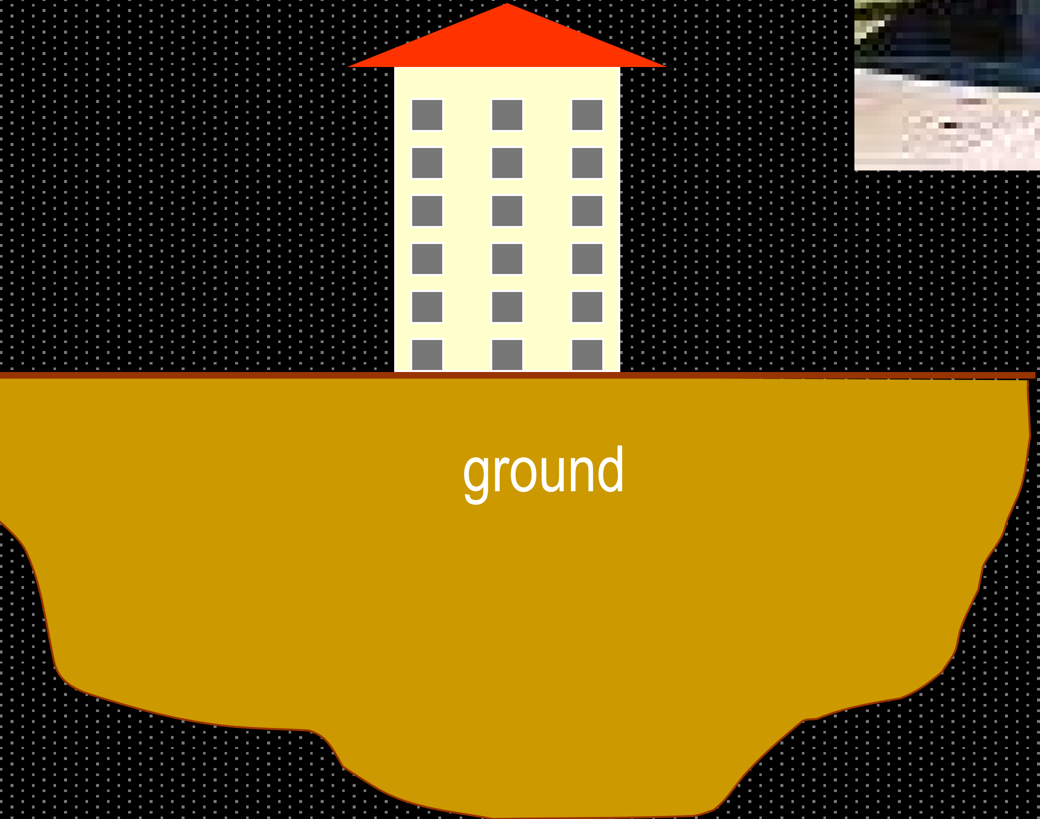
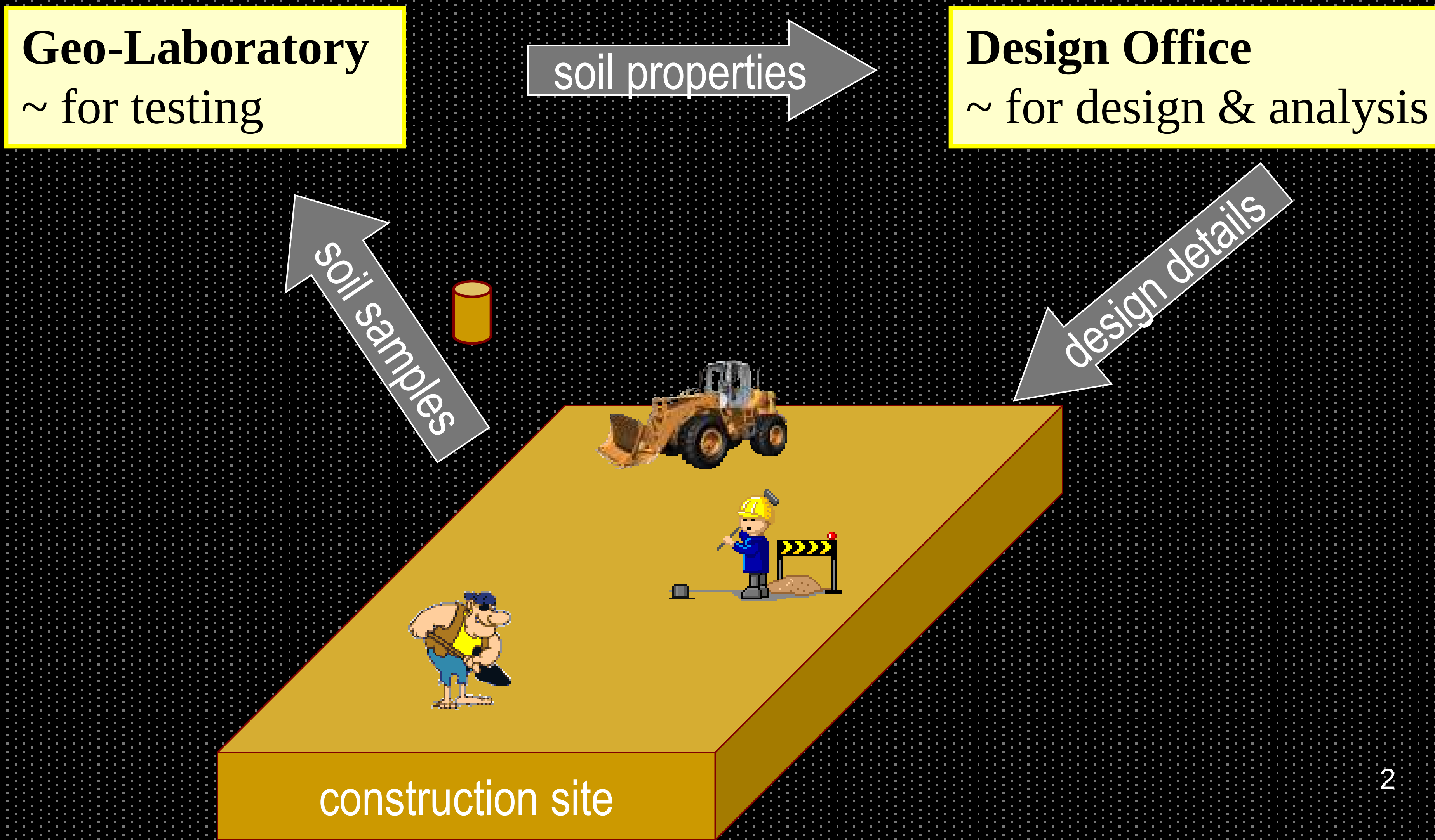
Introduction to Geotechnical Engineering



