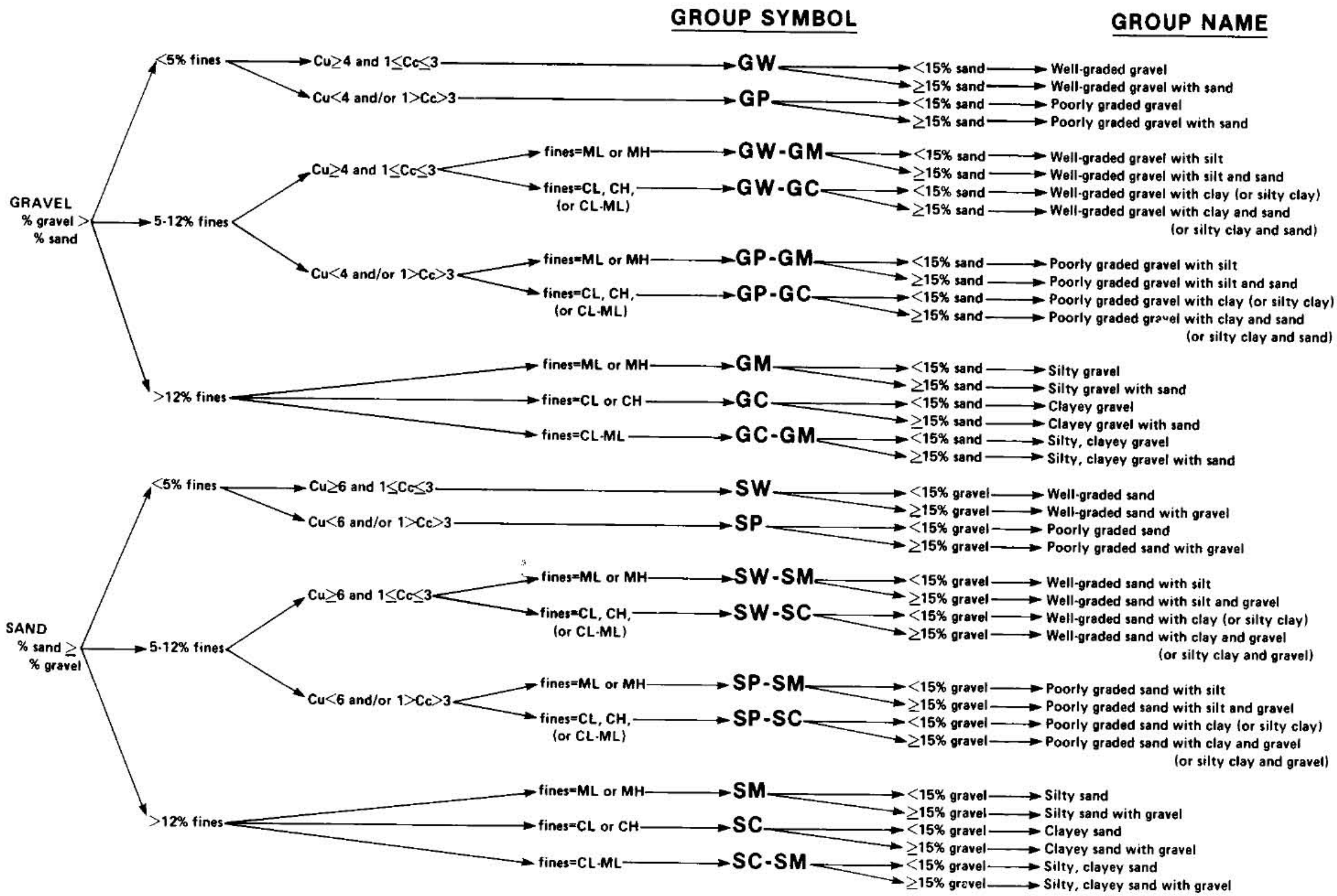


Figure 4-1: Flow chart to determine the group symbol and group name for coarse-grained soils (ASTM D 2487).

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:

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

- **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:

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

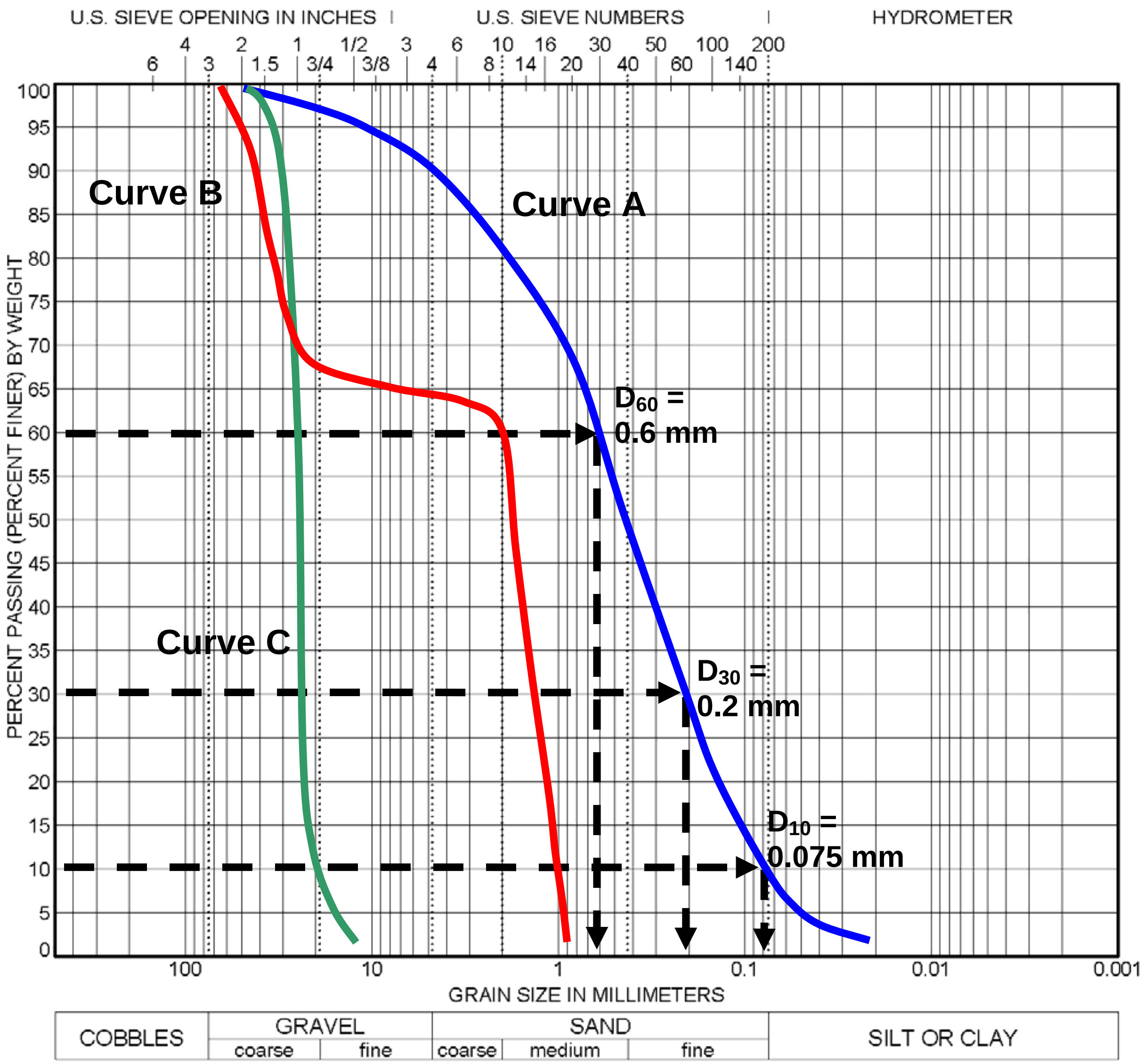
By use of the two "shape" parameters, C_u and C_c, the uniformity of the coarse-grained soil (gravel and sand) can now be classified as well-graded (non-uniform), poorly graded (uniform), or gap graded (uniform or non-uniform). Table 4-10 presents criteria for such classifications.

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)

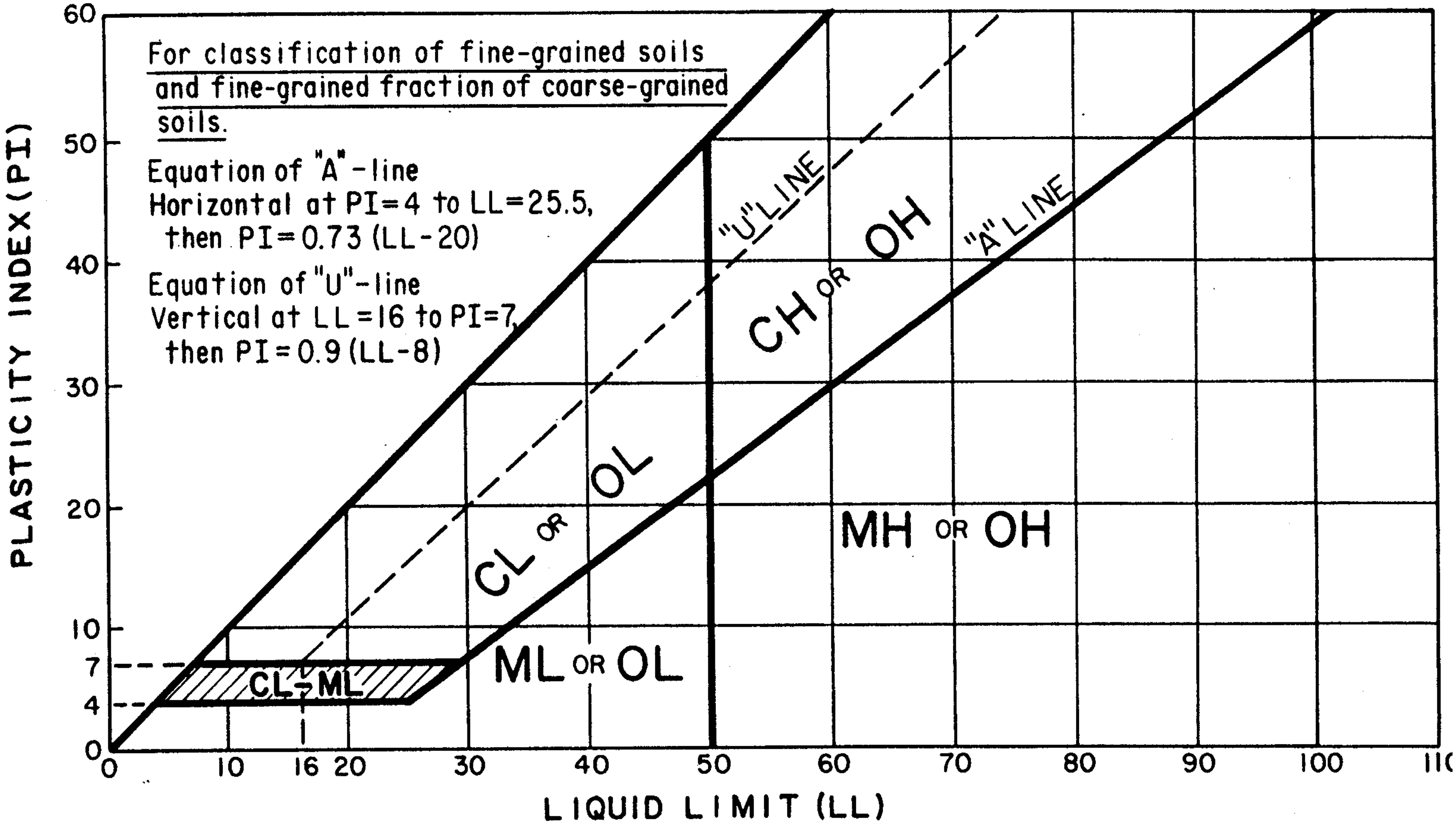


Figure 4-3. Plasticity chart for Unified Soil Classification System (ASTM D 2487).

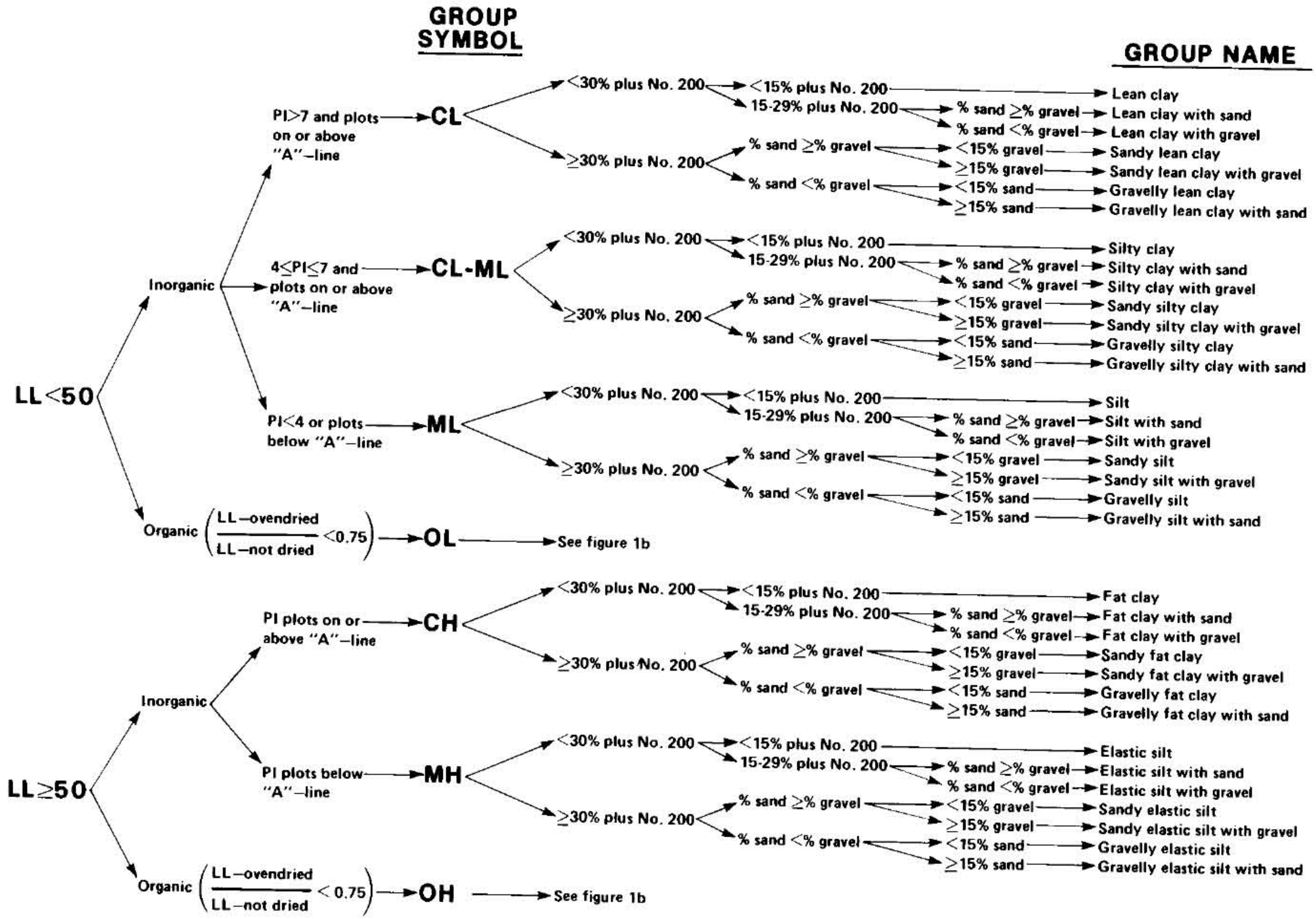


Figure 4-4a. Flow chart to determine the group symbol and group name for fine-grained soils (ASTM D 2487).

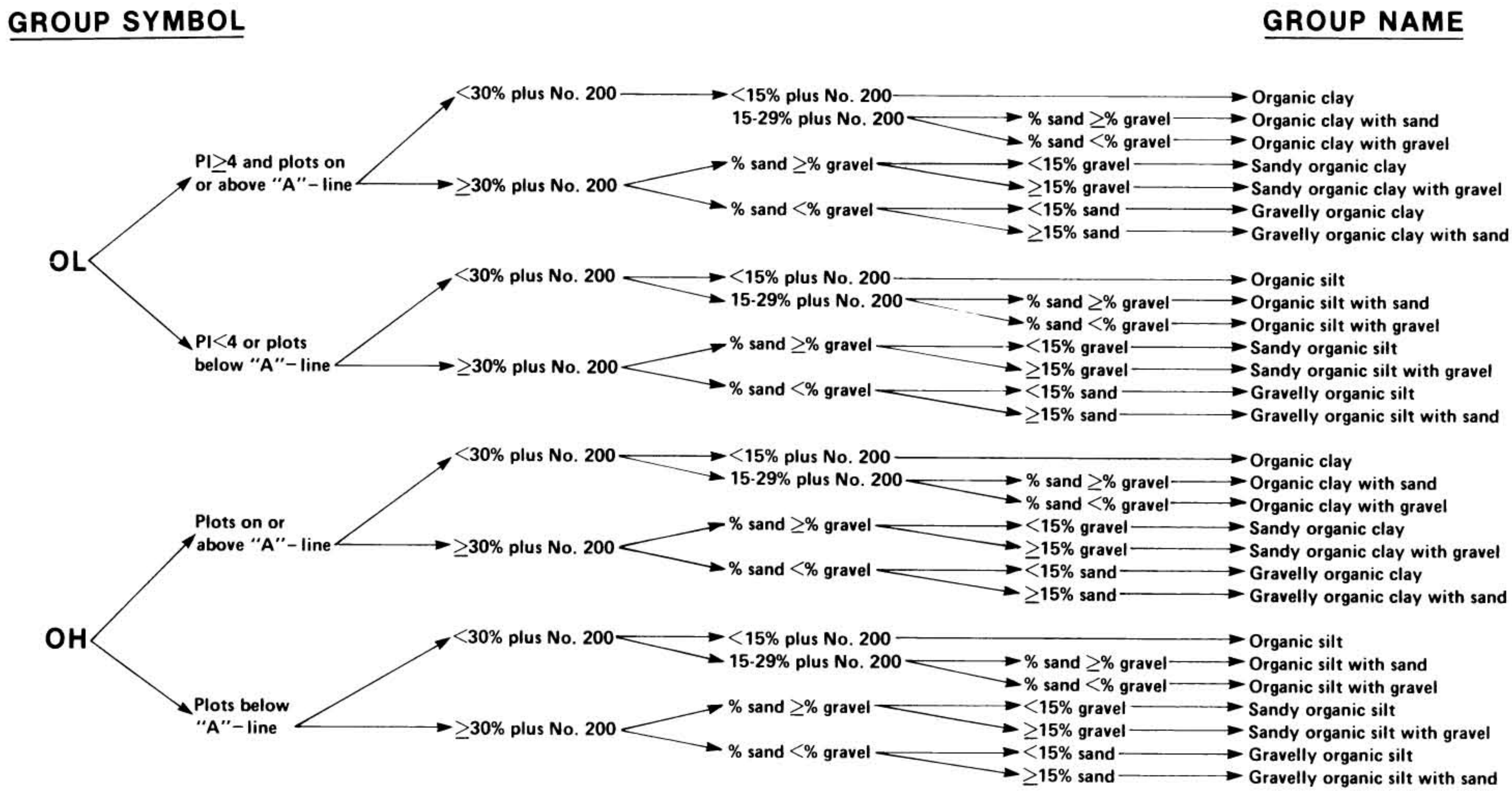


Figure 4-4b. Flow chart to determine the group symbol and group name for organic soils (ASTM D 2487).

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.

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

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.

<u>Classification procedure:</u>
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.

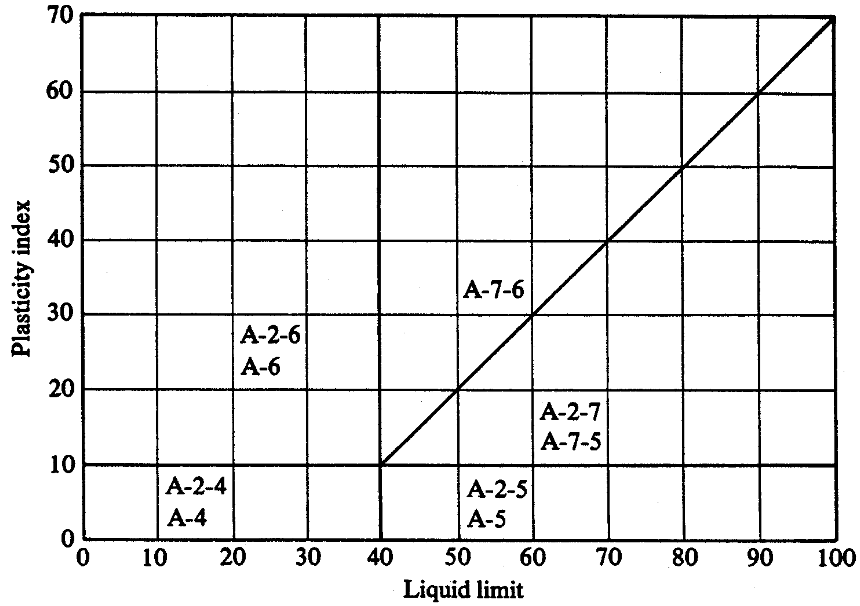


Figure 4-5. Range of liquid limit and plasticity index for soils in groups A-2, A-4, A-5, A-6 and A-7 per AASHTO M 145 (or ASTM D 3282).

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.

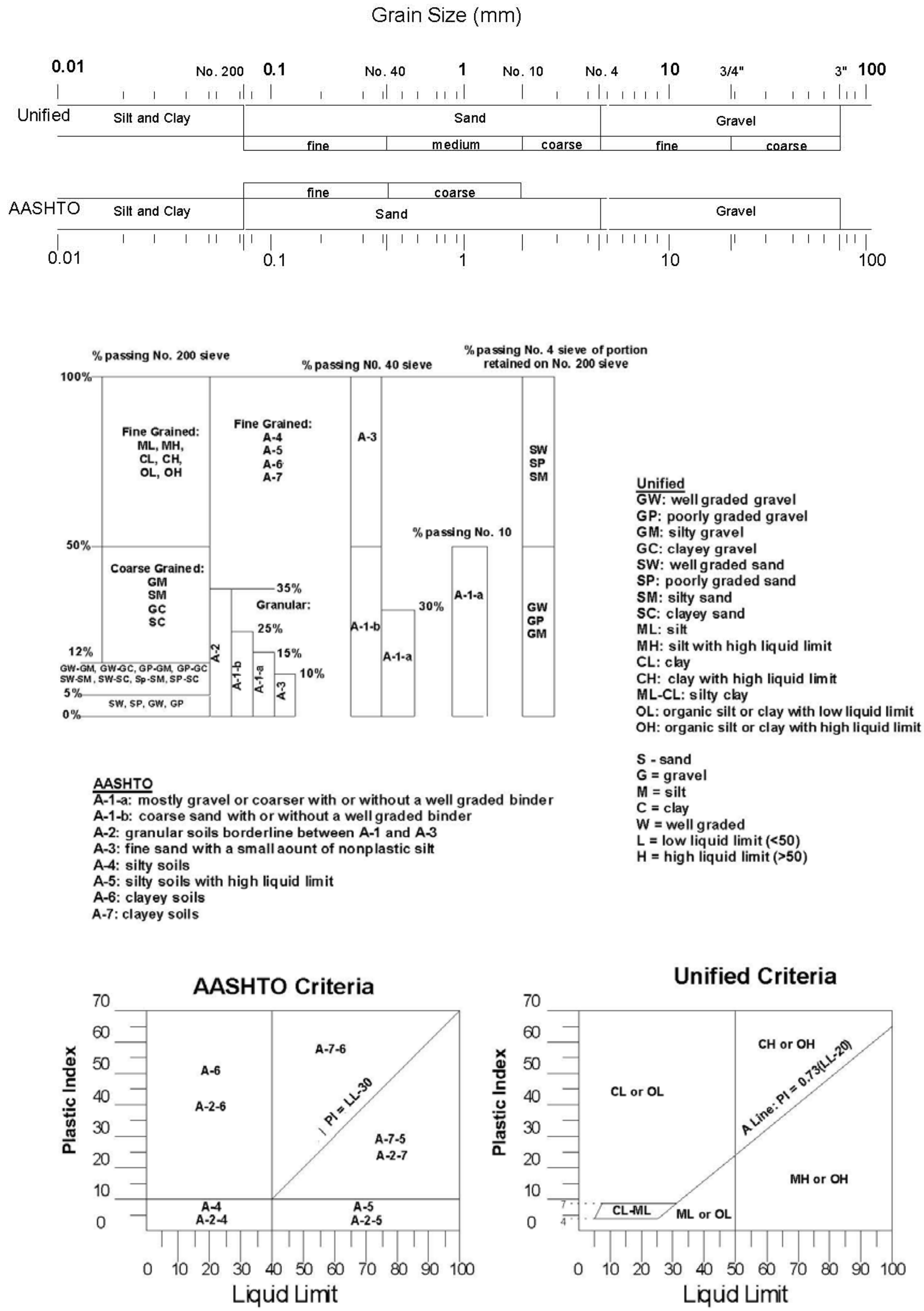


Figure 4-6. Comparison of the USCS with the AASHTO soil classification system (after Utah DOT – Pavement Design and Management Manual, 2005).

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			**Amount of Infilling**			**Discontinuity Spacing (m)***		
F	-	Fault	Su	-	Surface Stain	EW	-	Extremely Wide (>6)
J	-	Joint	Sp	-	Spotty	VW	-	Very Wide (2-6)
Sh	-	Shear	Pa	-	Partially Filled	W	-	Wide (0.6-2)
Fo	-	Foliation	Fi	-	Filled	M	-	Moderate (0.2-0.6)
V	-	Vein	No	-	None	C	-	Close (0.06-0.2)
B	-	Bedding				VC	-	Very Close (0.02-0.06)
						EC	-	Extremely close (<0.02)

Discontinuity Width (mm)*			**Surface Shape of Joint**		
W	-	Wide (12.5-5.0)	Wa	-	Wavy
MW	-	Moderately Wide (2.5-12.5)	Pl	-	Planar
N	-	Narrow (1.25-2.5)	St	-	Stepped
VN	-	Very Narrow (<1.25)	Ir	-	Irregular
T	-	Tight (~ 0)			

Type of Infilling			**Roughness of Surface**		
Cl	-	Clay	Slk	-	Slickensided (surface has smooth, glassy finish with visual evidence of striations)
Ca	-	Calcite	S	-	Smooth (surface appears smooth and feels so to the touch)
Ch	-	Chlorite	SR	-	Slightly Rough (asperities on the discontinuity surface are distinguishable and can felt)
Fe	-	Iron Oxide	R	-	Rough (some ridges and side-angle steps are evident; asperities are clearly visible, and discontinuity surface feels very abrasive)
Gy	-	Gypsum/Talc	VR	-	Very Rough (near-vertical steps and ridges occur on the discontinuity surface
H	-	Healed			
No	-	None			
Py	-	Pyrite			
Qz	-	Quartz			
Sd	-	Sand			

* 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

	PARAMETER						
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).

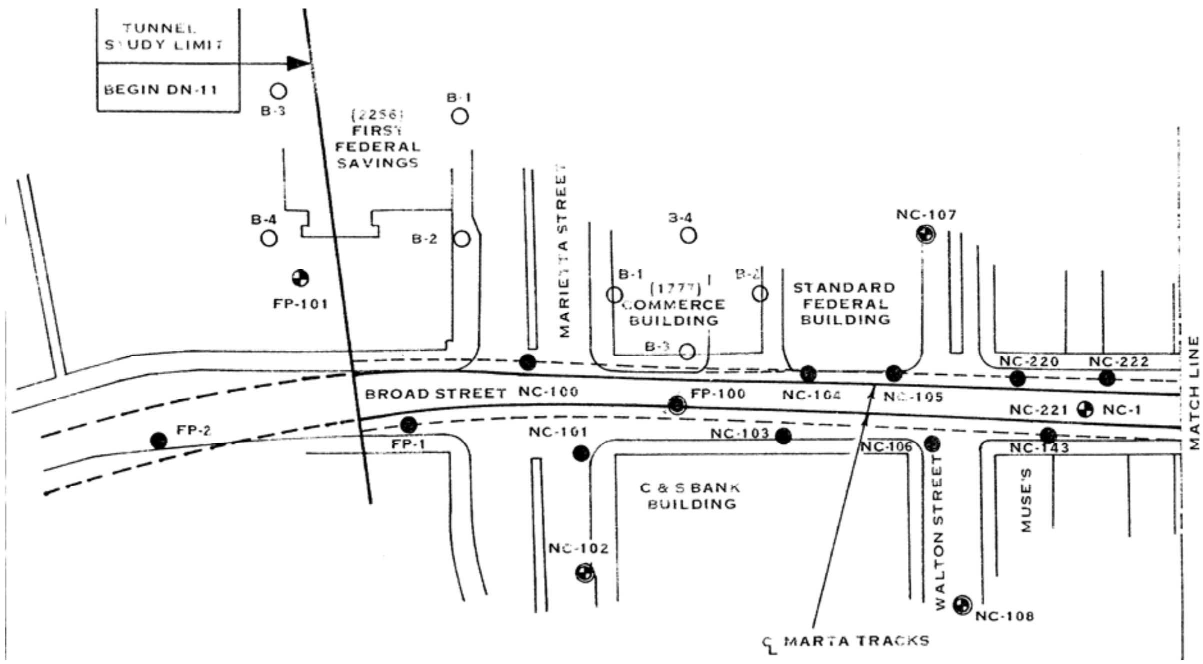


Figure 4-8. Example boring location plan (FHWA, 2002a).

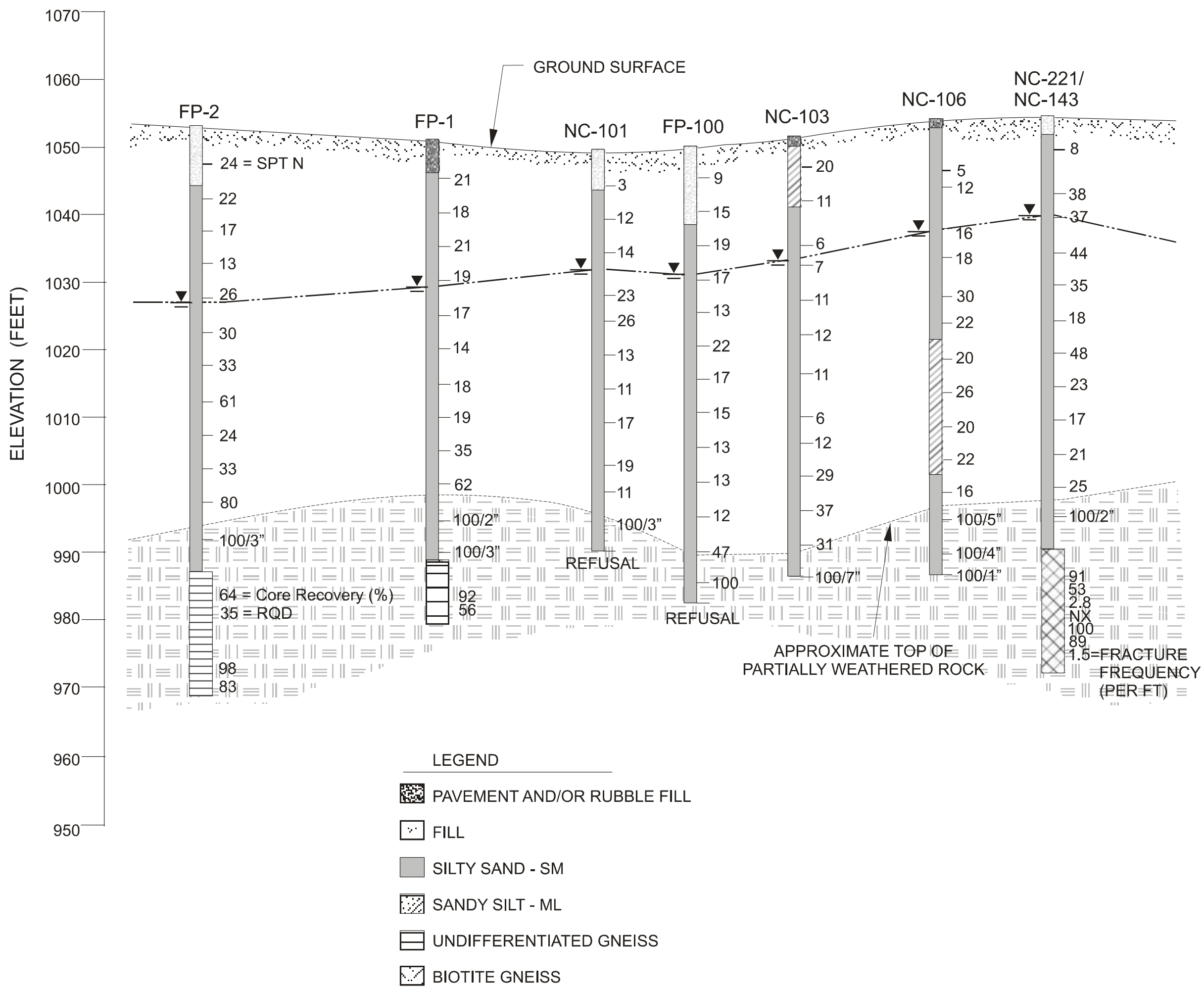


Figure 4-9. Example interpreted subsurface profile (FHWA, 2002a).

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.

CHAPTER 5.0
LABORATORY TESTING
FOR GEOTECHNICAL DESIGN AND CONSTRUCTION

Laboratory testing of soils and rocks is a fundamental element of geotechnical engineering. The complexity of testing required for a particular project may range from a simple moisture content determination to sophisticated triaxial strength testing. A laboratory test program should be well-planned to optimize the test data for design and construction. The geotechnical specialist, therefore, should recognize the project's issues ahead of time so as to optimize the testing program, particularly strength and consolidation testing.

Laboratory testing of samples recovered during subsurface investigations is the most common technique to obtain values of the engineering properties necessary for design. A laboratory-testing program consists of "index tests" to obtain general information on categorizing materials, and "performance tests" to measure specific properties that characterize soil behavior for design and constructability assessments (e.g., shear strength, compressibility, hydraulic conductivity, etc.). This chapter provides information on common laboratory test methods for soils and rocks including testing equipment, general procedures related to each test, and parameters measured by the tests.

5.01 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.

5.1 QUALITY ASSURANCE FOR LABORATORY TESTING

Laboratory testing will be required for most projects. Therefore, it is necessary to select the appropriate types and quantities of laboratory tests to be performed. A careful review of all data obtained during the field investigation and a thorough understanding of the preliminary design of geotechnical, structural and hydraulic features of the project are essential to develop an appropriately scoped laboratory testing program. In some cases owners may hire external testing laboratories to perform select tests. It is necessary that testing requests be clear and sufficiently detailed. Unless specialized testing is required, the owner should require that all testing be performed in accordance with the appropriate specifications for laboratory testing such as those codified in AASHTO and/or ASTM. Several tables are presented in this chapter that summarize various common tests for soils and rocks per AASHTO and ASTM standards. In order to assure that the results of laboratory testing are representative, several precautions must be taken before the tests themselves are performed. These precautions include: sample tracking, sample storage, sample handling to prevent sample disturbance, and sample selection. Discussion of each of these precautions follows.

5.1.1 Sample Tracking

Whether the laboratory testing is performed in-house or is subcontracted, samples will likely be assigned a laboratory identification number that differs from the identification number assigned in the field. A list should be prepared to match the laboratory identification number with the field identification number. This list can also be used to provide tracking information to ensure that each sample arrived at the lab. When laboratory testing is requested, both the field identification number and the laboratory identification number should be used on the request form. An example request form is shown in Figure 5-1. A spreadsheet or database program is useful to manage sample identification data.

5.1.2 Sample Storage

Undisturbed soil samples should be transported and stored so that that the moisture content is maintained as close as possible to the natural conditions (AASHTO T 207, ASTM D 4220 and D 5079). Samples should not be placed, even temporarily, in direct sunlight. Shelby tubes should be stored in an upright position with the top side of the sample up in a humid room with relative humidity above 90%.

Long-term storage of soil samples in Shelby tubes is not recommended. As storage time increases, moisture will migrate within the tube. Potential for disturbance and moisture migration within the sample will increase with time, and samples tested 30 days after their retrieval should be noted on the laboratory data sheet. Excessive storage time can lead to additional sample disturbance that will affect strength and compressibility properties. Additionally, stress relaxation, temperature changes, and storage in a room with humidity below 90 percent may have detrimental effects on the samples. Long-term storage of soil samples should be in temperature- and humidity-controlled environments. The temperature control requirements may vary from sub freezing to ambient and above, depending on the environment of the parent formation. The relative humidity for soil storage normally should be maintained at 90 percent or higher to prevent moisture evaporation from the samples.

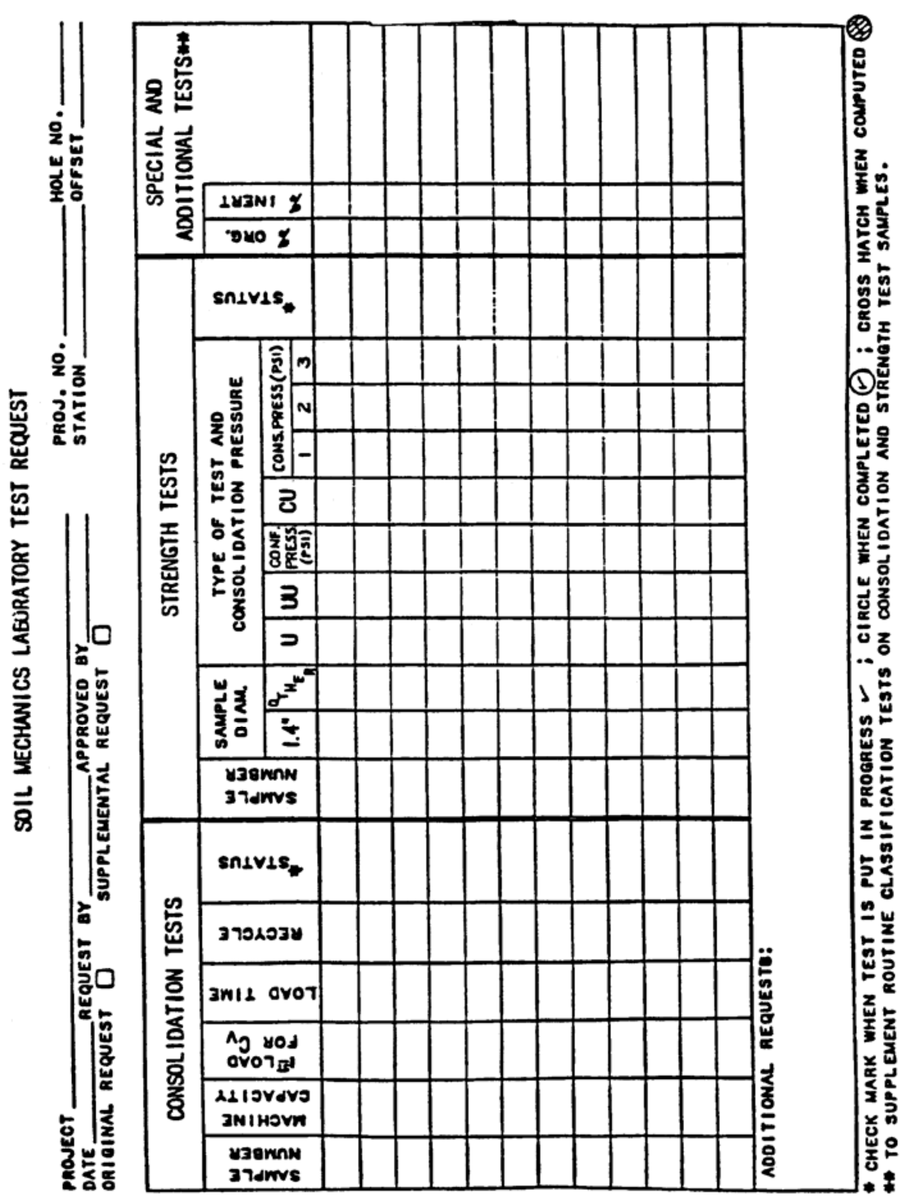


Figure 5-1. Sample laboratory test request form (Note: only some tests are included in this sample form).

Long-term storage of soil samples in Shelby tubes is not recommended for another reason. During long term storage the sample tubes may corrode. Corrosion accompanied by adhesion of the soil to the tube may result in the development of such a large sidewall resistance that some soils may experience internal failures during extrusion. Often these failures cannot be seen by the naked eye; x-ray radiography (ASTM D 4452) will likely be necessary to confirm the presence of such conditions. If these samples are tested as "undisturbed" specimens, the results may be misleading.

5.1.3 Sample Handling

Careless handling of nominally undisturbed soil samples after they have been retrieved may cause major disturbances that could influence test results and lead to serious design and construction consequences. Samples should always be handled by experienced personnel in a manner that ensures that the sample maintains structural integrity and its natural moisture condition. Saws and knives used to prepare soil specimens should be clean and sharp. Preparation time should be kept to a minimum, especially where the maintenance of the moisture content is critical. Specimens should not be exposed to direct sun, freezing, or precipitation.

5.1.4 Effects of Sample Disturbance

As a soil sample is removed from the ground during a conventional soil investigation, its in-situ effective stress condition is being changed. In addition, nominally undisturbed specimens taken from samples obtained from drilled boreholes will become disturbed as a result of the drilling itself, sampling, sample extrusion, and sample trimming to form a specimen for testing. These processes will also change the effective stress condition in the soil sample, i.e., the effective stress in the soil at the time after a sample is trimmed and prepared for testing is different from that of the same soil in the ground. Therefore the utmost care should be taken to minimize the effect of these processes in order for the results of laboratory tests to represent the in-situ soil behavior accurately.

5.1.5 Specimen Selection

The selection of representative specimens for testing is one of the most important aspects of sampling and testing procedures. Selected specimens must be representative of the formation or deposit being investigated. The geotechnical specialist should study the boring logs, understand the geology of the site, and visually examine the samples before selecting the test specimens. Samples should be selected on the basis of their color, physical appearance, structural features and an understanding of the disturbance of the samples. Specimens should be selected to represent all types of materials present at the site, not just the worst or the best.

Samples with discontinuities and intrusions may fail prematurely in the laboratory. The first inclination would be to test these samples. However, if these features are small and randomly located, they may not necessarily cause such failures in the field. Therefore samples having such local features should be noted, but not necessarily selected for testing since such samples may not be representative of the stratum in terms of its response to applied loads.

Certain considerations regarding laboratory testing, such as when, how much, and what type, can be decided only by an experienced geotechnical specialist. The following minimal criteria should be considered when the scope of the laboratory testing program is being determined:

- Project type (bridge, embankment, building, reconstruction or new construction, etc.)
- Size of the project (geographic extent).
- Loads to be imposed on the foundation soils (geometry, type, direction and magnitude).
- Performance requirements for the project (e.g., settlement and lateral deformation limitations).
- Vertical and horizontal variations in the subsurface profile as determined from boring logs and visual identification of subsurface material types in the laboratory.
- Known or suspected peculiarities of subsurface strata at the project location (e.g., swelling soils, collapsible soils, organics, etc.)
- Presence of visually observed intrusions, slickensides, fissures, concretions, etc.

The selection of tests should be considered preliminary until the geotechnical specialist is satisfied that the test results are sufficient to develop reliable subsurface profiles and provide the parameters needed for design.

5.2 LABORATORY TESTING FOR SOILS

Table 5-1 provides a listing of commonly-performed soil laboratory tests. Tables 5-2 and 5-3 provide a summary of typical soil index and performance tests, respectively. Additional information on these tests is provided in subsequent sections.

Table 5-1
Commonly performed laboratory tests on soils (after FHWA, 2002a)

Test Category	Name of Test	Test Designation AASHTO	Test Designation ASTM
Visual Identification	Practice for Description and Identification of Soils (Visual-Manual Procedure)	-	D 2488
	Practice for Description of Frozen Soils (Visual-Manual Procedure)	-	D 4083
Index Properties	Test Method for Determination of Water (Moisture) Content of Soil by Direct Heating Method	T 265	D 2216
	Test Method for Specific Gravity of Soils	T 100	D 854; D 5550
	Method for Particle-Size Analysis of Soils	T 88	D 422
	Test Method for Classification of Soils for Engineering Purposes	M 145	D 2487; D 3282
	Test Method for Amount of Material in Soils Finer than the No. 200 (0.075 mm) Sieve		D 1140
	Test Method for Liquid Limit, Plastic Limit, and Plasticity Index of Soils	T 89; T 90	D 4318
Compaction	Test Method for Laboratory Compaction Characteristics of Soil Using Standard Effort (12,375 ft. lbs/ft^3)	T 99	D 698
	Test Method for Laboratory Compaction Characteristics of Soil Using Modified Effort (56,250 ft.lbs/ft^3)	T 180	D 1557
Strength Properties	Test Method for Unconfined Compressive Strength of Cohesive Soil	T 208	D 2166
	Test Method for Unconsolidated, Undrained Compressive Strength of Cohesive Soils in Triaxial Compression	T 296	D 2850
	Test Method for Consolidated, Undrained Compressive Strength of Cohesive Soils in Triaxial Compression	T 297	D 4767
	Method for Direct Shear Test of Soils under Consolidated Drained Conditions	T 236	D 3080
	Test Methods for Modulus and Damping of Soils by the Resonant-Column Method	-	D 4015
	Test Method for Laboratory Miniature Vane Shear Test for Saturated Fine-Grained Clayey Soil	-	D 4648
	Test Method for CBR (California Bearing Ratio) of Laboratory-Compacted Soils	-	D 1883
	Test Method for Resilient Modulus of Soils	T 294	-
	Test Method for Resistance R-Value and Expansion Pressure of Compacted Soils	T 190	D 2844
Consolidation, Swelling, Collapse Properties	Test Method for One-Dimensional Consolidation Properties of Soils	T 216	D 2435
	Test Method for One-Dimensional Consolidation Properties of Soils Using Controlled-Strain Loading	-	D 4186
	Test Methods for One-Dimensional Swell or Settlement Potential of Cohesive Soils	T 258	D 4546
	Test Method for Measurement of Collapse Potential of Soils	-	D 5333
Permeability	Test Method for Permeability of Granular Soils (Constant Head)	T 215	D 2434
	Test Method for Measurement of Hydraulic Conductivity of Saturated Porous Materials Using a Flexible Wall Permeameter	-	D 5084
Corrosivity (Electro-chemical)	Test Method for pH for Peat Materials	-	D 2976
	Test Method for pH of Soils	-	D 4972
	Test Method for pH of Soil for Use in Corrosion Testing	T 289	G 51
	Test Method for Sulfate Content	T 290	D 4230
	Test Method for Resistivity	T 288	D 1125; G57
	Test Method for Chloride Content	T 291	D 512
Organic Content	Test Methods for Moisture, Ash, and Organic Matter of Peat and Other Organic Soils	T 194	D 2974

Table 5-2
Methods for index testing of soils (after FHWA, 2002a)

Test	Procedure	ASTM and/or AASHTO	Applicable Soil Types	Applicable Soil Properties	Limitations / Remarks
Moisture content, w_n	Dry soil in oven at 100 $\pm$ 5 ^oC	D 2216 T 265	Gravel, sand, silt, clay, peat	e_o, γ	Simple index test for all materials.
Unit weight and density	Extract a tube sample; measure dimensions and weight;	D 2216 T 265	Soils where undisturbed samples can be taken, i.e., silt, clay, peat	γ_t, γ_{dry}, ρ_{tot}, ρ_{dry}, p_t	Not appropriate for clean granular materials where undisturbed sampling is not possible. Very useful index test.
Atterberg limits, LL, PL, PI, SL, LI	LL – Moisture content associated with closure of the groove at 25 blows of specimen in Casagrande cup PL – Moisture content associated with crumbling of rolled soil at 1/8-in (3mm)	D 4318 T 89 T 90	Clays, silts, peat; silty and clayey sands to determine whether SM or SC	Soil classification and used in consolidation parameters	Not appropriate in non-plastic granular soil. Recommended for all plastic materials.
Mechanical sieve	Place air dry material on a series of successively smaller screens of known opening size and vibrate to separate particles of a specific equivalent diameter	D 422 T 88	Gravel, sand, silt	Soil classification	Not appropriate for clay soils. Useful, particularly in clean and dirty granular materials.
Wash sieve	Flush fine particles through a U.S. No. 200 (0.075 mm) sieve with water.	C 117 D 1140 T 88	Sand, silt, clay	Soil classification	Needed to assess fines content in dirty granular materials.
Hydrometer	Allow particles to settle, and measure specific gravity of the solution with time.	D 422 D 1140 T 88	Fine sand, silt, clay	Soil classification	Helpful to assess relative quantity of silt and clay.
Sand Equivalent	Sample passing No. 4 (4.75 mm) sieve is separated into sand and clay size particles	D 2419 T 176	Gravel, Sand, silt, clay	Aggregate classification Compaction	Useful for aggregates
Specific gravity of solids	The volume of a known mass of soil is compared to the known volume of water in a calibrated pyncnometer	D 854 D 5550 T 100	Sand, silt, clay, peat	Used in calculation of e_o	Particularly helpful in cases where unusual solid minerals are encountered.
Organic content	After performing a moisture content test at 110 ^oC (230^o F), the sample is ignited in a muffle furnace at 440 ^oC (824^o F) to measure the ash content.	D 2974 T 194	All soil types where organic matter is suspected to be a concern	Not related to any specific performance parameters, but samples high in organic content will likely have high compressibility.	Recommended on all soils suspected to contain organic materials.

Symbols used in Table 5-2

e_o: in-situ void ratio　γ_{dry}:dry unit weight　γ: unit weight　p_t: total vertical stress
ρ_{dry}:dry density　ρ_{tot}:total density　γ_t:total unit weight

Table 5-3
Methods for performance testing of soils (after FHWA, 2002a)

Test	Procedure	Applicable Soil Types	Soil Properties	Limitations / Remarks
1-D oedometer	Incremental loads are applied to a soil specimen confined by a rigid ring; deformation values are recorded with time; loads are typically doubled for each increment and applied for 24 hours each.	Primarily clays and silts; granular soils can be tested, but typically are not.	p_c, OCR, C_c, $C_{c\varepsilon}$, C_r, $C_{r\varepsilon}$, C_α, $C_{\alpha\varepsilon}$, c_v, k	Recommended for fine grained soils. Results can be useful index to other critical parameters.
Constant rate of strain oedometer	Loads are applied such that Δu is between 3 and 30 percent of the applied vertical stress during testing	Clays and silts; not applicable to free draining granular soils.	p_c, C_c, $C_{c\varepsilon}$, C_r, $C_{r\varepsilon}$, c_v, k	Requires special testing equipment, but can reduce testing time significantly.
Unconfined compression (UC)	A specimen is placed in a loading apparatus and sheared under axial compression with no confinement.	Clays and silts; cannot be performed on granular soils or fissured and varved materials	$s_{u,UC}$	Provides rapid means to approximate undrained shear strength, but disturbance effects, test rate, and moisture migration will affect results.
Unconsolidated undrained (UU) triaxial shear	The specimen is not allowed to consolidate under the confining stress, and the specimen is loaded at a quick enough rate to prevent drainage.	Clays and silts	$s_{u,UU}$	Sample must be nearly saturated. Sample disturbance and rate effects will affect measured strength.
Isotropic consolidated drained compression (CIDC)	The specimen is allowed to consolidate under the confining stress, and then is sheared at a rate slow enough to prevent build-up of pore water pressures.	Sands, silts, clays	ϕ', c', E	Can be run on clay specimen, but time consuming. Best triaxial test to obtain deformation properties.
Isotropic consolidated undrained compression (CIUC)	The specimen is allowed to consolidate under the confining stress with drainage allowed, and then is sheared with no drainage allowed, but pore water pressures measured.	Sands, silts, clays, peats	ϕ', c', $s_{u,CIUC}$, E	Recommended to measure pore pressures during test. Useful test to assess effective stress strength parameters. Not recommended for measuring deformation properties.
Direct shear	The specimen is sheared on a forced failure plane at a constant rate, which is a function of the hydraulic conductivity of the specimen.	Compacted fill materials; sands, silts, and clays	ϕ', ϕ'_r	Requires assumption of drainage conditions. Relatively easy to perform.

Test	Procedure	Applicable Soil Types	Soil Properties	Limitations / Remarks
Flexible Wall Permeameter	The specimen is encased in a membrane, consolidated, backpressure saturated, and measurements of flow with time are recorded for a specific gradient.	Relatively low permeability materials ($k \leq 1x10^{-5}$ cm/s); clays & silts	k	Recommended for fine grained materials. Backpressure saturation required. Confining stress needs to be provided. System permeability must be at least an order of magnitude greater than that of the specimen. Time needed to allow inflow and outflow to stabilize.
Rigid Wall Permeameter	The specimen is placed in a rigid wall cell, vertical confinement is applied, and flow measurements are recorded with time under constant head or falling head conditions.	Relatively high permeability materials; sands, gravels, and silts	k	Need to control gradient. Not for use in fine grained soils. Monitor for sidewall leakage.

Symbols used in Table 5-3

- ϕ': peak effective stress friction angle
- ϕ'_r residual effective stress friction angle
- c': effective stress cohesion intercept
- s_u: undrained shear strength
- p_c: preconsolidation stress
- OCR: overconsolidation ratio
- c_v: vertical coefficient of consolidation
- E: Young's modulus
- k: hydraulic conductivity
- C_c: compression index
- $C_{c\varepsilon}$: modified compression index
- C_r: recompression index
- $C_{r\varepsilon}$: modified recompression index
- C_α: secondary compression index
- $C_{\alpha\varepsilon}$: modified secondary compression index

5.3 LABORATORY INDEX TESTS FOR SOILS

5.3.1 General

Data generated from laboratory index tests provide an inexpensive way to assess soil consistency and variability among samples collected from a site. Information obtained from index tests is used to select samples for engineering property testing as well as to provide an indicator of general engineering behavior. For example, a soil with a high plasticity index (PI) can be expected to have high compressibility, low hydraulic conductivity, and high swell potential. Common index tests discussed in this section include moisture content, unit weight (wet density), Atterberg limits, particle size distribution, visual classification, specific gravity, and organic content. Index testing should be conducted on each type of soil material on every project. Information from index tests should be assessed prior to a final decision regarding the specimens selected for subsequent performance testing.

5.3.2 Moisture Content

The moisture (or water) content test is one of the simplest and least expensive laboratory tests to perform. Moisture content is defined as the ratio of the weight of the water in a soil specimen to the dry weight of the specimen. Natural moisture contents (w_n) of sands are typically $0 \le w_n \le 20$ %, whereas for inorganic and insensitive silts and clays, the typical range is $10 \le w_n \le 40$ %. However, for clays it is possible to have more water than solids (i.e., $w_n > 100\%$), depending upon the mineralogy, formation environment, and structure of the clay. Therefore, soft and highly compressible clays, as well as sensitive, quick, or organically rich clays, can exhibit water contents in the range of $40 \le w_n \le 300$ % or more.

Moisture content can be tested a number of different ways including: (1) a drying oven (ASTM D 2216); (2) a microwave oven (ASTM D 4643); or (3) a field stove or blowtorch (ASTM D 4959). While the microwave or field stove (or blowtorch) methods provide a rapid evaluation of moisture content, potential errors inherent with these methods require confirmation of results obtained by using ASTM D 2216. The radiation heating induced by the microwave oven and the excessive temperature induced by the field stove may release water entrapped in the soil structure (adsorbed water) that would normally not be released at 230^o F (110^o C), the maximum temperature specified by ASTM D 2216. Therefore, the microwave oven and field stove methods may yield greater values of moisture content than would occur from ASTM D 2216.

Field measurements of moisture content often rely on a field stove or microwave due to the speed of testing. For control of compacted material, it is common to use a nuclear gauge

(ASTM D 3017) in the field to assess moisture contents rapidly. Nuclear gage readings may indicate widely varying moisture contents for micaceous soils, i.e., soils containing a significant amount of mica particles. Results from nuclear techniques should be "calibrated" or confirmed by using the drying oven method (ASTM D 2216).

Moisture contents of soils as determined from in-situ moisture content tests may be altered during sampling, sample handling, and sample storage. Because the top end of the sample tube may contain water or collapse material from the borehole, moisture content tests should not be performed on material near the top of the tube. Also, as storage time increases, moisture will migrate within a specimen and lead to altered values of moisture content. If the sample is not properly sealed, moisture loss through drying of the sample will likely occur.

5.3.3 Unit Weight

The terms density (ρ) and unit weight (γ) are often incorrectly used interchangeably. The correct usage is that density implies mass while unit weight implies weight measurements. Density and unit weight are related through the gravitational constant (g) as follows: $\gamma = \rho g$. In this document they will be referenced as "density (unit weight)" if the usage is independent of the specific definition.

In the laboratory, soil unit weight and mass density are easily measured on tube (undisturbed) samples of natural soils. The moist (total) mass density is $\rho_t = M_t/V_t$, where M_t is the total mass of the soil sample including the mass of the moisture in the pores and V_t is the total volume of the soil sample. Similarly the dry mass density is given by $\rho_d = M_s/V_t$, where M_s is the mass of the solid component of the soil sample and V_t is the total volume of the soil sample. Likewise, the moist unit weight is $\gamma_t = W_t/V_t$, where W_t is the total weight including the weight of the water in the pores and V_t is the total volume of the soil sample. Similarly, the dry unit weight is defined as $\gamma_d = W_s/V_t$ where W_s is the weight of the solid component of the soil sample and V_t is the total volume of the soil sample. The relationship between the total and dry mass density and unit weight in terms of natural moisture content, w, is given by:

$$\rho_d = \frac{\rho_t}{1 + w} \qquad \text{5-1}$$

Since $\gamma = \rho g$ the relationship between total and dry unit weight is given by:

$$\gamma_d = \frac{\gamma_t}{1 + w} \tag{5-2}$$

Field measurements of soil mass density (unit weight) are generally restricted to shallow surface samples such as those obtained during placement of compacted fills. In those cases, field measurements of soil mass density (unit weight) can be accomplished by using drive tubes (ASTM D 2937), the sand cone method (ASTM D 1556), or a nuclear gauge (ASTM D 2922). To obtain unit weights or mass densities with depth, either high-quality thin-walled tube samples must be obtained (ASTM D 1587), or relatively expensive geophysical logging by gamma ray techniques (ASTM D 5195) can be employed.
Table 5-4 presents typical unit weights along with a range of void ratios for a variety of soils.

5.3.4 Particle Size Distribution

Particle size distributions by mechanical sieve and hydrometer analyses are useful for soil classification purposes. Procedures for grain size analyses are contained in ASTM D 422 and AASHTO T88. Testing is accomplished by shaking air-dried material through a stack of sieves having decreasing opening sizes. Table 2-3 in Chapter 2 listed U.S. standard sieve sizes and their associated opening sizes. Each successive screen in the stack has a smaller opening to capture progressively smaller particles. The amount retained on each sieve is collected, dried and weighed to determine the percentage of material passing that sieve size. An example of how to determine the grain size distribution from sieve data is shown in Figure 5-2. The grain size distribution curve corresponding to the data in Figure 5-2 is presented in Figure 5-3.

Testing of the finer grained particles is accomplished by suspending the chemically dispersed particles in a water column and measuring the change in the specific gravity of the liquid as the particles fall from suspension. This part of the test is commonly referred to as a hydrometer analysis.

Obviously, obtaining a representative specimen is an important aspect of this test. When soil samples are dried or washed for testing, it may be necessary to break up the soil clods. Care should be taken to avoid crushing of soft carbonate or sand particles. If the soil contains a substantial amount of fibrous organic materials, these may tend to plug the sieve openings during washing. The material settling over the sieve during washing should be constantly stirred to avoid plugging.

Table 5-4

Typical particle sizes, uniformity coefficients, void ratios and unit weights (from Kulhawy and Mayne, 1990)

Soil Type	Approximate Particle Size, mm			Uniformity Coefficient	Void Ratio		Normalized Unit Weight			
							Dry γ_{dry}/γ_w		Saturated γ_{sat}/γ_w	
	D_{max}	D_{min}	D_{60}	D_{60}/D_{10}	e_{max}	e_{min}	Min	Max	Min	Max
Uniform granular soil										
Equal spheres (theoretical)	-	-	-	1.0	0.92	0.35	-	-	-	-
Standard Ottawa sand	0.84	0.59	0.67	1.1	0.80	0.50	1.47	1.76	1.49	2.10
Clean, uniform sand	-	-	-	1.2 to 2.0	1.00	0.40	1.33	1.89	1.35	2.18
Uniform, inorganic silt	0.05	0.005	0.012	1.2 to 2.0	1.10	0.40	1.28	1.89	1.30	2.18
Well-graded granular soil										
Silty sand	2.0	0.005	0.02	5 to 10	0.90	0.30	1.39	2.04	1.41	2.28
Clean, fine to coarse sand	2.0	0.05	0.09	4 to 6	0.95	0.20	1.36	2.21	1.38	2.37
Micaceous sand	-	-	-	-	1.20	0.40	1.22	1.92	1.23	2.21
Silty sand and gravel	100	0.005	0.02	15 to 300	0.85	0.14	1.43	2.34	1.44	2.48
Silty or sandy clay	2.0	0.001	0.003	10 to 30	1.80	0.25	0.96	2.16	1.60	2.36
Gap-graded silty clay with gravel or larger	250	0.001	-	-	1.00	0.20	1.35	2.24	1.84	2.42
Well-graded gravel, sand, silt, and clay	250	0.001	0.002	25 to 1,000	0.70	0.13	1.60	2.37	2.00	2.50
Clay (30 to 50% < 2μ size)	0.05	0.5μ	0.001	-	2.40	0.50	0.80	1.79	1.51	2.13
Colloidal clay (over 50% < 2μ size)	0.01	10Å	-	-	12.00	0.60	0.21	1.70	1.14	2.05
Organic silt	-	-	-	-	3.00	0.55	0.64	1.76	1.39	2.10
Organic clay (30 to 50% < 2μ size)	-	-	-	-	5.40	0.70	0.48	1.60	1.30	2.00

Note: γ_w= 62.4 pcf (9.80 kN/m^3); μ = 10^{-3} mm; Å : Angstrom = 10^{-7} mm

U.S. Standard sieve Nos	Total dry weight of soil sample sieved 200.0 g	Retained on sieve		Cumulative retained		Cumulative passing	
		g	%	g	%	g	%
1	2	3	4	5	6	7	8
	G						
4		0.00	0.00	0.00	0.00	200.00	100.00
10		2.84	1.42	2.84	1.42	197.16	98.58
20		5.66	2.83	8.50	4.25	191.50	95.75
40		46.04	23.02	54.54	27.27	145.46	72.72
60		44.00	22.00	98.54	49.27	101.46	50.73
100		23.64	11.82	122.18	61.09	77.82	38.91
140		11.26	5.63	133.44	66.72	66.56	33.28
200		63.16	31.58	196.60	98.30	3.40	1.70
-200 (Pan)		3.40	1.70	200.00	100.00		
		200.00g	100.00%				

Figure 5-2. Example grain size distribution based on sieve analysis (Jumikis, 1962).

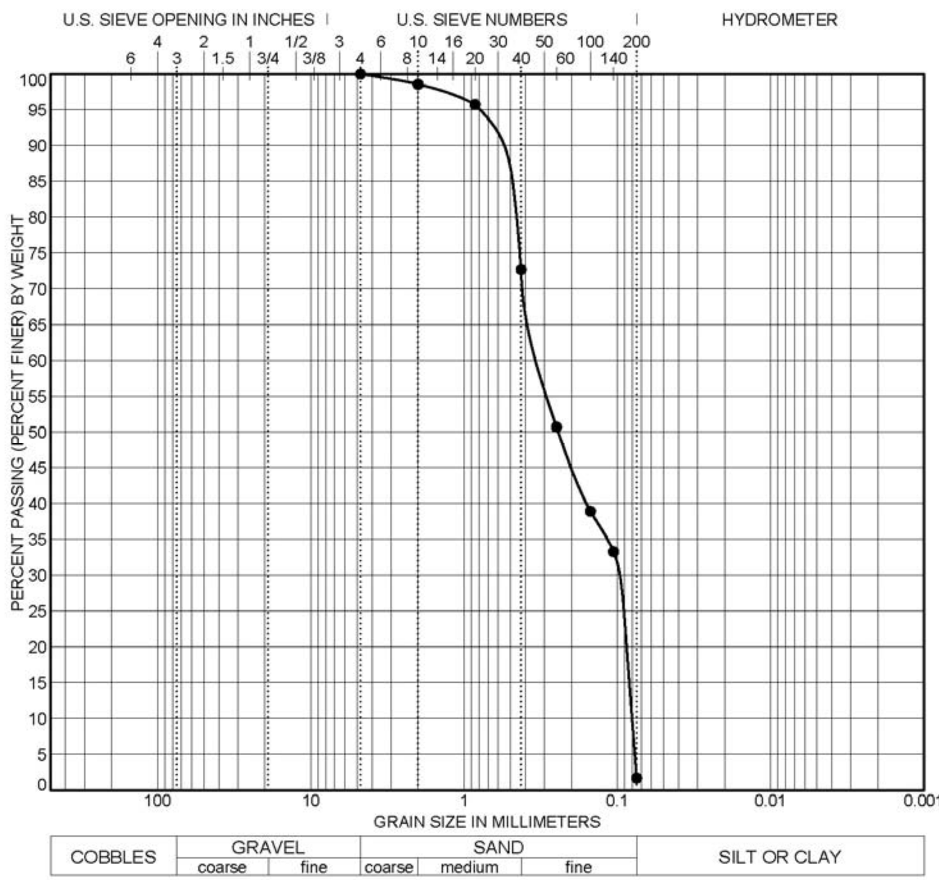


Figure 5-3. Grain size distribution curve based on data in Figure 5-2.

Particle size testing is relatively straightforward, but the results can be misleading if procedures are not performed correctly and/or if equipment is not maintained in good working condition. If the sieve screen is distorted, large particles may be able to pass through sieve openings that typically would retain the particles. Material lodged within the sieve from previous tests could become dislodged during shaking, thereby increasing the weight of material retained on the following sieve. Therefore, sieves should be cleaned thoroughly after each test. A wire brush may distort finer sieve meshes during cleaning, so a plastic brush should be used to clean the U.S. No. 40 (0.425 mm) sieve and finer. Openings of fine mesh No. 200 sieve (0.075 mm)) are easily distorted as a result of normal handling and use. Therefore, fine-mesh sieves should be replaced often. A simple way to determine whether sieves should be replaced is to examine the stretch of the sieve fabric on its frame periodically. The fabric should remain taut; if it sags, it has been distorted and should be replaced. A common cause of serious errors is the use of "dirty" sieves. Some soil particles, because of their shape, size or adhesion characteristics, have a tendency to lodge in the sieve openings. This is especially true of the fine mesh sieves.

Representative samples of fine-grained soils (i.e., samples containing more than 50% of particles with diameter less than the U.S. No. 200 sieve size (075 mm) should not be oven dried prior to testing because some particles may cement together leading to a calculated lower fines content from mechanical sieve analyses than is actually present. When fine-grained particles are a concern, the wash sieve method (ASTM D 1140) should be performed to assess the fines content.

If the clay-size content is an important parameter, hydrometer analyses should be performed even though the hydrometer test provides only approximate results due to oversimplified assumptions. However, the results can still be used as a general index of silt and clay-size content. Depending upon the chemical makeup of the fine grained particles, the traditional sodium hexametaphosphate solution used to disperse the clay-size particles may not provide adequate dispersion. If the clay-size particles are not dispersed, the hydrometer data leads to the interpretation of a lower than actual clay-size content. In some cases the concentration of the dispersing agent may need to be increased or a different dispersing agent may need to be used. If the sieve and hydrometer analyses are performed correctly, the gradation curve should be continuous over a range that includes all particle sizes.

