Soil and Soil Mechanics textbook

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Soil

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Soil Mechanics

Textbook

Collected by

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I started writing this simplified textbook at the beginning of the 1st semester of my doctoral study in Czech Republic at VŠB-Technical University of Ostrava . Through my research and preparations for my study, I got this idea to write this small textbook for civil Engineering students, in an effort to simplify the concepts as well help in making it easier for them by collecting necessary information in a concise manner.

I would like to thank and acknowledge all the researchers and authors whom I relied on their knowledge in my textbook and also, as well as recognize all the other internet resources that proved very valuable in putting together this textbook.

It's my pleasure if there is any mistake in this textbook to contact me by e-mail
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TO MY PARENTS PRICEY

TO MY DEAR WIFE

TO MY CHILDREN OF LOVED ONES

DEAR PROFESSORS WITH RESPECT

TO STUDENTS DEAR

I DEDICATE TO YOU MY SIMPLE WORK

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CONVERSION FACTORS FROM ENGLISH TO SI UNITS

Quantity	English	SI
Length:	1 ft	= 0.3048 m
	1 ft	= 30.48 cm
	1 ft	= 304.8 mm
	1 in.	= 0.0254 m
	1 in.	= 2.54 cm
	1 in.	= 25.4 mm
Area:	1 ft^2	= 929.03×10^{-4} m^2
	1 ft^2	= 929.03 cm^2
	1 ft^2	= 929.03×10^2 mm^2
	1 $in.^2$	= 6.452×10^{-4} m^2
	1 $in.^2$	= 6.452 cm^2
	1 $in.^2$	= 645.16 mm^2
Volume:	1 ft^3	= 28.317×10^{-3} m^3
	1 ft^3	= 28.317×10^3 cm^3
	1 $in.^3$	= 16.387×10^{-6} m^3
	1 $in.^3$	= 16.387 cm^3
Section modulus:	1 $in.^3$	= 0.16387×10^5 mm^3
	1 $in.^3$	= 0.16387×10^{-4} m^3
Hydraulic conductivity:	1 ft/min	= 0.3048 m/min
	1 ft/min	= 30.48 cm/min
	1 ft/min	= 304.8 mm/min
	1 ft/sec	= 0.3048 m/sec
	1 ft/sec	= 304.8 mm/sec
	1 in./min	= 0.0254 m/min
	1 in./sec	= 2.54 cm/sec
	1 in./sec	= 25.4 mm/sec
Coefficient of consolidation:	1 $in.^2$/sec	= 6.452 cm^2/sec
	1 $in.^2$/sec	= 20.346×10^3 m^2/yr
	1 ft^2/sec	= 929.03 cm^2/sec
Force:	1 lb	= 4.448 N
	1 lb	= 4.448×10^{-3} kN
	1 lb	= 0.4536 kgf
	1 kip	= 4.448 kN
	1 U.S. ton	= 8.896 kN
	1 lb	= 0.4536×10^{-3} metric ton
	1 lb/ft	= 14.593 N/m
Stress:	1 lb/ft^2	= 47.88 N/m^2
	1 lb/ft^2	= 0.04788 kN/m^2
	1 U.S. ton/ft^2	= 95.76 kN/m^2
	1 kip/ft^2	= 47.88 kN/m^2
	1 lb/$in.^2$	= 6.895 kN/m^2
Unit weight:	1 lb/ft^3	= 0.1572 kN/m^3
	1 lb/$in.^3$	= 271.43 kN/m^3
Moment:	1 lb-ft	= 1.3558 N · m
	1 lb-in.	= 0.11298 N · m
Energy:	1 ft-lb	= 1.3558 J
Moment of inertia:	1 $in.^4$	= 0.4162×10^6 mm^4
	1 $in.^4$	= 0.4162×10^{-6} m^4

CONVERSION FACTORS FROM SI TO ENGLISH UNITS

Length:	1 m	= 3.281 ft
	1 cm	= 3.281×10^{-2} ft
	1 mm	= 3.281×10^{-3} ft
	1 m	= 39.37 in.
	1 cm	= 0.3937 in.
	1 mm	= 0.03937 in.
Area:	1 m^2	= 10.764 ft^2
	1 cm^2	= 10.764×10^{-4} ft^2
	1 mm^2	= 10.764×10^{-6} ft^2
	1 m^2	= 1550 $in.^2$
	1 cm^2	= 0.155 $in.^2$
	1 mm^2	= 0.155×10^{-2} $in.^2$
Volume:	1 m^3	= 35.32 ft^3
	1 cm^3	= 35.32×10^{-4} ft^3
	1 m^3	= 61,023.4 $in.^3$
	1 cm^3	= 0.061023 $in.^3$
Force:	1 N	= 0.2248 lb
	1 kN	= 224.8 lb
	1 kgf	= 2.2046 lb
	1 kN	= 0.2248 kip
	1 kN	= 0.1124 U.S. ton
	1 metric ton	= 2204.6 lb
	1 N/m	= 0.0685 lb/ft

Stress:	1 N/m^2	= 20.885×10^{-3} lb/ft^2
	1 kN/m^2	= 20.885 lb/ft^2
	1 kN/m^2	= 0.01044 U.S. ton/ft^2
	1 kN/m^2	= 20.885×10^{-3} kip/ft^2
	1 kN/m^2	= 0.145 $lb/in.^2$
Unit weight:	1 kN/m^3	= 6.361 lb/ft^3
	1 kN/m^3	= 0.003682 $lb/in.^3$
Moment:	1 N · m	= 0.7375 lb-ft
	1 N · m	= 8.851 lb-in.
Energy:	1 J	= 0.7375 ft-lb
Moment of inertia:	1 mm^4	= 2.402×10^{-6} $in.^4$
	1 m^4	= 2.402×10^{6} $in.^4$
Section modulus:	1 mm^3	= 6.102×10^{-5} $in.^3$
	1 m^3	= 6.102×10^{4} $in.^3$
Hydraulic conductivity:	1 m/min	= 3.281 ft/min
	1 cm/min	= 0.03281 ft/min
	1 mm/min	= 0.003281 ft/min
	1 m/sec	= 3.281 ft/sec
	1 mm/sec	= 0.03281 ft/sec
	1 m/min	= 39.37 in./min
	1 cm/sec	= 0.3937 in./sec
	1 mm/sec	= 0.03937 in./sec
Coefficient of consolidation:	1 cm^2/sec	= 0.155 $in.^2/sec$
	1 m^2/yr	= 4.915×10^{-5} $in.^2/sec$
	1 cm^2/sec	= 1.0764×10^{-3} ft^2/sec

Glossary of symbols

a	Height of Coulomb's wedge of soil
a	Cross-sectional area of sample in axial-test
c_u	Half unconfined compression strength
c_{vc}	Coefficient of consolidation
d	Diameter, displacement, depth
e	Voids ratio
f, f'	General function and its derivative
h, h_w	Height, and height of water in standpipe
i	Hydraulic gradient
k	Maximum shear stress (Tresca)
k	Coefficient of permeability
$\boldsymbol{k}$	Cohesion in eq. (8.1)
l	Length of sample in axial-test
l	Constant in eq. (8.3) cf. λ in eq. (5.23)
m_{vc}, m_{vs}	Coefficients of volume compressibility
n	Porosity
n	Normal stress
p	Effective spherical pressure
p_e	Equivalent pressure cf. σ'_e
p_u	Undrained critical state pressure
p_x	Critical state pressure on yield curve
p_{LL}	Pressure corresponding to liquid limit
p_{PL}	Pressure corresponding to plastic limit
p_Ω	Pressure corresponding to Ω point
p^*, q^*	Generalized stress parameters
p, q	Uniformly distributed loading pressures
p'	Equivalent stress
q	Axial-deviator stress
q_u	Undrained critical state value of q
q_x	Critical state value of q on yield curve
r	Radial coordinate
r_1, r_2	Directions of planes of limiting stress ratio
s	Distance along a flowline
s_+, s_-	Parameters locating centres of Mohr's circles
s, t	Stresses in plane strain

t	Tangential stress
t	Time
$t_{1/2}$	Half-settlement time
u	Excess pore-pressure
u_w	Total pore-pressure
u	Velocity of stream
v	Velocity
v_a	Artificial velocity
v_s	Seepage velocity
v	Specific volume
v_x	Critical state value of v on yield curve
v_κ, v_λ	Ordinates of κ-line and λ-line
v_{LL}	Specific volume at liquid limit
v_{PL}	Specific volume at plastic limit
Δv_{PI}	Change of specific volume corresponding to plasticity index
v_Ω	Specific volume corresponding to Ω point
w	Water content
w	Weight
x, y, z	Cartesian coordinate axes
x_t	Transformed coordinate
A, A_t	Cross-sectional areas
A, A_m	Fourier coefficients
$A, B, \overline{B}$	Pore-pressure coefficients
C_c, C'_c	Compression indices
D	Diameter
D_0	A total change of specific volume
E	Young's modulus
$\dot{E}$	Loading power
F, F', F^*	Potential functions (Mises)
F	Frictional force
G	Shear modulus
G_s	Specific gravity
H	Maximum drainage path
H	Abscissa of Mohr-Coulomb lines
K	Bulk modulus
K, K_0	Coefficients of earth pressure
L	Lateral earth pressure force
M	Mach number
N	Overcompression ratio
N	Normal force
P, P_w, P_s	Vertical loads in consolidometer
P_A, P_P	Active and passive pressure forces
$\dot{P}$	Probing power
Q	Quantity of flow
R	Force of resistance
T	Tangential force
T_v	Time factor

$T_{½}$	Half-settlement time factor
U	Proportion of consolidation
$\dot{U}$	Recoverable power
V, V_t, V_v	Volumes
W	Weight
$\dot{W}$	Dissipated power
X_1, X_2, X_3	Loads in simple test system
Y	Yield stress in tension (Mises)
LL	Liquid limit
PL	Plastic limit
PI	Plasticity index
α, β	Pair of characteristics
α, β	Angles
γ	Engineering shear strain
γ	Saturated bulk density (by weight not mass)
γ_d	Dry bulk density (by weight not mass)
γ'	Submerged bulk density (by weight not mass)
γ_w	density of water (by weight not mass)
δ	Inclination of equivalent stress
δ	Displacement in simple test system
ε	Half angle between characteristics
$\dot{\varepsilon}$	Increment of shear strain
ε	Cumulative shear strain
$\dot{\varepsilon}_{ij}$	Components of strain increment
$\dot{\varepsilon}^*$	Generalized shear strain increment
η	Ratio of stresses q/p
η	Special parameter
θ	Angle
κ	Special parameter
κ	Gradient of swelling line
λ	Gradient of compression line
λ_+, λ_-	Inclinations of principal stress to a stress discontinuity
μ	Viscosity
v	Poisson's ratio
ν, ν^*	Scalar factors
ξ	Special parameter
ρ	Settlement
ρ	Angle of friction in eq. (8.1)
σ	Total stress
σ'	Effective stress
σ'_{ij}	Components of effective stress
σ'_e	Hvorslev's equivalent stress
σ'_+, σ'_-	Parameters locating centres of Mohr's circles
τ, τ_m	Shear stresses in direct shear test
τ_{yz} etc.	Shear stresses
ϕ	Potential function

ϕ, ϕ_m	Angles of friction in Taylor's shear tests
ϕ_+, ϕ_-	Inclinations of principal stress on either side of discontinuity
ψ	Streamline function
Γ	Ordinate of critical state line
Δ	Caquot's angle
Λ	Parameter relating swelling with compression
M	Critical state frictional constant
$\sum'$	Major principal stress
Ω	Common point of idealized critical state lines

Introduction:

The term Soil has various meanings, depending upon the general field in which it is being considered. The term 'Soil' has different meanings in different scientific fields. It has originated from the Latin word Solum. To an agricultural scientist, it means ''the loose material on the earth's crust consisting of disintegrated rock with an admixture of organic matter, which supports plant life''. To a geologist, it means the disintegrated rock material which has not been transported from the place of origin. But, to a civil engineer, the term 'soil' means, the loose unconsolidated inorganic material on the earth's crust produced by the disintegration of rocks, overlying hard rock with or without organic matter.
Foundations of all structures have to be placed on or in such soil, which is the primary reason for our interest as Civil Engineers in its engineering behaviour. The application of the principles of soil mechanics to the design and construction of foundations for various structures is known as ''Foundation Engineering''. ''Geotechnical Engineering'' may be considered to include both soil mechanics and foundation engineering.
In fact, according to Terzaghi, it is difficult to draw a distinct line of demarcation between soil mechanics and foundation engineering; the latter starts where the former ends. [DHARM N-GEO\GE1-1.PM5]
Soil mechanics is the science of equilibrium and motion of soil bodies. Here soil is understood to be the weathered material in the upper layers of the earth's crust. The non-weathered material in this crust is denoted as rock, and its mechanics is the discipline of rock mechanics. Soil mechanics has been developed in the beginning of the 20th century. The need for the analysis of the behavior of soils arose in many countries, often as a result of spectacular accidents, such as landslides and failures of foundations. [Delft, March 2012]

According to Terzaghi (1948):

"Soil Mechanics is the application of laws of mechanics and hydraulics to engineering problems dealing with sediments and other unconsolidated accumulations of solid particles produced by the mechanical and chemical disintegration of rocks regardless of whether or not they contain an admixture of organic constituent."
Some of the basic theories of soil mechanics are the basic description and classification of soil, effective stress, shear strength, consolidation, lateral earth pressure, bearing capacity, slope stability, and permeability. Foundations, embankments, retaining walls, earthworks and underground openings are all designed in part with theories from soil mechanics.
But to a geotechnical engineer, soil has a much broader meaning and can include not only agronomic material, but also broken-up fragments of rock, volcanic ash, alluvium, Aeolian sand, glacial material, and any other residual or transported product of rock weathering. Difficulties naturally arise because there is not a distinct dividing line between rock and soil. For example, to a geologist a given material may be classified as a formational rock because it belongs to a definite geologic environment, but to a geotechnical engineer it may be sufficiently weathered or friable that it should be classified as a soil.(Robert W. Day).
Study of soil behavior in a more methodical manner in the area of geotechnical engineering started in the early part of the 18thcentury, and last to 1927. The development of soil mechanics can be divided into four phases, according to Skempton (1985):

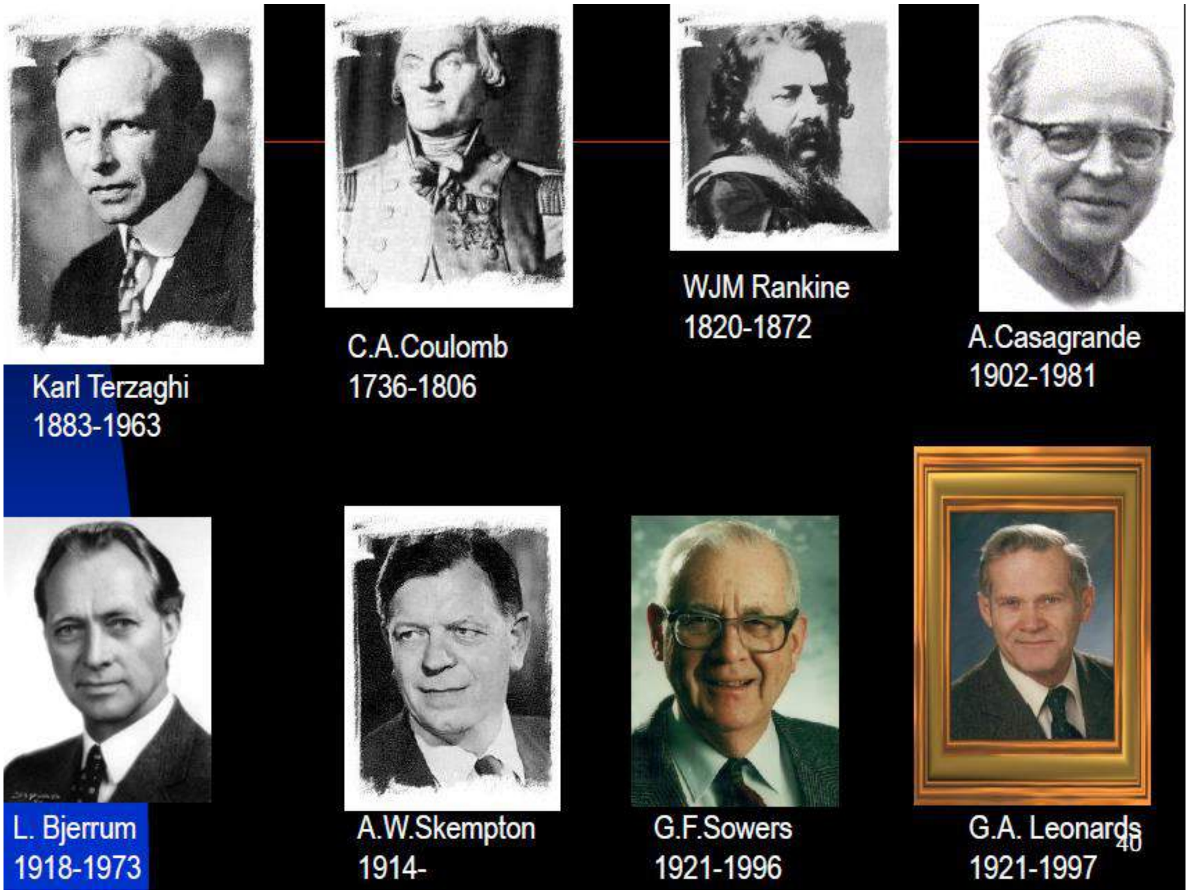


Fig.1- Hall of fame by Dr. Attaullah Shah

In this Textbook, different laboratory tests will be conducted to determine the following important index and mechanical properties of soils:

- **Water Content**
- **Organic Matter (Content)**
- **Unit Weight (Density)**
- **Specific Gravity**
- **Relative Density**
- **Atterberg Limits**
- **Grain Size Distribution (Sieve Analysis and Hydrometer Analysis)**
- **Visual Classification**
- **Moisture-Density Relationship (Compaction)**
- **Hydraulic Conductivity (Constant Head Method)**
- **Consolidation**
- **Shear Strength**
  - **Unconfined Compression Test**
  - **Direct Shear Test**

Objectives of Soil Mechanics, by Dr. Attaullah Shah

- To perform the Engineering soil surveys.
- To develop rational soil sampling devices and soil sampling methods.
- To develop suitable soil testing devices and soil testing methods.
- To collect and classify soils and their physical properties on the basis of fundamental knowledge of soil mechanics.
- To investigate the physical properties of soil and determine the coefficients to characterize these properties.
- To evaluate the soil test results and other applications as a construction material.
- To understand various factors such as static and dynamic loads, water and temperature.

Soil Modelling

“Model” assumed relationship between stress and strain for a soil. Underlying conventional design calculations in geotechnical engineering are different soil models based on concepts of elasticity and plasticity. Underlying most methods of calculating ground movements is the assumption of a linear elastic soil model.
Underlying most stability calculations is a soil model which assumes rigid, perfectly plastic behaviour.

Fields of Application of Soil Mechanics

The knowledge of soil mechanics has application in many fields of Civil Engineering

1- Foundation

The understanding of the mechanical behavior of granular material under different states of stress conditions it is of great importance for the analysis of possible failure within a soil mass that supports structures such as embankments, piles, spread footings, earth dams, etc. In diverse situations, these structures may induce a wide range of loading conditions to the soil mass beneath them, causing deformations that, in extreme cases, may lead to a total collapse. This mechanical behavior is directly associated with the strength and deformation characteristics of soils.
The loads from any structure have to be ultimately transmitted to a soil through the foundation for the structure. Thus, the foundation is an important part of a structure, the type and details of which can be decided upon only with the knowledge and application of the principles of soil mechanics.
One of the most famous examples of problems related to soil bearing capacity and foundations in the construction of structures prior to 18thcentury is the Leaning Tower of Pisa in Italy. The construction of the Tower began in 1173 A.D. and last over 200 years.

2- Underground and Earth-retaining Structures

Underground structures such as drainage structures, pipe lines, and tunnels and earth-retaining structures such as retaining walls and bulkheads can be designed and constructed only by using the principles of soil mechanics and the concept of 'soil-structure interaction'.
Various earth pressure theories assume that soils are homogeneous, isotropic and horizontally inclined. These assumptions lead to hydrostatic or triangular pressure distributions when calculating the lateral earth pressures being exerted against a vertical plane. Field measurements on deep retained excavations have shown that the average earth pressure load is approximately uniform with depth with small reductions at the top and bottom of the excavation. This type of distribution was first suggested by Terzaghi (1943) on the basis of empirical data collected on the Berlin Subway and Chicago Subway projects between1936-42.

3- Pavement Design

Pavement Design may consist of the design of flexible or rigid pavements. Flexible pavements depend more on the subgrade soil for transmitting the traffic loads. Problems peculiar to the design of pavements are the effect of repetitive loading, swelling and shrinkage of sub-soil and frost action. Consideration of these and other factors in the efficient design of a pavement is a must and one cannot do without the knowledge of soil mechanics.
Pavements are constructed on compacted soils that are typically unsaturated. The negative pore-water pressure (soil suction) due to the ingress of water in between soil particles has a significant effect on pavement foundation stiffness and strength. The study characterized the effects of soil suction on shear strength and resilient modulus of four soils representing different regions of Minnesota. The deviator stress in shear strength measurements followed a power function relationship with soil suction.

4- Excavations, Embankments and Dams

Excavations require the knowledge of slope stability analysis; deep excavations may need temporary supports—'timbering' or 'bracing', the design of which requires knowledge of soil mechanics. Likewise the construction of embankments and earth dams where soil itself is used as the construction material requires a thorough knowledge of the engineering behaviour of soil especially in the presence of water. Knowledge of slope stability, effects of seepage, consolidation and consequent settlement as well as compaction characteristics for achieving maximum unit weight of the soil in-situ, is absolutely essential for efficient design and construction of embankments and earth dams.

5- Small Housing

In the Middle East, especially in the villages, and so far there are some people build their homes from templates made of soil mixed with grass and even the roof is a soil surrounded by the large legs wood trees in the form of a beam and beyond are compacted both roof and walls to prevent leakage of water inside the house.

Soil Profile

A deposit of soil material, resulting from one or more of the geological processes described earlier, is subjected to further physical and chemical changes which are brought about by the climate and other factors prevalent subsequently. Vegetation starts to develop and rainfall begins the processes of leaching and eluviation of the surface of the soil material. Gradually, with the passage of geological time profound changes take place in the character of the soil.
These changes bring about the development of 'soil profile'.

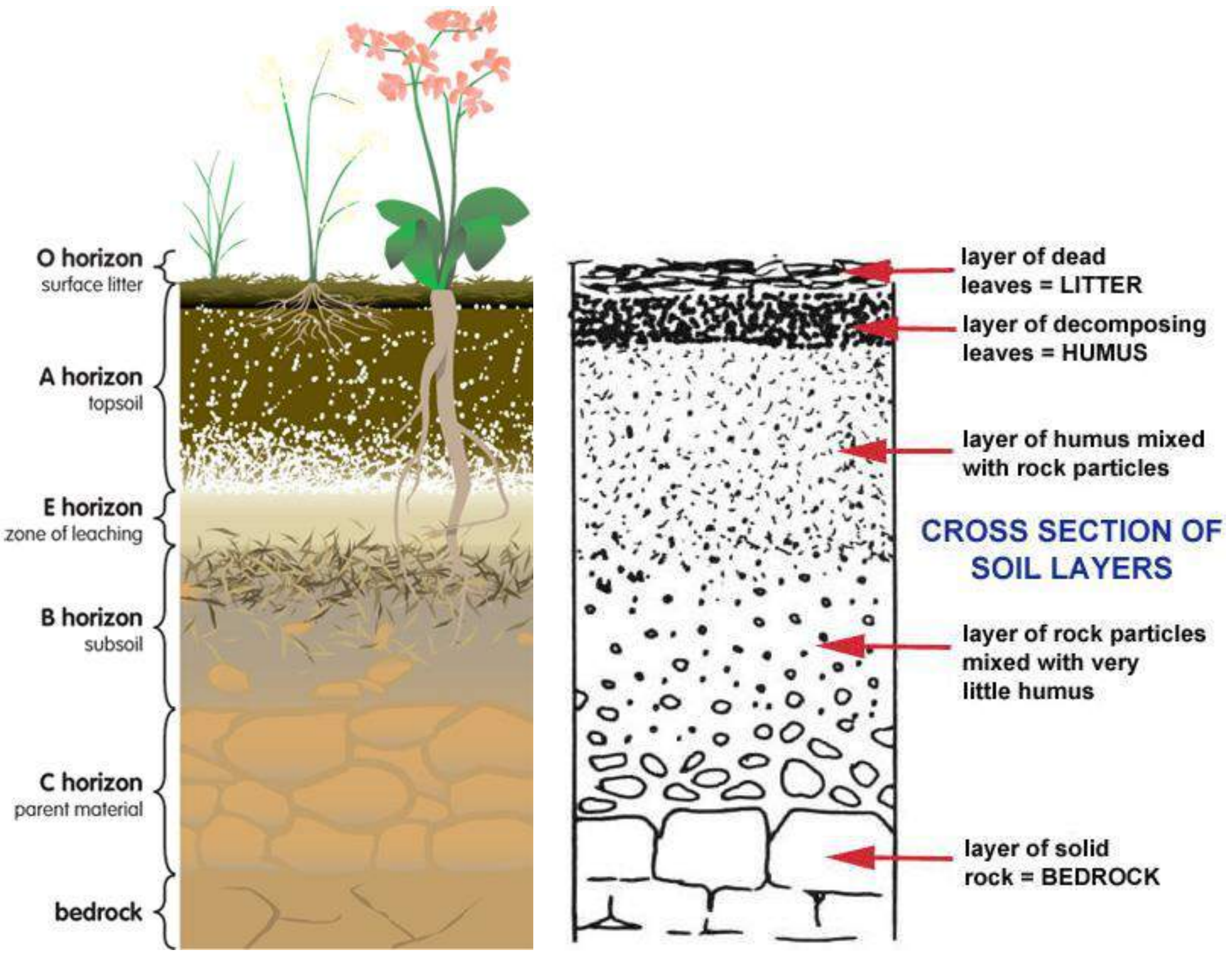


Fig.2- Topsoil – each soil horizon has specific physical and chemical properties

Generally, three distinct strata or horizons occur in a natural soil-profile; this number may increase to five or more in soils which are very old or in which the weathering processes have been unusually intense.
From top to bottom these horizons are designated as the A-horizon, the B-horizon and the C-horizon. The A-horizon is rich in humus and organic plant residue. This is usually elevated and leached; that is, the ultrafine colloidal material and the soluble mineral salts are washed out of this horizon by percolating water. It is dark in color and its thickness may range from a few centimeters to half a metre. This horizon often exhibits many undesirable engineering characteristics and is of value only to agricultural soil scientists.

The B-horizon is sometimes referred to as the zone of accumulation. The material which has migrated from the A-horizon by leaching and eluviation gets deposited in this zone. There is a distinct difference of color between this zone and the dark top soil of the A-horizon. This soil is very much chemically active at the surface and contains unstable fine-grained material.
Thus, this is important in highway and airfield construction work and light structures such as single storey residential buildings, in which the foundations are located near the ground surface. The thickness of B-horizon may range from 0.50 to 0.75 m.
The material in the C-horizon is in the same physical and chemical state as it was first deposited by water, wind or ice in the geological cycle. The thickness of this horizon may range from a few centimeters to more than 30 m. The upper region of this horizon is often oxidized to a considerable extent. It is from this horizon that the bulk of the material is often borrowed for the construction of large soil structures such as earth dams.
Each of these horizons may consist of sub-horizons with distinctive physical and chemical characteristics and may be designated as A1, A2, B1, B2, etc. The transition between horizons and sub-horizons may not be sharp but gradual. At a certain place, one or more horizons may be missing in the soil profile for special reasons.
The morphology or form of a soil is expressed by a complete description of the texture, structure, color and other characteristics of the various horizons, and by their thicknesses and depths in the soil profile. For these and other details the reader may refer ‘‘Soil Engineering’’ by M.G. Spangler.

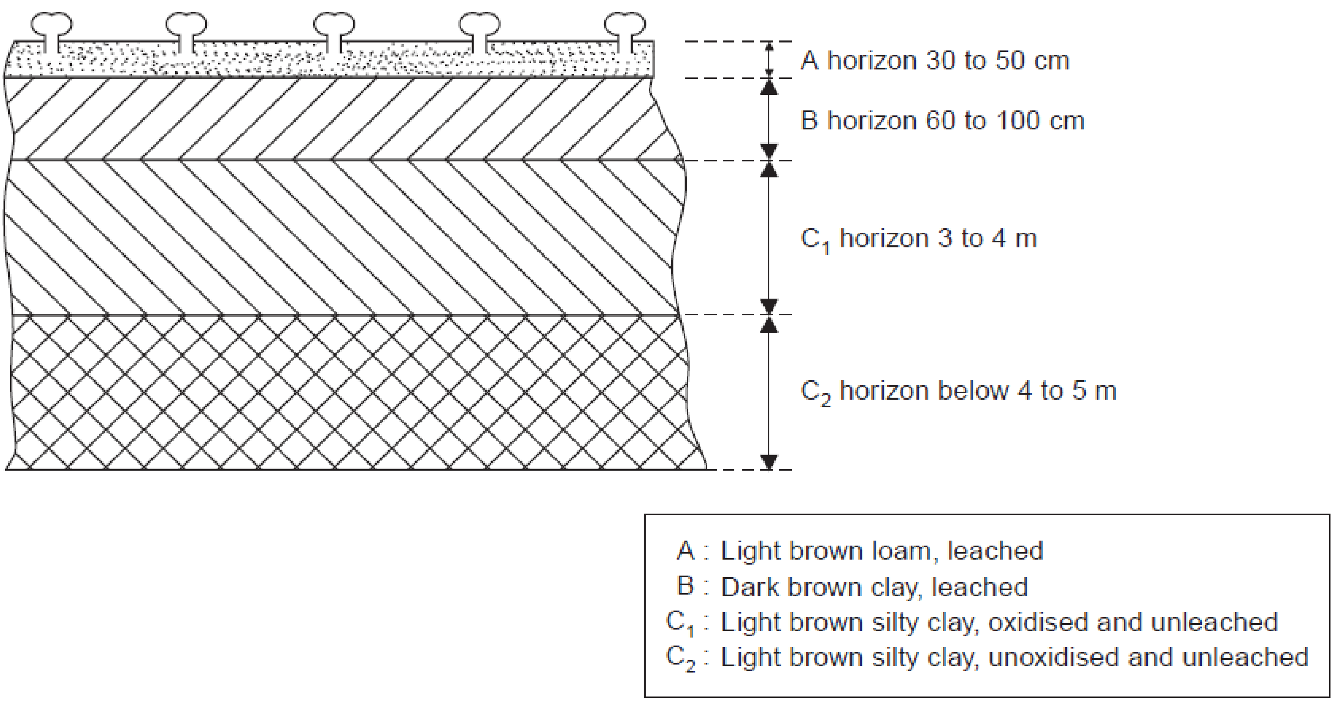


Fig.3- A typical soil profile [DHARM N-GEO\GE1-1.PM5]

Definitions and Terminology

Stiffness dependent upon stress level

Stiffness is the ratio of the force required to create a specified deflection or movement of a part. Stiffness is Force/deflection, which is expressed in lbs/in or grams/cm.
In civil engineering the non-linear property is used to great advantage in the pile foundation for a building on very soft soil, underlain by a layer of sand. In the sand below a thick deposit of soft clay the stress level is high, due to the weight of the clay. This makes the sand very hard and strong, and it is possible to apply large compressive forces to the piles, provided that they are long enough to reach well into the sand.

Shear

In compression soils become gradually stiffer. In shear, however, soils become gradually softer, and if the shear stresses reach a certain level, with respect to the normal stresses, it is even possible that failure of the soil mass occurs. This means that the slope of a sand heap, for instance in a depot or in a dam, cannot be larger than about 30 or 40 degrees. The reason for this is that particles would slide over each other at greater slopes. As a consequence of this phenomenon many countries in deltas of large rivers are very flat. It has also caused the failure of dams and embankments all over the world, sometimes with very serious consequences for the local population. Especially dangerous is that in very fine materials, such as clay, a steep slope is often possible for some time, due to capillary pressures in the water, but after some time these capillary pressures may vanish (perhaps because of rain), and the slope will fail.

Dilatancy

Shear deformations of soils often are accompanied by volume changes. Loose sand has a tendency to contract to a smaller volume, and densely packed sand can practically deform only when the volume expands somewhat, making the sand looser. This is called dilatancy, a phenomenon discovered by Reynolds, in 1885. This property causes the soil around a human foot on the beach near the water line to be drawn dry during walking. The densely packed sand is loaded by the weight of the foot, which causes a shear deformation, which in turn causes a volume expansion, which sucks in some water from the surrounding soil.

Creep

The deformations of a soil often depend upon time, even under a constant load. This is called creep. Clay and peat exhibit this phenomenon. It causes structures founded on soft soils to show ever increasing settlements. A new road, built on a soft soil, will continue to settle for many years. For buildings such settlements are particular damaging when they are not uniform, as this may lead to cracks in the building.

Groundwater

A special characteristic of soil is that water may be present in the pores of the soil. This water contributes to the stress transfer in the soil. It may also be flowing with respect to the granular particles, which creates friction stresses between the fluid and the solid material. In many cases soil must be considered as a two phase material. As it takes some time before water can be expelled from a soil mass, the presence of water usually prevents rapid volume changes.

Unknown initial stresses

Soil is a natural material, created in historical times by various geological processes. Therefore the initial state of stress is often not uniform, and often even partly unknown. Because of the non-linear behavior of the material, mentioned above, the initial stresses in the soil are of great importance for the determination of soil behavior under additional loads. These initial stresses depend upon geological history, which is never exactly known, and this causes considerable uncertainty. In particular, the initial horizontal stresses in a soil mass are usually unknown. The initial vertical stresses may be determined by the weight of the overlying layers. This means that the stresses increase with depth, and therefore stiffness and strength also increase with depth. The horizontal stresses, however, usually remain largely unknown.

Variability

The creation of soil by ancient geological processes also means that soil properties may be rather different on different locations. Even in two very close locations the soil properties may be completely different, for instance when an ancient river channel has been filled with sand deposits. Sometimes the course of an ancient river can be traced on the surface of a soil, but often it cannot be seen at the surface. When an embankment is built on such a soil, it can be expected that the settlements will vary, depending upon the local material in the subsoil. The variability of soil properties may also be the result of a heavy local load in the past.

Grain size

Soils are usually classified into various types. In many cases these various types also have different mechanical properties. A simple subdivision of soils is on the basis of the grain size of the particles that constitute the soil. Coarse granular material is often denoted as gravel and finer material as sand. In order to have a uniformly applicable terminology it has been agreed internationally to consider particles larger than 2 mm, but smaller than 63 mm as gravel. Larger particles are denoted as stones. Sand is the material consisting of particles smaller than 2 mm, but larger than 0.063 mm. Particles smaller than 0.063 mm and larger than 0.002 mm is denoted as silt. Soil consisting of even smaller particles, smaller than 0.002 mm, is denoted as clay , see Table 1.

Soil type	min.	max.
clay		0.002 mm
silt	0.002 mm	0.063 mm
sand	0.063 mm	2 mm
gravel	2 mm	63 mm

Table 1: Grain sizes

The grain size may be useful as a first distinguishing property of soils, but it is not very useful for the mechanical properties. The quantitative data that an engineer needs depend upon the mechanical properties such as stiffness and strength, and these must be determined from mechanical tests. Soils of the same grain size may have different mechanical properties. Sand consisting of round particles, for instance, can have a strength that is much smaller than sand consisting of particles with sharp points. Also, a soil sample consisting of a mixture of various grain sizes can have a very small permeability if the small particles just fit in the pores between the larger particles. The size of the particles in a certain soil can be represented graphically in a grain size diagram, see Figure 5. Such a diagram indicates the percentage of the particles smaller than a certain diameter, measured as a percentage of the mass (or weight). A steep slope of the curve in the diagram indicates a uniform soil; a shallow slope of the diagram indicates that the soil contains particles of strongly different grain sizes. For rather coarse particles, say larger than 0.05 mm, the grain size distribution can be determined by sieving.

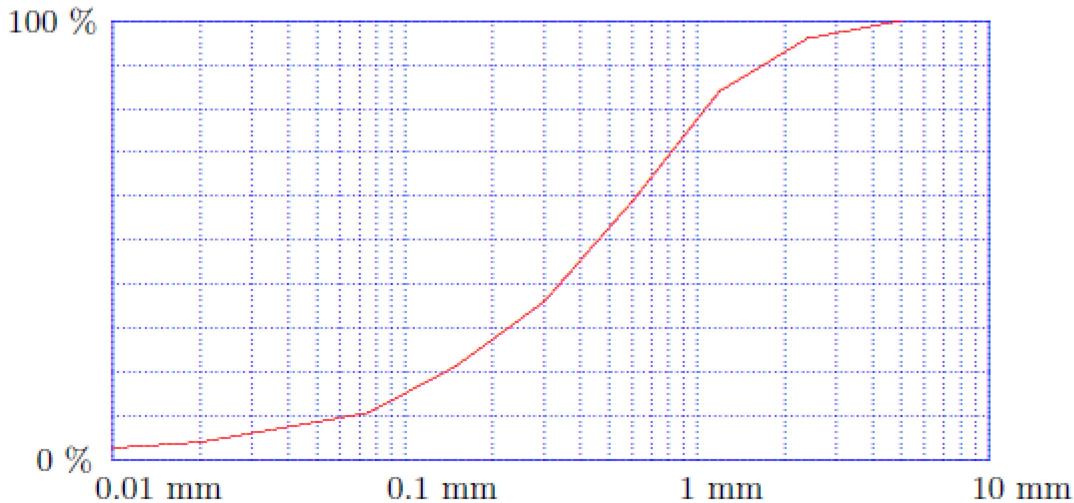


Figure 5: Grain size diagram.

Chemical composition

Besides the difference in grain size, the chemical composition of soil can also be helpful in distinguishing between various types of soils. Sand and gravel usually consist of the same minerals as the original rock from which they were created by the erosion process. This can be quartz, feldspar or glimmer. In Western Europe sand usually consists mainly of quartz. The chemical formula of this mineral is SiO2. Fine-grained soils may contain the same minerals, but they also contain the so-called clay minerals, which have been created by chemical erosion.

Consistency limits

For very fine soils, such as silt and clay, the consistency is an important property. It determines whether the soil can easily be handled, by soil moving equipment, or by hand. The consistency is often very much dependent on the amount of water in the soil. This is expressed by the water content w . It is defined as the weight of the water per unit weight of solid material,

$$w = W_w / W_k \quad \ldots\ldots\; 1$$

When the water content is very low (as in very dry clay) the soil can be very stiff, almost like a stone. It is then said to be in the solid state. Adding water, for instance if the clay is flooded by rain, may make the clay plastic, and for higher water contents the clay may even become almost liquid. In order to distinguish between these states (solid, plastic and liquid) two standard tests have been agreed upon, that indicate the consistency limits.

Porosity

An important basic parameter is the porosity n, defined as the ratio of the volume of the pore space and the total volume of the soil,

$$n = V_p / V_t \quad \ldots\ldots\ldots 2$$

For most soils the porosity is a number between 0.30 and 0.45 (or, as it is usually expressed as a percentage, between 30 % and 45 %). When the porosity is small the soil is called densely packed, when the porosity is large it is loosely packed.
It may be interesting to calculate the porosities for two particular cases. The first case is a very loose packing of spherical particles, in which the contacts between the spheres occur in three mutually orthogonal directions only. This is called a cubic array of particles, see Figure 6. If the diameter of the spheres is D, each sphere occupies a volume $\pi D^3/6$ in space. The ratio of the volume of the solids to the total volume then is $V_p/V_t = \pi/6 = 0.5236$, and the porosity of this assembly thus is n = 0.4764. This is the loosest packing of spherical particles that seems possible. Of course, it is not stable: any small disturbance will make the assembly collapse.

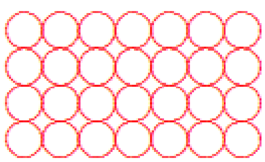


Figure 6: Cubic array

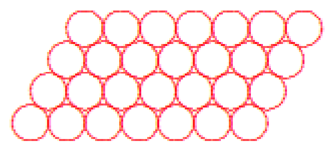


Figure 7: Densest array

Degree of saturation

The pores of a soil may contain water and air. To describe the ratio of these two the degree of saturation **S** is introduced as

$$S = V_w/V_p \quad \ldots\ldots\ldots 3$$

Here $\mathbf{V_w}$ is the volume of the water, and $\mathbf{V_p}$ is the total volume of the pore space. The volume of air (or any other gas) per unit pore space then is 1 − S. If S = 1 the soil is completely saturated, if S = 0 the soil is perfectly dry.