Typical Geotechnical Project

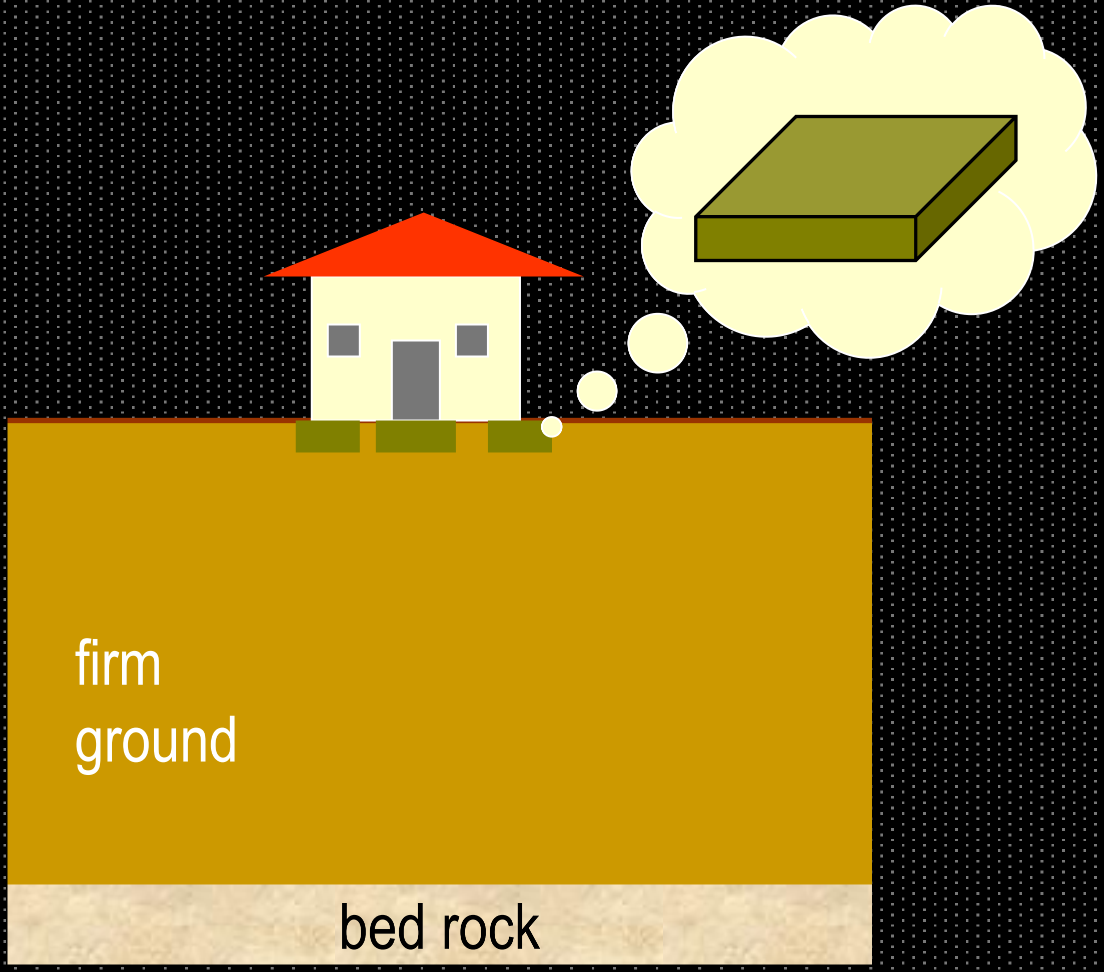


Geotechnical Applications

Shallow Foundations

~ for transferring building loads to underlying ground

~ mostly for firm soils or light loads

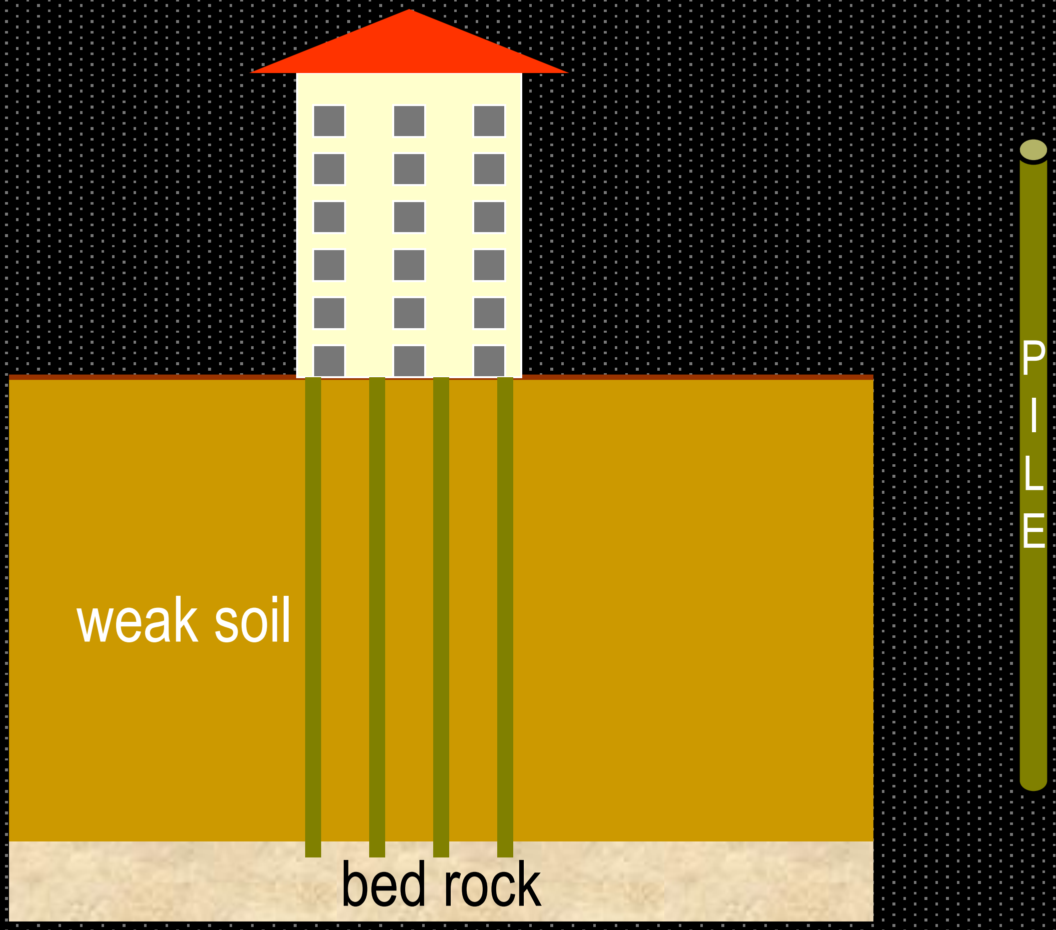


Shallow Foundations

Deep Foundations

~ for transferring building loads to underlying ground

~ mostly for weak soils or heavy loads



Deep Foundations

Driven timber piles, Pacific Highway

Pier Foundations for Bridges

<u>Millau Viaduct in France (2005)</u>

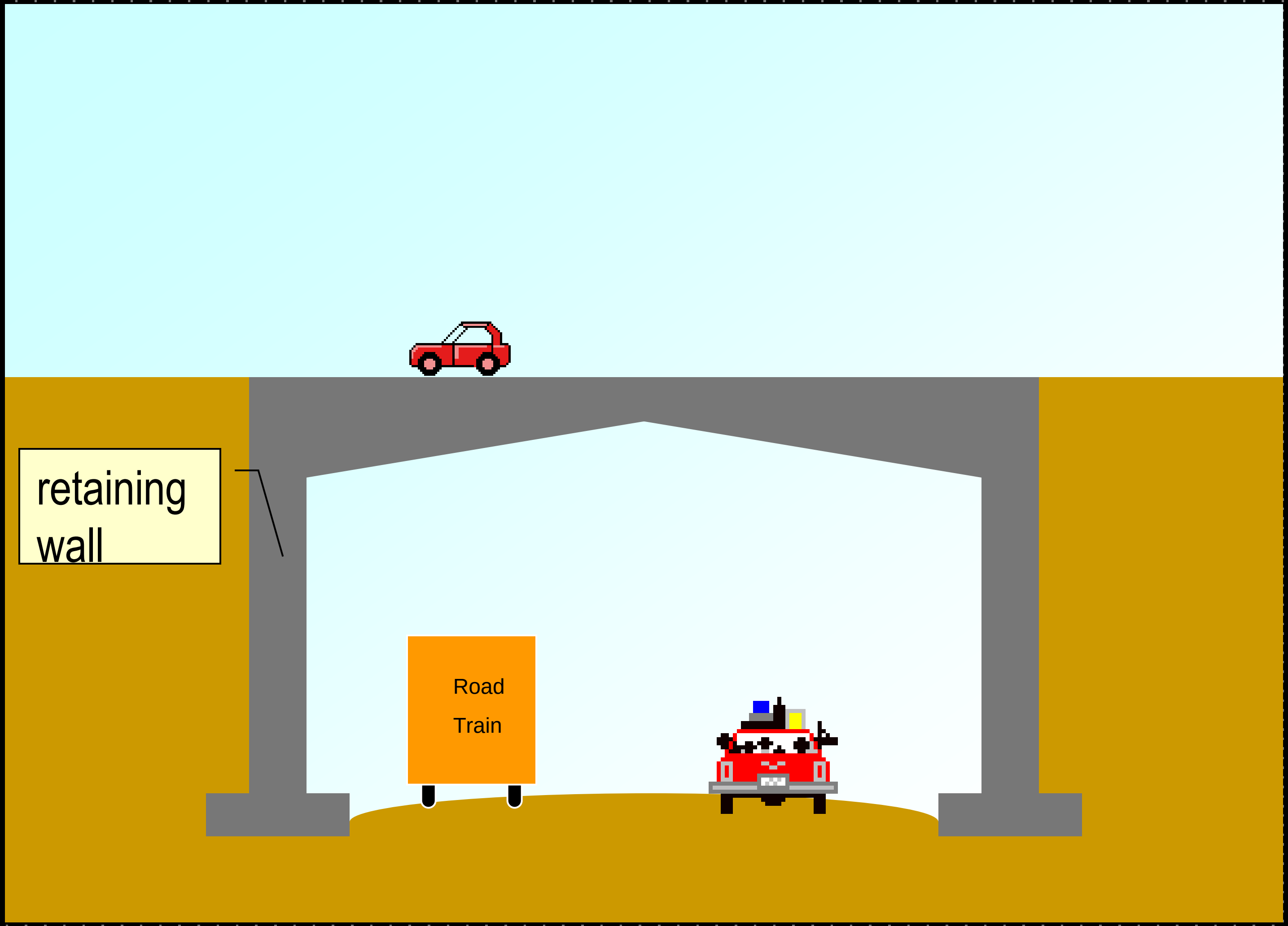
- Cable-stayed bridge
- Supported on 7 piers, 342 m apart
- Longest pier (336) in the world

Pier Foundations for Bridges

Millau Viaduct in France (2005)