5.3.4.1 Sand Equivalent

The sand equivalent test is a rapid test to show the relative proportions of fine dust or claylike materials in aggregate (or soils). A sample of aggregate passing the No. 4 sieve (4.75-mm) sieve and a small amount of flocculating solution are poured into a graduated cylinder and are agitated to loosen the claylike coatings from the sand particles. The sample is then irrigated with additional flocculation solution forcing the claylike material into suspension above the sand. After a prescribed sedimentation period, the height of flocculated clay and height of sand are determined. The sand equivalent is determined from the ratio of the height of the sand to height of the clay and expressed as a percentage. Cleaner aggregates will have higher sand equivalent values. For asphalt pavements, agencies often specify a minimum sand equivalent around 25 to 35 (Roberts, *et al.*, 1996). Higher values are used in case of compacted structural fill which may support structures (see Section 8.6).

5.3.5 Atterberg Limits

The Atterberg limits of a fine grained soil represent the moisture content at which the physical state of the soil changes. The tests for the Atterberg limits are referred to as index tests because they serve as an indication of several physical properties of the soil, including strength, permeability, compressibility, and shrink/swell potential. These limits also provide a relative indication of the plasticity of the soil, where plasticity refers to the ability of a silt or clay to retain water without changing state from a semi-solid to a viscous liquid. In geotechnical engineering practice, the Atterberg limits generally refer to the liquid limit (LL), plastic limit (PL), and shrinkage limit (SL). The limits were defined and discussed in Chapter 2. In this chapter the definition is extended further in terms of quantifiable parameters that permit their measurements in the laboratory. These quantifiable definitions are as follows:

- <u>Liquid Limit (LL)</u> - This limit represents the moisture content at which any increase in moisture content will cause a plastic soil to behave as a viscous liquid. The LL is defined as the moisture content at which a standard groove cut in a remolded sample will close over a distance of ½-inch (13 mm) at 25 blows of the liquid limit device (Figure 5-4). The test is performed on material passing a US Standard No. 40 sieve (0.425 mm). During the test the material is brought to various moisture contents, usually by adding water. The plot of moisture contents vs. blows required to close the groove is called a "flow curve" and the value of the liquid limit moisture content is obtained from the flow curve at 25 blows.

Figure 5-4. Some of the equipment used for Atterberg limits testing of soil.

- Plastic Limit (PL) - This limit represents the moisture content at which the transition between the plastic and semisolid state of a soil occurs. The PL is defined as the moisture content at which a thread of soil just crumbles when it is carefully rolled out by hand to a diameter of 1/8-inch (3 mm).

- Shrinkage Limit (SL) – This limit represents the moisture content corresponding to the change between the semisolid to solid state of the soil. The SL is also defined as the moisture content at which any further reduction in moisture content will not result in a decrease in the volume of the soil.

Based on the above index values, there are two useful related indices, namely, the Plasticity Index (PI) and the Liquidity Index (LI), which were defined in Chapter 2 as follows:

$$PI = LL - PL \qquad 2\text{-}11$$

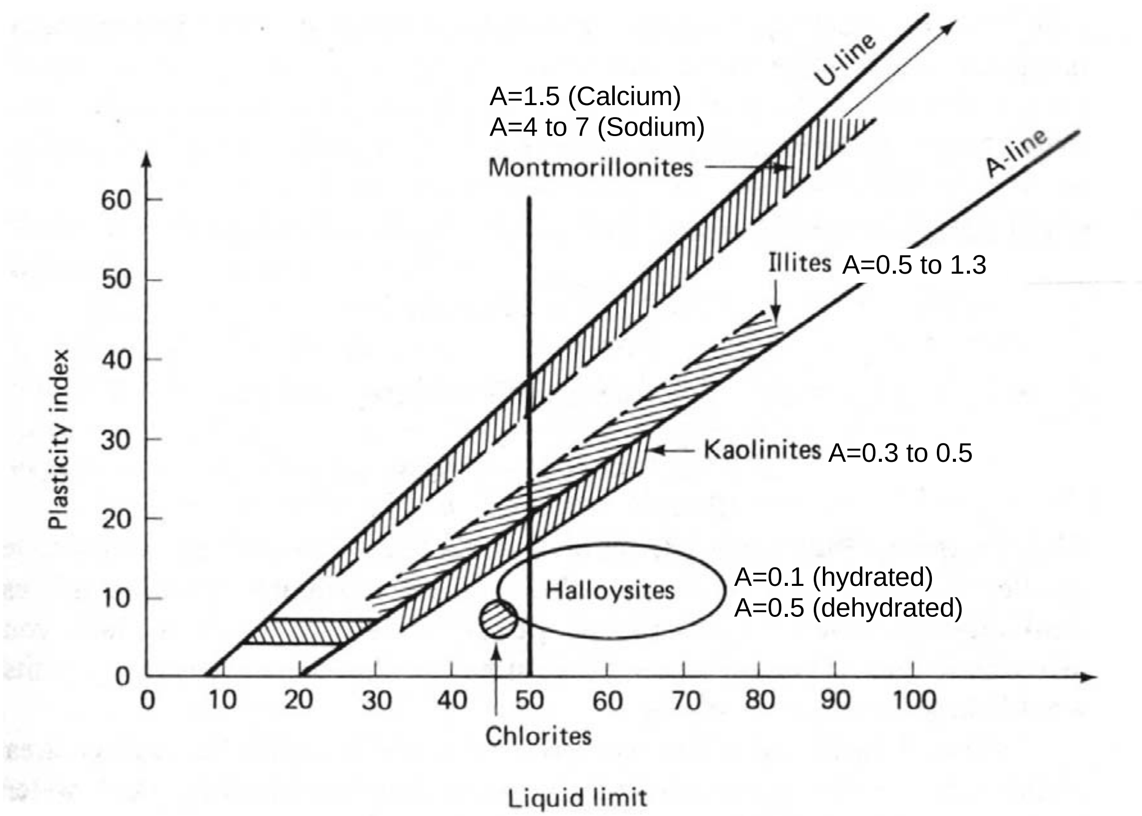
$$LI = \frac{w - PL}{PI} \qquad 2\text{-}12$$

where w is the natural (in-situ) water content of the soil. Numerous engineering correlations have been developed that relate PI and LI to clay soil properties, including undrained and drained strength to PI and compression index to LI.

Another useful index proposed by Skempton (1953) based on the proportion of clay and PI is known as the "Activity Index." The activity index of a clay soil is denoted by A and is generally defined as follows:

$$A = \frac{PI}{CF} \qquad 5\text{-}3$$

where CF is the clay fraction is usually taken as the percentage by weight of the soil with a particle size less than 0.002 mm. Clays with 0.75 < A < 1.25 are classified as "normal" clays while those with A < 0.75 are "inactive" and A > 1.25 are "active." Values of activity index, A, can be correlated to the type of clay mineral that, in turn, provides important information relative to the expected behavior of a clay soil. A clay soil that consists predominantly of the clay mineral montmorillonite behaves very differently from a clay soil composed predominantly of kaolinite. Figure 5-5 also shows the activities of various clay minerals and their location on the Casagrande's plasticity chart. The symbol for the activity index (A) in Figure 5-5 should not be confused with the "A-line" also shown in the figure.



Activity Index, A, of other minerals			
Attapulgite	0.5-1.2	Mica (muscovite)	0.2
Allophane	0.5-1.2	Quartz	0
Calcite	0.2		

Figure 5-5. Location of clay minerals on the Casagrande Plasticity Chart and Activity Index values (after Skempton, 1953, Mitchell, 1976, Holtz and Kovacs, 1981).

Modified Activity Index, A_m: Based on their studies regarding the swell potential of compacted natural and artificial clay soils, Seed *et al.* (1962) proposed that for natural clay soils compacted as per the requirements of ASTM D 698 and Atterberg limits determined by ASTM D 4318 (AASHTO T 89, T 90), a Modified Activity Index, A_m, defined as follows is more appropriate:

$$A_m = \frac{PI}{CF - 5} \tag{5-4}$$

The above definition is used to define the swell potential of soils (see Section 5.7).

5.3.5.1 Significance of the "A-line" and "U-line" on Plasticity Chart

As shown in Figure 4-3 in Chapter 4, the equation for the A-line and U-line are:

$$A - \text{line}: PI = 0.73\,(LL - 20) \tag{5-5}$$

$$U - \text{line}: PI = 0.9\,(LL - 8) \tag{5-6}$$

The A-line generally separates soils whose behavior is more claylike (points plotting above the A-Line) from those that exhibit a behavior more characteristic of silt (points plotting below the A-line). The A-line also separates organic (below) from inorganic (above) soils. The LL = 50 line generally represents the dividing line between silt, clay and organic fractions of the soil that exhibit low plasticity (LL<50) and high plasticity (LL>50). The U-line shown in Figure 5-5 represents the upper range of PI and LL coordinates that have been found for soils. When the limits of any soil plot above the U-line, the results should be considered spurious and the tests should be rerun. Note that in Figure 5-5 the clay mineral montmorillonite plots well above the A-line and just below the U-line. If a soil plots in this range, it probably contains a significant amount of the clay mineral montmorillonite that expands in presence of water.

5.3.6 Specific Gravity

The specific gravity of solids (G_s) is a measure of solid particle density and is referenced to an equivalent volume of water. Specific gravity of solids is defined as $G_s = (M_s/V_s)/\ \rho_d$ where M_s is the mass of the soil solids and V_s is the volume of the soil solids and ρ_d is the mass density of water = 1,000 kg/m^3 or 1 Mg/m^3. This formulation represents the theoretically correct definition of specific gravity and can be rewritten as $G_s = \rho_s\ /\ \rho_d$.

However, since $\gamma = \rho g$ the gravitational constant appears in both the numerator and denominator of the expression and the equation for G_s can also be given as $G_s = \gamma_s/\gamma_w$ where γ_s = unit weight of solid particles in the soil mass and γ_w = unit weight of water = 62.4 pcf (1,000 kg/m^3 or 1 Mg/m^3).

The typical values of specific gravity of most soils lie within the narrow range of $G_s = 2.7 \pm 0.1$. Exceptions include soils with appreciable organics (e.g., peat), ores (e.g., mine tailings), or calcareous (high calcium carbonate content) constituents (e.g., caliche). It is common to assume a reasonable G_s value within the range listed above for preliminary calculations. Laboratory testing by AASHTO T100 or ASTM D 854 or D 5550 can be used to confirm the magnitude of G_s, particularly on projects where little previous experience exists and unusually low or high unit weights are measured.

5.3.7 Organic Content

A visual assessment of organic materials may be very misleading in terms of engineering analysis. Laboratory test method AASHTO T194 or ASTM D 2974 should be used to evaluate the percentage of organic material in a specimen where the presence of organic material is suspected based on field information or from previous experience at a site. The test involves weighing and heating a previously dried sample to a temperature of 824°F (440°C) and holding this temperature until no further change in weight occurs. At this temperature, the organics in the sample turn to ash and the sample is re-weighed. Therefore, with the assumption that the weight of the ash is negligible, the percentage of organic matter is the ratio of the difference in weight before and after heating the sample to 824°F (440°C) to the weight of the original dried sample. The sample used for the test can be a previously dried sample from a moisture content evaluation. Usually organic soils can be distinguished from inorganic soils by their characteristic odor and their dark gray to black color. In doubtful cases, the liquid limit should be determined for an oven-dried sample (i.e., dry preparation method) and for a sample that is not pre-dried before testing (i.e., wet preparation method). If drying decreases the value of the liquid limit by about 30 percent or more, the soil may usually be classified as organic (Terzaghi, *et al.*, 1996).

Soils with relatively high organic contents have the ability to retain water. Water retention may result in higher moisture content, higher primary and secondary compressibility, and potentially higher corrosion potential. Organic soils may or may not be relatively weak depending on the nature of the organic material. Highly organic fibrous peats can exhibit high strengths despite having a very high compressibility. In some instances such soils may even exhibit tensile strength.

5.3.8 Electro Chemical Classification Tests

Electro chemical classification tests provide the geotechnical specialist with quantitative information related to the aggressiveness of the soil conditions with respect to corrosion and the potential for deterioration of typical foundation materials. Electro chemical tests include determination of pH, resistivity, sulfate ion content, sulfides, and chloride ion content. Depending on the application, limits of these electro chemical properties are established based on various factors such as corrosion rates for metals and disintegration rates for concrete. Tests to characterize the aggressiveness of a soil environment are important for design applications that include metallic elements, especially for ground anchors comprised of high strength steel and for metallic reinforcements in mechanically stabilized earth walls. ASTM and AASHTO test procedures are listed under "Corrosivity (Electrochemical)" in Table 5-1.

5.3.9 Laboratory Classification

In addition to field identification (ASTM D 2488), soils should be classified in the laboratory by using the Unified Soil Classification System (USCS) in accordance with ASTM D 2487 or by the AASHTO soil classification system in accordance with AASHTO T 145. These two systems were discussed in Chapter 4. The USCS will be used throughout the remainder of this document. Classification in the laboratory occurs in a controlled environment and more time can be spent on this classification than the identification exercise performed in the field. Laboratory and/or field identification is also important so that defects and features of the soil can be recorded that would not typically be noticed from index testing or standard classification. Some of the features include degree of calcium carbonate cementation, mica content, joints, and fractures.

5.4 CONSOLIDATION TESTING

5.4.1 Process of Consolidation

As discussed in Chapter 2, consolidation is a time-dependent decrease in the volume of a soil mass under applied loading. In highway design, static loading is represented by the permanent load placed on the soil by embankments and structures. Depending on the configuration of the load and the subsurface conditions, the stress increase due to the externally applied loads may extend below the water table where all the voids are filled with water. An applied load will cause the soil grains to readjust to a more compact position to carry the load. This readjustment cannot take place until the water, which is incompressible, escapes from the voids.

As discussed in Chapter 2, the rate of the readjustment of the soil particles is a function of the void size, which controls the rate at which the water can escape from the voids. The settlement associated with the readjustment of the soil particles due to migration of water out of the voids is known as **primary consolidation**.

The amount of primary consolidation will depend on the initial void ratio of the soil. The greater the initial void ratio, the more water that can be squeezed out, and the greater the primary consolidation. The rate at which primary consolidation occurs is dependent on the rate at which the water is squeezed out of the soil voids. **Secondary compression** occurs after primary consolidation is complete. Secondary compression occurs under constant load. It is caused by the soil particles reorienting or deforming under constant load at a very slow rate. This process is known as "**creep**" and it occurs in most soils when they are subjected to long-term applied loads. Therefore, secondary compression is also a time-dependent process. However, secondary compression is not dependent on water being squeezed out of the soil as is consolidation. That is why it is called "secondary compression" and not "secondary consolidation." Primary consolidation accounts for the major portion of settlement in saturated fine-grained soils. Primary consolidation and secondary compression both contribute significantly to settlements in organic soil.

Some natural deposits of fine-grained soils experienced compression in geologic history due to the weight of glaciers, due to the weight of overlying soil that has been eroded, or due to desiccation. Since their void ratios were substantially reduced in the past by these processes, these soils are less compressible today. Such soils are called "**preconsolidated**" or "**overconsolidated**" since they have been subjected to greater stresses in the past than exist at present. This concept is important because overconsolidated soils can be reloaded such as by the load from an embankment or bridge substructure without settling appreciably until the currently applied load exceeds the preconsolidation load. Saturated fine-grained soil deposits, which have never consolidated under loads other than the current loads, are called "**normally consolidated**." On the other end of the spectrum, soils whose present loading induces stresses in the soil that are greater than the maximum effective stress they have experienced in the past are called "**under consolidated.**" This means that the consolidation process under the existing loading is on-going and the soil will continue to consolidate until that process is complete, even if no additional loads are applied.

5.4.2 Consolidation Testing

To predict the amount of consolidation in saturated fine-grained and organic soils, adequate testing must be performed. An undisturbed soil sample should be obtained in the field with a Shelby tube sampler. The oedometer or one-dimensional consolidometer is the primary laboratory equipment used to evaluate consolidation and settlement potential of fine-grained soils. A consolidation test is typically performed on a specimen obtained from an undisturbed sample retrieved from the deposit of fine-grained soils to evaluate the consolidation characteristics of the soil and define the settlement-time relationship of the in-situ soils under proposed foundation loads. The equipment for a consolidation test includes:

1. A loading device that applies a vertical load to the soil specimen,

2. A metal ring (fixed or free) that laterally confines the soil specimen and restricts deformation to the vertical direction only (i.e., only one-dimensional compression is modeled),

3. Porous discs placed on the top and bottom of the sample to allow the sample to drain,

4. A dial indicator or linear variable differential transducer (LVDT) to measure vertical displacement. Properly calibrated, each device should provide the same accuracy, but the electronic output of an LVDT can be incorporated into an automated recording system for quicker, more efficient, and higher resolution readings.

5. A timer to assess the duration of loading increments. Monitoring of time for manual systems can be accommodated by use of a wall clock with a second hand. The internal clock of a computer is used for automated systems

6. A surrounding container to permit the specimen to remain submerged during the test.

Figure 5-6 shows a schematic of a consolidation test. The consolidation-loading device may be a weighted lever arm as shown in Figure 5-7b, a pneumatic device, or an automated loading frame as shown in Figure 5-7c. Automated loading frames are recommended for use in production testing because they provide the most flexibility in testing options. The pneumatic device provides flexibility in loads and load increment ratios (LIR) that can be applied during testing. A weighted lever arm provides a robust, relatively simple system for consolidation testing, however, because data are generally recorded manually, it is difficult to expedite testing or vary the loading schedule since data reduction cannot typically be performed in real time.

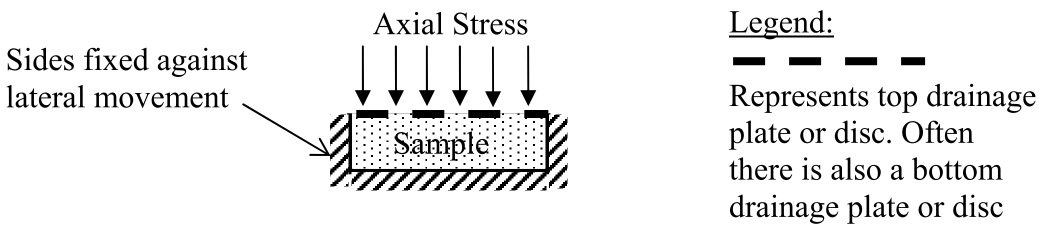


Figure 5-6. Schematic of a consolidation test.

Consolidation cells may be either fixed ring or floating ring. Friction and drag are created in the ring as the specimen compresses in relation to the ring. In a fixed ring test the sample compresses from the top only, potentially resulting in high incremental side shear forces. In a floating ring test the sample compresses from the top and bottom thus providing the advantage of minimizing drag forces. However, the floating ring method has the following disadvantages: it is more difficult to set up; it has the potential for sidewall leakage that would result in an inaccurate assessment of the rate of consolidation, and soil may squeeze out near the junction of the sidewall and the bottom porous disc. Because of these disadvantages, the fixed ring method is most commonly used.

5.4.3 Procedures

The consolidation properties of fine-grained soils are evaluated in the laboratory by using the one-dimensional consolidation test. The most common laboratory method is the incremental load (IL) method (ASTM D 2435). The weighted lever arm oedometer shown in Figure 5-7b is commonly used for performing the procedure. The automated load-frame apparatus shown in Figure 5-7c provides higher quality test results compared to the weighted lever apparatus. High-quality undisturbed samples obtained by using Shelby tubes (ASTM D 1587), piston samplers, or other special samplers are preferred for laboratory consolidation tests.

(a) (b)

(c)

Figure 5-7. (a) Components of consolidation test equipment, (b) Weighted lever arm - incremental load consolidation apparatus, (c) Automated load-frame and computerized consolidation apparatus (Photographs courtesy of GeoComp Corporation).

5.4.4 Presentation and Understanding the Consolidation Test Results

The consolidation test should be run in such a way that sufficient time is allowed for the applied pressure (total stress) increment to be transmitted from the pore water, where it acts initially as a excess pore water pressure, to the soil structure where it ultimately becomes an applied effective stress increment. The time it takes for this transfer to occur is the basis for the process being called "consolidation" and not "compression." Therefore, the effective stress corresponding to the applied pressure is generally plotted versus void ratio. The resulting "**consolidation curve**" permits an evaluation of the preconsolidation pressure and values for other parameters pertaining to the consolidation characteristics of the soil sample.

Plots of void ratio versus effective pressure on arithmetic and logarithmic scales are shown in Figure 5-8. The semi log plot is more widely used in practice and will be used in subsequent sections of this manual. The consolidation curve on the void ratio versus semi log pressure plot is commonly referred to as the "**e-log p**" relationship. As discussed in detail in Chapter 7 and as shown on Figure 5-8, the slope of the loading portion of the e-lop p curve is called the compression index, which is denoted by the symbol C_c. The slope of the re-load portion of the e-log p curve is called the re-compression index; it is denoted by the symbol C_r.

Some geotechnical specialists prefer to use a plot of percent strain versus log of pressure instead of the e-log p plot. In this case the slope of the virgin compression portion of the consolidation curve is called the modified compression index denoted by the symbol $C_{c\varepsilon}$ and the slope of the rebound portion of the curve is called the modified recompression index denoted by the symbol $C_{r\varepsilon}$. The modified indices reflect the relationship between strain and void ratio, i.e., strain (ε) = $\Delta e/(1+e_o)$. Therefore, to convert the strain-based indices ($C_{c\varepsilon}$ and $C_{r\varepsilon}$) to the void-ratio-based indices (C_c and C_r) multiply the strain based values by $(1 + e_o)$. Void-ratio-based values (e-log p) will be used in the remainder of this manual.

Analysis of consolidation test data allows the engineer to determine:

1. **<u>Initial Void Ratio</u> (e_o)**

   The value of the initial void ratio is very important because it defines the amount of void space at the start of the loading. It is this initial void space that will be reduced as the water is squeezed out of the voids with time. The initial void ratio e_o is a key parameter used in settlement computations to determine the magnitude of settlement.

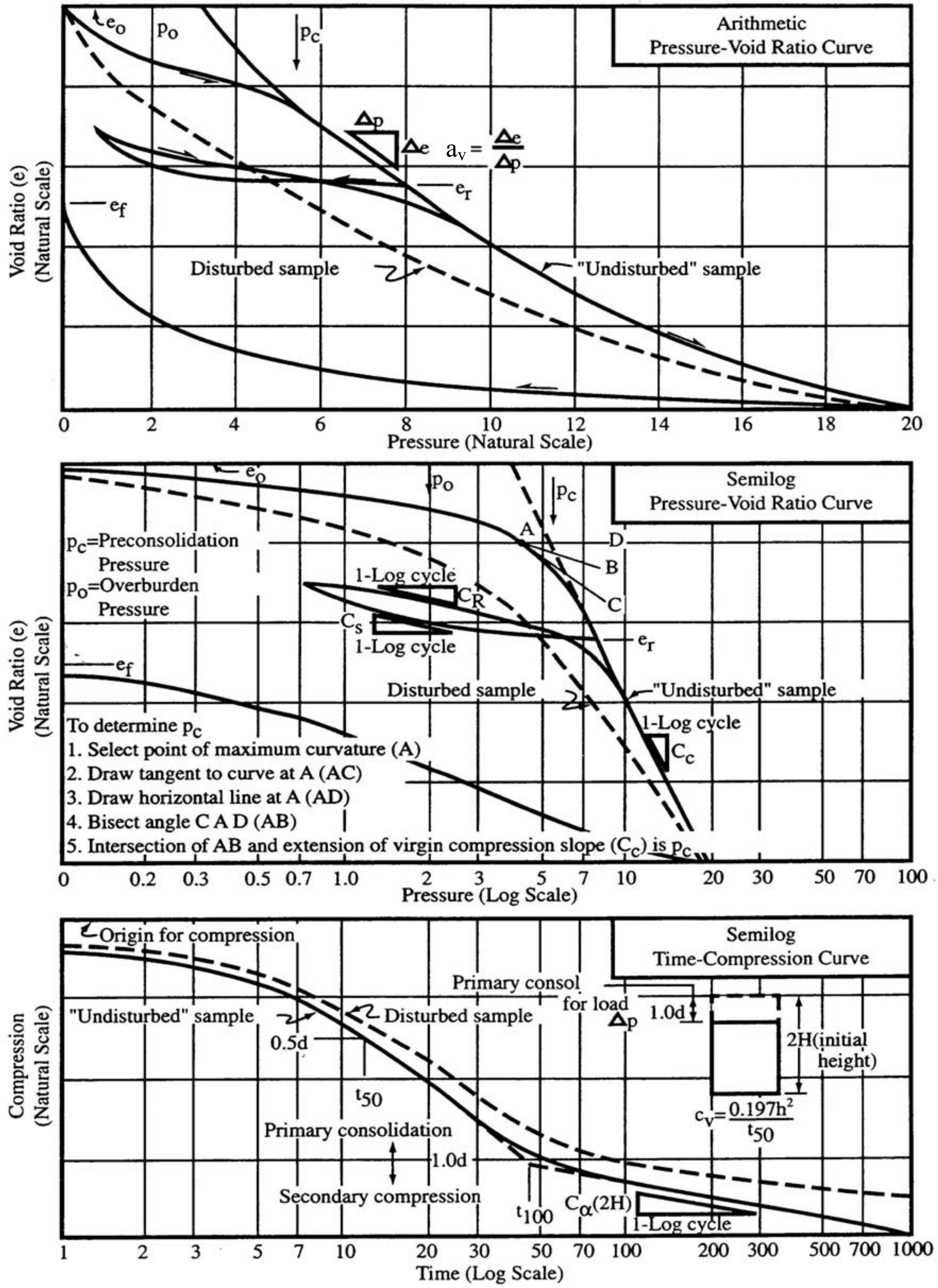


Figure 5–8. Consolidation test relationships (after NAVFAC, 1986a).

2. **<u>Preconsolidation Pressure</u> (p_c)**

The e-log p relationship generally displays a distinct break at approximately the maximum past effective stress (p_c). The graphical technique developed by Casagrande (1936) is generally used to determine the value of p_c, which is known as preconsolidation pressure. The Casagrande procedure is included in the middle portion of Figure 5-8 and is discussed in detail in Chapter 7.

The maximum effective stress to which a soil has been loaded in the past will have a major influence on the amount of settlement to be expected under a proposed loading. In fact, 10 times more settlement may occur in a normally consolidated soil than a preconsolidated soil for equal load increments up to the preconsolidation pressure. Values of preconsolidation pressure should be carefully established for the entire depth of the fine-grained soil deposit under consideration. Normally, a minimum, maximum and most probable value of p_c will be determined from laboratory test results and plotted as a range with depth.

3. **<u>Compression Index</u> (C_c)**

The slope of the consolidation curve beyond p_c is called the compression index (C_c). It is a measure of the load-deformation characteristic of the soil during "virgin" compression.

4. **<u>Recompression Index</u> (C_r)**

An unload/reload segment of the consolidation curve is also shown in Figure 5-8. The slope of the reload curve is called the recompression index (C_r). It is a measure of the load-deformation characteristic of the soil upon reloading after some amount of load release. As is obvious in Figure 5-8, the slope of the reload portion of the consolidation curve is not as steep as the slope of the virgin portion of the curve since the void ratio change accompanying the virgin loading is unrecoverable. Figure 5-8 also shows that if, upon reloading, the applied pressure exceeds the pressure from which the soil was unloaded, the slope of the reload curve reverts back to the virgin compression slope, C_c. In general, $C_c \approx 10\ C_r$.

5. **<u>Coefficient of Consolidation</u> (c_v)**

The coefficient of consolidation is an indicator of the rate of drainage during consolidation. The value may be determined by the t_{50} (log time) method or the t_{90} (square root of time) method. Both of these methods are described in Chapter 7 (Approach Roadway Deformations). As shown in the bottom portion of Figure 5-8, the compression-log time curve for a given load increment is used to determine the coefficient of consolidation (c_v), which is a measure of the time rate of primary consolidation. The value of c_v is determined for each load increment. These values are sometimes plotted on a separate axis below the consolidation curve .

6. **<u>Secondary Compression Index</u> (C_α)**

Of great importance in organic materials, secondary compression may account for the majority of settlement that takes place over a long period of time in such soils. The compression-log time curve for a given load increment is used to determine the secondary compression index (C_α), which is basically the slope of the curve over one log cycle beyond the time required for primary consolidation (t_{100}) as shown in the bottom portion of Figure 5-8.

7. **<u>Effects of Sample Disturbance on Consolidation Test Results</u>**

The influence of sample disturbance on consolidation test results is shown on Figure 5-8 by the dashed lines. The dash lines indicate that disturbance:

a. Eliminates the distinct break in the e-log p curve at the preconsolidation pressure (p_c).

b. Lowers the estimated value of the preconsolidation pressure (p_c) and the measured value of the compression index (C_c).

c. Decreases the measured values of c_v.

d. Increases the recompression index (C_r).

e. Decreases the secondary compression index (C_α).

The general effects of disturbance are (a) under- or over-prediction of the magnitude of expected settlement and (b) over-prediction of the time for its occurrence.

The importance of the consolidation test results as applied to design is summarized below. The test results may be applied to project design after a series of tests have been completed to represent the total depth of the fine-grained soil deposit. The two most important predictions are:

1. **The amount of settlement**. The value is determined by analyzing the consolidation curve between the existing overburden pressure and the final pressure induced by the highway load at various depths. The amount of settlement may vary dramatically depending upon the maximum past pressure to which the soil has been loaded. The total amount of long-term settlement should include an estimate of settlement due to secondary compression, especially for times past the time for 100% primary consolidation if that is less than the design life of the constructed facility.

2. **The time for settlement**. The time for primary consolidation to occur may be estimated from the results of the compression versus time plots at loads between the overburden pressure and final pressure induced by the applied load. The important factors in the settlement-time relationship are:

   (a) Time required is proportional to the square of the longest distance required for water to drain from the deposit. This distance is the thickness of the layer if water drains in one direction only (generally vertically upward to the surface), and one-half the layer thickness if more permeable soils exist above and below the consolidating layer.

   (b) Time required for consolidation varies inversely with the coefficient of consolidation.

   (c) Rate of settlement decreases as time increases.

Settlement computations based on consolidation test results are demonstrated in Chapter 7 (Approach Roadway Deformations).

5.4.5 Comments on the Consolidation Tests

The consolidation test results are necessary to assess the consolidation properties of the soil. As will be shown in subsequent sections of this document, the consolidation test is one of the most important tests for fine-grained soil as it provides data regarding stress history and compressibility. It is important to consider all laboratory testing variables and their potential effects on the values of soil properties computed from the test results. Information that will need to be provided to a laboratory for a consolidation test includes the loading schedule (i.e., magnitude and duration of loads). It is important to evaluate the loading schedule to be used, especially the duration of loading since time is required for the applied total stress increment to be transferred from the pore water to the soil structure so that it becomes an effective stress acting on the soil mass. Important issues related to consolidation tests are discussed below.

- **Loading Sequence**: The loading sequence selected for a consolidation test will depend on the type of soil being tested and the particular application being considered for the project (e.g., embankment, shallow foundation). The selection of a loading sequence should never be left to the discretion of the laboratory. As an example, if the clay soil is heavily overconsolidated, it is possible that a laboratory-determined maximum load for the consolidation test will not be sufficient to exceed p_c.

- **Range of Applied Loads**: The range of applied loads for the test should well exceed the effective stresses that are required for settlement analyses. This range should cover the smallest and largest effective stresses anticipated in the field and will depend on depth, foundation loads, and excavations. The anticipated preconsolidation stress should be exceeded by at least a factor of four during the laboratory test. If the preconsolidation stress is not significantly exceeded during the loading schedule, p_c, and C_c (or $C_{c\varepsilon}$) may be underestimated due to specimen disturbance effects.

- **Load Increment Ratio (LIR):** By definition the LIR= $\Delta\sigma/\sigma_{initial}$ where $\Delta\sigma$ is the incremental stress and $\sigma_{initial}$ is the previous stress. A LIR=1 corresponds to a doubling of the vertical stress applied to the specimen at each successive load increment during a consolidation test. A LIR of 1 is commonly used for most tests. Experience with soft sensitive soils suggests that as the stress approaches the value of p_c, a smaller LIR will facilitate a better estimate of p_c. Typically, laboratories provide a unit cost for a consolidation test that may be based on 6 to 8 load increments with a separate cost for each additional increment.

- **Unload-Reload Cycle**: It is recommended that an unload-reload cycle be performed, especially for cases where accurate settlement predictions are required, specifically to obtain a value for C_r. Since most samples will inevitably be somewhat disturbed, a C_r value based on the initial loading of a consolidation test sample will be greater than that for an undisturbed sample, resulting in an overestimation of settlements in the overconsolidated region. A value of C_r based on an unload-reload cycle is more likely to be representative of the actual behavior of the soil in the overconsolidated region.

- **Duration of Load Increment**: The duration of each load increment should be selected to ensure that the sample is approximately 100 percent consolidated prior to application of the next load increment. For relatively low to moderate plasticity silts and clays, durations of 3 to 12 hours will be appropriate for loads in the normally consolidated range. For fibrous organic materials, primary consolidation may be completed in 15 minutes for each load increment. For high plasticity materials, the duration for each load increment may need to be 24 hours or more to ensure complete primary consolidation and to evaluate secondary compression behavior. Conversely, primary consolidation may occur in less than 3 hours for loads less than p_c.

  If the time period is too short for a given load increment (i.e., the sample is not allowed to achieve approximately 100 percent consolidation before the next load increment is applied), then values of C_c may be underestimated and values of c_v may be overestimated. The duration of time required, however, can be optimized by using pneumatic, hydraulic, or electro-mechanical loading systems that include automated loading and data acquisition systems. Continuous deformation versus time measurements and the square root of time method described in Chapter 7 (Approach Embankment Deformations) can be used to estimate the beginning and end of primary consolidation during the test. Once the end of primary consolidation is detected, the system can automatically apply the next load increment. Alternatively, some laboratories can provide real-time deformation versus log time plots to enable the engineer to evaluate whether 100 percent primary consolidation has been achieved.

- **Secondary Compression**: In cases where secondary compression is important (e.g., organic soils), secondary compression should be assessed on the basis of the deformation versus log-time response. The consolidation test for each load increment should be run long enough to establish a linear trend between vertical displacement and log time.

5.4.6 Useful Correlations between Consolidation Parameters and Index Values

This section presents some useful correlations between consolidation parameters and other index values. These correlations can be used by the designer to check the validity of the laboratory tests results or to develop a prediction of the range of values of consolidation parameters that can be expected from yet-to-be performed consolidation tests. **It must be emphasized that predictions based on correlations should never be substituted for proper testing and that any assumptions regarding consolidation parameters should always be verified through testing.**

5.4.6.1 Compression Index, C_c

Over 70 different equations have been published for correlating C_c with the index properties of clays. Table 5-5 lists some of the more useful correlations. Figure 5-9 shows correlations between natural water content and C_c for fine-grained soils, peats and shales **Note that the coordinates in Figure 5-9 are both logarithmic so that values of C_c can vary by as much as a factor of 5 with respect to the average trend line in these empirical correlations. <u>Values of C_c obtained from Table 5-5 or Figure 5-9 should not be used for final design.</u>**

Table 5-5
Correlations for C_c (modified after Holtz and Kovacs, 1981)

Correlation	Soil
C_c=0.156 e_o + 0.0107[1] $C_c \approx 0.5G_s$(PI/100) [2]	All Clays
C_c=0.30 (e_o - 0.27)	Inorganic, silt, silty clay
C_c=0.009 (LL-10) [3]	Clay of medium to slight sensitivity (S_t < 4, LL<100)[4]
C_c=0.0115 w_n [5]	Organic Soils, Peat
C_c=0.75 (e_o - 0.50)	Low plasticity clays
[1] e_o = initial void ratio, [2] PI = Plasticity Index, [3] LL=Liquid Limit, [4] S_t = sensitivity =Undisturbed undrained shear strength/Remolded undrained shear strength (see Table 3-12 in Chapter 3, [5] w_n = natural water content	

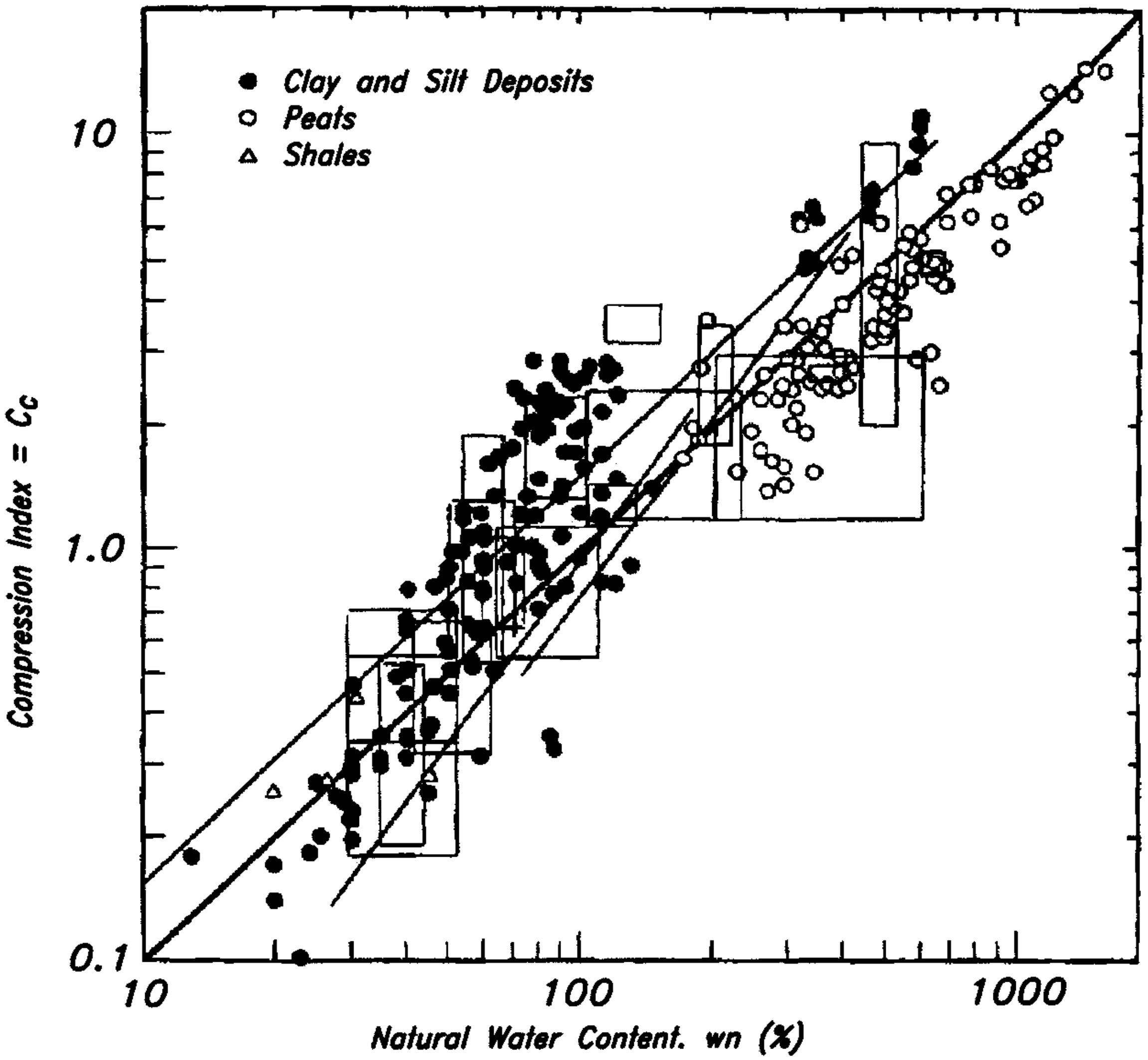


Figure 5-9. Empirical correlation between compression index and natural (in-situ) water content (from Terzaghi, *et al.*, 1996).

5.4.6.2 Recompression Index, C_r

The ratio of C_r / C_c typically ranges from 0.02 to 0.20 (Terzaghi, and Peck, 1967). The low value is typical of highly structured and bonded soft clay or silt, while the largest ratio corresponds to micaceous silts and fissured stiff clays and shales. In reality, the value of C_r depends on whether loading or unloading is occurring, since some hysteresis effects develop when the soil is subjected to cycles of loading and unloading.

Generally, it is sufficiently accurate to assume C_r is constant for most clay deposits. It may not be adequate to rely on a single value of C_r for loading and unloading in the case of highly structured soft clays or stiff clay shales. In the case of highly structured soft clays the initial value of C_r is steep as a result of flocculation (edge to face structure of clays) and bonding that allows the soil to be stable at high void ratios until the stress exceeds the preconsolidation pressure (Terzaghi and Peck, 1967). The subsequent rebound slope can be significantly different from the initial $C_{r.}$

5.4.6.3 Coefficient of Vertical Consolidation, c_v

Because of the wide range of permeabilities that exist in soils (see Table 5-10), the coefficient of consolidation can itself vary widely, from less than 10 ft^2/yr ($\approx$1 m^2/yr) for clays of low permeability to 10,000 ft^2/yr ($\approx$1,000 m^2/yr) or more for very sandy clays, fissured clays and weathered rocks. Some typical values for clays are given in Table 5-6 and an approximate correlation with liquid limit is shown in Figure 5-10.

Just as permeabilities in the horizontal and vertical directions can be significantly different due to variations in soil particle orientation, non-homogeneity, etc., so too can the in situ coefficient of horizontal consolidation, c_h, be much different from the coefficient of vertical consolidation c_v measured in the laboratory for the same reasons. For example, the in situ coefficient of horizontal consolidation, c_h, for clays containing fissures or fine bands of sand, may often be much greater than c_v measured in the laboratory for the clay alone. In such cases, the in-situ c_h may govern the actual rate of consolidation under field loading conditions.

Table 5-6
Typical values of coefficient of vertical consolidation, c_v
(after Carter and Bentley, 1991)

Soil	**c_v**		
	($cm^2/s \times 10^{-4}$)	**(m^2/yr)**	**(ft^2/yr)**
Boston Blue Clay (CL)	40±20	12±6	135±70
Organic silt (OH)	2-10	0.6-3	7-34
Glacial lake clays (CL)	6.5-8.7	2.0-2.7	22-30
Chicago silty clay (CL)	8.5	2.7	29
Swedish medium sensitive clays (CL-CH)			
1. laboratory	0.5-0.7	0.1-0.2	1.7-2.4
2. field	0.7-3.0	0.2-1.0	2.4-10.2
San Francisco Bay Mud (CL)	2-4	0.6-1.2	6.8-13.6
Mexico City clay (MH)	0.9-1.5	0.3-0.5	3.1-5.1

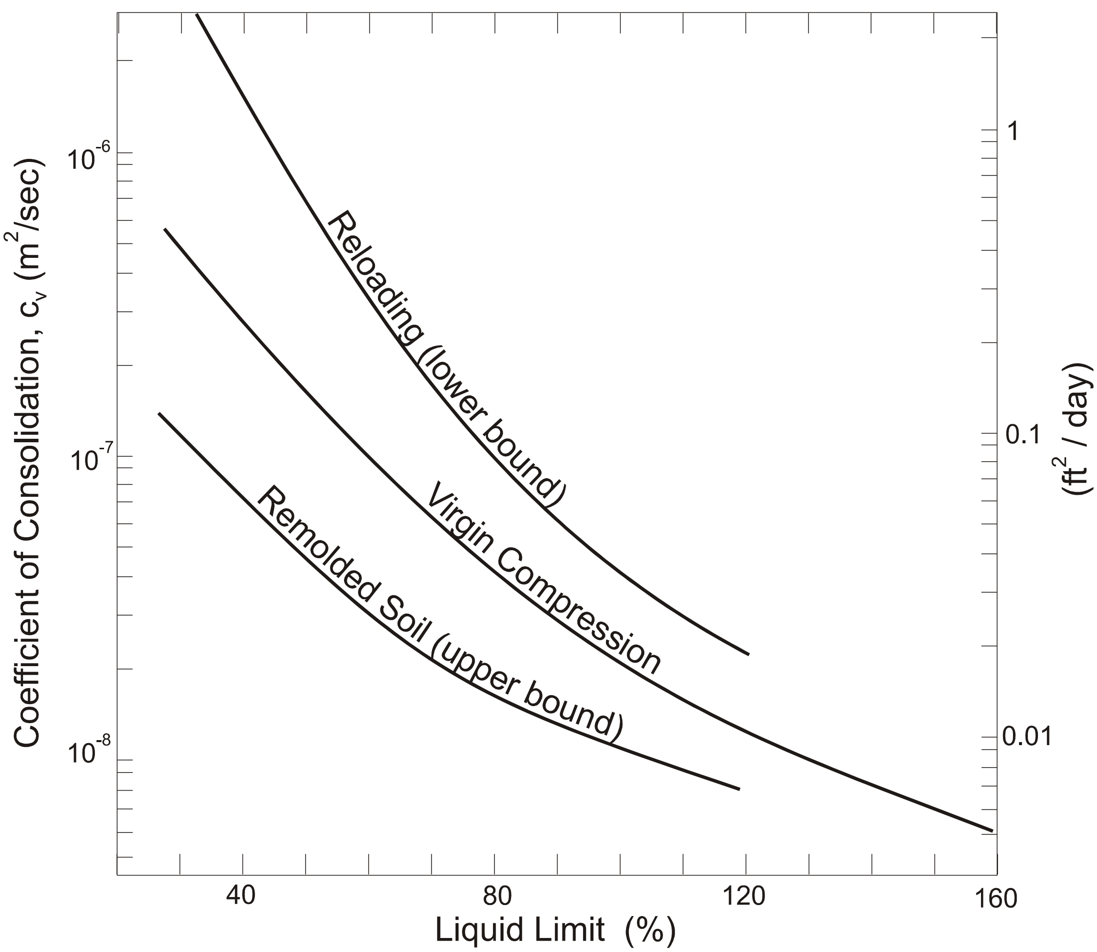


Figure 5-10. Approximate correlations between c_v and LL (NAVFAC, 1986a).

5.4.6.4 Coefficient of Secondary Compression, C_α

Table 5-7 presents typical values of C_α in terms of C_c for various geomaterials. As shown below by Equations 5-7, the coefficient, C_α, may be expressed either in units of strain ($C_{\alpha\varepsilon}$) or void ratio ($C_{\alpha e}$) per log cycle of time. As indicated previously, to convert void-ratio-based consolidation curve indices to strain-based indices divide the void-ratio-based values by (1 + e_o).

$$C_{\alpha\varepsilon} = \frac{d\varepsilon}{d(\log t)}; \qquad C_{\alpha e} = \frac{de}{d(\log t)}; \qquad C_{\alpha\varepsilon} = \frac{C_{\alpha e}}{(1+e_o)} \qquad 5\text{-}7$$

$C_{\alpha e}$ is usually assumed to be related to C_c with values of $C_{\alpha e}/C_c$ typically in the range 0.025-0.006 for inorganic soils and 0.035-0.085 for organic soils. Figure 5-11 presents a correlation between $C_{\alpha e}$ and natural water content.

Table 5-7
Typical Values of $C_{\alpha e}/C_{ce}$ (Terzaghi, *et al.*, 1996)

Soil	C_α/C_c
Granular soils including rockfill	0.02 ± 0.01
Shale and mudstone	0.03 ± 0.01
Inorganic clays and silts	0.04 ± 0.01
Organic clays and silts	0.05 ± 0.01
Peat and muskeg	0.06 ± 0.01

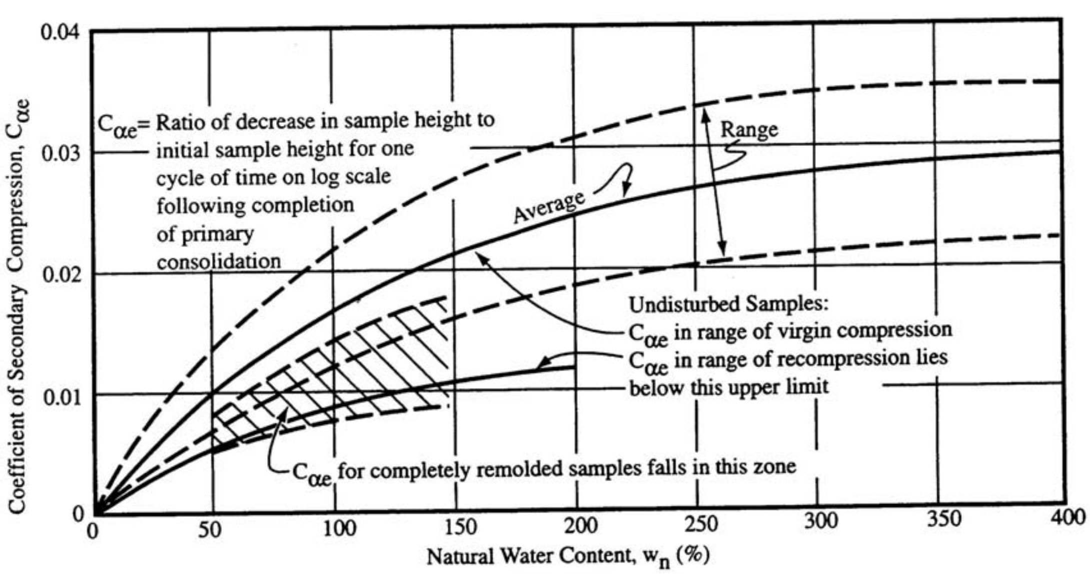


Figure 5-11. Correlation between $C_{\alpha e}$ and natural water content (NAVFAC, 1986a).

5.5 SHEAR STRENGTH OF SOILS

The shear strength of soils is extremely important to foundation design. In addition, slopes of all kinds, including hills, river banks, and man-made cuts and fills, stay in place only because of the shear strength of the material of which they are composed. Knowledge of the shear strength of soil is important for the design of structural foundations, embankments, retaining walls, pavements, and cuts. Table 5-8 provides a summary of specific issues related to the design and construction of typical highway design elements that should be considered in developing and implementing a laboratory and in-situ testing program for evaluating soil shear strength.

5.5.1 Concept of Frictional and Cohesive Strengths

The concept of shear strength was introduced in Chapter 2 where it was shown to be comprised of two components, **friction** **(ϕ)** and **cohesion** **(c)**. In terms of the classification of soils introduced in Chapter 3, these two components of the shear strength can be generalized as follows:

1. Coarse-grained soils, such as gravel and sand, and fine-grained silt, derive strength primarily from friction between particles. Therefore they are considered to be "**cohesionless**" or "**frictional**" soils and are often denoted as "ϕ-soils."

2. Fine-grained soils, composed mainly of clay, derive strength primarily from the electro-chemical attraction, or bond, between particles. Therefore they are considered to be "**cohesive**" soils and are often denoted as "c-soils."

3. Mixtures of cohesionless and cohesive soils derive strength from both interparticle friction and bonding. Such soils are commonly denoted as "**c-ϕ soils**."

5.5.1.1 Strength Due to Friction

The strength due to friction between soil particles is dependent on the stress state of the soil (e.g., overburden pressure) and the angle of internal friction (ϕ) between the particles. As discussed in Chapter 2, the frictional resistance of soil is equal to the normal stress, σ_n, times the tangent of friction angle, ϕ. The tangent of ϕ is equal to the coefficient of friction (μ) between the soil particles.

Table 5-8
Summary of issues relevant to shear strength evaluation in support of the design of typical geotechnical features (after FHWA, 2002a)

Design Element	Issues Relevant to Shear Strength Evaluation
Shallow Foundation	• Soil shear strength information required for depths up to 2 times the width of the footing, unless weak zones are found below this depth. The depth of bottom of footing will be based, in part, on requirements with respect to frost penetration depths and scour depths.
Drilled Shaft	• The excavation of a hole to construct a drilled shaft results in stress relief and disturbance in the soil that ultimately results in a reduction in shear strength from that corresponding to in-situ (i.e., before construction) conditions. The magnitude of the stress relief and disturbance will depend upon the method of construction, soil type, saturation condition, and type of strength (e.g., side shear or end bearing).
Driven Pile	• The shear strength of the soil may vary significantly between the time when the pile (or pile group) is first driven and tested to the time when the superstructure loads are applied to the pile (or pile group). The time-dependent phenomena of strength increase is referred to as "**pile set-up**" and is often observed for driven piles in saturated normally consolidated (NC) to moderately over consolidated (OC) clay and fine-grained material. A decrease in strength with time is referred to as "**relaxation**" and is often observed for heavily OC clays, dense silts, dense fine sands, and weak laminated rock. Shear strengths should therefore be evaluated for both long and short term conditions. • Changes in site conditions that affect the in-situ the effective stress state may increase or decrease shear strength and pile capacity. These may include site dewatering or additional surface loading from an embankment. • An increase in granular soil strength may occur due to densification during driving. This increased strength will need to be considered such that an appropriate pile driving system can be selected for construction.
Retaining Walls	• The analysis of non-gravity cantilevered and anchored walls requires an evaluation of earth pressures on the active side and passive side of the excavation. For undrained loadings in some clayey soils, particularly low to medium plasticity materials, there can be a large difference in undrained strength between the strength used for the active side and the passive side of the excavation. • For soils that may exhibit peak, fully softened, and residual conditions, an estimate of the tolerable deflection of the wall system needs to be made and this deflection used to select the appropriate strength condition for analysis.
Slopes	• The shear strength of discontinuities (e.g., fissures) in soil (and rock) needs to be evaluated since it may represent the critical (i.e., lowest) shear strength for design. • Weathering and other physiochemical reactions may occur at a quick enough rate to weaken soil bonds and reduce shear strength. • Strength loss may occur in cut slopes due to soil softening (in presence of water) and continuing deformations. Large deformation residual strengths should be used for long-term analyses.

The equation for frictional resistance, τ, is written in terms of normal stress, σ_n, as follows:

$$\tau = \sigma_n \tan \phi \tag{5-8}$$

The coefficient of friction, $\tan \phi$, between individual particles depends on both their mineral hardness and surface roughness. However, the measured friction angle of a soil sample or deposit also depends on the density of the mass caused by interlocking of particles. For a detailed discussion of factors affecting frictional resistance, the reader is referred to textbooks such as by Holtz and Kovacs (1981).

5.5.1.2 Strength Due to Cohesion

The concept of cohesive strength is more difficult to explain and often misunderstood. The designer must develop a good understanding of this concept otherwise there will be a disconnect between reality and the design of some structures, e.g., the first bench cut in shoring.

There are two types of cohesion in soils: **true cohesion** and **apparent cohesion**. These are briefly discussed as follows (after Mitchell, 1976):

1. **True cohesion** may result from chemical cementation (just like in rocks) and/or forces of attraction (e.g., electrostatic and electromagnetic attractions) between colloidal (10^{-3} mm to 10^{-6} mm) clay particles. True cohesion is stress-independent unlike frictional resistance that is a function of normal stress.

2. **Apparent cohesion** may develop because of capillary stresses and mechanical interlocking as follows:

   - **Capillary stresses** develop between particles in a partially saturated soil due to surface tension in the water. The surface tension (negative pressure) in the water produces an equal and opposite effective stress between the soil particles, which results in an apparent cohesion since it too is stress-independent. The magnitude of this type of apparent cohesion can be extremely large, especially in fine-grained soils. Such capillary stresses can be overcome by an increase in the degree of saturation.

- **Apparent mechanical forces** are often exhibited by the interlocking of rough (angular) soil particles. The interlock between the soil particles can offer some resistance to shear stresses even in the absence of a normal stress. This type of apparent cohesion is often the cause of cohesion measured in compacted soils. However, such apparent mechanical forces are susceptible to significant reduction by vibrations and other types of mechanical disturbance.

Figure 5-12 presents a graphical representation of the potential contribution of various mechanisms of cohesion. It can be seen that true cohesion in soils exists only when the particle size is colloidal. Unless the complete soil sample is composed of colloidal particles, true cohesion due to interparticle attraction cannot be relied on. Cementation by deposition is often observed in arid environments (e.g., desert southwest), but it is difficult to quantify. As indicated above, capillary stresses can provide a large apparent cohesion, but such cohesion can be overcome by saturation. Since cohesion cannot be defined with confidence, its contribution to long-term shear strength in c-ϕ soils is often disregarded or greatly minimized by using only a small value such as 100 to 500 psf (5 to 25 kPa). For purely cohesive soils, the designer should be careful in evaluating the cohesion for long-term design purposes. Further discussion on apparent cohesion in the context of compacted soils is included in Section 5.8.4.1.

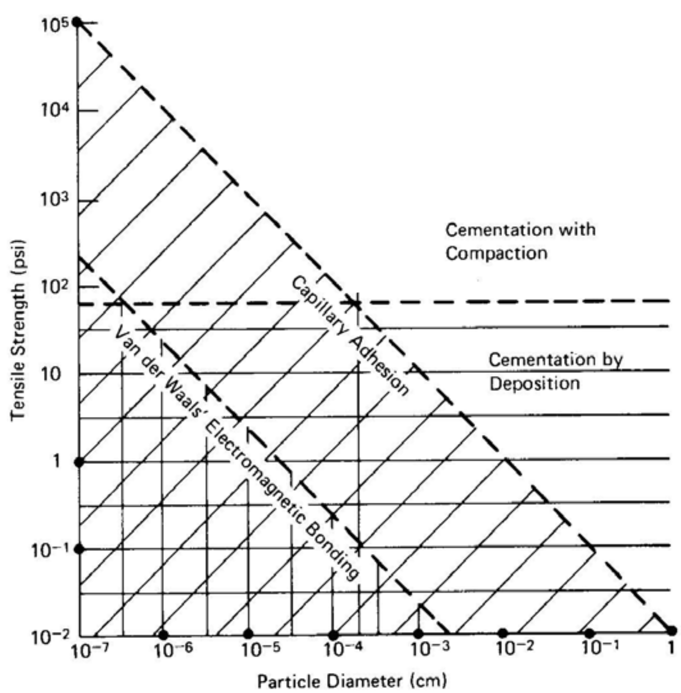


Figure 5-12. Potential contributions of various bonding mechanisms to cohesive strength (after Ingles, 1962).

5.5.1.3 Simplified Expression for Shear Strength of Soils

As indicated in Chapter 2, the shear strength, τ, of soils is expressed simplistically by two additive components as follows:

$$\tau = c + \sigma_n \tan \phi \qquad 5\text{-}9$$

In terms of effective stresses, the effective shear strength, τ', can be re-written as follows:

$$\tau' = c' + (\sigma_n - u) \tan \phi' = c' + \sigma' \tan \phi' \qquad 5\text{-}10$$

where u is the pore water pressure, c' is the effective cohesion, σ' is the effective normal stress, and ϕ' is the effective friction angle.

The shear strength soil is influenced by many factors including the effective stress state, mineralogy, packing arrangement of the soil particles, soil hydraulic conductivity, rate of loading, stress history, sensitivity, and other variables. As a result, **the shear strength of soil is not a unique property**. The following sections present and discuss various laboratory tests to determine the shear strength for various types of construction and loading conditions. Typical laboratory strength tests are introduced including the unconfined compression test (AASHTO T208; ASTM D 2166), the triaxial test (AASHTO T234; ASTM D 4767), and the direct shear test (AASHTO T236; ASTM D 3080). A detailed discussion on testing equipment and procedures is beyond the scope of this document. The interested reader should review the AASHTO and ASTM standards for detailed information on testing equipment and procedures. The following sections also describe information that must be conveyed to a laboratory testing firm to ensure that the strength testing is performed consistent with the requirements imposed by the design (e.g., selection of confining pressures consistent with the imposed loads).

5.5.2 Strength Testing of Soils in the Laboratory

The shear strength of a soil is the maximum shear stress that the soil structure can resist before failure. Failure is generally defined as continuing displacement without an increase in applied stress. Since the water filling the pores has no shear strength, shear stresses are carried by the structure of soil grains. However, the shear strength of the soil structure is indirectly dependent on the pressure in the pore water, which influences the friction term as shown by the excess pore water pressure term, u, in Equation 5-10. Foundation designers must consider the effects of expected construction operations on the subsoils when planning a test program. For example, when a highway embankment or structural footing is suddenly placed on a soft clay deposit, the pore water initially carries all the load and the available shear strength does not increase until drainage begins and the excess pore water pressure decreases. In planning a test program for such a situation the designer should request unconsolidated undrained (UU) triaxial tests to determine the undrained shear strength of the soil which, in this case, would be the critical strength value, i.e., the initial shear strength before consolidation begins. Additional consolidated undrained (CU) or consolidated drained (CD) tests would also be used to determine the increase in shear strength as consolidation occurs and excess pore water pressures dissipate. These results can be used to determine alternate methods of applying the loads safely, especially if the undrained strength is insufficient to sustain the proposed loading. Stage construction involves placement of an increment of load and a waiting period to allow strength gain through excess pore water pressure dissipation so the soil deposit can safely support the next load increment.

The majority of strength tests are conducted on cohesive soils since obtaining undisturbed samples of non-cohesive soils is difficult. Strength tests on cohesive soils are usually conducted on high quality undisturbed samples obtained from thin wall sampling tubes. The preferred test for most projects where cohesive soils are involved is the triaxial compression test. The number and types of tests must be selected by the designer to suit the project conditions. For each test the designer should clearly indicate the consolidation or confining pressure to be used. These pressures are determined from the p_o diagram for each specific project (refer to Chapter 2 for discussion of the p_o diagram). The range usually extends from the effective overburden pressure to the pressure induced by the highway loading. The program objective should be to establish a profile of soil strength with depth. Soil strength parameters are frequently expressed as a ratio of shear strength over the effective overburden pressure (τ/p_o).

The most common laboratory soil strength tests are:

- Unconfined compression test
- Triaxial compression test, and
- Direct shear test

Each of these tests is briefly discussed below. For the triaxial compression and the direct shear tests, it is important that each test be performed on a new sample. **The practice of performing multi-stage shear strength tests on a single sample is not recommended.**

5.5.2.1 Unconfined Compression (UC) Tests

The unconfined compression test is a quick, relatively inexpensive means to obtain an estimation of the undrained shear strength of cohesive specimens. In this test a cylindrical specimen of the soil is loaded axially as shown in Figure 5-13 without any lateral confinement to the specimen, at a sufficiently high rate to prevent drainage. Since there is no confinement, residual negative pore pressures that may exist in the sample following sample preparation generally control the state of effective stress in the sample. The shear stresses induced in the specimen by the axial load result in a shear failure. The magnitude of the shear stress at the moment of failure represents the shear strength of the soil under these conditions of loading and drainage. Therefore, the shear strength obtained from this test is called the "undrained shear strength (s_u)." In most cases, the value of undrained shear strength obtained from an unconfined compression test is conservative. The maximum axial compressive stress measured at failure represents the compressive strength of the soil under these conditions of loading, drainage, and confinement. Therefore, the compressive strength obtained from this test is called the "unconfined compressive strength (q_u)." These two strengths terms should not be confused; one is a shear strength the other a compressive strength. It can be shown graphically by a Mohr's circle construction (Appendix B) that the undrained shear strength (s_u) is equal to one-half the unconfined compressive strength (q_u).

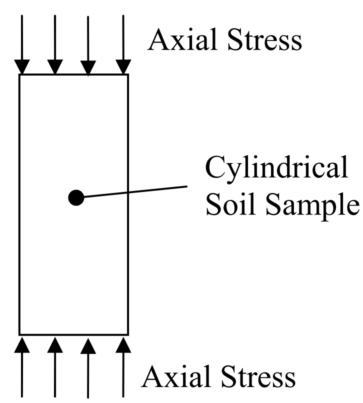


Figure 5-13. Schematic of an unconfined compression test.

The unconfined compression test cannot be performed on granular soils, dry or crumbly soils, silts, peat, or fissured or varved materials. Because there is no control over the effective stress state of the specimen, this test is not recommended for evaluating strength properties for compressible clay soils subjected to embankment or structural foundation loads. The reliability of this test decreases with increasing sampling depth because the sample tends to swell after removal from the ground due to confining stress release. The swelling results in greater particle separation and reduced shear strength. Testing the full diameter extruded specimen as soon as possible after removal from the tube can:

- minimize swelling
- reduce disturbance
- preserve the natural moisture content.

Unfortunately, despite these shortcomings, this test is commonly used in practice because of its simplicity and low cost.

5.5.2.2 Triaxial Tests

The triaxial test is very versatile in the sense that the shear strength can be evaluated under compression as well as extension loading conditions. A schematic of triaxial compression test is shown in Figure 5-14 where the axial stress is greater than the confining stress. Lateral pressures at various depths below the ground surface can be simulated by confining pressures. Note that the confining pressures acts on the entire sample and is equal to the axial stress before the application of an axial stress increment. Typically, failure of the sample is caused by increasing the axial stress (compression) until a shear failure takes place. In an extension test, the confining pressures are increased while keeping the axial stress constant. Pore water pressures during the test can be measured.

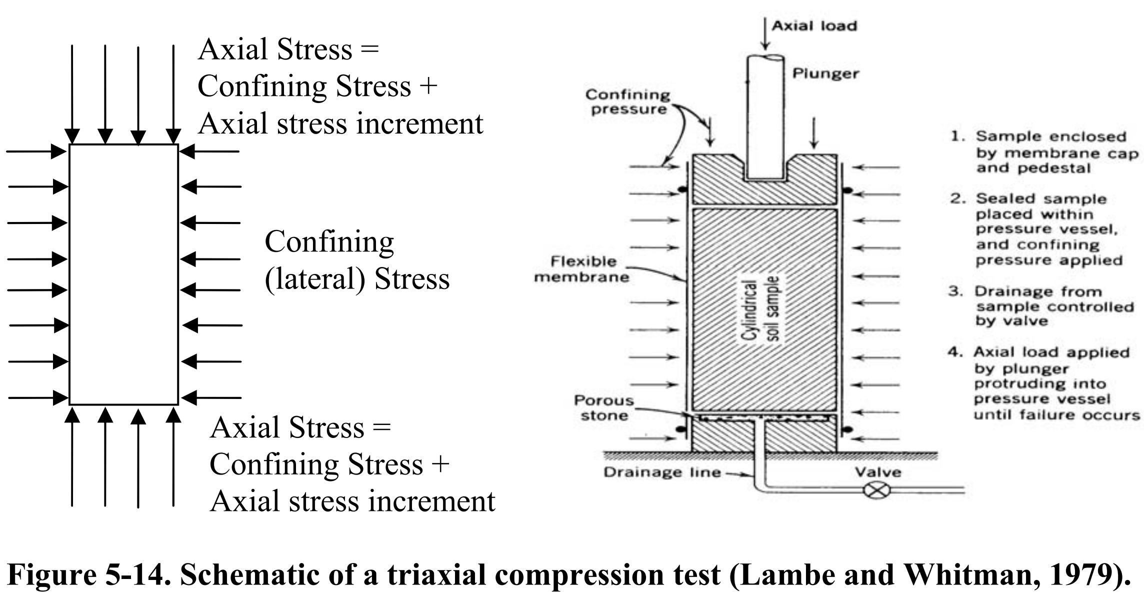
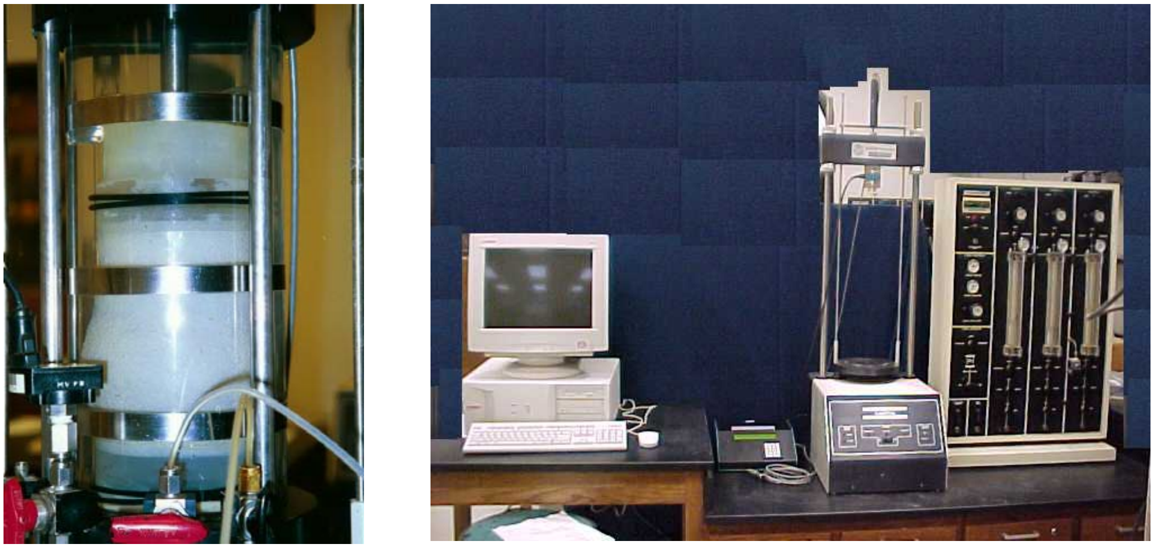


Figure 5-14. Schematic of a triaxial compression test (Lambe and Whitman, 1979).