Density

For the description of the density and the volumetric weight of a soil, the densities of the various components are needed. The density of a substance is the mass per unit volume of that substance. For water this is denoted by ρ_ω, and its value is about 1000 kg/m^3. Small deviations from this value may occur due to temperature differences or variations in salt content. In soil mechanics these are often of minor importance, and it is often considered accurate enough to assume that

$$\rho_\omega = 1000\ kg/m^3 \qquad 4$$

For the analysis of soil mechanics problems the density of air can usually be disregarded.

The density of the solid particles depends upon the actual composition of the solid material. In many cases, especially for quartz sands, its value is about

$$\rho_{\omega} = 2650\ kg/m^3 \qquad 5$$

Stresses in Soils

As in other materials, stresses may act in soils as a result of an external load and the volumetric weight of the material itself. Soils, however, have a number of properties that distinguish it from other materials. Firstly, a special property is that soils can only transfer compressive normal stresses, and no tensile stresses. Secondly, shear stresses can only be transmitted if they are relatively small compared to the normal stresses. Furthermore it is characteristic of soils that part of the stresses is transferred by the water in the pores.

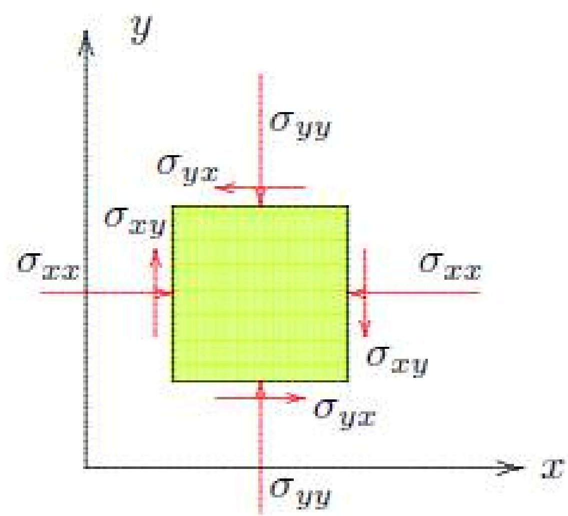


Figure 8: Stresses

Because the normal stresses in soils usually are compressive stresses only, it is standard practice to use a sign convention for the stresses that is just opposite to the sign convention of classical continuum mechanics, namely such that compressive stresses are considered positive, and tensile stresses are negative. The stress tensor will be denoted by σ.

The sign convention for the stress components is illustrated in Figure 8. Its formal definition is that a stress component is positive when it acts in positive coordinate direction on a plane with its outward normal in negative coordinate direction, or when it acts in negative direction on a plane with its outward normal in positive direction. This means that the sign of all stress components is just opposite to the sign that they would have in most books on continuum mechanics or applied mechanics.

Stresses in a Layer

Vertical stresses

In many places on earth the soil consists of practically horizontal layers. If such a soil does not carry a local surface load, and if the groundwater is at rest, the vertical stresses can be determined directly from a consideration of vertical equilibrium.

A simple case is homogeneous layers, completely saturated with water, see Figure 9. The pressure in the water is determined by the location of the phreatic surface. This is defined as the plane where the pressure in the groundwater is equal to the atmospheric pressure.

If the atmospheric pressure is taken as the zero level of pressures, as is usual, it follows that p = 0 at the phreatic surface. If there are no capillary effects in the soil, this is also the upper boundary of the water, which is denoted as the groundwater table. In the example it is assumed that the phreatic surface coincides with the soil surface, see Figure 9. The volumetric weight of the saturated soil is supposed to be = 20 kN/m^3. The vertical normal stress in the soil now increases linearly with depth,

$$\sigma_{zz} = \gamma d \qquad 6$$

This is a consequence of vertical equilibrium of a column of soil of height d. It has been assumed that there are no shear stresses on the vertical planes bounding the column in horizontal direction.

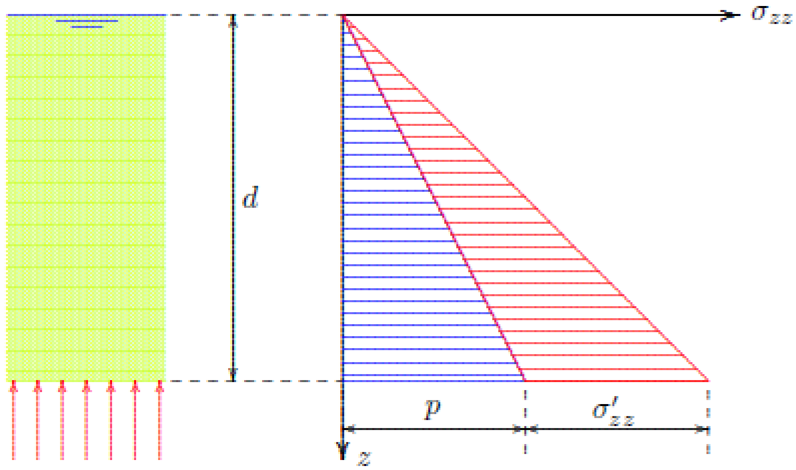


Figure 9: Stresses in a homogeneous layer

That seems to be a reasonable assumption if the terrain is homogeneous and very large, with a single geological history. Often this is assumed, even when there are no data.

At a depth of 10 m, for instance, the vertical total stress is 200 kN/m^2 = 200 kPa. Because the groundwater is at rest, the pressures in the water will be hydrostatic. The soil can be considered to be a container of water of very complex shape, bounded by all the particles, but that is irrelevant for the actual pressure in the water. This means that the pressure in the water at a depth d will be equal to the weight of the water in a column of unit area, see also Figure 9,

$$p = \gamma_w d \qquad 7$$

Where w is the volumetric weight of water, usually w = 10 kN/m3. It now follows that a depth of 10 m the effective stress is 200 kPa- 100 kPa=100 kPa.

Formally, the distribution of the effective stress can be found from the basic equation

$$\sigma'_{zz} = \sigma_{zz} - p \qquad \text{or, with 6 and 7}$$

$$\sigma'_{zz} = (\gamma - \gamma_w)d \qquad 8$$

The vertical effective stresses appear to be linear with depth. That is a consequence of the linear distribution of the total stresses and the pore pressures, with both of them being zero at the same level, the soil surface. It should be noted that the vertical stress components, both the total stress and the pore pressures, with both of them being zero at the same level, the soil surface.

Pore pressures

Soil is a porous material, consisting of particles that together constitute the grain skeleton. In the pores of the grain skeleton a fluid may be present: usually water. The pore structure of all normal soils is such that the pores are mutually connected. The water fills a space of very complex form, but it constitutes a single continuous body. In this water body a pressure may be transmitted, and the water may also flow through the pores. The pressure in the pore water is denoted as the pore pressure.

Residual and Transported Soils

Soils which are formed by weathering of rocks may remain in position at the place of region. In that case these are 'Residual Soils'. These may get transported from the place of origin by various agencies such as wind, water, ice, gravity, etc. In this case these are termed ''Transported soil''. Residual soils differ very much from transported soils in their characteristics and engineering behaviour. The degree of disintegration may vary greatly throughout a residual soil mass and hence, only a gradual transition into rock is to be expected. An important characteristic of these soils is that the sizes of grains are not definite because of the partially disintegrated condition. The grains may break into smaller grains with the application of a little pressure.

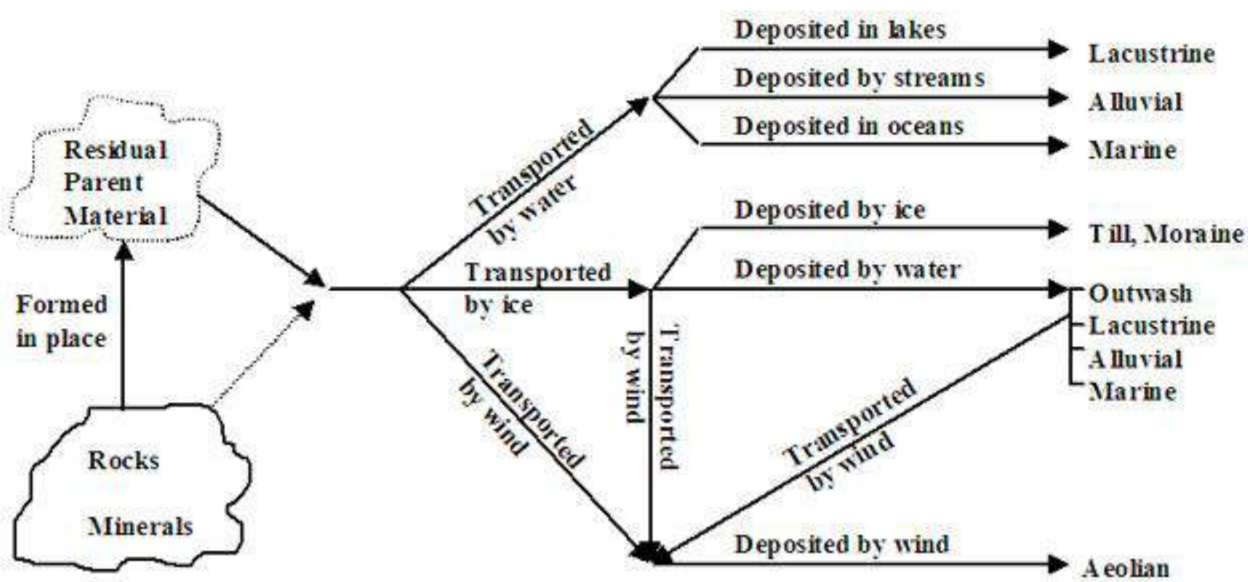


Fig. 10 Diagrammatic representation of sedentary and transported soils

[http://www.depi.vic.gov.au/agriculture-and-food/dairy/pastures-management/fertilising-dairy-pastures/how-do-the-properties-of-soils-affect-plant-growth]

The residual soil profile may be divided into three zones: (i) the upper zone in which there is a high degree of weathering and removal of material; (ii) the intermediate zone in which there is some degree of weathering in the top portion and some deposition in the bottom portion; and (iii) the partially weathered zone where there is the transition from the weathered material to the unweathered parent rock. Residual soils tend to be more abundant in humid and warm zones where conditions are favourable to chemical weathering of rocks and have sufficient vegetation to keep the products of weathering from being easily transported as sediments. Residual soils have not received much attention from geotechnical engineers because these are located primarily in undeveloped areas.

Transported soils may be further subdivided, depending upon the transporting agency and the place of deposition, as under:

- Alluvial soils. Soils transported by rivers and streams: Sedimentary clays.
- Aeolian soils. Soils transported by wind: loess.
- Glacial soils. Soils transported by glaciers: Glacial till.
- Lacustrine soils. Soils deposited in lake beds: Lacustrine silts and lacustrine clays.
- Marine soils. Soils deposited in sea beds: Marine silts and marine clays.

Broad classification of soils may be:

1. Coarse-grained soils, with average grain-size greater than 0.075 mm, e.g., gravels and sands.
2. Fine-grained soils, with average grain-size less than 0.075 mm, e.g., silts and clays.

These exhibit different properties and behaviour but certain general conclusions are possible even with this categorisation. For example, fine-grained soils exhibit the property of 'cohesion'—bonding caused by inter-molecular attraction while coarse-grained soils do not; thus, the former may be said to be cohesive and the latter non-cohesive or cohesionless.

Structure of Soils

The 'structure' of a soil may be defined as the manner of arrangement and state of aggregation of soil grains. In a broader sense, consideration of mineralogical composition, electrical properties, orientation and shape of soil grains, nature and properties of soil water and the interaction of soil water and soil grains, also may be included in the study of soil structure, which is typical for transported or sediments soils. Structural composition of sedimented soils influences, many of their important engineering properties such as permeability, compressibility and shear strength. Hence, a study of the structure of soils is important.

The following types of structure are commonly studied:

- Single-grained structure
- Honey-comb structure
- Flocculent structure

Soil Testing - In-situ Sampling and Preparation

In-situ Sampling and Preparation

The correct sampling, description and preparation of soil and soil mixtures is necessary if subsequent tests are to be meaningful and provide representative results.
Various national and international Standards specify a range of procedures and equipment necessary to ensure representative sampling.
With the use of simple hand tools, it is often possible to obtain detailed information regarding the sub-surface structure and hence the likely engineering characteristics of the area under investigation.

Soil Colour Charts

A standard identification of colour is an essential component of a soil-profile description. Soil colour charts are widely used by civil engineers, agronomists, soil scientists, geologists and archaeologists as a means of providing a standard colour reference.

Sample Mixers

A regular laboratory requirement is the mixing of samples with water and/or other constituents to provide a homogeneous mixture prior to subsequent testing. The following ranges of mixers provide an efficient means of mixing samples.

Hand Boring and Sampling

The items listed provide the engineer with an economic range of equipment for field survey work. Using this equipment it is possible to obtain disturbed or undisturbed samples at reasonable depths, subject to ground conditions. Most items may be inter-connected.

Soil and Gravel Auger Heads

These auger heads are suitable for boring in cohesive soils or sands and gravels. The soil augers are constructed of heavy duty steel plates forming an open tube partly interlocked at the cutting end. Gravel augers comprise a one piece steel casting with a spiral point and two plates designed to close when lifting samples from the borehole. The Dutch Auger is of similar construction to the Soil Augers and is particularly useful in very fine silt-clay sands.

Large Sample Splitter

This splitter is designed for the reduction of test samples which are too large in volume to be conveniently handled. It divides samples so that half is representative of the original total sample and handles material up to 6 inches in particle size.
The lever-actuated unit is constructed of heavy gauge welded steel with a hopper which holds up to 1 ft^3. The single splitter chute provides wide flexibility in sizes of opening and adjustment is provided for chutes of 0.5, 1.5, 2, 3, 4 or 6 inch by positioning of the chute bars. Overall height approximately is 1 metre. Hopper size 735 mm long x 480 mm wide (approx.).

Sample Reduction

The reduction of particles within the soil mass is necessary for a number of tests. For most purposes crushing of individual particles must be avoided. This reduction process is best achieved using a porcelain mortar and rubber headed pestle.

Moisture Content

The new range of Speedy Moisture testers now includes an electronic balance and a heavy duty plastic case. Designed for the most demanding on-site conditions, the new waterproof and durable case offers high levels of protection. The new model comprises: Speedy Moisture tester, electronic balance, beaker, cleaning cloth, cap, washer, scoop, steel pulverizing balls, and cleaning brushes.

Used to weigh a sample before placing it in the Speedy
Moisture Tester, the portable battery powered balance includes LCD display with a measuring range 0 – 200 g x 0.1 g. The % moisture content of the sample is read directly from the calibrated pressure gauge.

Soil Index Properties

Soil index properties are used extensively by engineers to discriminate between the different kinds of soil within a broad category, e.g. clay will exhibit a wide range of engineering properties depending upon its composition.
Classification tests to determine index properties will provide engineers with valuable information when the results are compared against empirical data relative to the index properties determined.

Determination of Liquid Limit

The condition of a soil can be altered by changing the moisture content. The liquid limit is the empirically established moisture content at which a soil passes from the plastic to the liquid state. Knowledge of the liquid limit allows the engineer to correlate several engineering properties with the soil. Two main types of test are used. The Casagrande's type (Cup), which has been used for many years, and the cone penetrometer method, which is now the definitive method specified in BS 1377.

Casagrande's Method

- Satisfies International Standards
- Motorized version with integral blow counter available

Particular design features of the instrument include a positive action horizontal lead screw, which is rapidly adjustable and rigidly fixes the height of cup in relation to the base during the test procedure. The cam mechanism and cup suspension assembly have been designed to withstand constant use with minimum readjustment.


Liquid Limit Device with accessories

Cone Penetrometer Method

The method is fundamentally more satisfactory than the Casagrande's method as it is essentially a static test depending on the soil shear strength. The test is based on the relationship between moisture content and the penetration of a cone into the soil sample under controlled conditions.


Penetration Test Cone and Test Gauge


EL24-0545/01 Semi-Automatic Cone Penetrometer

Determination of Plastic Limit

The plastic limit is defined as the lowest moisture content of a soil that will permit a sample to be rolled into threads of 3 mm diameter without the threads breaking.

The test procedure has remained, in principle, the same since 1932, when Casagrande's proposed to define the various limits by relating the moisture content characteristics of soil under certain conditions. The apparatus required is simple yet effective.
The majority of the apparatus required for this test is standard laboratory equipment. For full details see the Laboratory equipment section of the catalogue

Determination of Shrinkage Characteristics

When the water content of a fine-grained soil is reduced below the plastic limit, shrinkage of the soil mass continues until the shrinkage limit is reached.
Shrinkage can be significant in clays but less so in silts and sands.
The equipment listed below enables the engineer to determine a number of important parameters, including shrinkage ratio, volumetric shrinkage and linear shrinkage.

Determination of Density, Particle Density and Specific Gravity

The term density refers to mass per unit volume. The density of a mass of soil is of interest to the engineer for a variety of reasons including the design of earthworks and foundations and in slope stability analysis.
Particle density or specific gravity is a measure of the actual particles which make up the soil mass and is defined as the ratio of the mass of the particles to the mass of the water they displace.
Knowledge of the particle density is essential in relation to other soil tests. It is used when calculating porosity and voids ratio and is particularly important when compaction and consolidation properties are being investigated. The majority of apparatus used for the various tests is general laboratory equipment.

Particle Size Distribution and Sand Equivalent Value

The analysis of soils by particle size provides a useful engineering classification system from which a considerable amount of empirical data can be obtained.
Two separate and different procedures are used.
Sieving is used for gravel and sand size particles and sedimentation procedures are used for the finer soils. For soil containing a range of coarse and fine particles it is usual to employ a composite test of sieving and sedimentation procedures.
The Sand Equivalent Test serves as a rapid field test to show the relative proportions of clay-like or plastic fines and dusts in granular soils and fine aggregates.

Constant Temperature Bath

Specially designed for the sedimentation testing of soils and other fine grained material, the bath is supplied with a false bottom to assist in circulation of the bath liquid. Will accommodate six Sedimentation Cylinders.

Specification	
Dimensions (l x w x h)	535 x 210 x 610 mm external
Capacity	Holds up to 6 sedimentation cylinders
Construction	Stainless steel with toughened glass front
Temperature control	Heater/Thermostat/Circulation with Digital Controller Unit
Power	1500 Watts
Weight	12 kg

EL24-4865/01 Constant Temperature Bath

Sedimentation by the Hydrometer Method

This method determines particle size distribution in a soil from the coarse sand size down to clay size (about 2 µm). The test does not require the weighing accuracy necessary for pipette sedimentation and is suitable for use in site laboratories.

Automatic Compaction of Soils

The time and effort required preparing specimens for compaction studies and other test methods can often be costly and time-consuming. The use of an automatic, mechanical compactor will show considerable cost benefits over hand compaction methods. Two models meeting the requirements of BS/EN and ASTM are available.

Automatic Compactor

- Pre-set blow pattern ensures even compaction
- Solid state controls for reliability and ease of maintenance
- Automatic re-setting of counter after completion of blow pattern

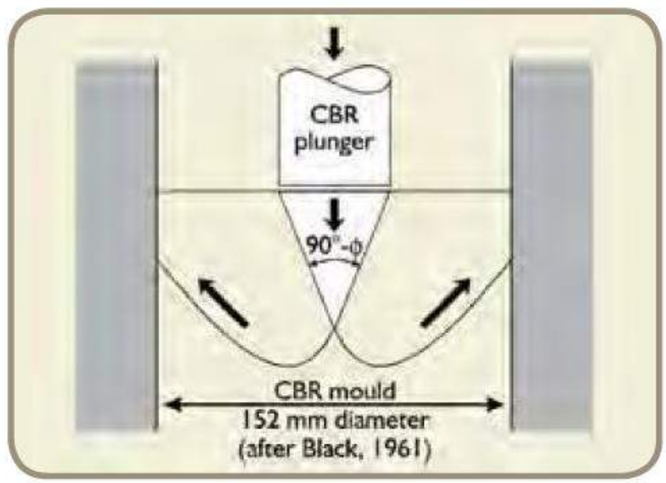
These machines automatically compact specimens eliminating the laborious hand compaction method. The height and weight of the rammer are adjustable to suit test requirements.
An automatic blow pattern ensures optimum compaction for each layer of soil. The rammer travels across the mould and the table rotates the mould in equal steps on a base that is extremely stable. The number of blows per layer can be set at the beginning of the test.

EL24 9090 series Automatic Soil Compactor with accessory mould

California Bearing Ratio

The California Bearing Ratio test, or CBR test as it is usually termed, is an empirical test first developed in California, USA, for estimating the bearing value of highway sub-bases and sub-grades. The test follows a standardized procedure and there is little difference between BS/EN and ASTM tests. However, there are numerous ways of preparing samples and in this respect American practice differs in detail from British practice.
This test can be performed in the laboratory on prepared samples or on location. It is important to appreciate that this test, being of an empirical nature, is valid only for the application for which it was developed, i.e. the design of highway base thicknesses.

EL24-9160 series CBR-Test 50 with accessories



Laboratory Test

1. Water Content Determination

Purpose:

This test is performed to determine the water (moisture) content of soils. The water content is the ratio, expressed as a percentage, of the mass of "pore" or "free" water in a given mass of soil to the mass of the dry soil solids.

Standard Reference:

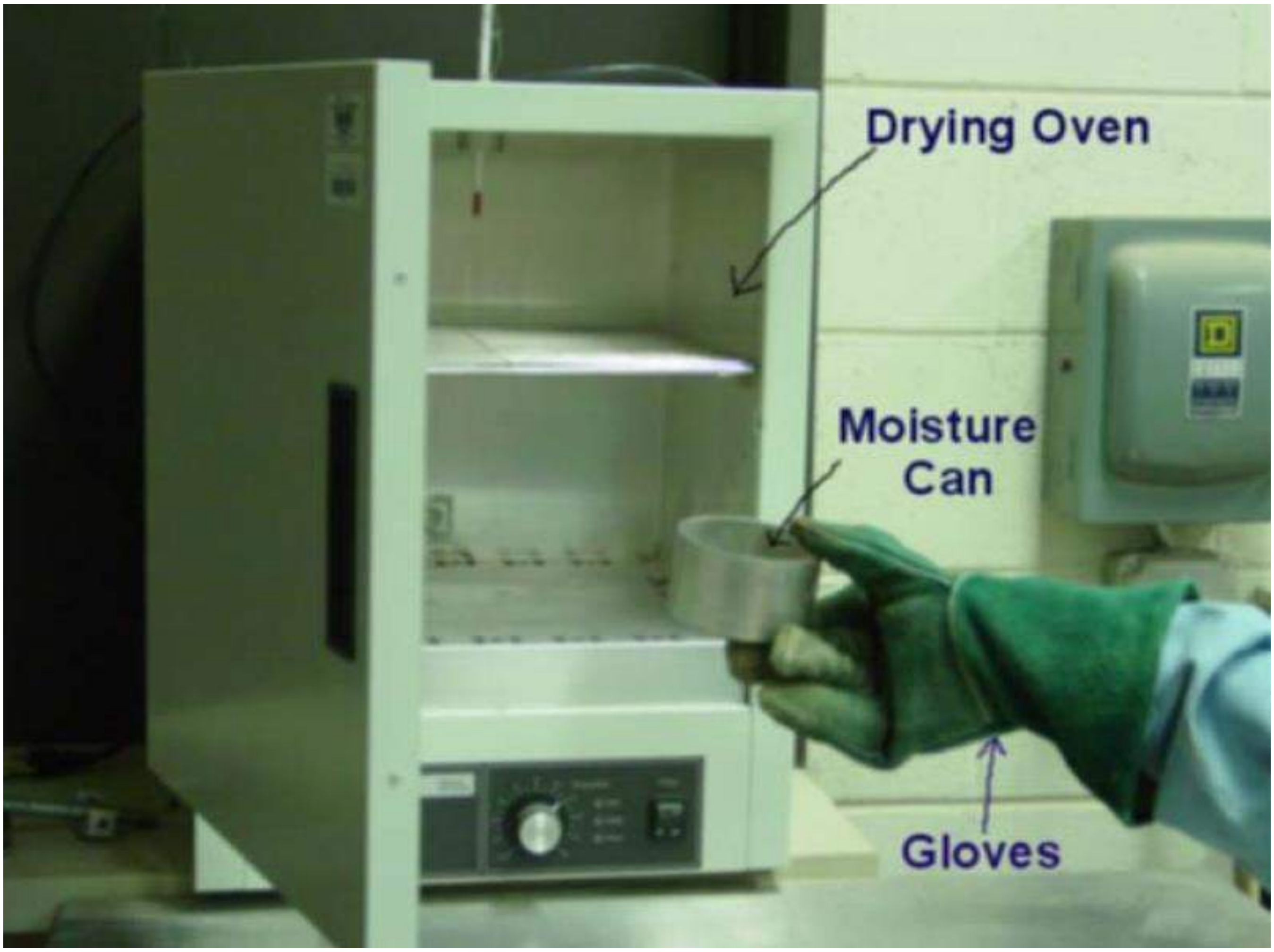
ASTM D 2216 - Standard Test Method for Laboratory Determination of Water (Moisture) Content of Soil, Rock, and Soil-Aggregate Mixtures

Significance:

For many soils, the water content may be an extremely important index used for establishing the relationship between the way a soil behaves and its properties. The consistency of a fine-grained soil largely depends on its water content. The water content is also used in expressing the phase relationships of air, water, and solids in a given volume of soil.

Equipment:

Drying oven, Balance, Moisture can, Gloves, Spatula.



Test Procedure:

1. Record the moisture can and lid number. Determine and record the mass of empty, clean, and dry moisture can with its lid (M_C)
2. Place the moist soil in the moisture can and secure the lid. Determine and record the mass of the moisture can (now containing the moist soil) with the lid (M_{CMS}).
3. Remove the lid and place the moisture can (containing the moist soil) in the drying oven that is set at 105 °C. Leave it in the oven overnight.
4. Remove the moisture can. Carefully but securely, replace the lid on the moisture can using gloves, and allow it to cool to room temperature. Determine and record the mass of the moisture can and lid (containing the dry soil) (M_{CDS}).
5. Empty the moisture can and cleans the can and lid.

Data Analysis:

1. Determine the mass of soil solids.

$$M_S = M_{CDS} - M_C$$

2. Determine the mass of pore water.

$$M_W = M_{CMS} - M_{CDS}$$

3. Determine the water content.

$$w = \frac{Mw}{Ms} x100$$

Example Data

Date Tested: August 30, 2002
Tested By: CEMM315 Class, Group A
Project Name: CEMM315 Lab
Sample Number: B-1,AU-1, 0'-2'
Sample Description: Gray silty clay

Specimen number	1	2
Moisture can and lid number	12	15
M_C = Mass of empty, clean can + lid (grams)	7.78	7.83
M_{CMS} = Mass of can, lid, and moist soil (grams)	16.39	13.43
M_{CDS} = Mass of can, lid, and dry soil (grams)	15.28	12.69
M_S = Mass of soil solids (grams)	7.5	4.86
M_W = Mass of pore water (grams)	1.11	0.74
w = Water content, w%	14.8	15.2

Example Calculation:

$$M_C = 7.78g,\ M_{CMS} = 16.39g,\ M_{CDS} = 15.28g$$

$$M_S = 15.28 - 7.78 = 7.5g$$

$$M_W = 16.39 - 15.28 = 1.11g$$

$$w = \frac{1.11}{7.5} x100 = 14.8\%$$

2. Organic Matter Determination

Purpose:

This test is performed to determine the organic content of soils. The organic content is the ratio, expressed as a percentage, of the mass of organic matter in a given mass of soil to the mass of the dry soil solids.

Standard Reference:

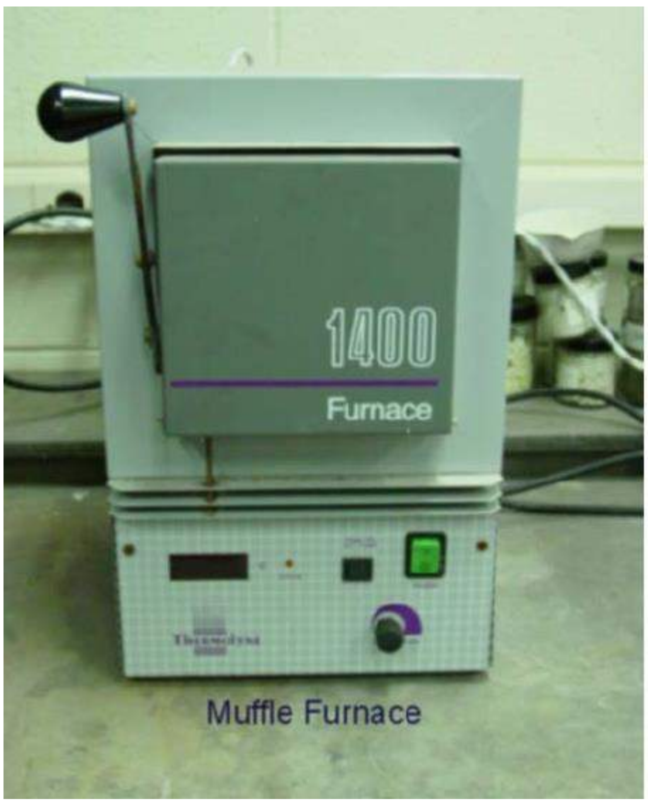
ASTM D 2974 – Standard Test Methods for Moisture, Ash, and Organic Matter of Peat and Organic Soils

Significance:

Organic matter influences many of the physical, chemical and biological properties of soils. Some of the properties influenced by organic matter include soil structure, soil compressibility and shear strength. In addition, it also affects the water holding capacity, nutrient contributions, biological activity, and water and air infiltration rates.

Equipment:

Muffle furnace, Balance, Porcelain dish, Spatula, Tongs



Test Procedure:

1. Determine and record the mass of an empty, clean, and dry porcelain dish (M_P).
2. Place a part of or the entire oven-dried test specimen from the moisture content experiment (Expt.1) in the porcelain dish and determine and record the mass of the dish and soil specimen (M_{PDS}).
3. Place the dish in a muffle furnace. Gradually increase the temperature in the furnace to 440oC. Leave the specimen in the furnace overnight.
4. Remove carefully the porcelain dish using the tongs (the dish is very hot), and allow it to cool to room temperature. Determine and record the mass of the dish containing the ash (burned soil) (MPA).
5. Empty the dish and clean it.

Data Analysis:

1. Determine the mass of the dry soil.
$$M_D=M_{PDS}-M_P$$
2. Determine the mass of the ashes (burned) soil.
$$M_A=M_{PA}-M_P$$
3. Determine the mass of organic matter
$$M_O = M_D - M_A$$
4. Determine the organic matter (content).
$$OM = \frac{M_O}{M_D} x100$$

Example Data

Date Tested: August 30, 2002

Tested By: CEMM315 Class, Group A

Project Name: CEMM315 Lab

Sample Number: B-1,AU-1, 0'-2'

Sample Description: Gray silty clay

Specimen number	1	2
Porcelain dish number	5	8
M_P = Mass of empty, clean porcelain dish (grams)	23.20	23.03
M_{PDS} = Mass of dish and dry soil (grams)	35.29	36.66
M_{PA} = Mass of the dish and ash (Burned soil) (grams)	34.06	35.27
M_D = Mass of the dry soil (grams)	12.09	13.63
M_A = Mass of the ash (Burned soil) (grams)	10.86	12.24
M_O = Mass of organic matter (grams)	1.23	1.39
OM = Organic matter, %	10.17	10.20

Example Calculation:

$$M_P = 23.2g,\ M_{PDS} = 35.29g,\ M_{PA} = 34.06g$$

$$M_D = 35.29 - 23.20 = 12.09g$$

$$M_A = 34.06 - 23.20 = 10.86g$$

$$M_O = 12.09 - 10.86 = 1.39g$$

$$OM = \frac{1.39}{12.09} x100 \quad = 10.17\%$$

3. *Density (Unit Weight) Determination*

Purpose:

This lab is performed to determine the in-place density of undisturbed soil obtained by pushing or drilling a thin-walled cylinder. The bulk density is the ratio of mass of moist soil to the volume of the soil sample, and the dry density is the ratio of the mass of the dry soil to the volume the soil sample.

Standard Reference:

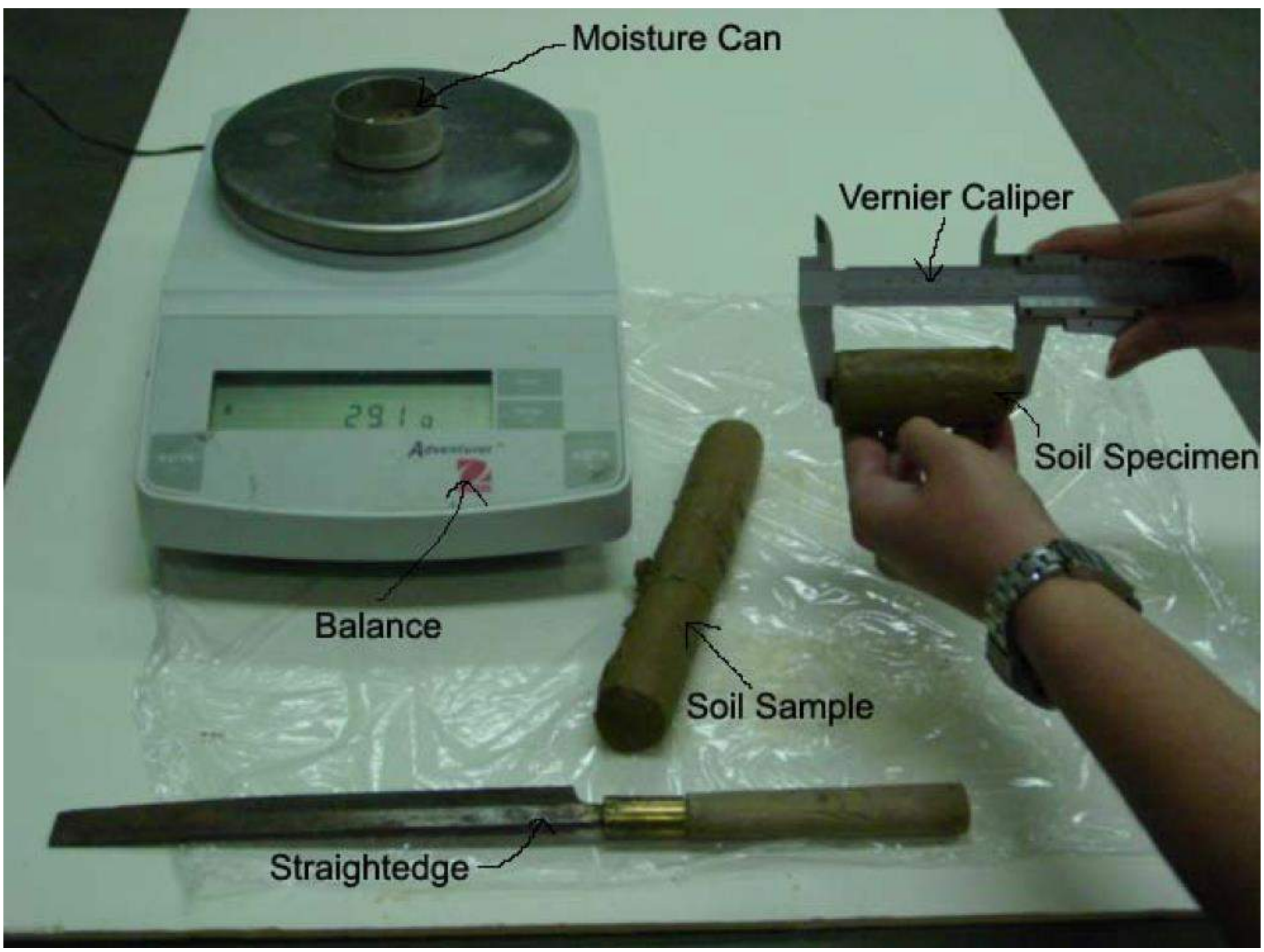
ASTM D 2937-00 – Standard Test for Density of Soil in Place by the Drive- Cylinder Method

Significance:

This test is used to determine the in-place density of soils. This test can also be used to determine density of compacted soils used in the construction of structural fills, highway embankments, or earth dams. This method is not recommended for organic or friable soils.

Equipment:

Straightedge, Balance, Moisture can, Drying oven, Vernier caliper



Test Procedure:

1. Extrude the soil sample from the cylinder using the extruder.
2. Cut a representative soil specimen from the extruded sample.
3. Determine and record the length (L), diameter (D) and mass (M_t) of the soil specimen.
4. Determine and record the moisture content of the soil (w). (See Experiment 1)
5. (Note: If the soil is sandy or loose, weigh the cylinder and soil sample together. Measure dimensions of the soil sample within the cylinder. Extrude and weigh the soil sample and determine moisture content)

Data Analysis:

Determine the moisture content as in Experiment 1

Determine the volume of the soil sample

$$V = \frac{\pi D^2 L}{4} \quad cm^3$$

Calculate bulk density (ρ_t) of soil

$$\rho_t = \frac{M_t}{V} \quad \frac{g}{cm^3} \qquad \text{or unit weight } \gamma_t = \rho_t g$$

Calculate dry density (ρ_d) of soil

$$\rho_d = \frac{\rho_t}{1+w} \quad \frac{g}{cm^3} \qquad \text{or dry unit weight } \gamma_d = \rho_d g$$

Example Data

Sample number: B-1, ST-1, 10'-12' Date Tested: September 10, 2002

Soil description: Gray silty clay

Mass of the soil sample (M_t): 125.20 grams

Length of the soil sample (L): 7.26 cm

Diameter of the soil sample (D): 3.41 cm

Moisture content determination:

Specimen number	1
Moisture can and lid number	15
M_C = Mass of empty, clean can + lid (grams)	7.83
M_{CMS} = Mass of can, lid, and moist soil (grams)	13.43
M_{CDS} = Mass of can, lid, and dry soil (grams)	12.69
M_S = Mass of soil solids (grams)	4.86
M_W = Mass of pore water (grams)	0.74
w = Water content, w%	15.2

Example calculations: $w=15.2\%$, $M_t=125.2g$, $L=7.26cm$, $D=3.41cm$

$$V=\frac{\pi(3.41)^2(7.26)}{4}=66.28\ \ cm^3$$

$$\rho_t=\frac{125.20}{66.28}=1.89\ \ \frac{g}{cm^3}\ \ or\ \ \gamma_t=1.89\times 62.4=118\ \ \frac{lb}{ft^3}$$

$$\rho_d=\frac{1.89}{1+\left(\frac{15.20}{100}\right)}=1.64\ \ \frac{g}{cm^3}\ \ or\ \ \gamma_d=1.64\times 62.4=102.3\ \ \frac{lb}{ft^3}$$

(Note: 62.4 is the conversion factor to convert density in g/cm^3 to unit weight in lb/ft^3)

4. Specific Gravity Determination

Purpose:

This lab is performed to determine the specific gravity of soil by using a pycnometer. Specific gravity is the ratio of the mass of unit volume of soil at a stated temperature to the mass of the same volume of gas-free distilled water at a stated temperature.