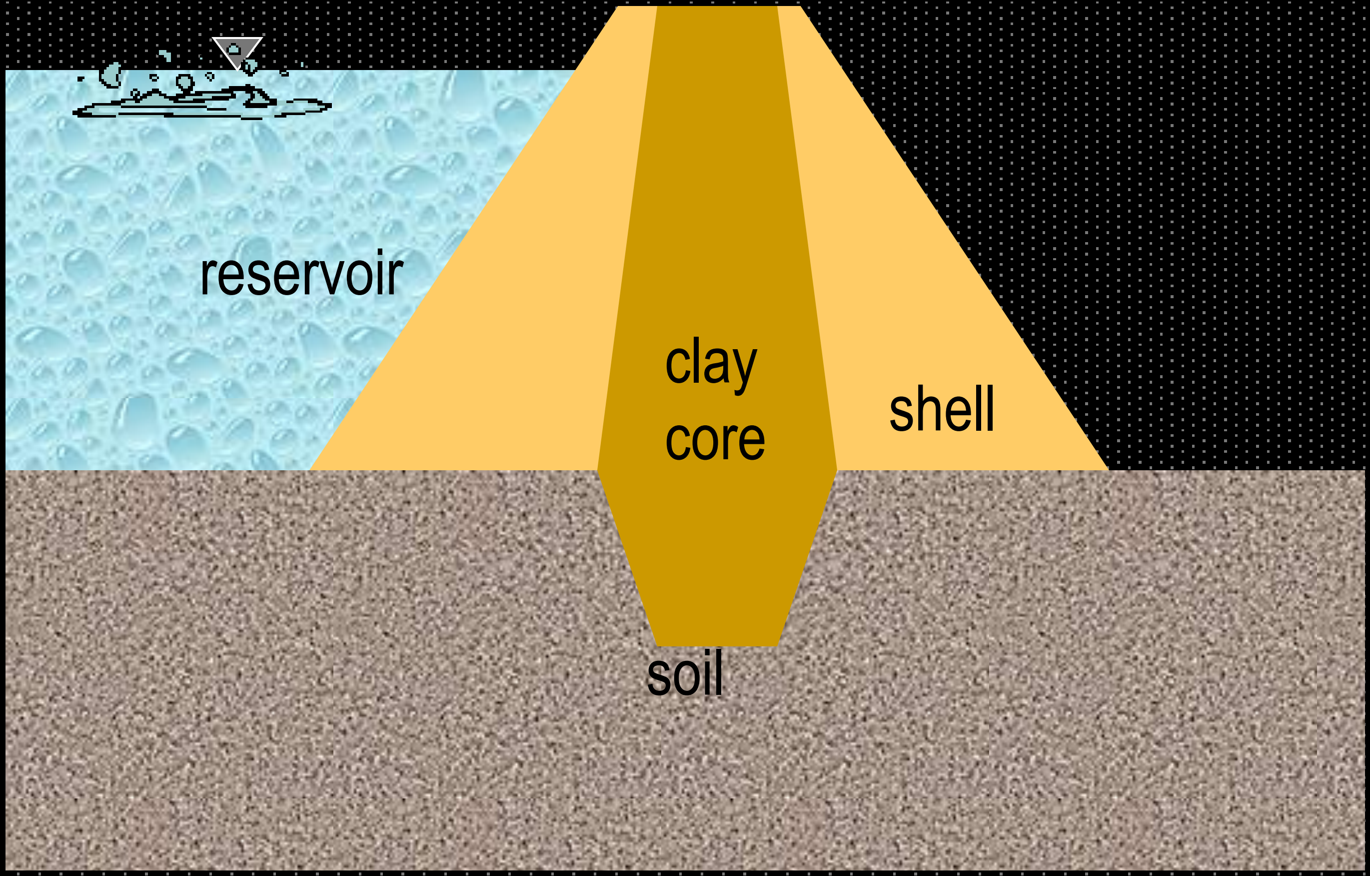
Retaining Walls

~ for retaining soils from spreading laterally

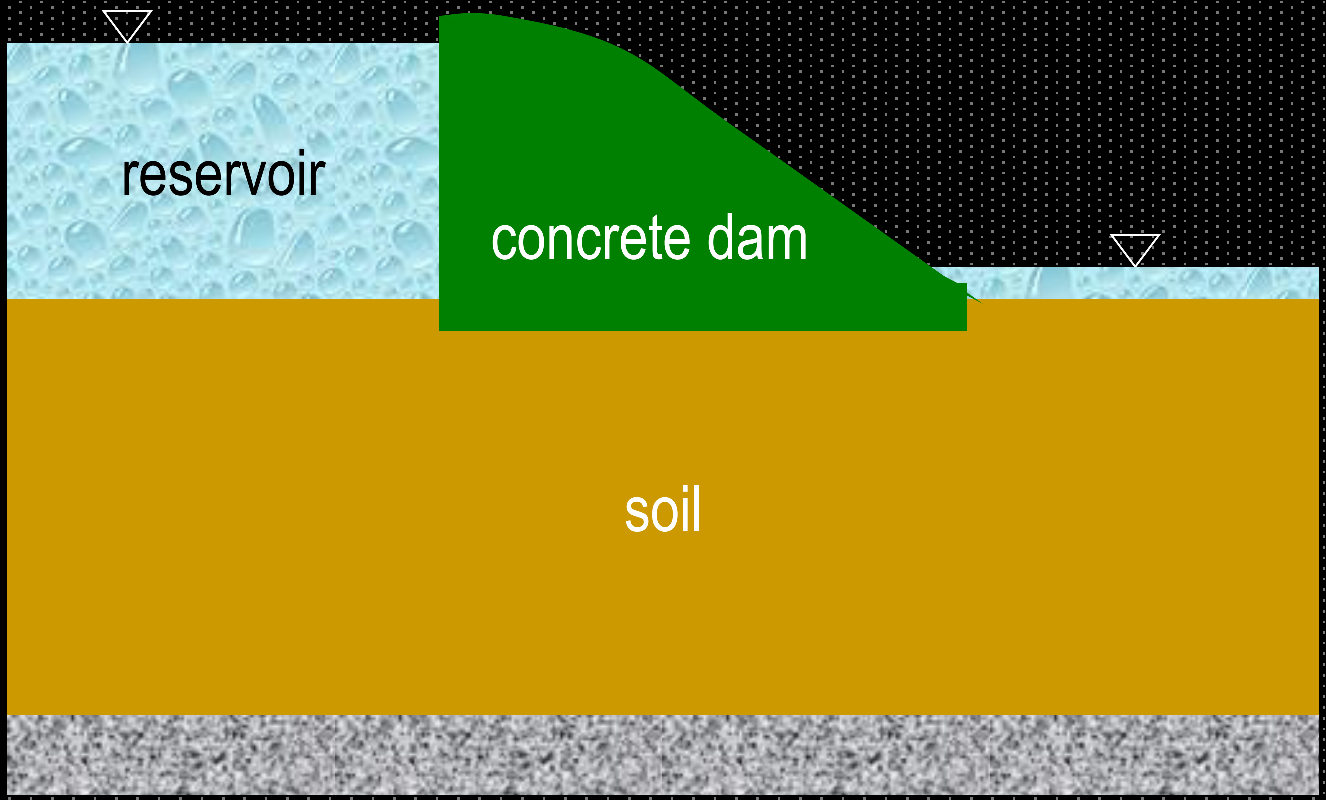


Earth Dams

~ for impounding water



Concrete Dams



Concrete Dams

Three Gorges Dam, Hong Kong

Concrete Dams

Earthworks

~ preparing the ground prior to construction

Roadwork, Pacific Highway