(a) **(b)**

Figure 5-15. (a) Failure of a loose sand specimen in a triaxial cell; and (b) Load frame, pressure panel, and computerized data acquisition system (Photographs courtesy of GeoComp Corporation).

Equipment – Triaxial systems today use electronic instrumentation to provide continuous monitoring and periodic acquisition of test data (see Figure 5-15b). Force is measured by using a force transducer or load cell that is typically mounted outside the triaxial cell. More advanced systems have the transducer incorporated within the testing cell to reduce the effects of rod-friction. Linear variable differential transducers (LVDTs) are used to monitor deformations. Additionally, volume measurements can be taken with a device that makes use of an LVDT to measure the rise or fall of a bellofram cylinder. This change in movement is calibrated to the volume of water taken in or pushed out of the sample. Pressure transducers are mounted on the base of the test cell to monitor the confining pressure within the cell and the pore water pressure within the sample.

Unconsolidated-Undrained (UU) Test

In the UU test, no drainage or consolidation is allowed during either the application of the confining pressure or the application of the axial load that induces shear stress. The shear stresses induced in the specimen by the axial load result in failure. As indicated in Section 5.5.2, the UU test models the response of a soil that has been subjected to a rapid application of an axial load such as that due to construction of an embankment. It is difficult to obtain repeatable results for UU testing due to the effects of sample disturbance. The accuracy of the UU test is dependent on the soil sample retaining its original structure until testing occurs. The undrained shear strength of the soil, s_u, is measured in this test.

Consolidated-Drained (CD) Test

In the CD test, the specimen is allowed to consolidate completely under the confining pressure prior to the application of axial load, i.e., the confining pressure acts as an effective stress throughout the soil specimen. The axial load is applied at a rate slow enough to allow drainage of pore water so that there is no buildup of excess pore water pressures, i.e., the stresses imposed by the axial load are effective stresses. The shear stresses induced in the specimen by the axial load result in failure. The time required to conduct this test in low permeability soil may be as long as several months. Therefore it is not common to conduct a CD test on low permeability soils. The CD test models the long-term (drained) condition in soil. Effective stress strength parameters (i.e., ϕ' and c') are evaluated from the results of the CD test.

Consolidated-Undrained (CU) Test

The initial part of the CU test is similar to the CD test in that the specimen is allowed to consolidate under the confining pressure. However, unlike the CD test, the axial load is applied with the drainage lines closed in the CU test. Thus, during shearing there is continual development (+ or -) of excess pore water pressure. The rate of axial load application for this test is more rapid than that for a CD test. Pore pressures are typically measured during the CU test so that both total stress and effective stress strength parameters can be obtained. Recall that total stress equals effective stress + pore water pressure as expressed by Equation 2-13. The pore water pressure may be + or – depending upon whether the specimen dilates or compresses during application of the axial load. The shear stresses induced in the specimen by the axial load result in failure. The effective stress parameters evaluated for most soils based on CU testing with excess pore water pressure measurements will be similar to those obtained from CD testing, thus making CD tests unnecessary for typical applications.

During triaxial testing, the confining pressure, which acts uniformly over the entire specimen, is considered to be the minor principal stress. By definition, a principal stress is one that acts on a plane where shear stress is zero. The interface between the soil and the membrane isolating it from the fluid in the chamber is assumed to be frictionless during the entire test, i.e. no shear stresses develop along the circumference of the specimen. Likewise, the applied axial load causes a normal stress to act on the top and bottom of the specimen. As shown in Figure 5-14, this vertical normal stress acts in addition to the confining pressure. Therefore, the combined vertical stress acting on the top and bottom of the specimen during the triaxial test is considered to be the major principal stress not only because the plane (horizontal) on which it acts is orthogonal to the minor principal stress plane (vertical), but mainly because the interface shear stress between the specimen and the top and bottom caps is assumed to be equal to or close to zero. To assure this condition, the end caps are usually coated with a lubricant to make them virtually frictionless. Because of these boundary stress conditions, the specimen is free to shear on a plane consistent with the directions of the major and minor principal stress planes and its inherent shear strength as expressed by c and ϕ.

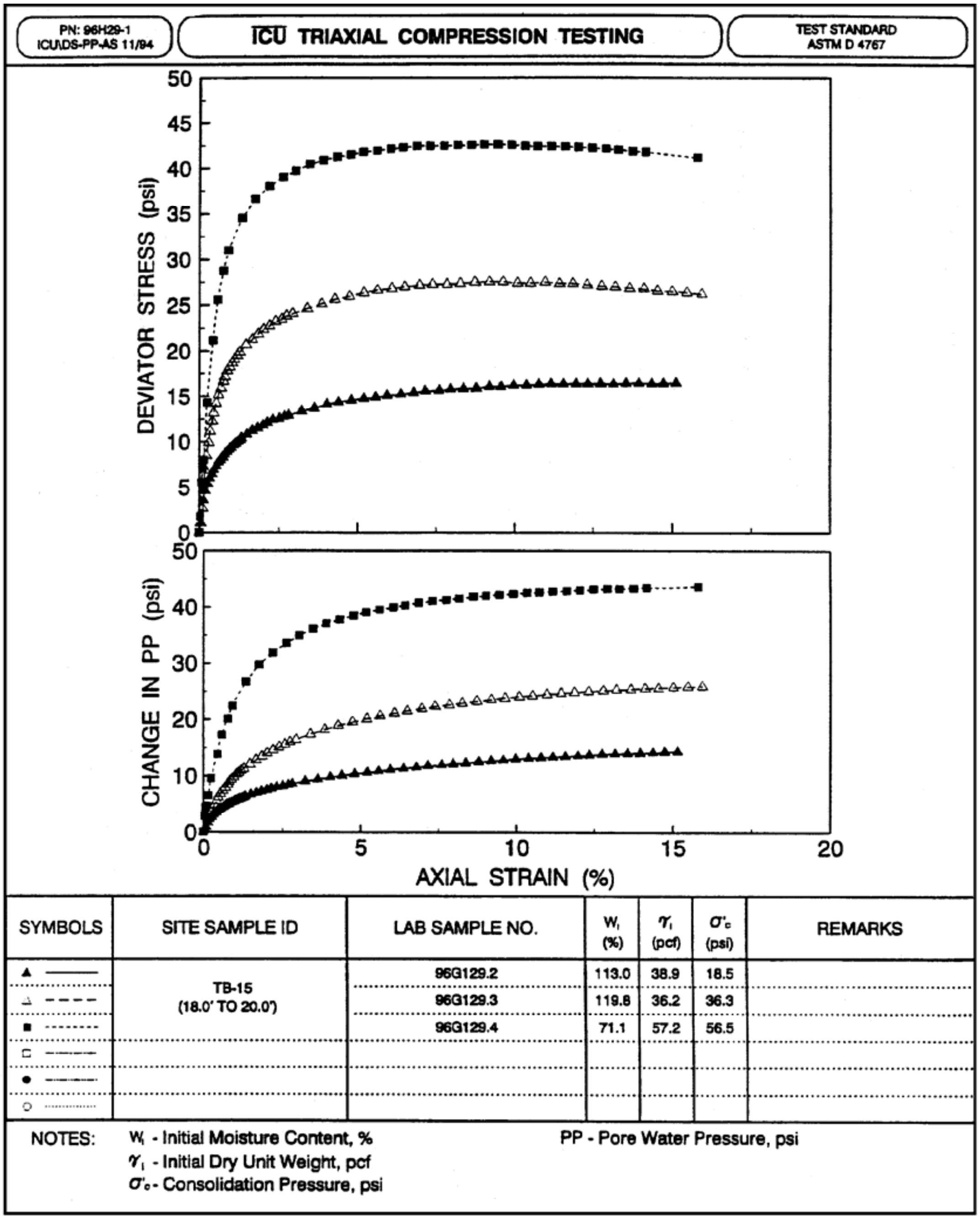


Figure 5-16. Typical stress-strain curves from CU test.

5.5.2.3 Direct Shear Tests

The oldest form of shear test upon soil is the direct shear test, first used by Coulomb (1776). A schematic of the essential elements of the direct shear apparatus are shown in Figure 5-17. The soil is held in a box that is split across its middle; the bottom portion of the box is usually fixed against lateral movement. A confining normal force, N, is applied, and then a tangential shear force, T, is applied so as to cause relative displacement between the two parts of the box. The magnitude of the shear force is recorded as a function of the shear displacement, and usually, the change in thickness of the soil sample is also recorded. Although it is widely used in practice, the direct shear device lacks a number of features that limit is applicability. For example, there is no way to control the confining pressure. Also, since there is no way to measure excess pore water pressures generated during shearing of saturated clay specimens, use of the direct shear test is generally limited to cohesionless soils.

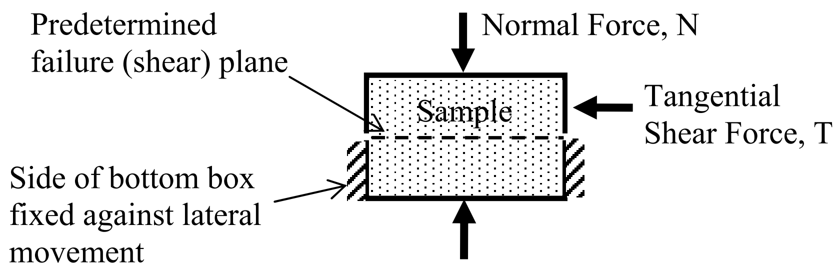


Figure 5-17. Schematic of the Direct Shear Test.

<u>Equipment</u> – The apparatus and procedures for direct shear testing are discussed in ASTM D 3080. A specimen is prepared in a split square or circular box. Figure 5-18 shows a circular specimen. The specimen is sheared as one part of the box is displaced horizontally with respect to the other. Generally, the lower part of the box is fixed against lateral movement and the shear force is applied to the upper part of the box through a loading frame as shown in Figure 5-18. The central two of the six screws visible in the top portion of the box extend into the bottom portion and are used to hold the assembly together while the specimen is being prepared. This shear box assembly is then placed in a reservoir which could be filled with water to allow saturation of the specimen prior to shearing. Before the test is begun, the two central screws are removed and the four corner screws, which rest on the top surface of the bottom portion of the box, are turned to slightly raise the top part of the box so that there is no contact between it and the bottom part of the box. This is done to prevent the error that would result from the frictional resistance between the two boxes at their contact. Load cells are used to monitor the shear force and LVDTs are used to monitor both horizontal and

vertical deformation. By use of this instrumentation, as well as a loading frame that provides a constant rate of horizontal deformation, it is possible to automate the direct shear test.

Figure 5-18. Direct shear testing box (Photograph courtesy of GeoComp Corporation).

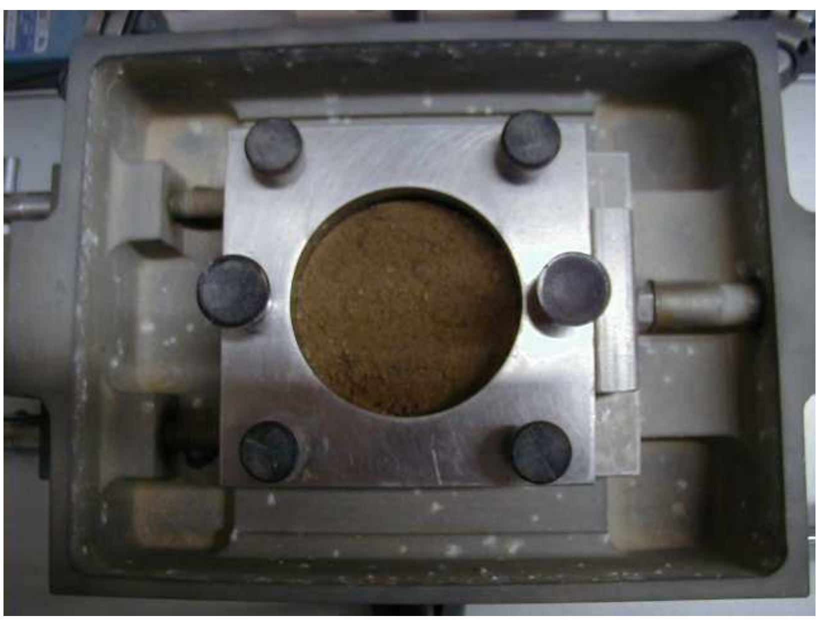


Figure 5-19. Soil sample mounted in direct shear testing apparatus (Photograph courtesy of GeoComp Corporation).

Test Procedures – In the direct shear test, the soil is first consolidated under an applied normal stress. After consolidation is completed, which will be virtually instantaneous in cohesionless soils, the specimen is sheared directly at a constant rate. The rate of shear is typically selected as a function of the hydraulic conductivity of the specimen. Direct shear testing is commonly performed on compacted materials used for embankment fills and retaining structures. Direct shear testing can also be performed on natural materials. However, the lack of control on soil specimen drainage makes the evaluation of undrained strength unreliable. The direct shear test can also be used to evaluate the drained strength of natural materials by shearing the sample at a rate slow enough to ensure that no significant pore water pressures develop, however there is no way to verify this condition by measurement.

In addition to providing data that allows the determination of the peak effective stress friction angle (ϕ'), the direct shear test data can also be used for the evaluation of effective stress residual strengths ($c'_r \approx 0$; ϕ'_r). The effective stress residual strength parameters are necessary for stability and landslide analyses. Residual strengths are associated with very large shear strains along a predefined or preferential slip surfaces that result in very large deformations. Data from a reversing direct shear test can also be used to evaluate residual shear strengths. In a reversing direct shear test, the direction of shearing in the test is reversed several times thereby causing the accumulation of displacements at the slip surface.

A characteristic of the direct shear test that distinguishes it from the triaxial test is that the shear failure in the direct shear device is forced to occur on a horizontal plane so that the orientations of the major and minor principal stress planes are not apparent. Ordinarily this characteristic is considered to be a disadvantage of the direct shear test. However this characteristic is advantageous for designs involving geosynthetics where the shearing resistance of the interface between the soil and the geosynthetic or between two pieces of geosynthetic is often required. Direct shear machines have been modified to test the interface shear strength between various types of engineering materials, as described in ASTM D 5321.

5.5.3 Factors Affecting Strength Testing Results

It is important for the designer to realize that all laboratory tests on soils must be carefully performed. This is particularly important for strength testing since the use of strength parameters are a key to successful foundation design. The following seven factors in particular affect the results of strength testing:

1. Sample disturbance
2. Mode of shearing
3. Confining pressures
4. Specimen size
5. Saturation
6. Displacement at failure
7. Rate of shearing and strain required to reach peak strength

A detailed discussion of each of these factors is presented in FHWA (2002a). In addition to recognizing the effect of the factors that can affect the strength testing results, it is extremely important for the designer to perform the proper tests depending on project requirements. The selection of an appropriate test to be used to provide relevant information for a particular geotechnical structure should consider, at a minimum, the following questions:

(1) How fast will construction occur relative to the hydraulic conductivity of the soil (i.e., should drained or undrained strength tests be performed)?
(2) How does the direction of applied load affect measured shear strengths and the appropriate strength to be used for an analysis?
(3) How do the expected levels of deformation for the geotechnical structure affect the selection of shear strength? and
(4) How does the manner in which the feature is constructed affect the shear strength to be used in analysis?

5.5.4 Comparison of Laboratory and Field Strengths

Soil samples are obtained from the ground for laboratory testing by sampling from boreholes and sealing and transporting these samples to the laboratory. The degree of disturbance affecting the samples will vary according to the type of soil, sampling method and the skill of the driller. At best some disturbance will occur from the removal of in-situ stresses during sampling and from the preparation of specimens in the laboratory for testing. In general, disturbance tends to reduce the shear strength obtained from unconfined or unconsolidated tests and increase the shear strength obtained from consolidated tests. There is, therefore, considerable merit in measuring the in-situ shear strength. The field vane shear test discussed in Chapter 3 is the most commonly used field test for direct measurement of the undrained shear strength of soft to medium clays. In reviewing the different types of field and laboratory tests available to determine the undrained shear strength in clays, the designer should expect the field vane shear test to provide the most accurate value of s_u, with UC and UU tests yielding lower results and CU tests yielding slightly higher results.

It is important that the results of in-situ tests be interpreted carefully and calibrated with laboratory tests. Without careful calibrations, the in-situ tests can yield inaccurate results.

5.5.5 Selection of Design Shear Strength

Frequently, on a large project the designer will receive a large quantity of undrained shear strength test results from both the field and laboratory. This data must be synthesized to permit rational interpretation of the results. The tests should be analyzed on a hole-by-hole basis. All tests from one hole should be reviewed and the results for each type of test should be plotted separately versus depth to determine the pattern of strength variation for each test type and to assess the reliability of the data, e.g., a CU test result that is lower than a UC test result at the same depth should be considered suspect. The general pattern of shear strength results should show an increase in strength with depth in a normally consolidated clay deposit. Overconsolidated clays may exhibit this pattern only at greater depths since the amount of preconsolidation increases shear strength in the upper portions of the soil deposit. Section 5.14 presents values of the coefficient of variation of measured soil properties that should be taken into account while selecting the final design shear strength.

5.5.6 Correlations of Shear Strength Parameters with Index Parameters

This section presents some useful correlations between shear strength parameters and other index values or field conditions. These correlations may be used by the designer to check the general validity of the laboratory test results or to develop a preliminary assessment about the shear strength characteristics of the soils on the project site. In the latter case, it must be emphasized that predictions based on correlations alone should never be used for design and that any assumptions regarding shear strength parameters must always be verified through testing.

5.5.6.1 Undrained Shear Strength of Cohesive Soils

For most saturated clays tested under quick undrained conditions, the angle of shearing resistance, ϕ_u, is zero. This means that the shear strength of the clay is a fixed value and is equal to the apparent cohesion, c_u, at a specific moisture content and preconsolidation pressure. A value for the undrained shear strength may be crudely estimated for a sample for which uncorrected SPT N-values are known by molding a specimen of the clay between the fingers and by applying the observations indicated in Table 4-2 in Chapter 4. However, as noted in the footnote of Table 4-2**, the values of SPT blow count listed there should not be used to determine the design strength of fine grained soils.**

Undrained Shear Strength

For most normally consolidated clays the undrained shear strength (s_u) is proportional to the effective overburden pressure (p_o). For such soils, Skempton (1957) proposed the relationship shown in Figure 5-20 between the s_u/p_o and plasticity index (PI). Figure 5-20 also includes results obtained by a number of other researchers. As can be seen in the figure, the composite of all findings varies so much that **such relations should be used with caution.** However, such correlations, particularly the correlation by Skempton (1957), are useful for obtaining preliminary estimates and for checking laboratory results of project-specific tests performed on normally consolidated clays.

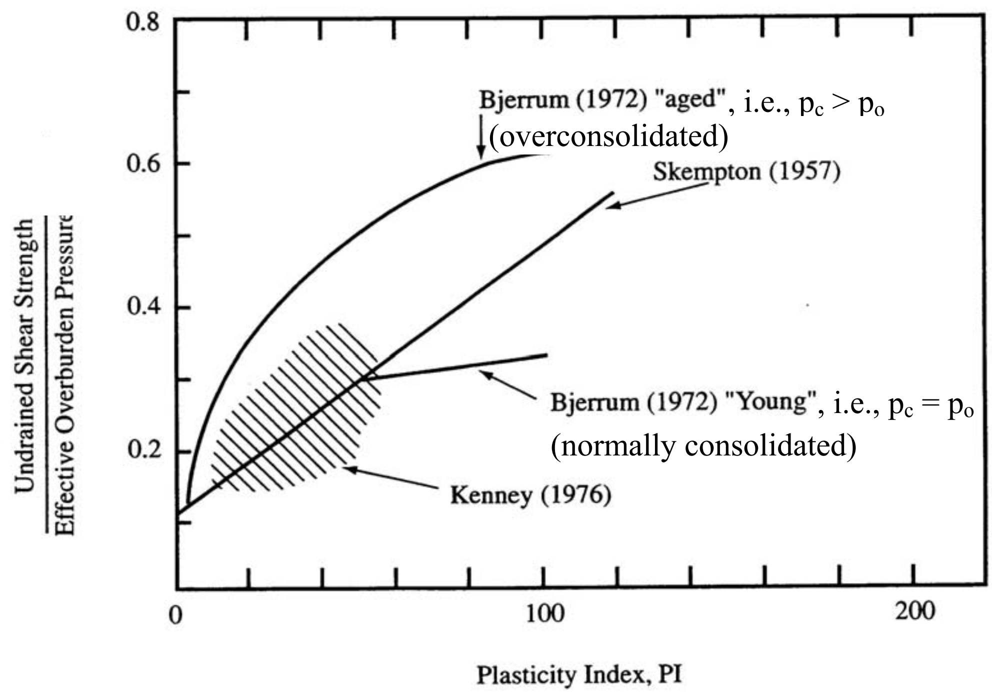


Figure 5-20. Relationship between the ratio of undrained shear strength to effective overburden pressure and plasticity index for normally consolidated and overconsolidated clays (after Holtz and Kovacs, 1981).

The shear strength of undisturbed clays depends on the consolidation history of the clay as well as its fabric characteristics. In general, the undrained strength ratio, s_u/p_o, increases with increasing overconsolidation as measured by the overconsolidation ratio, OCR. **It is recommended that laboratory tests be performed to determine the undrained strength ratio for overconsolidated clays rather than relying on any published correlations.** In practical terms, it is more straightforward to measure the undrained shear strength of overconsolidated clays than to predict it from other indices.

5.5.6.2 Drained and Effective Shear Strength of Cohesive Soils

It is often important to carry out stability calculations in terms of effective stresses. The soil strength parameters, c' and ϕ', used in these calculations should be obtained from either drained direct shear box or drained triaxial tests or from CU triaxial tests with pore water pressure measurements (giving ϕ'_{cu} and c'_{cu}). Generally, there is a minor difference in the results obtained from these two tests for saturated clays because the soil is being tested under different boundary conditions and stress paths. In-situ tests such as CPTs can also be used to estimate the drained and effective shear strength parameters of cohesive soils.

For clays, empirical correlations have been developed to relate ϕ' to the plasticity characteristics of the soil. Figure 5-21 shows a slight trend of ϕ' decreasing with increasing PI. The existence of these relationships arises because both PI and shear strength reflect the clay mineral composition of the soil; as the clay mineral content increases, the PI increases and the strength decreases. From Figure 5-21, it can be seen that the drained friction angle values can be $\pm$ 8° in variance with respect to the dashed trend line.

Considering the overall importance of ϕ' in stability calculations, foundations design, and landslide analyses, it is essential to assess ϕ' directly by means of consolidated drained direct shear tests, consolidated drained triaxial tests, or consolidated undrained triaxial tests with pore water pressure measurements. The consequences of merely estimating ϕ' can be economically unwise. As an example, in stability analyses for relatively long, shallow slip surfaces that may be associated with a landslide, the required forces that would need to be resisted by some form of stabilization system (e.g., retaining wall, micropiles) would vary significantly depending on the drained friction angle of the soil. It is highly recommended that geotechnical designers develop historical data summaries of ϕ' versus PI to check the validity of future test results.

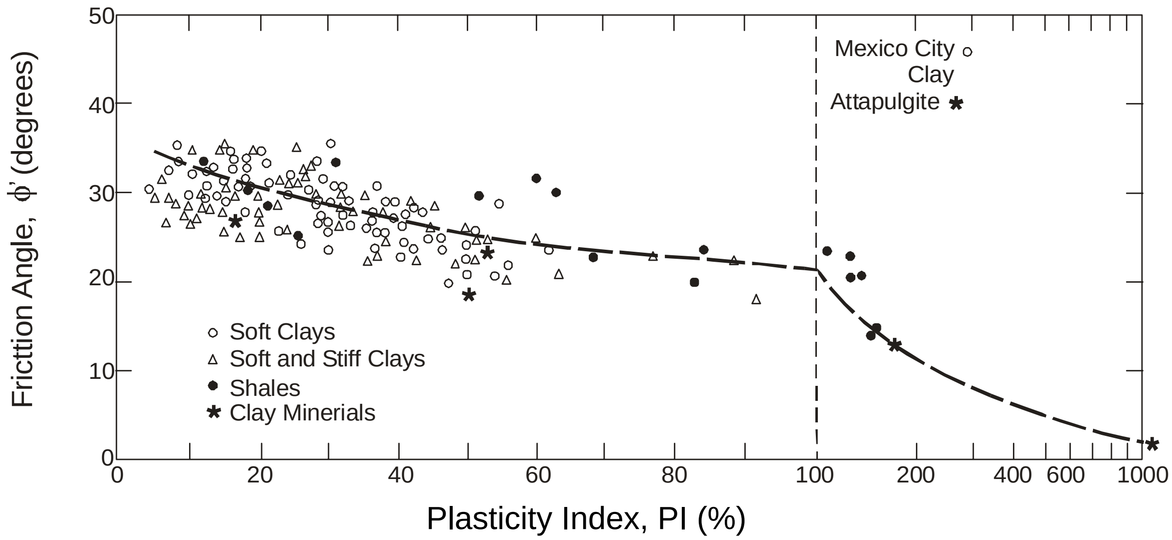
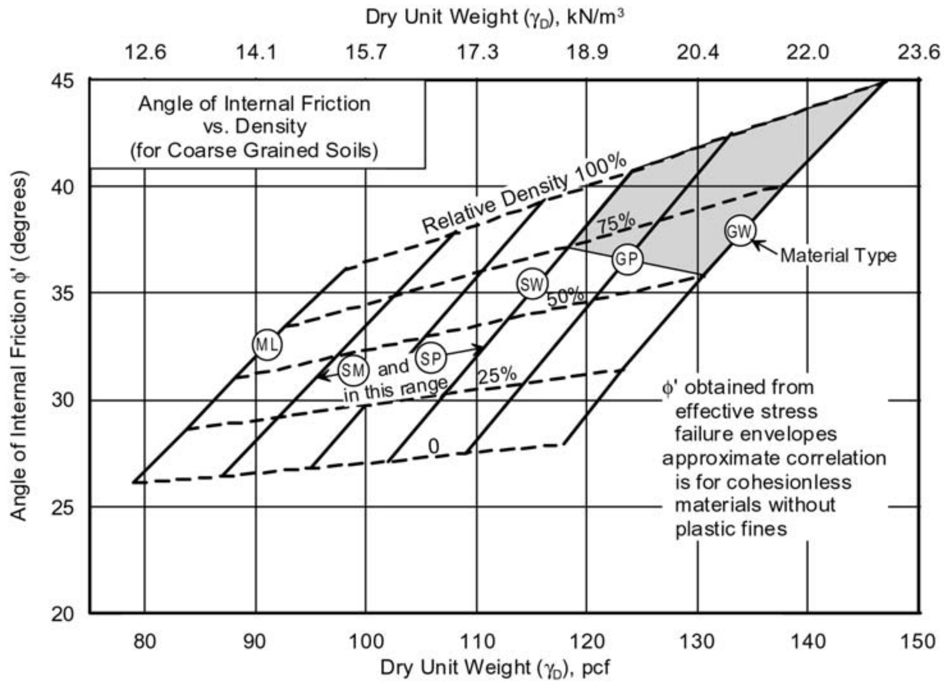


Figure 5-21. Relationships between ϕ and PI. (after Terzaghi, *et al.*, 1996).

5.5.6.3 Shear Strength of Cohesionless Soils

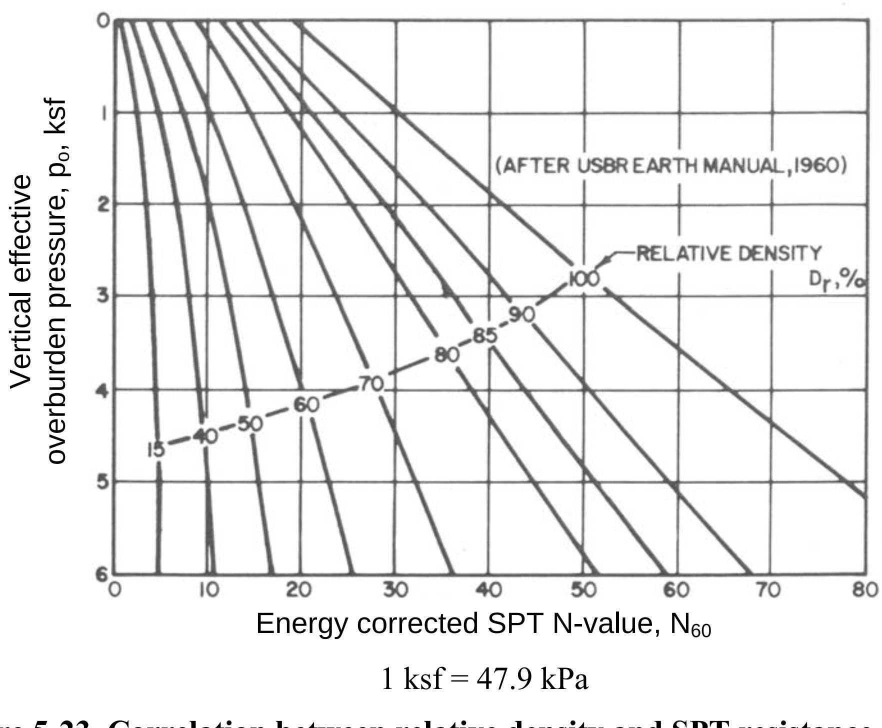
Because of their high permeability, pore water pressures do not build up significantly when cohesionless soils are subjected to shearing forces. The complication of total and effective stresses is therefore avoided and the phenomenon of apparent cohesion, or undrained shear strength does not occur. Consequently, the shear strength of cohesionless soils is defined exclusively in terms of frictional resistance between the grains, as measured by the angle of shearing resistance, ϕ. Typical values of ϕ for sands and gravels are given in Figure 5-22 as a function of dry unit weight and relative density. The material types indicated in the figure relate to the Unified Classification System (USCS).

Figure 5-22 requires determination of relative density. A reasonable estimate of relative density can be obtained from Figure 5-23. Figure 5-23 was originally developed based on data obtained using rope and cathead operated hammers. Thus, it is recommended that an energy corrected SPT N-value, i.e., N_{60}, be used as shown in Figure 5-23. However, note that Figure 5-23 is a function of both N-value and the vertical effective overburden pressure, p_o. Therefore, N_{60}-value should not be corrected for overburden pressure, i.e., C_N=1.0 (see Section 3.7.2) while using Figure 5-23.



Note: Use caution in the shaded portion of the chart due to the potential for unreliable SPT N-values in gravels

Figure 5-22. Correlation between relative density, material classification and angle of internal friction for coarse-grained soils (NAVFAC, 1986a).



1 ksf = 47.9 kPa

Figure 5-23. Correlation between relative density and SPT resistance (NAVFAC, 1986a).

5.6 PERMEABILITY

5.6.1 General

Permeability, also known as hydraulic conductivity, is one of the major parameters used in selecting soils for various types of construction. In some cases, it may be desirable to place a high-permeability fill immediately under a pavement surface or behind a wall to facilitate the removal of water. In other cases, such as retention pond dikes, it may be detrimental to use high-permeability materials. Permeability also significantly influences the choice of backfill materials for various elements such as trenches and retaining walls.

Unlike the fill soils discussed in the previous paragraph, the permeability of a natural soil is strongly influenced by its macroscopic structure, e.g., clays containing fissures or fine bands of sand will have permeabilities that are many times greater than that of the clay material itself. Also, stratified soils often have horizontal permeabilities that are many times the vertical permeability. Because of the small size of laboratory specimens and the way they are obtained and prepared, large scale in-situ features are absent and test results do not give a true indication of field values in soils with a pronounced macro-structure. Moreover, laboratory tests usually constrain water to flow vertically through the specimen whereas the horizontal permeability may be much greater, and hence of overriding importance, so far as site conditions are concerned. Field permeability tests overcome these shortcomings, but, since the pattern of water flow from a well used to determine in situ permeabilities can only be estimated, interpretation of the field test results is difficult and uncertain. Thus, one set of problems is exchanged for another.

5.6.2 Equipment

Laboratory permeability testing is performed to determine the permeability or hydraulic conductivity (k) of a soil specimen. For natural soils, tests are conducted on specimens from tube samples. For fill and borrow soils, tests are performed on compacted materials. Two types of tests are commonly performed, the rigid wall test (AASHTO T215; ASTM D 2434) and the flexible wall test (ASTM D 5084).

The equipment for the rigid wall test includes a rigid wall permeameter, a water tank, a vacuum pump, and manometer tubes (see Figure 5-24). The permeameter must be large enough to minimize sidewall leakage. Therefore, the diameter of the rigid wall device should be at least 8 to 12 times that of the largest soil particle in the sample being tested. Frequently, the side wall of the cylinder is coated with petroleum jelly to prevent sidewall leakage. Porous stones and filter paper placed on the top and bottom of the test specimen to prevent soil particle erosion by retaining fine particles must not restrict flow through the soil otherwise the permeability of the porous stones or filter paper will be measured. A vacuum pump is used to remove air from the sample and to help saturate the specimen prior to testing. In a rigid wall test saturation is performed from the bottom of the specimen upward (see ASTM D 2434). Manometer outlets should be available on the sides of the cell to measure head loss over the specimen.

Rigid wall permeameters are not recommended for low permeability (i.e., $k \leq 2 \times 10^{-5}$ ft/min (1×10^{-6} cm/s)) soils due to the potential for sidewall leakage. Rigid wall permeameters are typically used for sandy and gravelly soils (ASTM D 2434) with a permeability greater than 2×10^{-2} ft/min (1×10^{-3} cm/s). Rigid wall systems are used for granular materials because the permeability of the flexible wall system (valves, pore stones, tubing, etc.) may be less than that of the specimen. Therefore the flexible wall system itself may affect the permeability value of granular soils and a measured value of permeability lower than that of the specimen may result.

Figure 5-24. Rigid wall permeameter (Photograph courtesy of GeoComp Corporation).

Figure 5-25. Flexible wall permeameter (Photograph courtesy of GeoComp Corporation).

The equipment for a flexible wall test includes a flexible wall permeameter cell, a cell reservoir, a headwater reservoir, a tailwater reservoir, top and base caps, a flexible membrane, porous stones, and filter paper (see Figure 5-25). The specimen can be tested over a range of confining stresses under backpressure saturation. The separate headwater and tailwater reservoirs can be monitored, and falling head or constant head tests can be performed. Since the flexible membrane encases the specimen, side leakage is prevented. Flexible wall permeameter cells consist of influent and effluent lines as well as porous stones and filter paper. The hydraulic conductivity of the system should be tested before a soil specimen is tested to ensure that the system's conductivity is at least one order of magnitude greater than that anticipated for the soil. The triaxial cell can be used as a flexible wall permeameter. In fact, permeability measurements are often made as part of a drained triaxial (CD) test.

The determination of permeability by testing is predicated on the validity of Darcy's Law for laminar flow through porous media. If any of the assumptions of Darcy's Law are violated, the results of permeability testing will be invalid.

5.6.3 Procedures

Flexible wall permeameters (ASTM D 5084) are used for materials with a hydraulic conductivity (k) less than or equal to $2x10^{-2}$ ft/min ($1x10^{-3}$ cm/sec). The flexible membrane used to encase the specimen prevents sidewall leakage for fine-grained soils that are likely to occur in a rigid wall system. The confining stress of the hydraulic conductivity test should be specified to the laboratory. As confining stress increases, the hydraulic conductivity of fine grained soils will typically decrease due to consolidation of the specimen and reduction of void ratio. The confining stress should be equal to the anticipated effective stress-state in the soil.

The hydraulic gradient, defined as the difference in hydraulic head across the specimen divided by the length of the specimen, should also be specified to the laboratory. Typical hydraulic gradients in field situations are less than 5, however the use of such a small gradient in the laboratory will result in extremely long testing times for materials having hydraulic conductivities less than $2x10^{-5}$ ft/min ($1x10^{-6}$ cm/sec). If the hydraulic gradient across the specimen is too high, turbulent flow will occur and Darcy's Law will be violated with the result that the measured hydraulic conductivity will be less than that which will occur in the field. Suggested values of hydraulic gradient, as presented in Table 5-9, are a function of the anticipated hydraulic conductivity.

Saturation of the specimen is necessary to achieve accurate results. A hydraulic conductivity test should be ended when steady flow is occurring. The flow through the permeameter is considered to be steady when four or more consecutive hydraulic conductivity measurements fall within ±25 percent of the average k value if k is greater than 1×10^{-8} cm/sec, or if four or more measurements fall within ±50 percent of the average if k is less than 1×10^{-8} cm/sec.

Table 5-9
Recommended maximum hydraulic gradient for permeability testing

Hydraulic Conductivity, cm/sec*	Recommended Maximum Hydraulic Gradient
1×10^{-3} to 1×10^{-4}	2
1×10^{-4} to 1×10^{-5}	5
1×10^{-5} to 1×10^{-6}	10
1×10^{-6} to 1×10^{-7}	20
less than 1×10^{-7}	30
* Conventionally expressed in cm/sec. [1 cm/sec ≈ 2 ft/min = 0.6 m/min]	

5.6.4 Useful Correlations of Permeability with Index Values

The typical range of permeability values for various soil types and USCS groups is presented in Table 5-10, which is based on information originally presented by Casagrande and Fadum (1940). Superimposed on Table 5-10 are "typical soil groups" identified by their USCS symbols (Carter and Bentley, 1991). The range of permeability values corresponding to those groups is typical for compacted soils of that type where compaction is according to ASTM D 1557. Typical permeability values for highway construction materials are given in Table 5-11.

Numerous correlations of permeability with grain size can be found in the literature. Figure 5-26 presents logarithmic plots of k versus D_{10}, based on experimental results. Figure 5-26 includes the well known Hazen's formula. All the correlations shown in Figure 5-26 were developed for sands and gravels. The great range of particle size present in most clays and the effects of clay mineralogy make such correlations of limited use for clays.

Hazen's equation is the most common correlation equation used to estimate permeability for sands ($k \geq 10^{-3}$ cm/sec). This equation is written as:

$$k = C(D_{10})^2 \tag{5-14}$$

where: k is the permeability in cm/s;

- C = a coefficient ranging from 0.4 to 1.2 depending on sand size/sorting; and
- D_{10} = effective grain size in mm at 10% passing by weight as determined from sieve analysis.

Hazen's equation should be used with caution since it provides very approximate estimates of k applicable only to clean sands having less than 5% passing the No. 200 sieve (0.075 mm) and with D_{10} sizes between 0.1 and 3.0 mm (Holtz and Kovacs, 1981). Hazen's equation is valid only for $k \geq 10^{-3}$ cm/sec.

Table 5-10
Typical permeability values in soils (after Carter and Bentley, 1991)

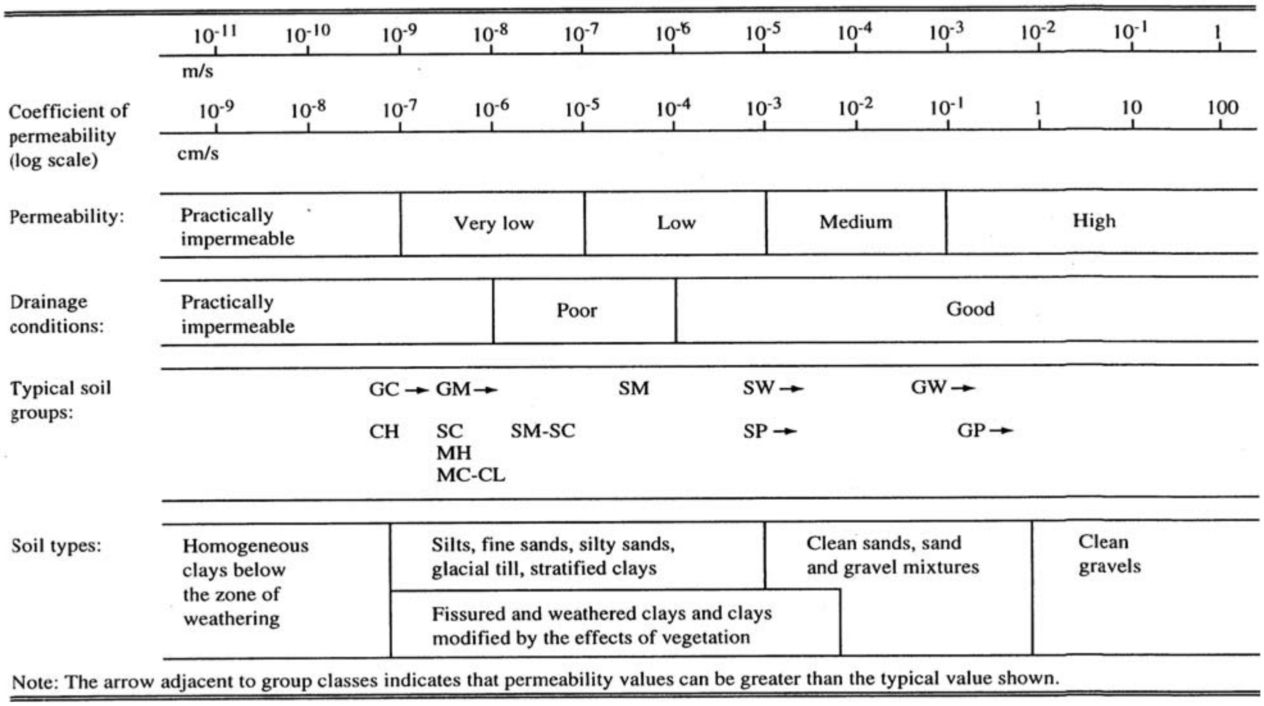


Table 5-11
Typical permeability values for highway materials (after Krebs and Walker, 1971)

Materials	**Permeability (cm/sec)**
Uniformly graded coarse aggregate	40 - $4x10^{-1}$
Well-graded aggregate without fines	$4x10^{-1}$ - $4x10^{-3}$
Concrete sand, low dust content	$7x10^{-2}$ - $7x10^{-4}$
Concrete sand, high dust content	$7x10^{-4}$ - $7x10^{-6}$
Silty and clayey sands	10^{-5} - 10^{-7}
Compacted silt	$7x10^{-6}$ - $7x10^{-8}$
Compacted clay	Less than 10^{-7}
Bituminous concrete (new pavements)*	$4x10^{-3}$ - $4x10^{-6}$
Portland cement concrete	less than 10^{-8}
* Values as low as 10^{-8} have been reported for sealed, traffic compacted highway pavement.	

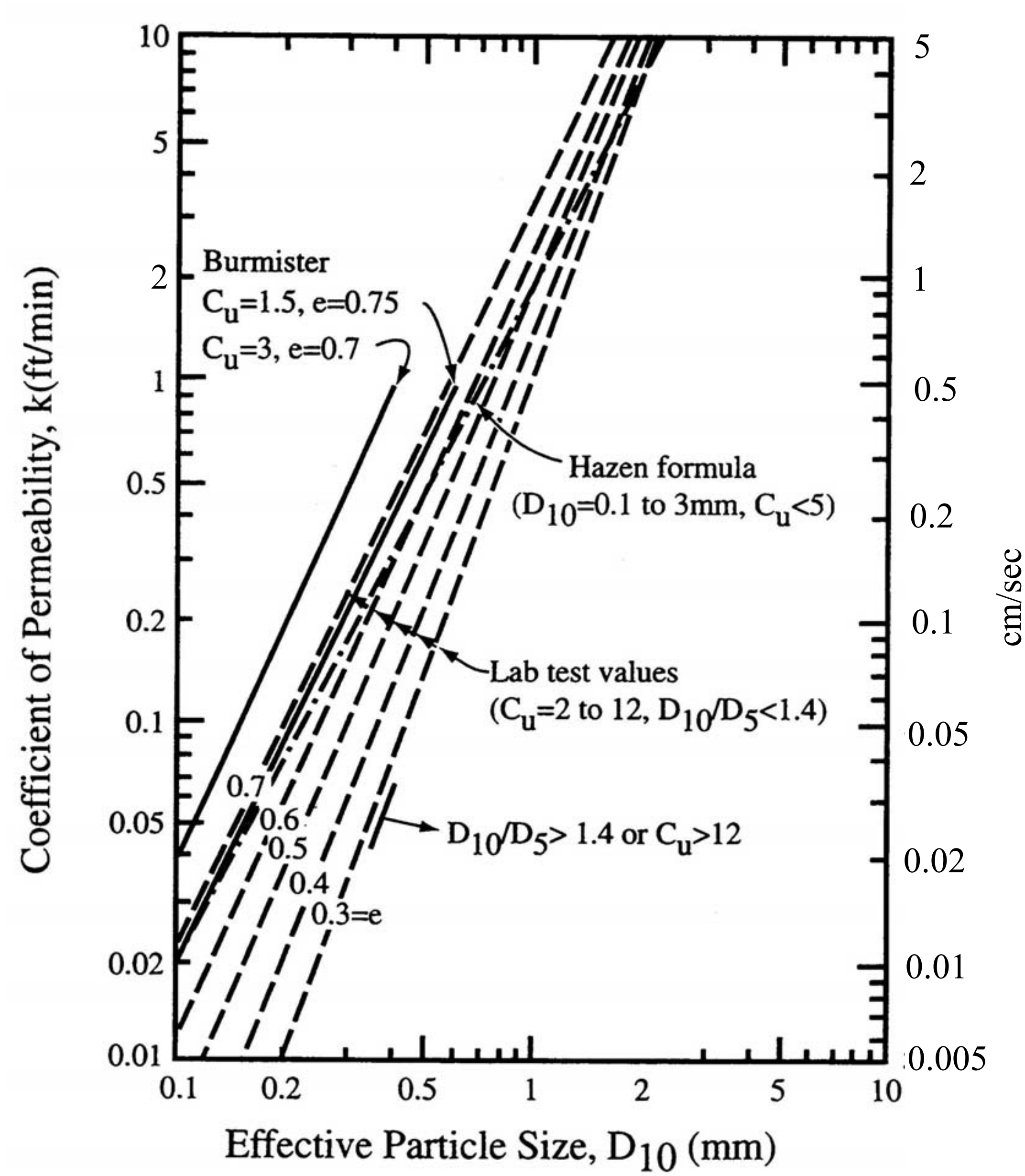


Figure 5-26. The permeability of sands and gravels (after NAVFAC, 1986a).

Note: In Figure 5-26, correlations shown are for remolded compacted sands and sand-gravel mixtures with C_u values as indicated.

5.7 VOLUME CHANGE PHENOMENA DUE TO LOADING AND MOISTURE

Depending on mineralogy and depositional patterns, natural soils can exhibit either swell (expansion) or collapse under various degrees of loading and moisture ingress. Moisture may be in liquid or frozen form. For foundation design, it is very important to recognize and evaluate the potential for soils to swell or collapse. It is important to realize that these phenomena happen in both natural and compacted soils. Every year millions of dollars are spent dealing with the consequences of swelling (expanding) and collapsing soils. This section briefly discusses these two mechanisms and the tests that can be performed to evaluate the swell (expansion) and collapse potentials.

5.7.1 Swell Potential of Clays

Swelling is a characteristic reaction of some clays to water ingress. The potential for swell depends on the mineralogical composition of the soil fines. While montmorillonite (smectite) exhibits a high degree of swell potential, illite has no to moderate swell potential, and kaolinite exhibits almost none. The percentage of volumetric swell of a soil depends on the amount and type of clay, its relative density, the compaction moisture content and dry density, permeability, location of the groundwater table, the presence of vegetation and trees, overburden pressure, etc. Expansive soils are found throughout the U.S., however, damage caused by expansive clays is most prevalent in certain parts of California, Wyoming, Colorado, and Texas, where the climate is considered to be semi-arid and periods of intense rainfall are followed by long periods of drought. This pattern of wet and dry cycles results in periods of extensive near-surface drying and desiccation crack formation. During intense precipitation, water enters the deep cracks causing the soil to swell; upon drying, the soil will shrink. This weather pattern results in cycles of swelling and shrinking that can be detrimental to the performance of pavements, slabs on-grade, and retaining walls built on or in such soils.

Deep-seated volume changes in expansive soils are rare. More common are volume changes within the upper 3-10 feet (1- 3 m) of a soil deposit. These upper few feet are more likely to be affected by seasonal moisture content changes due to climatic changes. The zone over which volume changes are most likely to occur is defined as the active zone. The active zone can be evaluated by plotting the moisture content with depth for samples taken during the wet season and for samples taken during the dry season at the same location. The depth at which the moisture content becomes nearly constant is the limit of the active zone depth, which is also referred to as the depth of seasonal moisture change. The active zone is an important consideration in foundation design. In the design of piles or drilled shafts, it is important to recognize that full side friction resistance may not be realized in this zone. As the soil undergoes cycles of shrinking and swelling, it may lose contact with the pile or shaft. Alternatively, as the soil swells, it may impose significant uplift pressures on the foundation element.

In the field, the presence of surface desiccation cracks and/or fissures in a clay deposit is an indication of expansion potential. Experience has indicated that the most problematic expansive near-surface soils are typically highly plastic, stiff, fissured, overconsolidated clays. Several classification methods are used to identify expansive soils in the laboratory. Currently, there is not a standard classification procedure; different methods are used in various locations across the U.S. Typically, methods include the use of Atterberg limits and/or clay size percentage to describe a soil qualitatively as having low, medium, high, or very high expansion potential. Generally, soils with a plasticity index less than 15 percent will not exhibit expansive behavior. For soils with a plasticity index greater than 15 percent, the clay content of the soil should be evaluated in addition to the Atterberg limits. Figure 5-27 shows the swelling potential of natural soils and soils compacted to standard Proctor procedures (ASTM D 698) as a function of modified activity index, A_m, (Equation 5-4) and clay fraction.

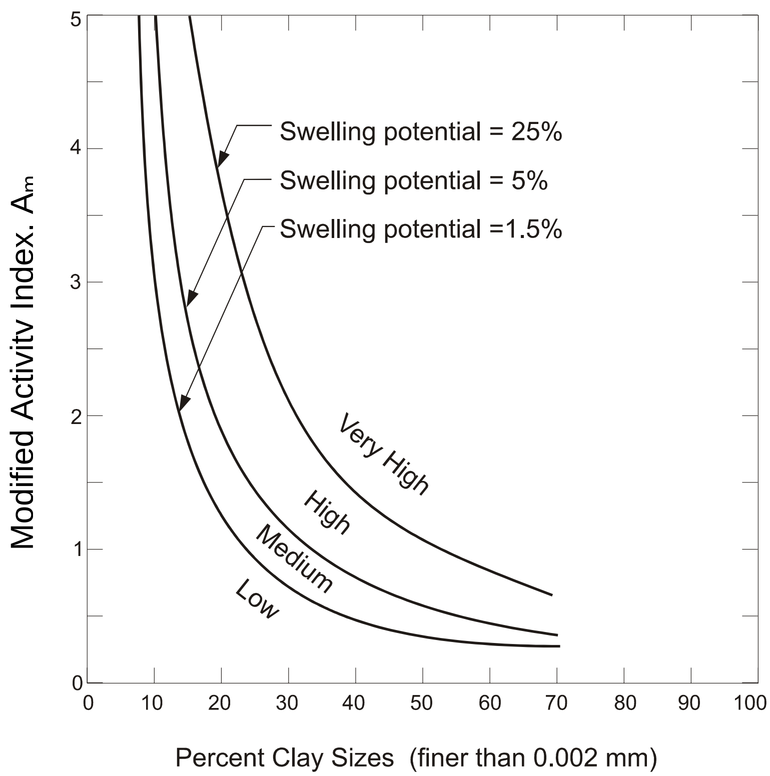


Figure 5-27. Classification of swell potential for soils (after Seed *et al.*, 1962).

5.7.1.1 Evaluation of Expansion (Swell) Potential

For situations where it is necessary to construct a facility in and around expansive soils, it will be necessary to estimate the magnitude of swell, i.e., surface heave, and the corresponding swelling pressures that may occur if the soil becomes wetted. The swelling pressure represents the magnitude of pressure that would be necessary to resist the tendency of the soil to swell. A one-dimensional swell potential test can be performed in an oedometer on undisturbed or recompacted samples according to AASHTO T256 or ASTM D 4546. In this test, the swell potential is evaluated by observing and measuring the swell of a laterally confined specimen when it is lightly surcharged and flooded with water. Alternatively, if the swelling pressure is to be measured, the height of the specimen is kept constant by adding load after the specimen is inundated. The swelling pressure is then defined as the vertical pressure necessary to maintain zero volume change. Swelling pressures in some expansive soils may be so large that the loads imposed by lightweight structures or pavements do little to counteract the swelling.

The use of the one-dimensional swell potential test to evaluate in-situ swell potential of natural and compacted clay soils has limitations including:

- Lateral swell and lateral confining pressure are not simulated in the laboratory. The calculated magnitude of swell in the vertical direction may not be a reliable estimate of soil expansion for structures that are not confined laterally (e.g., bridge abutments);

- The rate of swell calculated in the laboratory will not likely be indicative of the rate of swell experienced in the field. Laboratory tests cannot simulate the actual availability of water in the field.

It should be noted that there is a lack of a standard definition of swell potential in the technical literature based in part on variations in the test procedures, e.g., the condition of the test specimen (remolded or undisturbed), the magnitude of the surcharge, etc. Therefore the geotechnical specialist must be sure that the conditions used in the laboratory swell test simulate those expected in the field. In general, soils classified as CL or CH according to the USCS and A-6 or A-7 according to the AASHTO classification system should be considered potentially expansive.

5.7.2 Collapse Potential of Soils

There are several types of soils that can experience collapse under moisture ingress. Examples of such soils are wind-blown deposits such as loess, or alluvial soils deposited in arid or semi-arid environments where evaporation of soil moisture takes place at such a rapid rate that the deposits do not have time to consolidate under their self weight or where the deposits are cemented by precipitated salts. Such soils are predominantly composed of silts and some clay. Typically, the structure of such soils is flocculated and the soil particles are held together by "clay bridges" or some other cementing agent such as calcium carbonate. In both cases disturbed samples obtained from these deposits are generally classified as silt. When dry or at low moisture content the in-situ material gives the appearance of a stable deposit. At elevated moisture contents these soils generally undergo sudden changes in volume and collapse. **Full saturation is not required to realize collapse of such soils; often collapse of the soil structure occurs at moisture contents corresponding to pre-collapse degrees of saturation between 50 to 70%.** Such soils, unlike other non-cohesive soils, will stand on almost a vertical slope until inundated. Collapse-susceptible soils typically have a low relative density, a low unit weight and a high void ratio. Figure 5-28 is a useful tool for assessing whether a soil is collapsible or not based on LL and dry unit weight.

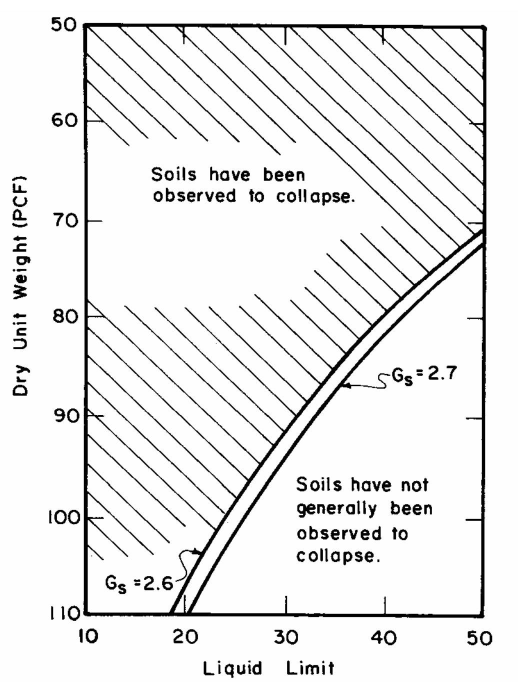


Figure 5-28. Chart for evaluation of collapsible soils (after Holtz and Hilf, 1961).

Structures founded on such soils may be seriously damaged if the soils are inundated and collapse. Therefore, if a soil is suspected to be collapse susceptible, then it is of primary importance to estimate the magnitude of potential collapse that may occur if the soil becomes wetted. To do this, a one-dimensional collapse potential test can be performed in an oedometer on undisturbed or recompacted samples according to ASTM D 5333. For this test, a sample is placed in an oedometer at it natural or compacted moisture content and the vertical pressure on the sample is increased in increments to the anticipated final loading in the field. Readings of vertical deformation are taken during the loading sequence. At the anticipated final load level, water is introduced to the sample and the resulting deformation due to collapse is recorded. The percent collapse (%C) is defined as:

$$\%C = \frac{100\,\Delta H_c}{H_o} \tag{5-15}$$

where ΔH_c is the change in height upon wetting and H_o is the initial height of the specimen. Conceptually, C is a strain. Therefore, for a soil layer with a given thickness, H, the settlement due to collapse, $s_{collapse}$, if the entire thickness is inundated may be calculated as:

$$s_{collapse} = H\left(\frac{(\%C)}{100}\right) \tag{5-16}$$

The collapse potential (CP) is calculated as the percent collapse (%C) of a soil specimen subjected to a total load of 4 ksf (200 kPa) as measured by using procedures specified in ASTM D 5333. The CP is an index value used to compare the susceptibility of various soils to collapse. Table 5-12 provides a relative indication of the degree of severity for various values of CP.

Table 5-12
Qualitative assessment of collapse potential (after ASTM D 5333)

Collapse Potential (CP)	Severity of Problem
0	None
0.1 to 2%	Slight
2.1 to 6%	Moderate
6.1 to 10%	Moderately Severe
>10%	Severe

5.7.3 Expansion of Soils due to Frost Action

The expansion of soils due to frost action is commonly known as **frost heave**. Three conditions are required for frost heave to occur. These are (a) freezing surface, (b) source of water, and (c) fine grained soils in which capillary rise can occur. Frost action in soils can have important engineering consequences as follows (after Holtz and Kovacs, 1981):

- The volume of the soil can immediately increase about 10% just due to the volumetric expansion of water upon freezing.

- The formation of ice crystals and lenses in the soil can cause heaving and damage to light surface structures such as small buildings, and highway pavements. Frost action can also displace retaining walls due to increased lateral pressures.

When water in saturated fine-grained soils freezes, it forms lenses of ice oriented roughly parallel to the surface exposed to low temperature (Terzaghi, *et al.,* 1996). Due to the inherent variability in the distribution of pore space, soils affected by frost action do not freeze and expand uniformly. Therefore, just as with swelling and collapsing soils, differential movement occur, and causes structural damage. Upon thawing, the moisture content of the soil increases which leads to reduction in shear strength and a consequent reduction in bearing capacity. Thus, the freeze-thaw cycle results in significant distress to structures and in particular highway pavements.

Only fine-grained soils are susceptible to frost action. However, the critical grain size marking the boundary between soils that are subject to ice-lens formation and those that are not depends on the uniformity of the soil, i.e., the distribution of pore space. The following conditions noted by Terzaghi, *et al.* (1996) may be used as a general guide for evaluating the frost susceptibility of soils:

- In perfectly uniform soils, i.e., a single particle size soils, ice lenses do not develop unless the grains are smaller than 0.01 mm.

- Uniform soils must contain at least 10% of grains smaller than 0.02 mm.

- In mixed-grain soils, ice lenses form when grains with a size less than 0.02 mm constitute at least 3% of the total aggregate.

- In soils with less than 1% of grains smaller than 0.02 mm, ice lenses are not formed under any conditions which may be encountered in the field.

Common frost susceptible soils include silts (ML, MH), silty sands (SM), and low plasticity clays (CL, CL-ML). One of the most common methods to mitigate the detrimental effects of frost is to place the foundations below the anticipated frost depth. The depth of frost action depends primarily on air temperature below freezing and duration, soil permeability and soil water content. Figure 5-29 can be used for a preliminary estimate of the frost depth. More positive measures to mitigate damage due to frost action include lowering of the ground water table and, depending on the depth of the frost penetration, removal of the frost susceptible soils in the subgrade or foundation. Use of impervious membranes, chemical additives, and even foamed insulation (Styrofoam) under highways, buildings, and railroads have been successfully employed (Holtz and Kovacs, 1981).

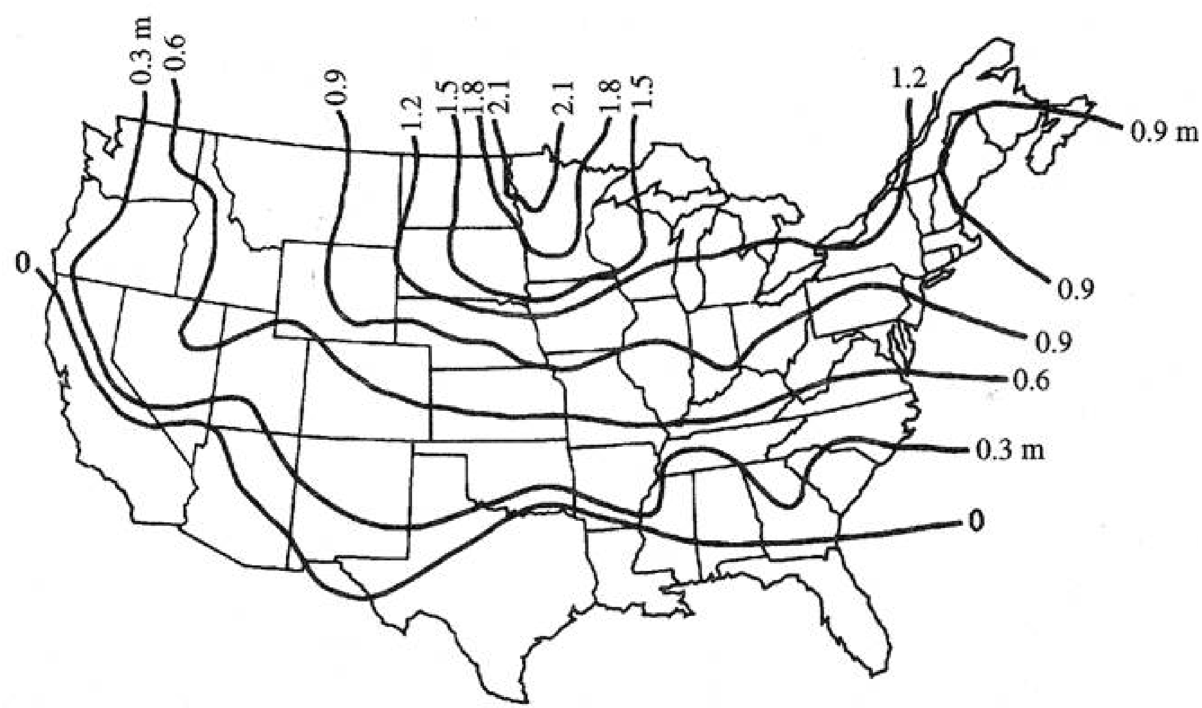


Figure 5-29. Approximate frost depth map for United States (Bowles, 1996). (1 m = 3.28 ft)

5.8 COMPACTION CHARACTERISTICS OF SOIL

5.8.1 Concept of Compaction

In the construction of highway embankments, earth dams, retaining walls, structural foundations and many other facilities, loose soils must be compacted to increase their densities. Compaction is the process of densifying soil under controlled moisture conditions by application of a given amount and type of energy. Compaction increases the density of the soil, which generally leads to:

- an increase in the strength and stiffness characteristics of the soil,
- a decrease in the amount of undesirable settlement of structures under both static and dynamic loads,
- a reduction in soil permeability, and
- an increase in the stability of slopes and embankments.

Unless compaction is properly controlled, there is a potential that the volume change phenomena described in Section 5.7 (swell, collapse and frost heave) can occur.

The density of compacted soils is measured in terms of the dry unit weight, γ_d, of the soil. The dry unit weight is a measure of the amount of solid materials present in a unit volume of soil. The greater the amount of solid materials, the stronger and more stable the soil will be. Pertinent parameters for evaluating the results of laboratory and field compaction tests are:

- dry “density” or dry “unit weight.”
- compaction water content.
- type of energy input, e.g., impact, static, vibratory, kneading.
- amount of energy input expressed in ft-lbs/ft^3.

Table 5-13 presents a summary of the characteristics of the most commonly used laboratory compaction tests. Figure 5-30 shows a typical hammer and a mold which is used for performing compaction tests in the laboratory. A comparison of the various values in Table 5-13 reveals that the energy level in the Modified Proctor compaction (MPC) test is 4.5 times that for the Standard Proctor compaction (SPC) test.

Table 5-13
Characteristics of laboratory compaction tests

Common Name	ASTM (AASHTO) Designation	Mold Dimensions			Hammer		No. of Layers	Blows/ Layer	Energy (ft-lbs/ft^3)
		Diam. (in)	Height (in)	Vol. (ft^3)	Wt. (lbs)	Drop Ht. (in)			
Standard Proctor	D 698 (T 99)	4	4½	1/30	5.5	12	3	25	12,375
Modified Proctor	D 1557 (T 180)	4	4½	1/30	10	18	5	25	56,250
Note: Both tests are performed on minus No. 4 (4.75 mm) fraction of the soil.									

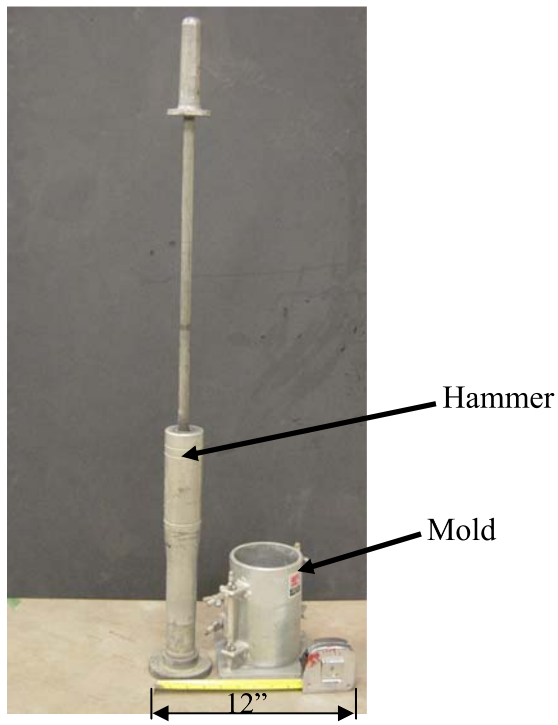


Figure 5-30. Hammer and mold for laboratory compaction test (tape measure is for scale purpose only).

5.8.2 Test Procedures

At least 3 (preferably 5) samples of the same type of soil are prepared at various water contents and compacted according to the requirements listed in Table 5-13. Following compaction, the moist unit weight of the compacted soil (γ_t) in the mold is easily calculated as the weight of the soil (measured) divided by the volume of the mold (constant = 1/30 ft^3). The water content (w) is determined as per ASTM D 2216-05 and the dry unit weight is then calculated as (see Table 2-2 in Chapter 2):

$$\gamma_d = \frac{\gamma_t}{(1+w)} \quad \text{5-17}$$

The dry unit weight (pcf (kN/m^3)) for each compacted sample is plotted versus its compaction moisture content (%). The resulting curve is called a **compaction curve**. Figure 5-31 shows compaction curves for the same soil using Standard Proctor compaction (SPC) test parameters and Modified Proctor compaction (MPC) test parameters as listed in Table 5-13. The typical compaction curves as presented in Figure 5-31 have the following characteristics:

- **Maximum dry density** ($\gamma_{d\text{-max}}$) is the dry density corresponding to the peak of the compaction curve for a given type and amount of input energy. Note from Figure 5-31, that the SPC $\gamma_{d\text{-max}}$ is less than the MPC $\gamma_{d\text{-max}}$. Note from Table 5-13 that although the type of energy (impact) is the same for both SPC and MPC, the amount of energy in the MPC test is 4.5 times that of the SPC test.