Standard Reference:

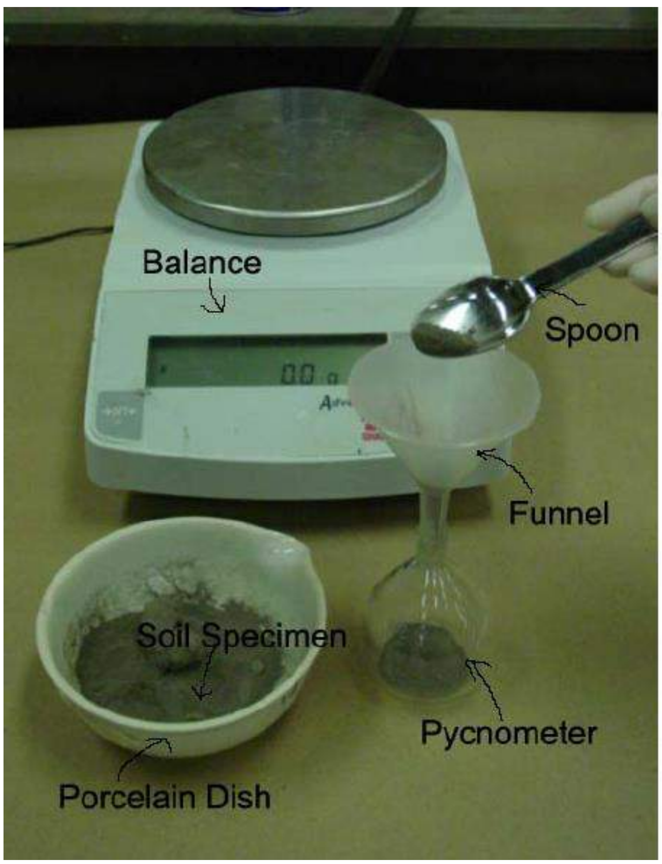
ASTM D 854-00 – Standard Test for Specific Gravity of Soil Solids by Water Pycnometer

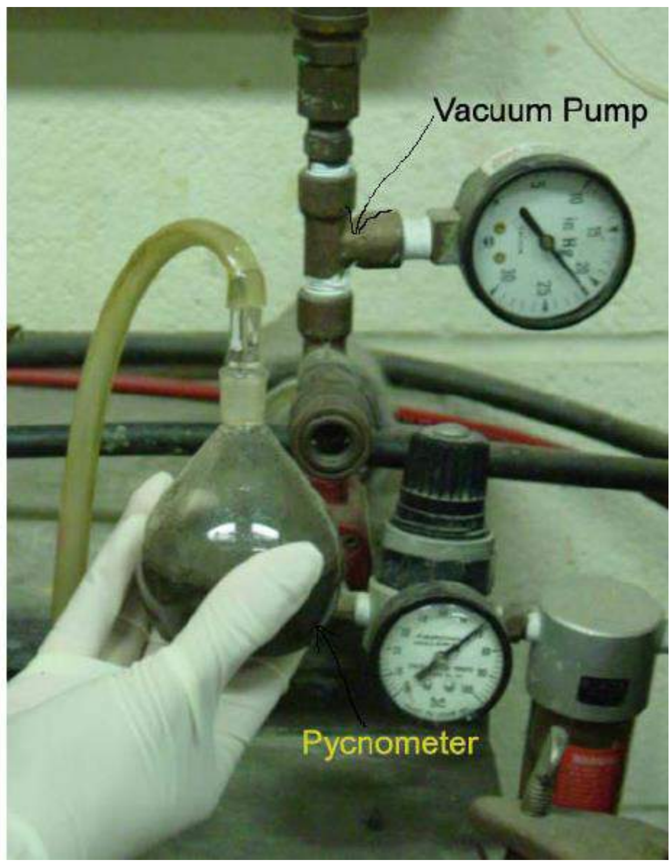
Significance:

The specific gravity of a soil is used in the phase relationship of air, water, and solids in a given volume of the soil.

Equipment:

Pycnometer, Balance, Vacuum pump, Funnel, Spoon





Test Procedure:

1. Determine and record the weight of the empty clean and dry pycnometer, W_P.
2. Place 10g of a dry soil sample (passed through the sieve No. 10) in the pycnometer. Determine and record the weight of the pycnometer containing the dry soil, W_{PS}.
3. Add distilled water to fill about half to three-fourth of the pycnometer. Soak the sample for 10 minutes.
4. Apply a partial vacuum to the contents for 10 minutes, to remove the entrapped air.
5. Stop the vacuum and carefully remove the vacuum line from pycnometer.
6. Fill the pycnometer with distilled (water to the mark), clean the exterior surface of the pycnometer with a clean, dry cloth. Determine the weight of the pycnometer and contents, W_B.
7. Empty the pycnometer and clean it. Then fill it with distilled water only (to the mark). Clean the exterior surface of the pycnometer with a clean, dry cloth. Determine the weight of the pycnometer and distilled water, W_A.
8. Empty the pycnometer and clean it.

Data Analysis:

Calculate the specific gravity of the soil solids using the following formula:

$$\text{Specific Gravity}, G_S = \frac{W_0}{W_0 + (W_A - W_B)}$$

Where:
W_0 = weight of sample of oven-dry soil, g = W_{PS} - W_P
W_A = weight of pycnometer filled with water
W_B = weight of pycnometer filled with water and soil

Example Data

Date Tested: September 10, 2002
Tested By: CEMM315 Class, Group A
Project Name: CEMM315 Lab
Sample Number: B-1, SS-1, 2'-3.5'
Sample Description: Gray silty clay

Specimen number	1	2
Pycnometer bottle number	96	37
W_P = Mass of empty, clean pycnometer (grams)	37.40	54.51
W_{PS} = Mass of empty pycnometer + dry soil (grams)	63.49	74.07
W_B = Mass of pycnometer + dry soil + water (grams)	153.61	165.76
W_A = Mass of pycnometer + water (grams)	137.37	153.70
Specific gravity (G_S)	2.65	2.61

Example Calculation: $W_P = 37.40\ g$, $W_{PS} = 63.49\ g$, $W_B = 153.61\ g$,

$$W_A = 137.37\ g$$

$$W_O = 63.49 - 37.40 = 26.09\ g$$

$$G_S = \frac{26.09}{26.09 + (137.37 - 153.61)} = 2.65$$

5. Relative Density Determination

Purpose:

This lab is performed to determine the relative density of cohesionless, free-draining soils using a vibrating table. The relative density of a soil is the ratio, expressed as a percentage, of the difference between the maximum index void ratio and the field void ratio of a cohesionless, free-draining soil; to the difference between its maximum and minimum index void ratios.

Standard References:

ASTM D 4254 – Standard Test Methods for Minimum Index Density and Unit Weight of Soils and Calculation of Relative Density
ASTM D 4253 – Standard Test Methods for Maximum Index Density and Unit Weight of Soils Using a Vibratory Table

Significance:

Relative density and percent compaction are commonly used for evaluating the state of compactness of a given soil mass. The engineering properties, such as shear strength, compressibility, and permeability, of a given soil depend on the level of compaction.

Equipment:

Vibrating Table, Mold Assembly consisting of standard mold, guide sleeves, surcharge base-plate, surcharge weights, surcharge base-plate handle, and dial-indicator gage, Balance, Scoop, Straightedge

Test Procedure:

1. Fill the mold with the soil (approximately 0.5 inch to 1 inch above the top of the mold) as loosely as possible by pouring the soil using a scoop or pouring device (funnel). Spiraling motion should be just sufficient to minimize particle segregation.
2. Trim off the excess soil level with the top by carefully trimming the soil surface with a straightedge.
3. Determine and record the mass of the mold and soil. Then empty the mold (M1).
4. Again fill the mold with soil (do not use the same soil used in step 1) and level the surface of the soil by using a scoop or pouring device (funnel) in order to minimize the soil segregation. The sides of the mold may be struck a few times using a metal bar or rubber hammer to settle the soil so that the surcharge base-plate can be easily placed into position and there is no surge of air from the mold when vibration is initiated.
5. Place the surcharge base plate on the surface of the soil and twist it slightly several times so that it is placed firmly and uniformly in contact with the surface of the soil. Remove the surcharge base-plate handle.
6. Attach the mold to the vibrating table.
7. Determine the initial dial reading by inserting the dial indicator gauge holder in each of the guide brackets with the dial gage stem in contact with the rim of the mold (at its center) on the both sides of the guide brackets. Obtain six sets of dial indicator readings, three on each side of each guide bracket. The average of these twelve readings is the initial dial gage reading, R_i. Record R_i to the nearest 0.001 in. (0.025 mm).
8. Firmly attach the guide sleeve to the mold and lower the appropriate surcharge weight onto the surcharge base-plate.
9. Vibrate the mold assembly and soil specimen for 8 min.
10. Determine and record the dial indicator gage readings as in step (7).
11. The average of these readings is the final dial gage reading, R_f.
12. Remove the surcharge base-plate from the mold and detach the mold from the vibrating table.
13. Determine and record the mass of the mold and soil (M_2)
14. Empty the mold and determine the weight of the mold.
15. Determine and record the dimensions of the mold (i.e., diameter and height) in order to calculate the calibrated volume of the mold, V_c. Also, determine the thickness of the surcharge base-plate, T_p.

Analysis:

1. Calculate the minimum index density (ρ_{dmin}) as follows:

$$\rho_{dmin} = \frac{M_{S1}}{V_C}$$

Where
M_{s1}= mass of tested-dry soil = Mass of mold with soil placed loose – mass of mold
V_c= Calibrated volume of the mold

2. Calculate the maximum index density (ρ_{dmax}) as follows:

$$\rho_{dmax} = \frac{M_{S2}}{V}$$

Where

M_{s2} = mass of tested-dry soil = Mass of mold with soil after vibration – Mass of mold

V = Volume of tested-dry soil = $V_c - (Ac*H)$

Where

Ac = the calibrated cross sectional area of the mold

H = $|R_f - R_i| + T_p$

3. Calculate the maximum and the minimum-index void ratios as follows (use Gs value determined from Experiment 4; ρ_w=1 g/cm^3):

$$e_{min} = \frac{\rho_W G_s}{\rho_{dmin}} - 1 \qquad e_{max} = \frac{\rho_W G_s}{\rho_{dmin}} - 1$$

4. Calculate the relative density as follows:

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

[Calculate the void ratio of the natural state of the soil based on ρ_d (Experiment 3) and $\rho s = G_S * \rho_w$ (G_s determined from Experiment 4) as follows:

$$e = \frac{\rho_s}{\rho_d} - 1]$$

Example Data

Date Tested: September 10, 2002

Tested By: CEMM315 Class, Group A

Project Name: CEMM315 Lab

Sample Number: B-1, ST-1, 2'-3.5'

Sample Description: Brown sand

Mass of empty mold:	9.878 Kg
Diameter of empty mold:	15.45 cm
Height of empty mold:	15.50 cm
Mass of mold and soil (M_1):	14.29 Kg
Average initial dial gauge reading (R_i):	0.88 inches
Average final dial gauge reading (R_f):	0.40 inches
Thickness of surcharge base plate (T_P):	0.123 inches
Mass of mold and soil (M_2):	14.38 Kg

Calculations:

$$M_{s1} = 14.29 - 9.878 = 4.412 \text{ kg} = 4412 \text{ g}, \quad M_{s2} = 14.38 - 9.878 = 4.502 \text{ kg} = 4502 \text{ g}$$

$$A_c = \frac{\pi(15.45)^2}{4} = 187.47 \text{ cm}^2, \quad H = (0.88 - 0.4 + 0.123) \times 2.54 = 1.53 \text{ cm}$$

$$V_c = \frac{\pi(15.45)^2 15.5}{4} = 2905.88 \text{ cm}^3, \quad V = 2905.88 - (187.47 \times 1.53) = 2618.75 \text{ cm}^3$$

$$\rho_{dmin} = \frac{4412}{2905.88} = 1.52 \frac{g}{cm^3}, \quad \rho_{dmax} = \frac{4502}{2618.75} = 1.72 \frac{g}{cm^3}$$

$$Gs = 2.65 (\text{Based on Experiment} - 4 \text{ conducted using the soil})$$

$$e_{min} = \frac{1 \times 2.65}{1.72} - 1 = 0.54, \quad e_{max} = \frac{1 \times 2.65}{1.52} - 1 = 0.74$$

$$\rho d = 1.65 \frac{g}{cm^3} (\text{Based on Experiment} - 3 \text{ conducted using this soil})$$

$$e = \frac{2.65}{1.65} - 1 = 0.61$$

$$D_d = \frac{0.74 - 0.61}{0.74 - 0.54} x100 = 65\%$$

6. Atterberg Limits

Purpose:

This lab is performed to determine the plastic and liquid limits of a fine grained soil. The liquid limit (LL) is arbitrarily defined as the water content, in percent, at which a part of soil in a standard cup and cut by a groove of standard dimensions will flow together at the base of the groove for a distance of 13 mm (1/2 in.) when subjected to 25 shocks from the cup being dropped 10 mm in a standard liquid limit apparatus operated at a rate of two shocks per second. The plastic limit (PL) is the water content, in percent, at which a soil can no longer be deformed by rolling into 3.2 mm (1/8 in.) diameter threads without crumbling.

Standard Reference:

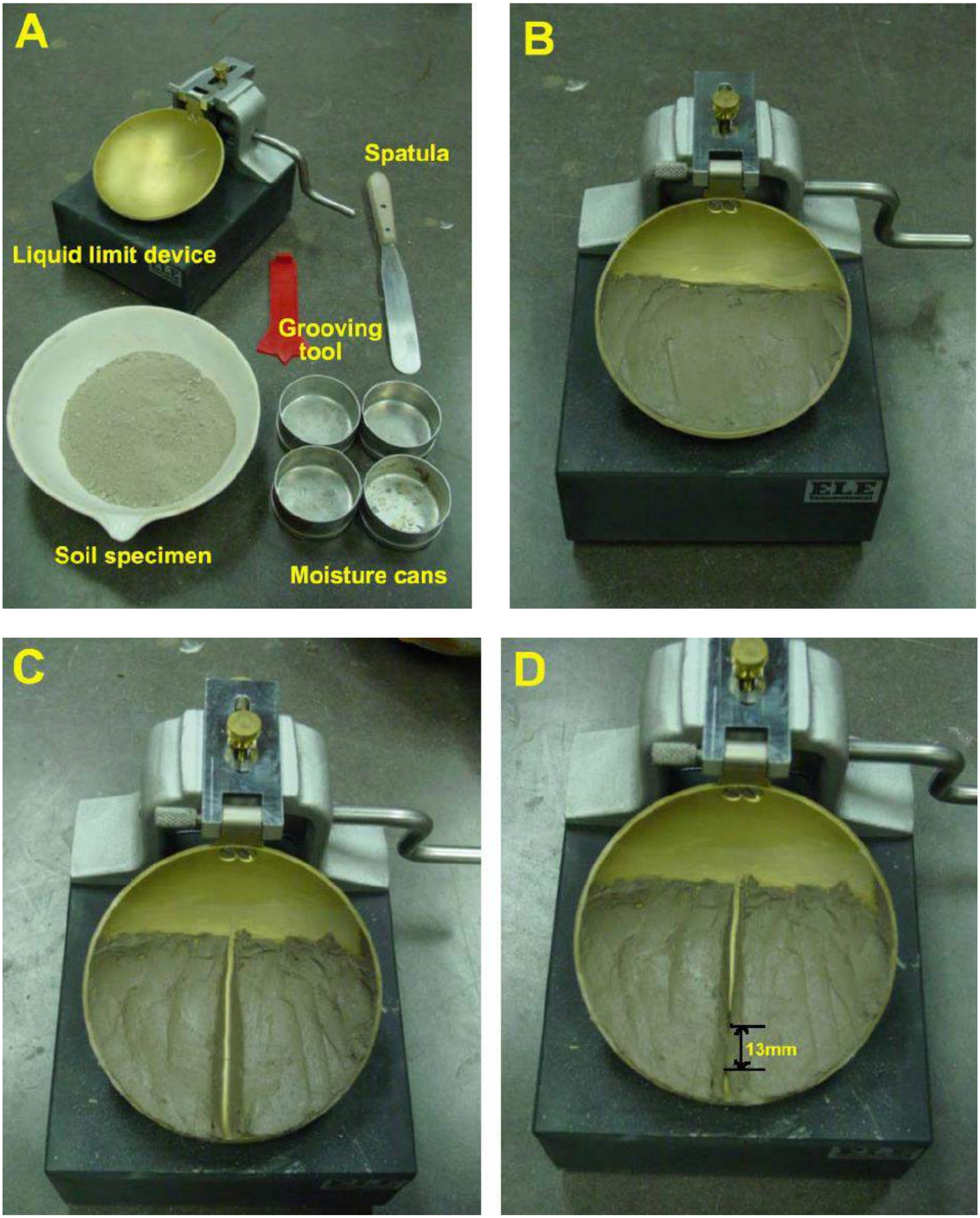
ASTM D 4318 - Standard Test Method for Liquid Limit, Plastic Limit, and Plasticity Index of Soils

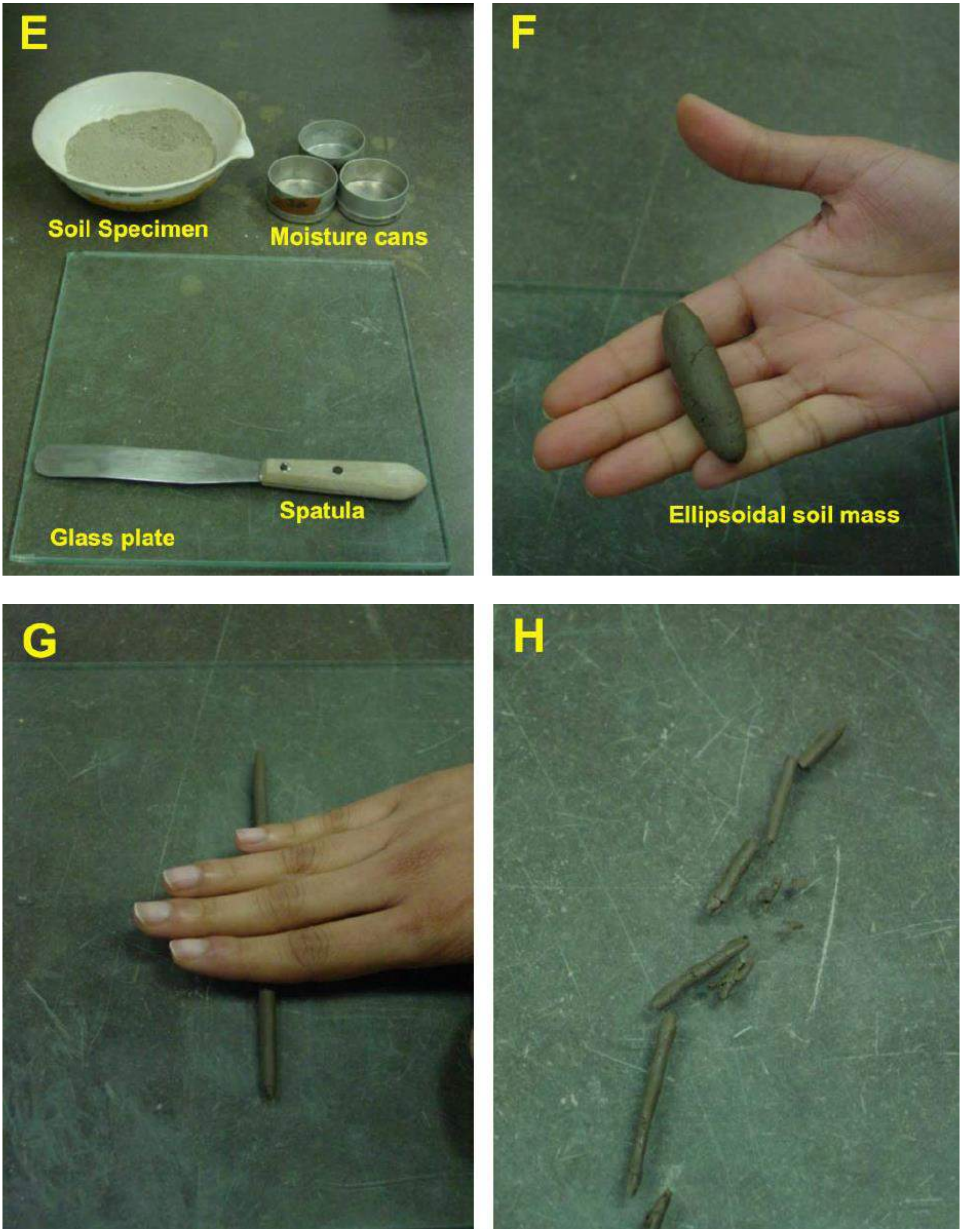
Significance:

The Swedish soil scientist Albert Atterberg originally defined seven “limits of consistency” to classify fine-grained soils, but in current engineering practice only two of the limits, the liquid and plastic limits, are commonly used. (A third limit, called the shrinkage limit, is used occasionally.) The Atterberg limits are based on the moisture content of the soil. The plastic limit is the moisture content that defines where the soil changes from a semi-solid to a plastic (flexible) state. The liquid limit is the moisture content that defines where the soil changes from a plastic to a viscous fluid state. The shrinkage limit is the moisture content that defines where the soil volume will not reduce further if the moisture content is reduced. A wide variety of soil engineering properties have been correlated to the liquid and plastic limits, and these Atterberg limits are also used to classify a fine-grained soil according to the Unified Soil Classification system or AASHTO system.

Equipment:

Liquid limit device, Porcelain (evaporating) dish, Flat grooving tool with gage, Eight moisture cans, Balance, Glass plate, Spatula, Wash bottle filled with distilled water, Drying oven set at 105°C.


A
Spatula
Liquid limit device
Grooving tool
Soil specimen
Moisture cans
B
ELE
C
D
13mm


E
Soil Specimen
Moisture cans
Spatula
Glass plate
F
Ellipsoidal soil mass
G
H

Test Procedure:

Liquid Limit:

1. Take roughly 3/4 of the soil and place it into the porcelain dish. Assume that the soil was previously passed through a No. 40 sieve, air-dried, and then pulverized. Thoroughly mix the soil with a small amount of distilled water until it appears as a smooth uniform paste. Cover the dish with cellophane to prevent moisture from escaping.
2. Weigh four of the empty moisture cans with their lids, and record the respective weights and can numbers on the data sheet.
3. Adjust the liquid limit apparatus by checking the height of drop of the cup. The point on the cup that comes in contact with the base should rise to a height of 10 mm. The block on the end of the grooving tool is 10 mm high and should be used as a gage. Practice using the cup and determine the correct rate to rotate the crank so that the cup drops approximately two times per second.
4. Place a portion of the previously mixed soil into the cup of the liquid limit apparatus at the point where the cup rests on the base. Squeeze the soil down to eliminate air pockets and spread it into the cup to a depth of about 10 mm at its deepest point. The soil pat should form an approximately horizontal surface (See Photo B).
5. Use the grooving tool carefully cut a clean straight groove down the center of the cup. The tool should remain perpendicular to the surface of the cup as groove is being made. Use extreme care to prevent sliding the soil relative to the surface of the cup (See Photo C).
6. Make sure that the base of the apparatus below the cup and the underside of the cup is clean of soil. Turn the crank of the apparatus at a rate of approximately two drops per second and count the number of drops, N, it takes to make the two halves of the soil pat come into contact at the bottom of the groove along a distance of 13 mm (1/2 in.)
7. (See Photo D). If the number of drops exceeds 50, then go directly to step eight and do not record the number of drops, otherwise, record the number of drops on the data sheet.
8. Take a sample, using the spatula, from edge to edge of the soil pat. The sample should include the soil on both sides of where the groove came into contact. Place the soil into a moisture can cover it. Immediately weigh the moisture can containing the soil, record its mass, remove the lid, and place the can into the oven. Leave the moisture can in the oven for at least 16 hours. Place the soil remaining in the cup into the porcelain dish. Clean and dry the cup on the apparatus and the grooving tool.
9. Remix the entire soil specimen in the porcelain dish. Add a small amount of distilled water to increase the water content so that the number of drops required closing the groove decrease.
10. Repeat steps six, seven, and eight for at least two additional trials producing successively lower numbers of drops to close the groove.
11. One of the trials shall be for a closure requiring 25 to 35 drops, one for closure between 20 and 30 drops, and one trial for a closure requiring 15 to 25 drops. Determine the water content from each trial by using the same method used in the first laboratory. Remember to use the same balance for all weighing.

Plastic Limit:

1. Weigh the remaining empty moisture cans with their lids, and record the respective weights and can numbers on the data sheet.
2. Take the remaining 1/4 of the original soil sample and add distilled water until the soil is at a consistency where it can be rolled without sticking to the hands.
3. Form the soil into an ellipsoidal mass (See Photo F). Roll the mass between the palm or the fingers and the glass plate (See Photo G). Use sufficient pressure to roll the mass into a thread of uniform diameter by using about 90 strokes per minute. (A stroke is one complete motion of the hand forward and back to the starting position.) The thread shall be deformed so that its diameter reaches 3.2 mm (1/8 in.), taking no more than two minutes.
4. When the diameter of the thread reaches the correct diameter, break the thread into several pieces. Knead and reform the pieces into ellipsoidal masses and re-roll them. Continue this alternate rolling, gathering together, kneading and re-rolling until the thread crumbles under the pressure required for rolling and can no longer be rolled into a 3.2 mm diameter thread (See Photo H).
5. Gather the portions of the crumbled thread together and place the soil into moisture can, and then cover it. If the can does not contain at least
6. 6 grams of soil, add soil to the can from the next trial (See Step 6). Immediately weigh the moisture can containing the soil, record it's mass, remove the lid, and place the can into the oven. Leave the moisture can in the oven for at least 16 hours.
7. Repeat steps three, four, and five at least two more times. Determine the water content from each trial by using the same method used in the first laboratory. Remember to use the same balance for all weighing.

Analysis:

Liquid Limit:

1. Calculate the water content of each of the liquid limit moisture cans after they have been in the oven for at least 16 hours.
2. Plot the number of drops, N, (on the log scale) versus the water content (w). Draw the best-fit straight line through the plotted points and determine the liquid limit (LL) as the water content at 25 drops.

Plastic Limit:

1. Calculate the water content of each of the plastic limit moisture cans after they have been in the oven for at least 16 hours.
2. Compute the average of the water contents to determine the plastic limit, PL. Check to see if the difference between the water contents is greater than the acceptable range of two results (2.6 %).
3. Calculate the plasticity index, PI=LL-PL. Report the liquid limit, plastic limit, and plasticity index to the nearest whole number, omitting the percent designation.

Example Data

Date Tested: September 20, 2002

Tested By: CEMM315 Class, Group A

Project Name: CEMM315 Lab

Sample Number: B-1, SS-1, 8'-10'

Sample Description: Grayey silty clay

Liquid Limit Determination

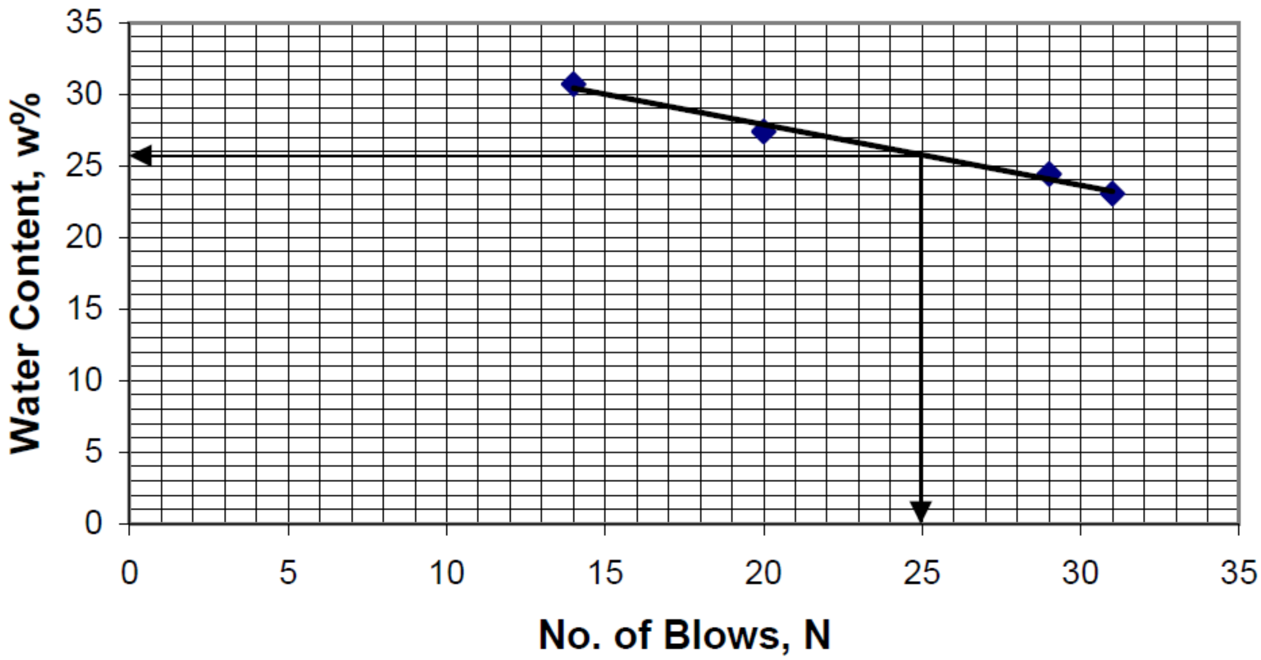
Sample no.	1	2	3	4
Moisture can and lid number	11	1	5	4
M_C = Mass of empty, clean can + lid (grams)	22.23	23.31	21.87	22.58
M_{CMS} = Mass of can, lid, and moist soil (grams)	28.56	29.27	25.73	25.22
M_{CDS} = Mass of can, lid, and dry soil (grams)	27.40	28.10	24.90	24.60
M_S = Mass of soil solids (grams)	5.03	4.79	3.03	2.02
M_W = Mass of pore water (grams)	1.16	1.17	0.83	0.62
w = Water content, w%	23.06	24.43	27.39	30.69
No. of drops (N)	31	29	20	14

Plastic Limit Determination

Sample no.	1	2	3
Moisture can and lid number	7	14	13
M_C = Mass of empty, clean can + lid (grams)	7.78	7.83	15.16
M_{CMS} = Mass of can, lid, and moist soil (grams)	16.39	13.43	21.23
M_{CDS} = Mass of can, lid, and dry soil (grams)	15.28	12 .69	20.43
M_S = Mass of soil solids (grams)	7.5	4.86	5.27
M_W = Mass of pore water (grams)	1. 11	0.74	0.8
w = Water content, w%	14.8	15.2	15.1

$$\text{Plastic Limit (PL)= Average w \%} = \frac{14.8+15.2+15.1}{3} = 15.0$$

LIQUID LIMIT CHART



From the above graph, Liquid Limit = 26

Final Results:

Liquid Limit = 26

Plastic Limit = 15

Plasticity Index =11

7. Grain Size Distribution (Sieve Analysis and Hydrometer Analysis)

Purpose:

This test is performed to determine the percentage of different grain sizes contained within a soil. The mechanical or sieve analysis is performed to determine the distribution of the coarser, larger-sized particles, and the hydrometer method is used to determine the distribution of the finer particles.

Standard Reference:

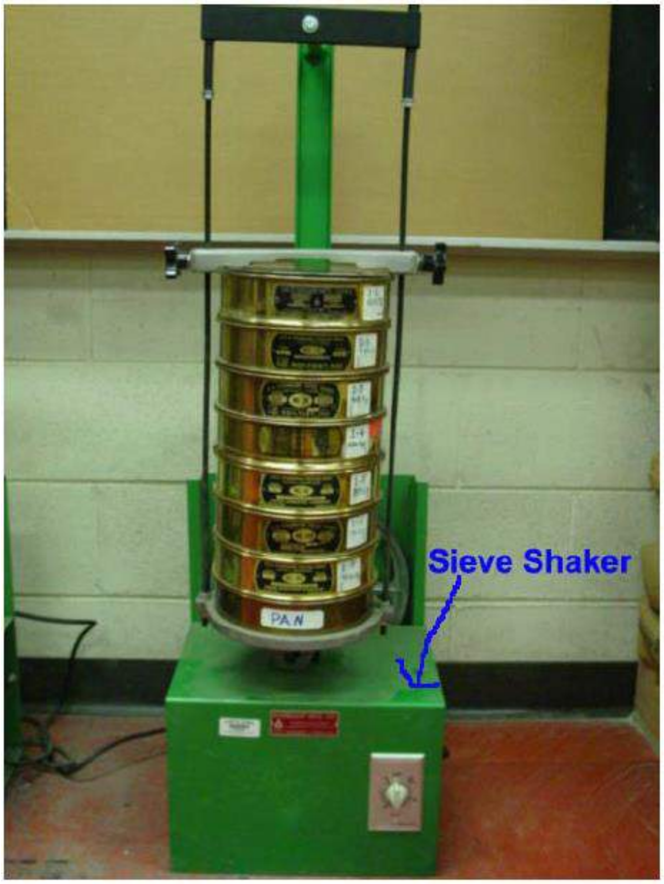
ASTM D 422 - Standard Test Method for Particle-Size Analysis of Soils

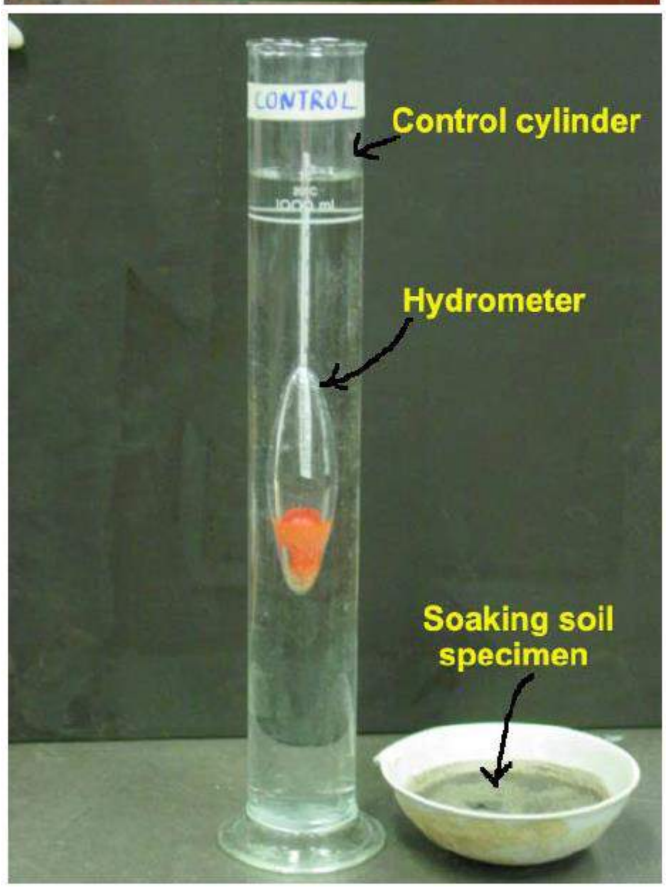
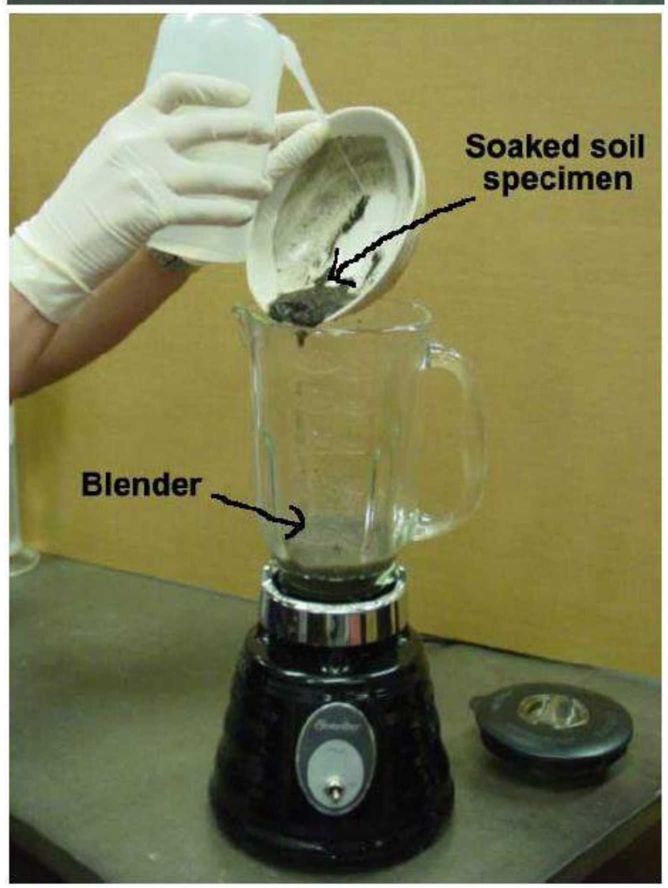
Significance:

The distribution of different grain sizes affects the engineering properties of soil. Grain size analysis provides the grain size distribution, and it is required in classifying the soil.

Equipment:

Balance, Set of sieves, Cleaning brush, Sieve shaker, Mixer (blender), 152H Hydrometer, Sedimentation cylinder, Control cylinder, Thermometer, Beaker, Timing device.


Sieve Shaker
PAN

Adventurer
OHAUS

CONTROL
Control cylinder
Hydrometer
Soaking soil specimen

Soaked soil specimen
Blender

Test Procedure:

Sieve Analysis:

1. Write down the weight of each sieve as well as the bottom pan to be used in the analysis.
2. Record the weight of the given dry soil sample.
3. Make sure that all the sieves are clean, and assemble them in the ascending order of sieve numbers (#4 sieves at top and #200 sieves at bottom). Place the pan below #200 sieves. Carefully pour the soil sample into the top sieve and place the cap over it.
4. Place the sieve stack in the mechanical shaker and shake for 10 minutes.
5. Remove the stack from the shaker and carefully weigh and record the weight of each sieve with its retained soil. In addition, remember to weigh and record the weight of the bottom pan with its retained fine soil.

Hydrometer Analysis:

1. Take the fine soil from the bottom pan of the sieve set, place it into a beaker, and add 125 mL of the dispersing agent (sodium hexametaphosphate (40 g/L)) solution. Stir the mixture until the soil is thoroughly wet. Let the soil soak for at least ten minutes.
2. While the soil is soaking, add 125mL of dispersing agent into the control cylinder and fill it with distilled water to the mark. Take the reading at the top of the meniscus formed by the hydrometer stem and the control solution. A reading less than zero is recorded as a negative

3. (-) correction and a reading between zero and sixty is recorded as a positive (+) correction. This reading is called the zero correction. The meniscus correction is the difference between the top of the meniscus and the level of the solution in the control jar (Usually about +1).
4. Shake the control cylinder in such a way that the contents are mixed thoroughly. Insert the hydrometer and thermometer into the control cylinder and note the zero correction and temperature respectively.
5. Transfer the soil slurry into a mixer by adding more distilled water, if necessary, until mixing cup is at least half full. Then mix the solution for a period of two minutes.
6. Immediately transfer the soil slurry into the empty sedimentation cylinder. Add distilled water up to the mark.
7. Cover the open end of the cylinder with a stopper and secure it with the palm of your hand. Then turn the cylinder upside down and back upright for a period of one minute. (The cylinder should be inverted approximately 30 times during the minute.)
8. Set the cylinder down and record the time. Remove the stopper from the cylinder. After an elapsed time of one minute and forty seconds, very slowly and carefully insert the hydrometer for the first reading. (Note: It should take about ten seconds to insert or remove the hydrometer to minimize any disturbance, and the release of the hydrometer should be made as close to the reading depth as possible to avoid excessive bobbing).
9. The reading is taken by observing the top of the meniscus formed by the suspension and the hydrometer stem. The hydrometer is removed slowly and placed back into the control cylinder. Very gently spin it in control cylinder to remove any particles that may have adhered.
10. Take hydrometer readings after elapsed time of 2 and 5, 8, 15, 30, 60 minutes and 24 hours.

Data Analysis:

Sieve Analysis:

1. Obtain the mass of soil retained on each sieve by subtracting the weight of the empty sieve from the mass of the sieve + retained soil, and record this mass as the weight retained on the data sheet. The sum of these retained masses should be approximately equals the initial mass of the soil sample. A loss of more than two percent is unsatisfactory.
2. Calculate the percent retained on each sieve by dividing the weight retained on each sieve by the original sample mass.
3. Calculate the percent passing (or percent finer) by starting with 100 percent and subtracting the percent retained on each sieve as a cumulative procedure.
   a. ***For example: Total mass = 500 g***
      Mass retained on No. 4 sieve = 9.7 g
      Mass retained on No. 10 sieve = 39.5 g
   b. ***For the No.4 sieve:***
      Quantity passing = Total mass - Mass retained = 500 - 9.7 = 490.3 g
      The percent retained is calculated as;
      % retained = Mass retained/Total mass = (9.7/500) X 100 = 1.9 %
      From this, the % passing = 100 - 1.9 = 98.1 %
   c. ***For the No. 10 sieve:***
      Quantity passing = Mass arriving - Mass retained = 490.3 - 39.5 = 450.8 g
      % Retained = (39.5/500) X 100 = 7.9 %

% Passing = 100 - 1.9 - 7.9 = 90.2 %

(Alternatively, use % passing = % Arriving - % Retained

d. For No. 10 sieve = 98.1 - 7.9 = 90.2 %)

4. Make a semilogarithmic plot of grain size vs. percent finer.
5. Compute C_c and C_u for the soil.

Hydrometer Analysis:

1. Apply meniscus correction to the actual hydrometer reading.
2. From Table 1, obtain the effective hydrometer depth L in cm (for meniscus corrected reading).
3. For known G_s of the soil (if not known, assume 2.65 for this lab purpose), obtain the value of K from Table 2.
4. Calculate the equivalent particle diameter by using the following formula:

$$D = K\sqrt{\frac{L}{t}}$$

Where t is in minutes, and D is given in mm.