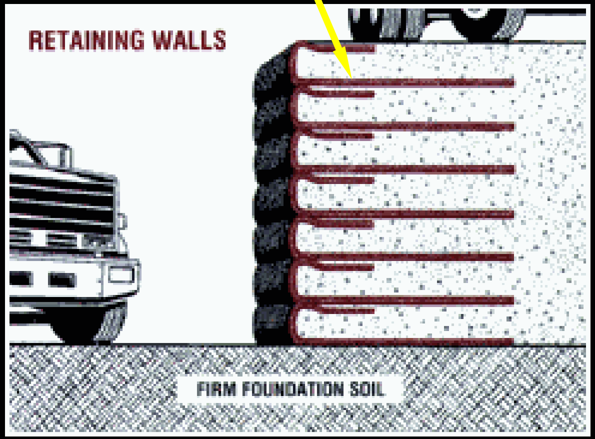
Geofabrics

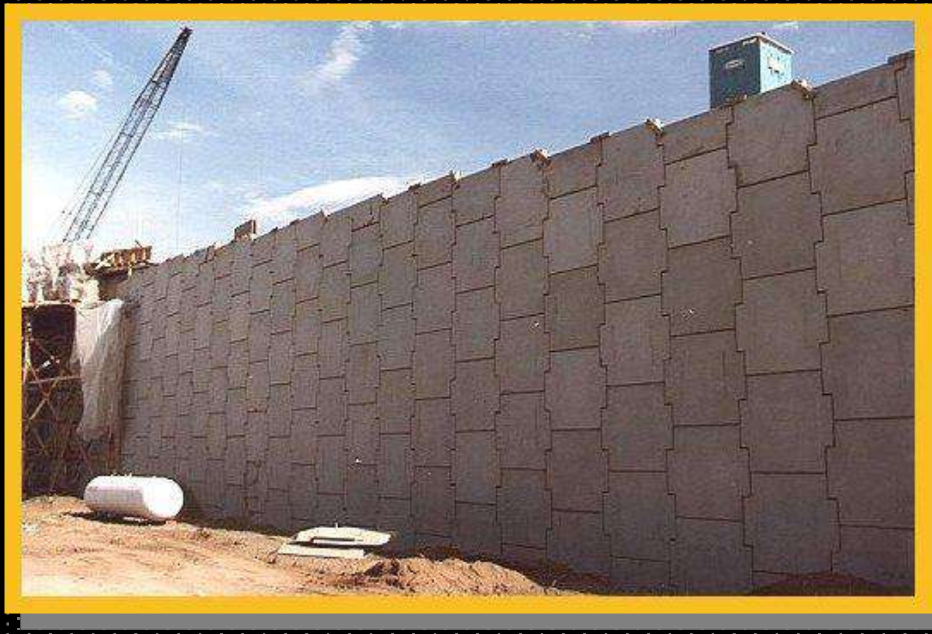
~ used for reinforcement, separation, filtration and drainage in roads, retaining walls, embankments…

Geofabrics used on Pacific Highway

Reinforced Earth Walls

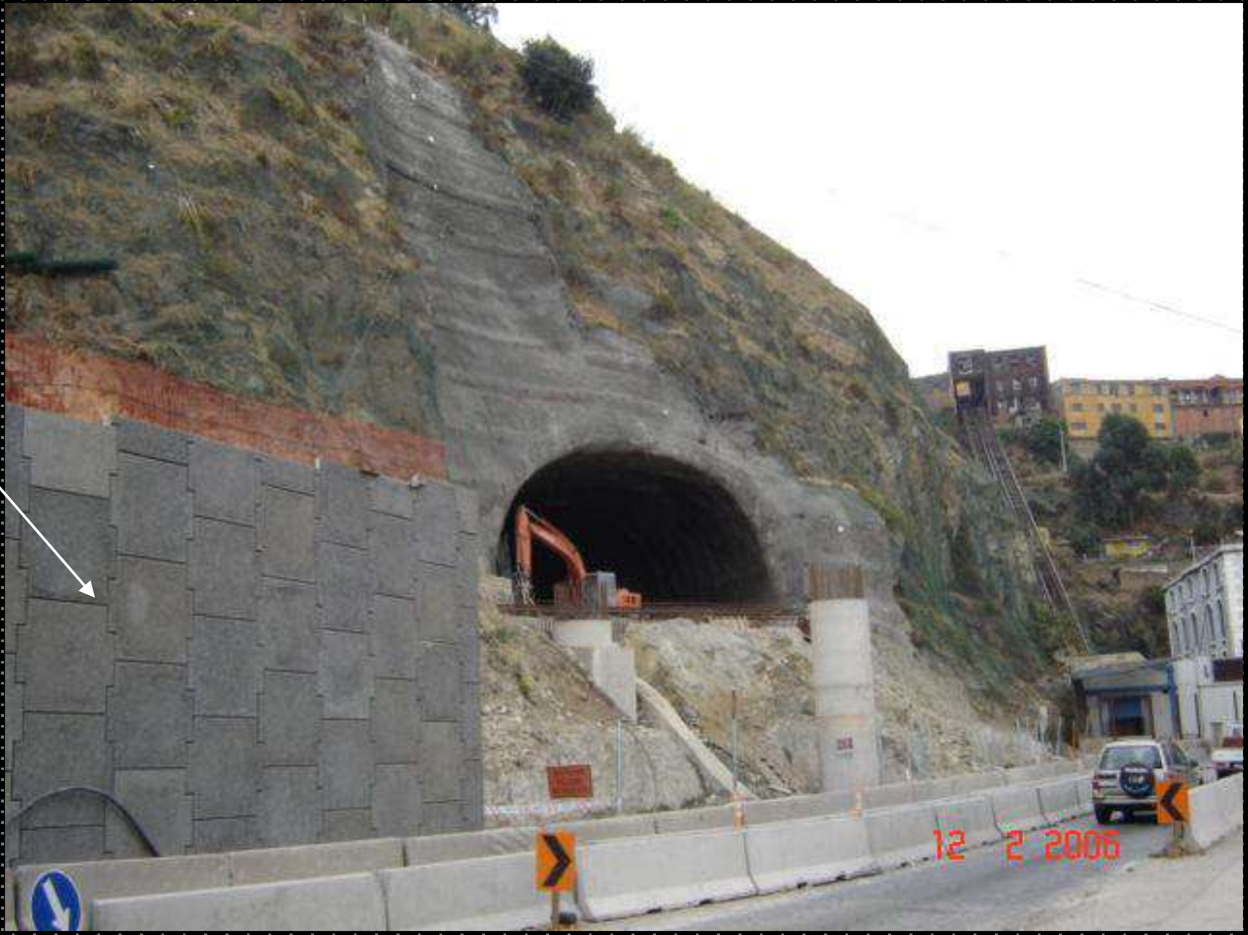
~ using geofabrics to strengthen the soil