- **Optimum moisture content** (w_{opt}) is the compaction water content at which the soil attains its maximum dry density for a given input energy. Note from Figure 5-31, that the SPC w_{opt} is greater than the MPC w_{opt}.

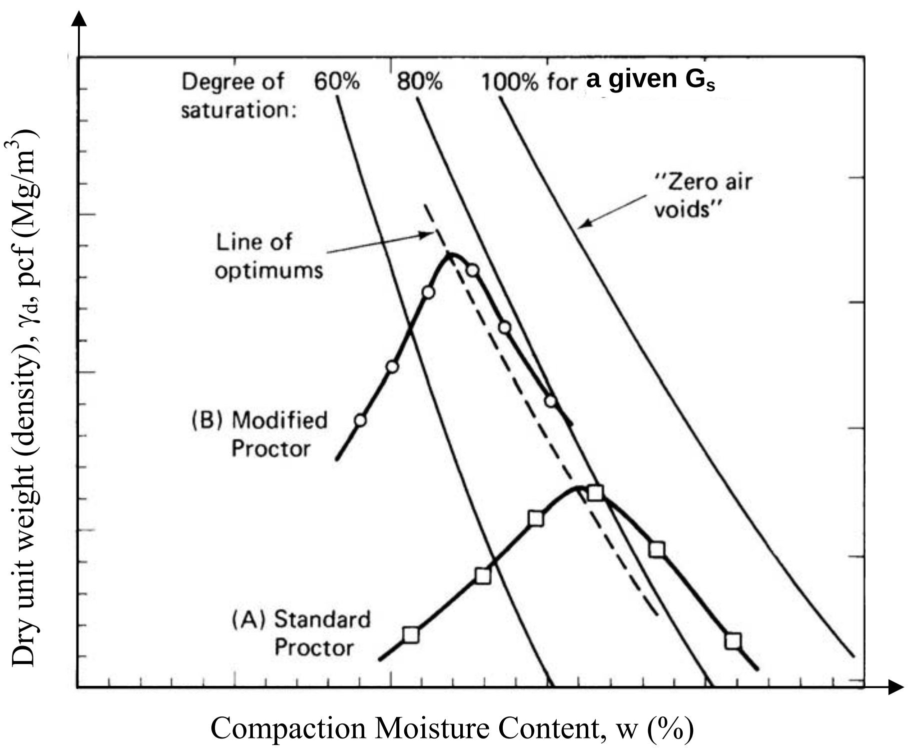


Figure 5-31. Compaction curves (after Holtz and Kovacs, 1981).

- **Zero air voids curve** is the curve that corresponds to S=100% regardless of the amount or type of energy input. The importance of the zero-air-voids curve is that it denotes the limits of compaction, i.e., if the moisture content of a fill is too high for a given amount of input energy, the compacted fill may begin to "pump" as its voids become fully saturated with moisture. This can happen even at low moisture contents if the input energy is very large as may be the case with too many passes of a too heavy a piece of compaction equipment. Points on the zero air voids curve are calculated from the basic equation for dry unit weight given by Equation 5-18 by setting S=1 and choosing arbitrary values of compaction moisture content within the range of the compaction curve.

$$\gamma_d = \frac{G_s\,\gamma_w}{(1+e)} = \frac{G_s\,\gamma_w}{\left(1+\dfrac{wG_s}{S}\right)} \qquad \text{5-18}$$

where: G_s = specific gravity of solid particles
γ_w = unit weight of water
e = void ratio
w = water content expressed as a decimal
S = degree of saturation expressed as a decimal.

Note that the S=100% (zero air voids) curve is calculated for a specific value of G_s, in this case 2.7. Curves corresponding to other degrees of saturation can be calculated in the same way by setting S=80 for the 80% saturation curve, S=60 for the 60% saturation curve and so forth. The saturation curve for a degree of saturation less than 100% is often useful for developing compaction specifications for silty soils since such soils frequently have sharply peaked compaction curves. Therefore, they can begin to "pump" even though the degree of saturation is less then 100%.

- **Line of optimums** - As its name suggests the "line of optimums" is obtained by passing a curve through the peaks of the compaction curves that were developed for a certain type of soil compacted at various energy input levels. Testing laboratories frequently develop such curves for various types of soil based on information in their job files. The line of optimums can be used as a guide for developing compaction specifications where no laboratory test data are available.

The above observations are true for all types of soils and apply to all methods of compaction. The most important concept about compaction curves as discussed above is that an increase in the amount of compaction (more energy) results in an increase in the maximum dry density and a corresponding decrease in the optimum moisture content. Therefore, this concept should be recognized when the geotechnical specialist is required to develop specifications for field compaction of soils.

5.8.3 Implication of Laboratory Tests on Field Compaction Specifications

With reference to Figure 5-31 it is obvious that for a given compaction curve the same dry unit weight can be obtained at two different compaction moisture contents, one below optimum and the other above optimum. For fine-grained soils this difference in moisture contents relates to a difference in soil structure that may affect engineering properties such as shear strength and permeability.

It is very important that compaction specifications be given in terms of three parameters: the compaction energy (Standard or Modified Proctor), the desired dry density expressed as a percentage of the maximum dry density, and the compaction moisture content expressed as a range (+ or -) with respect to the optimum moisture content. For example, since the input energy of Modified Proctor is greater than the input energy of Standard Proctor (see Table 5-13) the Modified Proctor curve plots above the Standard Proctor curve so that 95% of MPC $\gamma_{d\text{-}max}$ may be greater than 100% of SPC $\gamma_{d\text{-}max}$. Likewise, a compaction moisture content of 1 or 2% above optimum for modified Proctor compaction may be below the standard Proctor optimum moisture content.

Unfortunately, laboratory compaction curves mainly serve as guidelines for field compaction. This approach is inconsistent because the impact type of energy input in the laboratory is not the same as the type of energy delivered by the equipment commonly used in construction. Figure 5-32 illustrates this point by presenting the types of compactive effort (static, vibratory, kneading) corresponding to the equipment typically used in practice. Note that <u>none of the compaction processes</u> in Figure 5-32 involves impact type of energy that is used to determine the compaction characteristics of the soils in a laboratory SPC or MPC test.

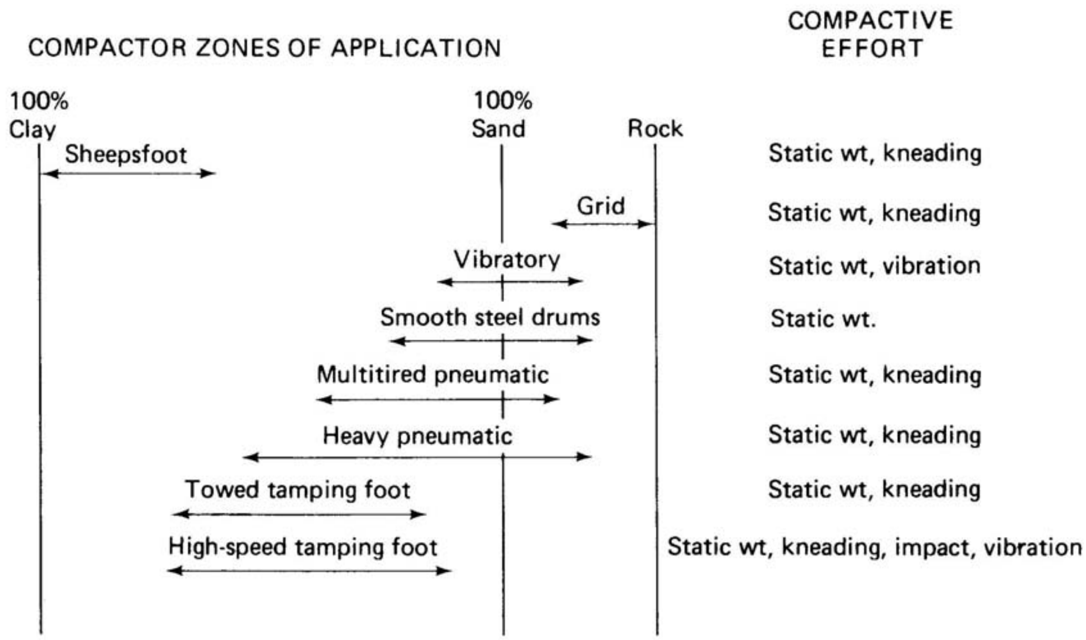


Figure 5-32. Compactors recommended for various types of soil and rock (Schroeder, 1980).

Due to the obvious disconnect between the types of energy in the laboratory and the field, some method is needed to express the laboratory-measured compaction parameters, i.e., maximum dry unit weight ($\gamma_{d\text{-}max}$) and optimum moisture content (w_{opt}), in terms of field compaction. Most commonly, this relationship is achieved by so-called **performance based or end-product specifications** wherein a certain **relative compaction, RC,** also known as **percent compaction**, is specified. The RC is simply the ratio of the desired field dry unit weight, $\gamma_{d\ field}$, to the maximum dry density measured in the laboratory, $\gamma_{d\ max}$, expressed in percent as follows:

$$RC = \frac{\gamma_{d\,field}}{\gamma_{d\,max}} \times 100\% \qquad 5\text{-}19$$

The relative compaction, RC, is not the same as relative density, D_r, that was defined in Chapter 2. Relative density applies only to granular soils with fines less than 12% (ASTM D 2049), while relative compaction is used across a wide variety of soils. Lee and Singh (1971) published the following relationship between RC and D_r based on a statistical evaluation of 47 different granular soils compacted by using Modified Proctor energy (Wright, *et al.*, 2003).

D_r = 0% for RC = 80%
D_r = 100% for RC = 100%

Assuming a linear interpolation, the above relationship can be expressed as follows:

$$D_r(\%) = 5[RC(\%) - 80] \qquad \text{5-20}$$

or

$$RC(\%) = 80 + \frac{D_r(\%)}{5} \qquad \text{5-21}$$

In terms of Standard Proctor, Equations 5-20 and 5-21 are approximately as follows:

$$D_r(\%) = 5[RC(\%) - 85] \qquad \text{5-22}$$

or

$$RC(\%) = 85 + \frac{D_r(\%)}{5} \qquad \text{5-23}$$

Figure 5-33 presents the above equations in a graphical format. Table 5-14 presents the values of D_r for values of RC values ranging from 85% to 100% for MPC and from 90% to 105% for SPC.

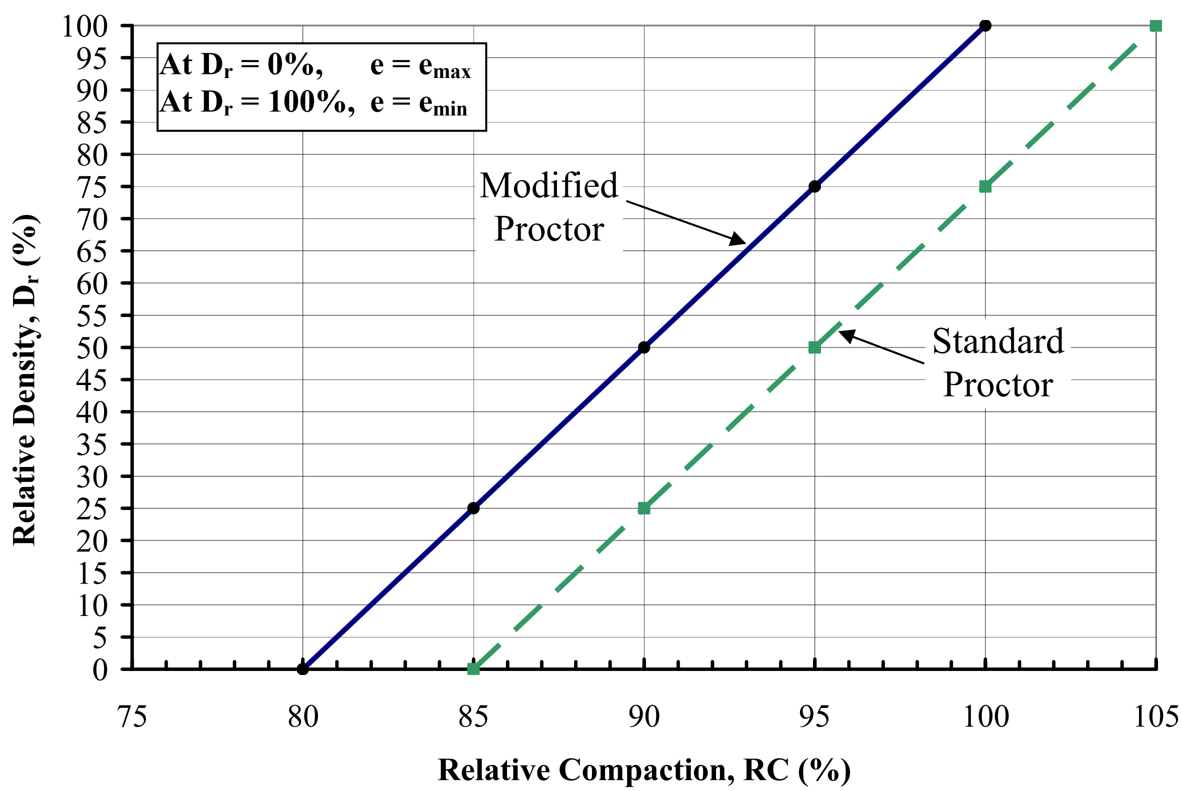


Figure 5-33. Relative density, relative compaction and void ratio concepts.

Table 5-14
Some values of D_r as a function of RC
based on Modified and Standard Proctor Compaction Test

RC (%) MPC (SPC)*	D_r (%)	RC (%) MPC (SPC)*	D_r (%)	RC (%) MPC (SPC)*	D_r (%)
85 (90)	25	90 (95)	50	95 (100)	75
86 (91)	30	91 (96)	55	96 (101)	80
87 (92)	35	92 (97)	60	97 (102)	85
88 (93)	40	93 (98)	65	98 (103)	90
89 (94)	45	94 (99)	70	99 (104)	95
				100 (105)	100
* MP: Modified Proctor; SP: Standard Proctor					

Figure 5-33 and Table 5-14 indicates that for every 1% increase in RC, the increase in D_r is 5% regardless of compaction energy. This is rather significant when it is realized that the shear strength parameter, ϕ, of granular soils is a direct function of relative density as shown in Figure 5-22 and as illustrated by the following simple computations:

- Based on Figure 5-22, the angle of internal friction for well-graded sands (SW soils) for values of D_r between 50% and 100% varies from 33º to 41º. From Table 5-14, values of D_r between 50% and 100% correspond to RC values of 90 and 100%, respectively for Modified Proctor and 95 and 105%, respectively for Standard Proctor. In other words, for SW soils, for every 1% increase in RC, the angle of internal friction increases by 0.8º.

- Alternatively, the increase in the coefficient of friction, tan ϕ, would be tan (41º)/ tan (33º) = 1.33 or a 33% increase over a 10% change in RC. In other words, there is a 3.3% increase in shear strength for every 1% increase in RC.

Select materials are often specified in the construction of transportation facilities such as embankments, foundations, and pavement sub-bases and bases. The select materials are granular soils as discussed in Chapters 6 (Slope Stability), 7 (Approach Roadway Deformations), 8 (Shallow Foundations) and 10 (Earth Retaining Structures). The above simple example illustrates the importance of carefully specifying RC for such materials. RC values of 90 to 100% of standard Proctor values are commonly used. Based on Table 5-14, this range of RC corresponds to a D_r between 25% and 75%.

For most transportation applications, the RC value is prescribed in **performance based specifications**. In this case, it does not matter which equipment or type of compaction energy the contractor chooses to use as long as the end-product meets the specified RC. The prudent contractor would choose the equipment according to the type of soil. Often the contractor chooses to use the equipment he/she owns or is cheapest to lease or rent. Unfortunately, this equipment may not always be the most efficient equipment for the work. Figure 5-32 can be used as a preliminary guide in selecting the type of equipment and mode of compaction energy as a function of soil type. In Figure 5-32 the "100%" above the word clay on the left and the word sand on the right indicates boundaries for the range of soils types in between, e.g. 100% clay means that the soil to be compacted is all fine grained, therefore use of a sheepsfoot roller is recommended. The figure also suggests that a sheepsfoot roller can be used for various soil mixtures consisting of up to approximately 35% fine grained soils and 70 % coarse grained soils.

An example of the influence of the choice of compaction equipment and energy is shown in Figure 5-34. Assume that Curve 1 is obtained from laboratory tests to develop the compaction curve for a borrow material that the contractor has identified for a given project. Further assume that the specification for the project requires that RC = 90%. If M represents the point of maximum dry density, γ_{dmax}, then RC=90% would mean that Points P and Y represent the limits of Curve 1 within which the contractor has to operate. In other words, the contractor cannot use compaction moisture contents less than a or c on the compaction moisture content axis.

To properly evaluate the choice of the compaction equipment, the contractor should perform compaction tests at various RC values in the laboratory to develop a line of optimums and a family of curves similar to Curve 2 and 3 shown in Figure 5-34. Once this data is developed, then it can be observed from Figure 5-34, that the most economical water content would be that corresponding to point R along the line of optimums, i.e., the moisture content given by Point b on the X-axis. Point R represents the minimum compactive effort to attain RC=90%. To avoid inadequate compaction and risk failed field quality control tests, a prudent contractor usually aims to achieve somewhat higher dry density. Thus, the contractor often chooses to select a target curve similar to Curve 2 and aim to maintain moisture content in Zone B.

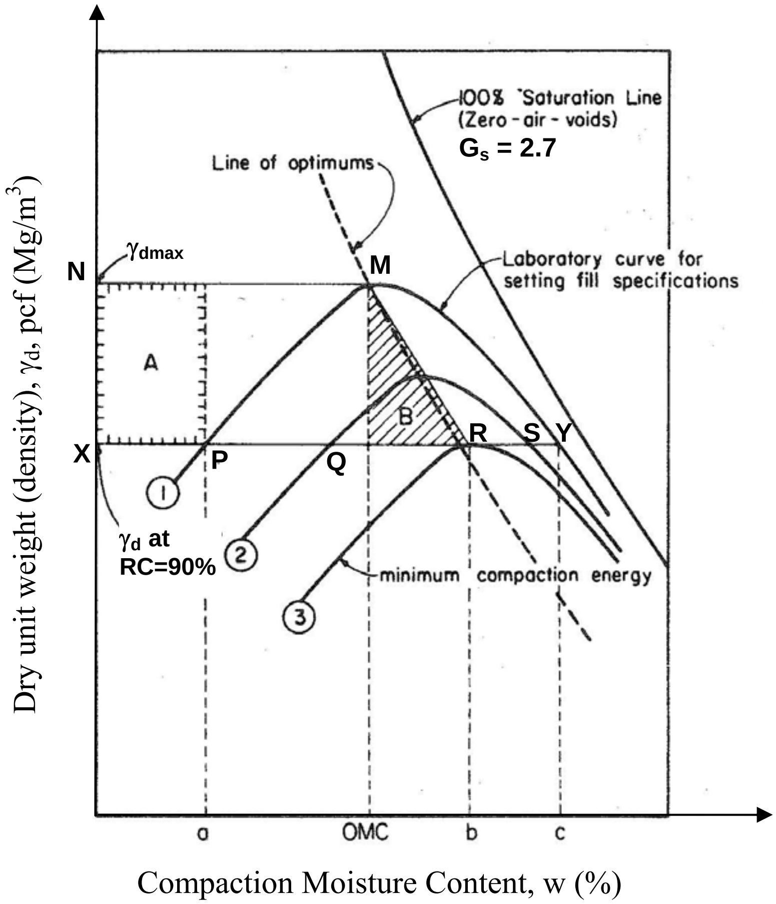


Figure 5-34. Example evaluation of economical field compaction conditions (after Bowles, 1979)

5.8.4 Engineering Characteristics of Compacted Soils

Typical values for the engineering characteristics of compacted soils are given in Table 5-15. The values of the engineering properties refer to soils compacted to maximum dry density by using the standard Proctor test. The data in Table 5-15 are based on more than 1500 soil tests performed by the Bureau of Reclamation in Denver, CO. The large majority of soils were from 17 western states in the U.S. with some foreign soils (USBR, 1960). The background information for the values in Table 5-15 is given in the notes section of the table.

Table 5-15
Average engineering properties of compacted inorganic soils (after USBR, 1960)

USCS	Standard Proctor Compaction (ASTM D 698/AASHTO T 99)		As Compacted Cohesion, c' psi (kPa)	Saturated Cohesion, c'_{sat} psi (kPa)	Friction Angle, ϕ' (deg)	Void Ratio, e [Permeability, k (ft/yr)]
	Maximum Dry Density, pcf (kN/m^3)	Optimum Moisture Content (%)				
GW	>119 (>18.7)	<13.3	*	*	>38	* [27,000±13,000]
GP	>110 (>17.3)	<12.4	*	*	>37	* [64,000±34,000]
GM	>114 (>17.9)	<14.5	*	*	>34	* [>0.3]
GC	>115 (>18.1)	<14.7	*	*	>31	* [>0.3]
SW	119±5 (18.7±0.8)	13.3±2.5	5.7±0.6 (39±4)	*	38±1	0.37±* [*]
SP	110±2 (17.3±0.3)	12.4±1.0	3.3±0.9 (23±6)	*	37±1	0.50±0.03 [>15.0]
SM	114±1 (17.9±0.2)	14.5±0.4	7.4±0.9 (51±6)	2.9±1.0 (20±7)	34±1	0.48±0.02 [7.5±4.8]
SM-SC	119±1 (18.7±0.2)	12.8±0.5	7.3±3.1 (50±21)	2.1±0.8 (14±6)	33±4	0.41±0.02 [0.8±0.6]
SC	115±1 (18.1±0.2)	14.7±0.4	10.9±2.2 (75±15)	1.6±0.9 (11±6)	31±4	0.48±0.01 [0.3±0.2]
ML	103±1 (16.2±0.2)	19.2±0.7	9.7±1.5 (67±10)	1.3±* (9±*)	32±2	0.63±0.02 [0.59±0.23]
ML-CL	109±2 (17.1±0.3)	16.8±0.7	9.2±2.4 (63±17)	3.2±* (22±*)	32±3	0.54±0.03 [0.13±0.07]
CL	108±1 (17.0±0.2)	17.3±0.3	12.6±1.5 (87±10)	1.9±0.3 (13±2)	28±2	0.56±0.01 [0.08±0.03]
MH	82±4 (12.9±0.6)	36.3±3.2	10.5±4.3 (72±30)	2.9±1.3 (20±9)	25±3	1.15±0.12 [0.16±0.10]
CH	94±2 (14.8±0.3)	25.5±1.2	14.9±4.9 (103±34)	1.6±0.86 (11±6)	19±5	0.80±0.04 [0.05±0.05]

Notes:
1. The entry ± indicates 90 percent confidence limits of the average value; * denotes insufficient data.
2. For permeability, 1 ft/yr ≈ 10^{-6} cm/sec.
3. All shear strengths, void ratios and permeabilities were determined on samples prepared at Standard Proctor maximum dry density and optimum moisture content.
4. The values of cohesion, c', and friction angle, ϕ', are based on a straight-line Mohr strength envelope on an effective stress basis. The value c'_{sat}, was obtained by saturating the sample and shearing it to failure. Consolidated-undrained (CU) triaxial tests were used to determine all the shear strengths.
5. Since all laboratory tests, except large-sized permeability tests, were performed on the minus No. 4 (4.75 mm) fraction of soil, data on average values for gravels are not available for most properties. However, an indication as to whether these average values will be greater than or less than the average values for the corresponding sand group are given in the table (note entries with > or < symbol).
6. Void ratio was derived from the maximum dry density and specific gravity of the soil.
7. In USCS, there are no upper boundaries of liquid limit of MH and CH soils. The maximum limits for MH and CH soils tested by USBR (1960) were 81% and 88%, respectively. Soils with higher liquid limits than these will have inferior engineering properties.

5.8.4.1 Effect of Increase in Moisture Content on Shear Strength of Compacted Soils

The cohesion values, c' (as-compacted) and c'_{sat} (after saturation of compacted soil) listed in Table 5-15 are instructive in the context of the apparent cohesion concept discussed in Section 5.5.1.2. In the soil's compacted state at optimum moisture content (OMC), the capillary stresses and the apparent mechanical forces assume their peak values at that particular compaction energy. Capillary stress, as discussed in Section 5.5.1.2, is due to surface tension in the water between individual soil grains. The magnitude of capillary stress is larger in fine-grained soils than coarse-grained soils as demonstrated by the increasing values of c' in Table 5-15 as the soil type changes from granular to fine-grained.

The same trend is observed with the c'_{sat} values. However, the values of c'_{sat} are approximately 10% (for CH soils) to 40% (for SM soils) of the corresponding c' values. This drastic reduction in cohesive strength is attributable to the effect of capillary stresses being significantly reduced by the increase in moisture content required to reach saturation resulting in much lower apparent cohesive strengths. The reduction may also represent loss of apparent mechanical forces due to reduction in the interlocking of the particles because of the lubricating effect of water.

Based on the above discussions, it is important to ensure that compacted soils are protected against increases in moisture content because the strength of such soils will decrease with associated detrimental effects on the facilities they support.

5.9 ELASTIC PROPERTIES OF SOILS

The stress-strain behavior of soils and rocks is highly nonlinear or inelastic. However, as indicated in Chapter 2, elastic theory provides a convenient first order approximation to stresses and strains induced in soils by external loads. A pair of elastic constants is required when elastic theory is used to solve such problems, e.g., elastic modulus (E) and Poisson's ratio (ν), or shear modulus (G) and bulk modulus (B), or some other pair of elastic constants. The pair of E and ν is most widely used since both parameters are readily measurable. Consequently, many of the elastic equations in geotechnical engineering are formulated with this pair. Therefore, typical values of E and ν for soils are presented in this section.

The elastic properties of soils may be measured from laboratory stress-strain curves such as those shown in Figure 5-16. The elastic properties, E_s and ν, of a soil may be estimated from empirical relationships presented in Table 5-16 for **preliminary** design or for final design where the prediction of deformation is not critical to the performance of the structure, i.e., when the structural design can tolerate the potential inaccuracies inherent in the correlations. The definition of E_s is not always consistent for the various correlations and methods of in-situ measurement. FHWA (2002a) provides additional details regarding the definition and determination of E_s. Where evaluation of elastic settlement is critical to the design of the foundation or selection of the foundation type, in-situ methods such as pressuremeter or dilatometer tests should be used for evaluating the modulus of the impacted strata.

The modulus of elasticity for normally consolidated cohesionless soils tends to increase with depth. An alternative method of defining the soil modulus for granular soils is to assume that the modulus, E_s, increases linearly with depth, starting at zero at the ground surface, in accordance with the following equation:

$$E_s \text{ (tsf)} = n_h \times z \qquad \text{5-24}$$

where: n_h = rate of increase of soil modulus with depth as defined in Table 5-17 (tsf/ft)
z = depth in feet below the ground surface (ft)

The formulation provided in Equation 5-24 is used primarily for analysis of lateral response or buckling of deep foundations.

5.10 COMMON SENSE GUIDELINES FOR LABORATORY TESTING OF SOILS

Sampling and testing of soils is one of the first and most important steps in the design and construction of all types of structures. Omissions or errors introduced here, if undetected, will be carried through the process of design and construction and will often result in costly and possibly unsafe facilities. Table 5-18 lists topics that should be considered for proper handling of samples, preparation of test specimens, and laboratory test procedures. Table 5-18 should in no way be construed as being a complete list of guidelines to avoid possible errors and omissions in handling or testing of soil specimens; there are more. These are just some of the more common ones.

Table 5-16
Elastic constants of various soils (after AASHTO 2004 with 2006 Interims)

Soil Type	Typical Range of Young's Modulus Values, E_s (tsf)	Poisson's Ratio, ν
Clay:		
Soft sensitive	25-150	0.4-0.5 (undrained)
Medium stiff to stiff	150-500	
Very stiff	500-1,000	
Loess	150-600	0.1-0.3
Silt	20-200	0.3-0.35
Fine Sand:		
Loose	80-120	0.25
Medium dense	120-200	
Dense	200-300	
Sand:		
Loose	100-300	0.20-0.36
Medium dense	300-500	
Dense	500-800	0.30-0.40
Gravel:		
Loose	300-800	0.20-0.35
Medium dense	800-1,000	
Dense	1,000-2,000	0.30-0.40

Estimating E_s from SPT N-value

Soil Type	E_s (tsf)
Silts, sandy silts, slightly cohesive mixtures	$4\ N1_{60}$
Clean fine to medium sands and slightly silty sands	$7\ N1_{60}$
Coarse sands and sands with little gravel	$10\ N1_{60}$
Sandy gravel and gravels	$12\ N1_{60}$

Estimating E_s (tsf) from q_c static cone resistance

Sandy soils	$2q_c$ where (q_c is in tsf)

Note: 1 tsf = 95.76 kPa

Table 5-17
Rate of increase of soil modulus with depth n_h (tsf/ft) for sand
(AASHTO 2004 with 2006 Interims)

Consistency	Dry or Moist	Submerged
Loose	30	15
Medium	80	40
Dense	200	100
Note: 1 tsf/ft = 314.7 kPa/m		

Table 5-18

Common sense guidelines for laboratory testing of soils

1. Protect samples to prevent moisture loss and structural disturbance.
2. Carefully handle samples during extrusion; samples being extruded should be properly supported upon their exit from the tube.
3. Avoid long term storage of soil samples in Shelby tubes.
5. Properly number and identify samples.
5. Store samples in properly controlled environments.
6. Visually examine and identify soil samples after removal of smear from the sample surface.
7. Use pocket penetrometer or miniature vane only for an indication of consistency not strength.
8. Carefully select “representative” specimens for testing.
9. Have a sufficient number of samples to select from.
10. Always consult the field logs for proper selection of samples.
11. Recognize disturbances caused by sampling, the presence of cuttings, drilling mud or other foreign matter.
12. Do not depend solely on the visual identification of soils for classification.
13. Always perform organic content tests when classifying soils as peat or organic. Visual classifications of organic soils may be very misleading.
15. Do not dry soils in overheated or underheated ovens.
15. Discard old worn-out equipment; old sieves for example, particularly fine (<No. 40) mesh ones need to be inspected and replaced often; worn compaction molds or compaction hammers should be checked and replaced if needed. An error in the volume of a compaction mold is amplified 30x when translated to unit volume.
16. Performance of Atterberg limits tests requires carefully adjusted drop height of the liquid limit machine and proper rolling of plastic limit specimens.
17. Do not use tap water for tests where distilled water is specified.
18. Properly cure stabilization test specimens.
19. Never assume that all samples are saturated as received.
20. Perform saturation by applying properly staged back pressures of adequate magnitude.
21. Use properly fitting o-rings, membranes, etc. in triaxial or permeability tests.
22. Evenly trim ends and sides of undisturbed samples.
23. Be careful to identify and report slickensides and natural fissures.
25. Do not mistakenly identify failures due slickensides as shear failures.
25. Do not use stress-strain curves from unconfined compression test results to determine elastic moduli.
26. Incremental loading of consolidation tests should be performed only after the completion of the primary stage.
27. Use proper loading rate for strength tests.
28. Do not guesstimate e-log p curves from accelerated, incomplete consolidation tests.
29. Avoid "reconstructing" soil specimens, disturbed by sampling or handling, for undisturbed testing.
30. Correctly label all laboratory test specimens.
31. Do not take shortcuts by using non-standard equipment or non-standard test procedures.
32. Periodically calibrate testing equipment and maintain calibration records.
33. Always test a sufficient number of samples to obtain representative results in variable material.

34 Take proper precautions to assure the safety of personnel when performing any test procedure.

5.11 LABORATORY TESTS FOR ROCK

5.11.1 Introduction

This section provides information on common laboratory test methods for rock including testing equipment, general procedures related to each test, and parameters measured by the tests. Table 5-19 provides a list of commonly performed laboratory tests for rock associated with typical projects for highway applications. Although other laboratory test methods for rock are available including triaxial strength testing, rock tensile strength testing, and durability testing related to rock soundness, most design procedures for structural foundations and slopes on or in rock are developed based on empirical rules related to RQD, degree of fracturing, and the unconfined compressive strength of the rock. The use of more sophisticated laboratory testing for rock properties is usually limited to the most critical projects. Details on other laboratory testing procedures for rock are provided in FHWA, 1997). Table 5-20 provides summary information on the typical rock index and performance tests.

Table 5-19
Common rock tests performed in the laboratory

Test Category	Name of Test	ASTM Test Designation
Point Load Strength	Suggested method for evaluating point-load strength	D 5731
Compressive Strength	Compressive strength of intact rock core specimen (in unconfined compression)	D 2938
Direct Shear Strength	Laboratory direct shear strength tests for rock specimens under constant normal stress	D 5607
Durability	Slake durability of shales and similar weak rocks	D 4644
Strength-Deformation	Elastic moduli of intact rock core specimens in uniaxial compression	D 3148

Table 5-20
Summary information on laboratory test methods for rock (FHWA, 2002a)

Test	Procedure	Applicable Rock Types	Applicable Rock Properties	Limitations / Remarks
Point-Load Strength Test	Rock specimens in the form of core, cut blocks, or irregular lumps are broken by application of concentrated load through a pair of spherically truncated, conical platens.	Generally not appropriate for rock with uniaxial compressive strength less than 520 ksf (25 MPa)	Provides an index of uniaxial compressive strength	Can be performed in the field with portable equipment or in the laboratory; in soft or weak rock, test results need to be adjusted to account for platen indentation
Unconfined Compressive Strength of Intact Rock Core	A cylindrical rock specimen is placed in a loading apparatus and sheared under axial compression with no confinement until peak load and failure are obtained.	Intact rock core	Uniaxial compressive strength	Simplest and fastest test to evaluate rock strength; fissures or other anomalies will often cause premature failure
Laboratory Direct Shear Test	A rock specimen is placed in the lower half of the shear box and encapsulated in either synthetic resin or mortar. The specimen must be positioned so that the line of shear force lies in the plane of the discontinuity to be investigated. The specimen is then mounted in the upper shear box and the normal load and shear force are applied.	Used to assess peak and residual shear strength of discontinuity	Peak and residual shear strength	May need to perform in-situ direct shear test if design is controlled by potential slip along a discontinuity filled with very weak material
Elastic Moduli of Intact Rock Core	Procedure is similar to that for unconfined compressive strength of intact rock. Lateral strains are also measured	Intact rock core	Modulus and Poisson's ratio	Modulus values (and Poisson's ratio) vary due to nonlinearity of stress-strain curve.
Slake Durability	Dried fragments of rock are placed in a drum made of wire mesh that is partially submerged in distilled water. The drum is rotated, the sample dried, and the sample is weighed. After two cycles of rotating and drying, the weight loss and the shape of size of the remaining rock fragments are recorded.	Shale or other soft or weak rocks	Index of degradation potential of rock	

5.11.2 Point-Load Strength Test

The point load strength test is used to estimate the unconfined compressive strength of rock. Both core samples and fractured rock samples can be tested. The test is conducted by compressing a piece of the rock between two points on cone-shaped platens (see Figure 5-35) until the rock specimen breaks in tension between these two points. Each of the cone points has a 1/5 in (5 mm) radius of curvature and the cone bodies themselves include a 60° apex angle. The equipment is portable, and tests can be carried out quickly and inexpensively in the field. Because the point load test provides an index value for the compressive strength, usual practice is to calibrate the results with a limited number of uniaxial compression tests on prepared core samples. The point load test is also used with other index values to assess the degradation potential of shales.

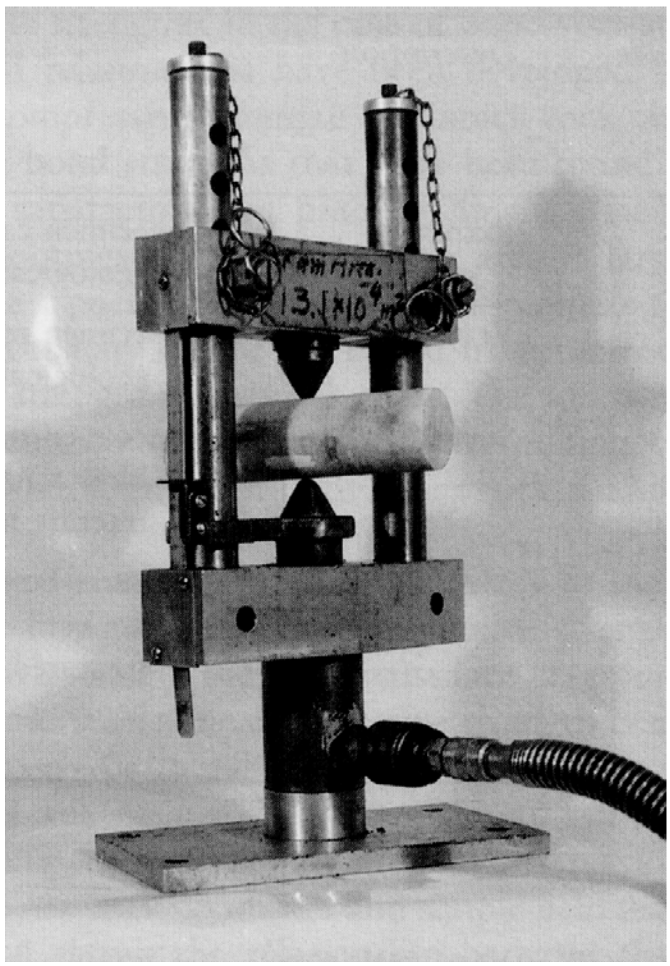


Figure 5-35. Point load strength test equipment (Wyllie, 1999).

If the distance between the contact points of the platens is D and the breaking load is P, then the point load strength, I_s, is calculated as:

$$I_s = \frac{P}{D_e^{\,2}} \qquad 5\text{-}25$$

where D_e is the equivalent core diameter given by:

(1) $D_e = D^2$ for diametral tests; or
(2) $D_e = 4\times A$ for axial, block, or lump tests where $A = W\times D$. The area A is the minimum cross-sectional area of a lump sample for a plane through the platen contact points where W is the specimen width.

The size-corrected point load strength index, $I_{s(50)}$ of a rock specimen is defined as the value of I_s that would have been measured by a diametral test with D = 2 in (50 mm). For tests performed on specimens other than 2 in (50 mm) in diameter, the results can be standardized to the size-corrected point load strength index according to:

$$I_{s(50)} = k_{PLT}\, I_s \qquad 5\text{-}26$$

The value of the size correction factor, k_{PLT}, is given by:

$$k_{PLT} = \left(\frac{D}{50}\right)^{0.45} \ (D \text{ in mm}) \qquad 5\text{-}27$$

It has been found that, on average, the uniaxial compressive strength, σ_c, is about 20 to 25 times the point load strength index, with a value of 24 commonly used, i.e.,

$$\sigma_c = 24\, I_{s(50)} \qquad 5\text{-}28$$

However, tests on many different types of rock show that the $\sigma_c / I_{s(50)}$ ratio can vary between 15 and 50, especially for anisotropic rocks. Consequently, the most reliable results are obtained if a series of uniaxial calibration tests are carried out. Point load test results are not acceptable if the failure plane lies partially along a pre-existing fracture in the rock, or is not coincident with the line between the platens. For tests in weak rock where the platens indent the rock, the test results should be adjusted by measuring the amount of indentation and correcting the distance D (Wyllie, 1999).

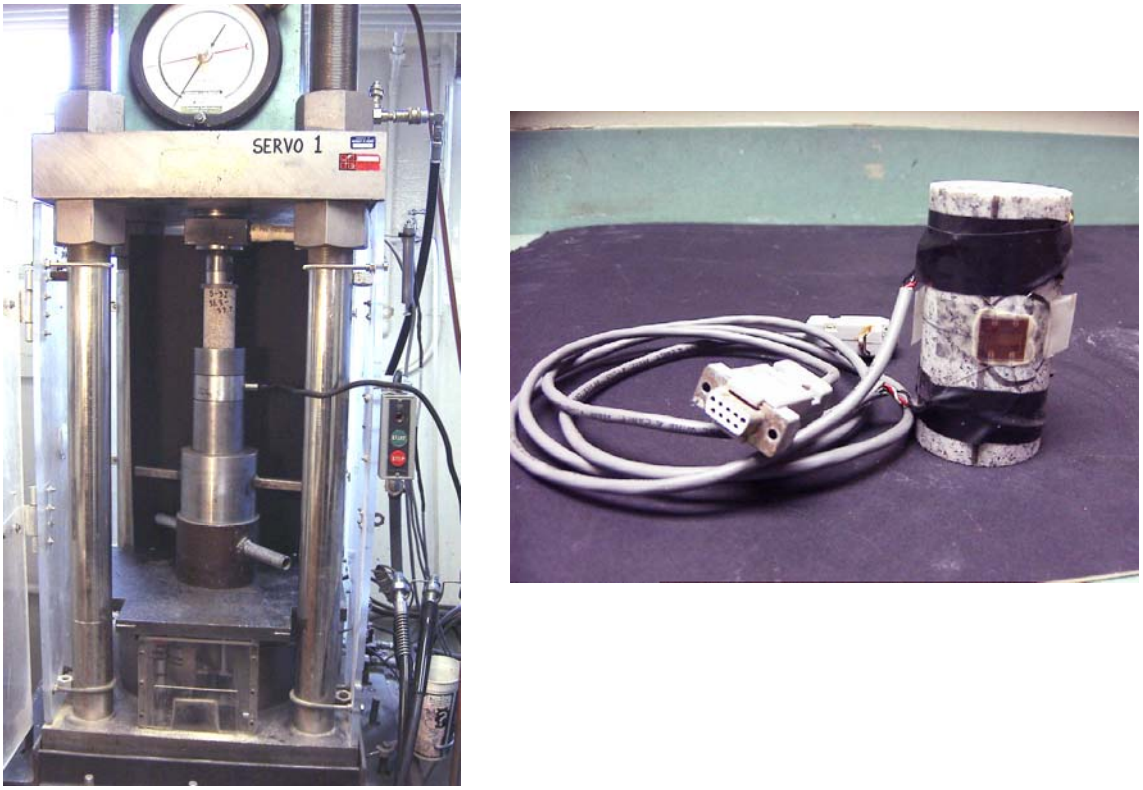
5.11.3 Unconfined Compressive Strength of Intact Rock Core

The unconfined compressive strength of intact rock core can be evaluated by using ASTM D 2938. In this test, rock specimens of regular geometry, generally rock cores, are used. The rock core specimen is cut to length so that the length to diameter ratio is 2.5 to 3.0 and the ends of the specimen are machined flat. ASTM D 2938 provides tolerance requirements related to the flatness of the ends of the specimen, the perpendicularity of the ends of the specimens, and the smoothness of the length of the specimen. The specimen is placed in a loading frame, see Figure 5-36a. Axial load is then continuously applied to the specimen at a uniform rate until peak load and failure are obtained. The unconfined or uniaxial compressive strength of the specimen is calculated by dividing the maximum load carried by the specimen during the test by the initial cross-sectional area of the specimen.

This test is more expensive than the point load strength test, but it is also more accurate with respect to in situ strength. Careful consideration of the design requirements should be made before deciding which test to perform, the unconfined compression test, a performance test, or the simpler point load strength test, an index test.

5.11.4 Elastic Modulus of Intact Rock Core

The test to determine the elastic modulus of intact rock is performed similarly to the unconfined compressive test discussed previously, except that deformation is monitored during application of load. This test is performed when it is necessary to estimate both the elastic modulus and the Poisson's ratio of the intact rock core. Because of this dual purpose, it is common to measure both axial (or vertical) and lateral (or diametral) strain during compression. It is preferable to use strain gauges glued directly to the rock surface (see Figure 5-36b) as compared to LVDTs mounted on the platens since slight imperfections at the contact between the platens and the rock may lead to movements that are not related to strain in the rock (Wyllie, 1999).



(a) (b)

Figure 5-36. (a) Unconfined compression strength test on intact rock core, (b) Use of strain gage on intact rock core sample for measurement of stress-strain characteristics. (Photographs courtesy of Geomechanics Laboratory, University of Arizona).

5.11.5 Laboratory Direct Shear Test

The apparatus and procedures for direct shear testing are discussed in ASTM D 5607. The direct shear test is typically used to evaluate the shear strength of a rock discontinuity. Overall, the equipment for the direct shear test on rock is similar to that for soil including a direct shear testing machine, a device for applying normal pressure, and vertical and horizontal displacement monitoring devices. A schematic of the test set up is shown in Figure 5-37. For testing rock specimens, an encapsulating material such as a high strength gypsum cement is poured around the specimen in the upper and lower holding ring. The specimen is sheared as one holding ring is displaced horizontally with respect to the other such that the discontinuity surface is exactly parallel to the direction of the shear load. Load cells are used to monitor the shear force and LVDTs or dial gauges are used to monitor both horizontal and vertical deformation. Multiple LVDTs should be used to monitor vertical deformation and potential rotation of the specimen in the vertical plane.

Typically, the results of a direct shear test on rock are presented on two separate plots: one a plot of shear stress versus shear displacement and the other a plot of normal displacement versus shear displacement. Normal stresses should be adjusted to account for potential decreases in the shear contact area. After the sample is sheared, the sample is reset to its original position, the normal load is increased, and another test is performed. Each test will produce a pair of shear stress and normal stress values for both peak and residual conditions. The friction angle of the discontinuity surface can be evaluated from this data.

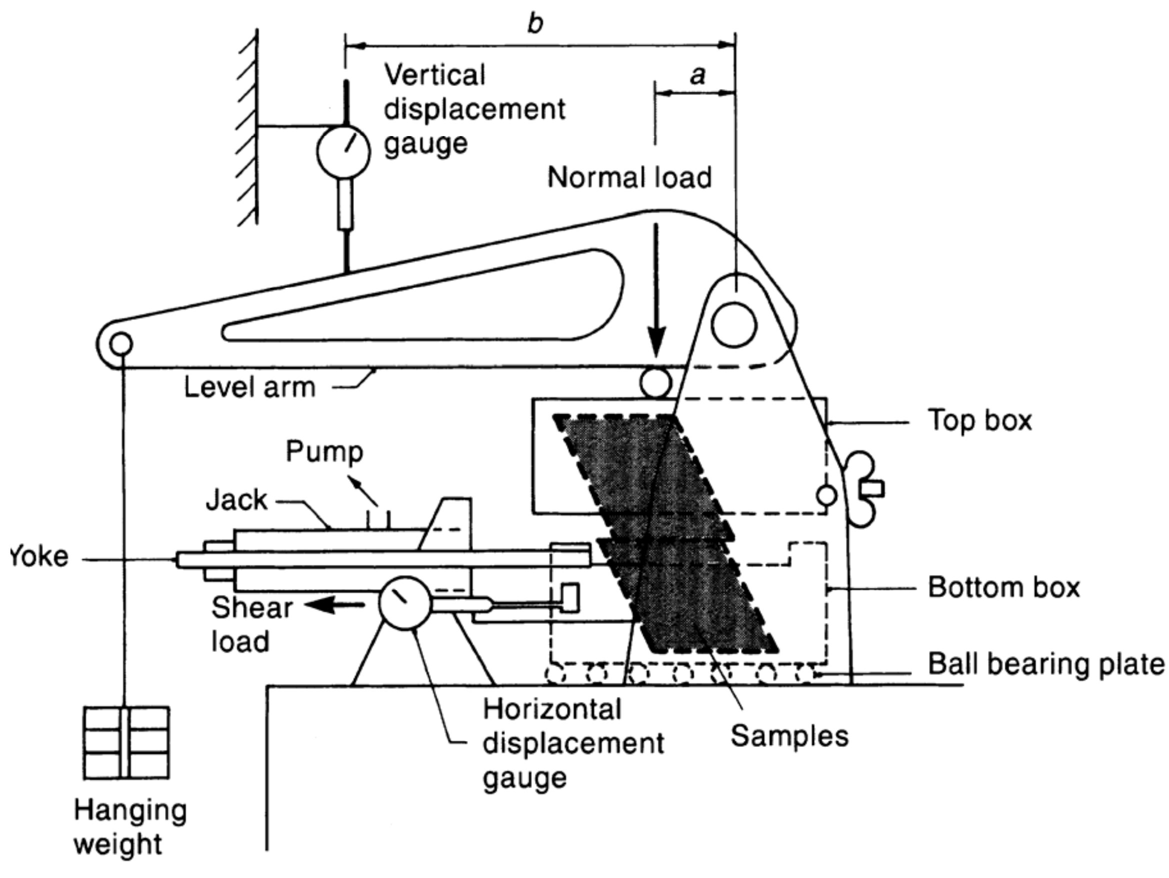


Figure 5-37. Laboratory direct shear testing equipment for rock (Wyllie, 1999).

5.12 ELASTIC PROPERTIES OF ROCKS

Preliminary estimates of the elastic modulus of intact rock can be made from Table 5-21. Note that some of the rock types identified in the table are not present in the U.S. As discussed in Chapter 4, **it is extremely important to use the elastic modulus of the rock mass for computation of in-situ displacements of rock under applied loads. Use of the intact modulus will result in unrealistic and unconservative estimates of displacement. Section 5.12.1 presents some guidance for estimating the elastic modulus of a rock mass.**

Poisson's ratio for rock should be determined from tests on intact rock core. Where tests on rock core are not practical, Poisson's ratio may be estimated from Table 5-22.

Table 5-21
Summary of elastic moduli for intact rock (AASHTO 2004 with 2006 Interims).

Rock Type	No. of Values	No. of Rock Types	Elastic Modulus, E_i (psi $\times 10^6$)			Standard Deviation (psi $\times 10^6$)
			Maximum	Minimum	Mean	
Granite	26	26	15.5	0.93	7.64	3.55
Diorite	3	3	16.2	2.48	7.45	6.19
Gabbro	3	3	12.2	9.8	11.0	0.97
Diabase	7	7	15.1	10.0	12.8	1.78
Basalt	12	12	12.2	5.20	8.14	2.60
Quartzite	7	7	12.8	5.29	9.59	2.32
Marble	14	13	10.7	0.58	6.18	2.49
Gneiss	13	13	11.9	5.13	8.86	2.31
Slate	11	2	3.79	0.35	1.39	0.96
Schist	13	12	10.0	0.86	5.97	3.18
Phyllite	3	3	2.51	1.25	1.71	0.57
Sandstone	27	19	5.68	0.09	2.13	1.19
Siltstone	5	5	5.76	0.38	2.39	1.65
Shale	30	14	5.60	0.001	1.42	1.45
Limestone	30	30	13.0	0.65	5.7	3.73
Dolostone	17	16	11.4	0.83	5.22	3.44
Note: 1 psi = 6.895 kPa						

Table 5-22
Summary of Poisson's ratio for intact rock (AASHTO 2004 with 2006 Interims)

Rock Type	No. of Values	No. of Rock Types	Poisson's Ratio, ν			Standard Deviation
			Maximum	Minimum	Mean	
Granite	22	22	0.39	0.09	0.20	0.08
Gabbro	3	3	0.20	0.16	0.18	0.02
Diabase	6	6	0.38	0.20	0.29	0.06
Basalt	11	11	0.32	0.16	0.23	0.05
Quartzite	6	6	0.22	0.08	0.14	0.05
Marble	5	5	0.40	0.17	0.28	0.08
Gneiss	11	11	0.40	0.09	0.22	0.09
Schist	12	11	0.31	0.02	0.12	0.08
Sandstone	12	9	0.46	0.08	0.20	0.11
Siltstone	3	3	0.23	0.09	0.18	0.06
Shale	3	3	0.18	0.03	0.09	0.06
Limestone	19	19	0.33	0.12	0.23	0.06
Dolostone	5	5	0.35	0.14	0.29	0.08

5.12.1 Elastic Modulus of Rock Mass

The elastic modulus of a rock mass (E_m) shall be taken as the lesser of the intact modulus of a sample of rock core (E_i) or the modulus computed from one of the following equations:

$$E_m = 145000 \times [10^{(RMR-10)/40}] \qquad 5\text{-}29$$

where: E_m = Elastic modulus of the rock mass (psi)
RMR = Rock Mass Rating (see Chapter 4)

Note that in almost all cases, the elastic modulus of the rock mass, E_m, is less than the elastic modulus of the intact rock, E_i.

The elastic modulus of the rock mass can also be determined from the following equation:

$$E_m = E_m/E_i \times E_i \qquad 5\text{-}30$$

where E_i is the elastic modulus of the intact rock. E_m/E_i is basically a reduction factor to account for discontinuities in the rock mass and can be determined by using the guidance in

Table 5-23. In using Table 5-23, it is important that the elastic modulus for the intact rock, E_i, be determined from tests rather than by using the data in Table 5-21. For critical or large structures, determination of rock mass modulus (E_m) by in-situ tests may be warranted. A discussion of suitable in-situ tests can be found in FHWA (2002a).

Table 5-23
Estimation of E_m based on RQD (AASHTO 2004 with 2006 Interims).

RQD (Percent)	E_m/E_i	
	Closed Joints	**Open Joints**
100	1.00	0.60
70	0.70	0.10
50	0.15	0.10
20	0.05	0.05
Note: Refer to Chapter 3 for guidance on determination of RQD and a description of rock joints.		

5.13 COMMON SENSE GUIDELINES FOR LABORATORY TESTING OF ROCKS

As with soils, omissions or errors introduced during laboratory testing of rock, if undetected, will be carried though the process of design and construction and will often result in costly and possibly unsafe facilities. Table 5-24 lists topics that should be considered and given proper attention so that a reasonable assessment of the rock properties will be assured and an optimization of the geotechnical investigation can be realized in terms of economy, performance, and safety. Guidance in proper handling and storage of rock cores may be found in ASTM D 5079.

Table 5-24
Common sense guidelines for laboratory testing of rocks

1. Provide protection of samples to avoid moisture loss and structural disturbance. 2. Clearly indicate proper numbering and identification of samples. 3. Store samples in controlled environments to prevent drying, overheating & freezing. 5. Take care in the handling and selection of "representative" specimens for testing. 5. Consult the field logs while selecting test specimens. 6. Recognize disturbances and fractures caused by coring procedures. 7. Maintain trimming and testing equipment in good operating condition. 8. Use properly fitting, platens, o-rings and membranes in triaxial, uniaxial, and shear tests. 9. Maintain tolerances in trimming of ends and sides of intact cores. 10. Document frequency, spacing, conditions and infilling of joints and discontinuities. 11. Periodically calibrate instruments used to measure load, deflection, temperatures and time. 12. Use a properly-determined loading rate for strength tests. 13. Photo document samples cores, fracture patterns and test specimens for possible use in a report. 15. Carefully align and level all specimens in directional loading apparatuses and test frames. 15. Record initial baselines, offsets, and eccentricities prior to testing. 16. Save remnant rock pieces after destructive testing by uniaxial, triaxial and direct shear tests. 17. Conduct nondestructive tests (i.e., porosity, unit weight, ultrasonics) prior to destructive strength tests (compression, tensile, shear). 18. Take proper precautions to assure the safety of personnel when any test procedure is performed.

5.14 PRACTICAL ASPECTS FOR LABORATORY TESTING

A poor understanding sometimes exists among geologists, structural engineers, and some foundation engineers about the type and amount of laboratory testing required for design of geotechnical features whether they happen to be structural foundations or earthwork. This weakness may render subsequent analyses useless. Organizations that have neither the proper testing facilities nor trained soils laboratory personnel should contract testing to competent AASHTO/ASTM certified private testing firms. This solution can be effective only if the project foundation designer can confidently request the necessary testing and review the results to select design values. A fair estimate of the costs associated with a private testing laboratory may be obtained by assuming the following number of person-days (pd) per test and multiplying by current labor costs:

- visual description of an SPT sample including moisture content (0.05 pd),
- visual description of a tube sample including moisture content and unit weight (0.1 pd),
- classification tests (0.7 pd),
- undrained triaxial test (0.9 pd),
- drained triaxial test (2.0 pd),
- consolidation test (2.0 pd).

These values include all work required to present a completed test result to the foundation designer. Alternatively, most private testing laboratories provide a schedule of services and associated costs that can be used to obtain a more accurate estimate of the cost of a proposed laboratory test program.

Blanket consultant contracts "to perform testing necessary for design" usually result in unnecessarily large quantities of testing being performed, much of which does not apply to the project geotechnical issues. For example, if a multi span structure is crossing an area having a soft clay surface deposit underlain by sands, inordinate amounts of time and money should not be spent to determine the strength and consolidation parameters of the soft clay layer at pier and abutment locations in great detail. Generally a pile foundation will be designed by using SPT N-values and the only laboratory testing that may be needed in the soft clay layer may be to estimate drag forces on the piles. Also, non-standard strength testing such as torvanes, penetrometers, etc., which are not covered by ASTM or AASHTO standards, should not be permitted. Such devices should be used only for field index tests to determine consistency.

5.15 VARIABILITY OF MEASURED PROPERTIES

Chapters 3, 4 and 5 presented methodologies to perform subsurface explorations, to describe identify, and classify subsurface materials, and to implement and interpret a laboratory test program. The topics covered in these chapters are presented in the temporal sequence in which they are performed on an actual project. Therefore, the geotechnical specialist should continuously evaluate the results of laboratory tests with respect to the initial subsurface model prepared as part of Step 1 of the flow chart presented in Figure 3-1 of Chapter 3. Table 5-25 presents the values of the coefficient of variation of measured properties that should be taken into consideration as the subsurface model is finalized for engineering design. The data in Table 5-25 can be used by the geotechnical specialist as follows:

- Perform sensitivity (parametric) studies to evaluate the effect of variability in properties with respect to the subsurface profile and the type, magnitude and direction of the anticipated loading and establish the best-case and worst-case scenarios,

- Evaluate the need to perform additional explorations,

- Exercise judgment with respect to the results of engineering analyses and designs and convey the uncertainty to the project team, in particular to the structural engineer, and

- Establish the need for instrumentation to monitor the performance of the facility during and after construction .

The finalization of the subsurface model should be performed by a geotechnical specialist who is experienced in the design and construction aspects of the proposed facility. Active input should be sought from the field inspectors and the laboratory personnel who were actually involved in the collection of the data. At this stage, it is recommended that the geotechnical specialist seek a peer review from one or more **qualified and experienced** geotechnical specialist(s) with the specific purpose of having them evaluate whether or not the collected data and subsurface model is adequate to permit a cost-effective design of the facility. The experienced geotechnical specialist(s) can also provide information on potential value analysis alternatives (value engineering) for the design of the facility based on the collected data. The importance of such peer reviews cannot be overemphasized. Finally, the owner of the facility should be informed of the findings so that the owner can make decisions such as authorizing more field explorations and/or laboratory tests or modifying the facilities as appropriate based on the results of the geotechnical investigation thus far.

Table 5-25
Values of coefficient of variation, V, for geotechnical properties and in situ tests (after Duncan, 2000)

Measured or interpreted parameter value	**Coefficient of Variation, V (Note 1)**
Unit weight, γ	3 to 7 %
Buoyant unit weight, γ_b	0 to 10 %
Effective stress friction angle, ϕ'	2 to 13 %
Undrained shear strength, s_u	13 to 40 %
Undrained strength ratio (s_u/p_o)	5 to 15 %
Compression index, C_c	10 to 37 %
Preconsolidation pressure, p_c	10 to 35 %
Hydraulic conductivity of saturated clay, k	68 to 90 %
Hydraulic conductivity of partially-saturated clay, k	130 to 240 %
Coefficient of consolidation, c_v	33 to 68 %
Standard penetration blow count, N	15 to 45 %
Electric cone penetration test, q_c	5 to 15 %
Mechanical cone penetration test, q_c	15 to 37 %
Vane shear test undrained strength, s_{uVST}	10 to 20 %
Note 1: Coefficient of Variation, V, is defined as standard deviation divided by the average (mean) value expressed as a percentage.	

CHAPTER 6.0
SLOPE STABILITY

Ground stability must be assured prior to consideration of other foundation related items. Embankment foundation problems involve the support of the embankment by natural soil. Problems with embankments and structures occasionally occur that could be prevented by initial recognition of the problem and appropriate design. Stability problems most often occur when the embankment is to be built over soft soils such as low strength clays, silts, or peats. Once the soil profile, soil strengths, and depth of ground water table have been determined by field explorations and/or field and laboratory testing, the stability of the embankment can be analyzed and a factor of safety estimated. If the embankment is found to be unstable, measures can then be taken to stabilize the foundation soils.

As illustrated in Figure 6-1, there are four major types of instability that should be considered in the design of embankments over weak foundation soils. Recommendations on how to recognize, analyze, and solve each of the first three problems are presented in this chapter. Lateral squeeze is more closely related to the evaluation of foundation deformation and is discussed in Chapter 7 (Approach Roadway Deformations).

The stability problems illustrated in Figure 6-1 can be classified as “internal” or "external." "Internal" embankment stability problems generally result from the selection of poor quality embankment materials and/or improper placement of the embankment fills and/or improper placement requirements. The infinite slope failure mode is an example of an “internal” stability problem; often such a failure is manifested as sloughing of the surface of the slope. Internal stability can be assured through project specifications by requiring granular materials with minimum gradation and compaction requirements. An example of a typical specification for approach roadway construction is presented in Chapter 7. The failure modes shown in Figure 6-1b, c and d, can be classified as “external” stability problems.

6.01 Primary Reference

The primary reference for this chapter is as follows:

FHWA (2001a). *Soil Slope and Embankment Design Reference Manual.* Report No. FHWA NHI-01-026, Authors: Collin, J. G., Hung, J. C., Lee, W. S., Munfakh, G., Federal Highway Administration, U.S. Department of Transportation.

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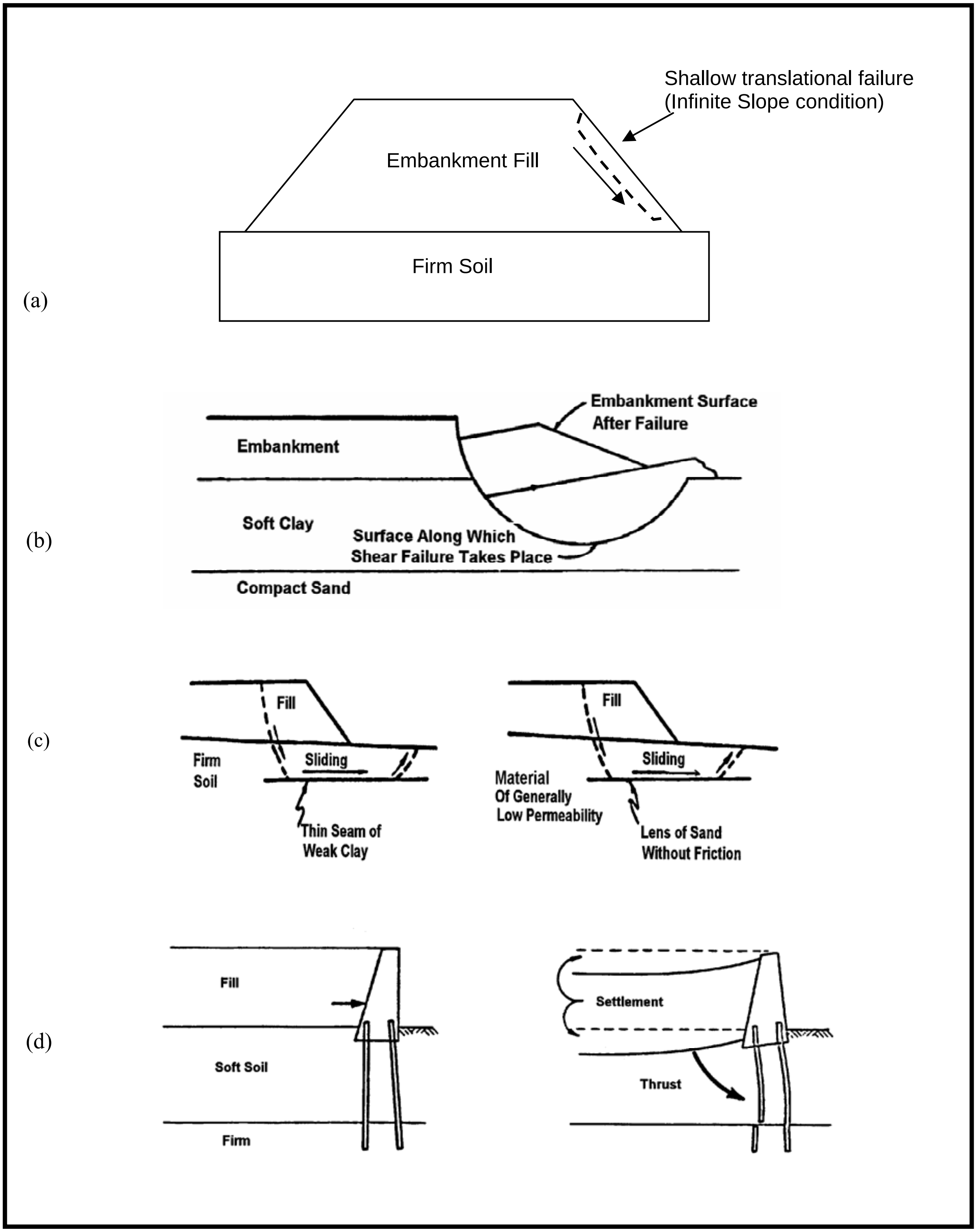


Figure 6-1. Embankment failures: (a) Infinite slope failure in embankment fill, (b) Circular arc failure in embankment fill and foundation soil, (c) Sliding block failure in embankment fill and foundation soil, and (d) Lateral squeeze of foundation soil.

6.1 EFFECTS OF WATER ON SLOPE STABILITY

Very soft, saturated foundation soils or ground water generally play a prominent role in geotechnical failures in general. They are certainly major factors in cut slope stability and in the stability of fill slopes involving both "internal" and "external" slope failures. The effect of water on cut and fill slope stability is briefly discussed below.

- **Importance of Water**

  Next to gravity, water is the most important factor in slope stability. The effect of gravity is known, therefore, water is the key factor in assessing slope stability.

- **Effect of Water on Cohesionless Soils**

  In cohesionless soils, water does not affect the angle of internal friction (ϕ). The effect of water on cohesionless soils below the water table is to decrease the intergranular (effective) stress between soil grains (σ'_n), which decreases the frictional shearing resistance (τ').

- **Effect of Water on Cohesive Soils**

  Routine seasonal fluctuations in the ground water table do not usually influence either the amount of water in the pore spaces between soil grains or the cohesion. The attractive forces between soil particles prevent water absorption unless external forces such as pile driving, disrupt the grain structure. However, certain clay minerals do react to the presence of water and cause volume changes of the clay mass.

  An increase in absorbed moisture is a major factor in the decrease in strength of cohesive soils as shown schematically in Figure 6-2. Water absorbed by clay minerals causes increased water contents that decrease the cohesion of clayey soils. These effects are amplified if the clay mineral happens to be expansive, e.g., montmorillonite.

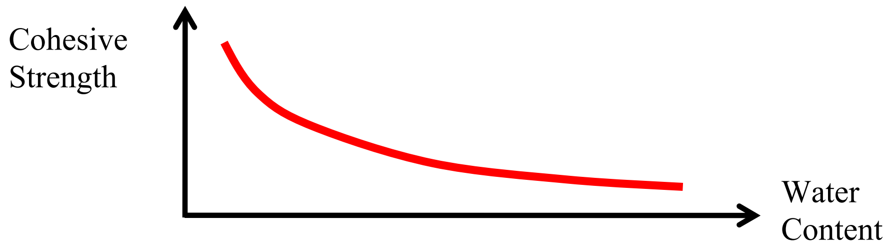


Figure 6-2. Effect of water content on cohesive strength of clay.

- **Fills on Clays**

  Excess pore water pressures are created when fills are placed on clay or silt. Provided the applied loads do not cause the undrained shear strength of the clay or silt to be exceeded, as the excess pore water pressure dissipates consolidation occurs, and the shear strength of the clay or silt increases with time. For this reason, the factor of safety increases with time under the load of the fill.