5. Determine the temperature correction CT from Table 3.
6. Determine correction factor "a" from Table 4 using Gs.
7. Calculate corrected hydrometer reading as follows:

$$Rc = R_{ACTUAL} - \text{zero correction} + C_T$$

8. Calculate percent finer as follows:

$$P = \frac{Rc \times a}{w_s} \times 100$$

Where WS is the weight of the soil sample in grams

9. Adjusted percent fines as follows:

$$P_A = \frac{P \times F_{200}}{100}$$

F_{200} = % finer of #200 sieve as a percent

10. Plot the grain size curve D versus the adjusted percent finer on the semilogarithmic sheet.

Table below show the values of Effective Depth Based on Hydrometer and Sedimentation Cylinder of Specific Sizes

Hydrometer 151H		Hydrometer 152H			
Actual Hydrometer Reading	Effective Depth, L (cm)	Actual Hydrometer Reading	Effective Depth, L (cm)	Actual Hydrometer Reading	Effective Depth, L (cm)
1.000	16.3	0	16.3	31	11.2
1.001	16.0	1	16.1	32	11.1
1.002	15.8	2	16.0	33	10.9
1.003	15.5	3	15.8	34	10.7
1.004	15.2	4	15.6	35	10.6
1.005	15.0	5	15.5	36	10.4
1.006	14.7	6	15.3	37	10.2
1.007	14.4	7	15.2	38	10.1
1.008	14.2	8	15.0	39	9.9
1.009	13.9	9	14.8	40	9.7
1.010	13.7	10	14.7	41	9.6
1.011	13.4	11	14.5	42	9.4
1.012	13.1	12	14.3	43	9.2
1.013	12.9	13	14.2	44	9.1
1.014	12.6	14	14.0	45	8.9
1.015	12.3	15	13.8	46	8.8
1.016	12.1	16	13.7	47	8.6
1.017	11.8	17	13.5	48	8.4
1.018	11.5	18	13.3	49	8.3
1.019	11.3	19	13.2	50	8.1
1.020	11.0	20	13.0	51	7.9
1.021	10.7	21	12.9	52	7.8
1.022	10.5	22	12.7	53	7.6
1.023	10.2	23	12.5	54	7.4
1.024	10.0	24	12.4	55	7.3
1.025	9.7	25	12.2	56	7.1
1.026	9.4	26	12.0	57	7.0
1.027	9.2	27	11.9	58	6.8
1.028	8.9	28	11.7	59	6.6
1.029	8.6	29	11.5	60	6.5
1.030	8.4	30	11.4		
1.031	8.1				
1.032	7.8				
1.033	7.6				
1.034	7.3				
1.035	7.0				
1.036	6.8				
1.037	6.5				
1.038	6.2				
1.039	5.9				

Table below show the Values of k for Use in Equation for Computing Diameter of Particle in Hydrometer Analysis

Temperature °C	Specific Gravity of Soil Particles								
	2.45	2.50	2.55	2.60	2.65	2.70	2.75	2.80	2.85
16	0.01510	0.01505	0.01481	0.01457	0.01435	0.01414	0.0394	0.01374	0.01356
17	0.01511	0.01486	0.01462	0.01439	0.01417	0.01396	0.01376	0.01356	0.01338
18	0.01492	0.01467	0.01443	0.01421	0.01399	0.01378	0.01359	0.01339	0.01321
19	0.01474	0.01449	0.01425	0.01403	0.01382	0.01361	0.01342	0.01323	0.01305
20	0.01456	0.01431	0.01408	0.01386	0.01365	0.01344	0.01325	0.01307	0.01289
21	0.01438	0.01414	0.01391	0.01369	0.01348	0.01328	0.01309	0.01291	0.01273
22	0.01421	0.01397	0.01374	0.01353	0.01332	0.01312	0.01294	0.01276	0.01258
23	0.01404	0.01381	0.01358	0.01337	0.01317	0.01297	0.01279	0.01261	0.01243
24	0.01388	0.01365	0.01342	0.01321	0.01301	0.01282	0.01264	0.01246	0.01229
25	0.01372	0.01349	0.01327	0.01306	0.01286	0.01267	0.01249	0.01232	0.01215
26	0.01357	0.01334	0.01312	0.01291	0.01272	0.01253	0.01235	0.01218	0.01201
27	0.01342	0.01319	0.01297	0.01277	0.01258	0.01239	0.01221	0.01204	0.01188
28	0.01327	0.01304	0.01283	0.01264	0.01244	0.01255	0.01208	0.01191	0.01175
29	0.01312	0.01290	0.01269	0.01269	0.01230	0.01212	0.01195	0.01178	0.01162
30	0.01298	0.01276	0.01256	0.01236	0.01217	0.01199	0.01182	0.01165	0.01149

Table below show the Temperature Correction Factors C_T

Temperature °C	factor C_T
15	1.10
16	-0.90
17	-0.70
18	-0.50
19	-0.30
20	0.00
21	+0.20
22	+0.40
23	+0.70
24	+1.00
25	+1.30
26	+1.65
27	+2.00
28	+2.50
29	+3.05
30	+3.80

Table below show Correction Factors a for Unit Weight of Solids

Unit Weight of Soil Solids, g/cm^3	Correction factor a
2.85	0.96
2.80	0.97
2.75	0.98
2.70	0.99
2.65	1.00
2.60	1.01
2.55	1.02
2.50	1.04

Example Data

Grain Size Analysis

Sieve Analysis

Date Tested: September 15, 2002

Tested By: CEMM315 Class, Group A

Project Name: CEMM315 Lab

Sample Number: B-1, ST-1, 2'-3. 5'

Visual Classification of Soil: Brown clayey to silty sand, trace fine gravel

Weight of Container: 198.5 gm

Wt. Container+Dry Soil: 722.3 gm

Wt. of Dry Sample: 523.8 gm

Sieve Number	Diameter (mm)	Mass of Empty Sieve (g)	Mass of Sieve+Soil Retained (g)	Soil Retained (g)	Percent Retained	Percent Passing
4	4.75	116.23	166.13	49.9	9.5	90.5
10	2.0	99.27	135.77	36.5	7.0	83.5
20	0.84	97.58	139.68	42.1	8.0	75.5
40	0.425	98.96	138.96	40.0	7.6	67.8
60	0.25	91.46	114.46	23.0	4.4	63.4
140	0.106	93.15	184.15	91.0	17.4	46.1
200	0.075	90.92	101.12	10.2	1.9	44.1
Pan	---	70.19	301.19	231.0	44.1	0.0
			Total Weight=	523.7		

* Percent passing=100-cumulative percent retained.

From Grain Size Distribution Curve:

% Gravel= 9.5 D_{10}= 0.002 mm
% Sand= 46.4 D_{30}= 0.017 mm
% Fines= 44.1 D_{60}= .0.25 mm

$$c_u = \frac{0.25}{0.002} = 125, \quad c_c = \frac{(0.017)^2}{0.25 \times 0.002} = 0.58$$

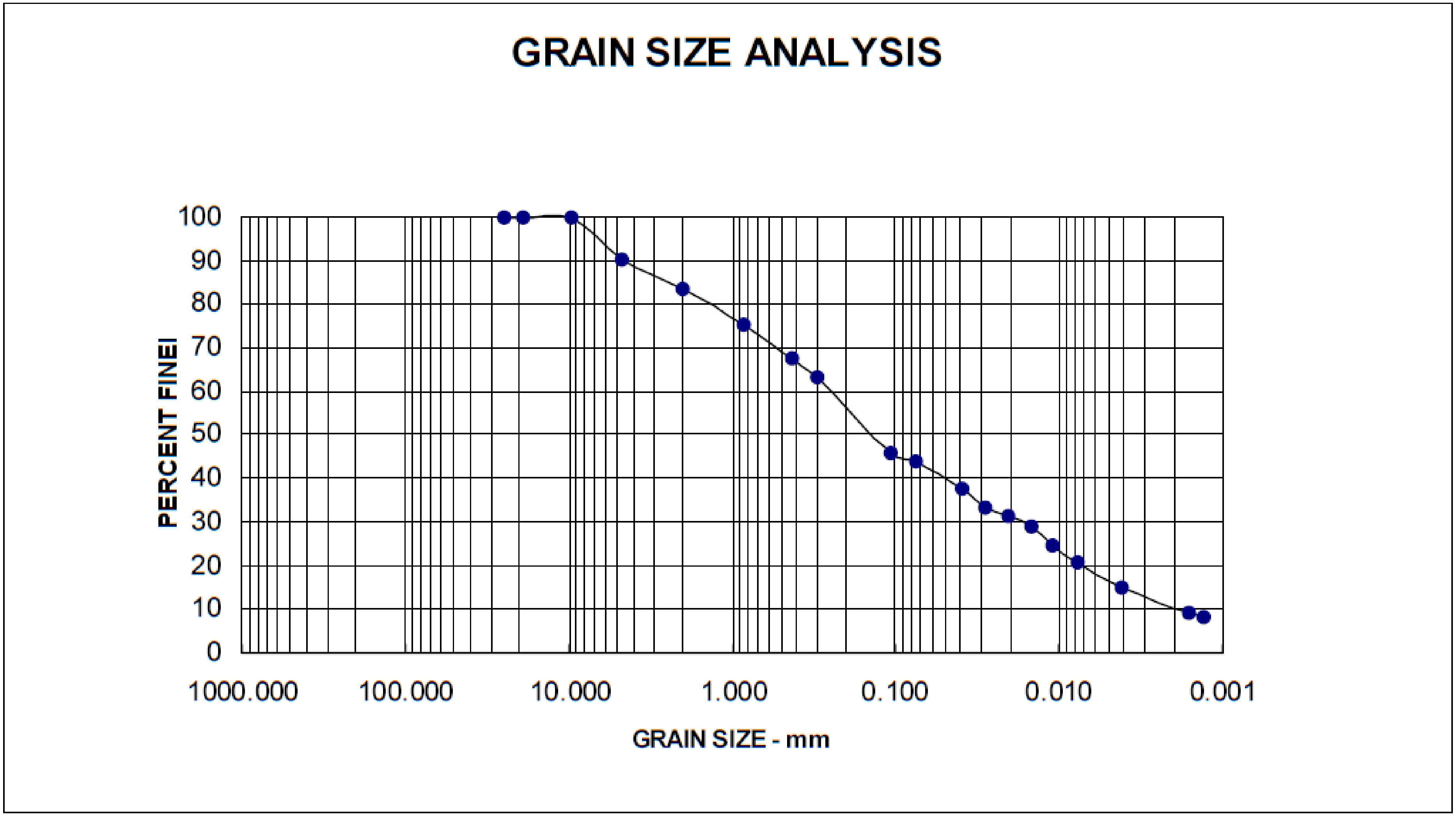
Unified Classification of Soil: SC/SM

Hydrometer Analysis

Test Date: September 15, 2002
Tested By: CEMM315 Class, Group A
Hydrometer Number (if known): 152 H
Specific Gravity of Solids: 2.56
Dispersing Agent: Sodium Hexametaphosphate
Weight of Soil Sample: 50.0 gm
Zero Correction: +6
Meniscus Correction: +1

Date	Time	Elapsed Time (min)	Temp. ^{o}C	Actual Hydro. Rdg. R_a	Hyd. Corr. for Meniscus	L from Table 1	K from Table 2	D mm	C_T from Table 3	a from Table 4	Corr. Hydr. Rdg. R_c	% Finer P	% Adjusted Finer P_A
09/15	4:06 PM	0	25	55	56	7.1	0.01326	0	+1.3	1.018	-	-	-
	4:07	1	25	47	48	8.6	0.01326	0.03029	+1.3	1.018	42.3	86.1	37.8
	4:08	2	25	42	43	9.2	0.01326	0.02844	+1.3	1.018	37.3	75.9	33.3
	4:10	4	25	40	41	9.6	0.01326	0.02054	+1.3	1.018	35.3	71.9	31.6
	4:14	8	25	37	38	10.1	0.01326	0.01490	+1.3	1.018	32.3	65.8	28.6
	4:22	16	25	32	33	10.9	0.01326	0.01094	+1.3	1.018	27.3	55.6	24.1
	4:40	34	25	28	29	11.5	0.01326	0.00771	+1.3	1.018	23.3	47.4	20.8
	6:22	136	23	22	23	12.5	0.01356	0.00411	+0.7	1.018	16.7	34	14.9
09/16	5:24 PM	1518	22	15	16	13.7	0.01366	0.00130	+0.4	1.018	9.4	19.1	8.4

Unified Classification of Soil: SC/SM


GRAIN SIZE ANALYSIS
PERCENT FINEI
100
90
80
70
60
50
40
30
20
10
0
1000.000
100.000
10.000
1.000
0.100
0.010
0.001
GRAIN SIZE - mm

Grain Size Analysis

Sieve Analysis

Date Tested:
Tested By:
Project Name:
Sample Number:
Visual Classification of Soil:

Weight of Container: ______gm
Wt. Container+Dry Soil: ______gm
Wt. of Dry Sample: ______gm

Sieve Number	Diameter (mm)	Mass of Empty Sieve (g)	Mass of Sieve+Soil Retained (g)	Soil Retained (g)	Percent Retained	Percent Passing
4	4.75					
10	2. 0					
20	0.84					
40	0.425					
60	0. 25					
140	0.106					
200	0.075					
Pan	---					
			Total Weight=			

* Percent passing=100-cumulative percent retained.

From Grain Size Distribution Curve:

% Gravel= ________ D_{10}= ______ mm

% Sand= ________ D_{30}= ______ mm

% Fines= ________ D_{60}= _______ mm

C_u= ________________ C_C= _________

Unified Classification of Soil: _________

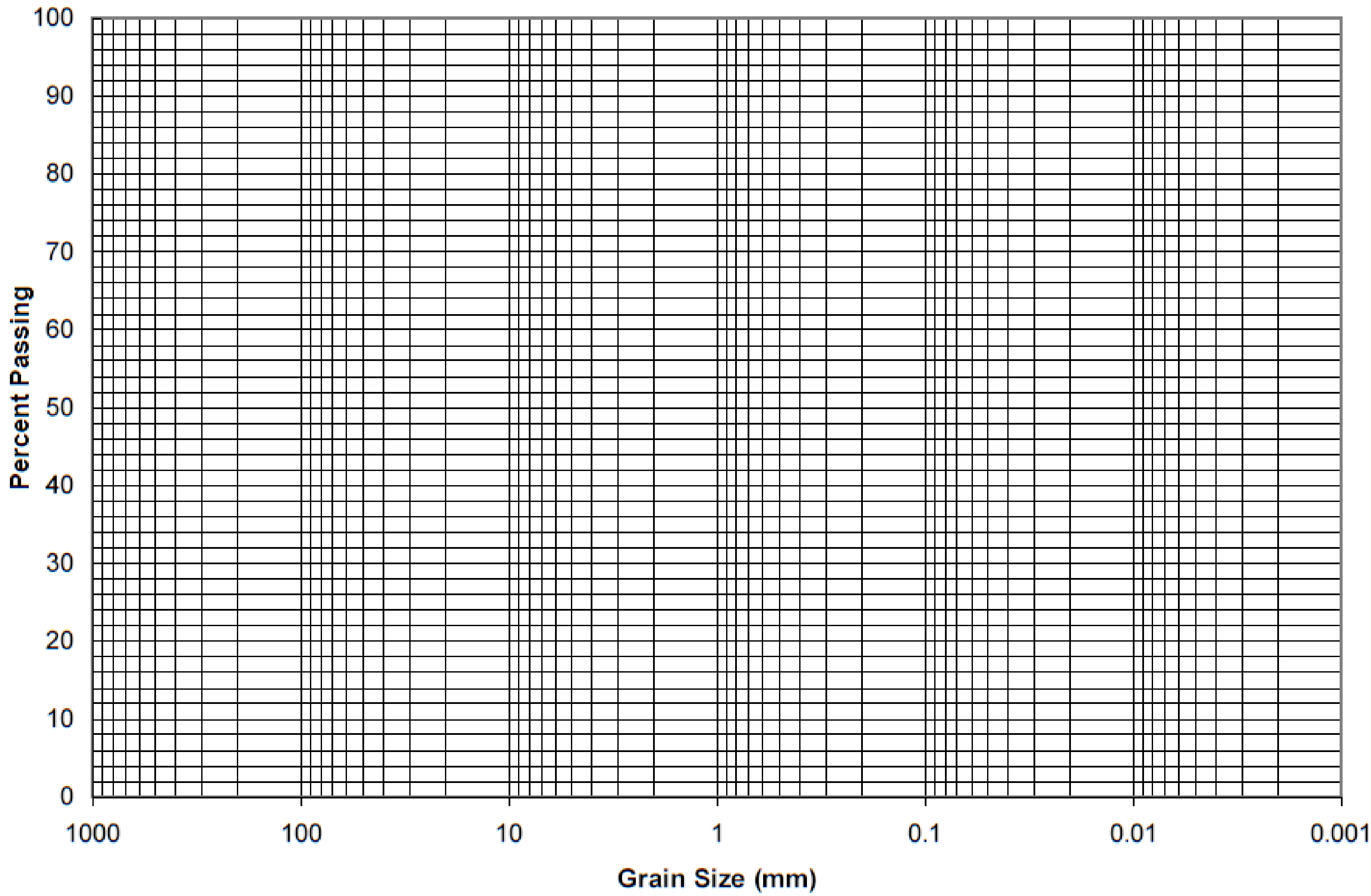
Hydrometer Analysis

Test Date: ___________
Tested By: ___________
Hydrometer Number (if known): __________
Specific Gravity of Solids:__________
Dispersing Agent: __________
Weight of Soil Sample: __________gm

Zero Correction: __________
Meniscus Correction: __________

Date	Time	Elapsed Time (min)	Temp. ^oC	Actual Hydro. Rdg. R_a	Hyd. Corr. for Meniscus	L from Table 1	K from Table 2	D mm	C_T from Table 3	a from Table 4	Corr. Hydr. Rdg. R_c	% Finer P	% Adjusted Finer P_A

Unified Classification of Soil:



Note: You can plot your data on this graph or generate similar graph using any graphics program (e.g., excel)

8. *Visual Classification*

Purpose:

Visually classify the soils.

Standard Reference:

ASTM D 2488 - Standard Practice for Description and Identification of Soils (Visual - Manual Procedure)

Significance:

The first step in any geotechnical engineering project is to identify and describe the subsoil condition. For example, as soon as a ground is identified as gravel, engineer can immediately form some ideas on the nature of problems that might be encountered in a tunneling project. In contrast, a soft clay ground is expected to lead to other types of design and construction considerations. Therefore, it is useful to have a systematic procedure for identification of soils even in the planning stages of a project. Soils can be classified into two general categories: (1) coarse grained soils and (2) fine grained soils. Examples of coarse-grained soils are gravels and sands. Examples of fine-grained soils are silts and clays. Procedures for visually identifying these two general types of soils are described in the following sections.

Equipment:

Magnifying glass (optional)

Identification Procedure:

a. Identify the color (e.g. brown, gray, brownish gray), odor (if any) and texture (coarse or fine-grained) of soil.
b. Identify the major soil constituent (>50% by weight) using Table 1 as coarse gravel, fine gravel, coarse sand, medium sand, fine sand, or fines.
c. Estimate percentages of all other soil constituents using Table 1 and the following terms:

Trace - 0 to 10% by weight
Little - 10 to 20%
Some - 20 to 30%
And - 30 to 50%
(Examples: trace fine gravel, little silt, some clay)

d. If the major soil constituent is sand or gravel:
Identify particle distribution. Describe as **well graded** or **poorly graded**. Well-graded soil consists of particle sizes over a wide range. Poorly graded soil consists of particles which are all about the same size. Identify particle shape (angular, sub-angular, rounded, sub-rounded) using Figure 1 and Table 2.
e. If the major soil constituents are fines, perform the following tests:

Dry strength test: Mold a sample into 1/8" size ball and let it dry. Test the strength of the dry sample by crushing it between the fingers. Describe the strength as none, low, medium, high or very high depending on the results of the test as shown in Table 3(a).

Dilatancy Test: Make a sample of soft putty consistency in your palm. Then observe the reaction during shaking, squeezing (by closing hand) and vigorous tapping. The reaction is rapid, slow or none according to the test results given in Table 3(b). During dilatancy test, vibration densities the silt and water appears on the surface. Now on squeezing, shear stresses are applied on the densified silt. The dense silt has a tendency for volume increase or dilatancy due to shear stresses. So the water disappears from the surface. Moreover, silty soil has a high permeability, so the water moves quickly. In clay, we see no change, no shiny surface, in other words, no reaction.

Plasticity (or Toughness) Test: Roll the samples into a thread about 1/8" in diameter. Fold the thread and reroll it repeatedly until the thread crumbles at a diameter of 1/8". Note (a) the pressure required to roll the thread when it is near crumbling, (b) whether it can support its own weight, (c) whether it can be molded back into a coherent mass, and (d) whether it is tough during kneading. Describe the plasticity and toughness according to the criteria in Tables 3(c) and 3(d). A low to medium toughness and non-plastic to low plasticity is the indication that the soil is silty; otherwise the soil is clayey.

Based on dry strength, dilatancy and toughness, determine soil symbol based on Table 4.

f. Identify moisture condition (dry, moist, wet or saturated) using Table 5.

g. Record visual classification of the soil in the following order: color, major constituent, minor constituents, particle distribution and particle shape (if major constituent is coarse-grained), plasticity (if major constituent is fine-grained), moisture content, soil symbol (if major constituent is fine-grained).

Examples of coarse-grained soils:

Soil 1: Brown fine gravel, some coarse to fine sand, trace silt, trace clay, well graded, angular, dry.

Soil 2: Gray coarse sand, trace medium to fine sand, some silt, trace clay, poorly graded, rounded, saturated.

Examples of fine-grained soils:

Soil A: Brown lean clay, trace coarse to fine sand, medium plasticity, moist, CL.

Soil B: Gray clayey silt, trace fine sand, non-plastic, saturated, ML.

Laboratory Exercise:

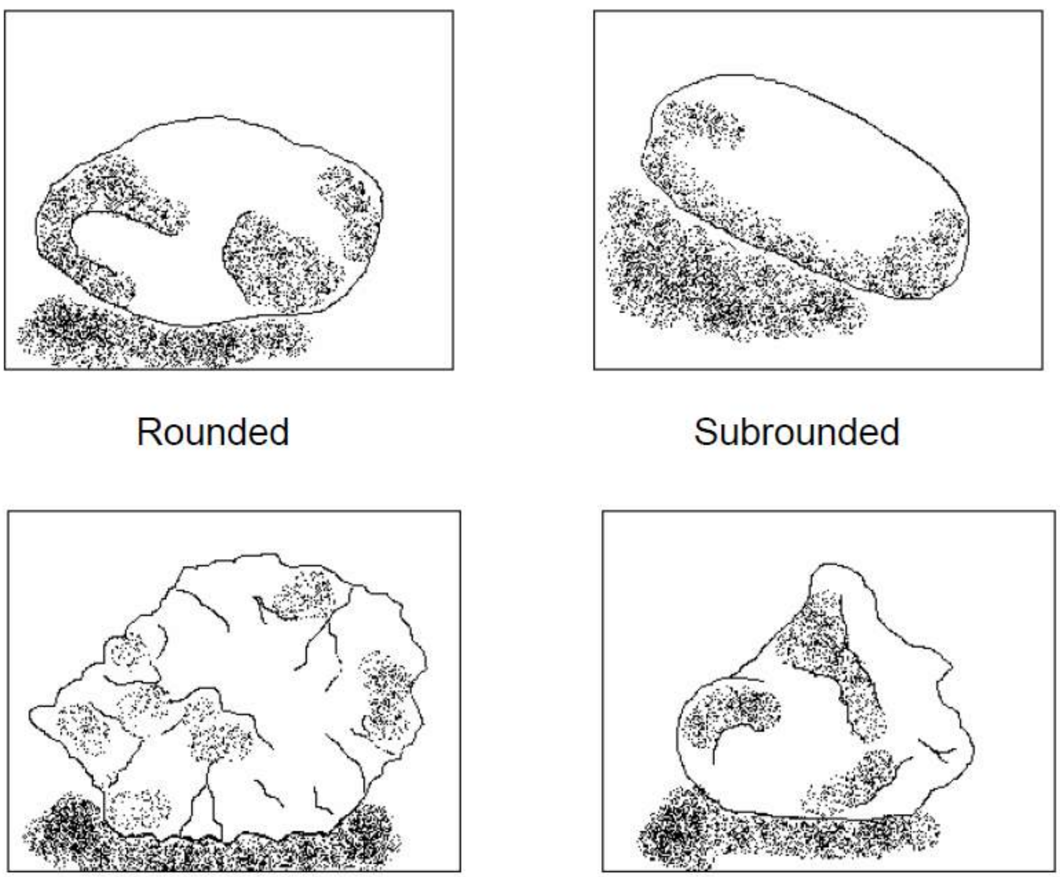
You will be given ten different soil samples. Visually classify these soils. Record all information on the attached forms.

Table 1. Grain Size Distribution

Soil Constituent	Size Limits	Familiar Example
Boulder	12 in. (305 mm) or more	Larger than basketball
Cobbles	3 in (76 mm) -12 in (305 mm)	Grapefruit
Coarse Gravel	¾ in. (19 mm) – 3 in. (76 mm)	Orange or Lemon
Fine Gravel	4.75 mm (No.4 Sieve) – ¾ in. (19 mm)	Grape or Pea
Coarse Sand	2 mm (No.10 Sieve) – 4.75 mm (No. 4 Sieve)	Rocksalt
Medium Sand	0.42 mm (No. 40 Sieve) – 2 mm (No. 10 Sieve)	Sugar, table salt
Fine Sand*	0.075 mm (No. 200 Sieve) – 0.42 mm (No. 40 Sieve)	Powdered Sugar
Fines	Less than 0.0075 mm (No. 200 Sieve)	-

*Particles finer than fine sand cannot be discerned with the naked eye at a distance of 8 in (20 cm).

Figure 1. Shape of Coarse-Grained Soil Particles



Rounded Subrounded

Angular Subangular

Table 2. Criteria for Describing Shape of Coarse-Grained Soil Particles

Description	Criteria
Angular	Particles have sharp edges and relatively plane sides with unpolished surfaces.
Subangular	Particles are similar to angular description, but have rounded edges.
Subrounded	Particles have nearly plane sides, but have well-rounded corners and edges.
Rounded	Particles have smoothly curved sides and no edges.

Table (3a). Criteria for Describing Dry Strength

Description	Criteria
None	The dry specimen ball crumbles into powder with the slightest handling pressure.
Low	The dry specimen crumbles into powder with some pressure form fingers.
Medium	The dry specimen breaks into pieces or crumbles with moderate finger pressure.
High	The dry specimen cannot be broken with finger pressure. Specimen will break into pieces between thumb and a hard surface.
Very High	The dry specimen cannot be broken between the thumb and a hard surface.

Table (3b). Criteria for Describing Dilatancy of a Soil Sample

Description	Criteria
None	There is no visible change in the soil samples.
Slow	Water slowly appears and remains on the surface during shaking or water slowly disappears upon squeezing.
Rapid	Water quickly appears on the surface during shaking and quickly disappears upon squeezing.

Table (3c). Criteria for Describing Soil Plasticity

Description	Criteria
Non-plastic	A 1/8” (3-mm) thread cannot be rolled at any water content.
Low	The thread is difficult to roll and a cohesive mass cannot be formed when drier than the plastic limit.
Medium	The thread is easy to roll and little time is needed to reach the plastic limit. The thread cannot be re-rolled after the plastic limit is reached. The mass crumbles when it is drier than the plastic limit.
High	Considerable time is needed, rolling and kneading the sample, to reach the plastic limit. The thread can be rerolled and reworked several times before reaching the plastic limit. A mass can be formed when the sample is drier than the plastic limit

Note: The plastic limit is the water content at which the soil begins to break apart and crumbles when rolled into threads 1/8” in diameter.

Table (3d). Criteria for Describing Soil Toughness

Description	Criteria
Low	Only slight pressure is needed to roll the thread to the plastic limit. The thread and mass are weak and soft.
Medium	Moderate pressure is needed to roll the thread to near the plastic limit. The thread and mass have moderate stiffness.
High	Substantial pressure is needed to roll the thread to near the plastic limit. The thread and mass are very stiff.

Table 4. Identification of Inorganic Fine-Grained Soils

Soil Symbol	Dry Strength	Dilatancy	Toughness
ML	None or Low	Slow to Rapid	Low or thread cannot be formed
CL	Medium to High	None to Slow	Medium
MH	Low to Medium	None to Slow	Low to Medium
CH	High to Very High	None	High

Note: ML = Silt; CL = Lean Clay (low plasticity clay); MH = Elastic Soil; CH = Fat Clay (high plasticity clay). The terms ‘lean’ and ‘fat’ may not be used in certain geographic regions (midwest).

Table 5. Criteria for Describing Soil Moisture Conditions

Description	Criteria
Dry	Soil is dry to the touch, dusty, a clear absence of moisture
Moist	Soil is damp, slight moisture; soil may begin to retain molded form
Wet	Soil is clearly wet; water is visible when sample is squeezed
Saturated	Water is easily visible and drains freely from the sample

VISUAL SOIL CLASSIFICATION

DATA SHEET

Soil Number: Soil A

Classified by: RES

Date: 09-29-02

1. Color brown
2. Odor none
3. Texture coarse
4. Major soil constituent : gravel
5. Minor soil constituents: sand, fines

Type	Approx. % by weight
gravel	60
sand	30
fines	10

6. For coarse-grained soils:

Gradation: well graded
Particle Shape: subrounded

7. For fine-grained soils:

Dry Strength ______
Dilatancy ______
Plasticity ______
Toughness ______
Soil Symbol ______

8. Moisture Condition: dry

Classification:

Brown gravel, some sand, trace fines, well graded, subrounded, dry

VISUAL SOIL CLASSIFICATION
DATA SHEET

Soil Number: Soil B
Classified by: RES
Date: 09-27-02

1. Color gray
2. Odor none
3. Texture coarse
4. Major soil constituent: sand
5. Minor soil constituents: gravel, fines

Type	Approx. % by weight
sand	80
fine gravel	15
fines	5

6. For coarse-grained soils:

Gradation: poorly graded
Particle Shape: rounded

7. For fine-grained soils:

Dry Strength ______
Dilatancy ______
Plasticity ______
Toughness ______
Soil Symbol ______

8. Moisture Condition: dry

Classification:

Gray sand, little fine gravel, trace fines, poorly graded, rounded, dry

VISUAL SOIL CLASSIFICATION

DATA SHEET

Soil Number: Soil C
Classified by: RES
Date: 09-29-02

1. Color gray
2. Odor none
3. Texture fine-grained
4. Major soil constituent : fines
5. Minor soil constituents: Fine Sand

Type	Approx. % by weight
Fines	95
Fine Sand	5

6. For coarse-grained soils:

   Gradation: ______
   Particle Shape: ______

7. For fine-grained soils:
   - Dry strength high
   - Dilatancy none
   - Plasticity medium
   - Toughness medium
   - Soil Symbol CL

8. Moisture Condition: moist

Classification:

Gray silty clay, trace fine sand, medium plasticity, moist, CL

VISUAL SOIL CLASSIFICATION
DATA SHEET

Soil Number: ________________
Classified by: _________________
Date: _________________

1. Color __________________
2. Odor __________________
3. Texture ___________________
4. Major soil constituent: ____________
5. Minor soil constituents: ___________

Type	Approx. % by weight
__________	________
__________	________
_________	_________

6. For coarse-grained soils:

   Gradation: ____________
   Particle Shape: ____________

7. For fine-grained soils:
   - Dry Strength ___________
   - Dilatancy ___________
   - Plasticity ___________
   - Toughness ___________
   - Soil Symbol ___________

8. Moisture Condition: _____________

Classification:

9. *Moisture-Density Relationship (Compaction)*

Purpose:

This laboratory test is performed to determine the relationship between the moisture content and the dry density of a soil for a specified compactive effort. The compactive effort is the amount of mechanical energy that is applied to the soil mass. Several different methods are used to compact soil in the field, and some examples include tamping, kneading, vibration, and static load compaction. This laboratory will employ the tamping or impact compaction method using the type of equipment and methodology developed by R. R. Proctor in 1933, therefore, the test is also known as the Proctor test. Two types of compaction tests are routinely performed: (1) The Standard Proctor Test, and (2) The Modified Proctor Test. Each of these tests can be performed in three different methods as outlined in the attached Table 1. In the Standard Proctor Test, the soil is compacted by a 5.5 lb hammer falling a distance of one foot into a soil filled mold. The mold is filled with three equal layers of soil, and each layer is subjected to 25 drops of the hammer. The Modified Proctor Test is identical to the Standard Proctor Test except it employs, a 10 lb hammer falling a distance of 18 inches, and uses five equal layers of soil instead of three. There are two types of compaction molds used for testing. The smaller type is 4 inches in diameter and has a volume of about 1/30 ft^3 (944 cm^3), and the larger type is 6 inches in diameter and has a volume of about 1/13.333 ft^3 (2123 cm^3). If the larger mold is used each soil layer must receive 56 blows instead of 25 (See Table 1).

Table 1 Alternative Proctor Test Methods

	Standard Proctor ASTM 698			Modified Proctor ASTM 1557		
	Method A	Method B	Method C	Method A	Method B	Method C
Material	≤ 20% Retained on No.4 Sieve	>20% Retained on No.4 ≤ 20% Retained on 3/8" Sieve	>20% Retained on No.3/8" <30% Retained on 3/4" Sieve	≤ 20% Retained on No.4 Sieve	>20% Retained on No.4 ≤ 20% Retained on 3/8" Sieve	>20% Retained on No.3/8" <30% Retained on 3/4" Sieve
For test sample, use soil passing	Sieve No.4	3/8" Sieve	¾" Sieve	Sieve No.4	3/8" Sieve	¾" Sieve
Mold	4" DIA	4" DIA	6" DIA	4" DIA	4" DIA	6" DIA
No. of Layers	3	3	3	5	5	5
No. of blows/layer	25	25	56	25	25	56

Note: Volume of 4" diameter mold = 944 cm^3 , Volume of 6" diameter mold = 2123 cm^3

(verify these values prior to testing)

Standard Reference:

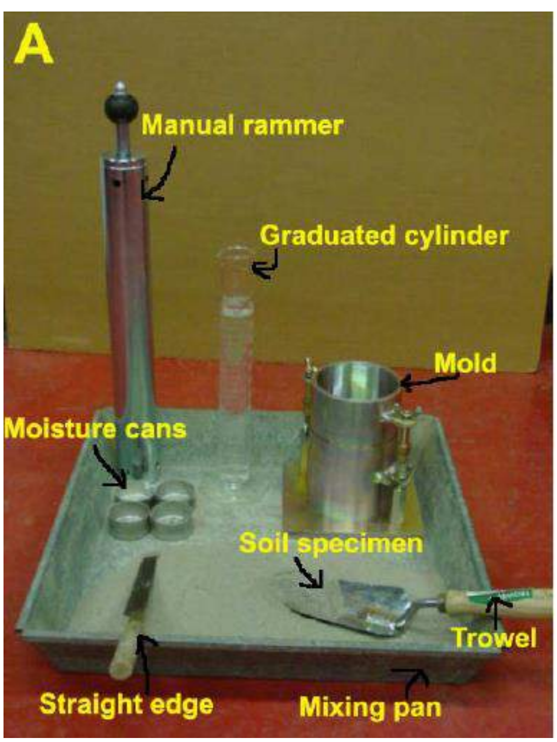
ASTM D 698 - Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Standard Effort (12,400 ft-lbs/ft^3 (600 KN-m/m^3)) ASTM D 1557 - Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Modified Effort (56,000 ft-lbs/ft^3 (2,700 KN-m/m^3))

Significance:

Mechanical compaction is one of the most common and cost effective means of stabilizing soils. An extremely important task of geotechnical engineers is the performance and analysis of field control tests to assure that compacted fills are meeting the prescribed design specifications. Design specifications usually state the required density (as a percentage of the “maximum” density measured in a standard laboratory test), and the water content. In general, most engineering properties, such as the strength, stiffness, resistance to shrinkage, and imperviousness of the soil, will improve by increasing the soil density. The optimum water content is the water content that results in the greatest density for a specified compactive effort. Compacting at water contents higher than (wet of) the optimum water content results in a relatively dispersed soil structure (parallel particle orientations) that is weaker, more ductile, less pervious, softer, more susceptible to shrinking, and less susceptible to swelling than soil compacted dry of optimum to the same density. The soil compacted lower than (dry of) the optimum water content typically results in a flocculated soil structure (random particle orientations) that has the opposite characteristics of the soil compacted wet of the optimum water content to the same density.

Equipment:

Molds, Manual rammer, Extruder, Balance, Drying oven, Mixing pan, Trowel, #4 sieve, Moisture cans, Graduated cylinder, Straight Edge.

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Test Procedure:

1. Depending on the type of mold you are using obtain a sufficient quantity of air-dried soil in large mixing pan. For the 4-inch mold take approximately 10 lbs, and for the 6-inch mold take roughly 15 lbs. pulverize the soil and run it through the # 4 sieve.
2. Determine the weight of the soil sample as well as the weight of the compaction mold with its base (without the collar) by using the balance and record the weights.
3. Compute the amount of initial water to add by the following method:
   (a) Assume water content for the first test to be 8 percent.
   (b) Compute water to add from the following equation:

$$\text{water to add (in ml)} = \frac{(\text{soil mass in grams})8}{100}$$

Where "water to add" and the "soil mass" are in grams. Remember that a gram of water is equal to approximately one milliliter of water.
4. Measure out the water, add it to the soil, and then mix it thoroughly into the soil using the trowel until the soil gets a uniform color (See Photos B and C).
5. Assemble the compaction mold to the base, place some soil in the mold and compact the soil in the number of equal layers specified by the type of compaction method employed (See Photos D and E). The number of drops of the rammer per layer is also dependent upon the type of mold used (See Table 1). The drops should be applied at a uniform rate not exceeding around 1.5 seconds per drop, and the rammer should provide uniform coverage of the specimen surface. Try to avoid rebound of the rammer from the top of the guide sleeve.
6. The soil should completely fill the cylinder and the last compacted layer must extend slightly above the collar joint. If the soil is below the collar joint at the completion of the drops, the test point must be repeated. (Note: For the last layer, watch carefully, and add more soil after about 10 drops if it appears that the soil will be compacted below the collar joint.)
7. Carefully remove the collar and trim off the compacted soil so that it is completely even with the top of the mold using the trowel. Replace small bits of soil that may fall out during the trimming process (See Photo F).
8. Weigh the compacted soil while it's in the mold and to the base, and record the mass (See Photo G). Determine the wet mass of the soil by subtracting the weight of the mold and base.
9. Remove the soil from the mold using a mechanical extruder (See
Photo H) and take soil moisture content samples from the top and bottom of the specimen (See Photo I). Fill the moisture cans with soil and determine the water content.
10. Place the soil specimen in the large tray and break up the soil until it appears visually as if it will pass through the # 4 sieve, add 2 percent more water based on the original sample mass, and re-mix as in step 4. Repeat steps 5 through 9 until, based on wet mass, a peak value is reached followed by two slightly lesser compacted soil masses.

Analysis:

1. Calculate the moisture content of each compacted soil specimen by using the average of the two water contents.
2. Compute the wet density in grams per cm3 of the compacted soil sample by dividing the wet mass by the volume of the mold used.
3. Compute the dry density using the wet density and the water content determined in step 1. Use the following formula:

$$\rho_d = \frac{\rho}{1 + w}$$

Where: w = moisture content in percent divided by 100, and ρ = wet density in grams per cm3.
4. Plot the dry density values on the y-axis and the moisture contents on the x-axis. Draw a smooth curve connecting the plotted points.
5. On the same graph draw a curve of complete saturation or "zero air voids curve". The values of dry density and corresponding moisture contents for plotting the curve can be computed from the following equation:

$$W_{sat} = \left(\frac{\rho_w}{\rho_d} - \frac{1}{G_s} \right) \times 100 \qquad \text{or} \qquad \rho_d = \frac{\rho_w}{\left(\frac{w}{100} + \frac{1}{Gs} \right)}$$

Where:

ρ_d = dry density of soil grams per cm^3
G_s = specific gravity of the soil being tested (assume 2.70 if not given)
ρ_w = density of water in grams per cm3 (approximately1 g/cm^3)
w_{sat} = moisture content in percent for complete saturation.
Example Calculations:
G_s=2.7 (assumed)
ρ_w=1.0 g/cm^3

Assumed w_{sat}%	Calculated ρ_d (g/cm^3)
8	2.22
10	2.13
12	2.04
14	1.96
16	1.89
18	1.82

6. Identify and report the optimum moisture content and the maximum dry density. Make sure that you have recorded the method of compaction used (e.g., Standard Proctor, Method A) on data sheet.