Tunneling

MSE (Mechanically stabilized Earth) wall



Chile (2006)

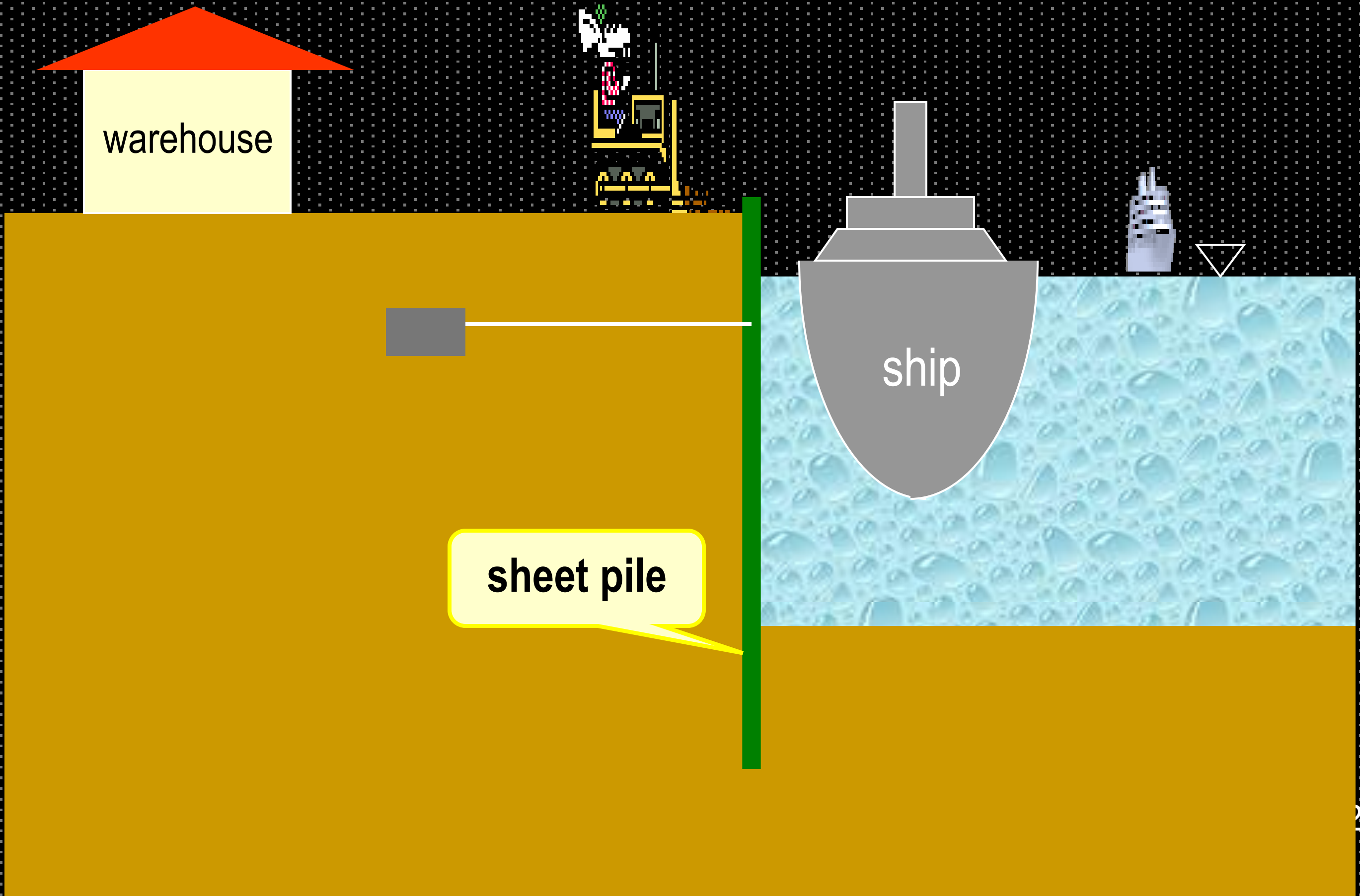
Tunneling

Retaining Walls

Rock anchors to support the vertical walls

Sheet Piles

~ sheets of interlocking-steel or timber driven into the ground, forming a continuous sheet



Sheet Piles

~ resist lateral earth pressures

~ used in excavations, waterfront structures, ..

Sheet Piles

~ used in temporary works

Cofferdam

~ sheet pile walls enclosing an area, to prevent water seeping in

Cofferdam

~ sheet pile walls enclosing an area, to prevent water seeping in

Landslides

Shoring

propping and supporting the exposed walls to resist lateral earth pressures

Excavations

Chile (2006)