- **Cuts in Clay**

  As a cut is made in clay the effective stress is reduced. This reduction will allow the clay to expand and absorb water, which will lead to a decrease in the clay strength with time. For this reason, the factor of safety of a cut slope in clay may decrease with time. Cut slopes in clay should be designed by using effective strength parameters and the effective stresses that will exist in the soil after the cut is made.

- **Slaking - Shales, Claystones, Siltstones, etc.**

  Sudden moisture increase in weak rocks can produce a pore pressure increase in trapped pore air accompanied by local expansion and strength decrease. The "slaking" or sudden disintegration of hard shales, claystones, and siltstones results from this mechanism. If placed as rock fill, these materials will tend to disintegrate into a clay soil if water is allowed to percolate through the fill. This transformation from rock to clay often leads to settlement and/or shear failure of the fill. Index tests such as the jar-slake test and the slake-durability test used to assess slaking potential are discussed in FHWA (1978).

6.2 DESIGN FACTOR OF SAFETY

A minimum factor of safety as low as 1.25 is used for highway embankment side slopes. This value of the safety factor should be increased to a minimum of 1.30 to 1.50 for slopes whose failure would cause significant damage such as end slopes beneath bridge abutments, major retaining structures and major roadways such as regional routes, interstates, etc The selection of the design safety factor for a particular project depends on:

- The method of stability analysis used (see Section 6.4.5).
- The method used to determine the shear strength.
- The degree of confidence in the reliability of subsurface data.
- The consequences of a failure.
- How critical the application is.

6.3 INFINITE SLOPE ANALYSIS

A slope that extends for a relatively long distance and has a consistent subsurface profile may be analyzed as an infinite slope. The failure plane for this case is parallel to the surface of the slope and the limit equilibrium method can be applied readily.

6.3.1 Infinite Slopes in Dry Cohesionless Soils

A typical section or "slice" through the potential failure zone of a slope in a dry cohesionless soil, e.g., dry sand, is shown in Figure 6-3, along with its free body diagram. The weight of the slice of width b and height h having a unit dimension into the page is given by:

$$W = \gamma \, b \, h \tag{6-1}$$

where γ is the effective unit weight of the dry soil. For a slope with angle β as shown in Figure 6-3, the normal (N) and tangential (T) force components of W are determined as follows:

$$N = W \cos \beta \quad \text{and} \tag{6-2}$$

$$T = W \sin \beta \tag{6-3}$$

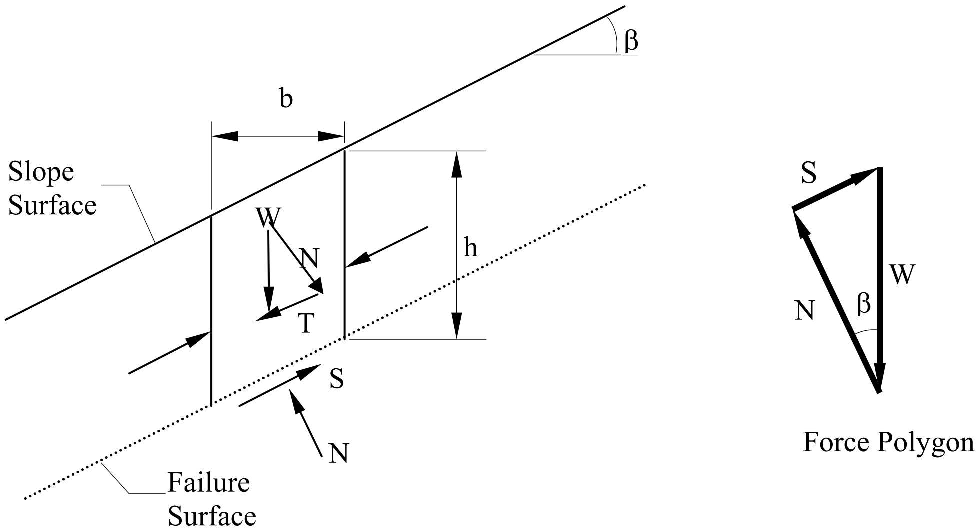


Figure 6-3. Infinite slope failure in dry sand.

The available shear strength along the failure plane is given by:

$$S = N \tan \phi \qquad 6\text{-}4$$

The factor of safety (FS) is defined as the ratio of available shear strength to strength required to maintain stability. Thus, the FS will be given by:

$$FS = \frac{S}{T} = \frac{N \tan \phi}{W \sin \beta} = \frac{(W \cos \beta) \tan \phi}{W \sin \beta} = \frac{\tan \phi}{\tan \beta} \qquad 6\text{-}5$$

For an infinite slope analysis, the FS is independent of the slope depth, h, and depends only on the angle of internal friction, ϕ, and the angle of the slope, β. The slope is said to have reached **limit equilibrium** when FS=1.0. Also, at a FS = 1.0, the maximum slope angle will be limited to the angle of internal friction, ϕ.

6.3.2 Infinite Slopes in c-ϕ Soils with Parallel Seepage

If a saturated slope in a c-ϕ soil has seepage parallel to the surface of the slope as shown in Figure 6-4, the same limit equilibrium concepts may be applied to determine the FS, which will now depend on the effective normal force (N'). In the following analysis, effective shear strength parameters, c' and ϕ' are used.

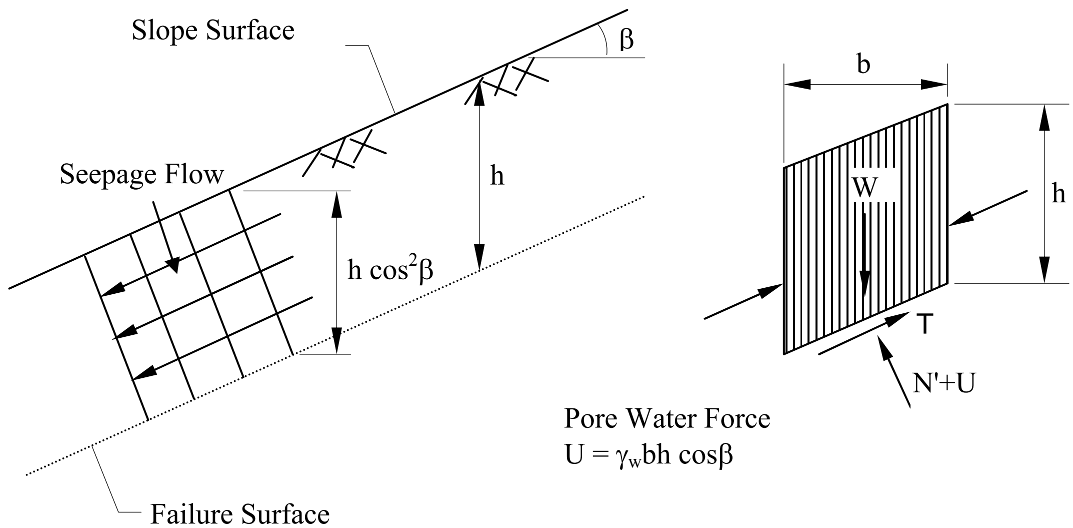


Figure 6-4. Infinite slope failure in a c-ϕ soil with parallel seepage.

From Figure 6-4, the pore water force acting on the base of a typical slice having a unit dimension into the page is:

$$U = \left(\gamma_w h \cos^2\beta\right)\frac{b}{\cos\beta} = \gamma_w b h \cos\beta \qquad 6\text{-}6$$

where h is any depth less than or equal to the depth of saturation and b is a unit width.

The available frictional strength, S, along the failure plane will depend on ϕ' and the effective normal force, N' =N-U, where N is the total normal force. The equation for S is:

$$S = c' \frac{b}{\cos\beta} + (N - U)\tan\phi' \qquad 6\text{-}7$$

The factor of safety for this case will be:

$$FS = \frac{S}{T} = \frac{(c' \, b/\cos\beta) + (N - U)\tan\phi'}{W \sin\beta} \qquad 6\text{-}8$$

By substituting $W = \gamma_{sat}\, b\, h$ into the above expression and rearranging terms, the FS is given by:

$$FS = \frac{c' + h\,(\gamma_{sat} - \gamma_w)\,(\cos^2\beta)\ \tan\phi'}{\gamma_{sat}\, h \sin\beta \cos\beta} \qquad 6\text{-}9$$

where $\gamma' = (\gamma_{sat} - \gamma_w)$.

For c' = 0, the above expression may be simplified to:

$$FS = \frac{\gamma'}{\gamma_{sat}}\ \frac{\tan\phi'}{\tan\beta} \qquad 6\text{-}10$$

From Equation 6-10 it is apparent that for a *cohesionless* material with parallel seepage, the FS is also independent of the slope depth, h, just as it is for a dry cohesionless material as given by Equation 6-5. The difference is that the FS for the dry material is reduced by the factor γ'/γ_{sat} for saturated cohesionless materials to account for the effect of seepage. For typical soils, this reduction will be about 50 percent in comparison to dry slopes.

The above analysis can be generalized if the seepage line is assumed to be located at a normalized height, m, above the failure surface where m =z/h. In this case, the FS is:

$$FS = \frac{c' + h \cos^2\beta\,[\,(1-m)\,\gamma_m + m\,\gamma'\,]\tan\phi'}{h \sin\beta \cos\beta\,[\,(1-m)\,\gamma_m + m\,\gamma_{sat}\,]} \qquad 6\text{-}11$$

and γ_{sat} and γ_m are the saturated and moist unit weights of the soil below and above the seepage line. The above equation may be readily reformulated to determine the critical depth of the failure surface in a c'-ϕ' soil for any seepage condition.

6.4 CIRCULAR ARC FAILURE

Experience and observations of failures of embankments constructed over relatively deep deposits of soft soils have shown that when failure occurs, the embankment sinks down, the adjacent ground rises and the failure surface follows a circular arc as illustrated in Figure 6-5.

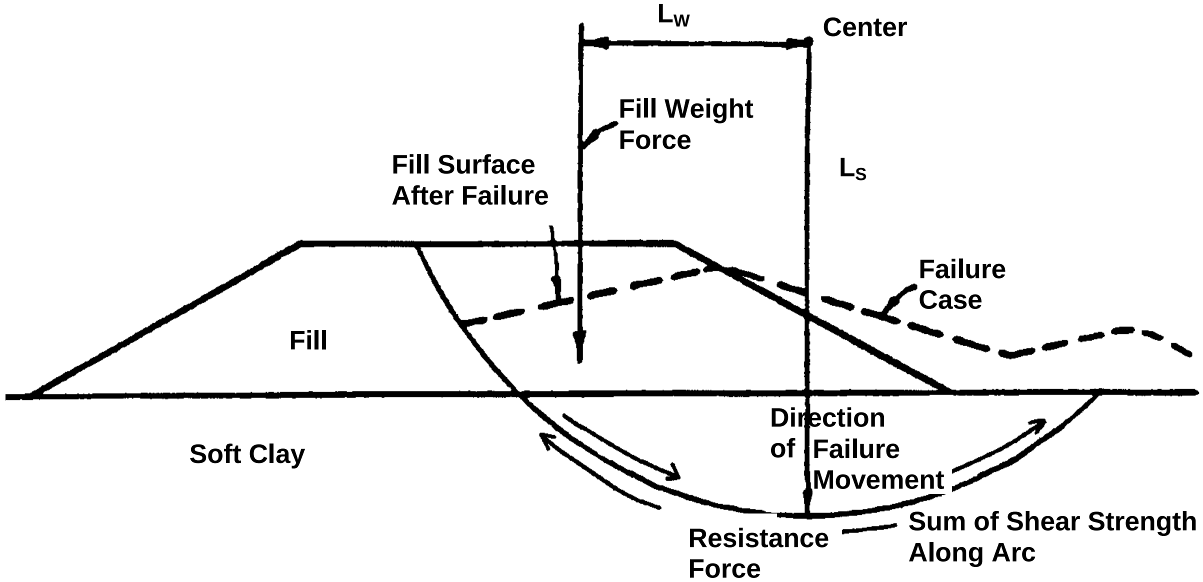


Figure 6-5. Typical circular arc failure mechanism.

At failure the driving and resistance forces act as follows:

- The force driving movement consists of the embankment weight. The driving moment is the product of the weight of the embankment acting through its center of gravity times the horizontal distance from the center of gravity to the center of rotation (L_W).

- The resisting force against movement is the total shear strength acting along the failure arc. The resisting moment is the product of the resisting force times the radius of the circle (L_S).

The factor of safety against slope instability is equal to the ratio of the resisting moment to driving moment.

$$\text{Factor of Safety} = \frac{\text{Total Shear Strength} \times L_S}{\text{Weight Force} \times L_W} = \frac{\text{Resisting Moment}}{\text{Driving Moment}} \qquad 6\text{-}12$$

Failure takes place when the factor of safety is less than 1, i.e., the driving moment > resisting moment.

6.4.1 Simple Rule of Thumb for Factor of Safety

A rule of thumb based on simplified bearing capacity theory can be used to make a preliminary "guestimate" of the factor of safety (FS) against circular arc failure for an embankment built on a clay foundation without presence of free water. The rule of thumb is as follows:

$$FS \cong \frac{6\ c}{\gamma_{Fill} \times H_{Fill}} \qquad \text{6-13}$$

Where: c = unit cohesion of clay foundation soil (psf)
γ_{Fill} = unit weight fill (pcf)
H_{Fill} = height of fill (feet)

Since the rule of thumb assumes that there is no influence from groundwater, c and γ_{Fill} are effective stress parameters.

For example, the factor of safety for the proposed embankment illustrated in Figure 6-6 can be computed as follows:

$$FS = \frac{(6)(1{,}100\ \text{psf})}{(130\ \text{pcf})(30\ \text{ft})} = 1.69 \qquad \text{Use Rule of Thumb 6-13}$$

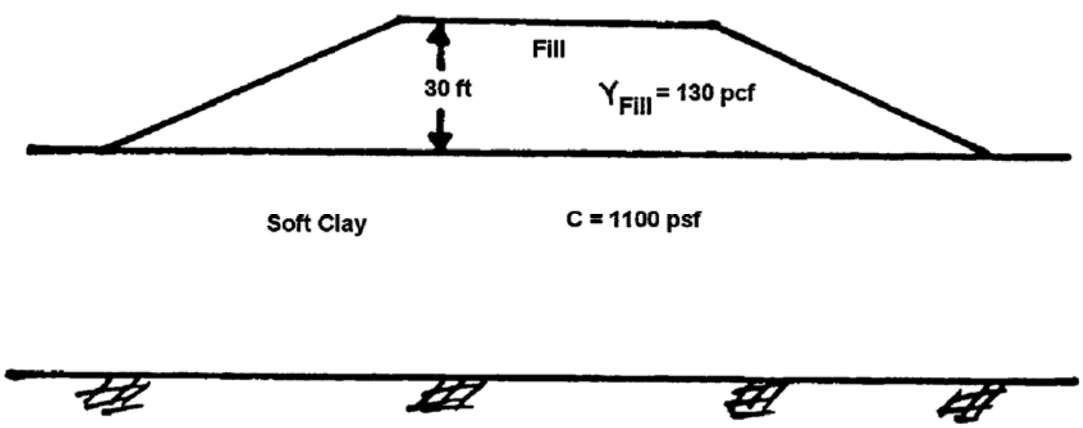


Figure 6-6. Example proposed embankment.

The factor of safety computed by using this rule of thumb should never be used for final design. This simple equation obviously does not take into account such factors as fill strength or fill slope angle and does not identify the location of a critical failure surface. **If**

the factor of safety computed by using the rule of thumb is less than 2.5, a more sophisticated stability analysis is required.

However, this rule of thumb can be helpful very early in the design stage to make a quick preliminary check on whether stability may be a problem and if more detailed analyses should be conducted. It can also be of use in the field while borings and sampling are being performed. For example, if in-situ vane shear tests are being carried out as part of the field investigation for a proposed embankment, the geotechnical specialist can use the vane strength with Equation 6-13 to estimate the FS in the field. This estimate can aid in directing the drilling, sampling, and testing program while the drill crew is at the site and help insure that critical strata are adequately explored and sampled. Finally, the FS calculated by the rule of thumb can be used to check for *gross* errors in computer output or input.

6.4.2 Stability Analysis Methods (General)

There are several available methods that can be used to perform a circular arc stability analysis for an approach embankment over soft ground. The simplest basic method is known as the **Normal** or **Ordinary Method of Slices**, also known as Fellenius' method (Fellenius, 1936) or the Swedish circle method of analysis. The Ordinary Method of Slices can easily be performed by hand calculations and is also a method by which the computation of driving and resisting forces is straightforward and easily demonstrated. For this method, the failure surface is assumed to be the arc of a circle as shown in Figure 6-7 and the factor of safety against sliding along the failure surface is defined as the ratio of the moment of the total available resisting forces on the trial failure surface to the net moment of the driving forces due to the embankment weight, that is:

$$FS = \frac{\text{Sum of Resisting Forces} \times \text{Moment Arm (R)}}{\text{Sum of Driving Forces} \times \text{Moment Arm (R)}} \qquad \text{6-14}$$

Note that since the method consists of computing the driving and resisting forces along the failure arc, the moment arm R is the same for both the driving and resisting forces. Thus, Equation 6-14 reduces to:

$$FS = \frac{\text{Sum of Resisting Forces}}{\text{Sum of Driving Forces}} \qquad \text{6-14a}$$

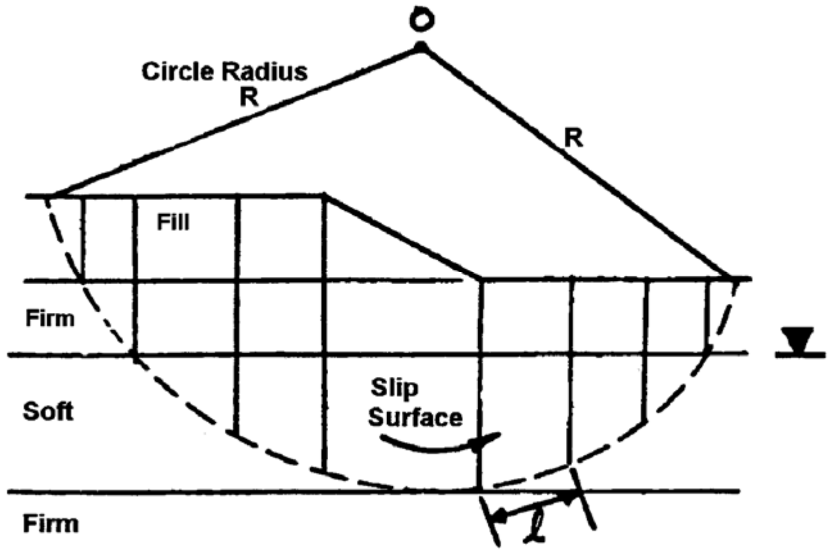


Figure 6-7. Geometry of Ordinary Method of Slices.

For slope stability analysis, the mass within the failure surface is divided into vertical slices as shown in Figures 6-7 and 6-8. A typical vertical slice and its free body diagram is shown in Figure 6-9 for the case where water is not a factor. The case with the presence of water is shown in Figure 6-10. The following assumptions are then made in the analysis using Ordinary Method of Slices:

1. The available shear strength of the soil can be adequately described by the Mohr-Coulomb equation:

$$\tau = c + (\sigma - u) \tan \phi \qquad \text{6-15}$$

where:

τ	= effective shear strength
c	= cohesion component of shear strength
$(\sigma - u) \tan \phi$	= frictional component of shear strength
σ	= total normal stress on the failure surface at the base of a slice due to the weight of soil and water above the failure surface
u	= water uplift pressure against the failure surface
ϕ	= angle of internal friction of soil
$\tan \phi$	= coefficient of friction along failure surface

2. The factor of safety is the same for all slices.

3. The factors of safety with respect to cohesion (c) and friction (tan ϕ) are equal.

4. Shear and normal forces on the sides of each slice are ignored.

5. The water pressure (u) is taken into account by reducing the total weight of the slice by the water uplift force acting at the base of the slice.

Equation 6-15 is expressed in terms of total strength parameters. The equation could easily have been expressed in terms of effective strength parameters. Therefore, the convention to be used in the stability analysis, be it total stress or effective stress, should be chosen and specified. In soil problems involving water, the engineer may compute the normal and tangential forces by using either total soil weights and boundary water forces (both buoyancy and unbalanced hydrostatic forces) or submerged (buoyant) soil weights and unbalanced hydrostatic forces. The results are the same. When total weight and boundary water forces are used, the equilibrium of the entire block is considered. When submerged weights and hydrostatic forces are used, the equilibrium of the mineral skeleton is considered. The total weight notation is used herein as this method is the simplest to compute.

6.4.3 Ordinary Method of Slices - Step-By-Step Computation Procedure

To compute the factor of safety for an embankment by using the Ordinary Method of Slices, the step-by-step computational procedure is as follows:

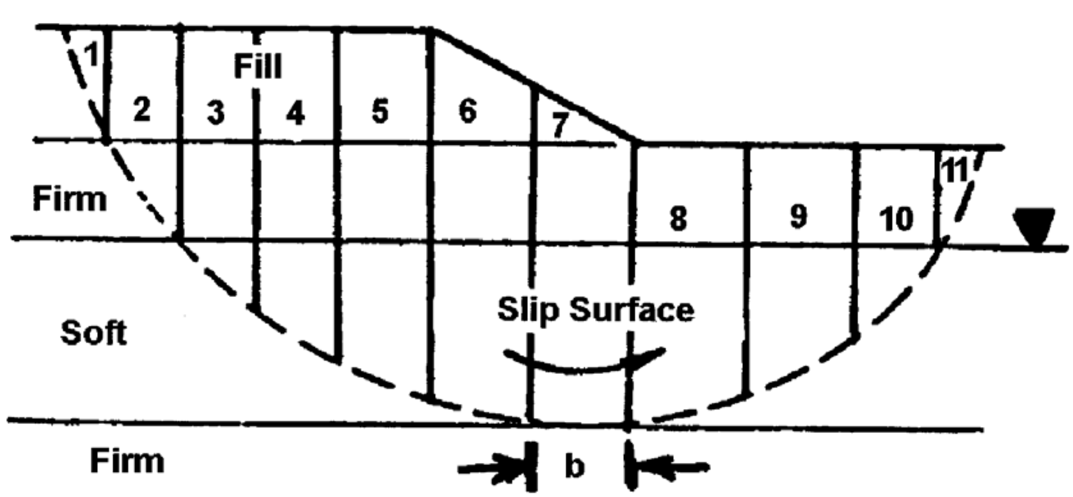


Figure 6-8. Example of dividing the failure mass in slices.

Step 1. **Draw a cross-section of the embankment and foundation soil profile on a scale of either 1-inch = 10 feet or 1-inch = 20 feet scale both horizontal and vertical.**

Step 2. **Select a circular failure surface such as shown in Figure 6-7.**

Step 3. **Divide the circular mass above the failure surface into 10 - 15 vertical slices as illustrated in Figure 6-8.**

To simplify computation, locate the vertical sides of the slices so that the bottom of any one slice is located entirely in a single soil layer or at the intersection of the ground water level and the circle.

Locate the top boundaries of vertical slices at breaks in the slope. The slice widths do not have to be equal. For convenience assume a one-foot (0.3 m) thick section of embankment. This unit width simplifies computation of driving and resisting forces.

Also, as shown in Figure 6-9 and 6-10 the driving and resisting forces of each slice act at the intersection of a vertical line drawn from the center of gravity of the slice to the failure circle to establish a centroid point on the circle. Lines (called rays) are then drawn from the center of the circle to the centroid point on the circular arc. The α angles are then measured from the vertical to each ray.

When the water table is sloping, use Equation 6-16 to calculate the water pressure on the base of the slice:

$$u = h_w \gamma_w \cos^2 \alpha_w \qquad \text{6-16}$$

where: α_w = slope of water table from horizontal in degrees.
h_w = depth from ground water surface to the centroid point on the circle.

Step 4: **Compute the total weight (W_T) of each slice.**

For illustration, the resisting and driving forces acting on individual slices with and without water pressure are shown on Figures 6-9 and 6-10.

To compute W_T, use total soil unit weight, γ_t, both above and below the water table.

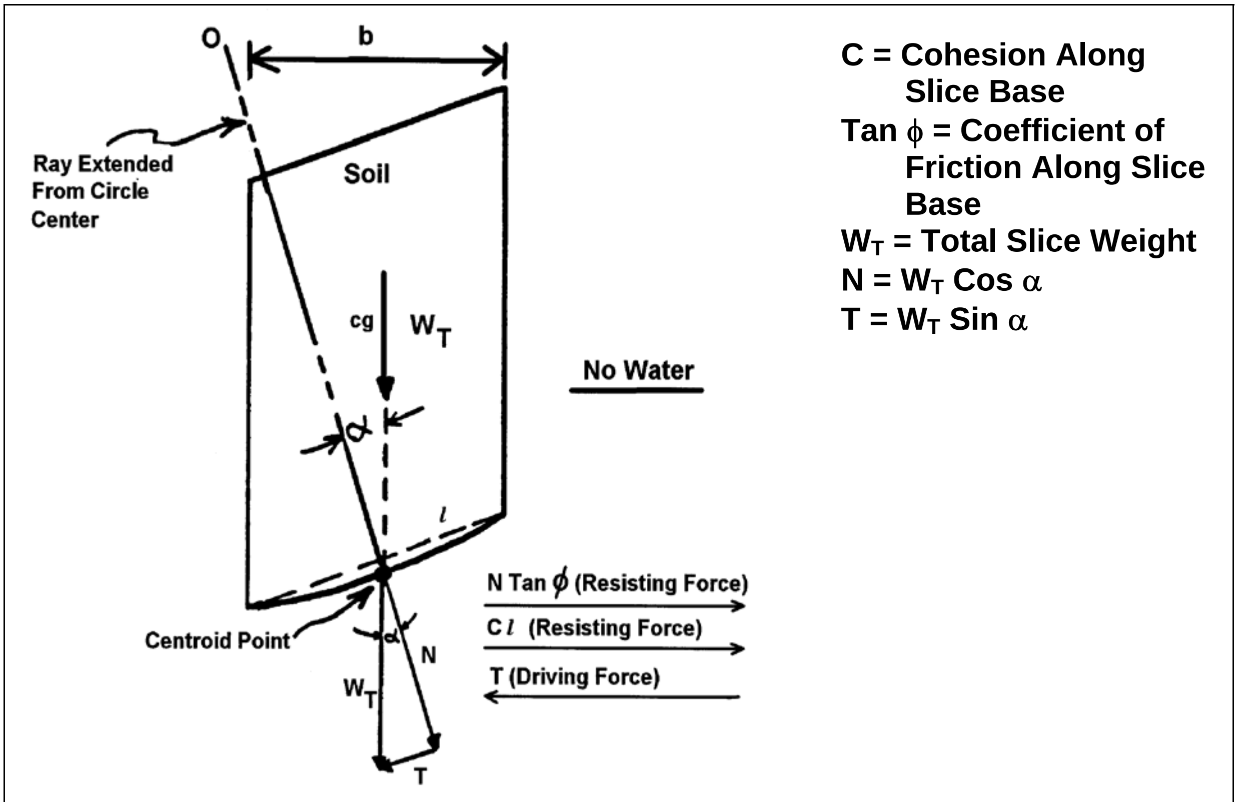


Figure 6-9. Forces on a slice without water effect.

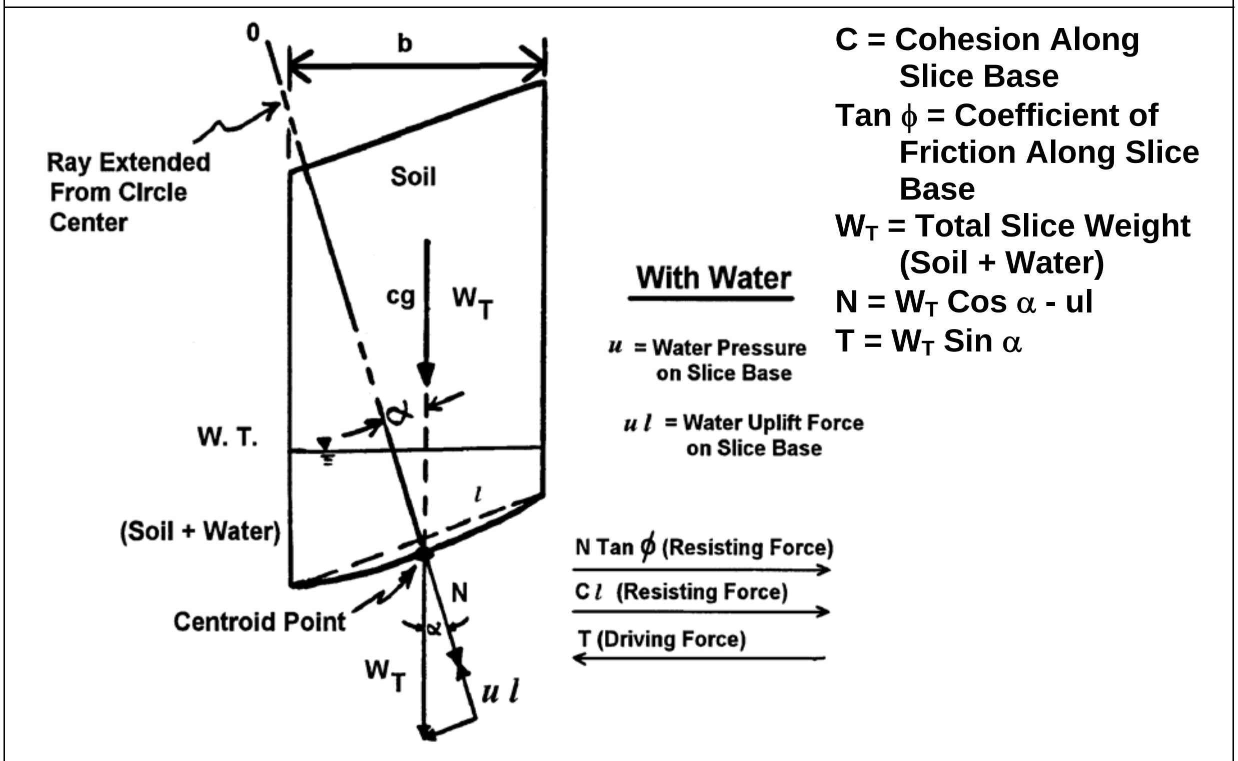


Figure 6-10. Forces on a slice with water effect.

$$W_T = \gamma_t \times \text{Average Slice Height} \times \text{Slice Width} \qquad 6\text{-}17$$

For example: Assume
$\gamma_t = 120$ pcf (18.9 kN/m^3)
Average height of slice = 10 ft (3 m)
Slice width = 10 ft (3 m)

Then for a unit thickness into the plane of the paper, W_T = (120 pcf) (10 ft) (10 ft) (1 ft) = 12,000 lbs (53.3 kN)

Step 5: Compute frictional resisting force for each slice depending on location of ground water table.

$$N = W_T \cos \alpha \qquad 6\text{-}18a$$

$$N' = W_T \cos \alpha - ul \qquad 6\text{-}18b$$

N	=	total normal force acting against the slice base
N'	=	effective normal force acting against the slice base
W_T	=	total weight of slice (from Step 4 above)
α	=	angle between vertical and line drawn from circle center to midpoint (centroid) of slice base (Note: α is also equal to the angle between the horizontal and a line tangent to the base of the slice)
u	=	water pressure on the base of the slice = average height of water, $h_w \times \gamma_w$. Use $\gamma_w = 62.4$ pcf (9.8 kN/m^3)
l	=	arc length of slice base. To simplify computations, take l as the secant to the arc.
$u\,l$	=	water uplift force against base of the slice per unit thickness into the plane of the paper.
ϕ	=	internal friction angle of the soil.
$\tan \phi$	=	coefficient of friction along base of the slice.

Note that **the effect of water is to reduce the normal force against the base of the slice and thus reduce the frictional resisting force**. To illustrate this reduction, take the same slice used in Step 4 and compute the friction resistance force for the slice with no water and then for the ground water table located 5 feet above the base of the slice.

Assume: $\phi = 25^o$ $\alpha = 20^o$ l = 11 ft (3.3 m)

If there is no water in the slice, u l = 0 and Equation 6-18b reverts to Equation 6-18a and the total frictional resistance can be computed as follows:

$$N = W_T \cos\alpha = (12{,}000 \text{ lbs})(\cos 20^o) = 11{,}276 \text{ lbs} \ (50.18 \text{ kN})$$
$$N \tan\phi = (11{,}276 \text{ lbs})(\tan 25^o) = 5{,}258 \text{ lbs} \ (23.4 \text{ kN})$$

If there is 5-ft of water above the midpoint of the slice, Equation 6-18b is used directly and the effective frictional resistance is computed as follows:

$$u\,l = (h_w)(\gamma_w)(l) = (5 \text{ ft})(62.4 \text{ pcf}))(11 \text{ ft})(1 \text{ ft}) = 3{,}432 \text{ lbs} \ (15.3 \text{ kN})$$
$$N' = W_T \cos\alpha - u\,l = 11{,}276 \text{ lbs} - 3{,}432 \text{ lbs} = 7{,}844 \text{ lbs} \ (34.9 \text{ kN})$$
$$N' \tan\phi = (7{,}844 \text{ lbs})(\tan 25°) = 3{,}658 \text{ lbs} \ (16.3 \text{ kN})$$

Step 6: Compute cohesive resisting force for each slice.

c = cohesive soil strength
l = length of slice base

Example: c = 200 psf (9.6 kPa)
l = 11 ft (3.6 m)
cl = (200 psf)(11 ft)(1 ft) = 2,200 lbs (9.8 kN)

Step 7: Compute tangential driving force, T, for each slice.

$$T = W_T \sin\alpha \qquad 6\text{-}19$$

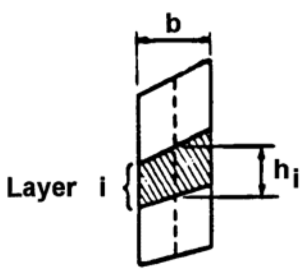
T is the component of total weight of the slice, W_T, acting tangent to the slice base. T is the driving force due to the weight of both soil and water in the slice.

Example: Given W_T = 12,000 lbs (53.3 kN)
$\alpha = 20^o$
$T = W_T \sin\alpha = (12{,}000 \text{ lbs})(\sin 20^o) = 4{,}104$ lbs (18.2 kN)

Step 8: Sum resisting forces and driving forces for all slices and compute factor of safety.

$$FS = \frac{\sum \text{Resisting Forces}}{\sum \text{Driving Forces}} = \frac{\sum N' \tan\phi + \sum c\, l}{\sum T} \qquad \text{6-20}$$

Tabular computation forms for use in performing a method of slices stability analysis by hand are included on Figures 6-11 and 6-12.



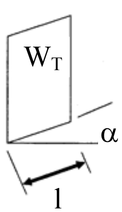
γ_i = unit weight of layer i
h_i = height of layer at center of slice
W_i = partial weight = $b\, h_i\, \gamma_i$
$\sum W_i$ = total weight of slice W_T

Slice No.	b	h_i	γ_i	W_i	$\sum W_i = W_T$

Figure 6-11a. Tabular form for computing weights of slices.

Slice No.	W_T (from Table 6-11a)	l	α	c	ϕ	u	ul	$W_T\cos\alpha$	$N' = W_T\cos\alpha$ -ul	$N'\tan\phi$	cl	$T= W_T\sin\alpha$
									Σ			

$$FS = \frac{\Sigma\,(W_T\ \cos\ \alpha - ul\,)\ \tan\ \phi + \Sigma\ cl}{\Sigma\,W_T\ \sin\ \alpha} = \frac{\Sigma\ N'\tan\ \phi + \Sigma\ cl}{\Sigma\,W_T\ \sin\ \alpha} = \underline{\qquad\qquad\qquad\qquad}.$$



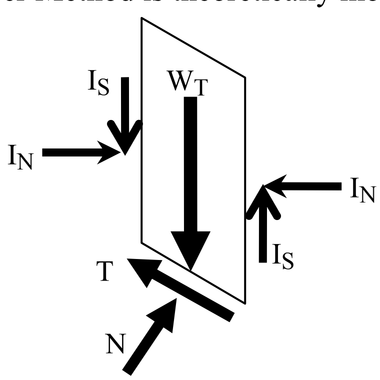
Legend: Refer to Figure 6-10 for definition of various slice quantities

W_T = Total weight of Slice (soil + water)
l = Base length of the slice
c = Cohesion at base of slice
ϕ = angle of internal friction
u = pore water pressure at base of slice

Figure 6-11b. Tabular form for calculating factor of safety by Ordinary Method of Slices.

6.4.4 Recommended Stability Methods

The basic static forces on a typical slice are shown in Figure 6-12. The limit equilibrium method of slices is based on the principles of statics, i.e., summation of moments, vertical forces, and horizontal forces. The Ordinary Method of Slices ignores both interslice shear (I_S) and interslice normal (I_N) forces and satisfies only moment equilibrium. There are many other methods available for performing a slope stability analysis besides the Ordinary Method of Slices. These include the Bishop Method (Bishop, 1955), the Simplified Janbu Method (Janbu, 1954) and the Spencer Method (Spencer, 1967). These methods are primarily variations and refinements of the Ordinary Method of Slices. The differences among these more refined methods lie in the assumptions made regarding the interslice shear and normal forces acting on the sides of slices. The Bishop Method, also known as the Simplified Bishop Method, includes interslice normal forces (I_N) but ignores interslice shear (I_S) forces. Again, Bishop's method satisfies only moment equilibrium. The Simplified Janbu Method is similar to the Bishop Method in that it includes the interslice normal (I_N) forces and ignores the interslice shear (I_S) forces. The difference between the Bishop Method and the Simplified Janbu Method is that the Simplified Janbu Method satisfies only horizontal force equilibrium, as opposed to moment equilibrium. The Spencer Method considers both normal and shear interslice side forces as well as moments. Therefore the Spencer Method is theoretically more rigorous than the other methods.



Legend:

W_T	Total weight of slice
N	Normal force at base of slice
T	Tangential force at base of slice
I_S	Interslice shear (vertical) force
I_N	Interslice normal (horizontal) force

Figure 6-12. Typical static forces on a slice of sliding mass without seepage.

The Ordinary Method of Slices is more conservative and gives unrealistically lower factors of safety than the Bishop Method or the other more refined methods. The only reason for inclusion of the Ordinary Method of Slides here is to demonstrate the principles of slope stability. For purely cohesive soils the Ordinary Method of Slices and Bishop's method give identical results. For soils that have frictional strength, the Bishop Method should be used as a minimum. While none of the methods is 100 percent correct theoretically, currently available procedures such as Bishop's method, Janbu's Simplified method or Spencer's method are sufficiently accurate for practical analysis and design. For more information on these and other slope stability methods, the reader is referred to FHWA (2001a).

The method of analysis that should be used to determine a factor of safety depends on the soil type, the source of the soil strength parameters, the level of confidence in the values, and the type of slope that is being designed. Slope stability analyses should be performed only by qualified and experienced geotechnical specialists. Guidelines recommended for the analysis of slope stability are given in Table 6-1.

Table 6 -1. Slope stability guidelines for design

Foundation Soil Type	**Type of Analysis**	**Source of Strength Parameters (see Chapter 5)**	**Remarks (see Note 1)**
Cohesive	**Short-term** (embankments on soft clays – immediate end of construction – $\phi = 0$ analysis).	• UU or field vane shear test or CU triaxial test. • Use undrained strength parameters at p_o	Use **Bishop Method**. An angle of internal friction should not be used to represent an increase of shear strength with depth. The clay profile should be divided into convenient layers and the appropriate cohesive shear strength assigned to each layer.
	Stage construction (embankments on soft clays – build embankment in stages with waiting periods to take advantage of clay strength gain due to consolidation).	• CU triaxial test. Some samples should be consolidated to higher than existing in-situ stress to determine clay strength gain due to consolidation under staged fill heights. • Use undrained strength parameters at appropriate p_o for staged height.	Use **Bishop Method** at each stage of embankment height. Consider that clay shear strength will increase with consolidation under each stage. Consolidation test data needed to estimate length of waiting periods between embankment stages. Piezometers and settlement devices should be used to monitor pore water pressure dissipation and consolidation during construction.
	Long-term (embankment on soft clays and clay cut slopes).	• CU triaxial test with pore water pressure measurements or CD triaxial test. • Use effective strength parameters.	Use **Bishop Method** with combination of cohesion and angle of internal friction (effective strength parameters from laboratory test).
	Existing failure planes	• Direct shear or direct simple shear test. Slow strain rate and large deflection needed. • Use residual strength parameters.	Use **Bishop, Janbu or Spencer Method** to duplicate previous shear surface.
Granular	**All types**	• Obtain effective friction angle from charts of standard penetration resistance (SPT) versus friction angle or from direct shear tests.	Use **Bishop Method** with an effective stress analysis.
Note 1: Methods recommended represent minimum requirement. More rigorous methods such as Spencer's method should be used when a computer program has such capabilities.			

6.4.5 Remarks on Safety Factor

For side slopes of routine highway embankments, a minimum design safety factor of 1.25 **as determined by the Ordinary Method of Slices** is used. For slopes that would cause greater damage upon failure, such as end slopes beneath bridge abutments, major retaining structures, and major roadways such as regional routes, interstates, etc., the design safety factor should be increased to at least 1.30 to 1.50. For cut slopes in fine-grained soils, which can lose shear strength with time, a design safety factor of 1.50 is desirable.

6.5 CRITICAL FAILURE SURFACE

The step-by-step procedure presented in the preceding section illustrates how to compute the factor of safety for one selected circular arc failure surface. The complete analysis requires that a large number of assumed failure surfaces be checked in order to find the critical one, i.e., the surface with the lowest factor of safety. This task would obviously be a tedious and time consuming operation if done by hand. Therefore a computer program becomes a valuable tool for performing such computations. Any method for stability analysis is easily adapted to computer solution. For critical circle methods a grid of possible circle centers is defined, and a range of radius values established for each. The computer can be directed to perform stability analyses for each circle center over the range of radii and then to print out all the safety factors or just the minimum one and its radius. A plot of minimum safety factor for each circle center in the form of contours can be used to define the location of the most critical circle and the minimum safety factor as shown in Figure 6-13. The radius of the most critical surface can be used to locate the intersection points of the circle with the ground surface above and below the slope. This is useful in identifying structures above and below the slope that may be potentially impacted by slope instability.

Figure 6-13 shows just one of several ways that computer programs can be used to search for the most critical failure surface. It is beyond the scope of this manual to discuss these in detail. However, the following points should be noted as one uses a computer program for locating the most critical failure surface:

1. Check multiple circle center locations and compare the lowest safety factors. There may be more than one “local” minimum and a single circle center location may not necessarily locate the lowest safety factor for the slope.

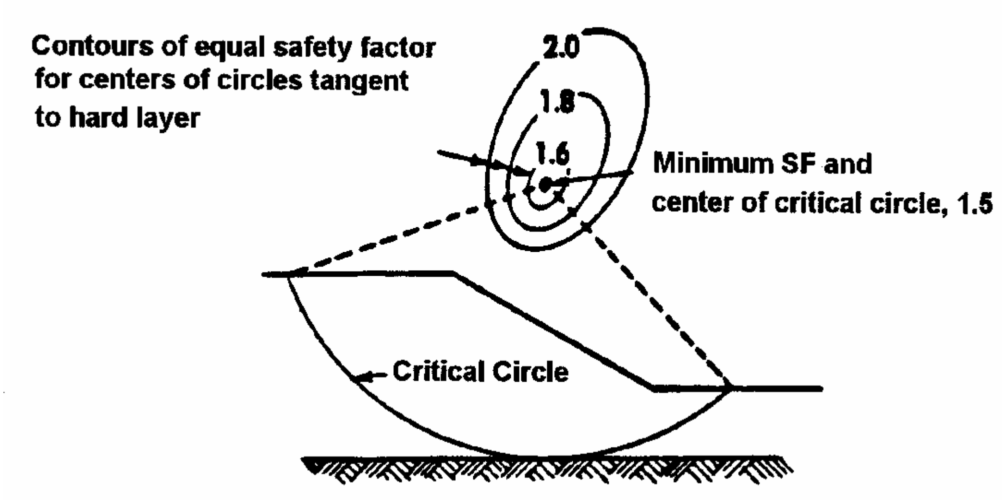


Figure 6-13. Location of critical circle by plotting contours of minimum safety factors for various trial circles.

2. Search all areas of the slope to find the lowest safety factor. The designer may find multiple areas of the slope where the safety factors are low and comparable. In this case, the designer should try to identify insignificant failure modes that lead to low safety factors for which the consequences of failure are small. This is often the case in cohesionless soils, where the lowest safety factor is found for a shallow failure plane located close the slope face.

3. Review the soil stratigraphy for "secondary" geological features such as thin relatively weak zones where a slip surface can develop. Often, circular failure surfaces are locally modified by the presence of such weak zones. Therefore computer software capable of simulating such failures should be used. Some of the weak zones may be man-made, e.g., when new fills are not adequately keyed into existing fills for widening projects.

4. Conduct stability analyses to take into account all possible loading and unloading schemes to which the slope might be subjected during its design life. For example, if the slope has a detention basin next to it, then it might be prudent to evaluate the effect of water on the slope, e.g., perform an analysis for a rapid drawdown condition.

5. Use the drained or undrained soil strength parameters as appropriate for the conditions being analyzed

6. Use stability charts to develop a "feel" for the safety factor that may be anticipated. Stability charts are discussed in the next section. Such charts may also be used to verify the results of computer solutions.

6.6 DESIGN (STABILITY) CHARTS

Slope stability charts are useful for preliminary analysis to compare alternates that may be examined in more detail later. Chart solutions also provide a quick means of checking the results of detailed analyses. Engineers are encouraged to use these charts before performing a computer analysis in order to determine the approximate value of the factor of safety. The chart solution allows some quality control and a check for the subsequent computer-generated solutions.

Slope stability charts are also used to back-calculate strength values for failed slopes, such as landslides, to aid in planning remedial measures. In back-calculating strength values a factor of safety of unity is assumed for the conditions at failure. Since soil strength often involves both cohesion and friction, there are no unique values that will give a factor of safety equal to one. Therefore, selection of the most appropriate values of cohesion and friction depends on local experience and judgment. Since the friction angle is usually within a narrow range for many types of soils and can be obtained by laboratory tests with a certain degree of confidence, it is generally fixed for the back-calculations in practice and the value of cohesion is varied until a factor of safety of one is obtained.

The major shortcoming in using design charts is that most charts are for ideal, homogeneous soil conditions that are not typically encountered in practice. Design charts have been devised with the following general assumptions:

1. Two-dimensional limit equilibrium analysis.
2. Simple homogeneous slopes.
3. Slip surfaces of circular shapes only.

It is imperative that the user understands the underlying assumptions for the charts before using them for the design of slopes.

Regardless of the above shortcomings, many practicing engineers use these charts for non-homogeneous and non-uniform slopes with different geometrical configurations. To do this correctly, one must use an average slope inclination and weighted averages of c, ϕ, or c', ϕ' or c_u calculated on the basis of the proportional length of slip surface passing through different relatively homogeneous layers. Such a procedure is extremely useful for preliminary analyses and saves time and expense. In most cases, the results are checked by performing detailed analyses using more suitable and accurate methods, for example, one of the methods of slices discussed previously.

6.6.1 Historical Background

Some of the first slope stability charts were published by Taylor (1948). Since then various charts were developed by many investigators. Two of the most common stability charts are presented in this manual. These were developed by Taylor (1948) and Janbu (1968).

6.6.2 Taylor's Stability Charts

Taylor's Stability Charts (Taylor, 1948) were derived from solutions based on circular failure surfaces for the stability of simple, homogeneous, finite slopes without seepage (i.e., condition of effective stress). The general equations that Taylor developed as the basis for his stability charts are relationships between the height (H) and inclination (β) of the slope, the unit weight of the soil (γ), and the values of the soil's *developed* (*mobilized*) shear strength parameters, c_d and ϕ_d. These developed (mobilized) quantities are as follows:

$$c_d = \frac{c'}{F_c}; \quad \tan \phi_d = \frac{\tan \phi'}{F_\phi} \tag{6-21}$$

Where F_c is the average factor of safety with respect to cohesion and F_ϕ, is the average factor of safety with respect to friction angle, i.e., $\phi_d = \arctan(\tan \phi' / F_\phi)$. As an approximation, the following equation may be used for the developed friction angle:

$$\phi_d \approx \frac{\phi'}{F_\phi} \tag{6-22}$$

However, for soils possessing both frictional and cohesive components of strength, the factor of safety in slope stability analyses generally refers to the overall factor of safety with respect to shearing strength, FS, which equals τ/τ_d where τ = shear strength and τ_d = the developed (mobilized) shear strength. Therefore, the general Mohr-Coulomb expression for developed shear strength in terms of combined factors of safety is:

$$\tau_d = \frac{\tau}{FS} = \frac{c'}{F_c} + \sigma' \frac{\tan \phi}{F_\phi} \tag{6-23}$$

Equation 6-23 can be re-written in terms of developed shear strength parameters as follows:

$$\tau_d = c_d + \sigma' \tan \phi_d \tag{6-24}$$

There are an unlimited number of combinations of F_c and F_ϕ that can result in a given value of FS. However, Equations 6-21 and 6-23 suggest that for the case where the value of $F_c = F_\phi$, the factor of safety with respect to shearing strength, FS, also equals that value. The importance of this condition will be illustrated in Section 6.6.2.1 by an example problem.

To simplify the determination of the factor of safety, Taylor calculated the stability of a large number of slopes over a wide range of slope angles and developed friction angles, ϕ_d. He represented the results by a dimensionless number that he called the "Stability Number," N_s, which he defined as follows:

$$N_s = \frac{c_d}{\gamma H} = \frac{c'}{F_c \gamma H} \qquad \text{6-25}$$

Equation 6-25 can be rearranged to provide an expression for F_c as a function of the Stability Number and three variables, c', H and γ, as follows:

$$F_c = \frac{c'}{N_s \gamma H} \qquad \text{6-26}$$

Taylor published his results in the form of curves that give the relationship between N_s and slope angle, β, for various values of developed friction angles, ϕ_d, as shown in Figure 6-14. Note that factors of safety do not appear in the chart. The chart is divided into two zones, A and B. As shown in the inset for Zone A, the critical circle for steep slopes passes through the toe of the slope with the lowest point on the failure arc at the toe of the slope. As shown in the inset for Zone B, for shallower slopes the lowest point of the critical circle is not at the toe, and three cases must be considered as follows:

- **Case 2:** For shallow slope angles or small developed friction angles the critical circle may pass below the toe of the slope. This condition corresponds to Case 2 in the inset for Zone B. The values of N_s for this case are given in the chart by the long dashed curves.

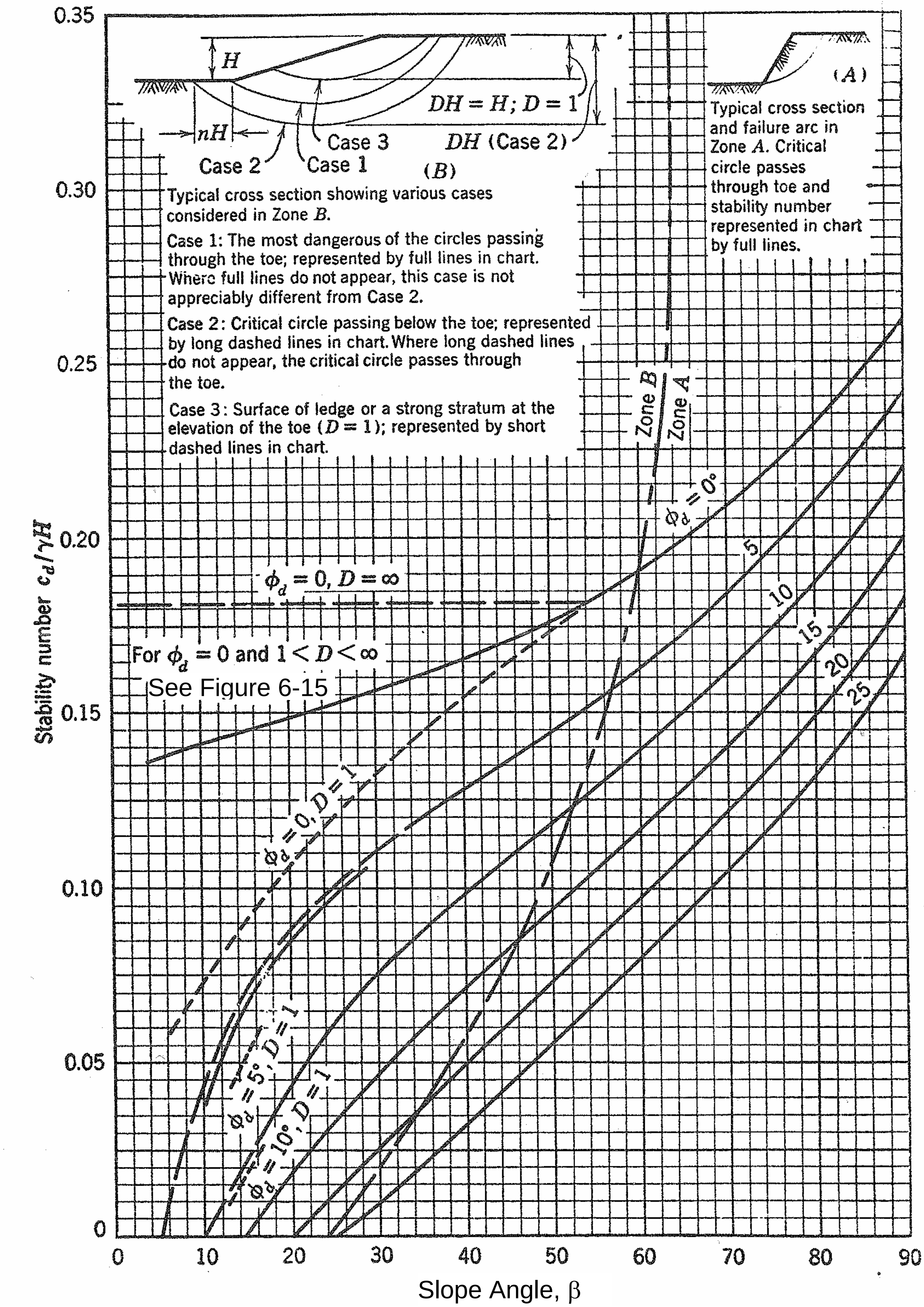


Figure 6-14. Taylor's chart for soils with friction angle (after Taylor, 1948).

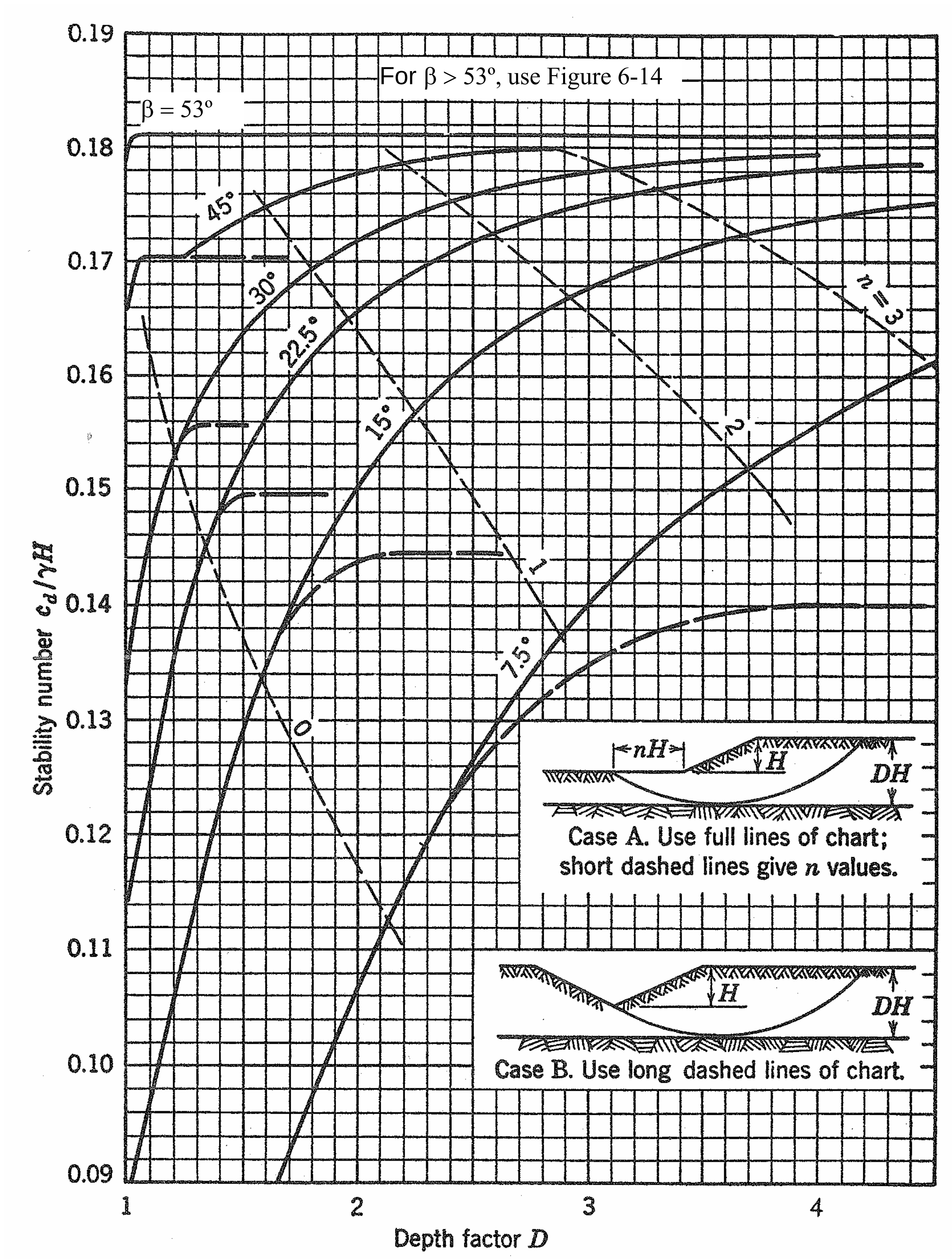


Figure 6-15. Taylor's chart for ϕ' =0 conditions for slope angles (β) less than 54° (after Taylor, 1948).

- **Case 1:** Where long dashed curves do not appear in the chart, the critical circle passes through the toe. This condition corresponds to Case 1. Stability numbers for Case 1 are given by the solid lines in the chart both when there is and when there is not a more dangerous circle that passes below the toe, i.e., the curves for Case 1 are an extension of the curves that correspond to a toe circle failure in Zone A. In both Case 1 and Case 2 the failure circle passes through the soil below the toe of the slope. The depth ratio, *D*, which is a multiple of the slope height H, is used to define the depth (*D*H) from the top of the slope to an underlying strong material through which the failure circle does not pass.

- **Case 3**: This case corresponds to the condition where there is an underlying strong layer at the elevation of the toe (*D*=1). This case is represented by short dashed lines in the chart.

Comment on ϕ_d = 0 condition: The condition of $\phi_d = 0$ in Taylor's Stability Chart is somewhat misleading since, as noted previously, Taylor's charts were derived for simple slopes without seepage, i.e., for an effective stress analysis. The condition of "$\phi_d = 0$" was used by Taylor to simplify the analysis and permit generation of the stability charts by assuming that shear strength is constant with depth. Basically, in the Mohr-Coulomb equation, Taylor assumed an average intergranular pressure, σ'_{avg} instead of an actual value of σ' which varies with depth. Since stability analyses are much simpler to perform when the shear strength is constant, he introduced this concept into his stability charts by considering the effective cohesion to be the average shear strength and by considering the friction angle to be zero. Thus the condition where $\phi_d = 0$ is merely an example of substitution of an average value for a variable quantity. However, in the context of Taylor's definition of $\phi_d = 0$, the stability charts are often used in practice for estimating the factor of safety and location of the critical circle in a homogeneous saturated clay in undrained shear.

As shown in Figure 6-14, the critical circle for the "$\phi_d = 0$" case passes below the toe for slopes with inclinations less than 53°. In practice the depth to which the failure circle extends is limited by an underlying strong material. Thus, the value of N_s for this case is greatly dependent on this limiting value of depth. The chart shown in Figure 6-15 is used exclusively for the "$\phi_d = 0$" case and supplements the curves shown in Figure 6-14 for that condition. The coordinates in Figure 6-15 allow the chart to be used easily and enable the user to evaluate a number of parameters that may be of interest in practice. For example, the chart can be used to determine nH, which is the distance from the toe of the slope to where the critical failure surface passing below the toe may be expected to emerge.

6.6.2.1 Determination of the Factor of Safety for a Slope

As indicated in Section 6.6.2, in order to use Taylor's charts to determine the minimum overall factor of safety with respect to shear strength for a slope of given height H and inclination β having soil properties γ, c′ and φ′, the condition $FS = F_c = F_\phi$ must be satisfied. The general computational approach is as follows:

1. Assume a reasonable value for the common factors of safety $FS = F_c = F_\phi$.
2. Use Equations 6-21 to calculate the corresponding values of c_d and ϕ_d.
3. For the given value of β and the calculated value of ϕ_d, read the corresponding value of the stability number N_s from Figure 6-14.
4. Use an inverted form of Equation 6-26 to calculate the slope height H corresponding to the assumed factor of safety.
5. If the calculated value of H is within an acceptable distance of the actual height, e.g., ± 0.5 feet, the assumed value of the common factor of safety represents the minimum overall factor of safety of the slope with respect to shear strength, F_s.
6. If the calculated value of H is not within the desired acceptable range, the process is repeated with a new assumed value of the common factor of safety until the recomputed value of H falls within that range.
7. The new assumed value of the common factor of safety for subsequent iterations is generally less than the previously chosen value if the calculated value of H is less than the actual value of H. Conversely, a. larger value of the new common factor of safety is assumed if the calculated value of H is greater than the actual value of H.

The use of Taylor's chart is illustrated by the following example.

Example 6-1: Determine the factor of safety for a 30 ft high fill slope. The slope angle is 30º. The fill is constructed with soil having the following properties:
Total unit weight, γ = 120 pcf; Effective cohesion, c′ = 500 psf
Effective friction angle, φ′ = 20º

Solution:

First assume a common factor of safety of 1.6 for both cohesion and friction angle so that $F_c = F_\phi = 1.6$. Since $F_\phi = 1.6$, the developed friction angle, ϕ_d, can be computed as follows:

$$\phi_d = \arctan\left(\frac{\tan\phi'}{F_\phi}\right) = \arctan\left(\frac{\tan 20^o}{1.6}\right) = 12.8^o$$

For ϕ_d = 12.8°, and β = 30°, the value of the stability number N_s from Figure 6-14 is approximately 0.065. Thus, from Equation 6-26

$$0.065 = \frac{500 \text{ psf}}{(1.6)\ (120 \text{ pcf})\ (H)}$$

or

$$H = \frac{500 \text{ psf}}{(1.6)\ (120 \text{ pcf})\ (0.065)} = 40.1 \text{ ft}$$

Since computed height H = 40.1 ft is greater than the actual height of 30-ft, the value of the common safety factor must be greater than 1.6. Assume $F_c = F_\phi = 1.9$ and recompute as follows:

If $F_\phi = 1.9$, then ϕ_d = 10.8° and N_s from Figure 6-14 is approximately 0.073 based on which the recomputed value of H is as follows:

$$H = \frac{500 \text{ psf}}{(1.9)\ (120 \text{ pcf})\ (0.073)} = 30.04 \text{ ft}$$

The height of 30.04 ft is virtually identical to the correct height of 30 ft. Therefore, the minimum factor of safety with respect to shearing strength is approximately 1.9.

Alternate Graphical Approach

An alternate graphical approach for determining the minimum factor of safety with respect to shearing strength is also available. The procedure is as follows:

1. Assume a reasonable value for F_ϕ and calculate ϕ_d.
2. For the given value of β and the calculated value of ϕ_d read the corresponding value of the stability number N_s from Figure 6-14.
3. Use Equation 6-26 to calculate F_c
4. Repeat the process for at least two other assumed values of F_ϕ over a range of expected factors of safety so that at least three pairs of F_ϕ and F_c are obtained.
5. Plot the calculated points on F_c versus F_ϕ coordinates and draw a curve through the points.
6. Draw a line through the origin that represents $F_c = F_\phi$.
7. The minimum overall factor of safety of the slope with respect to shear strength is the value of factor of safety at the intersection of the calculated with the $F_c = F_\phi$ line.

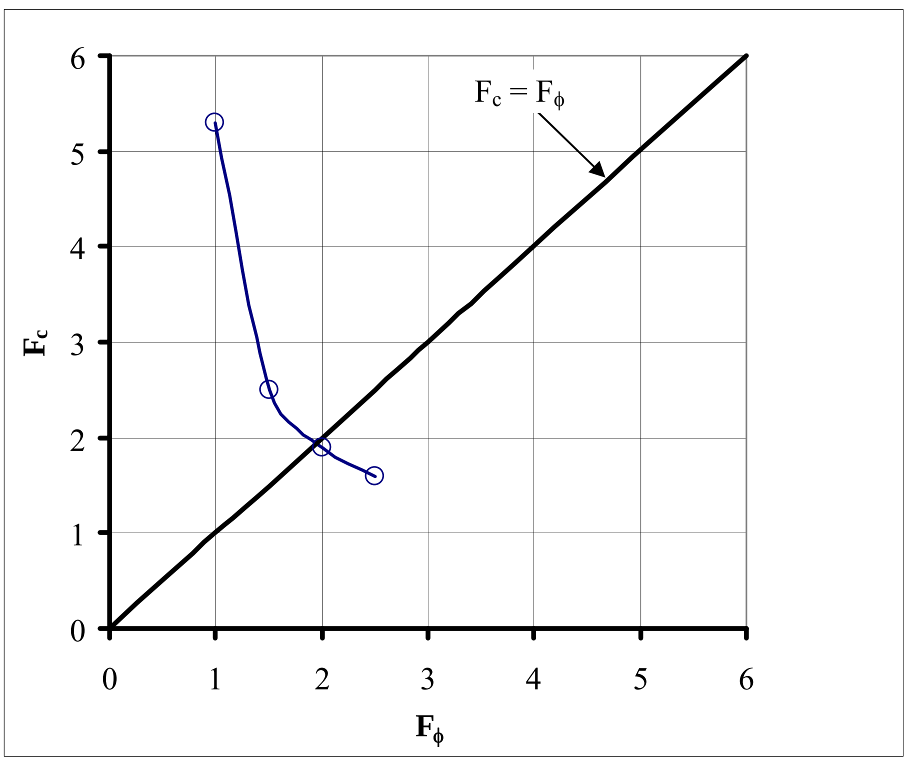
Example 6-1a: Solve Example 6-1 by using the alternate graphical approach.

Solution:

Set up a table for ease of computation (steps 1 through 4) as follows:

Assumed F_ϕ	Calculated ϕ_d	N_s from Figure 6-14	Calculated F_c
1.0	20	0.026	5.3
1.5	14	0.055	2.5
2.0	10	0.075	1.9
2.5	8	0.087	1.6

As shown in the figure below, plot the data as per steps 5 and 6 of the procedure. Read the value of $F_c = F_\phi = 1.95$ at the intersection of plotted curves. This value is the minimum factor of safety with respect to shearing strength, FS. This value is close to the value of 1.9 calculated in Example 6-1. This example problem is also solved by the use of a computer program, ReSSA, in Appendix D and the computer analysis yielded a FS =1.96 which is close to the values computed by the use of stability chart.



6.6.3 Janbu's Stability Charts

Janbu (1968) published stability charts for slopes in soils with uniform strength for $\phi = 0$ and $\phi > 0$ conditions. These charts are presented in Figures 6-16 through 6-19. This series of charts accounts for several different conditions and provides factors for surcharge loading at the top of the slope, submergence, and tension cracks that can be expected to influence the design of typical highway slopes.

The stability chart for slopes in soils with uniform shear strength throughout the depth of the layer and with $\phi = 0$ is shown in Figure 6-16. Charts for correction factors for the conditions when surcharge loads, submergence and tension cracks are present are shown in Figures 6-17 through 6-19. Step-by-step guidance for the use Janbu's charts follows.