Example Data

Moisture-Density (Compaction) Test

Data Sheets

Test Method: Standard Proctor, Method A (ASTM 698)

Date Tested: October 05, 2002

Tested By: CEMM315 Class, Group A

Project Name: CEMM315 Lab

Sample Number: Bag-1, 2'-6'

Visual Classification of Soil: Gray silty clay, trace fine sand, low plasticity, moist, CL

Water Content Determination:

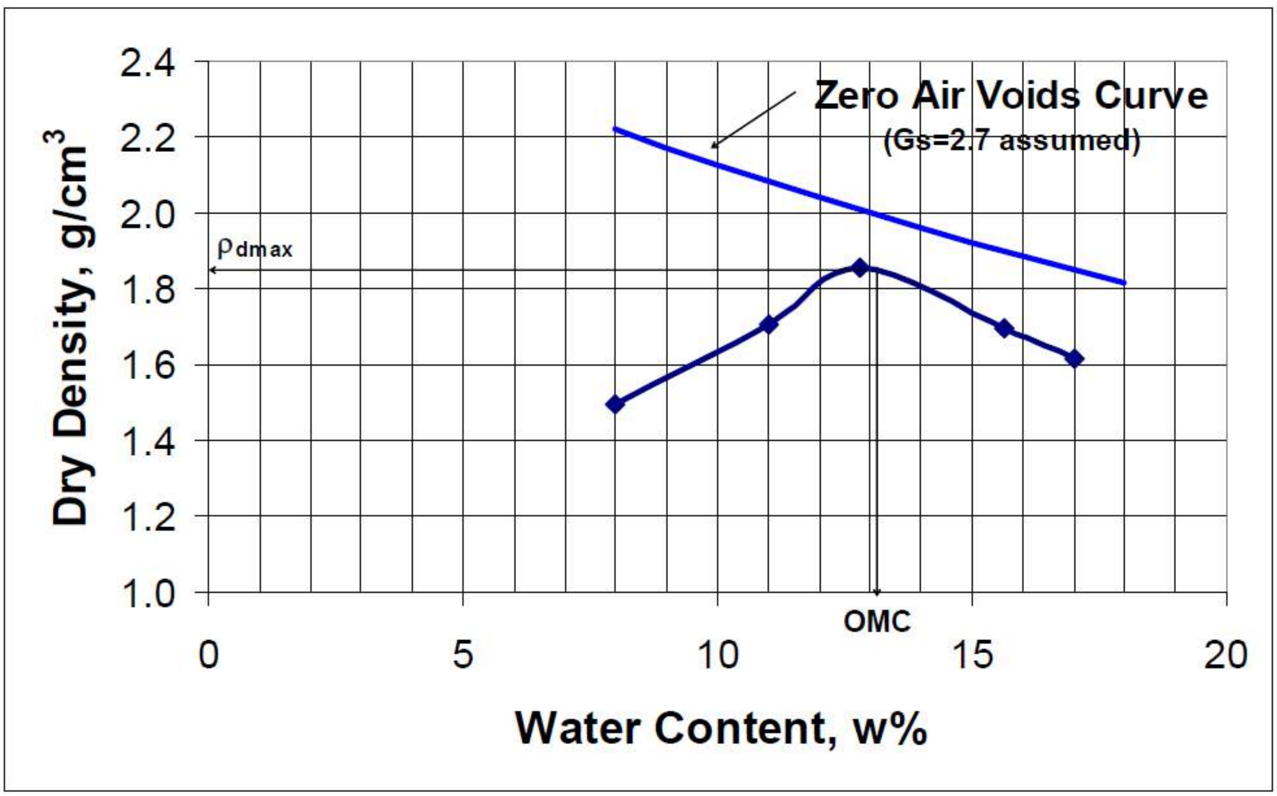
Compacted Soil - Sample no.	1		2		3	
Water content - Sample no.	1A	1B	2A	2B	3A	3B
Moisture can number - Lid number	1	2	3	4	5	6
M_C = Mass of empty, clean can + lid (grams)	7.78	7.83	7.71	7.9	7.5	7.9
M_{CMS} = Mass of can, lid, and moist soil (grams)	11.78	11.05	10.71	11 . 1	10.7	12 .0
M_{CDS} = Mass of can, lid, and dry soil (grams)	11.48	10.81	10.41	10.75	10.3	11.52
M_S = Mass of soil solids (grams)	3.70	2.98	2.7	2.85	2.84	3.62
M_W = Mass of pore water (grams)	0.29	0.24	0.30	0.35	0.40	0.53
w = Water content, w%	7.9	8.1	11 . 1	10.9	12.5	13.1

Compacted Soil - Sample no.	4		5		6	
Water content - Sample no.	4A	4B	5A	5B	6A	6B
Moisture can number - Lid number	7	8	9	10		
M_C = Mass of empty, clean can + lid (grams)	8.1	7.6	7.7	7.65		
M_{CMS} = Mass of can, lid, and moist soil (grams)	11. 1	10.2	10.3	10.33		
M_{CDS} = Mass of can, lid, and dry soil (grams)	10.70	9.84	10.02	9.92		
M_S = Mass of soil solids (grams)	2.60	2.24	2.32	2.27		
M_W = Mass of pore water (grams)	0.40	0.35	0.4	0.39		
w = Water content, w%	15.3	16.0	17. 1	17.6		

Density Determination:

Mold Volume=944 cm^3

Compacted Soil - Sample no.	1	2	3	4	5	6
w = Assumed water content, w%	10	12	14	16	18	
Actual average water content, w%	8.0	11.0	12.8	15.65	17	
Mass of compacted soil and mold (grams)	3457.2	3721.2	3909.0	3782.5	3715.2	
Mass of mold (grams)	1933	1933	1976.0	1849.5	1782.2	
Wet mass of soil in mold (grams)	1524.2	1788.2	2176	2149	2082	
Wet density, ρ, (g/cm^3)	1.615	1.894	2.093	1.959	1.888	
Dry density, ρ_d, (g/cm^3)	1.50	1.71	1.86	1.69	1.61	



Optimum Moisture Content = 13.1 %

Maximum Dry Density = 1.87 g/cm^3

Moisture-Density (Compaction) Test

Data Sheets

Test Method:

Date Tested:

Tested By:

Project Name:

Sample Number:

Visual Classification of Soil:

Water Content Determination:

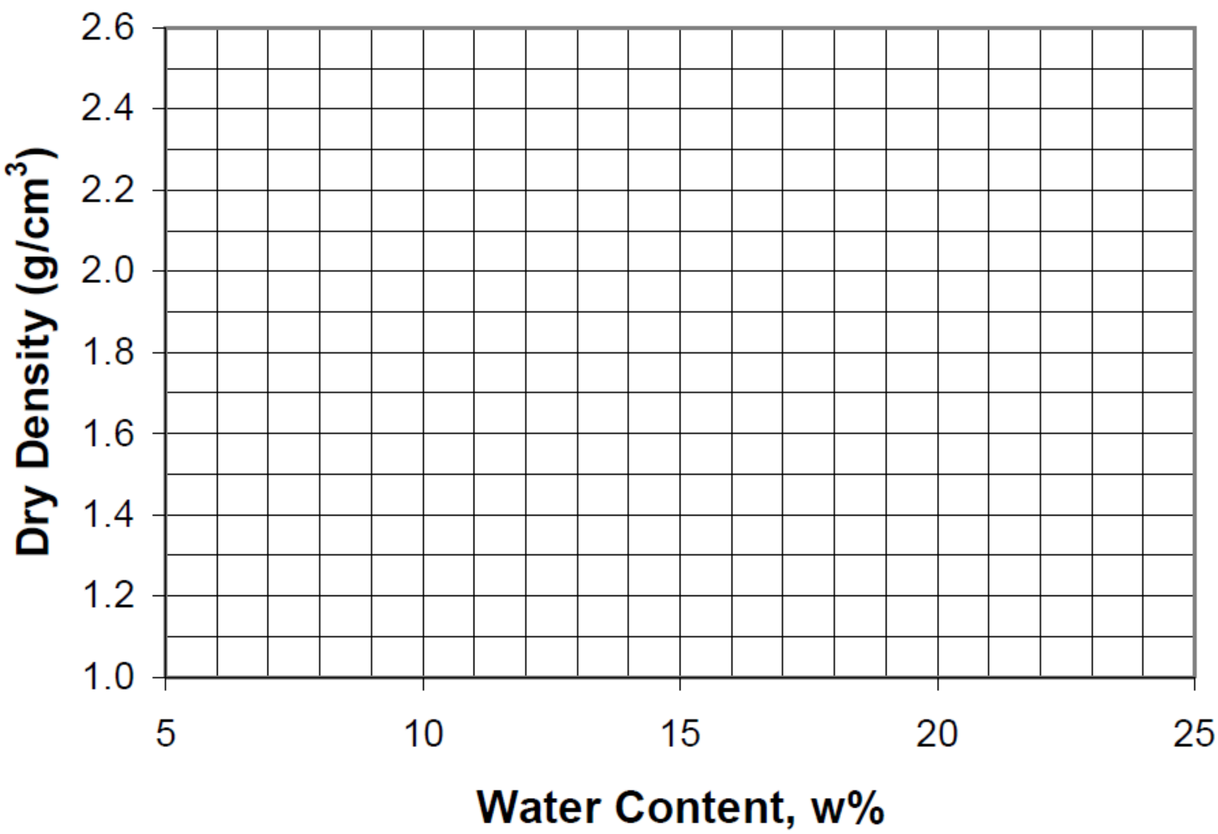
Compacted Soil - Sample no.	1		2		3	
Water content - Sample no.	1A	1B	2A	2B	3A	3B
Moisture can number - Lid number						
M_C = Mass of empty, clean can + lid (grams)						
M_{CMS} = Mass of can, lid, and moist soil (grams)						
M_{CDS} = Mass of can, lid, and dry soil (grams)						
M_S = Mass of soil solids (grams)						
M_W = Mass of pore water (grams)						
W = Water content, w%						

Compacted Soil - Sample no.	4		5		6	
Water content - Sample no.	4A	4B	5A	5B	6A	6B
Moisture can number - Lid number						
M_C = Mass of empty, clean can + lid (grams)						
M_{CMS} = Mass of can, lid, and moist soil (grams)						
M_{CDS} = Mass of can, lid, and dry soil (grams)						
M_S = Mass of soil solids (grams)						
M_W = Mass of pore water (grams)						
W = Water content, w%						

Density Determination:

Volume of mold=

Compacted Soil - Sample no.	1	2	3	4	5	6
w = Assumed water content, w%						
Actual average water content, w%						
Mass of compacted soil and mold (grams)						
Mass of mold (grams)						
Wet mass of soil in mold (grams)						
Wet density, ρ , (kg/m^3)						
Dry density, ρ_d , (kg/m^3)						



Optimum Moisture Content = ________________ %

Maximum Dry Density = __________________ g/cm^3

10.Permeability (Hydraulic Conductivity) Test Constant Head Method

Purpose:

The purpose of this test is to determine the permeability (hydraulic conductivity) of a sandy soil by the constant head test method. There are two general types of permeability test methods that are routinely performed in the laboratory: (1) the constant head test method, and (2) the falling head test method. The constant head test method is used for permeable soils (k>10-4 cm/s) and the falling head test is mainly used for less permeable soils (k<10-4 cm/s).

Standard Reference:

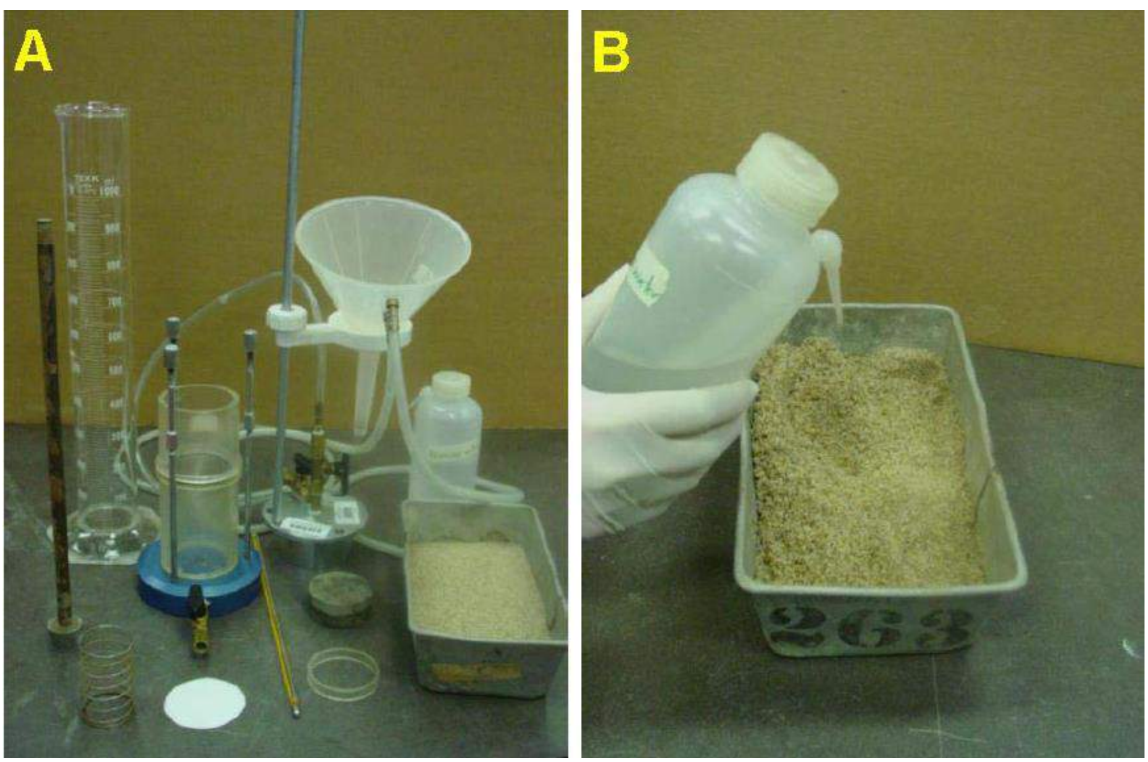
ASTM D 2434 - Standard Test Method for Permeability of Granular Soils (Constant Head)
(Note: The Falling Head Test Method is not standardized)

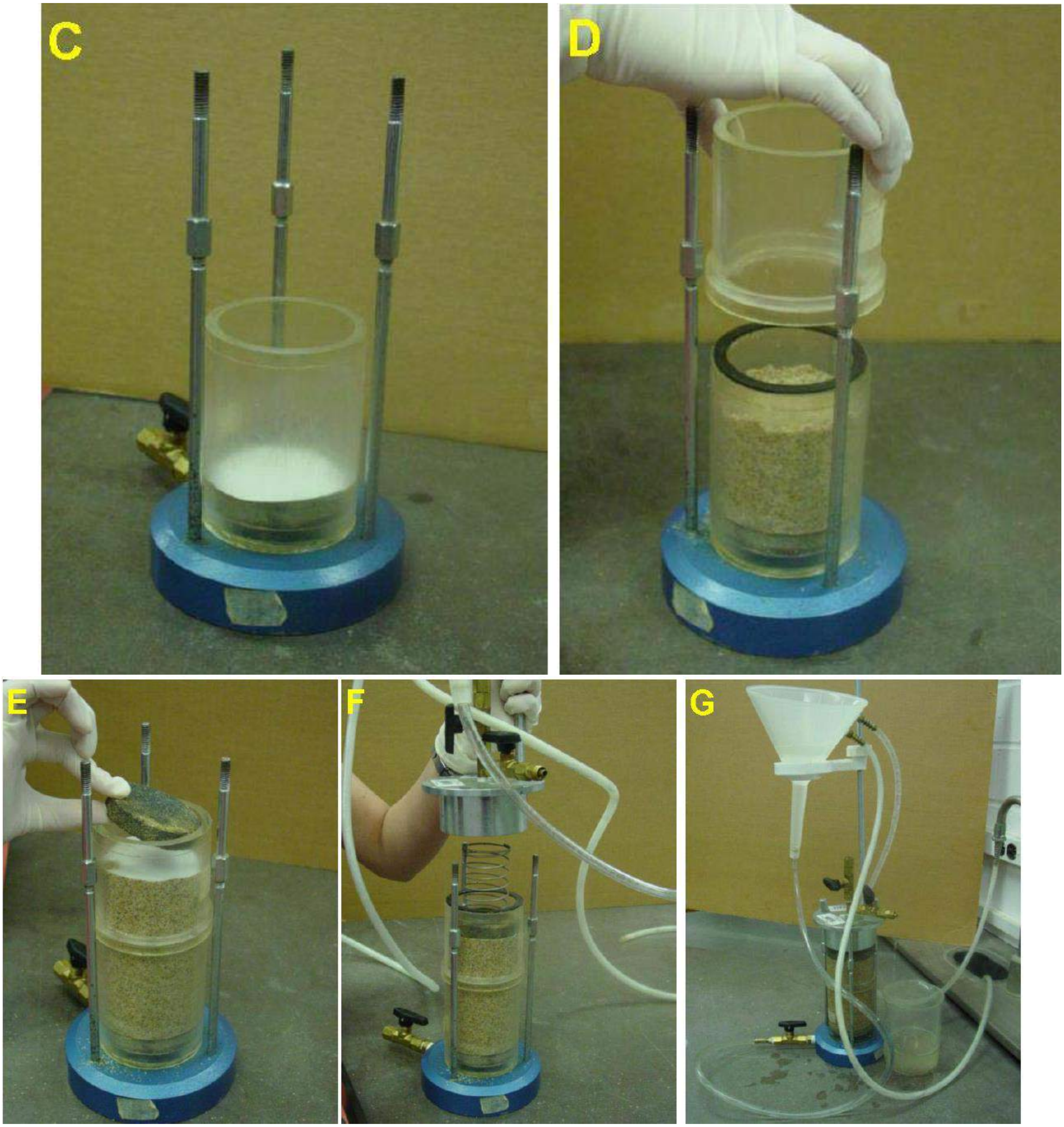
Significance:

Permeability (or hydraulic conductivity) refers to the ease with which water can flow through a soil. This property is necessary for the calculation of seepage through earth dams or under sheet pile walls, the calculation of the seepage rate from waste storage facilities (landfills, ponds, etc.), and the calculation of the rate of settlement of clayey soil deposits.

Equipment:

Permeameter, Tamper, Balance, Scoop, 1000 mL Graduated cylinders, Watch (or Stopwatch), Thermometer, Filter paper.

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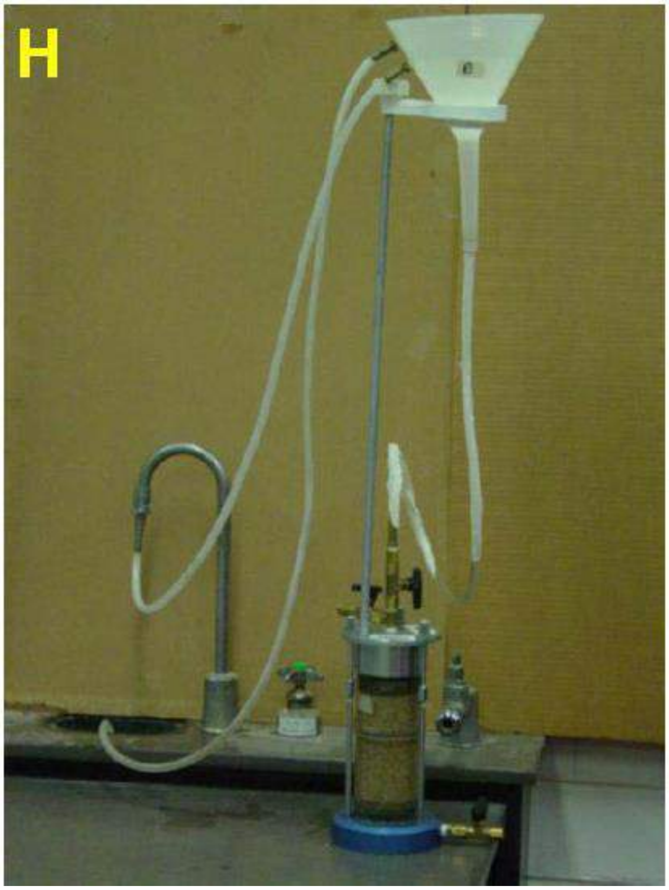


***Test Procedure**:*

a. Measure the initial mass of the pan along with the dry soil (M_1).
b. Remove the cap and upper chamber of the permeameter by unscrewing the knurled cap nuts and lifting them off the tie rods. Measure the inside diameter of upper and lower chambers. Calculate the average inside diameter of the permeameter (D).
c. Place one porous stone on the inner support ring in the base of the chamber then place a filter paper on top of the porous stone (see Photo C).
d. Mix the soil with a sufficient quantity of distilled water to prevent the segregation of particle sizes during placement into the permeameter. Enough water should be added so that the mixture may flow freely (see Photo B).
e. Using a scoop, pour the prepared soil into the lower chamber using a circular motion to fill it to a depth of 1.5 cm. A uniform layer should be formed.
f. Use the tamping device to compact the layer of soil. Use approximately ten rams of the tamper per layer and provide uniform coverage of the soil surface. Repeat the compaction procedure until the soil is within 2 cm. of the top of the lower chamber section (see Photo D).
g. Replace the upper chamber section, and don't forget the rubber gasket that goes between the chamber sections. Be careful not to disturb the soil that has already been compacted. Continue the placement operation until the level of the soil is about 2 cm. below the rim of the upper chamber. Level the top surface of the soil and place a filter paper and then the upper porous stone on it (see Photo E).

h. Place the compression spring on the porous stone and replace the chamber cap and its sealing gasket. Secure the cap firmly with the cap nuts (see Photo F).
i. Measure the sample length at four locations around the circumference of the permeameter and compute the average length. Record it as the sample length.
j. Keep the pan with remaining soil in the drying oven.
k. Adjust the level of the funnel to allow the constant water level in it to remain a few inches above the top of the soil.
l. Connect the flexible tube from the tail of the funnel to the bottom outlet of the permeameter and keep the valves on the top of the permeameter open (see Photo G).
m. Place tubing from the top outlet to the sink to collect any water that may come out (see Photo G).
n. Open the bottom valve and allow the water to flow into the permeameter.
o. As soon as the water begins to flow out of the top control (deairing) valve, close the control valve, letting water flow out of the outlet for some time.
p. Close the bottom outlet valve and disconnect the tubing at the bottom. Connect the funnel tubing to the top side port (see Photo H).
q. Open the bottom outlet valve and raise the funnel to a convenient height to get a reasonable steady flow of water.
r. Allow adequate time for the flow pattern to stabilize (see Photo I).
s. Measure the time it takes to fill a volume of 750 - 1000 mL using the graduated cylinder, and then measure the temperature of the water. Repeat this process three times and compute the average time, average volume, and average temperature. Record the values as t, Q, and T, respectively (see Photo I).
t. Measure the vertical distance between the funnel head level and the chamber outflow level, and record the distance as h.
u. Repeat step 17 and 18 with different vertical distances.
v. Remove the pan from the drying oven and measure the final mass of the pan along with the dry soil (M_2).

***Analysis*:**

1. Calculate the permeability, using the following equation:

$$K_T = \frac{QL}{Ath}$$

Where:

K_T = coefficient of permeability at temperature T, cm/sec.
L = length of specimen in centimeters
t = time for discharge in seconds
Q = volume of discharge in cm^3 (assume 1 mL = 1 cm^3)

A = cross-sectional area of permeameter (= $\frac{\pi}{4}D^2$, D= inside diameter of the permeameter)

h = hydraulic head difference across length L, in cm of water; or it is equal to the vertical distance between the constant funnel head level and the chamber overflow level.

2. The viscosity of the water changes with temperature. As temperature increases viscosity decreases and the permeability increases. The coefficient of permeability is standardized at 20°C, and the permeability at any temperature T is related to K_{20} by the following ratio:

$$K_{20} = K_T \frac{\eta_T}{\eta_{20}}$$

Where:

η_T and η_{20} are the viscosities at the temperature T of the test and at 20° C, respectively. From Table 1 obtain the viscosities and compute K_{20}.

3. Compute the volume of soil used from: V = LA.

4. Compute the mass of dry soil used in permeameter (M) = initial mass - final mass:

$$M = M_1 - M_2$$

5. Compute the dry density (ρ_d) of soil

$$\rho_d = \frac{M}{V}$$

Table 1. Properties of Distilled Water (η = absolute)

Temperature $^\circ C$	Density (g/cm^3)	Viscosity (Poise*)
4	1.00000	0.01567
16	0.99897	0.01111
17	0.99880	0.01083
18	0.99862	0.01056
19	0.99844	0.01030
20	0.99823	0.01005
21	0.99802	0.00981
22	0.99780	0.00958
23	0.99757	0.00936
24	0.99733	0.00914
25	0.99708	0.00894
26	0.99682	0.00874
27	0.99655	0.00855
28	0.99627	0.00836
29	0. 99598	0.00818
30	0. 99568	0.00801

$$\text{*Poise} = \frac{dyne \cdot s}{cm^2} = \frac{g}{cm \cdot s}$$

Example Data
HYDRAULIC CONDUCTIVITY TEST CONSTANT HEAD METHOD DATA SHEET

Date Tested: October 10, 2002

Tested By: CEMM315 Class, Group A

Project Name: CEMM315 Lab

Sample Number: B-1, ST-10, 8'-10'

Visual Classification: Brown medium to fine sand, poorly graded, subrounded, dry.

Initial Dry Mass of Soil + Pan (M_1) = 1675.0 g
Length of Soil Specimen, L = 17 cm
Diameter of the Soil Specimen (Permeameter), D = 6.4 cm
Final Dry Mass of Soil + Pan (M_2) = 865.6 g
Dry Mass of Soil Specimen (M) = 809.4 g
Volume of Soil Specimen (V) = 846.9 cm^3
Dry Density of Soil (ρ_d) = 1.48 g/cm^3

Trial Number	Constant Head, h (cm)	Elapsed Time, t (seconds)	Outflow Volume, Q (cm^3)	Water Temp., T (°C)	K_T cm/sec	K_{20} cm/sec
1	30	84	750	22	0.157	0.149
2	50	55	750	22	0.144	0.137
3	60	48	750	22	0.137	0.130
4	70	38	750	22	0.149	0.142

Average K_{20}= 0.139 cm/sec

HYDRAULIC CONDUCTIVITY TEST

CONSTANT HEAD METHOD

DATA SHEET

Date Tested:

Tested By:

Project Name:

Sample Number:

Visual Classification:

Initial Dry Mass of Soil + Pan (M_1) = ___________________ g
Length of Soil Specimen, L = __________________ cm
Diameter of the Soil Specimen (Permeameter), D = ______________ cm
Final Dry Mass of Soil + Pan (M_2) = __________________ g
Dry Mass of Soil Specimen (M) = ______________________ g
Volume of Soil Specimen (V) = ____________________ cm^3
Dry Density of Soil (ρ_d) = ___________________ g/cm^3

Trial Number	Constant Head, h (cm)	Elapsed Time, t (seconds)	Outflow Volume, Q (cm^3)	Water Temp., T (°C)	K_T	K_{20}
1						
2						
3						
4						

Average K_{20}= _______________________ cm/sec

11. Consolidation

Consolidation is a process by which soils decrease in volume. It occurs when stress is applied to a soil that causes the soil particles to pack together more tightly, therefore reducing volume. When this occurs in a soil that is saturated with water, water will be squeezed out of the soil. The magnitude of consolidation can be predicted by many different methods. In the Classical Method, developed by Karl Terzaghi, soils are tested with an oedometer test to determine their compression index. This can be used to predict the amount of consolidation.

When stress is removed from a consolidated soil, the soil will rebound, regaining some of the volume it had lost in the consolidation process. If the stress is reapplied, the soil will consolidate again along a recompression curve, defined by the recompression index. The soil which had its load removed is considered to be overconsolidated. This is the case for soils which have previously had glaciers on them. The highest stress that it has been subjected to is termed the preconsolidation stress. A soil which is currently experiencing its highest stress is said to be normally consolidated.

Purpose:

This test is performed to determine the magnitude and rate of volume decrease that a laterally confined soil specimen undergoes when subjected to different vertical pressures. From the measured data, the consolidation curve (pressure-void ratio relationship) can be plotted. This data is useful in determining the compression index, the recompression index and the preconsolidation pressure (or maximum past pressure) of the soil. In addition, the data obtained can also be used to determine the coefficient of consolidation and the coefficient of secondary compression of the soil.

Standard Reference:

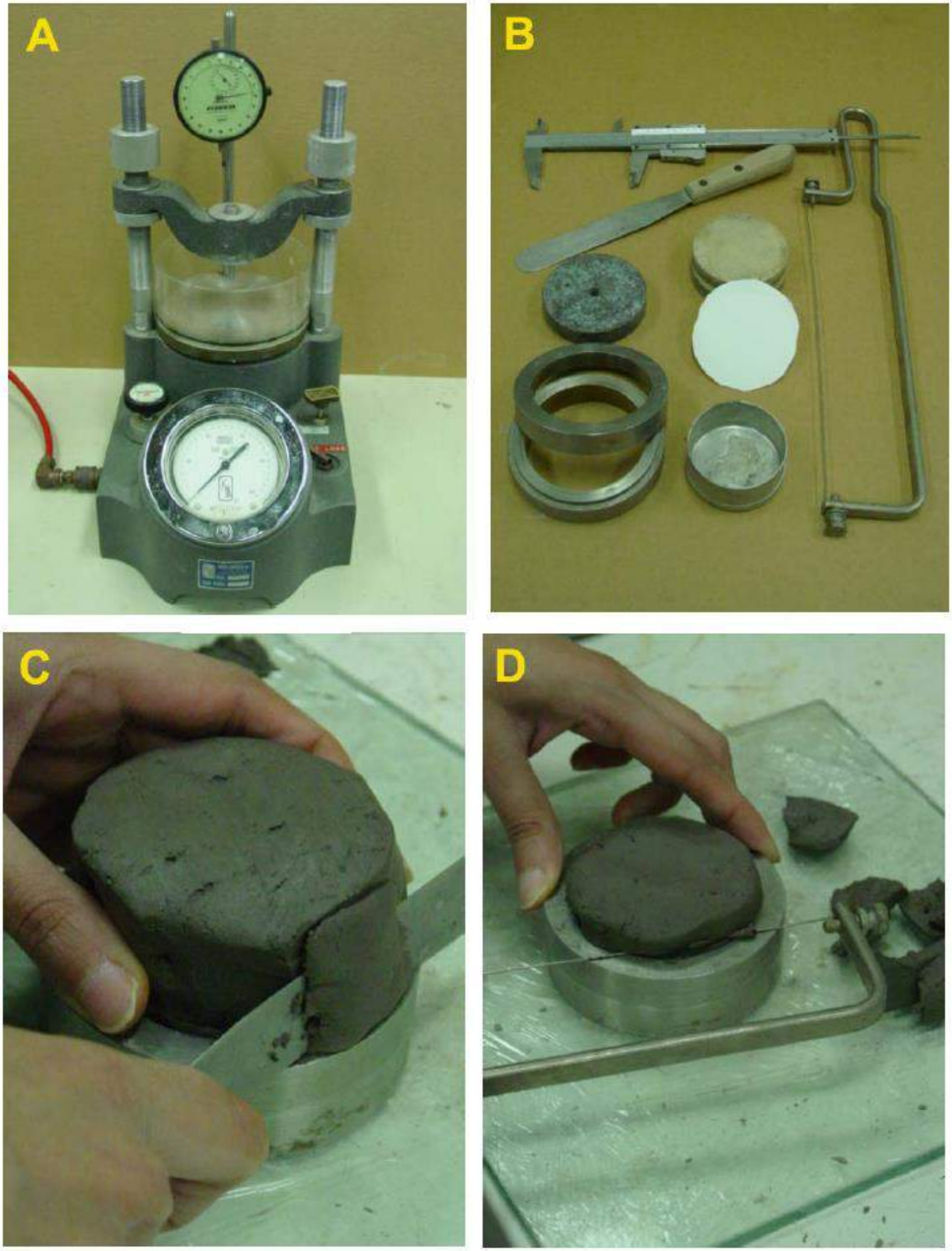
ASTM D 2435 - Standard Test Method for One-Dimensional Consolidation Properties of Soils

Significance:

The consolidation properties determined from the consolidation test are used to estimate the magnitude and the rate of both primary and secondary consolidation settlement of a structure or an earth-fill. Estimates of this type are of key importance in the design of engineered structures and the evaluation of their performance.

Equipment:

Consolidation device (including ring, porous stones, water reservoir, and load plate), Dial gauge (0.0001 inch = 1.0 on dial), Sample trimming device, glass plate, Metal straight edge, Clock, Moisture can, Filter paper.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Engineering Properties of Soils Based on Laboratory Testing by Prof. Krishna Reddy, UIC

Test Procedure:

1. Weigh the empty consolidation ring together with glass plate.
2. Measure the height (h) of the ring and its inside diameter (d).
3. Extrude the soil sample from the sampler, generally thin-walled Shelby tube. Determine the initial moisture content and the specific gravity of the soil as per Experiments 1 and 4, respectively (Use the data sheets from these experiments to record all of the data).
4. Cut approximately a three-inch long sample. Place the sample on the consolidation ring and cut the sides of the sample to be approximately the same as the outside diameter of the ring. Rotate the ring and pare off the excess soil by means of the cutting tool so that the sample is reduced to the same inside diameter of the ring. It is important to keep the cutting tool in the correct horizontal position during this process.
5. As the trimming progresses, press the sample gently into the ring and continue until the sample protrudes a short distance through the bottom of the ring. Be careful throughout the trimming process to insure that there is no void space between the sample and the ring.
6. Turn the ring over carefully and remove the portion of the soil protruding above the ring. Using the metal straight edge, cut the soil surface flush with the surface of the ring. Remove the final portion with extreme care.
7. Place the previously weighed Saran-covered glass plate on the freshly cut surface, turn the ring over again, and carefully cut the other end in a similar manner.
8. Weigh the specimen plus ring plus glass plate.
9. Carefully remove the ring with specimen from the Saran-covered glass plate and peel the Saran from the specimen surface. Center the porous stones that have been soaking, on the top and bottom surfaces of the test specimen. Place the filter papers between porous stones and soil specimen. Press very lightly to make sure that the stones adhere to the sample. Lower the assembly carefully into the base of the water reservoir. Fill the water reservoir with water until the specimen is completely covered and saturated.
10. Being careful to prevent movement of the ring and porous stones, place the load plate centrally on the upper porous stone and adjust the loading device.
11. Adjust the dial gauge to a zero reading.
12. With the toggle switch in the down (closed) position, set the pressure gauge dial (based on calibration curve) to result in an applied pressure of 0.5 tsf (tons per square foot).
13. Simultaneously, open the valve (by quickly lifting the toggle switch to the up (open) position) and start the timing clock.
14. Record the consolidation dial readings at the elapsed times given on the data sheet.
15. Repeat Steps 11 to 13 for different preselected pressures (generally includes loading pressures of 1.0, 2.0, 4.0, 8.0, and 16.0 tsf and unloading pressures of 8.0, 4.0, 2.0, 1.0 and 0.5 tsf)
16. At the last elapsed time reading, record the final consolidation dial reading and time, release the load, and quickly disassemble the consolidation device and remove the specimen. Quickly but carefully blot the surfaces dry with paper toweling. (The specimen will tend to absorb water after the load is released.)
17. Place the specimen and ring on the Saran-covered glass plate and, once again, weigh them together.

18. Weigh an empty large moisture can and lid.
19. Carefully remove the specimen from the consolidation ring, being sure not to lose too much soil, and place the specimen in the previously weighed moisture can. Place the moisture can containing the specimen in the oven and let it dry for 12 to 18 hours.
20. Weigh the dry specimen in the moisture can.

Analysis:

1. Calculate the initial water content and specific gravity of the soil.
2. For each pressure increment, construct a semilog plot of the consolidation dial readings versus the log time (in minutes). Determine D_0, D_{50}, D_{100}, and the coefficient of consolidation (c_v) using Casagrande's logarithm of time fitting method. See example data. Also calculate the coefficient of secondary compression based on these plots.
3. Calculate the void ratio at the end of primary consolidation for each pressure increment (see example data). Plot log pressure versus void ratio. Based on this plot, calculate compression index, recompression index and preconsolidation pressure (maximum past pressure).
4. Summarize and discuss the results.