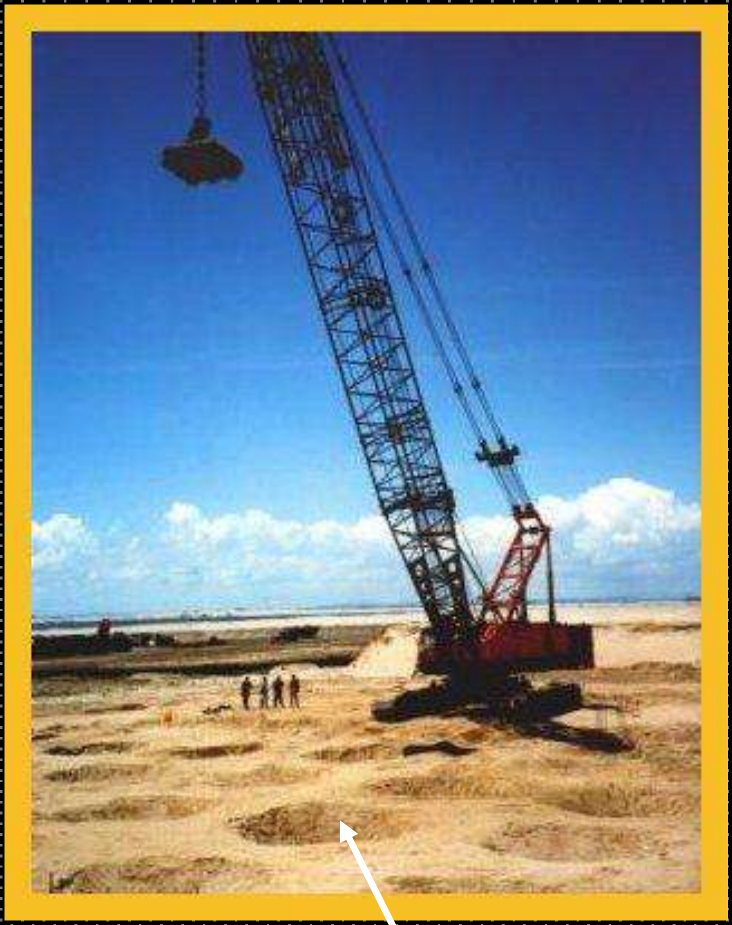
Earthquake Engineering

Loma Preita Earthquake, San Francisco (1989)

Ground Improvement

Impact Roller to Compact the Ground

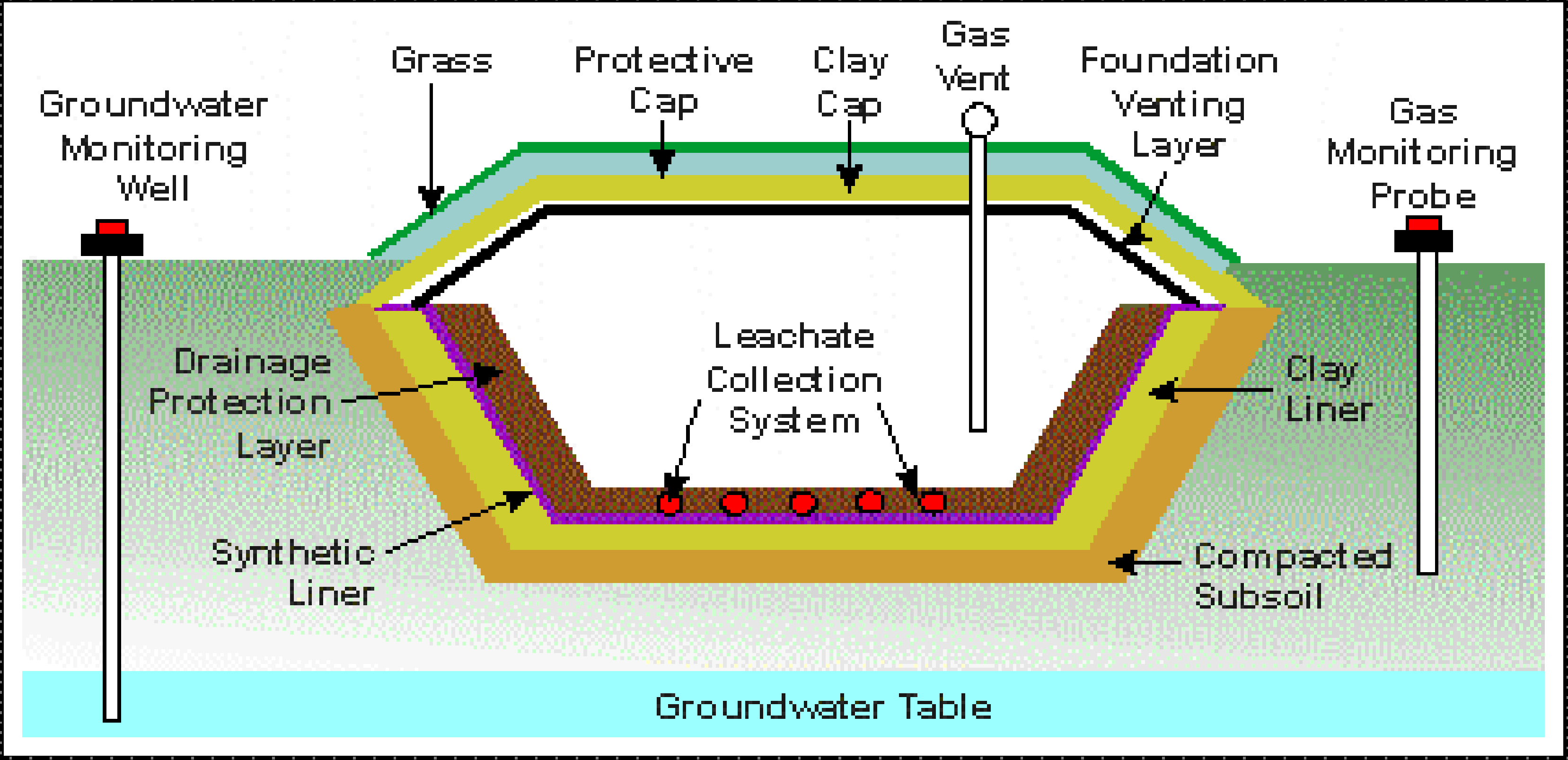
Ground Improvement



Big weights dropped from 25 m, compacting the ground.