<u>Steps for using Janbu's Charts on Figures 6-16 through 6-19, for $\phi = 0$ material.</u>

Step 1. Use the chart at the bottom of Figure 6-16 to determine the position of the center of the critical circle, which is located at a coordinate point defined by X_o, Y_o with respect to a cartesian coordinate system whose origin is at the toe of the slope. Following are some guidelines that can be used to identify the critical center:

- For slopes steeper than 53°, the critical circle passes through the toe. For slopes flatter than 53°, the critical circle passes below the toe.
- In addition to the toe circle, at least four circles with different depths below the toe, D, should be analyzed to ensure that the actual minimum factor of safety and the actual critical circle have been found. The following suggestions may be used to select the circles (Duncan and Wright, 2005):
  - If there is water outside the slope, a circle passing above the water may be critical.
  - If a soil layer is weaker than the one above it, the critical circle may extend into the lower (weaker) layer. This applies to layers both above and below the toe.
  - If a soil layer is stronger than the one above it, the critical circle may be tangent to the top of the stronger layer.

For each of the assumed circles, perform Steps 2 to 6.

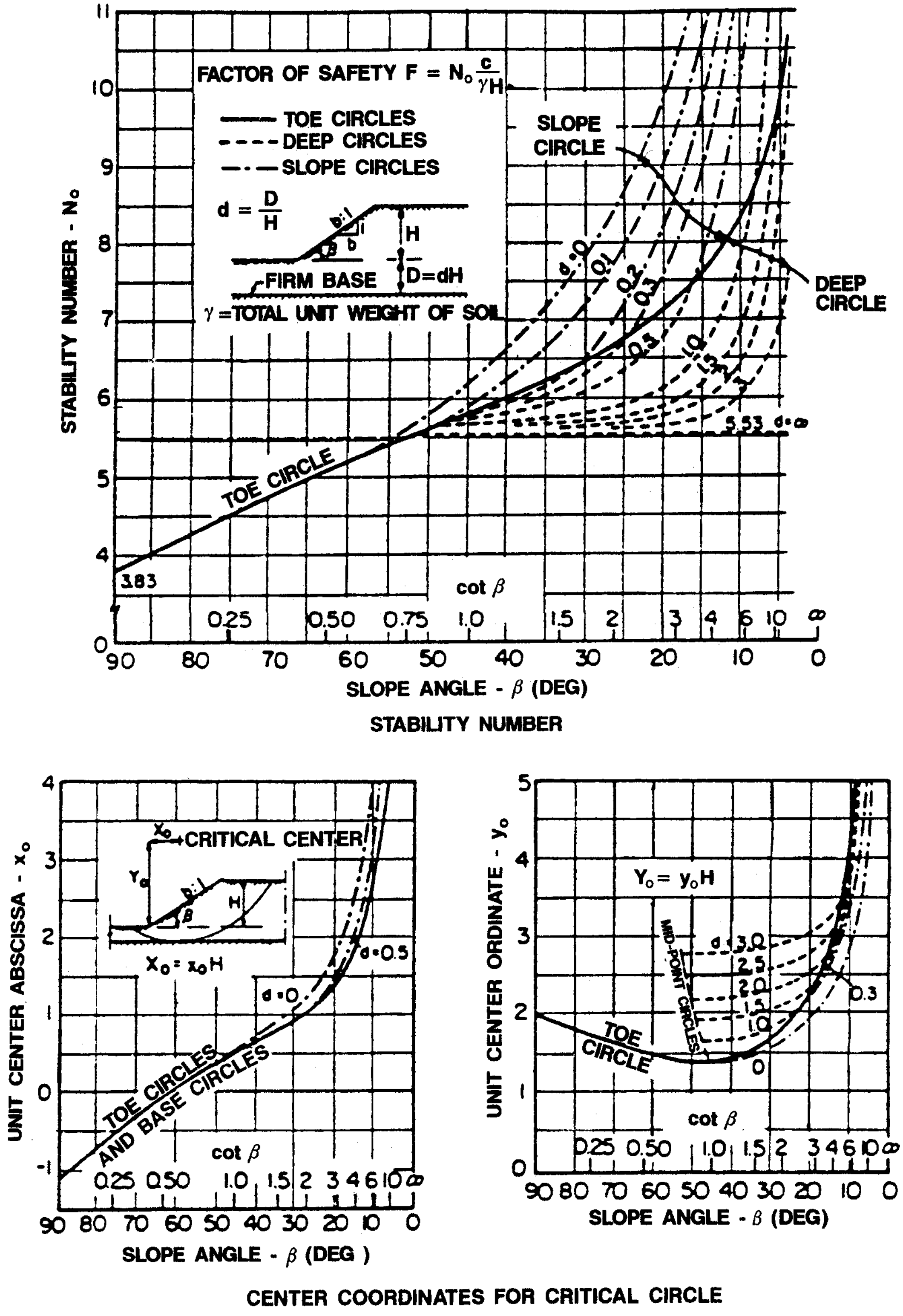


Figure 6-16. Stability charts for $\phi = 0$ soils (Janbu, 1968).

Step 2. Using the assumed critical circle as a guide, estimate the average value of strength, c, by calculating the weighted average of the strengths along the failure surface. The number of degrees intersected along the arc by each soil layer as a percentage of the entire angle subtended by the arc is used as the weighting factor.

Step 3. Calculate the depth factor, d where d = D/H. (Note that the depth factor, d, for Janbu's charts is different from the depth ratio *D* for Taylor's chart.)

Step 4. Calculate P_d by using the following equation:

$$P_d = (\gamma H + q - \gamma_w H_w)/(\ \mu_t\ \mu_q \mu_w) \qquad \text{6-27}$$

where: q = surcharge load
γ_w = unit weight of water
H_w = depth of water outside the slope
μ_t = tension crack correction factor (Figure 6-17)
μ_q = surcharge correction factor (Figure 6-18, top)
μ_w = submergence correction factor (Figure 6-18, bottom)

Step 5. Use the chart at the top of Figure 6-16 to determine the value of the stability number, N_o, which depends on the slope angle β, and the value of d.

Step 6. Calculate the factor of safety (FS) by using the following equation:

$$FS = N_o c/P_d \qquad \text{6-28}$$

Step 7. Repeat Steps 2 to 6 for all the circles assumed in Step 1. Compare the FS to obtain the most critical circle as the circle with the lowest FS. If it appears that the minimum FS is for a circle close to the toe, i.e., d=0, then it is prudent to check if the critical failure surface is within the height of the slope, H. In this case, the toe of the slope is adjusted to the point of intersection of assumed circle with the slope and all dimensions, (i.e., D, H, and H_w) are adjusted accordingly in the calculations and steps 1 to 6 are repeated (Duncan and Wright, 2005).

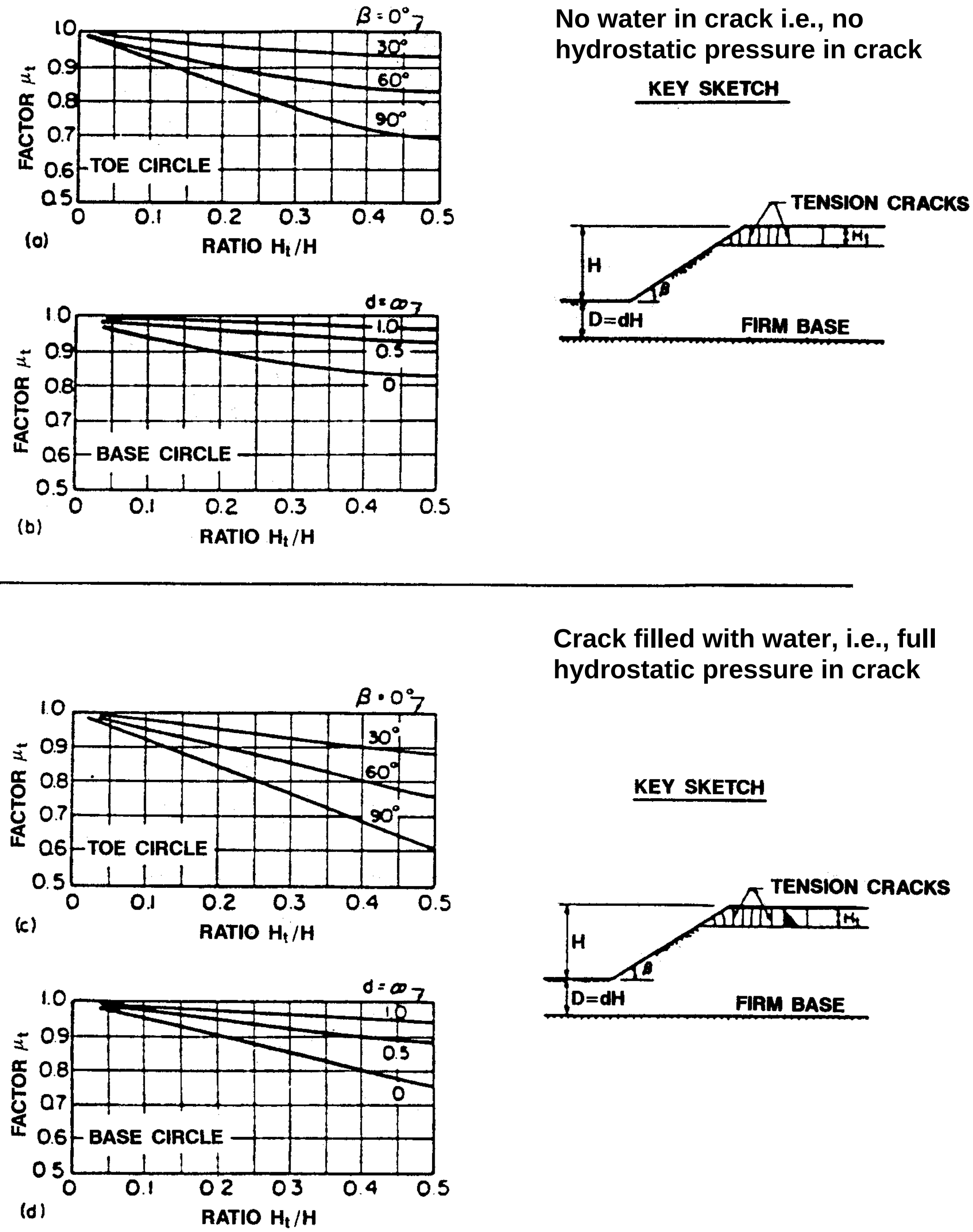


Figure 6-17. Reduction factors to account for tension cracks to be used with stability charts for ϕ=0 and ϕ > 0 soils (Janbu, 1968).

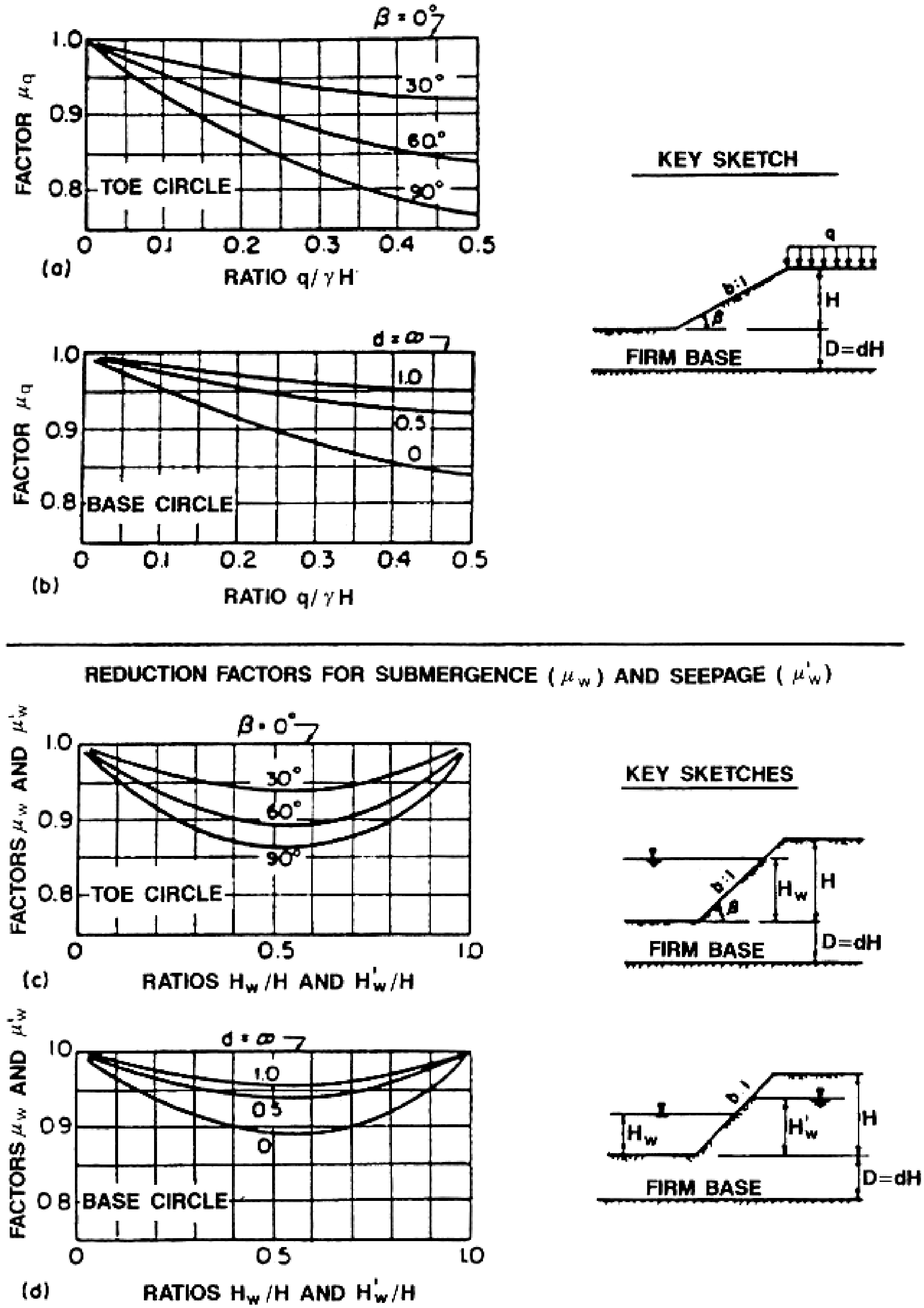


Figure 6-18. Reduction factors to account for surcharge (upper) and submergence and/or seepage (lower) to be used with stability charts for ϕ=0 and ϕ > 0 soils (Janbu, 1968).

Steps for using Janbu's Charts on Figures 6-17 through 6-19, for $\phi > 0$ materials.

Step 1. Use judgment to estimate the location of the critical circle. For most conditions of simple slopes in uniform soils with $\phi > 0$, the critical circle passes through the toe of the slope. The critical stability numbers given in Figure 6-19 were developed from analyses of toe circles.

Where conditions are not uniform and there is a weak layer beneath the toe of the slope, a circle passing beneath the toe may be more critical than a toe failure. Figure 6-19 may be used to calculate the factor of safety for such cases provided the values of c and ϕ used in the analysis represent the correct average values for the circle considered.

If there is a weak layer above the toe of the slope, a circle passing above the toe of the slope may be more critical. Similarly, if there is water outside the toe of the slope, a circle passing above the water may be more critical. When these types of circles are analyzed, the value of H should be equal to the height from the base of the weak layer, or the water level, to the top of the slope.

Step 2. Use the estimated circle in Step 1 as a guide to estimate the average values of c and ϕ. This can be done by calculating the weighted average values of c and ϕ. The number of degrees intersected along the arc by each soil layer as a percentage of the entire angle subtended by the arc is used as the weighting factor for each parameter.

Step 3. Calculate P_d by using Equation 6-27.

Step 4. Calculate P_e by using the following equation:

$$P_e = (\gamma H + q - \gamma_w H'_w)/(\mu_q \mu'_w) \qquad \text{6-29}$$

where: H'_w = height of water within the slope (Figure 6-18, bottom)
μ_q = surcharge correction factor (Figure 6-18, top)
μ'_w = seepage correction factor (Figure 6-18, bottom)

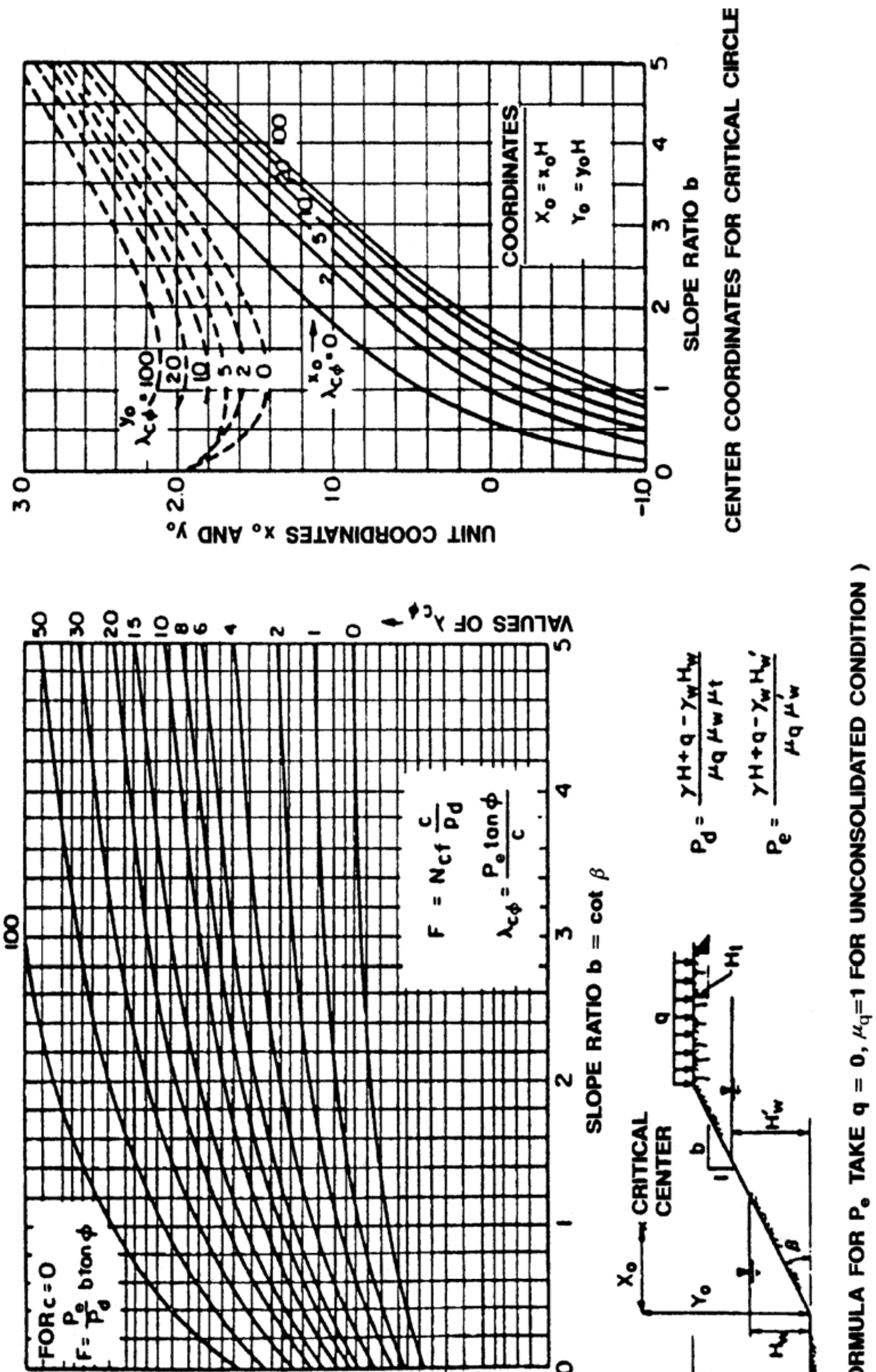


Figure 6-19. Stability charts for ϕ > 0 (Janbu, 1968).

Step 5. Calculate the dimensionless parameter $\lambda_{C\phi}$ by using the following equation:

$$\lambda_{C\phi} = P_e \tan\phi / c \qquad \text{6-30}$$

For c=0, $\lambda_{C\phi}$ is infinite therefore skip to Step 6.

Step 6. Use the chart in Figure 6-19 to determine the value of the critical stability number, N_{cf}, which is dependent on the slope angle, β, and the value of $\lambda_{C\phi}$.

Step 7. Calculate the factor of safety for the slope as follows:

For $c > 0$ $\qquad FS = N_{cf}\, c / P_d$ $\qquad$ 6-31

For $c = 0$ $\qquad FS = P_e\, b \tan\phi / P_d$ $\qquad$ 6-32

Step 8. Determine the actual location of the critical circle by using the chart on the right side of Figure 6-19. The center of the circle is located at a coordinate point defined by X_o, Y_o with respect to a cartesian coordinate system whose origin is at the toe of the slope. The circle passes through the toe of the slope (the origin), except for slopes flatter than 53°, where the critical circle passes tangent to the top of firm soil or rock. If the critical circle is much different from the one assumed in Step 1 for the purpose of determining the average strength, Steps 2 through 8 should be repeated.

If a slope contains more than one soil layer, it may be necessary to calculate the factor of safety for circles at more than one depth. If the underlying soil layer is weaker than the layer above it, the critical circle will extend into the lower layer, and either a toe circle or a deep circle within this layer will be critical. If the underlying soil layer is stronger than the layer above it, the critical circle may or may not extend into the lower layer, depending on the relative strengths of the two layers. Both possibilities should be examined (Duncan and Wright, 2005).

The use of Janbu's charts is illustrated by the following example.

Example 6-2: Figure 6-20 shows a 35 ft high slope with a grade of 1.5H:1V. The soil properties within the slope and under it are shown on the figure. Groundwater is immediately under the slope. Calculate the factor of safety for a toe circle by using total stress analysis based on the soil properties shown. (Note: The circle in Figure 6-20 is plotted in Step 5 of the solution.)

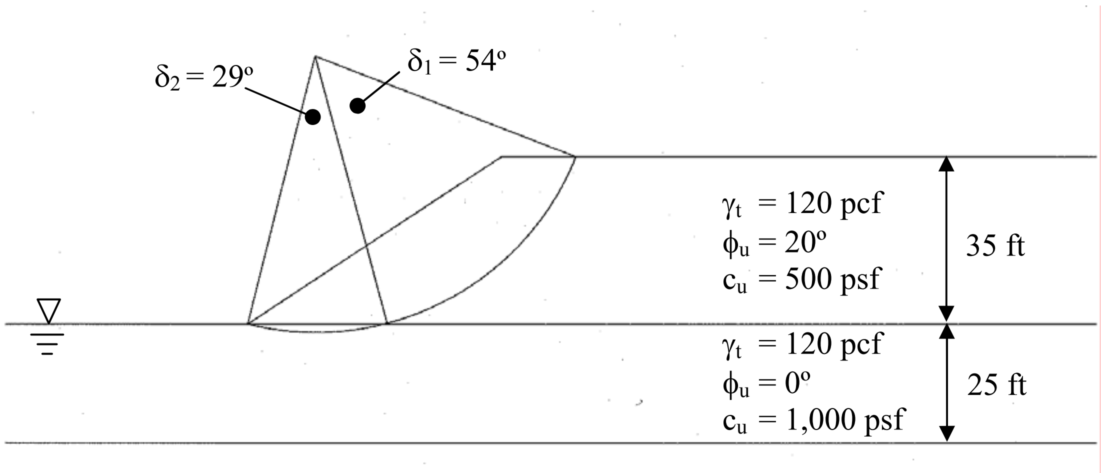


Figure 6-20. Data for Example 6-2.

Solution:

The correction factors μ_t, μ_q, μ_w and μ'_w are all equal to 1.0 since there is no tension crack (H_t = 0), no surcharge on the slope (q = 0), no water above the toe of the slope ($\gamma_w H_w$= 0), and no seepage out of the slope ($\gamma_w H'_w$= 0).

1. Calculate P_d by using Equation 6-27 as follows:

   $P_d = (\gamma H + q - \gamma_w H_w)/(\mu_t\ \mu_q \mu_w)$
   P_d = (120 pcf)(35 ft)/[(1)(1)(1)] = 4,200 psf

2. Calculate P_e by using Equation 6-29 as follows:

   $P_e = (\gamma H + q - \gamma_w H'_w)/(\mu_q \mu'_w)$
   P_e = (120 pcf)(35 ft)/[(1)(1)] = 4,200 psf

3. For a toe circle, it is likely that a segment of the circle will pass through the soil below the toe and the average shear strength parameters along the circle will be different than those for the two layers. However, since at this stage the length of the segment passing through the soil below the toe is unknown, assume that the shear strength values of the soil within the slope height are representative and calculate the parameter $\lambda_{C\phi}$ by using Equation 6-30 as follows:

   $\lambda_{C\phi} = P_e \tan\phi/c$ = (4,200 psf) (tan 20º)/(500 psf) = 3.06

4. From Figure 6-19 obtain the approximate center coordinates of the critical circle by using b=1.5 and $\lambda_{C\phi}$ = 3.06 as follows

$x_o \approx 0.4$ $y_o \approx 1.6$ Thus, $X_o = (H)(x_o) = (35 \text{ ft}) (0.4) = 14 \text{ ft}$
$Y_o = (H)(y_o) = (35 \text{ ft}) (1.6) = 56 \text{ ft}$

5. Plot the critical circle on the given slope, as shown in Figure 6-20. Note that the subtended angles for the failure circle within the slope and the foundation are $\delta_1 = 54^o$ and $\delta_2 = 29$ degrees, respectively.

6. Calculate c_{av}, tan ϕ_{av} and $\lambda_{C\phi}$ based on the angular distribution of the failure surface within the slope and foundation soil using δ_1 and δ_2 as follows:

   $c_{av} = [(54^o) (500 \text{ psf}) + (29^o) (1{,}000 \text{ psf})] / (54^o+29^o) = 674.7 \text{ psf}$

   $\tan \phi_{av} = [(54^o) (\tan 20^o) + (29^o) (\tan 0^o)] / (54^o+29^o) = 0.236$ (or $\phi_{av} = 13.3^o$)

   Thus, according to Equation 6-30;
   $\lambda_{C\phi} = P_e \tan\phi/c = (4{,}200 \text{ psf}) (0.236)/(674.7 \text{ psf}) = 1.47 \approx 1.5$

7. From Figure 6-19 obtain the center coordinates of the critical circle by using b=1.5 and $\lambda_{C\phi} = 1.5$ as follows

   $x_o \approx 0.5$ $y_o \approx 1.55$ Thus, $X_o = (H)(x_o) = (35 \text{ ft}) (0.5) = 17.5 \text{ ft}$
   $Y_o = (H)(y_o) = (35 \text{ ft}) (1.55) = 54.3 \text{ ft}$

   This circle is close to the circle obtained in the previous iteration, so retain $\lambda_{C\phi} = 1.5$ and $c_{av} = 674.7$ psf .

8. From Figure 6-19, obtain $N_{cf} = 10.0$ for b = 1.5 and $\lambda_{C\phi} = 1.5$.

9. Calculate the factor of safety, FS, by using Equation 6-28 as follows :

   $FS = N_o c/P_d = (10) (674.7 \text{ psf}) / (4{,}200 \text{ psf})$
   $FS = 1.61$

   This calculation sequence is only for a given circle. This sequence is repeated for several circles and the resulting FS compared to find the minimum FS. With the advent of computer programs, this method is now more often used to verify the results of the computer generated most critical circle rather than computing the minimum FS by repeating the above sequence of calculations for several circles.

6.7 SLIDING BLOCK FAILURE

A "sliding block" type failure can occur where:

1. the foundation soil contains thin seams of weak clay or organic soils,

2. a shallow layer of weak soil exists at the ground surface and is underlain by firm soil, and

3. the foundation soil contains thin sand or silt lenses sandwiched between less permeable soil. The weak layers or lenses provide a plane of weakness along which sliding can occur. In the case of sand or silt lenses trapped between less permeable soils, the mechanism that can cause sliding is as follows. As the fill load is placed, the water pressure is increased in the sand or silt lense. Since the water cannot escape due to the impermeable soil above and below, the sand or silt loses frictional strength as a result of the intergranular effective stress between soil grains being decreased due to the excess pore water pressure.

Typical "sliding block" type failures are illustrated in Figure 6-21. When sliding occurs, an active wedge type failure occurs through the fill and a passive wedge type failure occurs below the fill toe as soil in the toe area is pushed out of the way. The sliding mass moves essentially as a block, thus the term "sliding block." These concepts are illustrated in Figure 6-22.

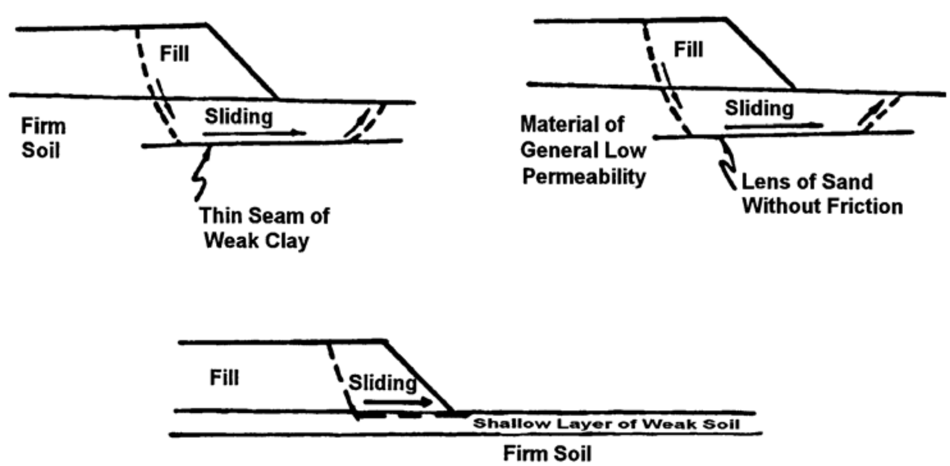


Figure 6-21. Sliding block failure mechanism.

6.7.1 Sliding Block – Hand Method of Analysis

A simple sliding block analysis to estimate the factor of safety against sliding is straightforward and can be performed easily and quickly by hand. For the analysis, the potential sliding block is divided into three parts; (1) an active wedge at the head of the slide, (2) a central block, and (3) a passive wedge at the toe as shown in Figure 6-22.

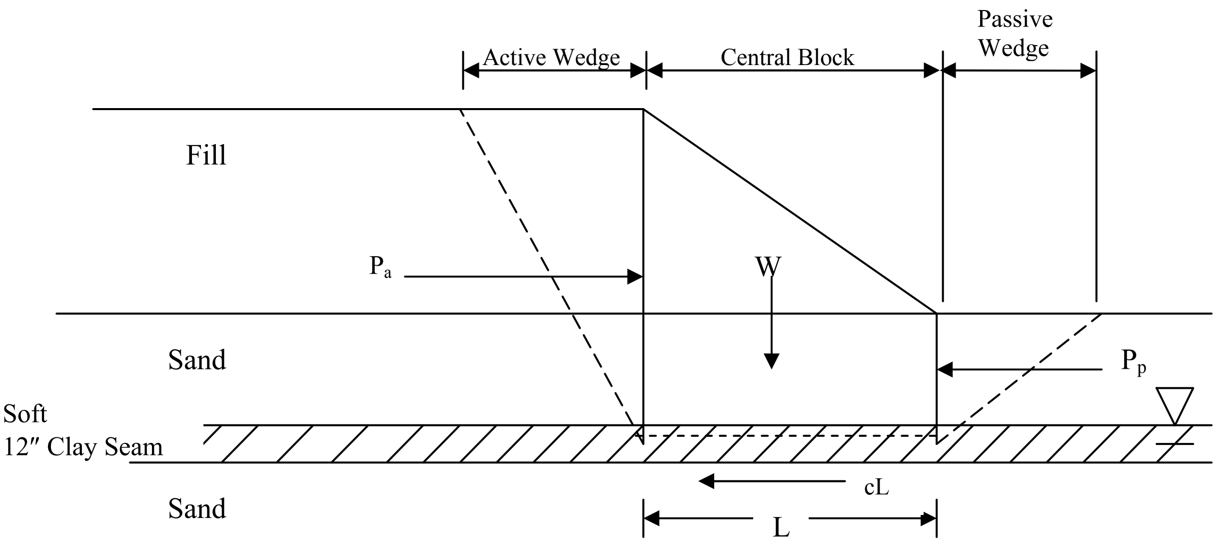


Figure 6-22. Geometry and force components for sliding block analysis.

For the problem illustrated in Figure 6-22, the factor of safety would be computed by summing forces horizontally, to give:

$$FS = \frac{\text{Horizontal Resisting Forces}}{\text{Horizontal Driving Forces}} = \frac{P_p + cL}{P_a} \qquad \text{6-33}$$

where: P_a = Active force (driving)
P_p = Passive force (resisting)
cL = Resisting force due to cohesion of clay

The assumption is made that the loading is rapid so that there is no frictional component of resistance. For convenience of computation of a 1 ft thick slice of embankment is assumed.

Several trial locations of the active and passive wedges must be checked to determine the minimum factor of safety. Note that since wedge type failures occur at the head and toe of the slope, similar to what occurs behind retaining walls, the active and passive forces are assumed to act against vertical planes that are treated as "imaginary" retaining walls, and the active and passive forces are computed the same as for retaining wall problems.

6.7.1.1 Computation of Forces - Simple Sliding Block Analysis

For the simple sliding block problem illustrated Figure 6-22 the forces used to compute the factor of safety can be calculated by using the Rankine approach as follows:

Driving Force – Rankine Active Force

$$P_a = 1/2\ \gamma\ H^2\ K_a \qquad \text{6-34}$$

Where: P_a = active force (kips) (kN)
γ = unit weight of soil in the active wedge (kcf) (kN/m^3)
H = height of soil layer in active wedge (ft) (m)
K_a = active earth pressure coefficient for level ground surface
K_a = $(1-\sin\phi)/(1+\sin\phi) = \tan^2(45^o - \phi/2)$ (see Chapter 2)
ϕ = angle of internal friction of soil in the active wedge.

Resisting Force – Rankine Passive Force

$$P_p = 1/2\ \gamma\ H^2\ K_p \qquad \text{6-35}$$

Where: P_p = passive force (kips) (kN)
γ = unit weight of soil in the passive wedge (kcf) (kN/m^3)
H = height of soil layer in passive wedge (ft) (m)
K_p = passive earth pressure coefficient for level ground surface
K_p = $(1+\sin\phi)/(1-\sin\phi) = \tan^2(45^o + \phi/2)$ (see Chapter 2)
ϕ = angle of internal friction of soil in the passive wedge.

Resisting Force (kips or kN) = Clay cohesion (c in ksf or kPa) x Length of central wedge (L in ft or m)

Computation Tips:

The following design tips should be kept in mind when a sliding block analysis is performed.

1. Be aware that the active or passive wedge can pass through more than one soil type with different strengths or unit weights. If that is the case then the active or passive

pressure distribution changes at the boundary between the different soils. This abrupt change in pressure is due to a change in either the angle of internal friction that affects the value of the earth pressure coefficient K_a or K_p and/or a change in the unit weight of the soil. The easiest way to handle this condition is to compute the active or passive earth pressure distribution diagram for each soil. There may be a discontinuity in the pressure diagram at the boundary between the two different soil layers. Then compute the active or passive force for each segment of the pressure distribution diagram from the area of each segment.

2. When the active or passive pressure is being computed for soils below the ground water table, the buoyant (effective) unit weight of the soil must be used.

The step-by step procedure for the Sliding Block Method of Analysis is illustrated by the following numerical example.

Example 6-3: Find the safety factor for the 20 ft high embankment illustrated in Figure 6-23 by using the simple sliding block method and Rankine earth pressure coefficients. Consider a 1 ft wide strip of the embankment into the plane of the paper.

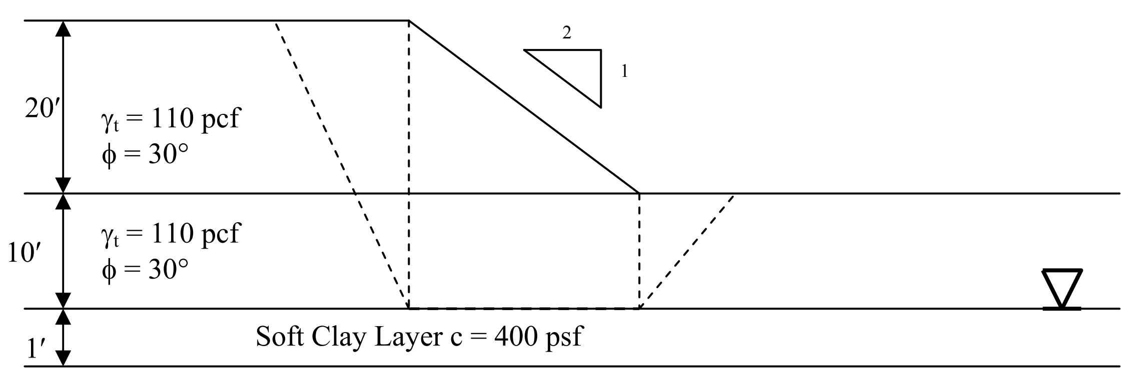


Figure 6-23. Example simple sliding block method using Rankine pressure coefficients.

Solution

Step 1: Compute driving force, P_a, by using Equation 6-34

- Active Driving Force (P_a) by using Equation 6-34

$$P_a = \frac{1}{2}\gamma_t\, H^2\, K_a$$ (use γ_t as the water table is below the failure plane)

$$K_a = \tan^2\left(45 - \frac{\phi}{2}\right) = \tan^2\left(45 - \frac{30^o}{2}\right) = 0.33$$

$$P_a = \frac{1}{2}(0.110\,\text{kcf})(30\,\text{ft})^2(0.33)(1\,\text{ft}) = 16.5\ \text{kips}$$

Step 2: Compute resisting forces

- Central Block Resistance

$$\text{cl} = (0.400\,\text{ksf})(40\,\text{ft})(1\ \text{ft}) = 16.0\ \text{kips}\ (71.1\ \text{kN})$$

- Passive Resisting Force by using Equation 6-35

$$P_p = \frac{1}{2}\gamma_t H^2 K_p$$

$$K_p = \tan^2\left(45 + \frac{\phi}{2}\right) = \tan^2\left(45 + \frac{30^o}{2}\right) = 3.0$$

$$P_p = \left(\frac{1}{2}\right)(0.110\,\text{kcf})(10\,\text{ft})^2(3)(1\,\text{ft}) = 16.5\,\text{kips}$$

Step 3: Compute factor of safety by using Equation 6-33

$$\text{Safety Factor} = \frac{\text{cl} + P_p}{P_a} = \frac{16.0\,\text{kips} + 16.5\,\text{kips}}{16.5\,\text{kips}} = 1.97$$

6.7.2 Computation of Forces - Complicated Sliding Block Analysis

The Rankine approach is a useful tool to portray the mechanism of a planar failure condition. However a general force diagram applicable to a more difficult sliding block type problem can account for the effects of water pressure, cohesion, friction, and a sloping failure plane in the analysis. This analysis procedure, which is described in FHWA (2001a), can be used both to estimate the factor of safety for assumed failure surfaces in design or to "back-analyze" sliding block landslide problems.

Computer solutions are also available for failure modes defined by planar and non-circular surfaces. However most of those solutions do not use the simplified Rankine block approach but rather a more complex failure plane such as that used in Janbu's method. In general a computer solution is preferred for these planar failure problems because of the flexibility they offer in handling a variety of conditions that result in a more complex failure plane.

6.8 SLOPE STABILITY ANALYSIS USING COMPUTER PROGRAMS

Slope stability procedures are well suited to computer analysis due to the interactive nature of the solution. Also, the simplified hand solution procedures do not properly account for interslice forces, irregular failure surfaces, seismic forces, and external loads such as line load surcharges or tieback forces. Several user-friendly computer programs exist to analyze two-dimensional slope stability problems. One of the advantages of a computer program is that it allows parametric studies to be performed by varying parameters of interest, e.g., shear strength parameters. More complex computer programs are available for three dimensional slope stability analysis. As a minimum, a basic two-dimensional slope stability program is recommended for routine use.

Desirable geotechnical features of such a program should include:

- Multiple analysis capability
  a. Circular arc (Bishop)
  b. Non-circular (Janbu)
  c. Sliding block

- Variable input parameters to account for specific conditions
  a. Heterogeneous soil systems
  b. Pseudo-static seismic loads
  c. Ground anchor forces
  d Piezometric levels

- Random generation of multiple failure surfaces with an option to analyze a specific failure surface.

Desirable software features include:

- User-friendly input screens including a summary screen that shows the cross section and soil boundaries in profile.
- Help screens and error tracking messages.
- Expanded output options for both resisting forces in friction, cohesion or tieback computations and driving forces in static or dynamic computations.
- Ordered output and plotting capability for the failure surface of 10 minimum safety factors.
- Documentation of program.

A major problem for software users is technical support, maintenance and update of programs. Slope stability programs are in a continual process of improvement that can be expected to continue indefinitely. Highway agencies should implement only software that is documented and verified and for which the seller agrees to provide full technical support, maintenance and update. The following web page for the FHWA National Geotechnical Team contains links to distributors of FHWA software:

http://www.fhwa.dot.gov/engineering/geotech/index.cfm

Similar services are provided for commercially available slope stability programs such as the ReSSA (2001), SLOPE/W, SLIDE, STABL series (e.g., PCSTABL, XSTABL, GSTABL), and UTEXAS. Appendix D provides an overview of use of the ReSSA program.

Finally, it is extremely important for the designer to understand that the design is only as good as the input parameters. Therefore, the designer should put major emphasis where it belongs, which is on:

- Investigation
- Sampling
- Testing
- Development of soil profile
- Design soil strengths
- Ground water table location

Computer programs are only tools that aid in the design. The answers are only as good as the input data. Don't get carried away with plugging in the numbers and examining the results. You may learn the "garbage in - garbage out" principle the hard way.

6.9 IMPROVING THE STABILITY OF EMBANKMENTS

There are usually several technically feasible solutions to a stability problem. The chosen solution should be the most economical considering the following factors:

1. Available materials.
2. Quantity and cost of materials.
3. Construction time schedules.
4. Line and grade requirements.
5. Right-of-way issues.

6.9.1 Embankment Stability Design Solutions

Table 6-2 presents a summary of practical solutions to mitigate embankment stability problems. Figures 6-24 to 6-26 illustrate some of the mitigation methods listed in Table 6-2. One of the solutions listed in Table 6-2 is the use of ground improvement. This solution can be used for cases where the internal stability of the embankment is not an issue due to the use of competent embankment materials, but the foundation materials are weak enough to affect the stability of the embankment slope. By improving the ground under the embankment, the resistance along the failure surface within the foundation is improved, thereby increasing the safety factor against slope failure. Relatively poor soils can be reinforced with geosynthetics to offset their low shear strength so that acceptable embankments can be constructed.

Another solution is related to reinforcement of the embankment soils themselves. This solution can be used where the foundation is adequate but the locally available soils may not be suitable for construction of embankments at the desired slope angles. In this case, the embankment soils may be strengthened by the inclusion of reinforcements. Such slopes are called reinforced soil slopes (RSS). The RSS technology can be used to construct slopes at angles up to 69-degrees from horizontal. The RSS design method is discussed here as an example of a remediation method. Only the basics of the RSS design method are presented herein. Detailed design procedures for RSS technology can be found in FHWA (2001b).

6.9.2 Design Approach for Reinforced Soil Slopes

The design of internal reinforcement for safe, steep slopes requires a rigorous analysis. The design of the reinforcement for this application is critical, as failure of the reinforcement would result in failure of the slope. The overall design requirements for reinforced slopes are similar to those for unreinforced slopes. The factor of safety must be adequate for both the short-term and long-term conditions and for all possible modes of failure.

Table 6-2
Practical design solutions to mitigate embankment stability problems

*1. Relocate highway alignment.	A line shift of the highway to an area having better soils may be the most economical solution.
*2. Reduce grade line. (flatten slope)	A reduction in grade line will decrease the weight of the embankment and will improve stability (Figure 6-24).
3. Counterweight berms.	A counterweight berm outside of the center of rotation, as illustrated in (Figure 6-25), provides an additional resisting moment that increases the factor of safety. Berms should be built concurrently with the embankment. The embankment should never be completed prior to berm construction since the critical time for shear failure is at the end of embankment construction. The top surface of a berm should be sloped to drain water away from the embankment. Also, care should be exercised in selection of materials and compaction specifications to assure the design unit weight will be achieved for berm construction.
4. Excavation of soft soil and replacement with shear key.	The strength of soft soils is often insufficient to support embankments. In such cases, the soft soils are excavated and replaced with granular material that acts like a shear key (Figure 6-26).
5. Displacement of soft soil.	For deep soft deposits, excavation is difficult. The soft soil can be displaced by generating continuous shear failures along the advancing fill front until the embankment is on firm bottom. The mudwave forced up in front of the fill must be excavated to insure continuous displacement and prevent large pockets of soft soil from being trapped under the fill
6. Slow rate or stage	Many weak subsoils will tend to gain strength during the loading process as consolidation occurs and pore water pressures dissipate. For soils that consolidate relatively fast, such as some silts and silty clays, this method is practical. Proper instrumentation is desirable to monitor the state of stress in the soil during the loading period to insure that loading does not proceed so rapidly that a shear failure occurs. Typical instrumentation consists of slope inclinometers to monitor stability, piezometers to measure excess pore water pressure, and settlement devices to measure the amount and rate of settlement. Planning of the instrumentation program and data interpretation should be done by a qualified and experienced geotechnical engineer. This option could also be used if weak subsoils are pretreated with wick drains
7. Lightweight embankment.	In some areas of the country lightweight materials such as blast furnace slag, shredded rubber tires, or expanded shale are available. The slag material weighs about 80 pcf (12.6 kN/m^3). Sawdust fill weighs about 50 pcf (7.9 kN/m^3) and has a friction angle of 35^o or more. Expanded Polystyrene Foam (EPS) is available throughout the country and weighs 1 to 3 pcf (0.15 to 0.5 kN/m^3). Use of such materials decreases the driving force. Typical advantages and disadvantages of the use of such materials, and specifications for lightweight fills are included in FHWA (2006b).
8. Ground improvement	Recently developed techniques such as stone columns, soil mixing, geosynthetics, soil nailing, ground anchors, and grouting can be used to increase resisting forces. Specialty contractors should be considered for these design solutions.
9. Reinforcement of embankment soils.	The embankment soils can be strengthened by incorporating reinforcements with the compacted soil. The reinforcement generally permits steeper slopes compared to unreinforced embankments.

*Always consider these solutions first since they are relatively simple and inexpensive.

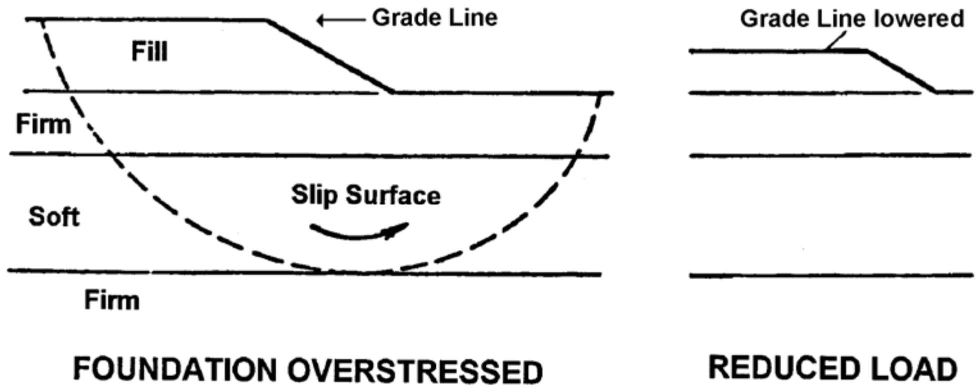


Figure 6-24. Reduction of grade line to improve slope stability.

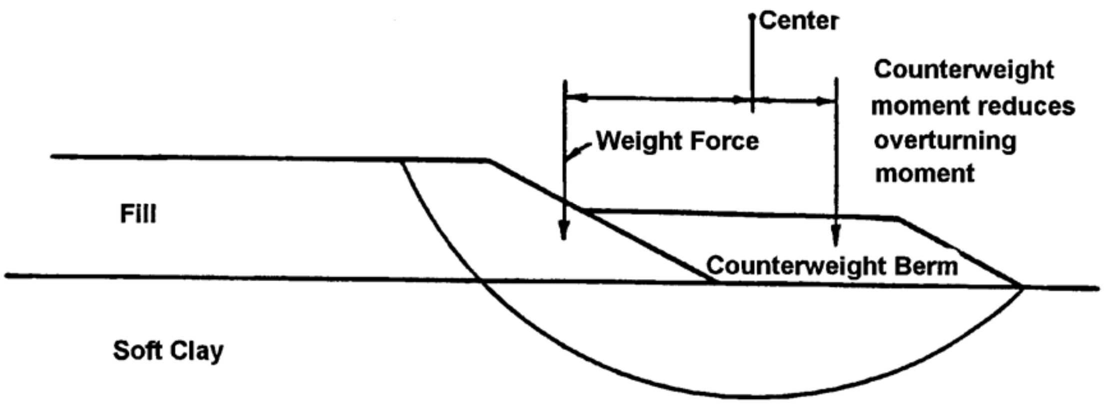


Figure 6-25. Use of counterweight berm to improve slope stability.

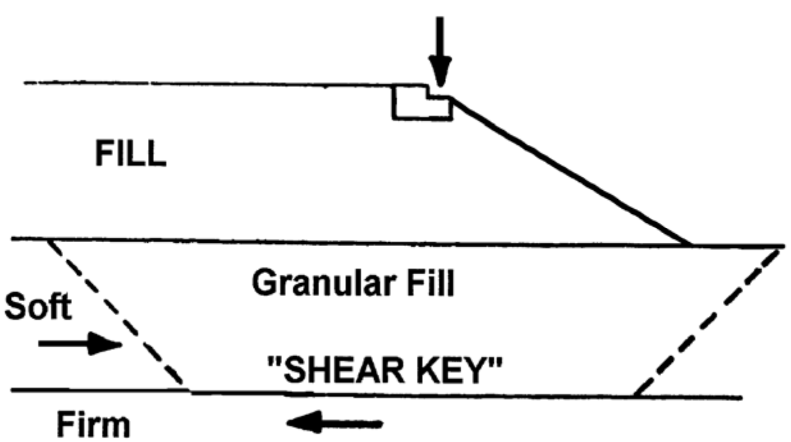


Figure 6-26. Use of shear key to improve slope stability.

As illustrated in Figure 6-27, there are three possible failure modes for reinforced slopes:

1. Internal - the failure plane passes through the reinforcing elements.

2. External - the failure surface passes behind and underneath the reinforced mass. The reinforced mass is the mass of soil that contains the reinforcements.

3. Compound - the failure surface passes behind and through the reinforced soil mass.

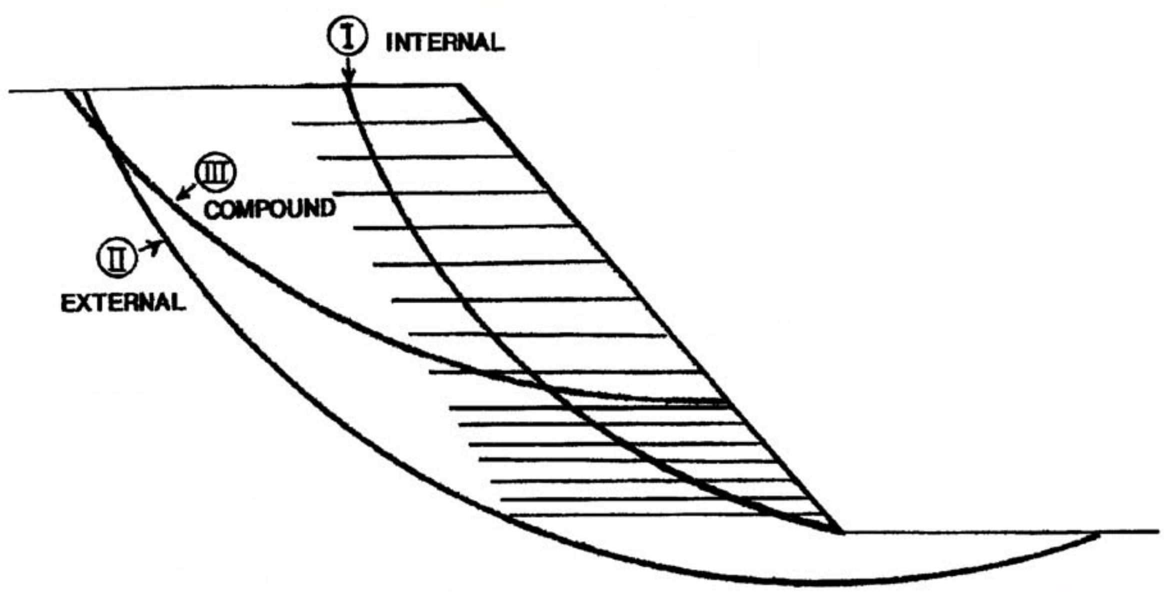


Figure 6-27. Failure modes for Reinforced Soil Slopes.

In some cases, the calculated minimum safety factor can be approximately equal in two or even all three modes if the reinforcement strengths, lengths, and vertical spacing are optimized (FHWA, 2001b). FHWA (2001b) contains a detailed discussion of the analysis and design of RSS'. A convenient chart solution is presented in this manual for preliminary feasibility-level design of the RSS.

6.9.2.1 Preliminary Feasibility Design of RSS

A preliminary design for a feasibility evaluation can be easily made by the use of design charts. These charts can also be used for the final design of low slopes, i.e., slope height less than 20 ft (6 m), where the consequences of failure are not critical. Figure 6-28 is a widely used chart that presents a simplified method based on a two-part, wedge-type failure surface. Use of the chart is limited by the assumptions noted on the figure. Figure 6-28 is not intended to be a single design tool. Other design charts available from the literature could also be used, e.g., FHWA (2001b), Leshchinsky and Perry (1987).

The procedure for using the charts shown in Figure 6-28 is as follows:

1. For an assumed (desired) safety factor, F, determine the factored friction angle, ϕ'_f, in degrees as follows (Note: this is similar to the factored friction angle in Taylor's stability chart):

$$\phi'_f = \arctan\left(\frac{\tan\ \phi'}{F}\right)$$

2. Using ϕ'_f read the force coefficient K from Part A and determine $T_{S\text{-}MAX}$ as follows:

$$T_{S\text{-}MAX} = 0.5\ K\ \gamma_f (H')^2$$

   where $H' = H + q/\gamma$ is the effective height, q = surcharge, and γ_f = fill unit weight.

3. Determine the length of the reinforcement at the top, L_T, and bottom, L_B, of the slope from Part B.

4. Determine the distribution of reinforcement:

   - For low slope heights ($H \leq 20$ ft) assume a uniform reinforcement distribution, and use $T_{S\text{-}MAX}$ to determine the spacing or the required tension, T_{MAX},, for each reinforcement layer.

   - For high slope heights ($H > 20$ ft), divide the slope into two or three reinforcement zones of equal height, and use a factored $T_{S\text{-}MAX}$ in each zone for spacing or design tension requirements.

     <u>For 2 zones:</u>

     $T_{Bottom} = 3/4\ T_{S\text{-}MAX}$

     $T_{Top} = 1/4\ T_{S\text{-}MAX}$

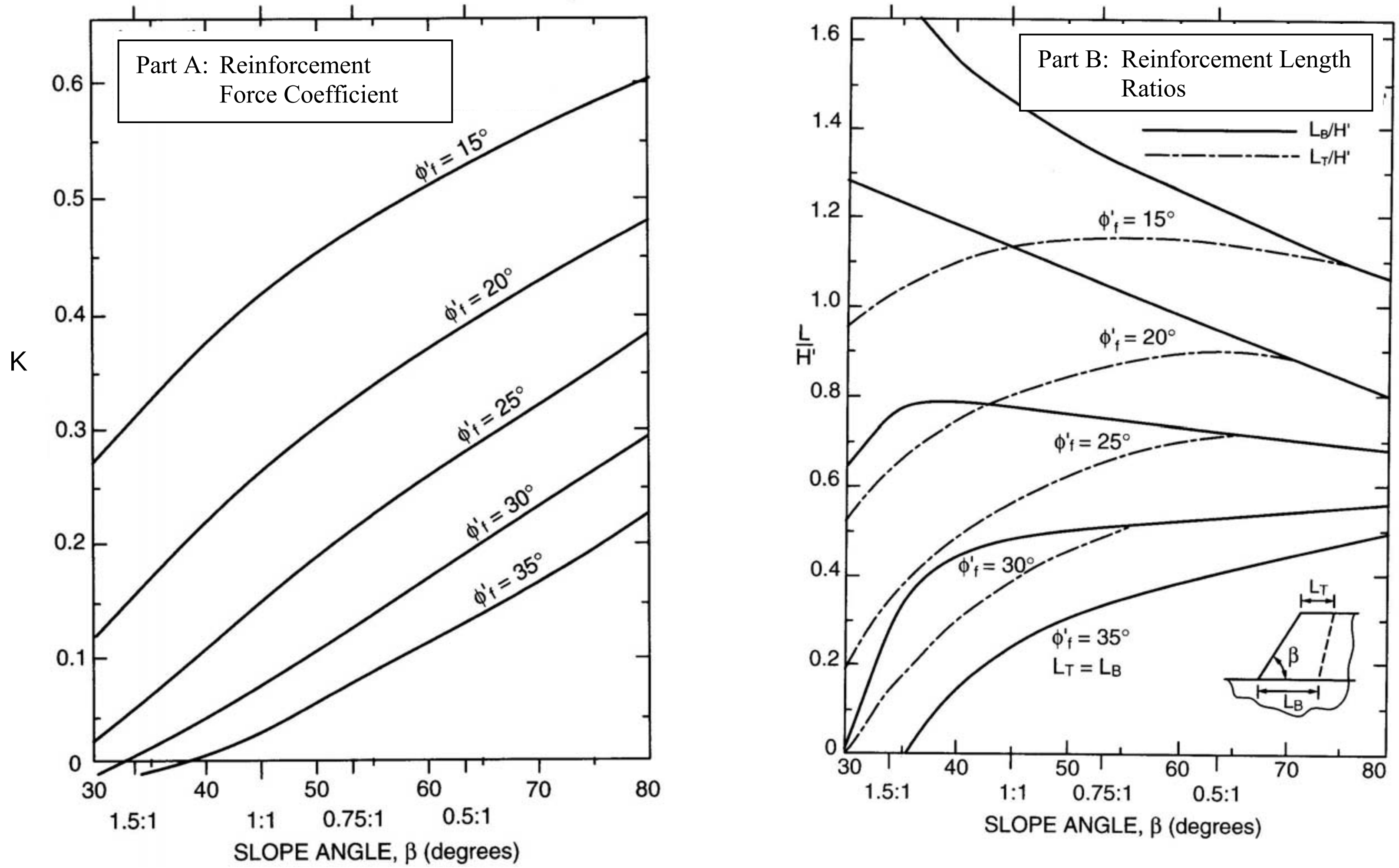
     <u>For 3 zones:</u>

     $T_{Bottom} = 1/2\ T_{S\text{-}MAX}$

     $T_{Middle} = 1/3\ T_{S\text{-}MAX}$

     $T_{Top} = 1/6\ T_{S\text{-}MAX}$

     The force is assumed to be uniformly distributed over the entire zone.



<u>Chart assumptions:</u>

(1) extensible reinforcement, (2) slopes constructed with uniform cohesionless soils (c=0), (3) no pore water pressures within slope, (4) competent, level foundation soils, (5) no seismic forces, (6) uniform surcharge, q, not greater than $0.2\gamma_f H$, (7) relatively high soil/reinforcement interface friction angle = $0.9\phi'$ (may not be appropriate for some geotextiles).

Figure 6-28. Chart solution for determining the reinforcement strength requirements (after Schmertmann, *et al.*, 1987).

- Determine the requirements for vertical spacing of the reinforcement, S_v, or the maximum design tension, T_{MAX}, for each reinforcement layer.

- For each zone, calculate T_{MAX} for each reinforcing layer in that zone based on an assumed S_v or, if the allowable reinforcement strength is known, calculate the minimum vertical spacing and number of reinforcing layers, N, required for each zone based on Equation 6-36 and the use of consistent units.

$$T_a R_c = T_d = \frac{T_{zone} S_v}{H_{zone}} = \frac{T_{zone}}{N} \qquad \text{6-36}$$

where:

T_a = sum of available tensile force per width of reinforcement for all reinforcement layers.

R_c = coverage ratio of the reinforcement that equals the width of the reinforcement, b, divided by the horizontal spacing S_h.

S_v = vertical spacing of reinforcement; multiples of compacted layer thickness for ease of construction.

T_{zone} = maximum reinforcement tension required for each zone.
= $T_{S\text{-}MAX}$ for low slopes (H< 20 ft)

H_{zone} = height of zone.
= T_{top}, T_{middle}, and T_{Bottom} for high slopes (H > 20 ft)

N = number of reinforcement layers.

- In general, use short (4 - 6.5 ft (1.2 – 2 m)) lengths of reinforcement layers to maintain a maximum vertical spacing of 16 in (400 mm) or less for face stability and compaction quality. This short reinforcement should be placed in continuous layers and need not be as strong as the primary load bearing reinforcement, but it must be strong enough to survive construction (e.g., minimum survivability requirements for geotextiles in road stabilization applications in AASHTO M-288) and provide localized tensile reinforcement to the surficial soils.

For detailed analyses required for final design, refer to FHWA (2001b). The computer program ReSSA (2001) noted earlier, can perform analysis and design of reinforced soil slopes using the methods described in FHWA (2001b).

6.10 IMPROVING THE STABILITY OF CUT SLOPES

The two most common types of cut slope failures are deep-seated and shallow surface failures. Both of these types of failure and their mitigation are discussed in this section.

6.10.1 Deep Seated Failure

A deep seated failure usually occurs in slopes cut into clay. The clay has insufficient shear strength to support the slope, and shear failure generally occurs along a circular arc. If the clay contains water-bearing silt or sand layers, seepage forces will also contribute to the instability. Figure 6-29 shows an example of a deep seated failure and a possible design solution. Table 6-3 lists typical design solutions to potential cut slope stability problems in clay.

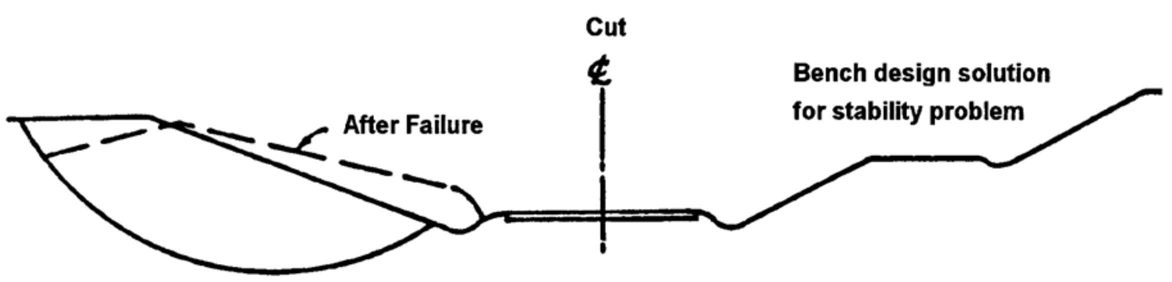


Figure 6-29. Deep seated slope failure (left) and bench slope design (right) to prevent slope failure.

Table 6-3
Typical design solutions to mitigate cut slope stability problems

Design Solution	Effect on Stability
a. Flatten slope.	Reduces driving force.
b. Bench slope.	Reduces driving force.
c. Buttress toe.	Increases resisting force.
d. Lower water table.	Reduces seepage force.
e. Reinforcement (e.g., nails)	Increases resisting force

The design of cut slopes in clay should NOT be based on the undrained strength of the clay determined by tests on samples obtained before the cut is made. Designs based on undrained strength will be unconservative since the effective stress is reduced when the cut is made because load is removed. This decrease in effective stress allows the clay to swell and lose strength if water is made available to the clay as shown in Figure 6-30. Therefore, the design of cut slopes in clays should be based on effective strength parameters so that the reduction in effective stress resulting from the excavation can be taken into account. It is important to remember that an undrained clay in a cut gradually weakens and may fail long after construction.

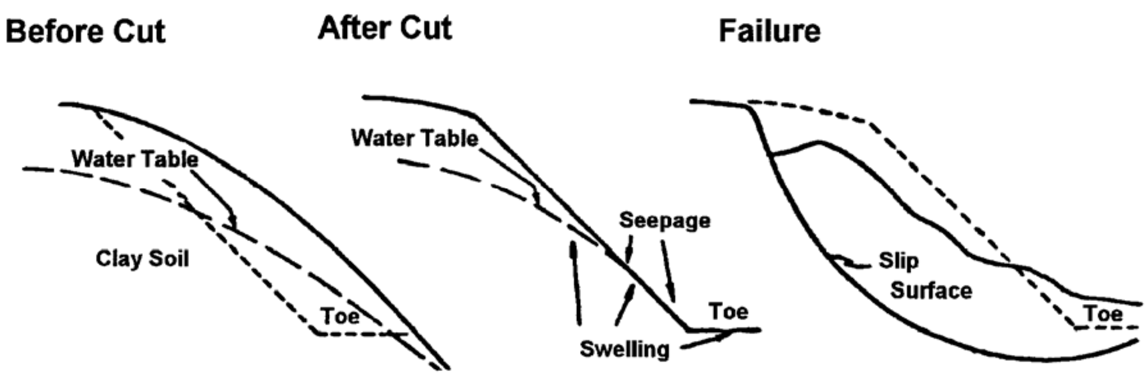


Figure 6-30: Typical cut slope failure mechanism in clay soils.

6.10.2 Shallow Surface Failures

Shallow surface failures (sloughs) are most common in cut slopes in layered clay or silt. This type of failure may involve either an entire slope or local areas in the slope. The prime cause of shallow surface failures is water seepage. Water seepage reduces the strength of the surface soils, causing them to slide or flow.

Sloughing of slopes due to ground water seepage can often be remedied by placing a 2-3 ft (0.6-1 m) thick rock or gravel blanket over the critical area. The blanket reduces the seepage forces, drains the water, and acts as a counter-weight on the unstable soil. The blanket should be "keyed" into the ditch at the toe of the slope. The key should extend about 4 feet (1.2 m) below the ditch line and be about 4 ft (1.2 m) wide. A geotextile should be placed both under the key and against the slope before placement of the gravel blanket. Construction of the gravel blanket should proceed from the toe upwards. The most effective placement is by a dozer that will track over and compact the lower areas of the gravel blanket while the upper areas are being constructed.

6.10.3 Factor of Safety - Cut Slopes

As indicated previously, a minimum design safety factor of 1.25 is used for routine highway embankment side slopes. A minimum factor of safety against sliding of 1.50 is recommended for the stability of cut slopes in fine-grained soils. The greater factor of safety for cut slopes is based upon the knowledge that cut slopes may deteriorate with time as a result of natural drainage conditions that embankments generally do not experience. In addition, there is a greater degree of uncertainty about the homogeneity of the soils in cut slopes than in embankment slopes that are engineered and constructed under controlled conditions.