Example Data

Date Tested: October 05, 2002
Tested By: CEMM315 Class, Group A
Project Name: CEMM315 Lab
Sample Number: GB-08-ST-13'-15'
Visual Classification: Gray silty clay

<u>Before test</u>

Consolidation type	= Floating type
Mass of the ring + glass plate	= 465.9 g
Inside diameter of the ring	= 6.3 cm
Height of specimen, H_i	= 2.7 cm
Area of specimen, A	= 31.172 cm^2
Mass of specimen + ring	= 646.4 g
Initial moisture content of specimen, w_i (%)	= 19.5 %
Specific gravity of solids, G_s	= 2.67

After test

Mass of wet sample + ring + glass plate = 636.5 g
Mass of can = 59.3 g
Mass of can + wet soil = 229.8 g
Mass of wet specimen = 170.50 g
Mass of can + dry soil = 208.5 g
Mass of dry specimen, M_s = 149.2 g
Final moisture content of specimen, w_f = 14.27 %

Calculations

Mass of solids in specimen, M_s = 149.2 g
(Mass of dry specimen after test)

Mass of water in specimen before test, M_{wi} = w_i x M_s
= 0.195*149.2 = 29.094 g

Mass of water in specimen after test, M_{wf} (g) = w_f x M_s
= 0.1427*149.2 = 21.29 g

Height of solids, H_s = $\frac{M_s}{A \times G_s \times \rho_w} = \frac{149.2}{31.172 \times 2.67 \times 1}$ = 1.792 cm
(same before and after test and note ρ_w = 1 g/cm^3)

Height of water before test, H_{wi} = $\frac{M_{wi}}{A \times \rho_w} = \frac{29.09}{31.172 \times 1}$ = 0.933 cm

Height of water after test, H_{wf} = $\frac{M_{wf}}{A \times \rho_w} = \frac{21.29}{31.172 \times 1}$ = 0.683 cm

Change in height of specimen after test, $\Sigma\Delta H$ = 0.257 cm
($\Sigma\Delta H$ for all pressures – see t vs Dial Reading plots)

Height of specimen after test, $H_f = H_i - \Sigma\Delta H$ = 2.7-0.257 = 2.443 cm

Void ratio before test, $e_o = \frac{H_i - H_s}{H_s} = \frac{2.7 - 1.792}{1.792}$ = 0.506

Void ratio after test, $e_f = \frac{H_f - H_s}{H_s} = \frac{2.443 - 1.792}{1.792}$ = 0.3617

Degree of saturation before test, $S_i = \dfrac{H_{wi}}{H_i - H_s} = \dfrac{0.933}{2.7 - 1.792} \times 100$

$$= 102.7\ \%$$

Degree of saturation after test, $S_f = \dfrac{H_{wf}}{H_f - H_s} = \dfrac{0.683}{2.443 - 1.792} \times 100$

$$= 105.08\%$$

Dry density before test, $\rho_d = \dfrac{M_s}{H_i \times A} = \dfrac{149.2}{2.7 \times 31.172} = 1.77\ g/cm^3$

$$= (110.6\ pcf)$$

Table 1: Time - Settlement Data (1 unit on dial guage = 0.0001 inches)

loading= ¼ tsf		loading=1/8 tsf		loading=1/2 tsf		loading=1 tsf	
time	dail reading	time	dail reading	time	dail reading	time	dail reading
0	0	0	0	0	0	0	0
0.1	0	0.1	0	0.1	13	0.1	6
0.25	0	0.25	0	0.25	18	0.25	8
0.5	0	0.5	0	0.5	25	0.5	11.5
1		1		1	34	1	15
2		2		2	40	2	20.5
4		4		4	54	4	27
8		8		8	77	10	42
15		15		15	90	15	46
30		30		30	126	31	58
60		60		60	144.5	60	79
120		120		130	160	121	81
				300	162	240	85
				1380	169	562	86

loading=2 tsf	
time	dail reading
0	255
0.1	255.5
0.25	256
0.5	256.5
1	257
2	257.5
4	258
8	258.5
15	262.5
30	283
60	286
128	292.5
240	297
335	299
390	300
678	303
1380	303.5
1520	304

loading=4 tsf	
time	dail reading
0	313
0.06	319
0.15	328
0.3	336
1	357
2	375
4	398
8	428
15	453
30	464
60	472.5
120	479.5
290	486
395	488
1230	496

loading=2 tsf (unloading)	
time	dail reading
0	496
0.1	496
0.25	496
0.5	495.5
1	495
2	494
4	493.5
8	493
15	492.5
30	492.5
70	492.5
140	492.5
215	492.5

loading=1 tsf (unloading)	
time	dail reading
0	492.5
0.1	492.5
0.25	492.5
0.5	492
1	490.5
2	486.5
4	481.5
8	477.5
15	474.5
44	472.5
60	471.5
218	470.5

loading=1/2 tsf (unloading)	
time	dail reading
0	470.5
0.06	469.5
0.5	466
1	464.5
2	461.5
4	458.5
8	454
15	450.5
30	447
60	444.5
110	443.5
930	440.5

loading=1 tsf (reloading)	
time	dail reading
0	440.5
0.1	440.7
0.25	441
0.5	441.2
1	441.5
2	441.6
4	441.8
8	442
15	442.1
30	442.4
60	442.4
120	442.4

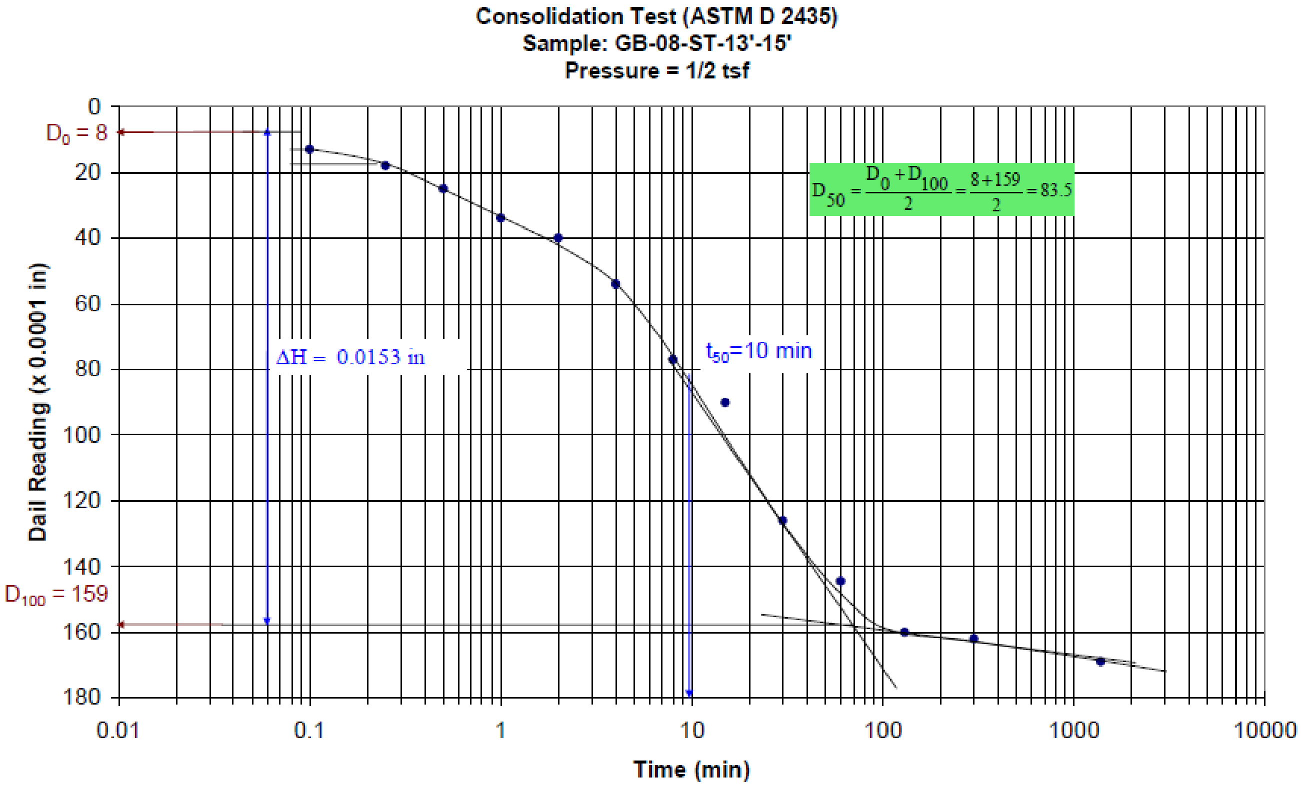
loading=2 tsf (reloading)	
time	dail reading
0	442.4
0.1	442.9
0.25	443.4
0.5	444.4
1	445.1
2	445.3
4	445.4
8	445.9
15	446.3
30	446.4
60	446.5
120	446.5

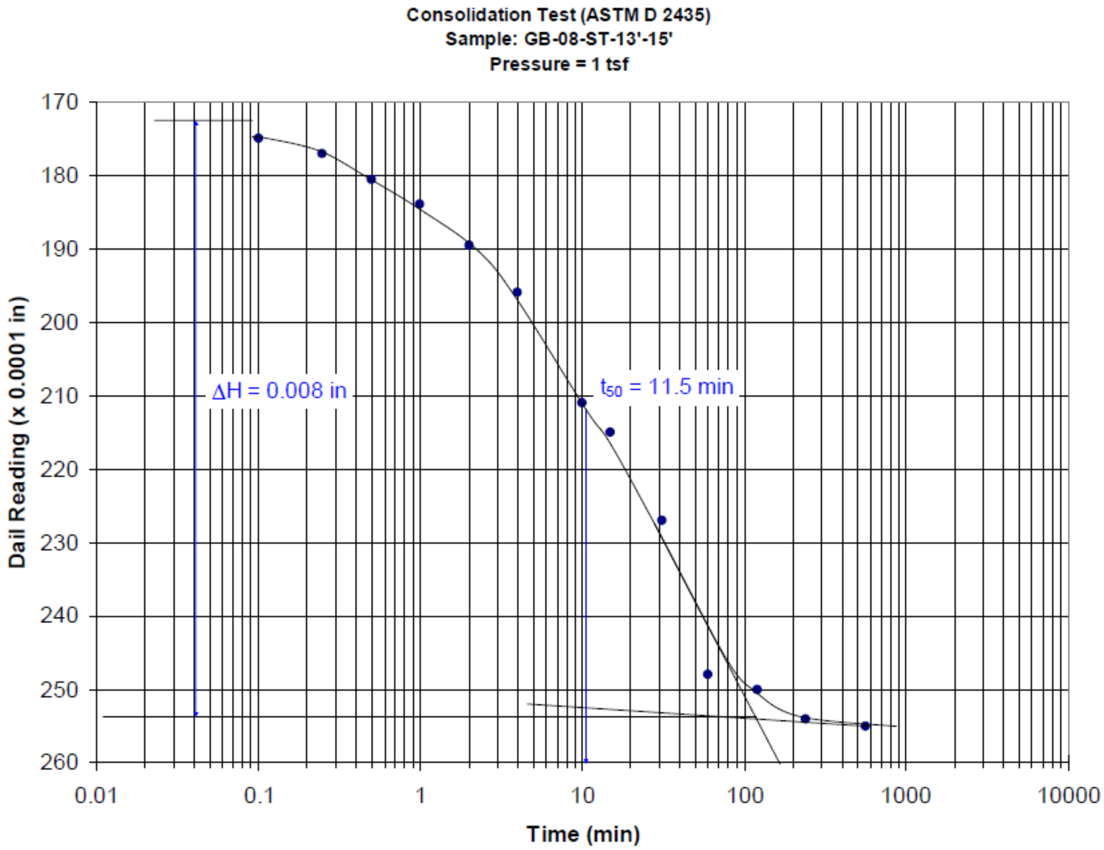
loading=4 tsf (reloading)	
time	dail reading
0	446.5
0.1	446.5
0.25	446.6
0.5	449.5
1	456.5
2	465.5
4	473.5
8	481
17	485.5
30	488
108	490.5
947	500

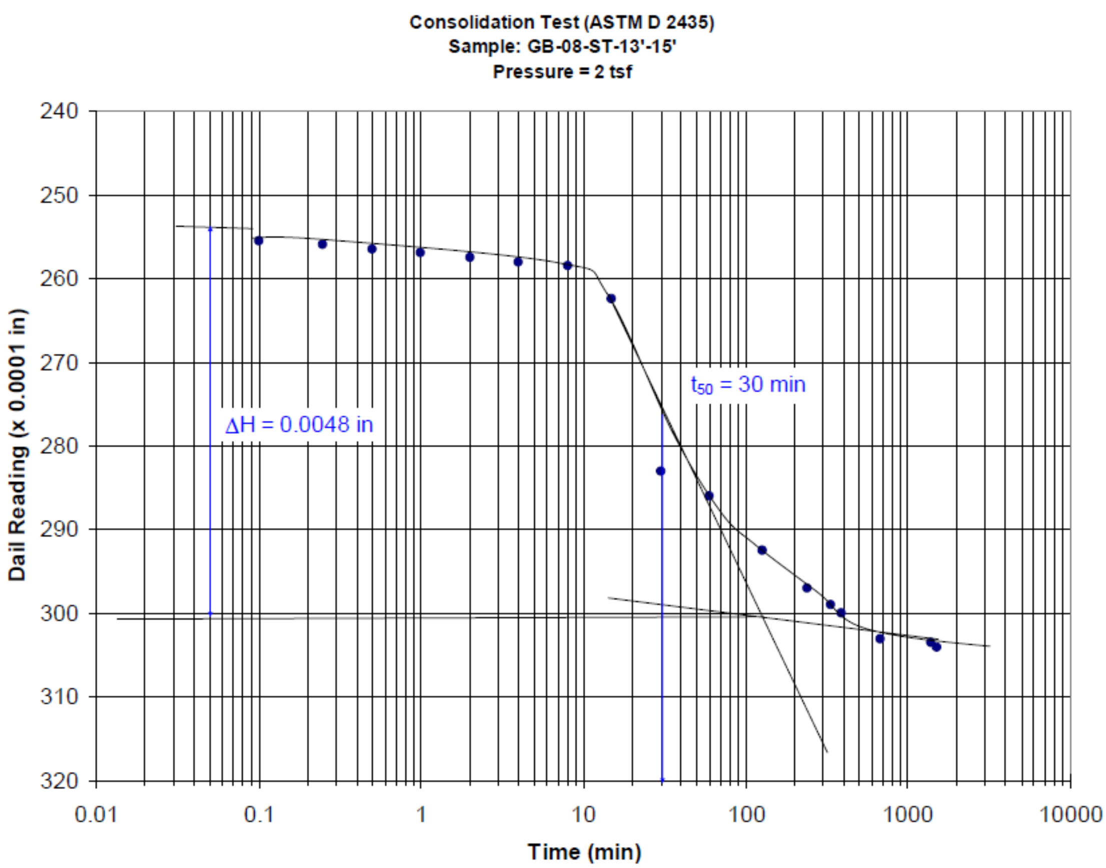
loading=8 tsf	
time	dail reading
0	500
0.1	510
0.25	518
0.5	528
1	542
2	561.5
4	580
8	604
15	619.5
30	631.8
60	640
127	642
205	651
228	652

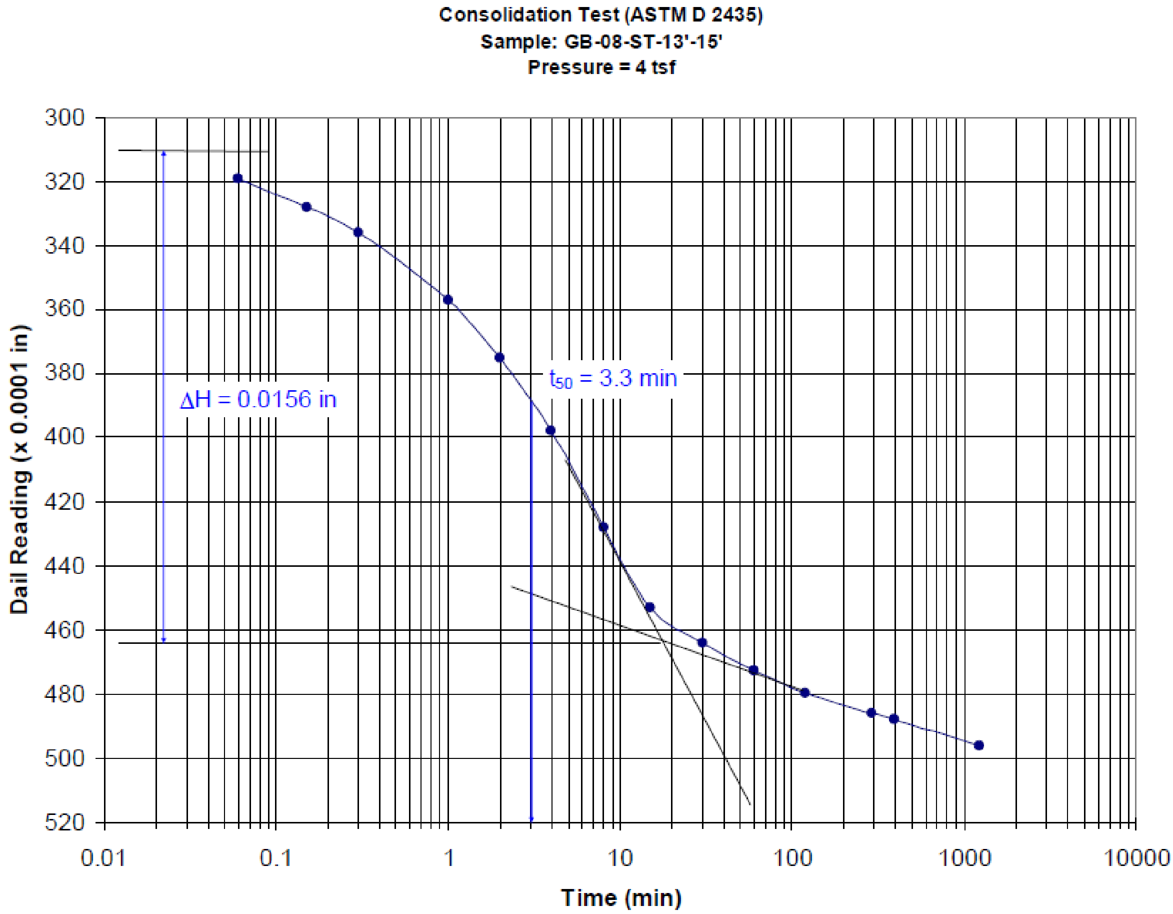
loading=16 tsf	
time	dail reading
0	652
0.1	672
0.25	687
0.5	702
1	727
2	754
4	800.5
8	816
15	836.5
30	850
60	860
115	867

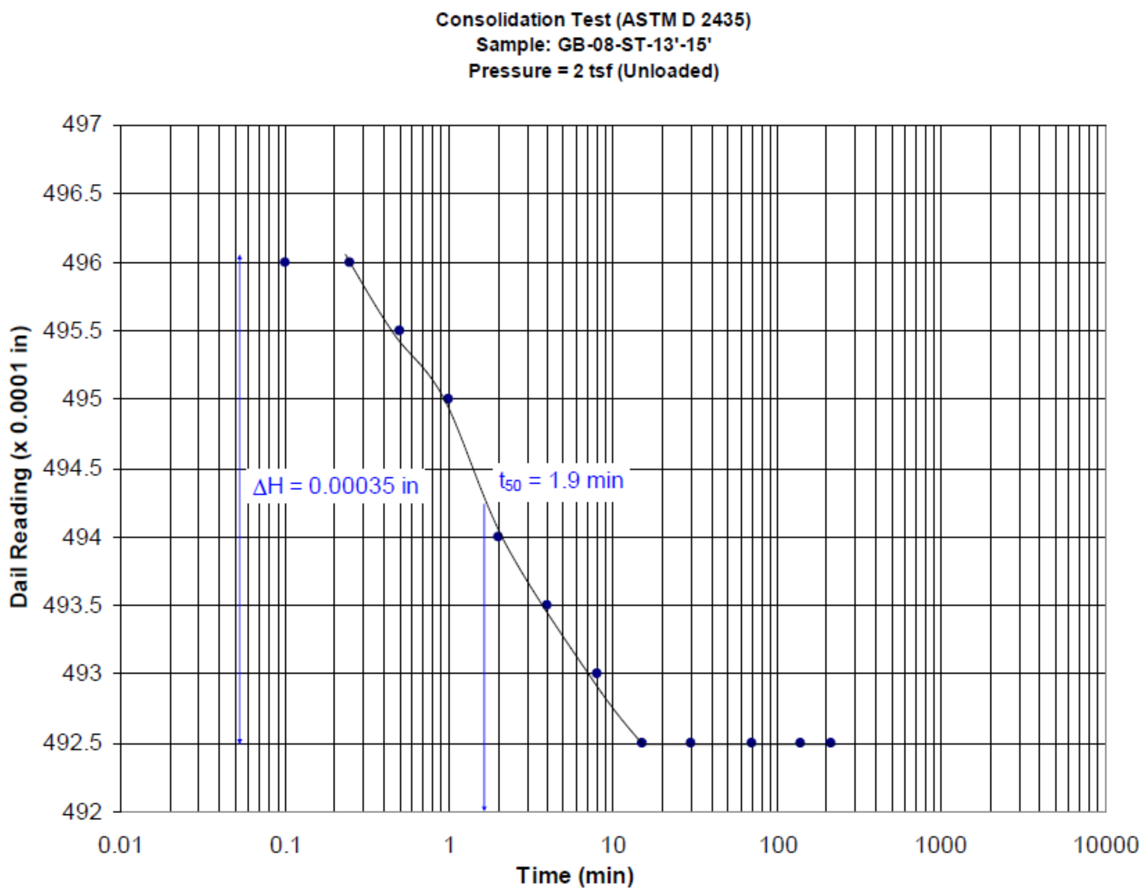
loading=32 tsf	
time	dail reading
0	867
0.1	877
0.25	893
0.5	908
1	928
2	953
4	983
8	1012
15	1027
30	1040
50	1047.5
76	1052.5
138	1060
240	1063

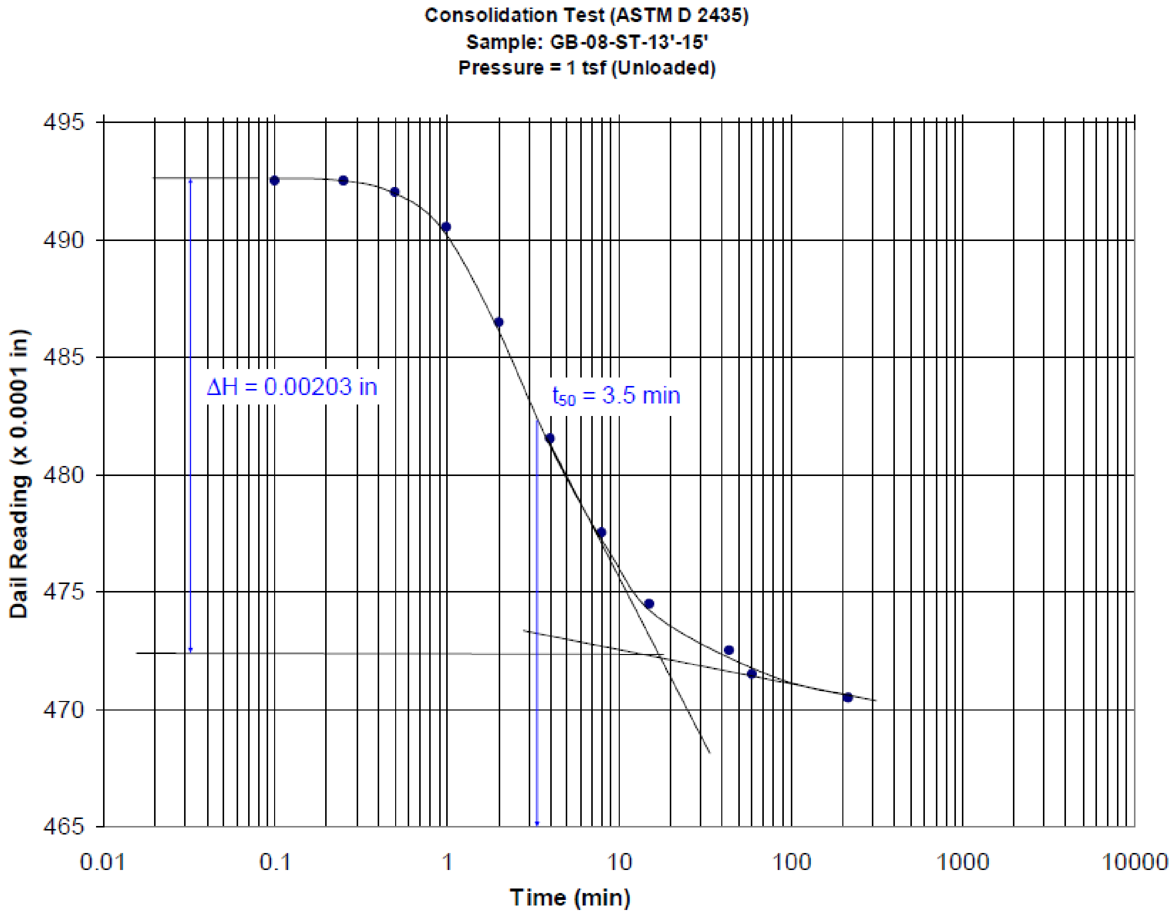


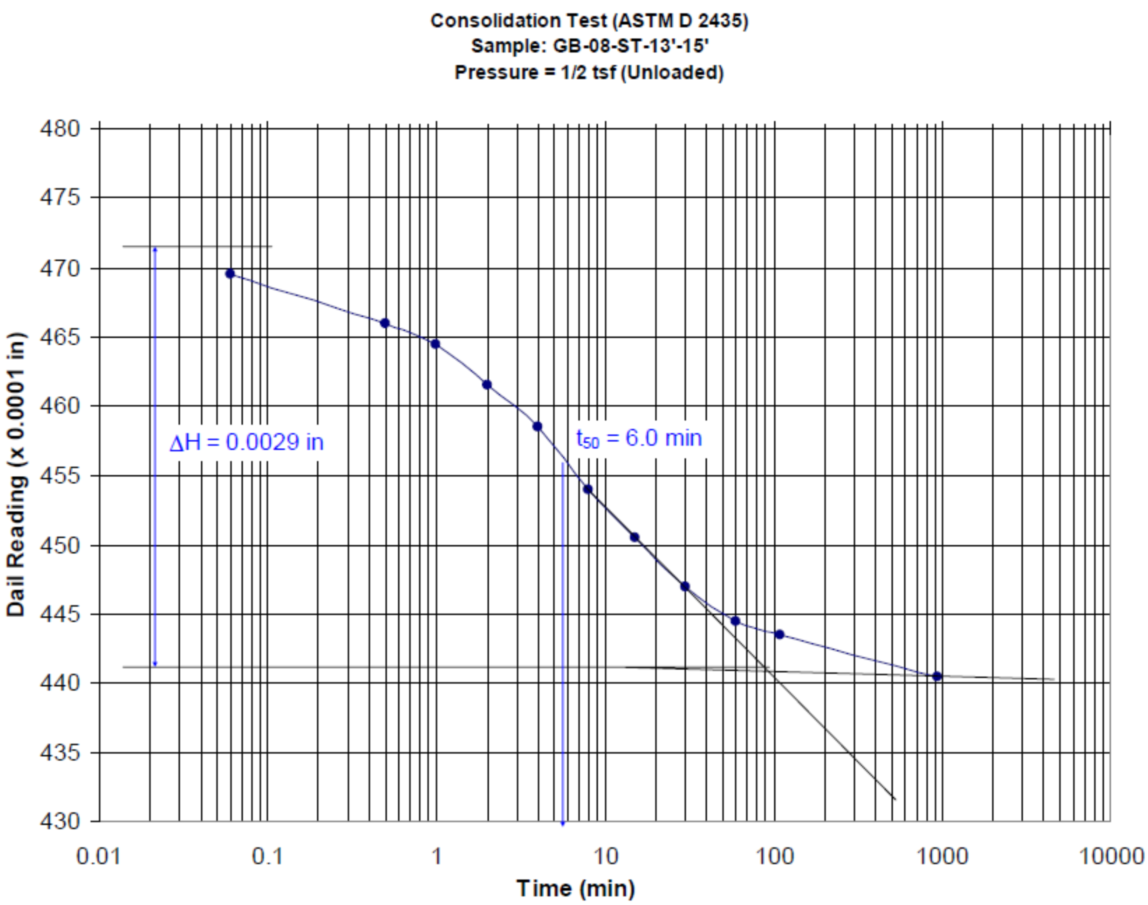

Consolidation Test (ASTM D 2435)
Sample: GB-08-ST-13'-15'
Pressure = 1 tsf
Dail Reading (x 0.0001 in)
ΔH = 0.008 in
t_{50} = 11.5 min
170
180
190
200
210
220
230
240
250
260
0.01
0.1
1
10
100
1000
10000
Time (min)

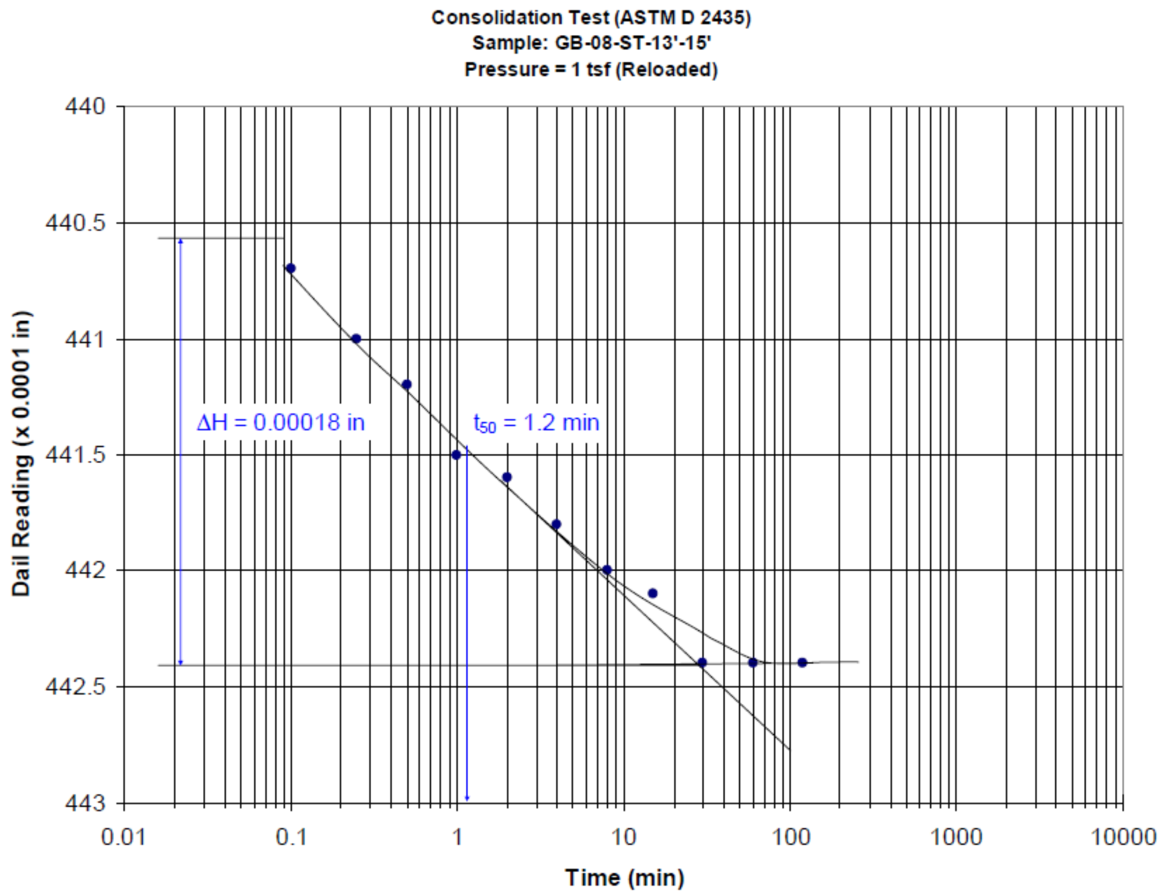

Consolidation Test (ASTM D 2435)
Sample: GB-08-ST-13'-15'
Pressure = 2 tsf
Dail Reading (x 0.0001 in)
ΔH = 0.0048 in
t_{50} = 30 min
240
250
260
270
280
290
300
310
320
0.01
0.1
1
10
100
1000
10000
Time (min)

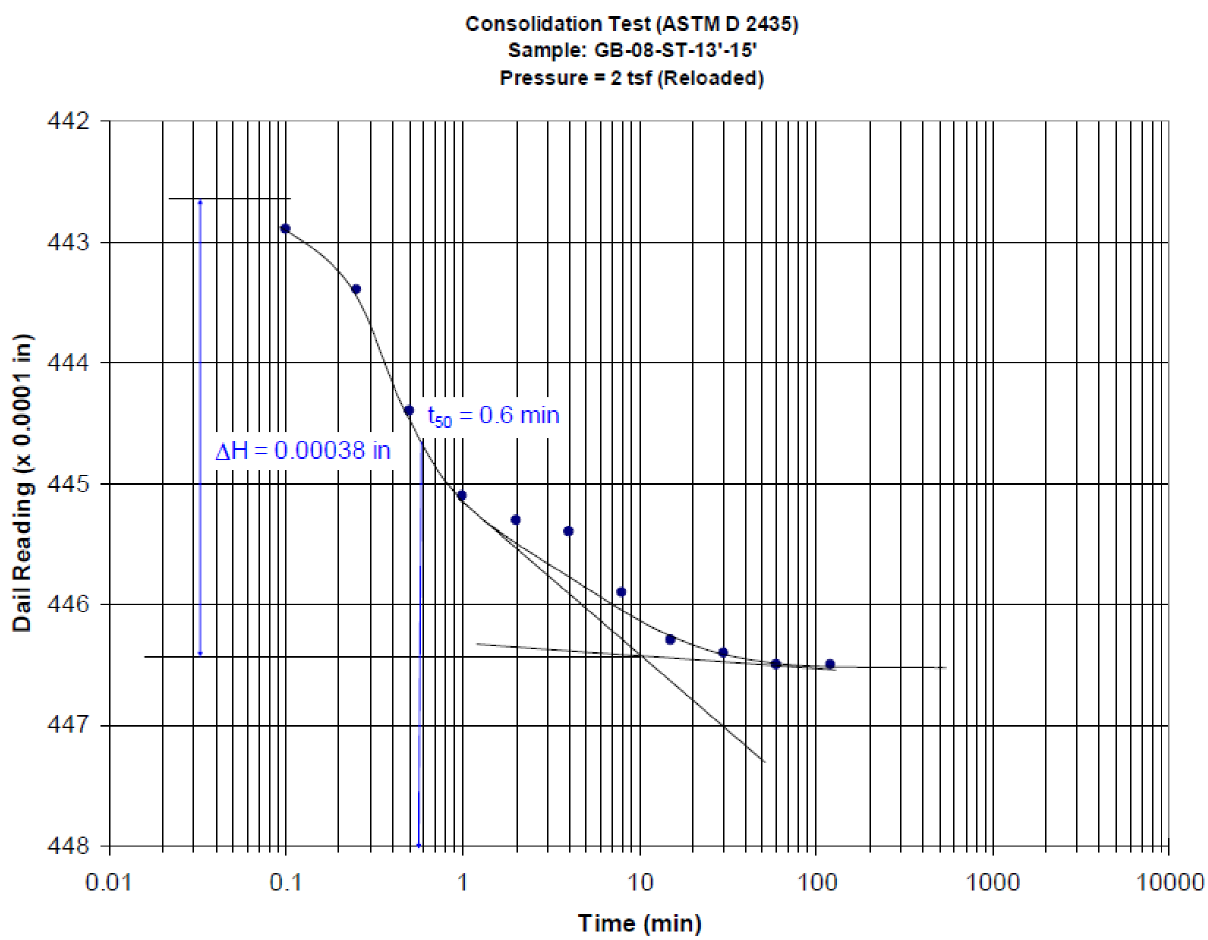

Consolidation Test (ASTM D 2435)
Sample: GB-08-ST-13'-15'
Pressure = 4 tsf
ΔH = 0.0156 in
t50 = 3.3 min
Dail Reading (x 0.0001 in)
Time (min)
300
320
340
360
380
400
420
440
460
480
500
520
0.01
0.1
1
10
100
1000
10000

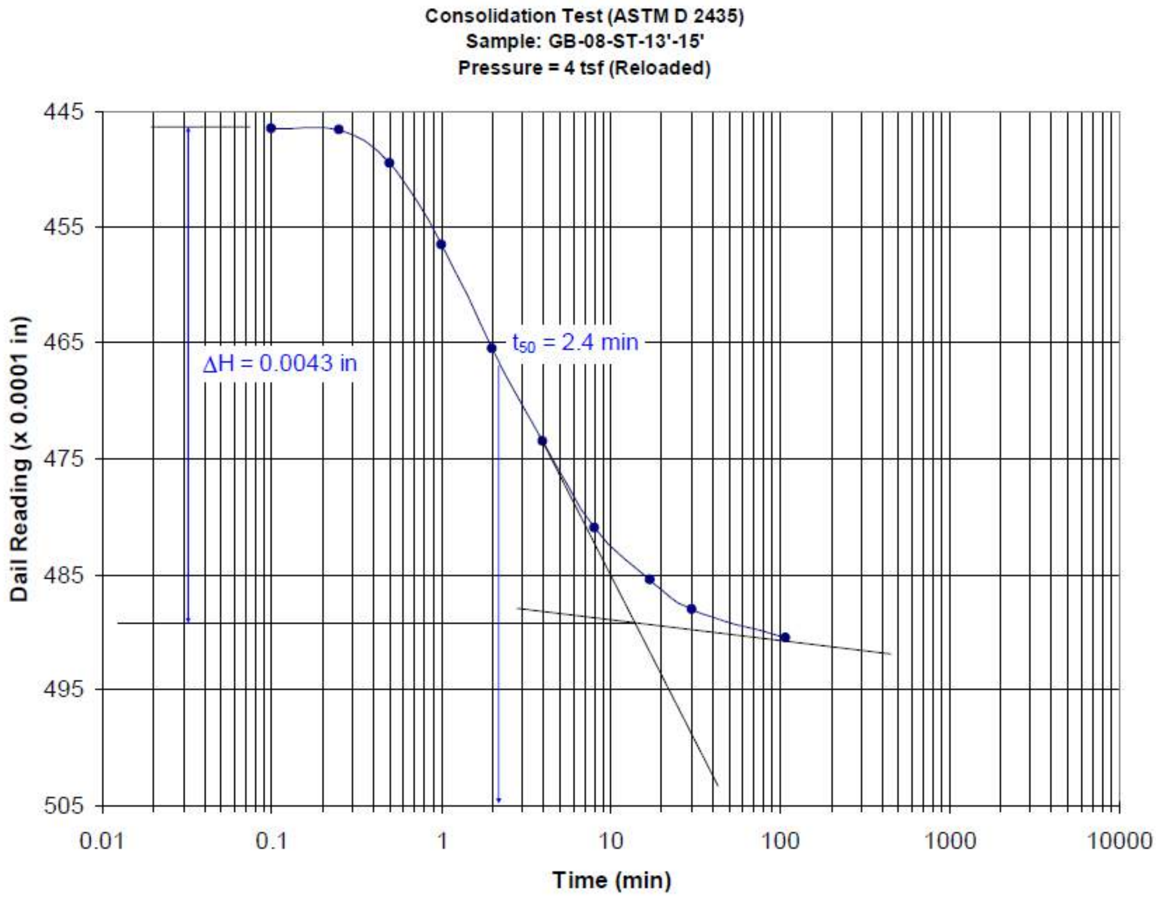

Consolidation Test (ASTM D 2435)
Sample: GB-08-ST-13'-15'
Pressure = 2 tsf (Unloaded)
ΔH = 0.00035 in
t50 = 1.9 min
Dail Reading (x 0.0001 in)
Time (min)
497
496.5
496
495.5
495
494.5
494
493.5
493
492.5
492
0.01
0.1
1
10
100
1000
10000

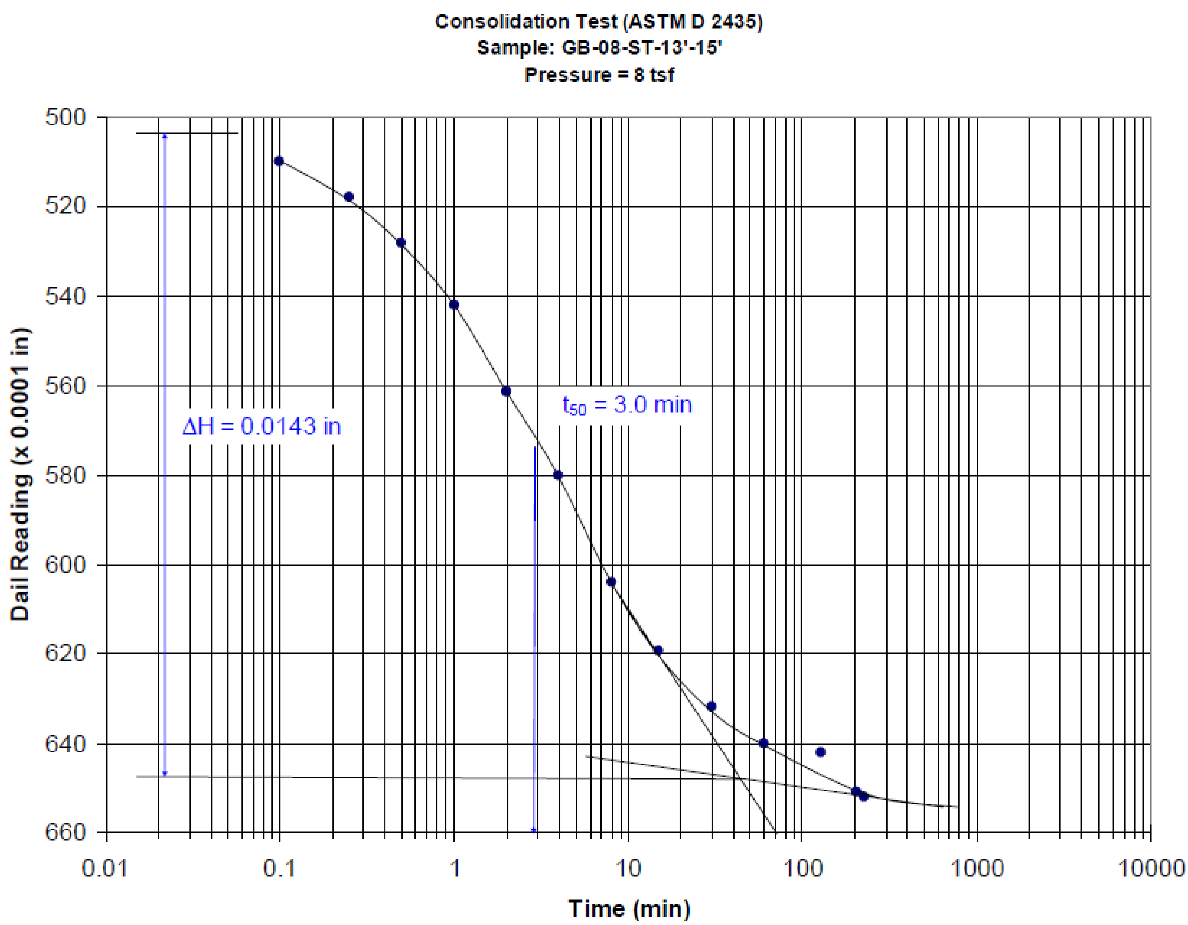

Consolidation Test (ASTM D 2435)
Sample: GB-08-ST-13'-15'
Pressure = 1 tsf (Unloaded)
Dail Reading (x 0.0001 in)
495
490
485
480
475
470
465
0.01
0.1
1
10
100
1000
10000
Time (min)
ΔH = 0.00203 in
t_{50} = 3.5 min


Consolidation Test (ASTM D 2435)
Sample: GB-08-ST-13'-15'
Pressure = 1/2 tsf (Unloaded)
Dail Reading (x 0.0001 in)
480
475
470
465
460
455
450
445
440
435
430
0.01
0.1
1
10
100
1000
10000
Time (min)
ΔH = 0.0029 in
t_{50} = 6.0 min