Craters formed in compaction

Environmental Geomechanics



Waste Disposal in Landfills

Instrumentation

~ to monitor the performances of earth and earth supported structures

~ to measure loads, pressures, deformations, strains,…

Soil Testing

Vane Shear Test

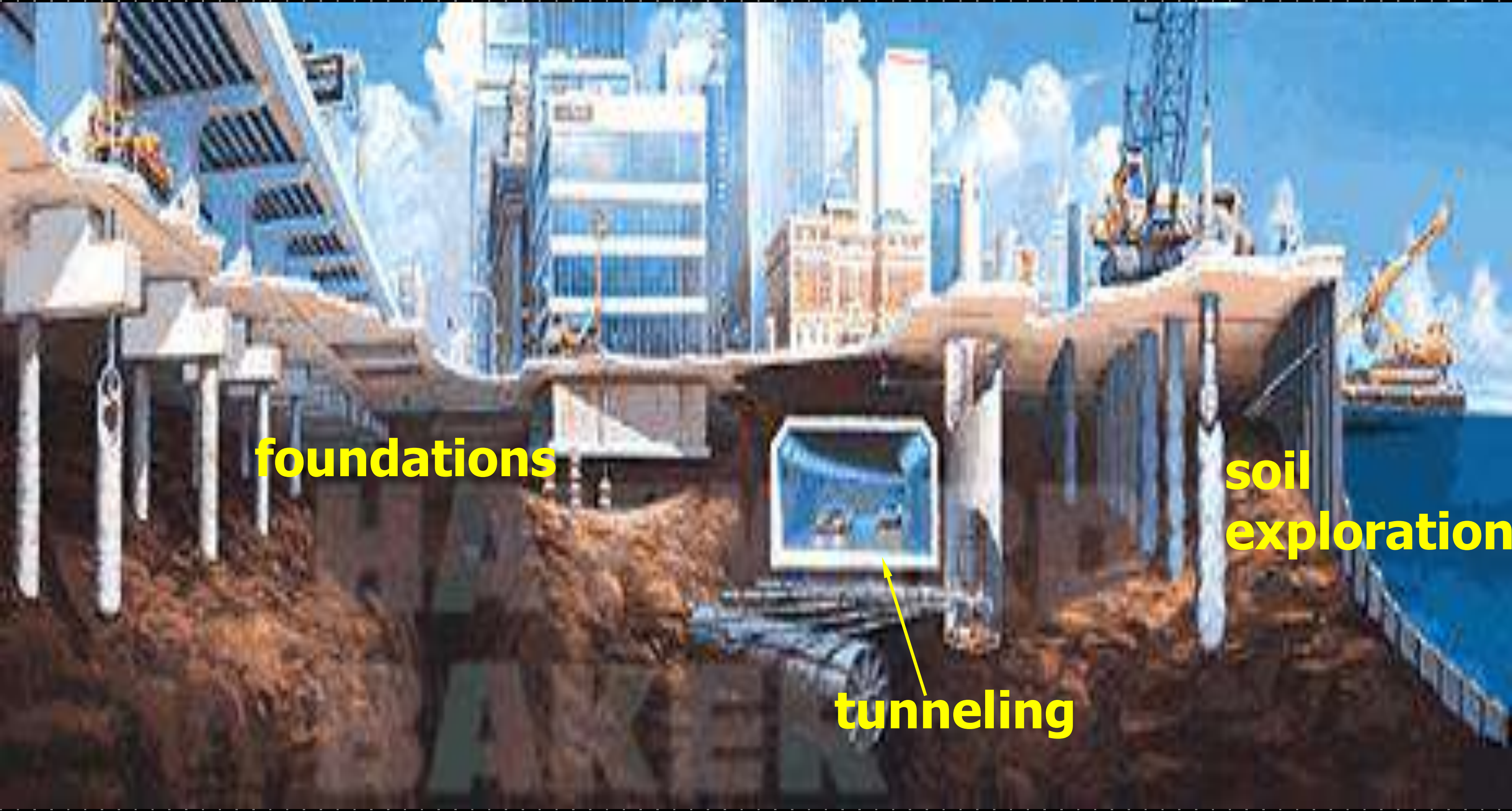
Standard Penetration Test

More Field Tests

Soil Testing

Triaxial Test on Soil Sample in Laboratory

Some Civil Engineering marvels ....



... buried right under your feet.

Sea wall in Brisbane (2005)

Courtesy: Coffey Geosciences Pty Ltd.

Hall of Fame

Great Contributors to the Developments in Geotechnical Engineering

Karl Terzaghi
1883-1963

C.A.Coulomb
1736-1806

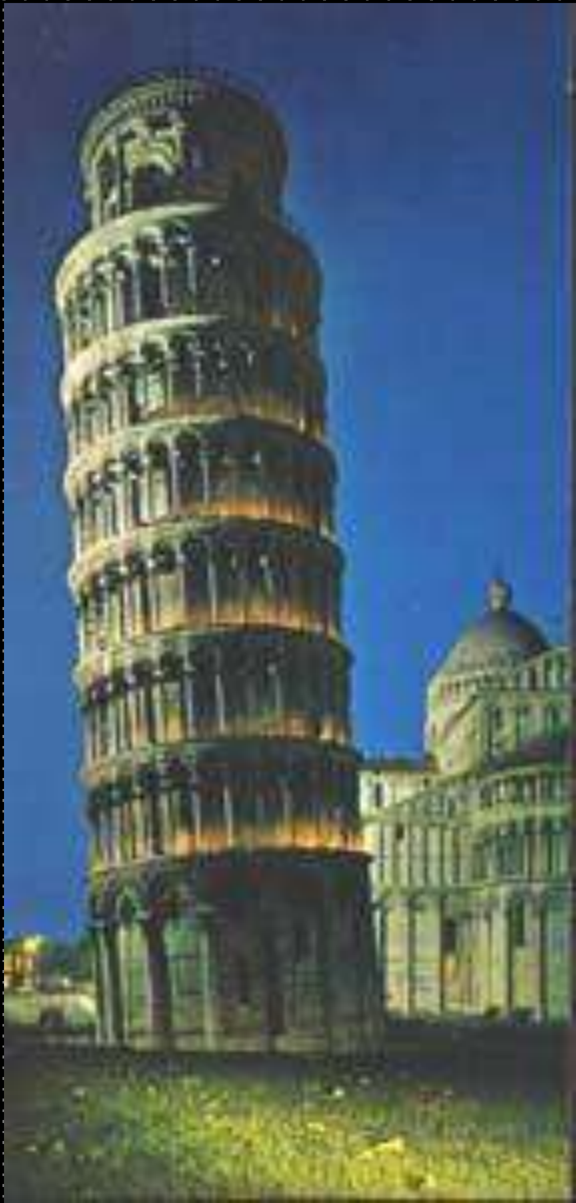
M. Rankine
1820-1872

Challenges

Geotechnical Engineering Landmarks

Leaning Tower of Pisa

Our blunders become monuments!



Hoover Dam, USA

Tallest (221 m) concrete dam

Burj Dubai (Khalifa Tower)

the tallest man-made structure ever built, at 818 m (2,684 ft).

Geotechnical Engineering (CE 336)

Geo-engineering at HU

110401336 (CE 336)
Introduction to Geotechnical Engineering

110401338 (CE338)
Geotechnical Engineering Laboratory

110401436 (CE436)
Engineering Geology

110401435 (CE435)
Foundation Engineering and Design

110401531(CE531)
Soil Stabilization and Ground Reinforcement

Course Description

- Index and classification of soils, water flow in soils (one and two dimensional water flow), soil stresses, soil compaction, distribution of stresses in soil due to external loads, consolidation and consolidation settlement, shear strength of soils, slope stability.

Course Objectives

1. To know and understand the formation and mineralogy of soils especially the clay minerals
2. To understand the classification and index properties of soils and the importance of soil classification on defining and integrating the engineering properties of soils, which in tern affect the engineering purpose
3. To know and understand the mechanical stabilization of soil (compaction)
4. To evaluate soil stresses due to the weight of overburden soil and external stresses. Stress evaluation is very important for soil shear strength and settlement calculations

Course Objectives

5. To understand the mechanism of water flow through the soil mass and the effect of this flow on soil effective stresses.
6. To understand consolidation (compression), rate of consolidation, and settlement of soils under the change in soil stresses.
7. To understand and evaluate the soil shear strength which is a very important aspect in geotechnical engineering. Soil shear strength is very important in evaluating foundation bearing capacity, slope stability, earth retaining wall design, pavement design, and so on.
8. To understand and evaluate the slope stability problems. Slopes could be natural, sloped formed by excavation, embankment slopes, and earth dam slopes.

Course Outline

- ***(weeks-1&2)***
- **Introduction to Geotechnical Engineering**
- **Formation of Soils and Mineralogy of Soil Solids as Geotechnical Materials**
  - Formation of Soils
  - Soil Profile
  - Mineralogy of Soil Solids
  - Clay Minerals

Course Outline

- ***(week-3&part of week-4)***
- **Index Properties and Classification of Soils**
  - Basic Definitions and Phase Relations
  - Solution of Phase Problems
  - Role of Classification System in Geotechnical Engineering
  - Soil Texture, Grain Size, and Grain Size Distribution
  - Atterberg Limits and Consistency Indices
  - Unified Soil Classification System
- ***(Rest of week-4)***
- **Soil Compaction**
  - Compaction
  - Theory of Compaction
  - Density-Water content (Compaction Curve) of Soils
  - Field Compaction Control and Specification
  - Relative Density of Cohesionless Soils

Course Outline

- Stress*(week-5)*
- **Soil Effective Stresses**
  - Effective Vertical Stress
  - Capillarity and Stresses in Capillarity Zone
  - Response of Effective Stress to a Change in Total Stress
  - Relationship between Horizontal and Vertical
- ***(Weeks-6&7***)
- **Water in Soils (Permeability, Seepage, and Effective Stresses)**
  - Introduction
  - Darcy's Law for Flow
  - Bernoulli Energy Equation for Steady Flow
  - Total, Pressure, and Elevation Heads
  - One Dimensional Flow and Measurement of Permeability
  - Factors Affect the Permeability
  - Permeability in Multi-layer Soil Profile
  - Seepage Forces, Quicksand, and Liquifaction
- Seepage and Flow nets (Two-Dimensional Flow