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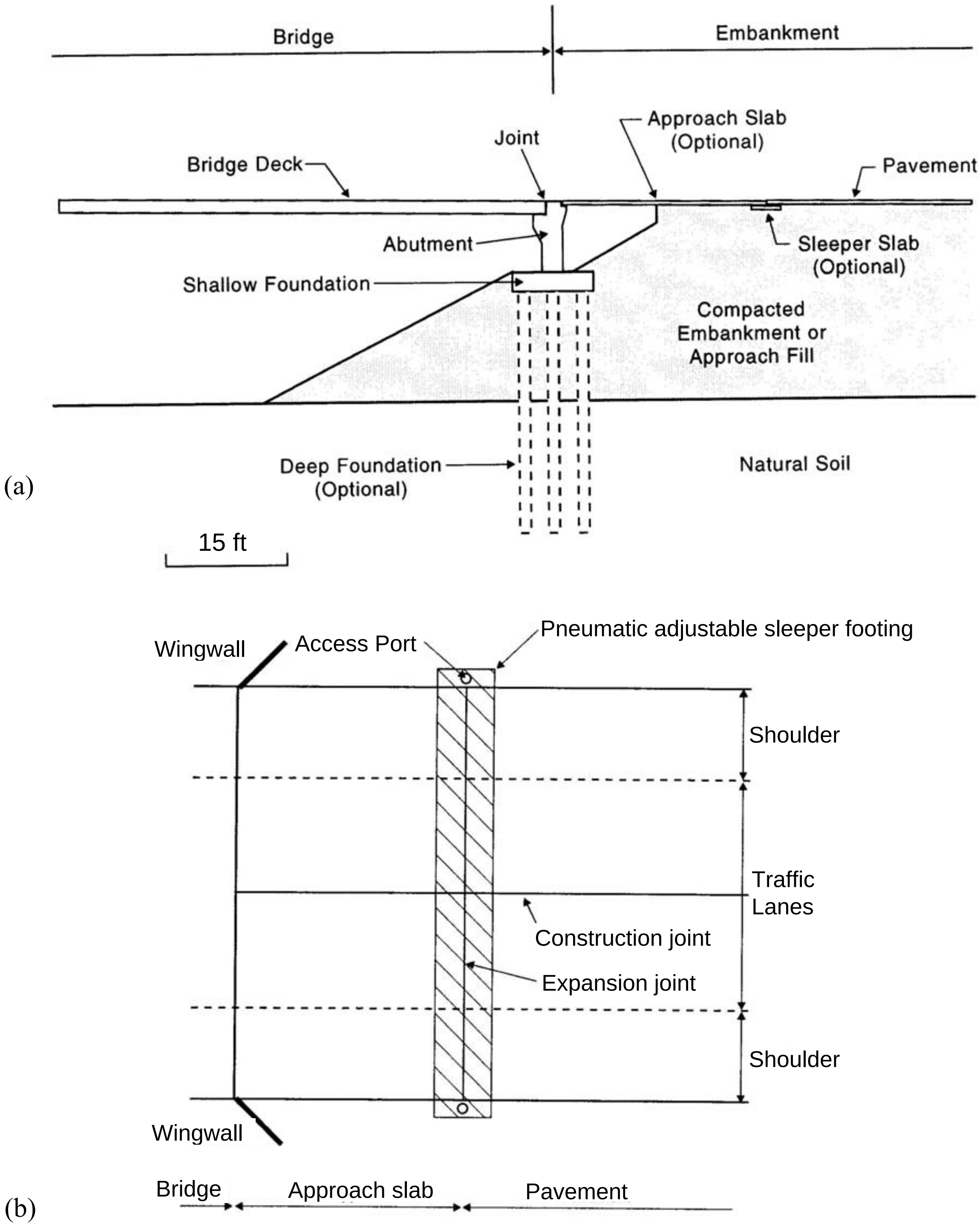
CHAPTER 7.0
APPROACH ROADWAY DEFORMATIONS

Often roadways are constructed on embankment fills to meet the requirements of the vertical grade of a roadway alignment. Fills placed to accommodate the vertical profile as the roadway approaches a bridge are often referred to as “approach embankment fills” or “approach roadway fills.” Typical elements of a bridge approach system are shown in Figure 7-1. The abutment configuration may vary as shown in Figure 7-2. An abutment fill slope is also referred to as an “end-slope.” The common element to all types of abutments is an approach fill. Deformation, both vertical and lateral, of approach fills is the most prevalent foundation problem in highway construction. The embankment deformation near a bridge structure, leads to the ubiquitous “bump at the end of the bridge.” Figure 7-3 shows some of the problems leading to the existence of the bump.

Approach slabs are often used by most state agencies to provide a smooth transition between the bridge deck and the roadway pavement. The slab usually is designed to withstand some embankment settlement and a reduction in subgrade support near the abutment. Joints must be provided to accommodate cyclic thermal movements of the bridge deck, abutment and roadway pavement. Figure 7-1b shows one common joint set. However, if the approach embankments are not properly engineered, the approach slab merely moves the bump at the end of the bridge to the approach slab-roadway interface. Unlike stability problems, the results of approach embankment deformations are seldom catastrophic but the cost of perpetual maintenance of continuing deformation can be immense. The difficulty in preventing these problems is not so much a lack of technical knowledge as a lack of communication between personnel involved in the roadway design and those involved in the structural design and construction.

7.1 TYPICAL APPROACH ROADWAY DEFORMATION PROBLEMS

Roadway designers allow use of inexpensive available soils for approach fills to reduce project costs. The bridge structures are necessarily designed for little or no deformation to maintain specified highway clearances and to insure integrity of structural members. In most agencies the responsibility for approach embankment design is not defined as a structural issue, which results in roadway embankment requirements being used up to the structure. In reality, **the approach embankment requires special materials and placement criteria to prevent internal deformations and to mitigate external deformations.** A discussion of the types of deformation associated with approach fills follows.



Note: This plan detail is only one way of handling the bridge/fill interface. An approach slab with expansion between the superstructure and the approach slab without a sleeper slab is another.

Figure 7-1. (a) Elements of a bridge approach system, (b) Plan view of an approach system (modified after NCHRP, 1997).

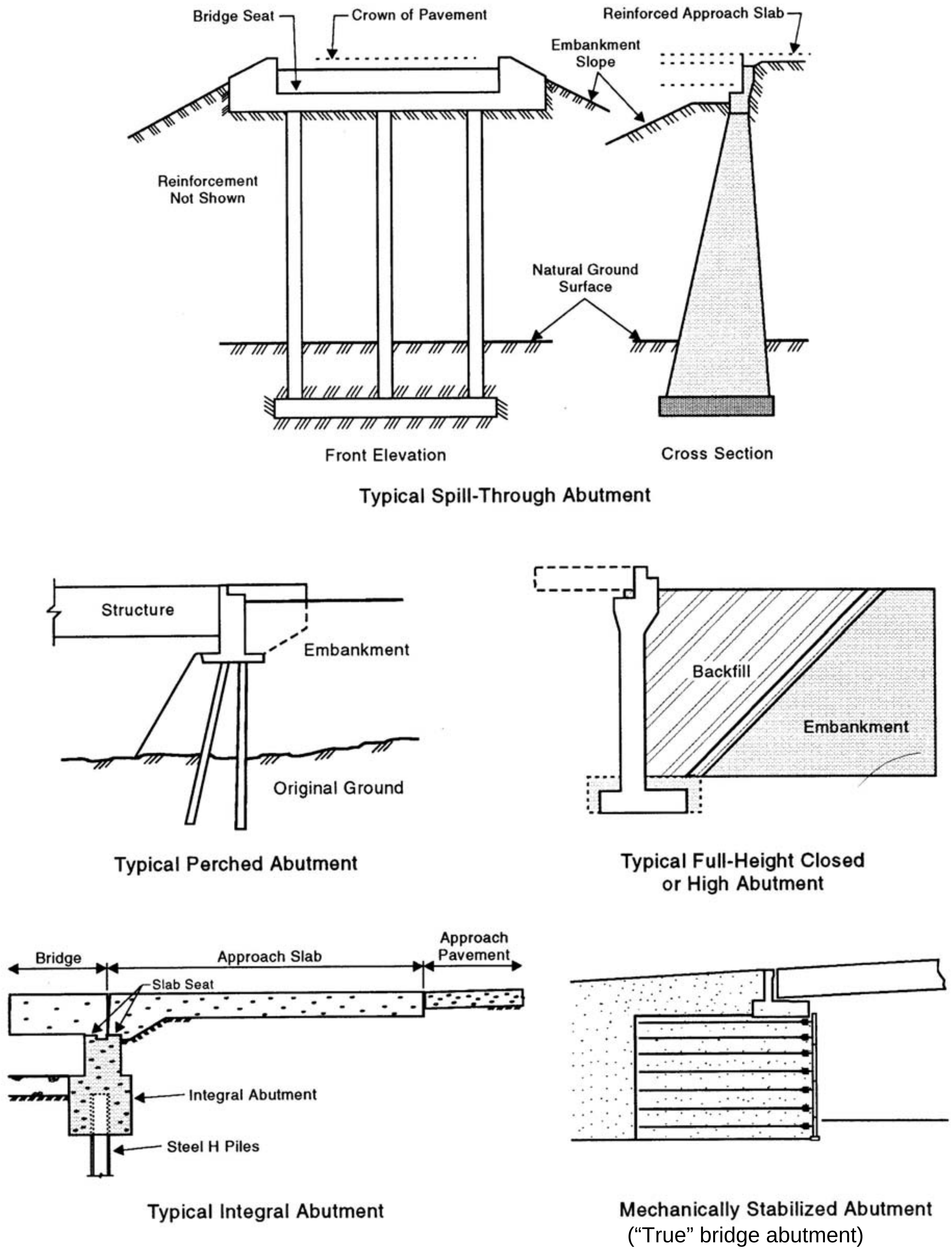


Figure 7-2. Types of abutments (modified after NCHRP, 1990).

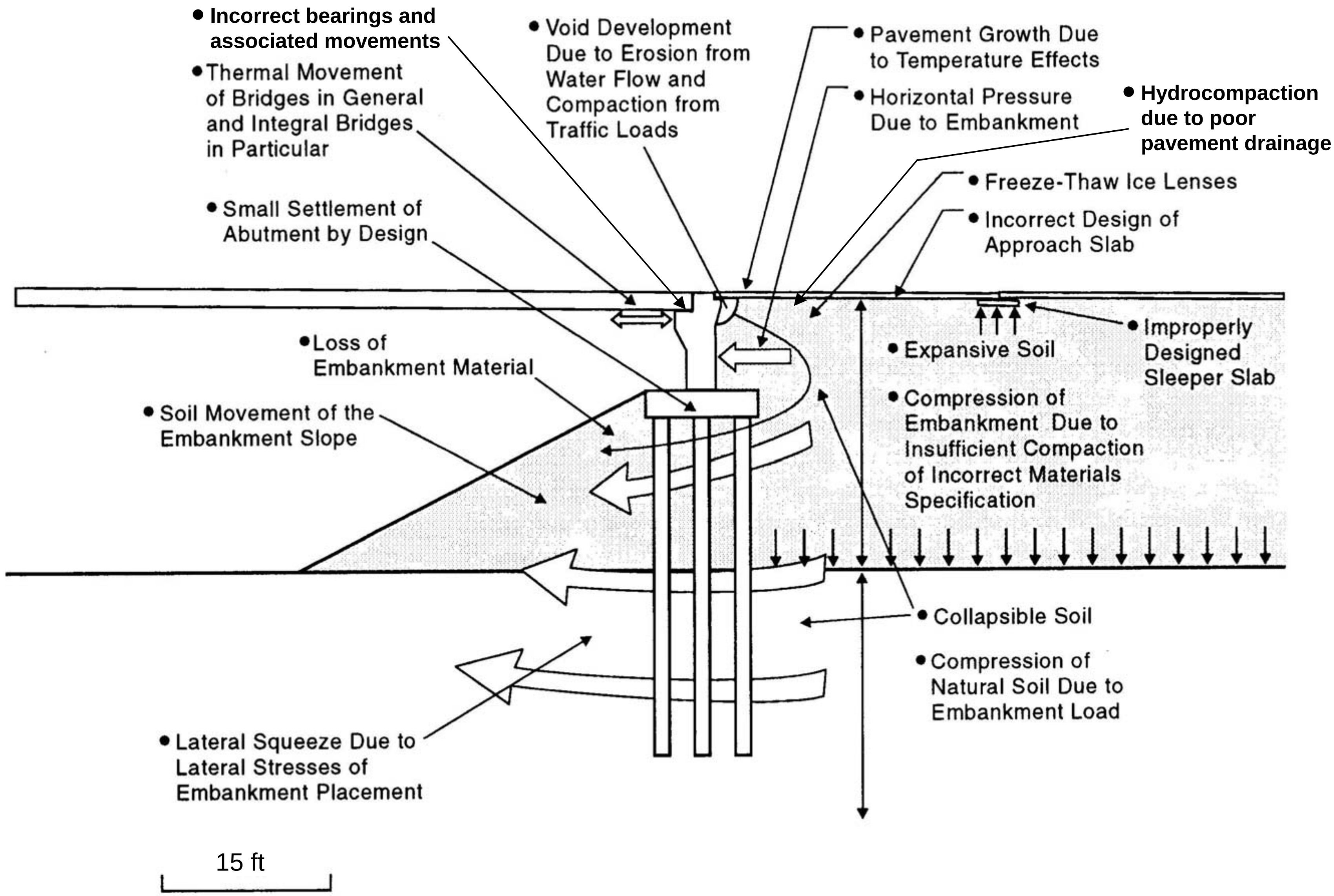


Figure 7-3. Problems leading to the existence of a bump (modified after NCHRP, 1997).

Most state agencies, as noted earlier, use bridge approach slabs to provide a transitional roadway between the pavement on the approach embankment and the actual structure of the bridge. Due to the deformation of the approach embankment fills for various reasons shown in Figure 7-3, these slabs can settle and/or rotate creating problems for the abutment as well as the joints. Depending on the configuration of the approach slab, e.g., how the slab is connected to the abutment and/or the wing walls, voids may develop under the slab as the approach fill settles. Such voids can then fill with water, which can further compound the problem, e.g., water pressures acting against structural elements, softening of the soils with associated reduction in strength, freeze-thaw issues, etc. Due to the above considerations, design problems with approach roadway embankments are classified as follows:

- Internal deformation **within** the embankment
- External deformation in native soils **below** the embankment

As mentioned previously, it is important to realize that the deformation considerations for the embankment include both vertical as well as lateral deformations. Vertical deformations are commonly referred to as “settlements.” Lateral deformations can result in rotation of the structure that is commonly referred to as “tilting.”

Internal deformation is a direct result of compression of the materials used in the construction of the embankment fill. The importance of adequate drainage with respect to the internal behavior of the embankment cannot be overemphasized. Poor drainage can (a) cause softening of the embankment soils leading to vertical and lateral deformations, (b) reduce the stability of soils near the slope leading to lateral deformations and associated vertical deformations near the crest of the slope, and (c) potentially lead to migration of fill material and creation of voids or substantial vertical and lateral deformations.

External deformation is due to the vertical and lateral deformation of the foundation soils on which the embankment is placed. Furthermore, deformation of foundation soils may include both immediate and consolidation deformations depending on the type of foundation soils. Lateral squeeze of the foundation soils can occur if the soils are soft and if their thickness is less than the width of the end slope of the embankment. Consolidation settlement and lateral squeeze are not an issue within embankment fills since coarse-grained soils placed under controlled compaction conditions are generally used.

This chapter discusses internal as well as external deformations of approach fills. Design solutions to mitigate the detrimental effect of these deformations are presented. Guidelines for construction monitoring are also provided.

7.2 INTERNAL DEFORMATION WITHIN EMBANKMENTS

Internal deformation within embankments can be easily controlled by using fill materials that have the ability to resist the anticipated loads imposed on them. A well constructed soil embankment will not excessively deform internally if quality control is exercised with regard to material and compaction. Standard specifications and construction drawings should be prepared for the approach embankment area, normally designated to extend 50 ft (15 m) behind the wingwall. The structural designer should have the responsibility for selecting the appropriate cross section for the approach embankment depending on selection of the foundation type. A typical approach embankment cross section is shown in Figure 7-4.

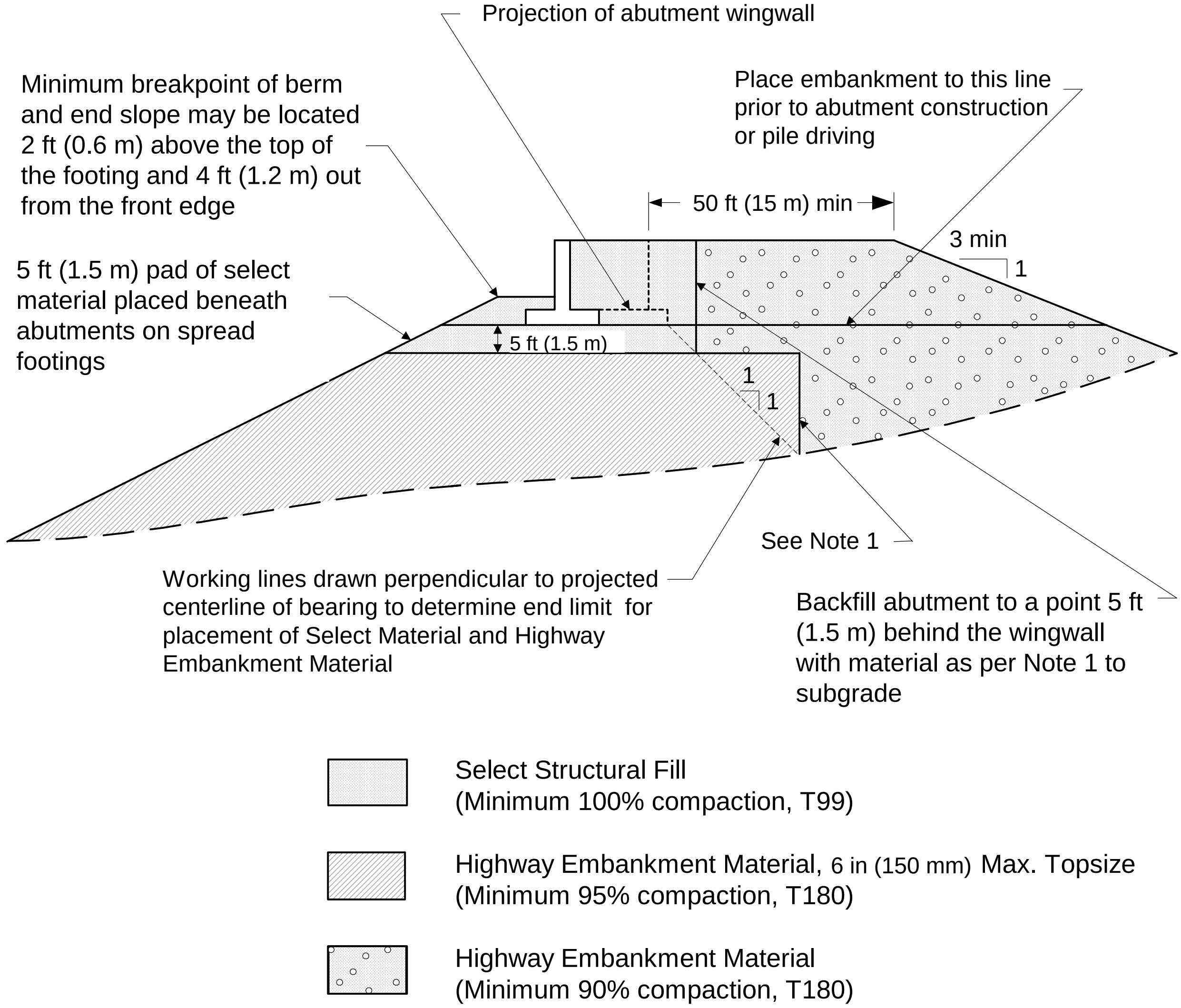
Special attention should be given to the interface area between the structure and the approach embankment, as this is where the "bump at the end of the bridge" occurs. The reasons for the bump are (a) poor compaction of embankment material near the structure, (b) migration of fine soil into drainage material, and (c) loss of embankment material due to poor drainage details as discussed earlier. Poor compaction is usually caused by restricted access of standard compaction equipment. Proper compaction can be achieved by optimizing the soil gradation in the interface area to permit compaction to maximum density with minimum effort. Figure 7-5 shows a detail for placement of drainage material. Considerations for the specification of select structural backfill and underdrain filter material to minimize the “bump” problem are included in the next two sections. Similar drainage results can be obtained by the use of prefabricated geocomposite drains that are attached to the backwall and connected to an underdrain.

7.2.1 General Considerations for Select Structural Backfill

Select structural backfill is usually placed in relatively small quantities and in relatively confined areas. Structural backfill specifications must be designed to ensure construction of a durable, dense backfill. Table 7-1 lists considerations for the specification of select structural backfill.

7.2.2 General Considerations for Drainage Aggregate

The drainage aggregate, such as that used for underdrain filters, should consist of crushed stone, sand, gravel or screened gravel. Suggested gradation for drainage aggregate is provided in Table 7-2. The AASHTO standard gradation No. 57 or 67 should be equally suitable.



Note 1: Highway embankment material and select material shall be placed simultaneously of the vertical payment line

Figure 7-4. Suggested approach embankment details.

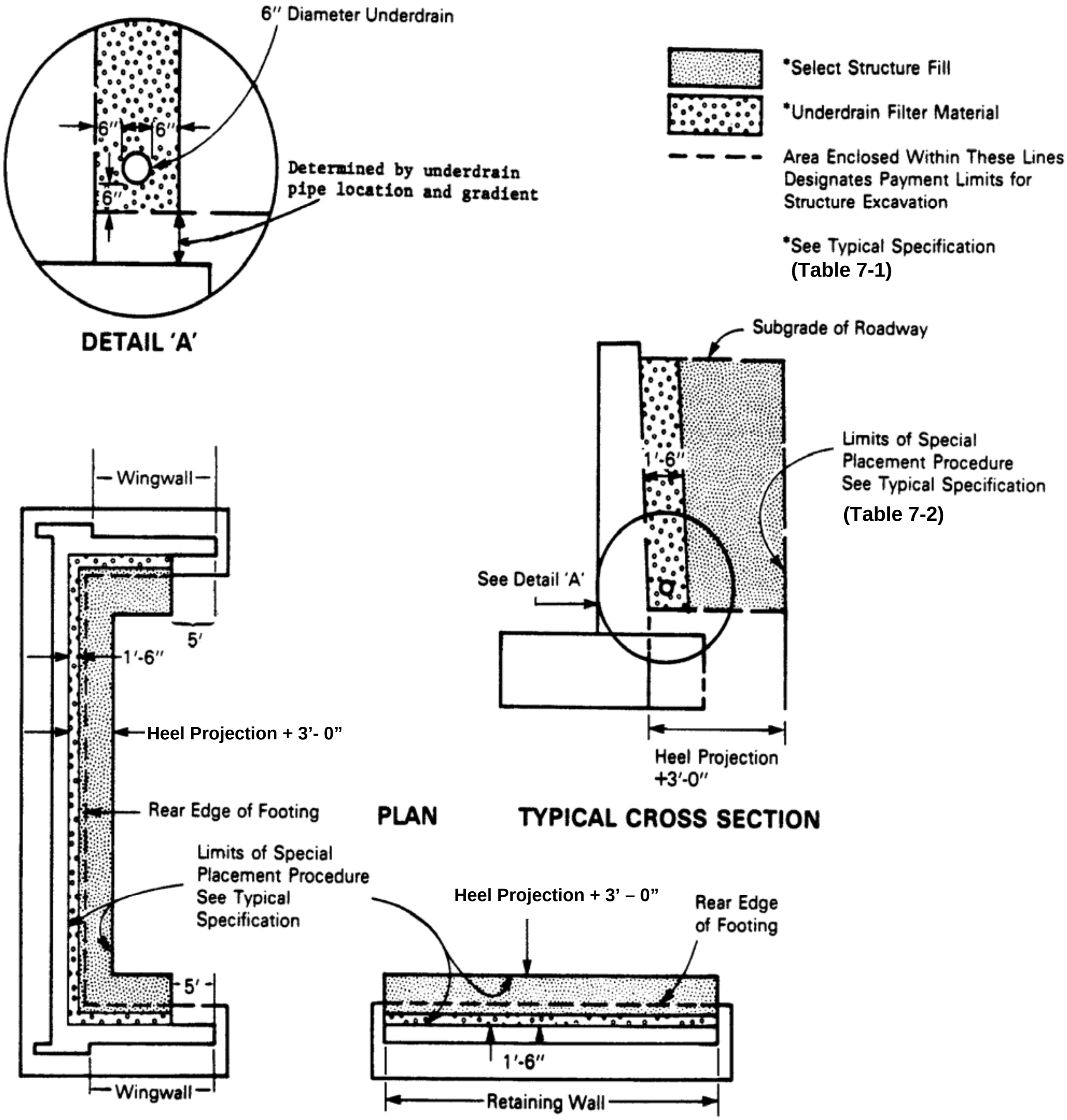


Figure 7-5. Structural backfill placement limits for porous drainage aggregate.
(1 ft = 0.3 m; 1 in = 25.4 mm)

Table 7-1
General considerations for specification of select structural backfill

<table>
<tr><th>Consideration</th><th>Comment</th></tr>
<tr><td>Lift Thickness</td><td>Limit to 6" to 8" (150 mm to 200 mm), so compaction is possible with small equipment.</td></tr>
<tr><td>Topsize (largest particle size)</td><td>Limit to less than ¾ of lift thickness.</td></tr>
<tr><td>Gradation/Percent Fines</td><td>Use well graded soil for ease of compaction. Typical gradation is as follows:
<table>
<tr><th>Sieve Size</th><th>Percent Passing (by weight)</th></tr>
<tr><td>4-in (100 mm)</td><td>100</td></tr>
<tr><td>No. 40 (0.425 mm)</td><td>0 to 70</td></tr>
<tr><td>No. 200 (0.075 mm)</td><td>0 to 15</td></tr>
</table>
The limitation on percent fines (particles smaller than No. 200 sieve) is to prevent piping and allow gravity drainage. For rapid drainage, consideration may be given to limiting the percent fines to 5%.</td></tr>
<tr><td>Plasticity Index</td><td>The plasticity index (PI) should not exceed 10 to control long-term deformation.</td></tr>
<tr><td>Durability</td><td>This consideration attempts to address breakdown of particles and resultant settlement. The material should be substantially free of shale or other soft, poor-durability particles. Where the agency elects to test for this requirement, a material with a magnesium sulfate soundness loss exceeding 30 should be rejected.</td></tr>
<tr><td>T99 Density Control</td><td>Small equipment cannot achieve AASHTO T180 densities. Minimum of 100 percent of standard Proctor maximum density is required.</td></tr>
<tr><td>Compatibility</td><td>Particles should not move into voids of adjacent fill or drain material</td></tr>
</table>

Table 7-2
Suggested gradation for drainage aggregate

Sieve Size	Percent Passing (by weight)
1-in (25.4 mm)	100
½-in (12.7 mm)	30 to 100
No. 3 (6.3 mm)	0 to 30
No. 10 (2.00 mm)	0 to 10
No. 20 (0.85 mm)	0 to 5

As with the select backfill, the soundness of the drainage aggregate should be tested. The drainage aggregate should have a loss not exceeding 20 percent by weight after four (4) cycles of the magnesium sulfate soundness test.

The maximum loose lift thickness for the drainage aggregate should not exceed 6 in (150 mm). Placement and compaction operations should be conducted in a manner so as to insure that the top surface of each lift of the drainage aggregate should not be contaminated by the adjacent backfill materials. Compaction of the drainage aggregate is commonly achieved by two passes of a vibratory compactor approved by the engineer. No compaction control tests are normally required for the drainage aggregate.

7.2.3 Use of Geosynthetics to Control Internal Deformations

In geographic areas where select materials are not available, the use of geosynthetic materials to reinforce the abutment backfill and approach area can reduce the bump at the end of the bridge. Such reinforced fills can be designed by using the principles of Reinforced Soil Slopes (RSS) discussed in Chapter 6 (Slope Stability)

It is suspected that high dynamic loads are routinely induced in the abutment backfill due to vehicle impact loads. Poorly designed or constructed drainage layers or non-durable drainage aggregate can cause either piping of fines or accelerated pavement subsidence due to breakdown of aggregates. As indicated previously, the use of geotextiles or geocomposite drains can be an effective method of minimizing internal embankment deformation and the resulting "bump at the end of the bridge."

7.3 EXTERNAL DEFORMATION IN FOUNDATION SOILS BELOW EMBANKMENTS

Once the issue of internal deformation within fills has been addressed, the designer must concentrate on the evaluation of the deformation of foundation soils and any engineered soils on which the fills will be placed. As explained in Chapter 2, deformations in foundation soils under embankments occur due to the pressure imposed by the embankments. Depending on the type of foundation soils, one or both of the following deformations may occur:

- Immediate (elastic) deformation
- Consolidation (or long-term) deformation

Immediate or elastic deformations occur in all soils regardless of whether they are cohesive or cohesionless. Consolidation deformations typically occur in fine-grained soils that are saturated at the time additional loads are applied. Many and varied procedures exist for computation of these types of deformations. Two methods are presented in this chapter; one each for cohesionless and cohesive soils. However, there is a critical first step that is common to both modes of deformation. This first step involves the estimation of the stress distribution within the foundation soils due to the pressures imposed by the embankment fills. This step is discussed next.

7.3.1 Procedure for Estimating Stress Distribution in Foundation Soils under Fills

The basic steps involved in estimating stresses in native soils under fills are as follows:

1. Develop a soil profile including soil unit weights, SPT results ($N1_{60}$), moisture contents and interpreted consolidation test values.

2. Draw effective overburden pressure (p_o) diagram with depth.

3. Plot total embankment pressure (p_f) on the p_o diagram at ground surface level.

4. Distribute the total embankment pressure with depth by using the appropriate pressure coefficient charts presented in Figure 7-6.

   (Note: The charts in Figure 7-6 are limited to only two locations, Section B-B and Section C-C, and assume that the end and side slopes have the same grades. Programs such as FoSSA (2003) may be used for case of unequal end and side slopes, or if pressure coefficients at locations other than along Section B-B or C-C are desired.)

The principles to remember are: (1) stresses induced in the soil from an embankment load are distributed with depth in proportion to the embankment width, and (2) the additional stresses in the soil decrease with depth.

Following is a step-by-step procedure to use the chart in Figure 7-6. A worked example is presented afterwards to illustrate the use of the chart numerically:

Step 1. Determine the distance b_f from the centerline of the approach embankment to the midpoint of side slope. Multiply the numerical value of "b_f" by the appropriate values shown to the right of the chart to develop the depths at which the distributed pressures will be computed, e.g., $0.2b_f$, $0.4b_f$, etc.

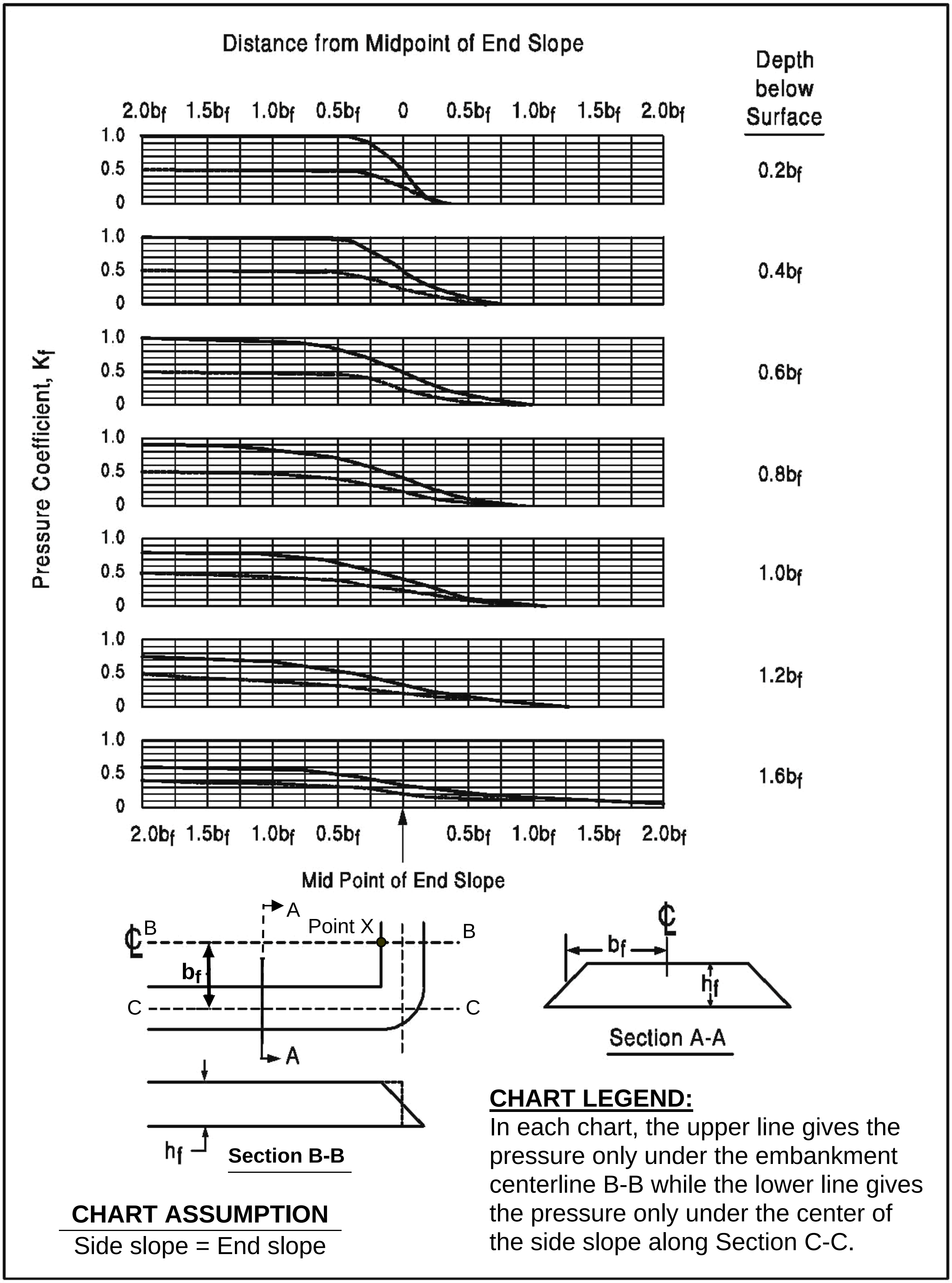


Figure 7-6. Pressure coefficients beneath the end of a fill (after NYSDOT).

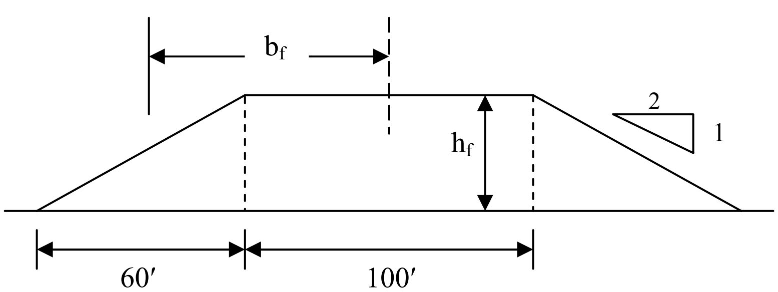
Step 2. Select the point X on the approach embankment where the vertical stress prediction is desired, normally at the intersection of the centerline of the embankment and the abutment. In this case the side slope is called the end slope. Measure the distance from X to the midpoint of the end slope. Return to the chart and scale that distance as a multiple of b_f on the horizontal axis from the appropriate side of the midpoint centerline line of the end slope.

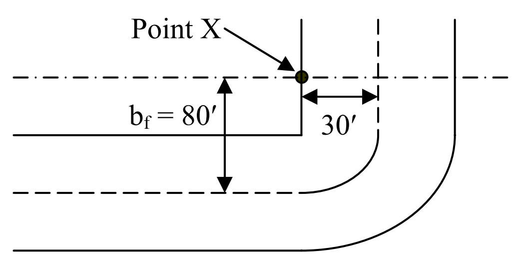
Step 3. Read vertically up or down from the plotted distance on the horizontal axes to the various curves corresponding to depth below surface. The "K_f" value on the left vertical axis should be read and recorded on a computation sheet with the corresponding depth. Note that the upper line gives the pressure under the embankment centerline (Section B-B) while the lower line gives the pressure under the mid of the side slope (Section C-C).

Step 4. Multiply each "K_f" value by the value of total embankment pressure ($\gamma_f h_f$) to determine the amount of the pressure increment (Δp) transmitted to each depth, where γ_f is the unit weight of the embankment fill soil and h_f is height of the embankment fill.

The application of this step-by-step procedure and the charts shown on Figure 7-6 to a typical embankment problem is illustrated by the following worked example problem.

Example 7-1: The geometry of a fill slope is as follows:
Fill height h_f = 30 ft; Fill unit weight γ_f = 100 pcf
End and side slopes (2H:1V); Embankment top width = 100 ft





Find: The stress increase (Δp) under the proposed abutment centroid (Point X) at a depth of 0.8 b_f below the base of the fill.

Solution:

Figure 7-6 will be used to determine the stress increase. To use the chart first compute the following quantities:

- Distance from midpoint of end slope to Point X = 30 ft.
- Distance from centerline to mid point of side slope b_f = (100 ft/2) + (60 ft/2) = 80 ft.

Enter stress distribution chart for a depth of $0.8b_f$ = (0.8)(80 ft) = 64 ft and a distance measured from the ***midpoint of the end slope to Point X expressed as a multiple of b_f*** = (30 ft/80 ft) b_f = 0.38 b_f. Enter the plot with this value to the left of the value of zero on the abscissa, i.e., upslope from the midpoint on the end slope.

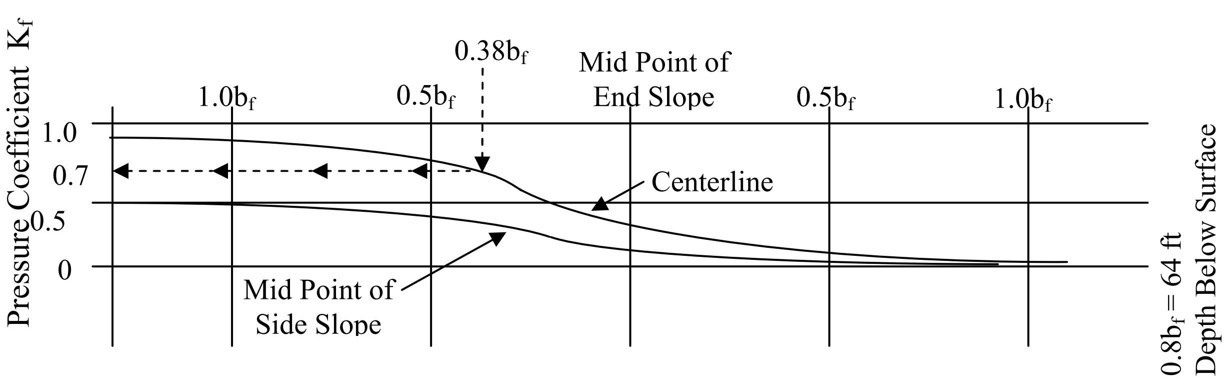
In Figure 7-6 read K_f = 0.7 from the chart for $0.8b_f$. Therefore, at a depth of 64 ft below the embankment at Point X

$$\Delta p = K_f \gamma_f h_f$$

$$\Delta p = (0.7)\ (100\ \text{pcf})\ (30\ \text{ft}) = 2{,}100\ \text{psf}$$

Repeat the above steps for distances to other points along the centerline of the embankment expressed as a multiple of b_f and measured (+ and -) from the midpoint of end slope to develop the horizontal distribution of vertical stress increases due to the embankment at a depth of 64 ft below and beyond the base of the end slope along the embankment centerline.

Horizontal Distribution of Vertical Stress Increases Below and Beyond the End Slope at a Depth of 64-ft Below the Embankment



7.4 COMPUTATION OF IMMEDIATE SETTELEMENT

All geomaterials, whether cohesionless or cohesive, will experience settlements immediately after application of loads. Whether or not the settlements will continue with time after the application of the loads will be a function of how quickly the water can drain from the voids as explained in Chapter 2. Long-term consolidation-type settlements are generally not experienced in cohesionless soils where pore water can drain quickly or in dry or slightly moist cohesive soils where significant amounts of pore water are not present. Therefore, embankment settlements caused by consolidation of cohesionless or dry cohesive soil deposits are frequently ignored as they are much smaller compared to immediate settlements in such soils. Consolidation type settlement for saturated cohesive soils is discussed in Section 7.5.

Many methods have been published in the geotechnical literature for the computation of immediate settlements in soils or rocks. These methods vary from the use of rules of thumb based on experience to the use of complex nonlinear elasto-plastic constitutive models. All methods are based on some form of estimate of soil compressibility. In the geotechnical literature, soil compressibility is expressed using several different terms such as "bearing capacity index," "compression index," "elastic modulus," "constrained modulus," etc.

For computing external embankment settlements, the method by Hough (1959) as modified by AASHTO (2004 with 2006 Interims) can be used since it is simple and provides a first-order conservative estimate of immediate settlements. The original Hough method (Hough, 1959) was based on uncorrected SPT N-values and included recommendations for cohesionless as well as cohesive soils such as sandy clay and remolded clay. AASHTO modified the Hough (1959) method for use with $N1_{60}$ values and eliminated the recommendations for sandy clay and remolded clay. Since the method presented here is AASHTO's version of the Hough method, it will be referred to as the "Modified Hough" method.

Even after the modifications, the settlements estimated by Modified Hough method are usually overestimated by a factor of 2 or more based on the data in FHWA (1987). While such conservative estimates may be acceptable from the viewpoint of the earthwork quantities (see discussion regarding compaction factor in Section 7.4.1.1), they may be excessive with respect to the behavior of the structures founded within, under or near the embankment. In cases where structures are affected by embankment settlement, more refined estimates of the immediate settlements are warranted. **For more refined estimates of immediate settlements it is recommended that the designer use either the modified method of Schmertmann, *et al.* (1978), which takes into account the strain distribution**

with depth, or the D'Appolonia (1968, 1970) method, which takes into account the effect of preconsolidation. Both methods provide equally suitable results. Schmertmann's modified method is presented in Chapter 8 (Shallow Foundations).

7.4.1 Modified Hough Method for Estimating Immediate Settlements of Embankments

The following steps are used in Modified Hough method to estimate immediate settlement:

Step 1. Determine the bearing capacity index (C′) by entering Figure 7-7 with $N1_{60}$ value and the visual description of the soil.

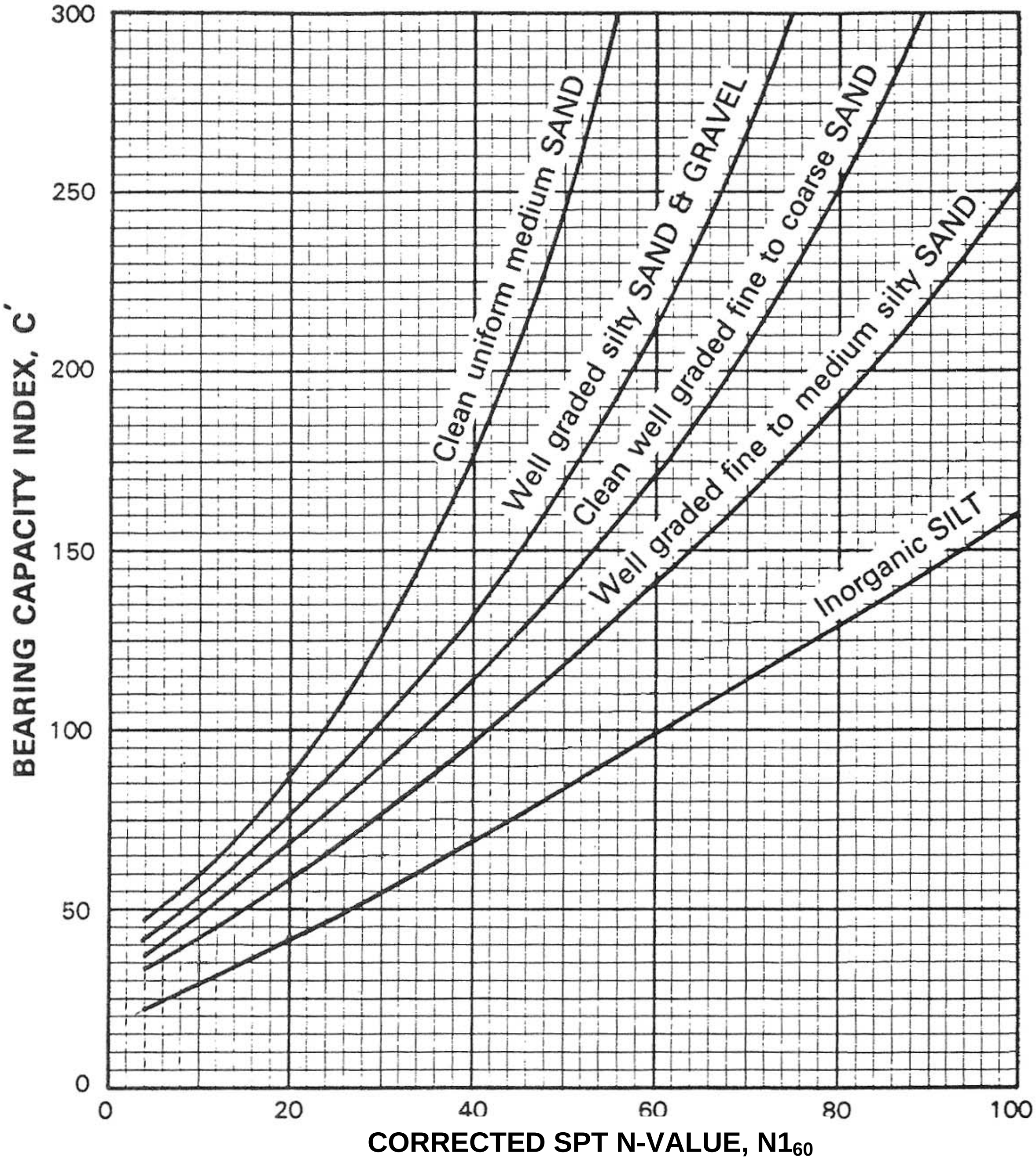
Step 2. Compute settlement by using the following equation. Subdivide the total thickness of the layer impacted by the applied loads into 10 ft ± (3 m ±) increments and sum the incremental solutions:

$$\Delta H = H\left(\frac{1}{C'}\right)\log_{10}\frac{p_o + \Delta p}{p_o} \qquad 7\text{-}1$$

where:
- ΔH = settlement of subdivided layer (ft)
- H = thickness of subdivided soil layer considered (ft)
- C' = bearing capacity index (Figure 7-7)
- p_o = existing effective overburden pressure (psf) at center of the subdivided layer being considered. For shallow surface deposits, a minimum value of 200 psf should be used to prevent unrealistic settlement predictions.
- Δp = distributed embankment pressure (psf) at center of the subdivided layer being considered

Note that the term $p_o + \Delta p$ represents the final pressure applied to the foundation subsoil, p_f.

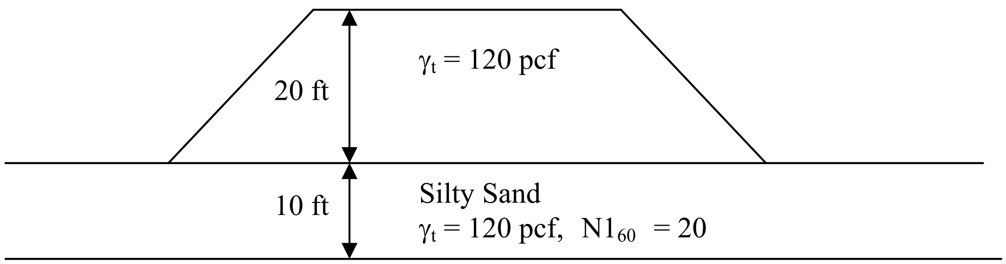
A key point is that the logarithm term in Equation 7-1 incorporates the fundamental feature of dissipation of applied stress with depth. The use of Modified Hough method is illustrated numerically in Example 7-2.



(Note: The "Inorganic SILT" curve should generally not be applied to soils that exhibit plasticity because N-values in such soils are unreliable)

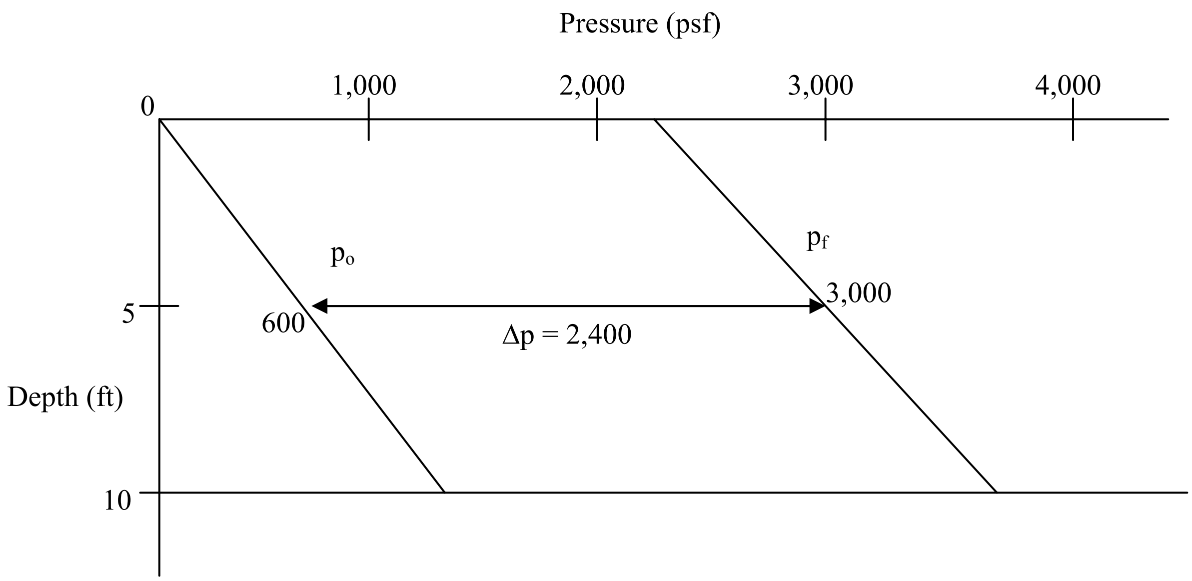
Figure 7-7. Bearing capacity index (C') values used in Modified Hough method for computing immediate settlements of embankments (AASHTO, 2004 with 2006 Interims; modified after Hough, 1959).

Example 7-2: For the geometry shown in the following figure, determine the settlement at the center of a wide embankment placed on a silty sand layer by using Modified Hough method and the p_o diagram.



Solution:

The original overburden pressure at the center of the 10 ft thick silty sand deposit can be computed as p_o = (10 ft/2) (120 pcf) = 600 psf. Since, the embankment is "wide" the stress does not practically dissipate with depth. Therefore, increase in the stress at this depth due to the 20 ft high wide embankment can be computed as Δp = (20 ft) (120 pcf) = 2,400 psf. The p_o diagram based on these values of p_o and Δp is shown below.



From Figure 7-7, find C′ for "silty sand." Using $N1_{60}$ = 20 and the "silty sand" curve, C′ ≈ 58. Find immediate settlement using Equation 7-1 as follows:

$$\Delta H = H\left(\frac{1}{C'}\right)\log_{10}\frac{p_o + \Delta p}{p_o}$$

$$\Delta H = 10 \text{ ft}\left(\frac{1}{58}\right)\log_{10}\frac{600 \text{ psf} + 2{,}400 \text{ psf}}{600 \text{ psf}} = 0.12 \text{ ft} = 1.44 \text{ in}$$

7.4.1.1 Comments on the Computed Settlement of Embankments

The implication of the amount of embankment settlement is that when the embankment is completed, additional fill will be required to bring the top of the embankment to the design grade. For example, a 1 in (25 mm) settlement on a 60-ft (18 m) wide, 1-mile (5,280 ft or 1,610 m) long embankment will result in a need for approximately 1,000 yd^3 (~750 m^3) of additional fill. Some state agencies refer to such settlement estimates as the "compaction factor" and note it in the contract plans so that the contractor can make appropriate allowances in the bid price to accommodate the additional embankment fill material needed to achieve the required design grades. It is in this regard the conservative estimate of the settlement resulting from the Modified Hough method may be acceptable and may even be preferable to prevent construction change orders.

7.5 COMPUTATION OF CONSOLIDATION (LONG-TERM) SETTLEMENTS

Unless the geomaterial is friable, consolidation settlements in fine-grained saturated soils occur over a period of time as a function of the permeability of the soils. This concept was introduced in discussed in Chapter 2 by using the spring-piston analogy. The features of the laboratory consolidation test were discussed in Chapter 5. In this chapter the data obtained from the consolidation test are used to demonstrate the computation of long-term settlements due to the consolidation phenomena, i.e., primary consolidation and secondary compression.

Theoretically, a necessary condition for consolidation settlement is that the soil must be saturated, i.e., degree of saturation, S = 100%. While the laboratory test for moisture content of a soil is inexpensive and relatively straightforward to perform and generally yields reliable, reproducible results, there are a number of parameters in consolidation analysis that cannot be determined with confidence as indicated by the data in Table 5-25. Therefore, depending on the magnitude and configuration of the load with respect to the size and moisture content of the compressible soil layer, it is possible that consolidation settlements may occur in soils that are judged to be "nearly saturated" but not "fully saturated." This is because such nearly saturated soils may approach full saturation after application of a load of sufficient magnitude to cause the pore spaces filled with air to compress (immediate settlement) to the extent that the degree of saturation is virtually 100%. Therefore, the geotechnical specialist should carefully evaluate the in-situ degree of saturation with respect to the degree of saturation of the soil sample at the beginning and end of the consolidation test. The geotechnical specialist should also carefully evaluate the reliability of other parameters determined during the performance of the consolidation test to make an informed judgment regarding the potential for consolidation settlements to occur. Unnecessarily conservative assumptions regarding the magnitude and time rate of consolidation settlements may lead to recommendations for deep foundations or for unnecessary implementation of costly ground improvement measures.

Settlement resulting from primary consolidation may take months or even years to be completed. Furthermore, because soil properties may vary beneath the location of loading, the duration of the primary consolidation and the amount of settlement may also vary with the location of the applied load, resulting in differential settlement. If such settlements are not within tolerable limits the geotechnical feature as well as a structure founded on or in it may be damaged. In the case of embankments, differential settlements that occur along the longitudinal axis of the embankment because of changes in thickness and/or consolidation properties of underlying clays can cause transverse cracking on the surface of the embankment where pavement structures are usually constructed.

When the areal extent of the applied load is wide compared to the thickness of the compressible layer beneath it, a large portion of the soil will consolidate vertically (one-dimensionally) with very little lateral displacement because of the constraining forces exerted by the neighboring soil elements. However, when the areal extent of the applied load is smaller than the thickness of the compressible layer or when there is a finite soft layer at a certain depth below the loaded area, significant lateral stresses and associated deformations can occur as shown earlier in Figure 2-16 in Chapter 2. Back-to-back retaining walls and a narrow embankment for an approach ramp on soft soils are examples of this condition. Due to the potential for significant lateral stresses and associated lateral deformations, the geotechnical specialist should carefully evaluate the loading geometry with respect to subsurface conditions and ascertain whether the problem is 1-D or 3-D. This type of evaluation is important because 3-D deformations can affect a number of facilities such as buried utilities, bridge foundations, and the stability of embankment slopes.

The determination of the vertical component of 3-D consolidation deformation is commonly based on the one-dimensional consolidation test (ASTM D 2435). Typically, the results of the one-dimensional consolidation test are expressed in an e-log p plot which is the so-called "consolidation curve." As indicated in Chapter 5, settlement due to consolidation can be estimated from the slope of the consolidation curve. This procedure is generally used in practice despite the fact that not all of the points beneath the embankment undergo one-dimensional consolidation. However, before the laboratory test results are used, it is very important to correct the consolidation curves for the effects of sampling. Thus, before proceeding with the discussion of computing consolidation settlements, the correction of the laboratory consolidation curves is discussed.

7.5.1 Correction of Laboratory One-Dimensional Consolidation Curves

As indicated in Chapter 3, the process of sampling soils will cause some disturbance no matter how carefully the samples are taken. This sampling disturbance will affect virtually all measured physical properties of the soil. The sampling disturbance will usually cause the "break" in the laboratory consolidation curve to occur at a lower maximum past vertical pressure (p_c) than would be measured for a truly undisturbed specimen. The effect of disturbance from the sampling procedure is illustrated in Figure 7-8 where, for the sake of comparison, the vertical strain rather than void ratio (e) is plotted versus the logarithm of the vertical effective stress.

Figure 7-8 shows three consolidation curves for a red-colored plastic clay from Fond du Lac, Wisconsin. Samples were taken alternately with 3 in (75 mm) and 2 in (50 mm) thin walled samplers. The 3 in (50 mm) sampler apparently caused less disturbance than the 2 in (50 mm) sampler. The curve for the remolded sample is the flattest curve without a well defined break between reloading and virgin compression.

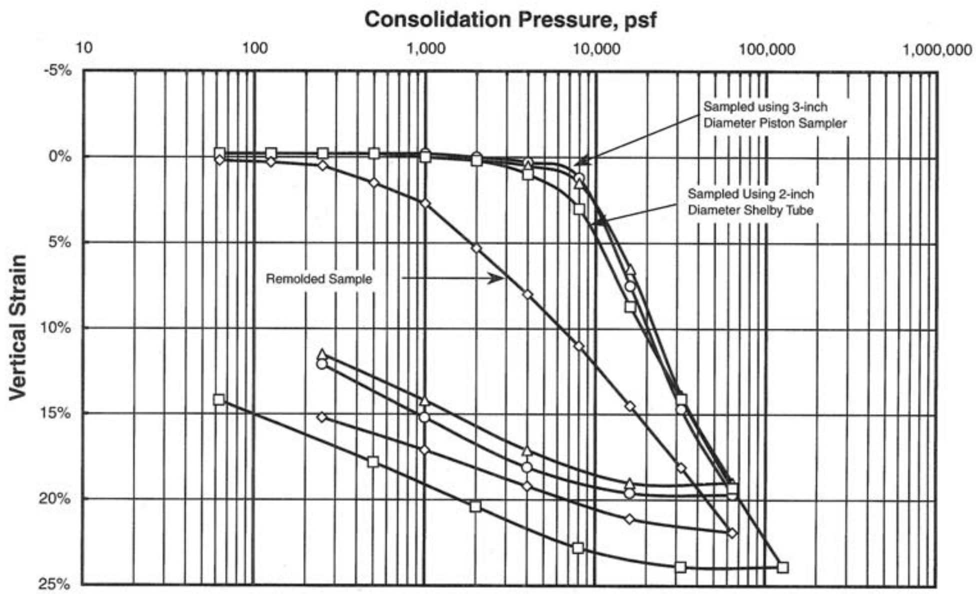


Figure 7-8. Effect of sample disturbance on the shape of the one-dimensional consolidation curve (Reese, *et al.*, 2006).

Even for good quality samples, it is still necessary to "correct" the e-log p curve since no sampling technique is perfect. There are several methods available to correct the consolidation curve. The laboratory curve can be corrected according to Figures 7-9a and 7-9b for normally consolidated and overconsolidated soils, respectively. Table 7-3 presents the reconstruction procedures.

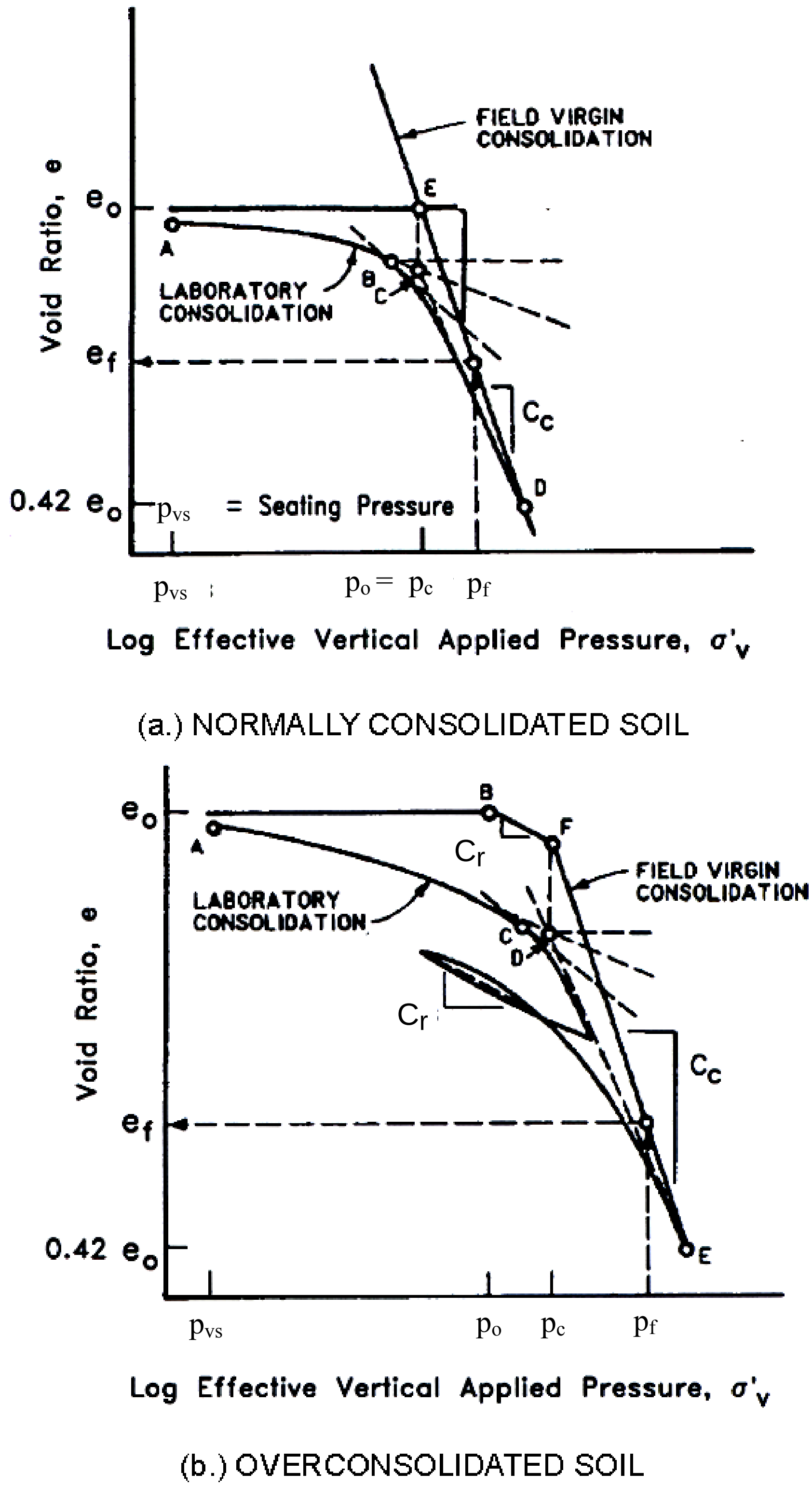


Figure 7-9. Construction of field virgin consolidation relationships (adapted from USACE, 1994).

Table 7-3

Reconstruction of virgin field consolidation curve (modified from USACE, 1994).

Step	Description
a. Normally Consolidated Soil (Figure 7-9a)	
1	By eye choose the point B at the point of minimum radius of curvature (maximum curvature) of the laboratory consolidation curve.
2	Plot point C by the Casagrande construction procedure: (1) Draw a horizontal line through point B; (2) Draw a line tangent to the consolidation curve at point B; (3) Draw the bisector between the horizontal and tangent lines; and (4) Draw a line tangent to the "virgin" portion of the laboratory consolidation curve. Point C is the intersection of the tangent to the virgin portion of the laboratory curve with the bisector. Point C indicates the maximum preconsolidation (past) pressure p_c.
3	Plot point E at the intersection of a horizontal line through e_o and the vertical extension of point C, that corresponds to p_c as found from Step 2. The value of e_o is given as the initial void ratio prior to testing in the consolidometer.
4	Plot point D on the laboratory virgin consolidation curve at a void ratio $e = 0.42e_o$. Extend the laboratory virgin consolidation curve to that void ratio if necessary. On the basis of many laboratory tests, Schmertmann (1955) found that the laboratory curve for various degrees of disturbance intersects the field virgin curve at a value of $e= 0.42e_o$.
5	The field virgin consolidation curve is the straight line determined by points E and D.
6	The field compression index, C_c, is the slope of the line ED.
b. Overconsolidated Soil (Figure 7-9b)	
1	Plot point B at the intersection of a horizontal line through the given e_o and the vertical line representing the initial estimated in situ effective overburden pressure p_o.
2	Draw a line through point B parallel to the mean slope, C_r, of the rebound laboratory curve.
3	Plot point D by using Step 2 in Table 7-3a for normally consolidated soil.
4	Plot point F by extending a vertical line through point D up through the intersection of the line of slope C_r extending through B.
5	Plot point E on the laboratory virgin consolidation curve at a void ratio $e = 0.42e_o$.
6	The field virgin consolidation curve is the straight line through points F and E. The field reload curve is the straight line between points B and F.
7	The field compression index, C_c, is the slope of the line FE.

7.5.2 Computation of Primary Consolidation Settlements

Depending upon the magnitude of the existing effective stress relative to the maximum past effective stress at a given depth, in-situ soils can be considered normally consolidated, overconsolidated (preconsolidated), or underconsolidated. The behavior of in-situ soils to additional loads is highly dependent upon the stress history. The overconsolidation ratio, OCR, which is a measure of the degree of overconsolidation in a soil is defined as p_c/p_o. The value of OCR provides a basis for determining the effective stress history of the clay at the time of the proposed loading as follows:

- OCR = 1 - the clay is considered to be "normally consolidated" under the existing load, i.e., the clay has fully consolidated under the existing load ($p_c = p_o$).
- OCR > 1 - the clay is considered to be "overconsolidated" under the existing load, i.e., the clay has consolidated under a load greater than the load that currently exists ($p_c > p_o$).
- OCR < 1 – the clay is considered to be "underconsolidated" under the existing load, i.e., consolidation under the existing load is still occurring and will continue to occur under that load until primary consolidation is complete, even if no additional load is applied ($p_c < p_o$).

The manner in which primary settlements are computed for each of these three conditions varies as will be discussed in the following sections.

7.5.2.1 Normally Consolidated Soils

The settlement of a geotechnical feature or a structure resting on n layers of normally consolidated soils ($p_c = p_o$) can be computed from Figure 7-10a where n is the number of layers into which the consolidating layer is divided:

$$S_c = \sum_{i}^{n} \frac{C_c}{1+e_o} H_o \log_{10}\left(\frac{p_f}{p_o}\right) \qquad 7\text{-}2$$

where:

C_c = compression index

e_o = initial void ratio

H_o = layer thickness

p_o = initial effective vertical stress at the center of layer n

p_f = $p_o + \Delta p$ = final effective vertical stress at the center of layer n.

The final effective vertical stress is computed by adding the stress change due to the applied load to the initial vertical effective stress. The total settlement will be the sum of the compressions of the n layers of soil.

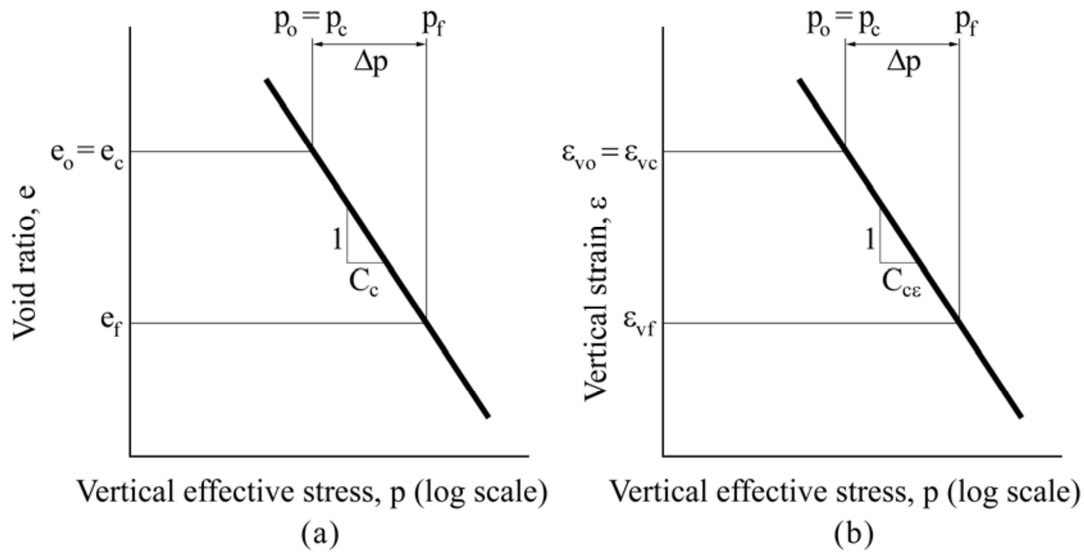


Figure 7-10. Typical consolidation curve for normally consolidated soil, (a) Void ratio versus vertical effective stress and (b) Vertical strain versus vertical effective stress.