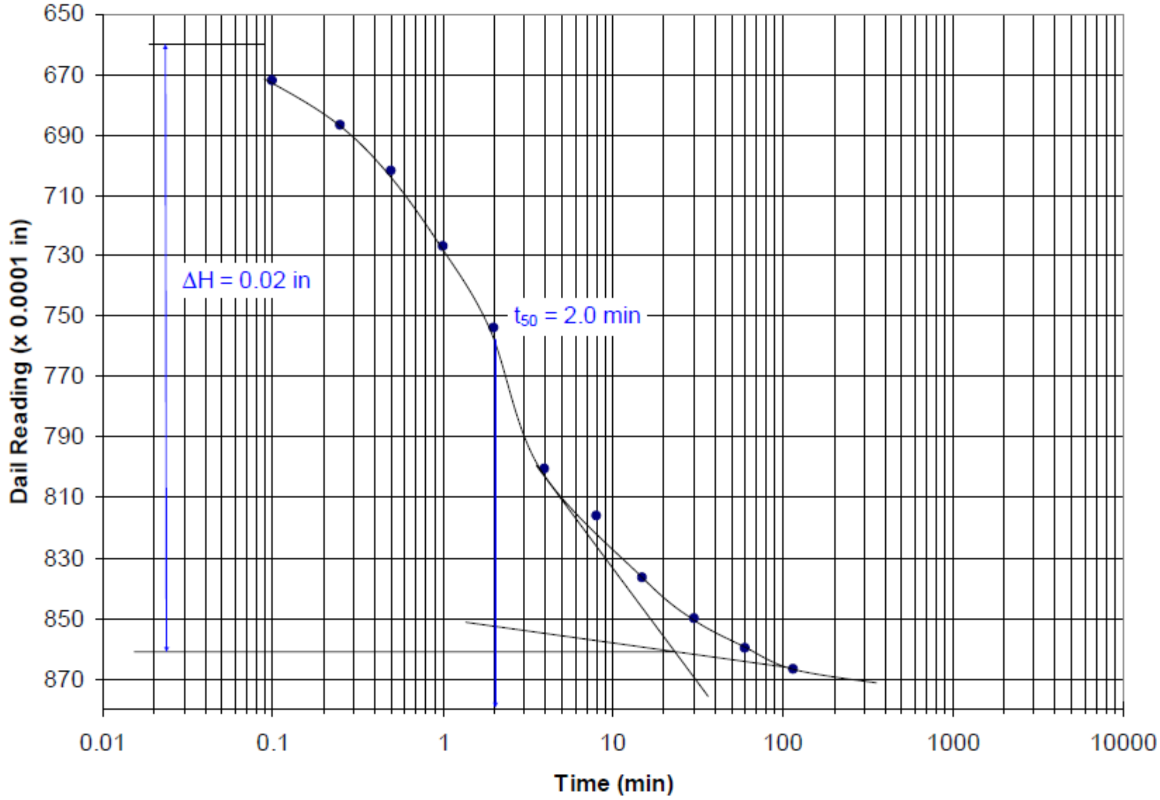

Consolidation Test (ASTM D 2435)
Sample: GB-08-ST-13'-15'
Pressure = 1 tsf (Reloaded)
ΔH = 0.00018 in
t50 = 1.2 min
Dail Reading (x 0.0001 in)
Time (min)


Consolidation Test (ASTM D 2435)
Sample: GB-08-ST-13'-15'
Pressure = 2 tsf (Reloaded)
ΔH = 0.00038 in
t50 = 0.6 min
Dail Reading (x 0.0001 in)
Time (min)


Consolidation Test (ASTM D 2435)
Sample: GB-08-ST-13'-15'
Pressure = 4 tsf (Reloaded)
Dail Reading (x 0.0001 in)
445
455
465
475
485
495
505
ΔH = 0.0043 in
t50 = 2.4 min
0.01
0.1
1
10
100
1000
10000
Time (min)


Consolidation Test (ASTM D 2435)
Sample: GB-08-ST-13'-15'
Pressure = 8 tsf
Dail Reading (x 0.0001 in)
500
520
540
560
580
600
620
640
660
ΔH = 0.0143 in
t50 = 3.0 min
0.01
0.1
1
10
100
1000
10000
Time (min)

Consolidation Test (ASTM D 2435)
Sample: GB-08-ST-13'-15'
Pressure = 16 tsf



Consolidation Test (ASTM D 2435)
Sample: GB-08-ST-13'-15'
Pressure = 32 tsf

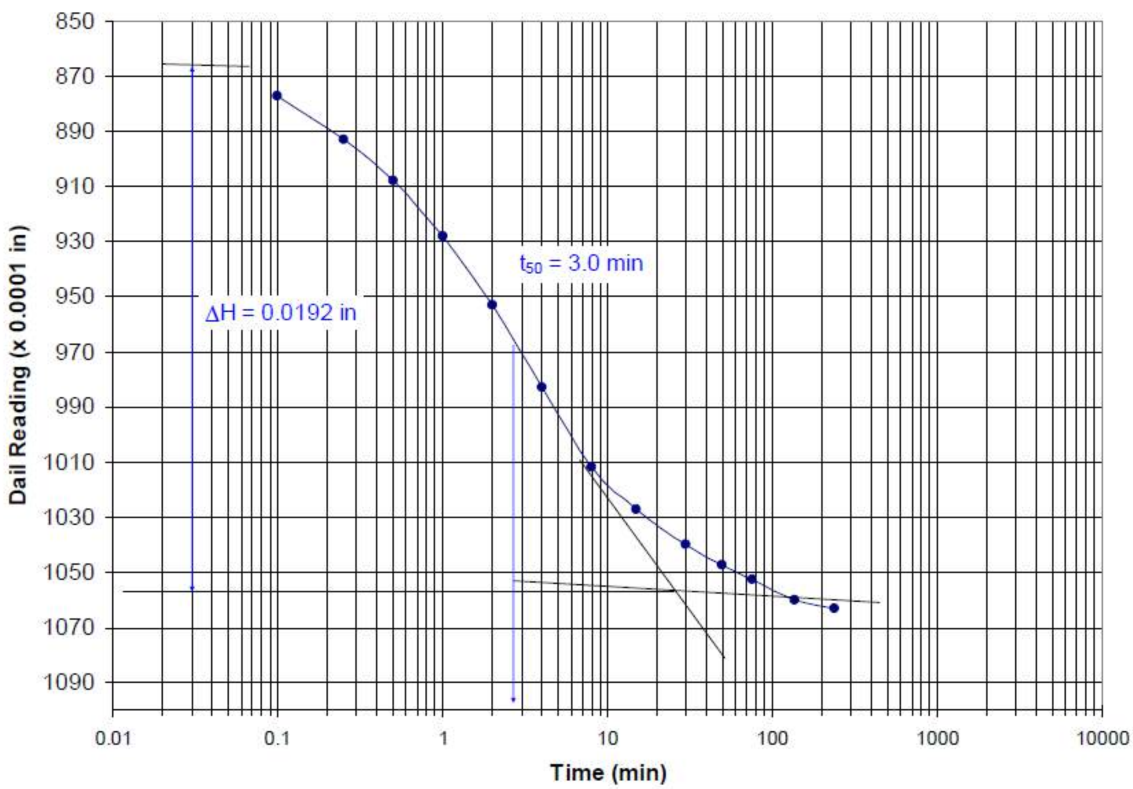


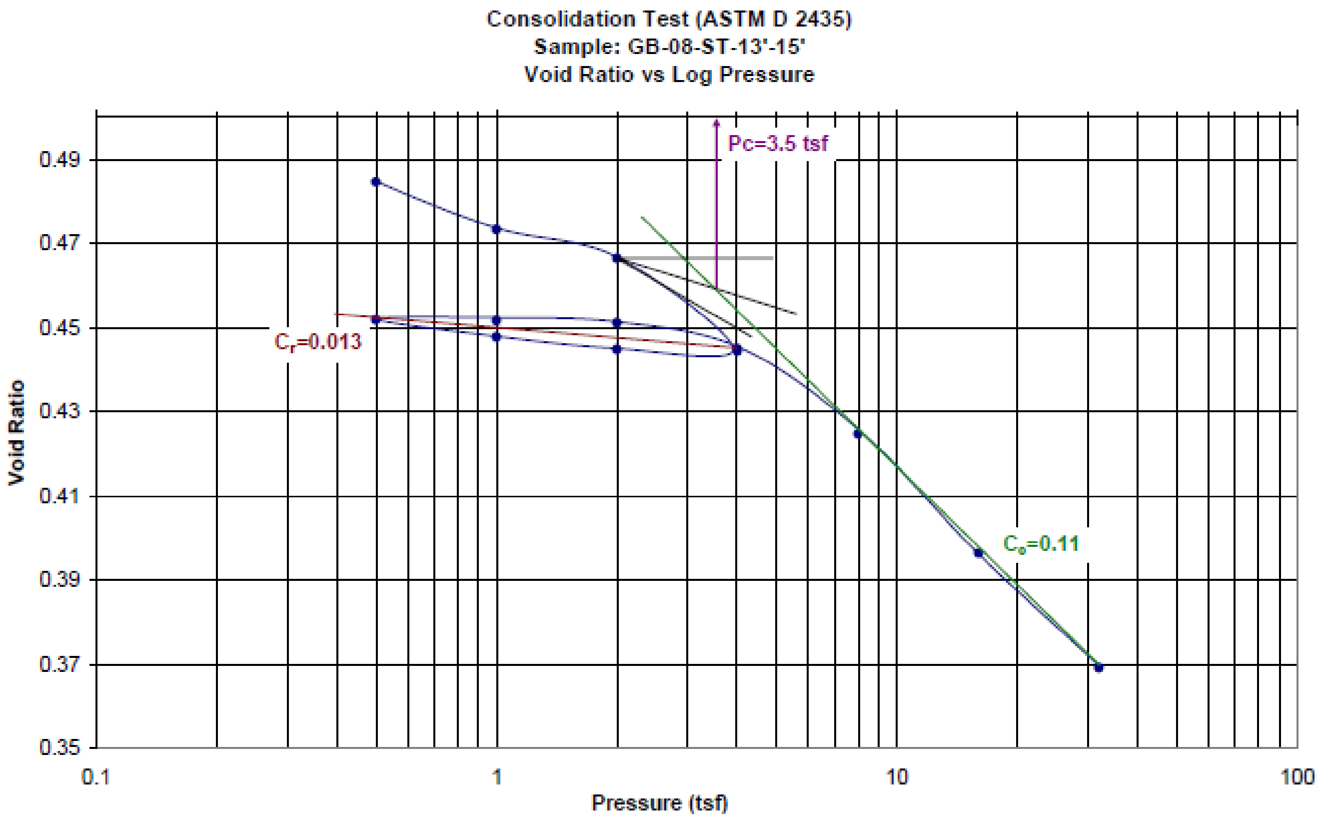
Table 2: Analysis of Consolidation Test Data

Pressure (tsf)	Time for 50% consolidation t_{50} (min)	D_0 (from graph)	D_{100} (from graph)	$D_{50} = (D_0+D_{100})*0.5$	$H_j = D_{50} * 0.0001$	ΔH (from graph)	$\Sigma \Delta H^{*}$	H^{**}	H_d^{**}	Coefficient of consolidation C_v (in^2/min)***	H_v^{***}	e^{***}
0								1.06299				
0.5	10	8	159	83.5	0.00835	0.0153	0.0153	1.04769	0.52593	5.45E-03	0.34	0.48
1	11.5	173	254	213.5	0.02135	0.008	0.0233	1.03969	0.52518	4.72E-03	0.33	0.47
2	30	254	301	277.5	0.02775	0.0048	0.0281	1.03489	0.52438	1.81E-03	0.33	0.47
4	3.3	310	362	336	0.03360	0.0156	0.0437	1.01929	0.51805	1.60E-02	0.31	0.44
2	1.9	496	492.5	494.25	0.04943	0.0004	0.04335	1.01964	0.52218	2.83E-02	0.31	0.44
1	3.5	493	472.5	482.5	0.04825	0.002	0.04132	1.02167	0.52290	1.54E-02	0.32	0.45
0.5	6	472	442	457	0.04570	0.0029	0.03842	1.02457	0.52371	9.01E-03	0.32	0.45
1	1.2	441	442.4	441.5	0.04415	0.0002	0.0386	1.02439	0.52323	4.49E-02	0.32	0.45
2	0.6	443	446.5	444.55	0.04446	0.0004	0.03898	1.02401	0.52312	8.98E-02	0.32	0.45
4	2.4	446	489	467.5	0.04675	0.0043	0.04328	1.01971	0.52154	2.23E-02	0.31	0.44
8	3	504	650	577	0.05770	0.0143	0.05758	1.00541	0.51713	1.76E-02	0.30	0.42
16	2	660	861	760.5	0.07605	0.02	0.07758	0.98541	0.51172	2.58E-02	0.28	0.40
32	3	869	1060	964.5	0.09645	0.0192	0.09678	0.96621	0.50722	1.69E-02	0.26	0.37

* $\Sigma\Delta H$ for applied pressure = $\Sigma\Delta H$ of all previous pressures + ΔH under applied pressure

** $H_{dj} = \dfrac{H_j}{2} \pm \dfrac{\Delta H_j}{4}$ and $H_j = H_i \pm \Delta H_{j-1}$ (- for Loading and + for Unloading)

*** $C_V = 0.197 \times \dfrac{H_d^2}{t_{50}}$, $H_V = (H_i - H_s) - \Sigma\Delta H$ and $e = \dfrac{H_V}{H_s}$



Final Results:

Compression Index (C_c) = 0.11
Recompression Index (C_r) = 0.013
Preconsolidation pressure (P_c) or Maximum past pressure (σ_{vmax}) = 3.5 tsf
Coefficient of consolidation (C_v)= 1.54×10^{-2} to 9.01×10^{-3} in^2/min (depends on the pressure)
Coefficient of secondary compression ($C\alpha$) = 0.001
(It is the slope of time vs settlement curve beyond the end of primary consolidation)

Consolidation Test

Data Sheets

Date Tested:
Tested By:
Project Name:
Sample Number:
Sample Description:

<u>Before test</u>

Consolidation type =
Mass of the ring + glass plate =
Inside diameter of the ring =
Height of specimen, H_i =
Area of specimen, A =
Mass of specimen + ring =
Initial moisture content of specimen, w_i (%) =
Specific gravity of solids, G_s =

<u>After test</u>

Mass of wet sample + ring + glass plate =
Mass of can =
Mass of can + wet soil =
Mass of wet specimen =
Mass of can + dry soil =
Mass of dry specimen, M_s =
Final moisture content of specimen, w_f =

<u>Calculations</u>

Mass of solids in specimen, M_s =
(Mass of dry specimen after test)

Mass of water in specimen before test, M_{wi} = $w_i \times M_s$=

Mass of water in specimen after test, M_{wf} (g) = $w_f \times M_s$=

Height of solids, $H_s = \dfrac{M_s}{A \times G_s \times \rho_w} =$

(same before and after test and note $\rho_w = 1\ g/cm^3$)

Height of water before test, $H_{wi} = \dfrac{M_{wi}}{A \times \rho_w} =$

Time - Settlement Data

Conversion: 0.0001 inch = 1.0 on dial reading (confirm this before using)

LOADING = _____ tsf	
ELAPSED TIME, min	DIAL READING
0	
0.1	
0.25	
0.5	
1	
2	
4	
10	
15	
30	
60	
120	
240	

LOADING = _____ tsf	
ELAPSED TIME, min	DIAL READING
0	
0.1	
0.25	
0.5	
1	
2	
4	
10	
15	
30	
60	
121	
240	

LOADING = _____ tsf	
ELAPSED TIME, min	DIAL READING
0	
0.1	
0.25	
0.5	
1	
2	
4	
10	
15	
30	
60	
120	
240	

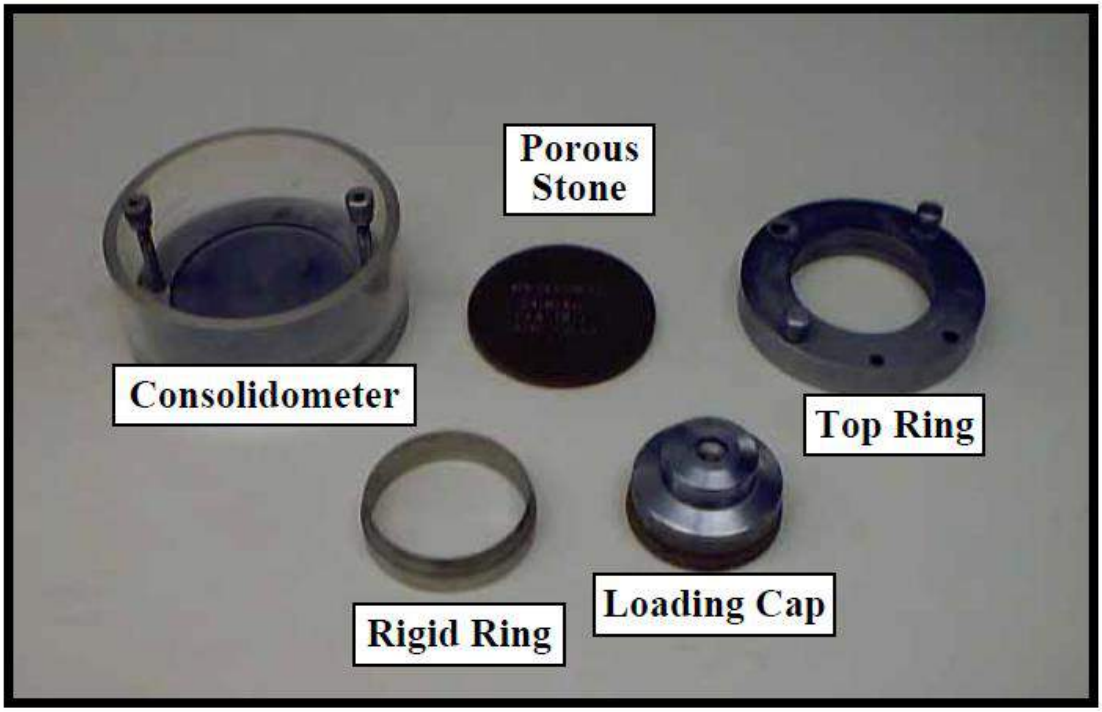
Analysis of Consolidation Test Data

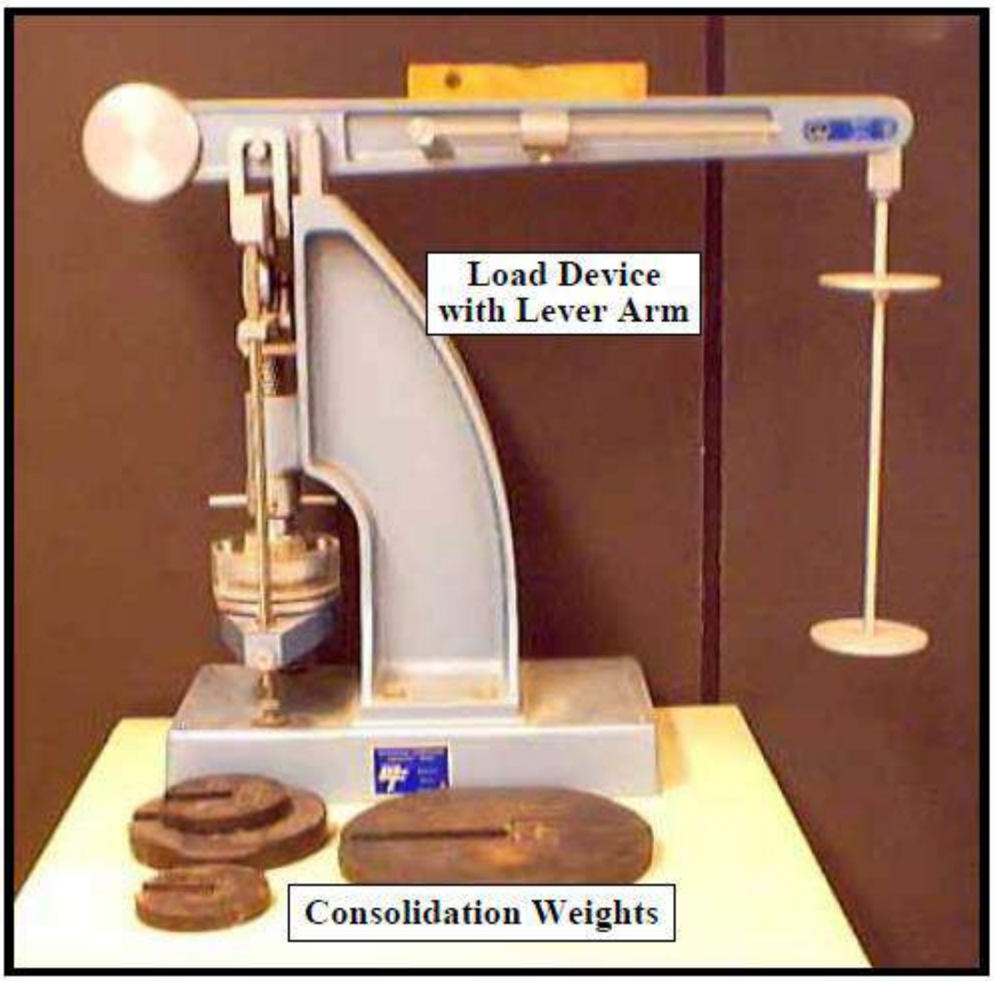
Pressure (tsf)	Time for 50% consolidation t_{50} (min)	D_0 (from graph)	D_{100} (from graph)	$D_{50} = (D_0+D_{100})*0.5$	$H_j = D_{50} * 0.0001$	ΔH (from graph)	$\Sigma \Delta H^{*}$	H^{**}	H_d^{**}	Coefficient of consolidation C_v (in^2/min)	H_v^{***}	e^{***}

* ΣΔH for applied pressure = ΣΔH of all previous pressures + ΔH under applied pressure

** $H_{dj} = \frac{H_j}{2} \pm \frac{\Delta H_j}{4}$ and $H_j = H_i \pm \Delta H_{j-1}$ (- for Loading and + for Unloading)

*** $C_V = 0.197 \times \frac{H_d^2}{t_{50}}$, $H_V = (H_i - H_s) - \Sigma\Delta H$ and $e = \frac{H_V}{H_s}$

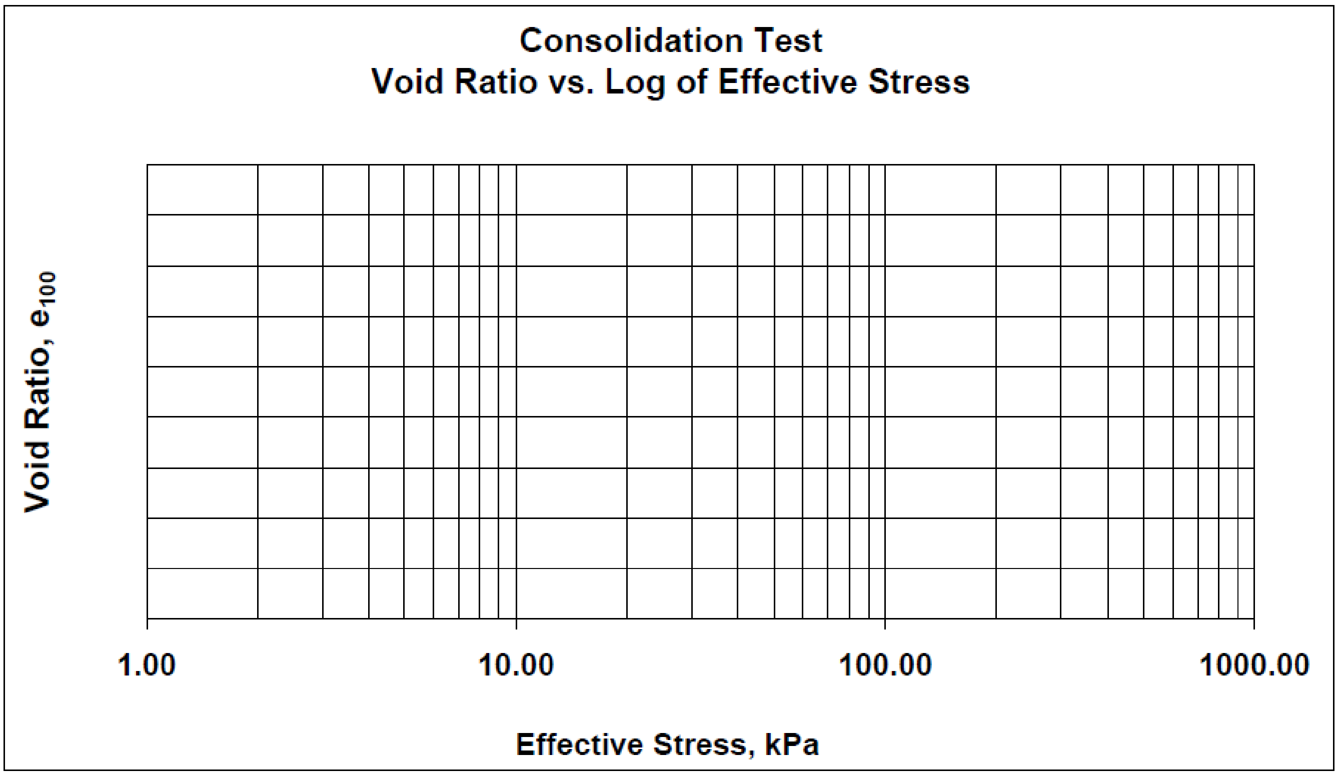




Consolidation Test Load Device

Effective Stress (kPa)						
Deformation, d_{100} (cm)						
Final Sample Height (cm)						
Height of Voids, H_v, (cm)						
Void Ratio, e_{100}						
d_{50} (cm)						
H_{dr} (cm)						
t_{90}, min						
c_v, cm^2/min						

Consolidation Test Summary Table



Void Ratio Vs. Effective Stress

Sample Calculations for Consolidation Test

12. Direct Shear Test

Purpose:

This test is performed to determine the consolidated-drained shear strength of a sandy to silty soil. The shear strength is one of the most important engineering properties of a soil, because it is required whenever a structure is dependent on the soil's shearing resistance. The shear strength is needed for engineering situations such as determining the stability of slopes or cuts, finding the bearing capacity for foundations, and calculating the pressure exerted by a soil on a retaining wall.

Standard Reference:

ASTM D 3080 - Standard Test Method for Direct Shear Test of Soils Under Consolidated Drained Conditions

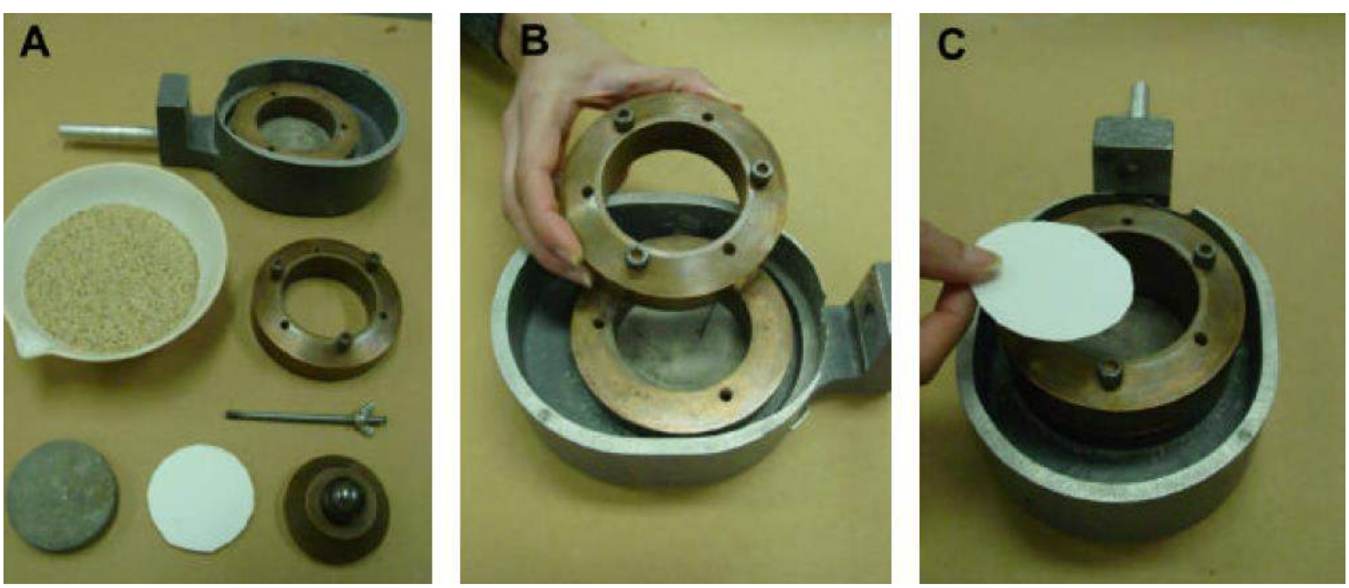
Significance:

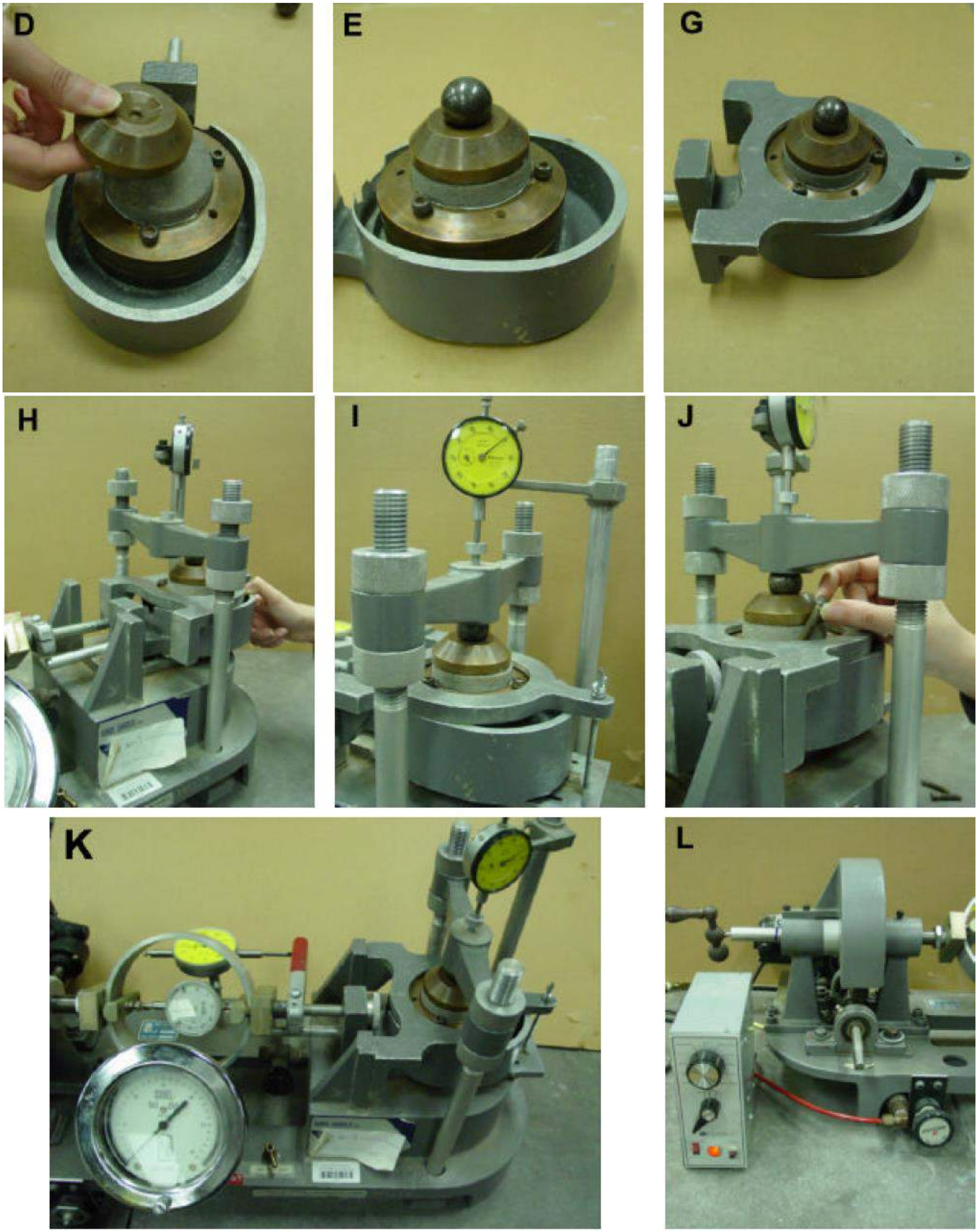
The direct shear test is one of the oldest strength tests for soils. In this laboratory, a direct shear device will be used to determine the shear strength of a cohesionless soil (i.e. angle of internal friction (f)). From the plot of the shear stress versus the horizontal displacement, the maximum shear stress is obtained for a specific vertical confining stress. After the experiment is run several times for various vertical-confining stresses, a plot of the maxi mum shear stresses versus the vertical (normal) confining stresses for each of the tests is produced. From the plot, a straight-line approximation of the Mohr-Coulomb failure envelope curve can be drawn, f may be determined, and, for cohesionless soils (c = 0), the shear strength can be computed from the following equation:

$$s = s \tan f$$

Equipment:

Direct shear device, Load and deformation dial gauges, Balance.




D
E
G
H
I
J
K
L

Test Procedure:

1. Weigh the initial mass of soil in the pan.
2. Measure the diameter and height of the shear box. Compute 15% of the diameter in millimeters.
3. Carefully assemble the shear box and place it in the direct shear device. Then place a porous stone and a filter paper in the shear box.
4. Place the sand into the shear box and level off the top. Place a filter paper, a porous stone, and a top plate (with ball) on top of the sand
5. Remove the large alignment screws from the shear box! Open the gap between the shear box halves to approximately 0.025 in. using the gap screws, and then back out the gap screws.
6. Weigh the pan of soil again and compute the mass of soil used.
7. Complete the assembly of the direct shear device and initialize the three gauges (Horizontal displacement gage, vertical displacement gage and shear load gage) to zero.
8. Set the vertical load (or pressure) to a predetermined value, and then close bleeder valve and apply the load to the soil specimen by raising the toggle switch.
9. Start the motor with selected speed so that the rate of shearing is at a selected constant rate, and take the horizontal displacement gauge, vertical displacement gage and shear load gage readings. Record the readings on the data sheet. (Note: Record the vertical displacement gage readings, if needed).
10. Continue taking readings until the horizontal shear load peaks and then falls, or the horizontal displacement reaches 15% of the diameter.

Analysis:

1. Calculate the density of the soil sample from the mass of soil and volume of the shear box.
2. Convert the dial readings to the appropriate length and load units and enter the values on the data sheet in the correct locations. Compute the sample area A, and the vertical (Normal) stress s_v.

$$s_v = \frac{N_v}{A}$$

3. Where: Nv = normal vertical force, and sv = normal vertical stress
4. Calculate shear stress (t) using

$$t = \frac{F_h}{A}$$

5. Where F_h= shear stress (measured with shear load gage)
6. Plot the horizontal shear stress (t) versus horizontal (lateral) displacement H.
7. Calculate the maximum shear stress for each test.
8. Plot the value of the maximum shear stress versus the corresponding vertical stress for each test, and determine the angle of internal friction (f) from the slope of the approximated Mohr-Coulomb failure envelope.

DIRECT SHEAR TEST
DATA SHEET

Date Tested: August 30, 2002

Tested By: CEMM315 Class, Group A

Project Name: CEMM315 Lab

Sample Number: K-3,AU-10, 2'-4'

Visual Classification: Brown uniform sand

Shear Box Inside Diameter: 6.3 cm
Area (A): $31.17\ cm^2 = 4.83\ in^2$
Shear Box Height: 4.9 cm
Soil Volume: $119.9\ cm^3$
Initial mass of soil and pan: 1000. g
Final mass of soil and pan: 720.82 cm
Mass of soil: 279.18 g
Density of soil (?) : $1.65\ g/cm^3$

Direct Shear Test Data

Displacement rate: ______

Normal stress: 2.27 psi

Horizontal Dial Reading (0.001 in)	Horizontal Displacement (in)	Load Dial Reading	Horizontal Shear Force (lb)	Shear Stress (psi)
0	0	0	0	0
10	0.01	4	5.142	1.064
19	0.019	4.3	5.231	1.082
29	0.029	4.8	5.379	1.113
36	0.036	5	5.439	1.126
44	0.044	7	6.033	1.248
51	0.051	8	6.33	1.31
57	0.057	13.5	7.963	1.648
63	0.063	15	8.409	1.740
70	0.07	17	9.002	1.863
76	0.076	19	9.597	1.986
84	0.084	20	9.893	2.047
91	0.091	22	10.488	2.170
100	0.1	22.5	10.636	2.201
107	0.107	23	10.785	2.232
114	0.144	23.5	10.933	2.262
121.5	0.1215	25	11.379	2.355
129	0.129	25.5	11.527	2.385
137	0.137	26	11.675	2.416
145	0.145	27	11.973	2.478
152	0.152	27.5	12.121	2.508
160	0.16	28	12.270	2.539
179	0.179	25	11.379	2.355

Direct Shear Test Data

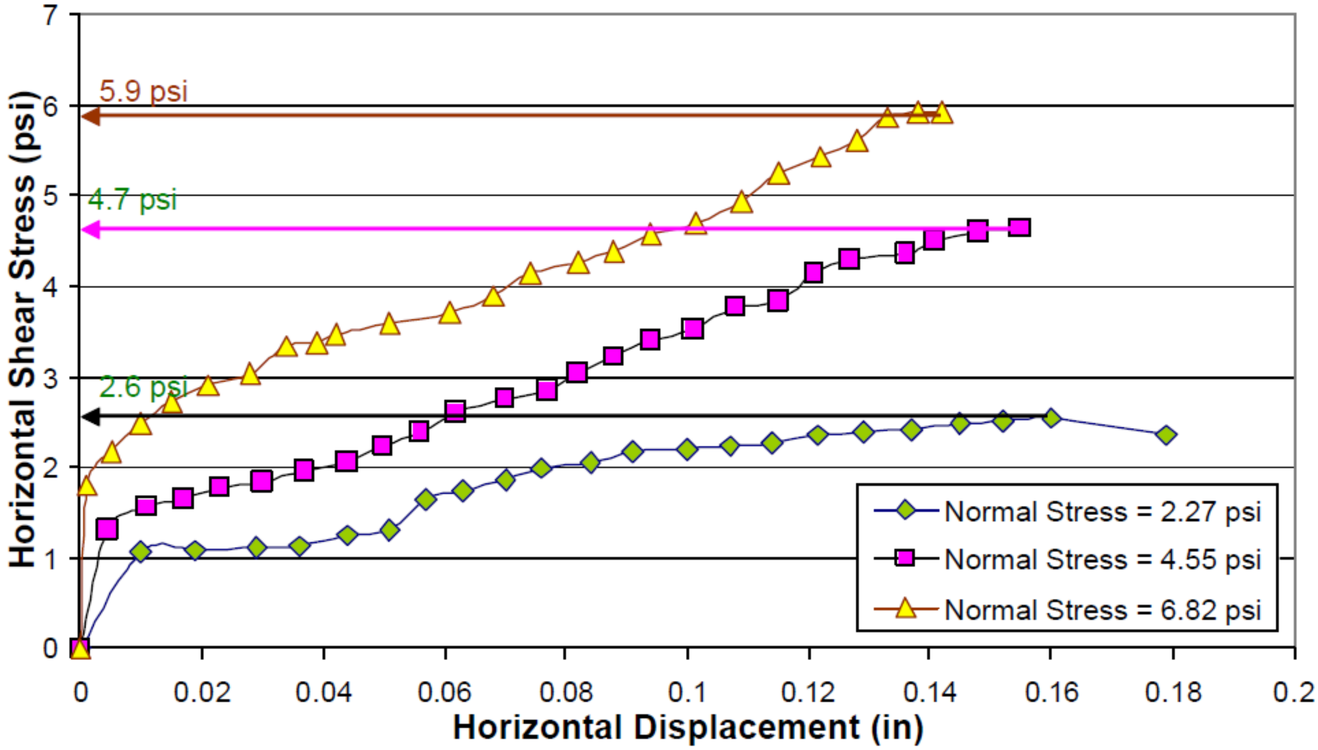
Normal stress: 4.55 psi

Horizontal Dial Reading (0.001 in)	Horizontal Displacement (in)	Load Dial Reading	Horizontal Shear Force (lb)	Shear Stress (psi)
0	0	0	0	0
4.5	0.0045	8	6.330	1.31
11	0.011	12	7.517	1.556
17	0.017	13.5	7.963	1.648
23	0.023	15.5	8.557	1.77
30	0.030	16.5	8.854	1.832
37	0.037	18.5	9.448	1.955
44	0.044	20	9.894	2.047
50	0.05	23	10.785	2.232
56	0.056	25.5	11.527	2.385
62	0.062	29	12.567	2.60
70	0.07	31.5	13.309	2.754
77	0.077	33	13.755	2.846
82	0.082	36	14.646	3.031
88	0.088	39	15.537	3.215
94	0.094	42	16.428	3.4
101	0.101	44	17.022	3.522
108	0.108	48	18.210	3.768
115	0.115	49	18.507	3.83
121	0.121	54	19.991	4.13
127	0.127	56.5	20.734	4.291
136	0.136	57.5	21.031	4.352
141	0.141	60	21.774	4.506
148	0.148	61.5	22.219	4.599
155	0.155	62	22.368	4.62

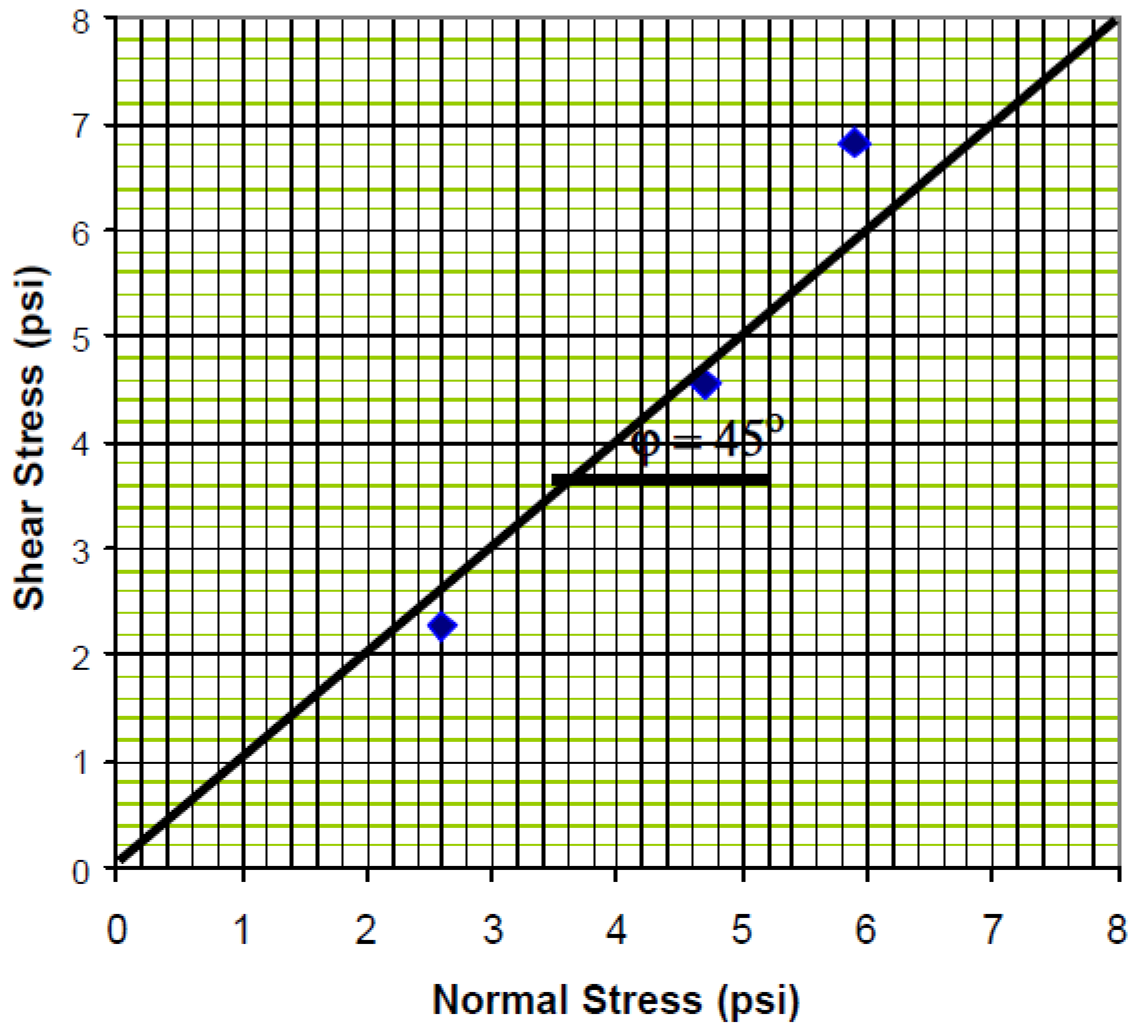
Direct Shear Test Data

Normal stress: 6.82 psi

Horizontal Dial Reading (0.001 in)	Horizontal Displacement (in)	Load dial Reading	Horizontal Shear Force (lb)	Shear Stress (psi)
0	0	0	0	0
1	0.001	16	8.706	1.801
5	0.005	22	10.488	2.170
10	0.01	27	11.972	2.478
15	0.015	31	13.16	2.723
21	0.021	34	14.052	2.908
28	0.028	36	14.646	3.031
34	0.034	41	16.131	3.338
39	0.039	41.5	16.279	3.37
42	0.042	43	16.725	3.461
51	0.051	45	17.319	3.584
61	0.061	47	17.913	3.707
68	0.068	50	18.804	3.891
74	0.074	54	19.99	4.13
82	0.082	56	20.586	4.26
88	0.088	58	21.18	4.383
94	0.094	61	22.071	4.568
101.5	0.1015	63	22.665	4.690
109	0.109	67	23.85	4.937
115	0.115	72	25.337	5.244
122	0.122	75	26.228	5.428
128	0.128	78	27.119	5.612
133	0.133	82	28.307	5.858
138	0.138	83	28.605	5.92
142	0.142	83	28.60	5.92



5.9 psi
4.7 psi
2.6 psi
Horizontal Shear Stress (psi)
Horizontal Displacement (in)
Normal Stress = 2.27 psi
Normal Stress = 4.55 psi
Normal Stress = 6.82 psi



φ = 45°
Shear Stress (psi)
Normal Stress (psi)

DIRECT SHEAR TEST
DATA SHEET

Date Tested:

Tested By:

Project Name:

Sample Number:

Visual Classification:

Shear Box Inside Diameter:
Area (A):
Shear Box Height:
Soil Volume:
Initial mass of soil and pan:
Final mass of soil and pan:
Mass of soil:
Density of soil (?) :

Direct Shear Test Data

Displacement rate: ______

Normal stress: ________ psi

Horizontal Dial Reading (0.001 in)	Horizontal Displacement (in)	Load Dial Reading	Horizontal Shear Force (lb)	Shear Stress (psi)

Direct Shear Test Data

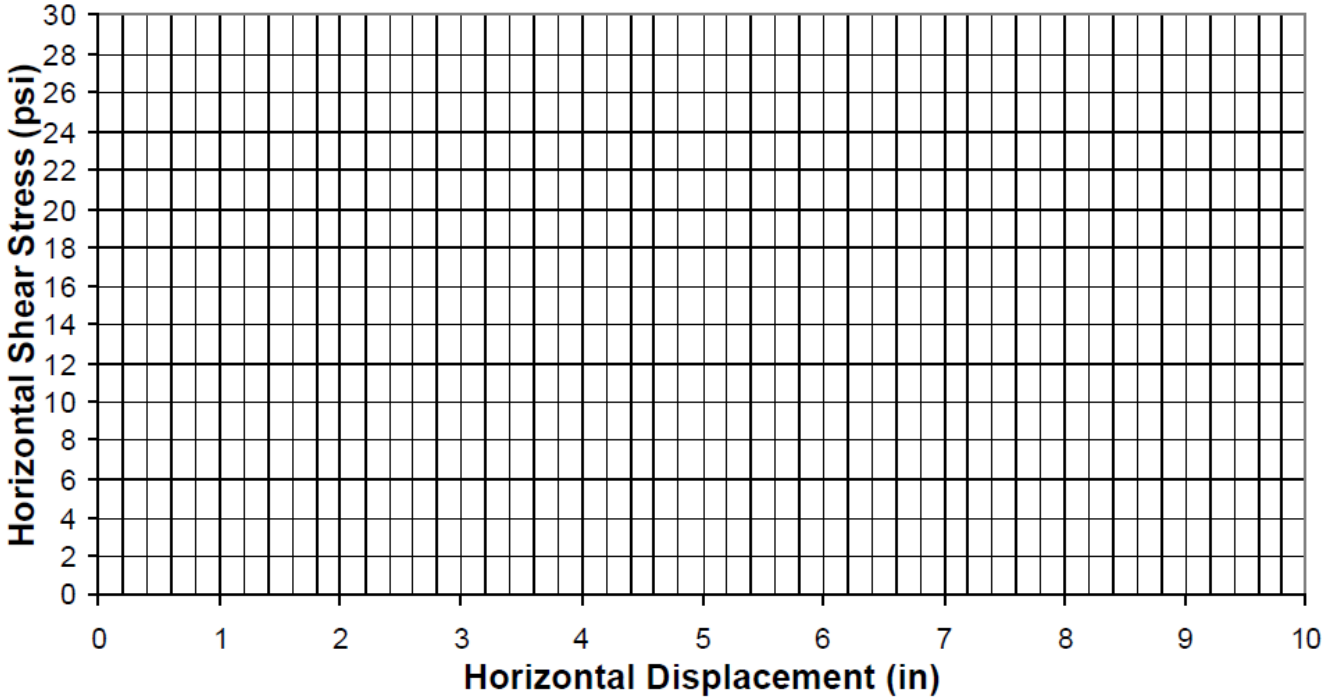
Normal stress: ________ psi

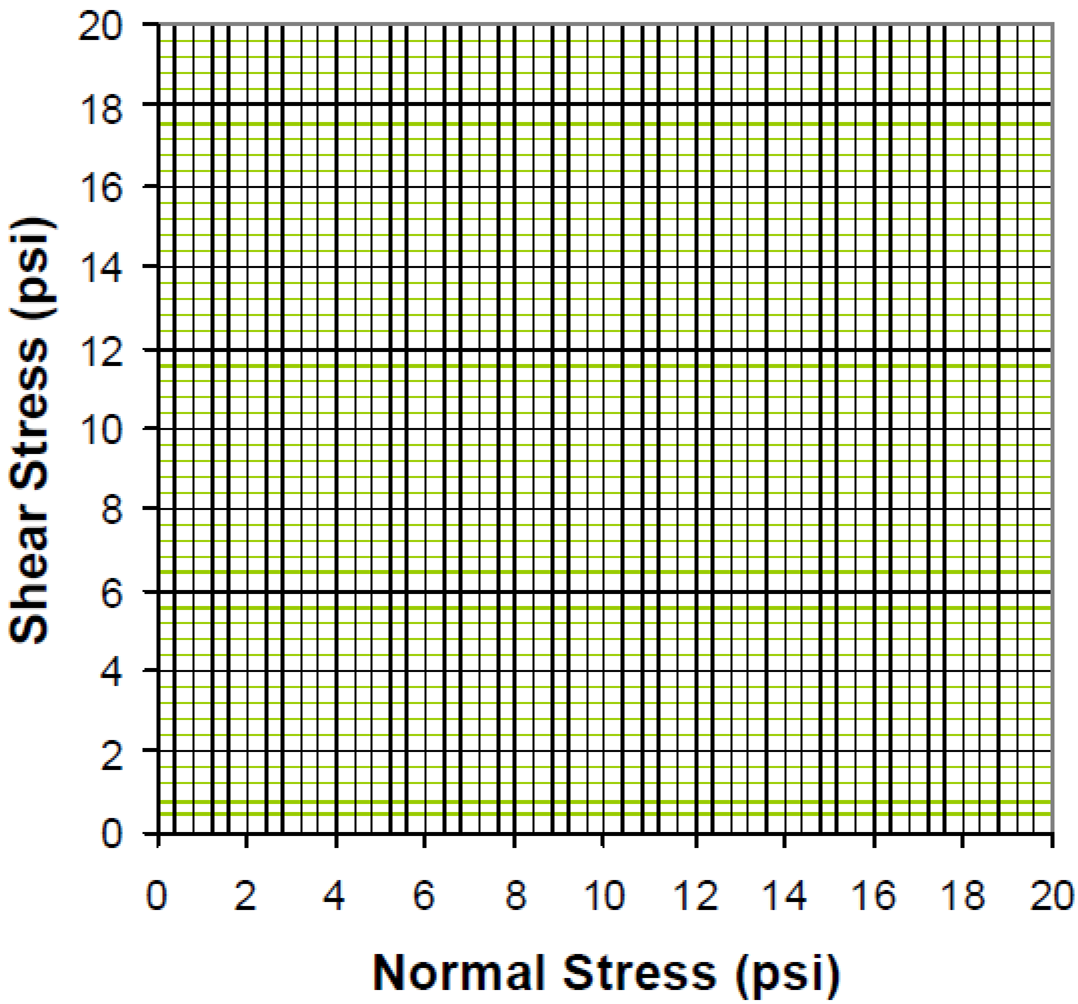
Horizontal Dial Reading (0.001 in)	Horizontal Displacement (in)	Load Dial Reading	Horizontal Shear Force (lb)	Shear Stress (psi)

Direct Shear Test Data

Normal stress: ______ psi

Horizontal Dial Reading (0.001 in)	Horizontal Displacement (in)	Load dial Reading	Horizontal Shear Force (lb)	Shear Stress (psi)


Horizontal Shear Stress (psi)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
0 1 2 3 4 5 6 7 8 9 10
Horizontal Displacement (in)


Shear Stress (psi)
0
2
4
6
8
10
12
14
16
18
20
0 2 4 6 8 10 12 14 16 18 20
Normal Stress (psi)

13. Unconfined Compression (UC) Test

Purpose:

The primary purpose of this test is to determine the unconfined compressive strength, which is then used to calculate the unconsolidated undrained shear strength of the clay under unconfined conditions. According to the ASTM standard, the unconfined compressive strength (q_u) is defined as the compressive stress at which an unconfined cylindrical specimen of soil will fail in a simple compression test. In addition, in this test method, the unconfined compressive strength is taken as the maximum load attained per unit area, or the load per unit area at 15% axial strain, whichever occurs first during the performance of a test.

Standard Reference:

ASTM D 2166 - Standard Test Method for Unconfined Compressive Strength of Cohesive Soil

Significance:

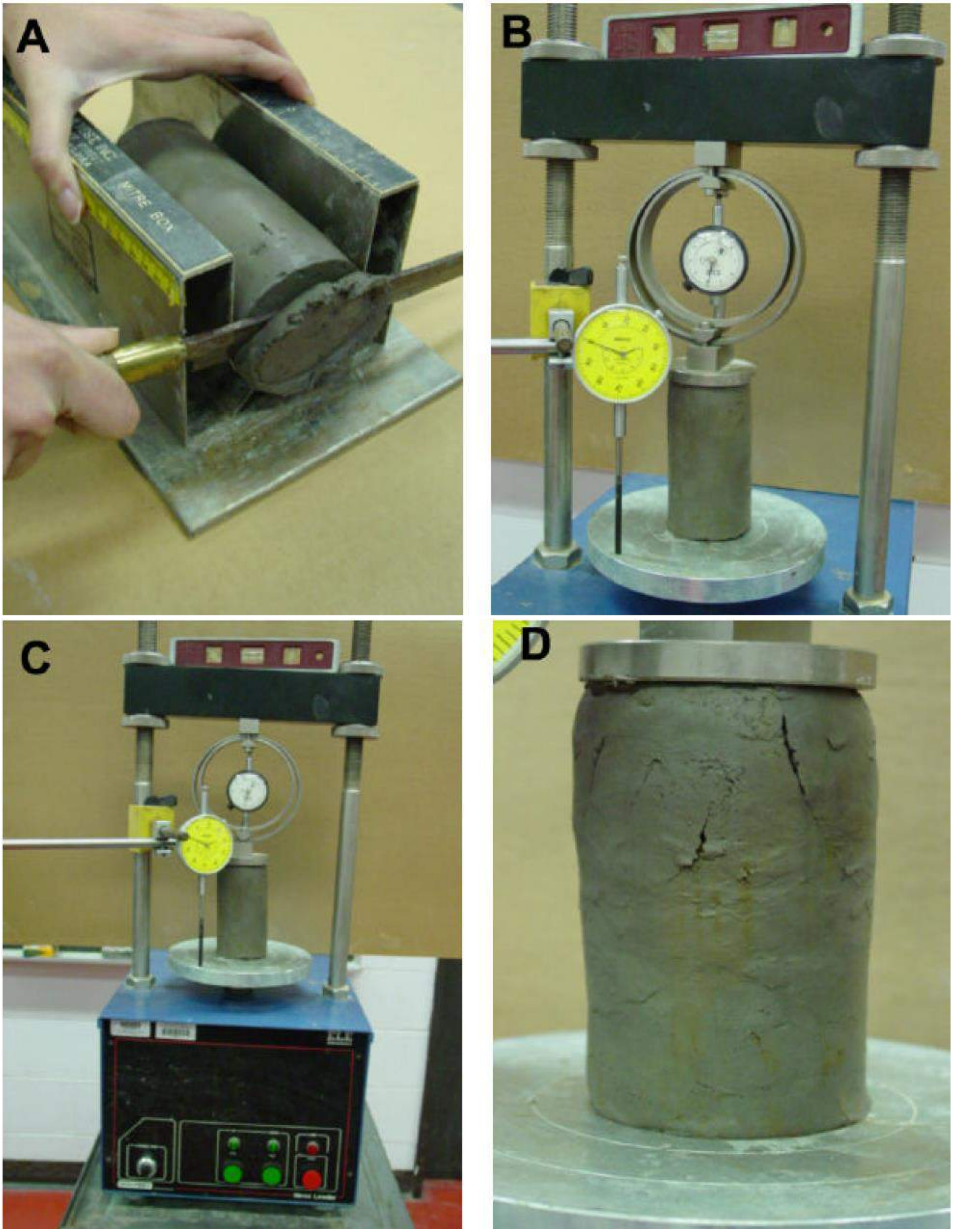
For soils, the undrained shear strength (s_u) is necessary for the determination of the bearing capacity of foundations, dams, etc. The undrained shear strength (s_u) of clays is commonly determined from an unconfined compression test. The undrained shear strength (s_u) of a cohesive soil is equal to one-half the unconfined compressive strength (q_u) when the soil is under the f = 0 condition (f = the angle of internal friction). The most critical condition for the soil usually occurs immediately after construction, which represents undrained conditions, when the undrained shear strength is basically equal to the cohesion (c). This is expressed as:

$$s_u = c = \frac{q_u}{2}$$

Then, as time passes, the pore water in the soil slowly dissipates, and the intergranular stress increases, so that the drained shear strength (s), given by s = c + s ‘tan f, must be used. Where s‘= intergranular pressure acting perpendicular to the shear plane; and s‘= (s - u), s = total pressure, and u = pore water pressure; c’ and j’ are drained shear strength parameters. The determination of drained shear strength parameters is given in Experiment 14

Equipment:

Compression device, Load and deformation dial gauges, Sample trimming equipment, Balance, Moisture can.


A
B
C
D
MITRE BOX

Test Procedure:

1. Extrude the soil sample from Shelby tube sampler. Cut a soil specimen so that the ratio (L/d) is approximately between 2 and 2.5. Where L and d are the length and diameter of soil specimen, respectively.
2. Measure the exact diameter of the top of the specimen at three locations 120° apart, and then make the same measurements on the bottom of the specimen. Average the measurements and record the average as the diameter on the data sheet.
3. Measure the exact length of the specimen at three locations 120° apart, and then average the measurements and record the average as the length on the data sheet.
4. Weigh the sample and record the mass on the data sheet.
5. Calculate the deformation (DL) corresponding to 15% strain (e).

$$\text{Strain (e)} = \frac{\Delta L}{L_o}$$

6. Where L_0 = Original specimen length (as measured in step 3).
7. Carefully place the specimen in the compression device and center it on the bottom plate. Adjust the device so that the upper plate just makes contact with the specimen and set the load and deformation dials to zero.
8. Apply the load so that the device produces an axial strain at a rate of 0.5% to 2.0% per minute, and then record the load and deformation dial readings on the data sheet at every 20 to 50 divisions on deformation the dial.
9. Keep applying the load until (1) the load (load dial) decreases on the specimen significantly, (2) the load holds constant for at least four deformation dial readings, or (3) the deformation is significantly past the 15% strain that was determined in step 5.
10. Draw a sketch to depict the sample failure.
11. Remove the sample from the compression device and obtain a sample for water content determination. Determine the water content as in Experiment 1.

Analysis:

(1) Convert the dial readings to the appropriate load and length units, and enter these values on the data sheet in the deformation and total load columns. (Confirm that the conversion is done correctly, particularly proving dial gage readings conversion into load)
(2) Compute the sample cross-sectional area

$$A_0 = \frac{\pi}{4} \times (d)^2$$

(3) Compute the strain, $e = \frac{?L}{L_0}$

(4) Computed the corrected area, $A' = \frac{A_0}{1-e}$

(5) Using A', compute the specimen stress, $s_c = \frac{P}{A'}$

(Be careful with unit conversions and use constant units).

(6) Compute the water content, w%.

(7) Plot the stress versus strain. Show q_u as the peak stress (or at 15% strain) of the test. Be sure that the strain is plotted on the abscissa. See example data.
(8) Draw Mohr's circle using q_u from the last step and show the undrained shear strength, s_u = c (or cohesion) = $q_u/2$. See the example data.

Example Data

Date Tested: August 30, 2002

Tested By: CEMM315 Class, Group A

Project Name: CEMM315 Lab

Sample Number: ST-1, 8'-10'

Visual Classification: Brown silty clay, medium plasticity, moist CL.

Sample data:

Diameter (d) = 7.29 cm

Length (L_0) = 14.78 cm

Mass = 1221.4 g

Table 1: Moisture Content determination

Sample no.	ST-1, 8-10
Moisture can number - Lid number	A
M_C = Mass of empty, clean can + lid (grams)	15.6
M_{CMS} = Mass of can, lid, and moist soil (grams)	45.7
M_{CDS} = Mass of can, lid, and dry soil (grams)	39.5
M_S = Mass of soil solids (grams)	23.9
M_W = Mass of pore water (grams)	6.2
W = Water content, w%	25.94

$$\text{Area } (A_0) = \frac{p}{4} \times (7.29) = 41..74 \text{ cm}^2$$

$$\text{Volume} = \frac{p}{4} \times (7.29)^2 \times 14.78 = 616.9 \text{ cm}^3$$

$$\text{Wet density} = \frac{1221.4}{616.9} = 1.98 \text{ g/cm}^3$$

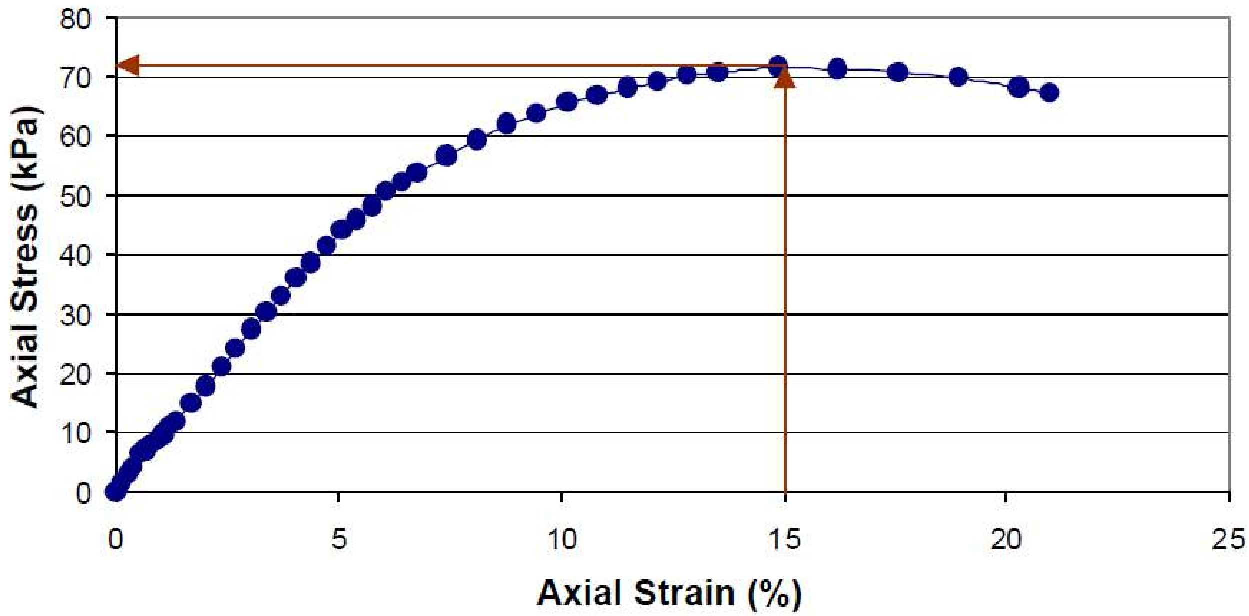
Water content (w%) = 25.9 %

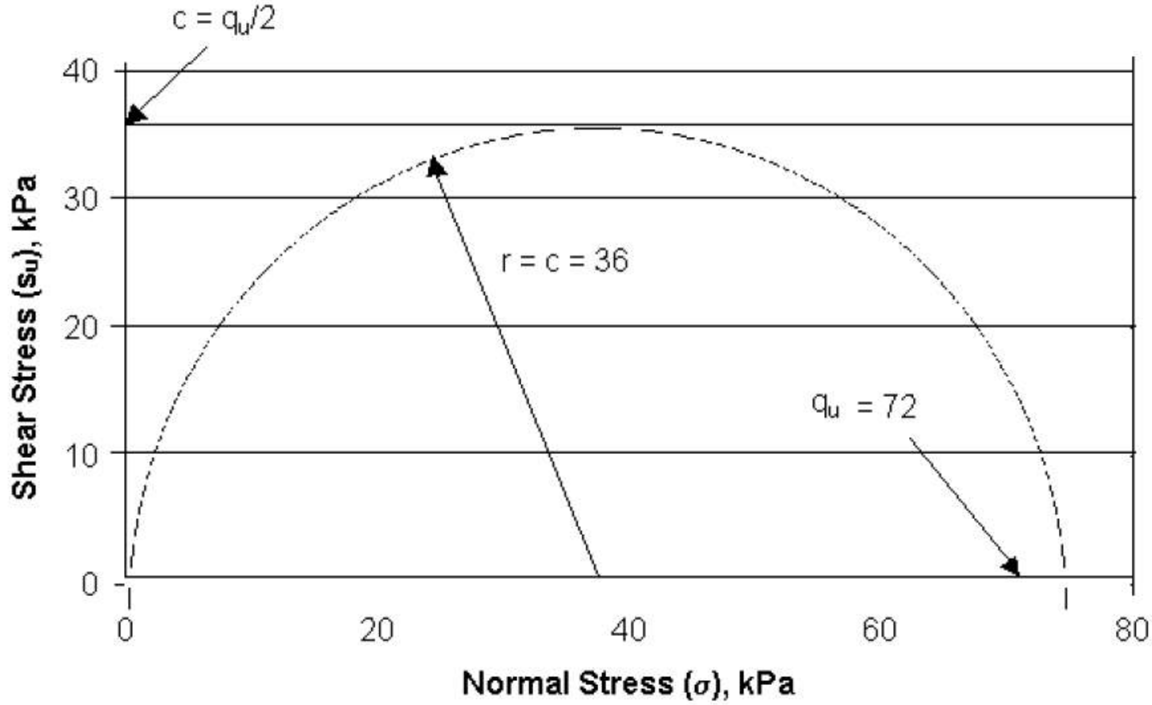
$$\text{Dry density } (?_d) = \frac{1.98}{\left(1 + \frac{25.9}{100}\right)} = 1.57 \text{ g/cm}^3$$

Table 2: Unconfined Compression Test Data (Deformation Dial: 1 unit = 0.10mm; Proving Ring No: 24691; Load Dial: 1 unit = 0.3154 lb)

Deformation Dial Reading	Load Dial Reading	Sample Deformation ΔL (mm)	Strain (ε)	% Strain	Corrected Area A'	Load (lb)	Load (KN)	Stress (kPa)
0	0	0	0.000	0.000	41.739	0.000	0.000	0.000
20	4	0.2	0.001	0.135	41.796	1.262	56.131	1.343
40	9	0.4	0.003	0.271	41.853	2.839	126.295	3.018
60	12	0.6	0.004	0.406	41.909	3.785	168.393	4.018
80	19	0.8	0.005	0.541	41.966	5.994	266.622	6.353
100	21	1	0.007	0.677	42.024	6.625	294.687	7.012
120	24	1.2	0.008	0.812	42.081	7.571	336.786	8.003
140	26	1.4	0.009	0.947	42.138	8.202	364.851	8.658
160	29	1.6	0.011	1.083	42.196	9.148	406.949	9.644
180	33	1.8	0.012	1.218	42.254	10.410	463.080	10.959
200	36	2	0.014	1.353	42.312	11.356	505.178	11.939
250	45	2.5	0.017	1.691	42.457	14.196	631.473	14.873
300	54	3	0.020	2.030	42.604	17.035	757.768	17.786
350	64	3.5	0.024	2.368	42.752	20.189	898.095	21.007
400	74	4	0.027	2.706	42.900	23.344	1038.422	24.205
450	84	4.5	0.030	3.045	43.050	26.498	1178.750	27.381
500	93	5	0.034	3.383	43.201	29.338	1305.044	30.209
550	102	5.5	0.037	3.721	43.353	32.177	1431.339	33.016
600	112	6	0.041	4.060	43.505	35.331	1571.666	36.126
650	120	6.5	0.044	4.398	43.659	37.855	1683.928	38.570
700	129	7	0.047	4.736	43.814	40.694	1810.223	41.316
750	138	7.5	0.051	5.074	43.971	43.533	1936.517	44.041
800	144	8	0.054	5.413	44.128	45.426	2020.714	45.792
850	152	8.5	0.058	5.751	44.286	47.950	2132.976	48.163
900	160	9	0.061	6.089	44.446	50.473	2245.237	50.516
950	166	9.5	0.064	6.428	44.606	52.366	2329.434	52.222
1000	171	10	0.068	6.766	44.768	53.943	2399.598	53.600
1100	182	11	0.074	7.442	45.096	57.413	2553.958	56.634
1200	192	12	0.081	8.119	45.428	60.568	2694.285	59.309
1300	202	13	0.088	8.796	45.765	63.722	2834.612	61.939
1400	209	14	0.095	9.472	46.107	65.931	2932.841	63.610
1500	217	15	0.101	10.149	46.454	68.454	3045.103	65.551
1600	223	16	0.108	10.825	46.806	70.347	3129.300	66.856
1700	229	17	0.115	11.502	47.164	72.240	3213.496	68.134
1800	234	18	0.122	12.179	47.527	73.817	3283.660	69.090
1900	240	19	0.129	12.855	47.896	75.710	3367.856	70.315
2000	243	20	0.135	13.532	48.271	76.656	3409.954	70.642
2200	250	22	0.149	14.885	49.039	78.864	3508.184	71.539
2400	253	24	0.162	16.238	49.831	79.811	3550.282	71.247
2600	255	26	0.176	17.591	50.649	80.442	3578.347	70.650
2800	256	28	0.189	18.945	51.495	80.757	3592.380	69.762
3000	254	30	0.203	20.298	52.369	80.126	3564.314	68.062

SAMPLE: ST-1, 8'-10'





From the stress-strain curve and Mohr's circle:

Unconfined compressive strength (q_u) = 72.0 KPa

Cohesion (c) = 36.0 KPa

UNCONFINED COMPRESSION TEST
DATA SHEET

Date Tested:
Tested By:
Project Name:
Sample Number:
Visual Classification:

Sample data:

Diameter (d) =
Length (L_0) =
Mass =

Table 1: Moisture Content determination

Sample no.	
Moisture can number - Lid number	
M_C = Mass of empty, clean can + lid (grams)	
M_{CMS} = Mass of can, lid, and moist soil (grams)	
M_{CDS} = Mass of can, lid, and dry soil (grams)	
M_S = Mass of soil solids (grams)	
M_W = Mass of pore water (grams)	
W = Water content, w%	

Area (A_0) =
Volume =
Wet density =
Water content (w%) =
Dry density ($?_d$) =

Table 2: Unconfined Compression Test Data (Deformation Dial: 1 unit = 0.10mm
Proving Ring No: 24691; Load Dial: 1 unit = 0.3154 lb)

Deformation Dial Reading	Load Dial Reading	Sample Deformation ΔL (mm)	Strain (ε)	% Strain	Corrected Area A'	Load (lb)	Load (KN)	Stress (kPa)
0								
20								
40								
60								
80								
100								
120								
140								
160								
180								
200								
250								
300								
350								
400								
450								
500								
550								
600								
650								
700								
750								
800								
850								
900								
950								
1000								
1100								
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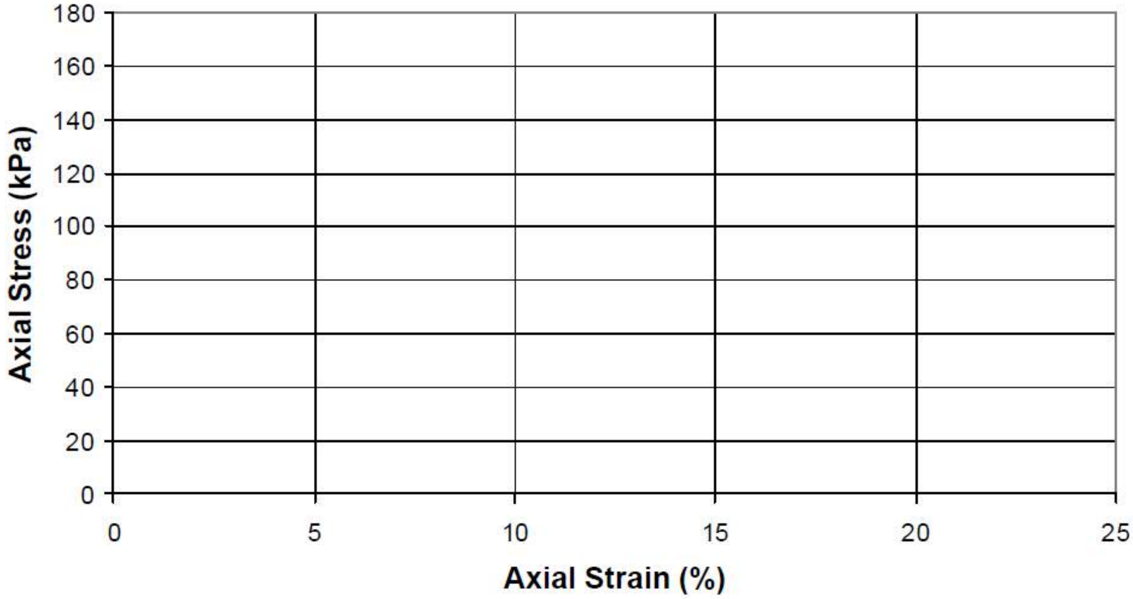
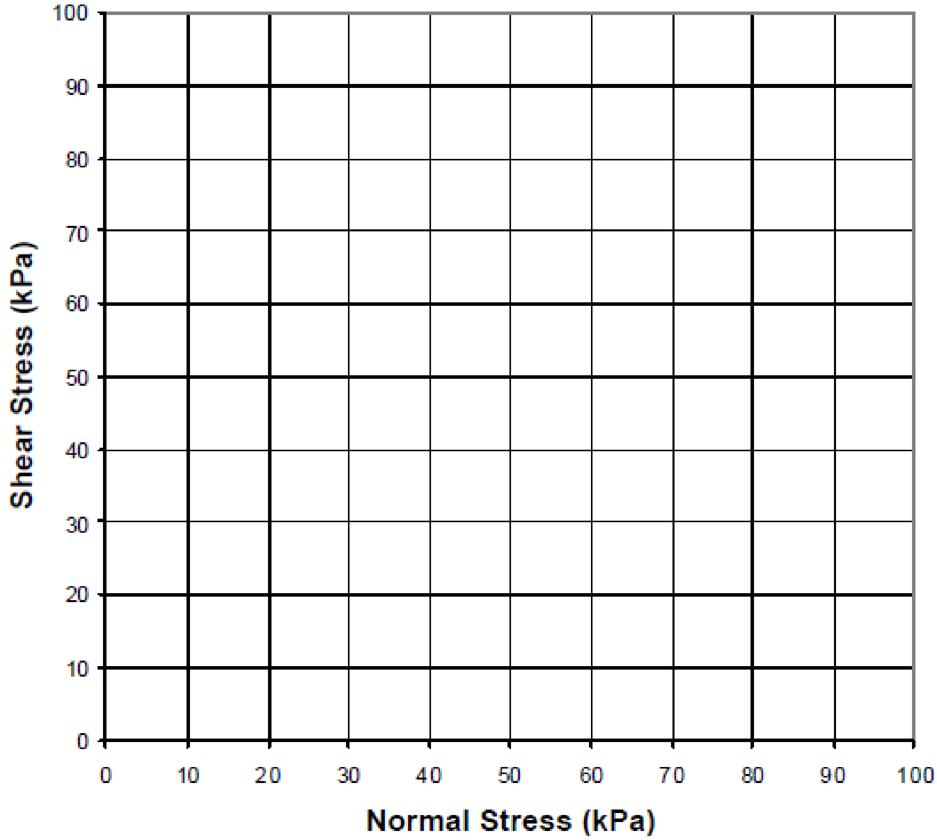
2200								
2400								
2600								
2800								
3000								





From the stress-strain curve and Mohr's circle:

Unconfined compressive strength (q_u) =

Cohesion (c) =

Summary

1. The term 'Soil' is defined and the development of soil mechanics or geotechnical engineering as a discipline in its own right is traced.
2. Foundations, underground and earth-retaining structures, pavements, excavations, embankments and dams are the fields in which the knowledge of soil mechanics is essential.
3. The formation of soils by the action of various agencies in nature is discussed, residual soils and transported soils being differentiated. Some commonly used soil designations are explained.
4. The structure and texture of soils affect their nature and engineering performance. Single-grained structure is common in coarse grained soils and honey-combed and flocculent structures are common in fine-grained soils.
5. The soil-material is considered to be a homogeneous mechanical mixture of two phases: one phase represents the structure of solid particles in the soil aggregate and the other phase represents the fluid water in the pores or voids of the aggregate. It is more difficult to understand this soil-material than the mechanically simple perfectly elastic or plastic materials, so most of the book is concerned with the mechanical interaction of the phases and the stress – strain properties of the soil-material in bulk. Much of this work is of interest to workers in other fields, but as we are civil engineers we will take particular interest in the standard tests and calculations of soil mechanics and foundation engineering.
6. A useful tool in engineering is the analysis of the behavior of a structure by doing a model test, at a reduced scale. The purpose of the test may be just to investigate a phenomenon in a qualitative way, but more often its purpose is to obtain quantitative information. In that case the scale rules must be known. For a soil a special difficulty is that the mechanical properties often depend upon the state of stress, which is determined to a large extent by the weight of the soil itself. This means that in a scale model the soil properties are not well represented, because in the model the stresses are much smaller than in reality (the prototype).

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Appendix

Engineering Properties of Soil and Rock
Table 6-1 Commonly Performed Laboratory Tests on Soils

Test Category	Name of Test	Test Designation	
		AASHTO	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

(Samtani and Nowatzki, 2006)

Table 6-2 Methods for Index Testing of Soils (Samtani and Nowatzki, 2006)

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 °C	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 °C (230° F), the sample is ignited in a muffle furnace at 440 °C (824° 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
- e_o: in-situ void ratio
- ρ_{dry}: dry density
- γ_{dry}: dry unit weight
- ρ_{tot}: total density
- γ: unit weight
- γ_t: total unit weight
- p_t: total vertical stress

Table 6-3 Methods for Performance Testing of Soils (Samtani and Nowatzki, 2006)

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.
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 $\leq 1\text{x}10^{-5}$ 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

- 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

Table 1.1 Soil—separate size limits

Agency	*Classification*	*Size limits (mm)*
U.S. Department of Agriculture (USDA)	Gravel	> 2
	Very coarse sand	2–1
	Coarse sand	1–0.5
	Medium sand	0.5–0.25
	Fine sand	0.25–0.1
	Very fine sand	0.1–0.05
	Silt	0.05–0.002
	Clay	< 0.002
International Society of Soil Mechanics (ISSS)	Gravel	> 2
	Coarse sand	2–0.2
	Fine sand	0.2–0.02
	Silt	0.02–0.002
	Clay	< 0.002
Federal Aviation Agency (FAA)	Gravel	> 2
	Sand	2–0.075
	Silt	0.075–0.005
	Clay	< 0.005
Massachusetts Institute of Technology (MIT)	Gravel	> 2
	Coarse sand	2–0.6
	Medium sand	0.6–0.2
	Fine sand	0.2–0.06
	Silt	0.06–0.002
	Clay	< 0.002
American Association of State Highway and Transportation Officials (AASHTO)	Gravel	76.2–2
	Coarse sand	2–0.425
	Fine sand	0.425–0.075
	Silt	0.075–0.002
	Clay	< 0.002
Unified (U.S. Army Corps of Engineers, U.S. Bureau of Reclamation, and American Society for Testing and Materials)	Gravel	76.2–4.75
	Coarse sand	4.75–2
	Medium sand	2–0.425
	Fine sand	0.425–0.075
	Silt and clay (fines)	< 0.075

Braja M. Das, 2008 "Advanced Soil Mechanics, Third edition"