Normally the slope of the virgin portion of the e-log p curve is determined from the corrected one-dimensional consolidation curve measured on specimens taken from each relevant soil in the stratigraphic profile. The procedure for determining the corrected curve is presented in Table 7-6a. Common correlations for estimating C_c were presented in Section 5.4.6.1 of Chapter 5 and can be used to check laboratory results.

Sometimes the consolidation data is presented in terms of vertical strain (ε_v) instead of void ratio. In this case the slope of the virgin portion of the modified consolidation curve is called the modified compression index and is denoted as $C_{c\varepsilon}$ as shown in Figure 7-10b. Settlement is computed by using Equation 7-3 for normally consolidated soils where all of the other terms are defined as for Equation 7-2.

$$S_c = \sum_{1}^{n} H_o C_{c\varepsilon} \log_{10}\left(\frac{p_f}{p_o}\right) \qquad 7\text{-}3$$

By comparing Equations 7-2 and 7-3, it can be seen that $C_{c\varepsilon} = C_c / (1+e_o)$

7.5.2.2 Overconsolidated (Preconsolidated) Soils

If the water content of a clay layer below the water table is closer to the plastic limit than the liquid limit, the soil is likely overconsolidated, i.e., OCR >1. This means that in the past the clay was subjected to a greater stress than now exists. Preconsolidation could have occurred because of any number of factors including but not limited to the weight of glaciers which is especially prevalent in the northern tier of states and in the northeast, the weight of a natural soil deposit that has since eroded away, the weight of a previously placed fill that has since been removed, loads due to structures that have since been demolished, desiccation, etc.

As a result of preconsolidation, the field state of stress will reside on the initially flat portion of the e-log p curve. Figures 7-11a and 7-11b illustrate the case where a load increment, Δp, is added so that the final stress, p_f, is greater than the maximum past effective stress, p_c. For this condition, the settlements for the case of n layers of overconsolidated soils will be computed from Equation 7-4 or Equation 7-5 that correspond to Figure 7-11a and 7-11b, respectively.

$$S = \sum_{1}^{n} \frac{H_o}{1+e_o}\left(C_r \log_{10}\frac{p_c}{p_o} + C_c \log_{10}\frac{p_f}{p_c} \right) \qquad 7\text{-}4$$

$$S = \sum_{1}^{n} H_o \left(C_{r\varepsilon} \log_{10}\frac{p_c}{p_o} + C_{c\varepsilon} \log_{10}\frac{p_f}{p_c} \right) \qquad 7\text{-}5$$

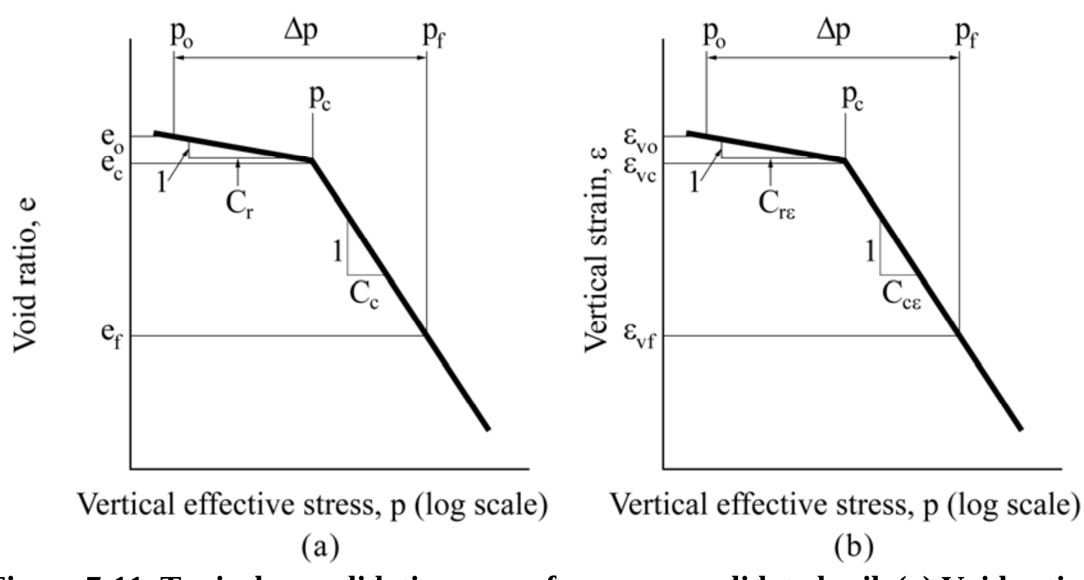


Figure 7-11. Typical consolidation curve for overconsolidated soil, (a) Void ratio versus vertical effective stress and (b) Vertical strain versus vertical effective stress.

The total settlement is computed by summing the settlements computed from each subdivided compressible layer within the zone of influence (Z_I). The assumption is made that the initial and final stress calculated at the center of each sublayer is representative of the average stress for the sublayer, and the material properties are reasonably constant within the sublayer. The sublayers are typically 5 ft (1.5 m) to 10 ft (3 m) thick in highway applications. In cases where the various stratigraphic layers represent combinations of both normally and overconsolidated soils, the settlement is computed by using the appropriate combinations of Equations 7-2 through 7-5.

7.5.2.3 Underconsolidated Soils

Underconsolidation is the term used to describe the effective stress state of a soil that has not fully consolidated under an existing load, i.e., OCR < 1. Consolidation settlement due to the existing load will continue to occur under that load until primary consolidation is complete, even if no additional load is applied. This condition is shown in Figure 7-12 by Δp_o. Therefore, any additional load increment, Δp, would have to be added to p_o. Consequently, if the soil is not recognized as being underconsolidated, the actual total primary settlement due to $\Delta p_o + \Delta p$ will be greater than the primary settlement computed for an additional load Δp only, i.e., the settlement may be under-predicted. As a result of under-consolidation, the field state of stress will reside entirely on the virgin portion of the consolidation curve as shown in Figure 7-12. The settlements for the case of n layers of under-consolidated soils are computed by Equation 7-6 or Equation 7-7 that correspond to Figure 7-12a and 7-12b, respectively.

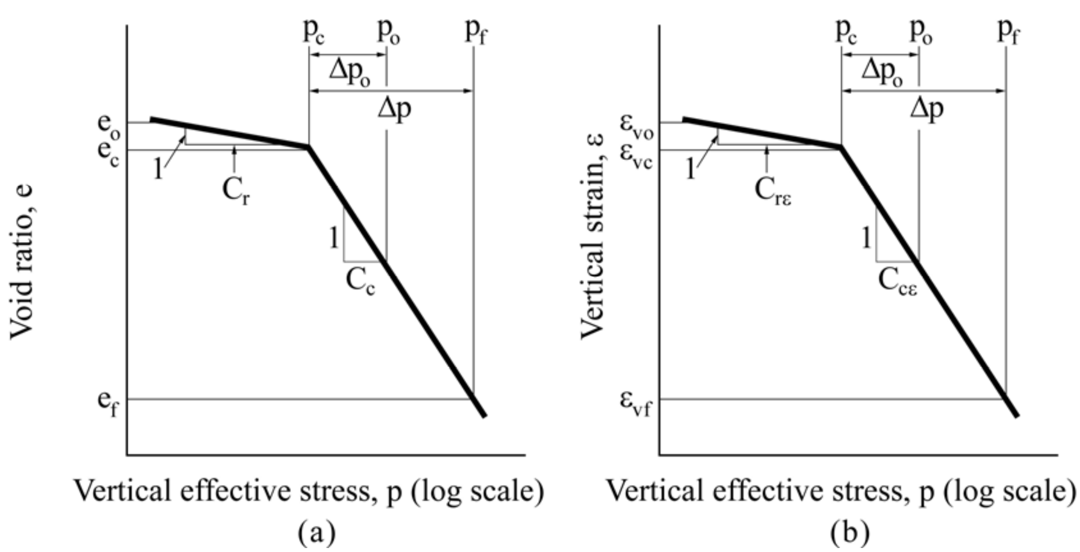


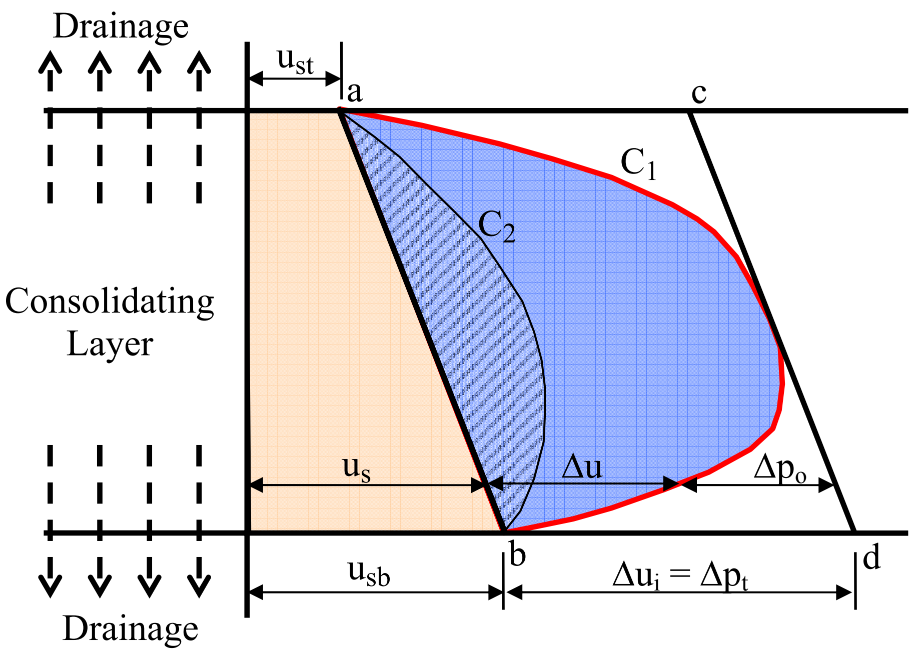
Figure 7-12. Typical consolidation curve for under-consolidated soil – (a) Void ratio versus vertical effective stress and (b) Vertical strain versus vertical effective stress.

$$S = \sum_{1}^{n} \frac{H_o}{1+e_o}\left(C_c \log_{10}\frac{p_o}{p_c} + C_c \log_{10}\frac{p_f}{p_o} \right) \qquad 7\text{-}6$$

$$S = \sum_{1}^{n} H_o \left(C_{c\varepsilon} \log_{10}\frac{p_o}{p_c} + C_{c\varepsilon} \log_{10}\frac{p_f}{p_o} \right) \qquad 7\text{-}7$$

7.5.3 Consolidation Rates (Time Rate of Consolidation Settlement)

The rate of consolidation should be considered for the design of geotechnical features and structures on compressible clay. For example, a geotechnical feature such as an embankment will settle relative to a bridge foundation supported on piles, creating an undesirable "bump at the end of the bridge." Hence, time rate of consolidation, as well as differential settlements between the bridge and embankment, is important. The concept of time rate of consolidation is explained with respect to Figure 7-13.



Definitions: u_{st} = hydrostatic pore water pressure at top of layer
u_{sb} = hydrostatic pore water pressure at bottom of layer
u_s = hydrostatic pore water pressure at any depth
Δu_i = initial excess pore water pressure
Δu = excess pore water pressure at any depth after time t
u_t = $u_s + \Delta u$ = total pore water pressure at any depth after time t

Figure 7-13. Diagram illustrating consolidation of a layer of clay between two pervious layers (modified after Terzaghi, *et al.* 1996).

- The initial hydrostatic pore water pressure distribution, u_s, is assumed to be linear in a layer of saturated clay. Line a-b in Figure 7-13 shows the initial hydrostatic pore water pressure distribution through a clay layer at a certain depth below the ground water elevation where u_{st} is the pore water pressure at the top of the clay layer and u_{sb} is the pore water pressure at the bottom of the clay layer. Experimental measurement of pore pressures in saturated clays subjected to one-dimensional loading indicate that when a load is applied the pore water pressure will instantaneously increase an amount equal to the total vertical stress increment, Δp_t, uniformly throughout the entire thickness of the consolidating layer as shown by a-c-d-b in Figure 7-13. The initial increase in the pore water pressure, Δu_i, above the static value is called the initial excess pore water pressure and it is equal to Δp_t. The total initial pore water pressure which is the sum of the hydrostatic pressure and the initial excess pore water pressures is shown as line c-d in Figure 7-13.

- With time, water will drain out of the consolidating layer to relieve the excess pore water pressure and the applied total vertical stress increment, Δp_t, will be slowly transferred to the soil particles, i.e., at any given time after application of the load, the initial excess pore water pressure will decrease at all depths to an excess pore water pressure having a value less than of Δu_i. The pattern of the excess pore water pressure at any given time is not parallel to line c-d, but is curvilinear similar to curve C_1 in Figure 7-13. Curves such as C_1 and C_2 are known as *iscochrones* because they are lines of equal time. The difference between the line a-b and curve C_1, for example, represents the excess pore water pressure, Δu, at any point within the consolidating layer at any time after application of the vertical load stress increment, Δp_t.

- If the clay layer is confined between two sand layers that are more permeable, the initial excess pore water pressure will drop immediately to zero at the drainage boundaries as shown in Figure 7-13 and the total vertical stress increment Δp_t and will be equal to the effective vertical stress increment, Δp_o. The rate of this transfer with depth depends upon the boundary drainage conditions. With time, the vertical distribution of excess pore water pressure within the consolidating layer will evolve from the initial distribution (a-c-d-b), to the C_1 distribution, to the C_2 distribution, and finally to the initial distribution of the hydrostatic pressure represented by line a-b.

- At any depth, the difference between a pore water pressure isochrone, such as C_1, and the initial excess pore water pressure c-d is equal to the effective vertical stress increment, Δp_o, i.e., the amount of Δp_t that has been transferred to the soil structure. Since the isochrone C_1 develops after a certain period of time, the difference between C_1 and c-d also represents the distribution of the effective stress increments with depth at a given time after application of load.

- Note that the distribution shown in Figure 7-13 pertains only to the specific boundary drainage condition where a more permeable material exists above and below the consolidating clay layer. In this case the clay layer is considered to be "doubly drained" with the longest distance to a drainage boundary being half the layer thickness. If the clay layer is underlain by a less permeably material (e.g., rock), drainage will occur in only one direction and the isochrones at a given time will be different from those shown for double drainage in Figure 7-13. In this case the clay layer is considered to be "singly drained" with the longest distance to a drainage boundary being the entire layer thickness. During the consolidation process the principle of effective stress will be in operation at every depth, i.e., $\Delta p_t = \Delta u + \Delta p_o$ and settlement will be occurring due to the effective stress increment Δp_o. The drainage boundary condition will affect the time it takes for settlement to occur, but it has no effect on the magnitude of settlement, which is determined by use of the equations presented previously in which settlement is a function of Δp_o only.

7.5.3.1 Percent Consolidation

As indicated previously, immediately after application of load, Δu, will drop to zero at the drainage boundaries because the water will drain immediately into the more pervious layers. Since the excess pore water pressure is zero at the drainage boundaries, the soil there has undergone 100% consolidation. However, at interior points, the pore water pressure dissipates more slowly with time depending on the permeability of the compressible soil. At any time after application of a load, the actual degree or percentage of consolidation at a given depth is defined as $(\Delta u_i - \Delta u)/\Delta u_i$, where Δu is the excess pore water pressure at that depth at that time and Δu_i = the initial excess pore water pressure which, as indicated previously, equals the total stress increment Δp_t. Thus, where $\Delta u_i = \Delta u$ (i.e., at the instant of loading), the percent consolidation is zero. When $\Delta u = 0$ (i.e., at the end of consolidation), the percent consolidation is 100. This relationship is valid at any depth within the consolidating layer at any time from the instant of loading to the completion of primary consolidation.

While plots of the type shown in Figure 7-13 give an indication of the pore pressure variation within the consolidating layer at any time and are useful to explain the theory of consolidation, from a practical viewpoint it is usually more beneficial to obtain the *average* degree or percent of consolidation, U, within the entire layer to indicate when the entire clay layer has undergone a certain average amount of consolidation of say 10, 50, or 80 percent. With reference to Figure 7-13, the average degree of consolidation at any time is defined as the difference between the area under the initial excess pore water pressure curve (a-c-d-b) and the area under the isochrone at that time, e.g., the cross hatched area under isochrone C_2 divided by the area under the initial excess pore water pressure curve (a-c-d-b). The result is expressed as a percentage. Therefore, at the instant Δp_t is applied the area under the isochrone is exactly equal to the area (a-c-d-b) as indicated above and the average percent consolidation (U) equals zero. At the end of primary consolidation all excess pore water pressures have dissipated and the area under the isochrone is zero. Thus, the average percent consolidation (U) equals 100. Since, according to the principle of effective stress, $\Delta p_t = \Delta u + \Delta p_o$, the amount of settlement at any time after the application of load is directly related to the amount of consolidation that has taken place up to that time. As a practicality the average degree of consolidation at any time, t, can be defined as the ratio of the settlement at that time, S_t, to the settlement at the end of primary consolidation, $S_{ultimate}$, when excess pore water pressures are zero throughout the consolidating layer, i.e., $U = S_t/S_{ultimate}$. This relationship is used to develop a so-called "settlement-time curve" as will be discussed later.

Table 7-4 shows the average degree of consolidation (U) corresponding to a normalized time expressed in terms of a time factor, T_v, where:

$$T_v = \frac{c_v t}{H_d^2} \quad , \quad \text{which can be written as} \quad t = \frac{T_v H_d^2}{c_v} \qquad \text{7-8}$$

where: c_v = coefficient of consolidation (ft^2/day) (m^2/day)
H_d = the longest distance to a drainage boundary (ft) (m)
t = time (day).

Any consistent set of units can be used in Equation 7-8 since T_v is dimensionless. As indicated previously, the longest drainage distance of a soil layer confined by more permeable layers on both ends is equal to one-half of the layer thickness. When confined by a more permeable layer on one side and an impermeable boundary on the other side, the longest drainage distance is equal to the layer thickness. The value of the dimensionless time factor T_v may be determined from Table 7-4 for any average degree of consolidation. U. The actual time, t, it takes for this percent of consolidation to occur is a function of the boundary drainage conditions, i.e., the longest distance to a drainage boundary, as indicated by Equation 7-8. By using the normalized time factor, T_v, settlement time can be computed for various percentages of settlement due to primary consolidation, to develop a predicted settlement-time curve. A typical settlement-time curve for a clay deposit under an embankment loading is shown in Figure 7-14.

Table 7-4
Average degree of consolidation, U, versus Time Factor, T_v, for uniform initial increase in pore water pressure

U %	T_v
0	0.000
10	0.008
20	0.031
30	0.071
40	0.126
50	0.197
60	0.287
70	0.403
80	0.567
90	0.848
93.1	1.000
95.0	1.163
98.0	1.500
99.4	2.000
100.0	Infinity

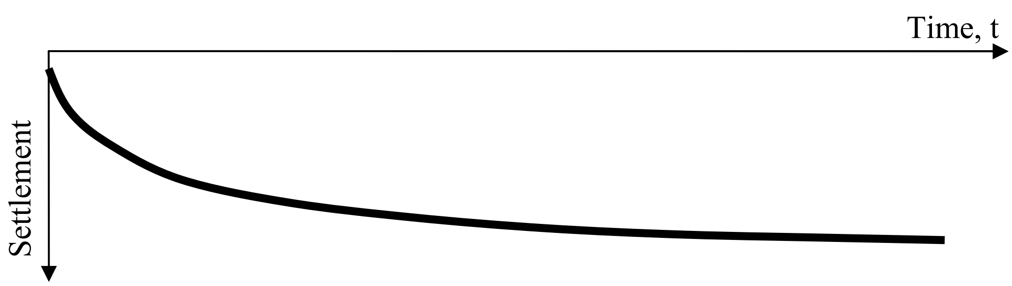


Figure 7-14. Typical settlement-time curve for clay under an embankment loading.

7.5.3.2 Step-by-Step Procedure to Determine Amount and Time for Consolidation

The step-by-step process for determining the amount of and time for consolidation to occur for a single-stage construction of an embankment on soft ground is outlined below:

1. From laboratory consolidation test data determine the e-log p curve and estimate the change in void ratio that results from the added weight of the embankment. Create the virgin field consolidation curve by using the guidelines presented in Table 7-3.

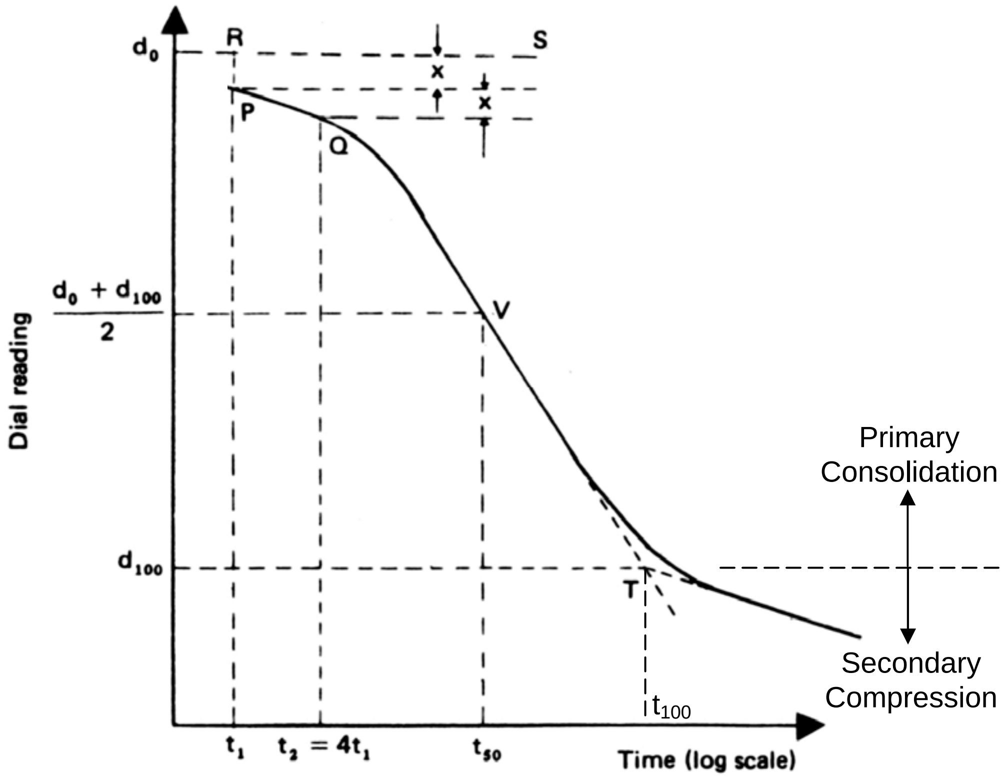
2. Determine if the foundation soil is normally consolidated, overconsolidated or under-consolidated.

3. Use Equations 7-2 to 7-7 to compute the primary consolidation settlement for normally consolidated, overconsolidated and under-consolidated foundation soils.

4. Determine c_v from laboratory consolidation test data. Two graphical procedures are commonly used for this determination are the **logarithm-of-time method** (log t) proposed by Casagrande and Fadum (1940) and the **square-root-of-time method** ($\sqrt{t}$) proposed by Taylor (1948). These methods are shown in Figures 7-15 and 7-16, respectively. Because both methods are different approximations of theory, they do not give the same answers. Often the $\sqrt{t}$ method gives slightly greater values of c_v than the log t method.

5. Use Equation 7-8 to calculate the time to achieve 90% - 95% primary consolidation.

For a more detailed discussion on the consolidation theory, the reader is referred to Holtz and Kovacs (1981). An alternative approach to hand calculations is the use of a computerized method. For example, program FoSSA (2003) by ADAMA Engineering, Version 1.0 licensed to FHWA, which was introduced in Chapter 2, calculates the time rate of settlement for various boundary conditions including the effects of staged construction and strip drains in addition to calculating the stresses and settlements. FoSSA (2003) also allows for simulation of multiple layers undergoing simultaneous consolidation. In any event, the step-by-step hand calculations can serve to verify the correctness of benchmark cases and thereby be used to ascertain the correctness of any computerized procedure.

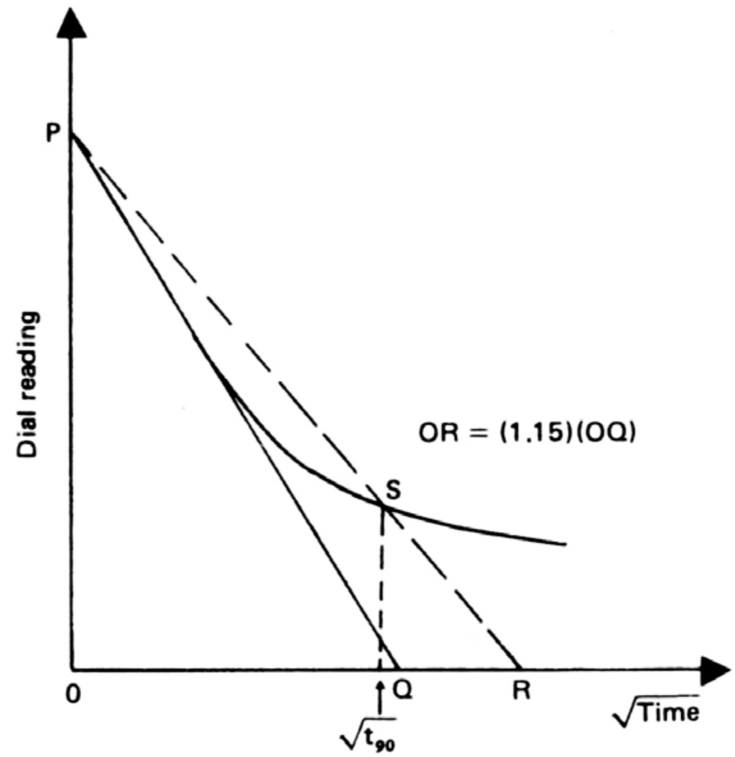


Step-by-step procedure:

1. Plot the dial readings for sample deformation for a given load increment against time on a semi-log paper.
2. Plot two points, P and Q on the upper portion of the consolidation curve which correspond to time t_1 and t_2, respectively. Note that $t_2 = 4\ t_1$.
3. The difference of the dial readings between P and Q is equal to x. Locate point R, which is at a distance x above point P.
4. Draw the horizontal line RS. The dial reading corresponding to this line is d_0, which corresponds to 0% consolidation.
5. Project the straight-line portions of the primary consolidation and the flatter portion towards the end of the consolidation curve to intersect at T. The dial reading corresponding to T is d_{100}, i.e., 100% primary consolidation. The sample deformation beyond t_{100} is due to secondary compression (see Section 7.5.4).
6. Determine the point V on the consolidation curve which corresponds to a dial reading of $(d_0+d_{100})/2 = d_{50}$. The time corresponding to the point V is t_{50}, i.e., 50% consolidation.
7. Determine c_v from Equation 7-8 for desired U. Example: For U=50% the value of T_v for is 0.197 from Table 7-4. Thus, c_v can be determined as follows:

$$c_v = \frac{0.197\,H_d^2}{t_{50}}$$

Figure 7-15. Logarithm-of-time method for determination of c_v.



Step-by-step procedure:

1. Plot the dial reading and the corresponding square-root-of-time, $\sqrt{t}$.
2. Draw the tangent PQ to the early portion of the plot.
3. Draw a line PR such that OR = (1.15) (OQ).
4. The abscissa of the point S (i.e., the intersection of PR and the consolidation curve) will give $\sqrt{t_{90}}$, i.e., the square-root-of-time for 90% consolidation.
5. Determine c_v from Equation 7-8 for U=90%. From Table 7-4, the value of T_v for U=90% is 0.848. Thus, c_v can be determined as follows:

$$c_v = \frac{0.848\ H_d^2}{t_{90}}$$

Figure 7-16. Square-root-of-time method for determination of c_v.

<u>Comments on c_v value</u>: The value of c_v is determined for a given load increment. It varies from increment to increment and is different for loading and unloading. Moreover, c_v, usually varies considerably among samples of the same soil. Therefore, **if the actual rate of consolidation is critical to the design, as in certain stability problems where excess pore water pressures must be known accurately, pore pressures must actually be measured in the field as construction proceeds.**

Regardless of whether hand-calculations or computerized methods are used, the important factors to remember are:

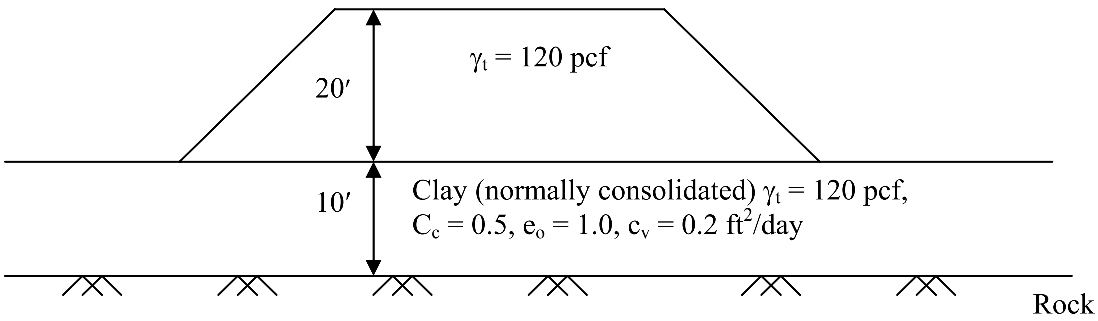
- the time required for consolidation is proportional to the square of the longest distance required for water to drain from the deposit and,
- the rate of settlement decreases as time increases.

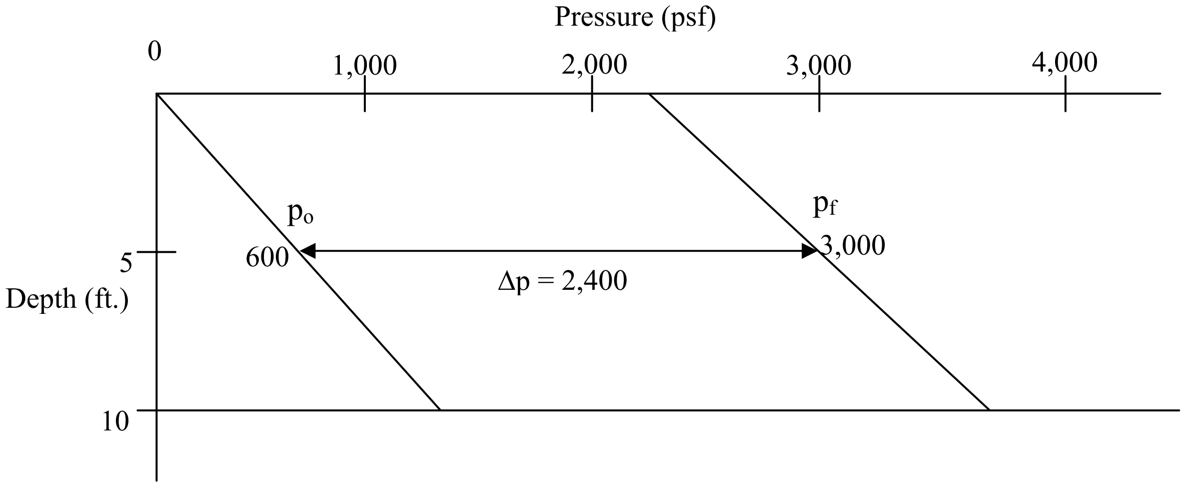
The maximum length of vertical drainage path, H_d, bears further explanation. This term should not be confused with the H term in the equation for the computation of the settlement magnitude. H is an arbitrarily selected value usually representing a portion of the total compressible layer thickness. For calculating the magnitude of settlement the sum of the sublayer H values must equal the total thickness of the clay layer. For calculating the time rate of settlement, the H_d term in Equation 7-8 is the maximum vertical distance that a water molecule must travel to escape from the compressible layer to a more permeable layer. In the case of a 20 ft (6 m) thick clay layer bounded by a sand layer on top and a virtually impermeable rock stratum on the bottom, the H_v term would equal to 20 ft (6 m). The water molecule must travel from the bottom of the layer to the top of the layer to escape, i.e., single drainage. However, if the clay layer was bounded top and bottom by more permeable sand deposits, the H_v distance would be 10 ft (3 m). The water molecule in this case, needs only to travel from the center of the layer to either boundary to escape, i.e., double drainage. However, regardless of the boundary drainage conditions, the sum of the sublayer H values must equal 20 ft (6 m) in the settlement computations.

The mechanism for determining the maximum horizontal path for escape of a water molecule is similar. The influence of horizontal drainage may be significant if the width of the loaded area is small. For instance, during consolidation under a long, narrow embankment, a water molecule can escape by traveling a distance equal to one half the embankment width. However, for very wide embankments the beneficial effect of lateral drainage may be small as the time for lateral escape of a water molecule increases as the square of one-half the embankment width.

The concepts of consolidation settlement and time rates of consolidation with reference to an embankment loading are illustrated by the following example.

Example 7-3: Determine the magnitude of and the time for 90% consolidation for the primary settlement of a "wide" embankment by using the p_o diagram.





Solution:

Since the embankment is "wide," the vertical stress at the base of the embankment is assumed to be the same within the 10-ft thick clay layer. Since soil is normally consolidated, use Equation 7-2 to determine the primary consolidation settlement as follows:

$$\Delta H = H \frac{C_c}{1 + e_o} \log_{10} \frac{p_o + \Delta p}{p_o}$$

$$\Delta H = 10 \text{ ft} \left(\frac{0.5}{1 + 1.0} \right) \log_{10} \frac{600 \text{ psf} + 2{,}400 \text{ psf}}{600 \text{ psf}} = 1.75 \text{ ft} = 21 \text{ inches (0.53 m)}$$

Find time for 90% consolidation use T_v = 0.848 from Table 7-4. Assume single vertical drainage due to impervious rock underlying clay layer and use Equation 7-8 to calculate the time required for 90% consolidation to occur.

$$t_{90} = \frac{TH_d^2}{c_v}$$

$$t_{90} = \frac{(0.848)(10\ \text{ft})^2}{0.2\ \text{ft}^2/\text{day}} = 424\ \text{days}$$

7.5.4 Secondary Compression of Cohesive Soils

Secondary compression is the process whereby the soil continues to displace vertically after the excess pore water pressures are dissipated to a negligible level i.e., primary compression is essentially completed. Secondary compression is normally evident in the settlement-log time plot when the specimen continues to consolidate beyond 100 percent of primary consolidation, i.e., beyond t_{100}, as shown in Figure 7-15. An example is shown in Figure 7-17, where secondary compression occurs beyond t_{100} = 392 mins. There are numerous hypotheses as to the reason for the secondary compression. The most obvious reason is associated with the simplifications involved in the theory of one-dimensional consolidation derived by Terzaghi. More rigorous numerical solutions accounting for the simplifications can often predict apparent secondary compression effects.

The magnitude of secondary compression is estimated from the coefficient of secondary compression, C_α, as determined from laboratory tests by using Equation 7-9 that is derived from Figure 7-17.

$$C_\alpha = \frac{\Delta e}{\log_{10}\left(\frac{t_{2\,lab}}{t_{1\,lab}}\right)} \qquad 7\text{-}9$$

where: $t_{1\,lab}$ = time when secondary compression begins and is typically taken as the time when 90 percent of primary compression has occurred

$t_{2\,lab}$ = an arbitrary time on the curve at least one log-cycle beyond t_{90} or the time corresponding to the service life of the structure

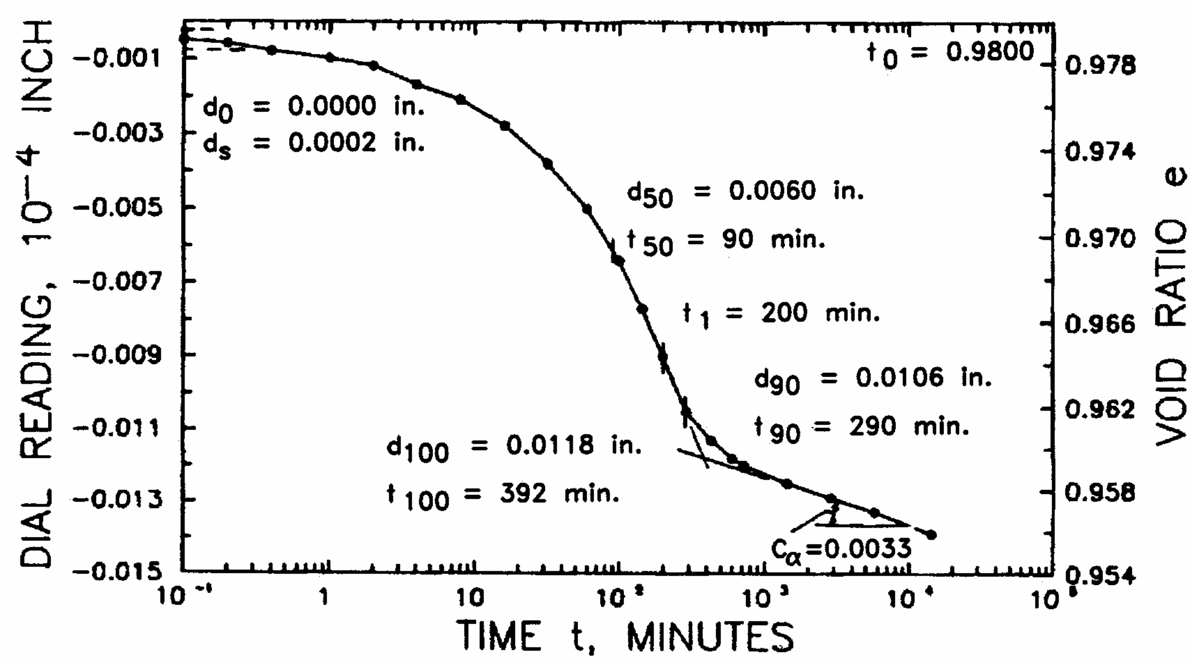


Figure 7-17: Example time plot from one-dimensional consolidometer test for determination of secondary compression (USACE, 1994). (1 in = 25.4 mm)

The settlement due to secondary compression (S_s) is then determined from Equation 7-10.

$$S_s = \frac{C_\alpha}{1+e_o} H_c \log_{10}\left(\frac{t_2}{t_1}\right) \qquad 7\text{-}10$$

where: t_1 = time when approximately 90 percent of primary compression has occurred for the actual clay layer being considered as determined from Equation 7-8.

t_2 = the service life of the structure or any other time of interest.

The values of C_α can be determined from the dial reading vs. log time plots associated with the one-dimensional consolidation test as shown in Figure 7-17. Typical ranges of the ratio of C_α/C_c presented in Section 5.4.6.4 of Chapter 5 can be used to check laboratory test results.

7.6 LATERAL SQUEEZE OF FOUNDATION SOILS

When the geometry of the applied load is larger than the thickness of the compressible layer or when there is a finite soft layer within the depth of significant influence (DOSI) below the loaded area, significant lateral stresses and associated lateral deformations can occur as shown earlier in Figure 2-16 in Chapter 2. For example, as shown in Figure 7-18, if the thickness of a soft soil layer beneath an embankment fill is such that it less than the width, b_e, of an end or side slope, then the soft soil may squeeze out.

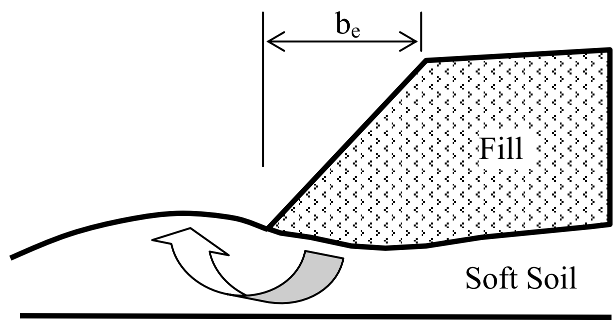


Figure 7-18. Schematic of lateral squeeze phenomenon.

The lateral squeeze phenomenon is due to an unbalanced load at the surface of the soft soil. The lateral squeeze behavior may be of two types, (a) short-term undrained deformation that results from a local bearing capacity type of deformation, or (b) long-term drained, **creep-**type deformation. **Creep refers to the slow deformation of soils under sustained loads over extended periods of time and can occur at stresses well below the shear strength of the soil.** As discussed in Section 5.4.1, secondary compression is a form of creep deformation while primary consolidation is not.

The lateral squeeze phenomenon can be observed in the field. For example, field observations and measurements have shown that some bridge abutments supported on piles driven through compressible soils tilted toward the embankment fill. Many of the abutments experienced large horizontal movements resulting in damage to the structure. The cause of this problem is attributed to the unbalanced fill load, which "squeezes" the soil laterally as discussed previously. This "lateral squeeze" of the soft foundation soil can apply enough lateral thrust against the piles to bend or even shear the piles. This problem is illustrated in Figure 7-19. The bridge abutment may tilt forward or backward depending on a number of factors including the relative configuration of the fill and the abutment, the relative stiffness of the piles or shafts and the soft deposit, the strength and thickness of the soft layer, rate of construction of the fill, and depth to bearing layer.

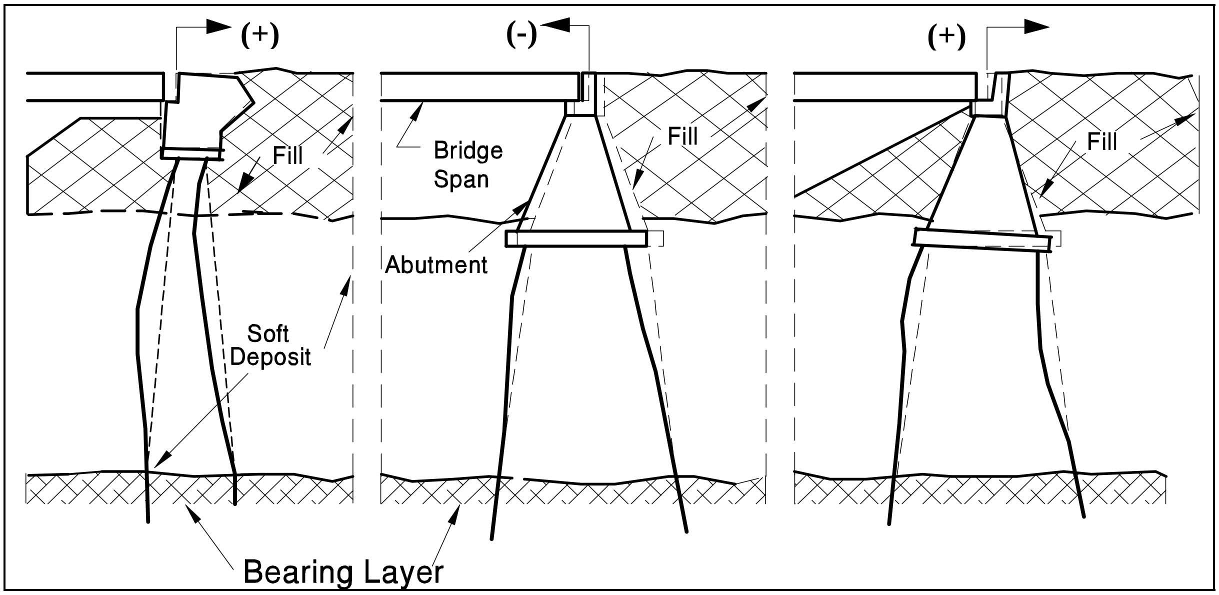


Figure 7-19. Examples of abutment tilting due to lateral squeeze (FHWA, 2006a).

7.6.1 Threshold Condition for Lateral Squeeze

Experience has shown that lateral squeeze of the foundation soil can occur and abutment tilting may result if the surface load applied by the weight of the fill exceeds 3 times the undrained shear strength, s_u, of the soft foundation soil, i.e.,

$$(\gamma)(H) > 3s_u \qquad 7\text{-}11$$

where, γ is the unit weigh of the fill and H is the height of the fill. The possibility of abutment tilting can be evaluated in design by using the above relationship. Whether the lateral squeeze will be short-term or long-term can be determined by evaluating the consolidation rate of settlement with respect to the rate of application of the load. For practical purposes, the unit weight of an embankment fill can be assumed to be approximately 125 pcf (19.7 kN/m^3). The undrained shear strength, s_u, of the foundation soil can be determined either from in-situ field vane shear tests or from laboratory triaxial tests on high quality undisturbed Shelby tube samples.

7.6.2 Calculation of the Safety Factor against Lateral Squeeze

The safety factor against failure by squeezing, FS_{SQ}, may be calculated by Equation 7-12 (Silvestri, 1983). The geometry of the problem and the forces involved are shown in Figure 7-20.

$$FS_{SQ} = \left[\frac{2s_u}{\gamma D_S \tan\theta}\right] + \left[\frac{4.14 s_u}{\gamma H}\right] \qquad 7\text{-}12$$

where:
- θ = angle of slope
- γ = unit weight of the fill
- D_S = depth of soft soil beneath the toe of the end slope or side slope of the fill
- H = height of the fill
- s_u = undrained shear strength of soft soil beneath the fill

Caution is advised when Equation 7-12 is used. It was found that when $FS_{SQ} < 2$, a rigorous slope stability analysis and possibly advanced numerical analysis, e.g., finite element analysis should be performed. When the depth of the soft layer, D_S, is greater than the base width of the end slope, $b=H/\tan\theta$, general slope stability behavior governs the design. In that case, the methods described in Chapter 6 (Slope Stability) may be used to evaluate the stability of the foundation soils.

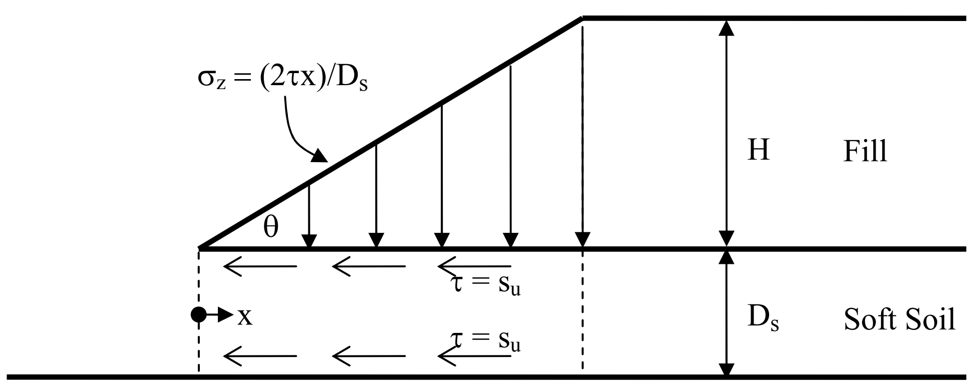


Figure 7-20. Definitions for calculating safety factor against lateral squeeze (after Silvestri, 1983).

7.6.3 Estimation of Horizontal Movement of Abutments

The amount of horizontal movement the abutment may experience can also be estimated in design. Information from case histories for nine structures where measurements of abutment movements occurred is documented in Table 7-5.

The data presented in Table 7-5 provides a basis for estimating horizontal movement for abutments under similar conditions, provided a reasonable estimate of the post-construction fill settlement is made by using data from consolidation tests on high quality undisturbed Shelby tube samples. Note that the data for the abutments listed in Table 7-5 shows the horizontal movement (tilt) to range from 6 to 33% of the vertical fill settlement, with the average being 21%. Therefore, as a first approximation, it can be said that if the fill load exceeds the $3s_u$ limit prescribed by Equation 7-11, then the horizontal movement (tilt) of an abutment can be reasonably estimated as approximately 25% of the vertical fill settlement for the conditions listed in Table 7-5.

Table 7-5
Summary of abutment movements (Nicu, *et al.*, 1971)

Foundation	Fill Settlement (inches)	Abutment Settlement (inches)	Abutment Tilting (inches)	Ratio of Abutment Tilting to Fill Settlement
Steel H-piles	16	Unknown	3	0.19
Steel H-piles	30	0	3	0.10
Soil bridge	24	24	4	0.17
Cast-in-place pile	12	3.5	2.5	0.19
Soil bridge	12	12	3	0.25
Steel H-piles	48	0	2	0.06
Steel H-piles	30	0	10	0.33
Steel H-piles	5	0.4	0.5 to 1.5	0.1 to 0.3
Timber Piles	36	36	12	0.33

7.7 DESIGN SOLUTIONS - DEFORMATION PROBLEMS

Both the magnitude and time rate of settlement can affect fill structures, which in turn may affect the performance of other structures such as bridge abutments that are built within or in the vicinity of the fills. There are various methods to reduce the magnitude and time rate of settlement. All of these methods can be considered as ground improvement and are discussed in detail in FHWA (2006b). Two of these methods are briefly discussed in this manual. The reader is referred to FHWA (2006b) for further details. Solutions to prevent abutment tilting due to lateral squeeze are discussed in Section 7.7.3.

7.7.1 Reducing the Amount of Settlement

Settlement can be reduced by either increasing the resistance or reducing the load. Several ground improvement methods that are particularly suitable for reducing the amount of settlement are noted below.

7.7.1.1 Category 1 - Increasing the Resistance

Common ground improvement techniques that increase resistance include the following:

- Excavation and recompaction.
- Excavation and replacement.
- Vertical inclusions such as stone columns, shafts and piles. Embankments supported in this way are known as column supported embankments.
- Horizontal inclusions such as geosynthetics.
- Grouting, e.g., soil mixing, jet grouting.
- Dynamic compaction.

7.7.1.2 Category 2 - Reducing the Load

Common load reduction techniques include the following:

- Reduce grade line (reduction in height and/or flattening the slope)
- Use lightweight fill material, e.g., expanded shale, foamed concrete, geofoam.
- Bypass the soft layer with a deep foundation. Deep foundations may be used in conjunction with a load transfer platform (see FHWA 2006b).

7.7.2 Reducing Settlement Time

Often the major design consideration related to a settlement problem is the time for the settlement to occur. Low permeability clays and silty clays can take a long time to consolidate under an applied load. The settlement time is critical on most projects because it has a direct impact on construction schedules and delays increase project costs. Settlement time is also important to the maintenance personnel of a highway agency. The life cycle cost of annual regrading and resurfacing of settling roadways is usually far greater than the cost of design treatments to eliminate settlement before or during initial construction.

The two most common methods used to accelerate settlement and reduce settlement time are:

1. Application of surcharge.
2. Installation of vertical drains in the foundation soils.

Note that both of the above techniques lead to an increase in the resistance. These techniques are briefly discussed below and their use is illustrated in the Apple Freeway example in Appendix A.

7.7.2.1 Surcharge Treatment

An embankment surcharge is constructed to a predetermined height, usually 1 to 10 ft (0.3 to 3 m) above final grade elevation based on settlement calculations. The surcharge is maintained for a predetermined waiting period (typically 3 to 12 months) based on settlement-time calculations. Depending upon the strength of the consolidating layer(s) the surcharge may have to be constructed in stages. The actual dimensions of the surcharge and the waiting period for each stage depend on the strength and drainage properties of the foundation soil as well as the initial height of the proposed embankment. The length of the waiting period can be estimated by using laboratory consolidation test data. The actual settlement occurring during embankment construction is then monitored with geotechnical instrumentation. When the settlement with surcharge equals the settlement originally estimated for the embankment, the surcharge is removed, as illustrated in Figure 7-21.

If the surcharge is not removed after the desired amount of settlement has occurred, then additional settlement will continue to occur. Note that the stability of a surcharged embankment must be checked as part of the embankment design to ensure that an adequate short term safety factor exists. The stability is often field verified by monitoring with instrumentation such as inclinometers, piezometers and settlement points as discussed later.

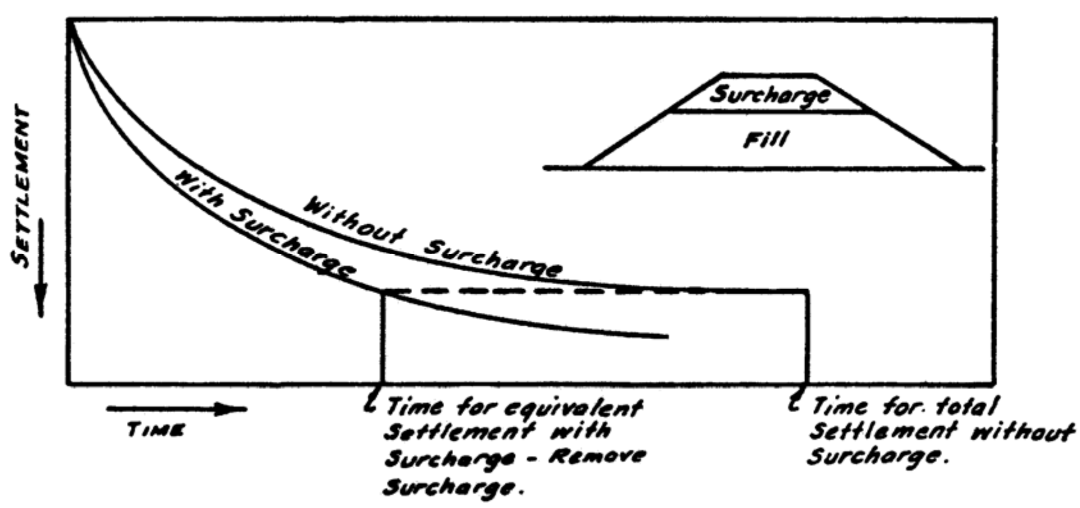


Figure 7-21. Determination of surcharge time required to achieve desired settlement.

7.7.2.2 Vertical Drains

Primary consolidation of some highly plastic clays can take many years to be completed. Surcharging alone may not be effective in reducing settlement time sufficiently since the longest distance to a drainage boundary may be significant. In such cases, vertical drains can be used to accelerate the settlement, either with or without surcharge treatment. The vertical drains accelerate the settlement rate by reducing the drainage path the water must travel to escape from the compressible soil layer to half the horizontal distance between drains, as illustrated in Figure 7-22. In most applications, a permeable sand blanket, 2 to 3 ft (0.6 to 1 m) thick, should be placed on the ground surface to permit free movement of water away from the embankment area and to create a working platform for installation of the drains. The drains are installed prior to placement of the embankment. The applied pressure from the embankment generates excess pore water pressure.

Recall that the consolidation time is proportional to the square of the length of the longest drainage path. Thus if the length of the drainage path is shortened by 50%, the consolidation time is reduced by a factor of four. Vertical drains and sand blankets should have high permeability to allow the water squeezed out of the subsoil to travel relatively quickly through the drains and the blanket.

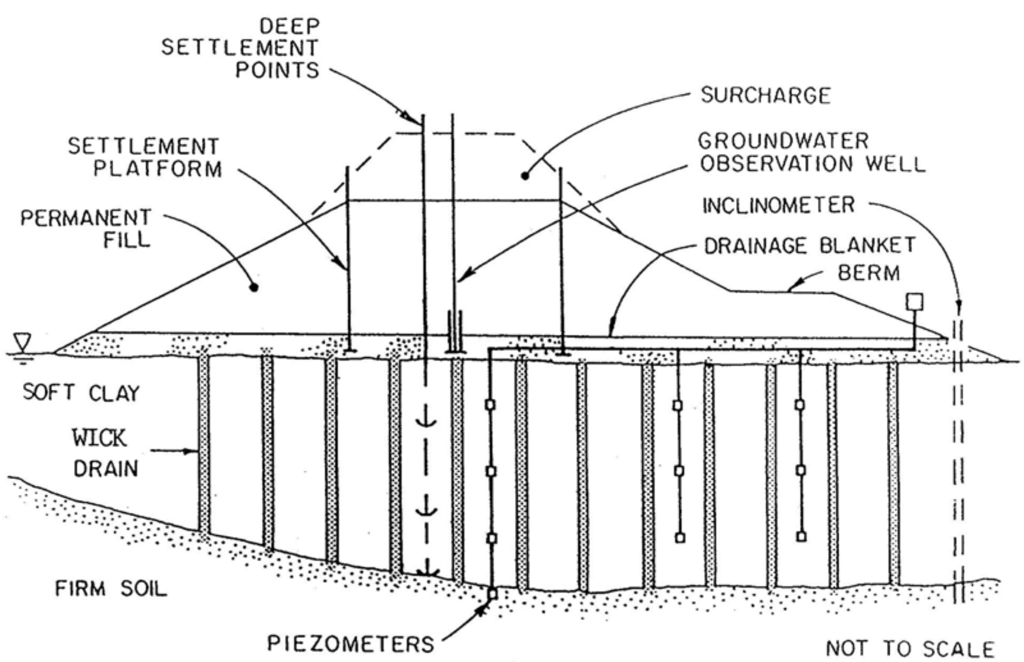


Figure 7-22. Use of vertical drains to accelerate settlement (NCHRP, 1989).

Wick drains are small prefabricated drains consisting of a plastic core that is wrapped with geotextile. Wick drains are typically 4 inches (100 mm) wide and about 1/4 inch (7 mm) thick. The drains are produced in rolls that can be fed into a mandrel. Wick drains are installed by pushing or vibrating a mandrel into the ground with the wick drain inside. When the bottom of the compressible soil is reached, the mandrel is withdrawn and the trimmed portion of the wick drain left in the ground. To minimize smear of the compressible soil, the cross-sectional area of the mandrel is recommended to be limited to a maximum of about 10 in^2 (6,450 mm^2). Predrilling of dense soil deposits may be required in some cases to reach the design depth. Use of wick drains in the United States began in the early 1970s. Design and construction guidance on the use of wick drains is provided in FHWA (1986, 2006b).

The feasibility of a surcharge solution should always be considered first since vertical drains are generally more expensive.

7.7.3 Design Solutions to Prevent Abutment Tilting

A recommended solution to minimize abutment-tilting is to induce settlement of the fill before the abutment piles or shafts are installed. If the construction time schedule or other factors do not permit pre-consolidation of the foundation soils before the piles or shafts are installed, then abutment tilting issues can be mitigated by the following design provisions:

1. Use sliding plate expansion shoes large enough to accommodate the anticipated horizontal movement.

2. Make provisions to fill in the bridge deck expansion joint over the abutment by inserting either metal plate fillers or larger neoprene joint fillers.

3. Design the deep foundations for downdrag forces due to settlement. This solution does not improve the horizontal displacement effects.

4. Use backward battered piles at the abutment and particularly at the wingwalls.

5. Use lightweight fill materials to reduce driving forces

Displacements should also be monitored during and after construction so that the predicted movements can be compared to actual displacements. Displacements should be monitored by survey monuments or protected prisms installed on the face of the abutment and wingwalls and should be tied into permanent benchmarks.

7.8 PRACTICAL ASPECTS OF EMBANKMENT SETTLEMENT

Few engineers realize the influence of embankment construction on the response of subsoils. The total weight of an embankment has an impact on the type of foundation treatment that may be appropriate. For instance, a relatively low height embankment of 10 ft (3 m) may be effectively surcharged because the additional surcharge weight could be 30 to 40 percent of the proposed embankment weight. However, when the embankment height exceeds 50 ft (15 m) the influence of a 5 or 10 ft (1.5 or 3 m) trapezoid of soil on top of this heavy 50 ft (15 m) mass is small and probably not cost-effective. Conversely, as the embankment height increases, the use of a shallow foundation for support of the abutment becomes more attractive. A 30 ft (9 m) high, 50 ft (15 m) long approach embankment weighs about 15,000 tons (130 MN) compared to the insignificant weight of a total (stub type) abutment loading that may equal 1,000 tons (9 MN). The width of an embankment also has an effect on total settlement. Wider embankments cause a pressure increase deeper into the subsoil. As might be expected, wider embankments may also cause more immediate and consolidation settlement and increase the time for consolidation to occur.

Recent developments in computer software readily permit computer analysis of approach embankment settlement. Programs such as FoSSA (2003), discussed in Chapter 2, allow the user to compute settlements along abutments and to evaluate the effects of settlements on pipes buried in end slopes or pipes placed diagonally under approach fills.

7.9 CONSTRUCTION MONITORING AND QUALITY ASSURANCE

Approach embankment construction should be clearly defined in standard drawings as to materials and limits of placement. Such standards assure uniformity in construction due to the familiarity of the construction personnel with the operations being performed and results expected. Designers should attempt to use standard details wherever possible. Attempts at small changes in materials or limits are generally counterproductive to good construction where repetition of good practice is an important factor.

The philosophy of approach embankment details is to insure adequate bearing capacity for abutments or piers placed in the embankment and to minimize settlement of the pavement or footing. Typical highway embankments require compaction to 90 percent of maximum dry density (AASHTO T180) to control pavement settlement. **Designers should specify materials and compaction control as shown in Figure 7-4, to limit differential settlement between the structure and approach fill**. If piles are used to support footings in fill, the largest particle size of embankment material should be limited to 6 in (150 mm) to ease pile installation either by driving or pre-drilling. If spread footings are used, a minimum of 5 ft (1.5 m) of select material compacted to 100 percent of maximum dry density (AASHTO T99) should be placed beneath the footing and extended beyond the wingwalls. This layer provides uniform support for the footing and a rigid transition between the structure and the fill to minimize differential settlement. Construction control is usually referenced to percent compaction on the standard design drawings.

7.9.1 Embankment Construction Monitoring by Instrumentation

The observational approach to design involves monitoring subsoil behavior during early construction stages to verify design and to predict responses to subsequent construction. Basic soil mechanics concepts can be used to predict future subsoil behavior accurately if data from instrumentation are analyzed after initial construction loads have been placed. Occasionally a design problem arises that is unique or extremely critical and that can be safely solved only by utilizing the observational approach.

Embankment placement must be carefully observed and monitored on projects where stability and/or settlement are critical. The monitoring should include visual observation by the construction inspection staff and the use of instrumentation. Without the aid of various forms of instrumentation, it is impossible to determine accurately what is happening to the foundation. Instrumentation can be used to warn of imminent failure or to indicate whether settlement is occurring as predicted. The type of instruments to be used and where they will be placed should be planned by a qualified and experienced geotechnical specialist. Actual interpretation and analysis of the data should also be performed by someone with a background in soil mechanics; however, the project engineer and inspector should understand the purpose of each type of instrumentation and how the data are to be used.

7.9.1.1 Inspector's Visual Observation

In areas of marginal embankment stability, the inspector should walk the surface of the embankment daily looking for any sign of cracking or movement. Hairline cracks often develop at the embankment surface just prior to failure. If the inspector discovers any such features, all fill operations should cease immediately. All instrumentation should be read immediately. The geotechnical specialist should be notified. Subsequent readings will indicate when it is safe to resume operations. Unloading by removal of fill material or other mitigation methods are sometimes necessary to prevent an embankment failure.

7.9.1.2 Types of Instrumentation

The typical instrumentation specified to monitor foundation performance on projects where stability and settlement are critical consists of:

1. **Slope Inclinometers** are used to monitor subsurface lateral deformation. A slope inclinometer typically consists of a 3 in (75 mm) internal diameter (ID) plastic tube with four grooves cut at 90-degree intervals around the inside. The slope inclinometer tube is installed in a borehole. The bottom of the slope inclinometer tube must be founded in firm soil or rock. A readout probe that fits into the grooves is lowered down the tube and angular deflection of the tube is measured. The amount and location of horizontal movement in the foundation soil can then be measured. For embankments built over very soft subsoils, telescoping inclinometer casing should be used to account for vertical consolidation. In soft ground conditions, several inches of lateral movement due to squeeze may occur without shear failure as the embankment is built. Therefore, from a practical construction control standpoint, the rate of movement rather than the amount is the better indicator of imminent failure. Slope inclinometer readings should be made often during the critical embankment placement period, daily if fill placement is proceeding rapidly, and readings should be plotted immediately on a movement versus time plot. Fill operations should cease if a sudden increase in the rate of movement occurs.

2. **Piezometers** indicate the amount of excess pressure build-up within the water-saturated pores of the soil. There are critical levels to which the water pressure in the subsoil will increase just prior to failure. The geotechnical specialist can estimate the critical water pressure level during design. Normally, the primary function of piezometers during fill placement is to warn of failures. Once the embankment placement is complete, the piezometers are used to measure the rate of consolidation. There are several different types of piezometers. The simplest is the open standpipe type, which is essentially a well point with a metal or plastic pipe attached to it. The pipe is extended up through the fill in sections as the fill height increases. This type of open well piezometer has the disadvantage that the pipes are susceptible to damage if hit by construction equipment. Also, the response time of open well piezometers is often too slow in soft clays to warn of potential embankment failure. There are several types of remote piezometers that eliminate the requirement for extending a pipe up through the fill. The remote units consist of a piezometer transducer that is sealed in a borehole with leads carried out laterally under the base of the embankment to a readout device that records the pore water pressure measured by the transducer. Pneumatic or vibrating wire piezometers

have a more rapid response to changes in pore water pressure than open-stand pipe piezometers.

3. **Settlement devices** are used to measure the amount and rate of settlement of the foundation soil due to the load from the embankment. Typically they are installed on or just below the existing ground surface before any fill is placed. The simplest settlement device is a settlement plate usually a 3 or 4 ft (0.9 to 1.2 m square plywood mat or steel plate with a vertical reference rod (usually ¾ in (19 mm) pipe) attached to the plate. The reference rods are normally added 4 ft (1.2 m) at a time as the height of the embankment increases. The elevation of the top of the reference rod is surveyed periodically to measure the foundation settlement. Remote pneumatic settlement devices are also available. As with the remote piezometer devices, the remote settlement devices have the advantage of not having a reference rod extending up through the fill.

7.9.1.3 Typical Locations for Instruments

Instrument installations should be spaced approximately 250 to 500 ft (75 to 150 m) along the roadway alignment in critical areas. Typical locations of instruments for an embankment over soft ground are shown in Figure 7-23:

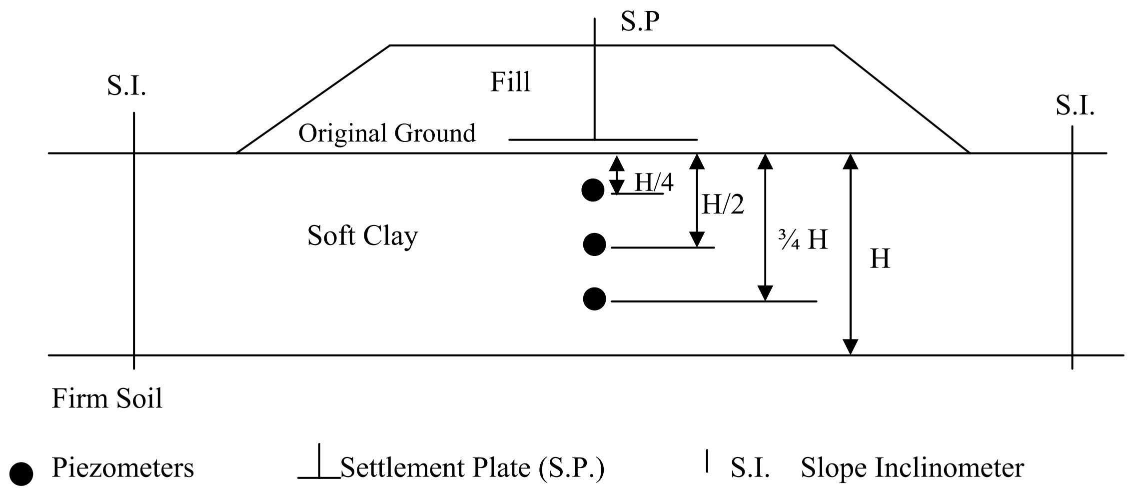


Figure 7-23. Typical locations for various types of monitoring instruments for an embankment constructed over soft ground.

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