Course Outline

- ***(week-8)***
- **Stress Distribution in Soils Due to External Loading**
  - Point Loading
  - Line Loading
  - Uniform loading Distributed over Rectangular and Circular Areas
  - Strip Loading
- ***(Weeks-9&10 and part of week-11)***
- **Soil Consolidation, Consolidation Settlement, and Rate of Consolidation**
  - Components of Settlements
  - The Oedometer and Consolidation Testing (One-Dim. Cons.)
  - Pre-consolidation Pressure
  - Settlement Calculations
  - Prediction of Field Consolidation Curves
  - Consolidation Process
  - Terzaghi’s One-Dim. Consolidation Theory
  - Evaluation of Secondary Settlement
  - Determination of Immediate Settlement

Course Outline

- ***(Rest of week 11 and weeks-12&13)***
- **Shear Strength of Soils**
  - Shearing Resistance
    - Granular Soils (Cohesionless Soils)
    - Clay Soils (Cohesive Soils)
    - Shear Strength
    - Failure
    - Mohr's Theory of Failure
    - Mohr-Coulomb Envelope in Terms of Principal Stress
    - Drained Versus Undrained Shear Strength
    - Measurement of Shear Strength in Laboratory (Triaxial Tests)
      - CD-Tests
      - CU-Tests
      - UU-Tests
    - Shear Strength of Cohesionless (Granular Soils)
    - Shear Induced Pore Water Pressures
    - Stress Paths
    - Soil Sensitivity

Course Outline

- ***(week-14)***
- **Stability of Slopes**
  - Type of slope failure
  - Analysis of a plane translational slip
  - Analysis of rotational, circular slips
  - The method of slices

Course Outline

Course Requirements

1. Attending the lectures (no make up between lectures)
2. Late coming to lectures (-3 minute from start consider absentee)
3. Home reading assignments for the related topics
4. Home works and quizzes
5. Exams
6. **No make up exams will be provided**

Grade distribution

1.	Quizzes	0 %
2.	Attendance	0 %
3.	First Exam	30%
4.	Second Exam	30%
5.	Final exam	40%

Office Hours

Office Hours

- Sunday 09:00-10:00
- Tuesday: 09:00-10:00
- Wednesday: 11:00-12:00
- Thursday: 09:00-10:00
- No office hours in the exams’ Day and if the OH coincides with Department or University Event

Dates of Exams

Dates of Exams:

- First exam: Sunday Nov. 12, 2017
- Second Exam: Sunday Dec. 17, 2017
- Final Exam: will be determined by the registrar

No make up exams whatsoever

(Absent =0.0)

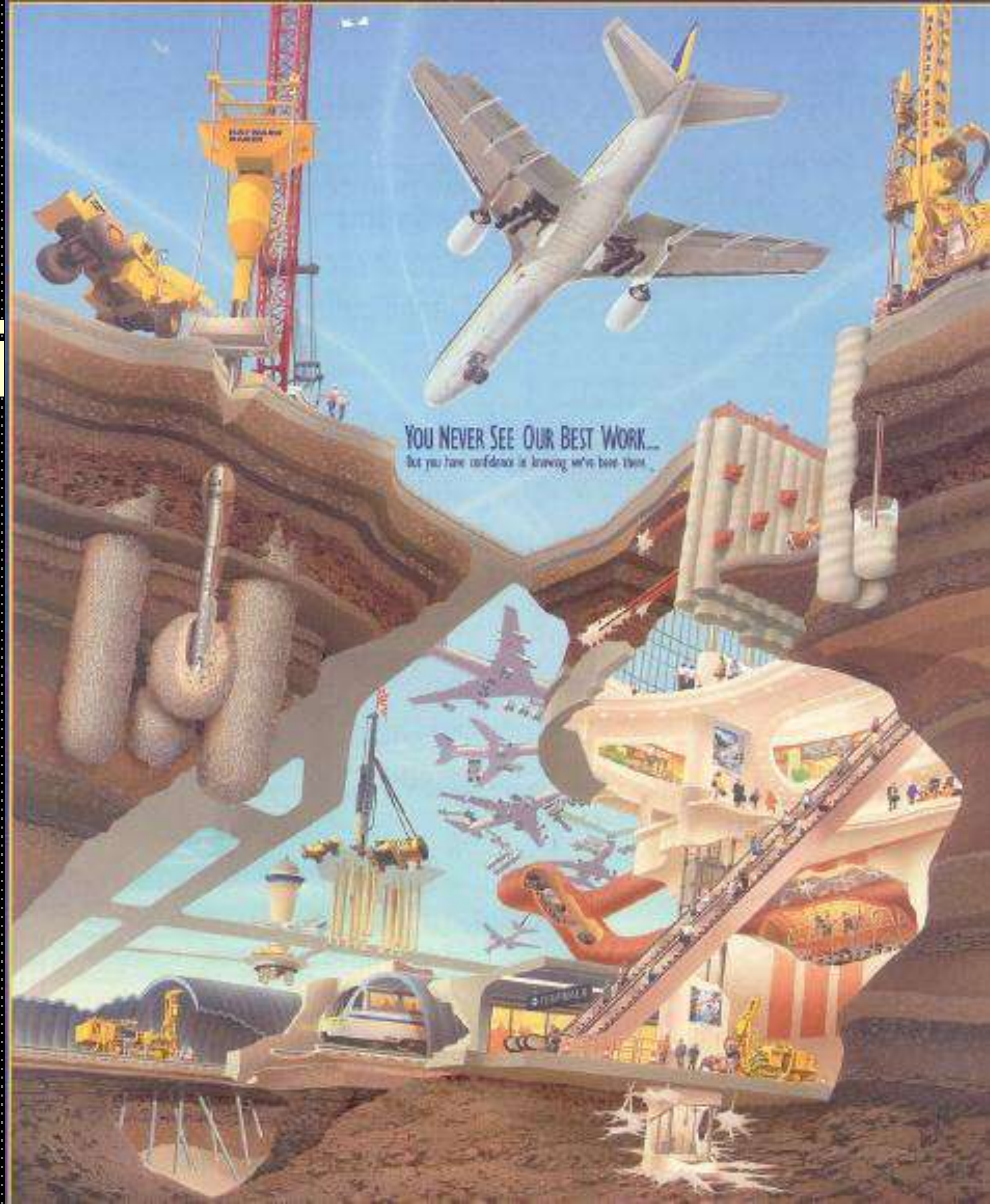
Text book

Text book

- Das, B.M., *Principles of Geotechnical Engineering*

- **<u>References</u>**
- Budhu" Soil Mechanics and foundation", John Wiley (for slope stability part)
- Terzaghi, Peck, and Mesri, " Soil Mechanics in Engineering Practice", John Wiley
- Holtz, R. D., and Kovacs W. D., "An Introduction to Geotechnical Engineering", Prentice-Hall

Civil Engineeri
challenges



II.
Physical Properties

Outline

1. Soil Texture
2. Grain Size and Grain Size Distribution
3. Particle Shape
4. Atterberg Limits
5. References

1.1 Origin of Clay Minerals

"The contact of rocks and water produces clays, either at or near the surface of the earth" (from Velde, 1995).

Rock +Water → Clay

For example,

The CO_2 gas can dissolve in water and form carbonic acid, which will become hydrogen ions H^+ and bicarbonate ions, and make water slightly acidic.

$$CO_2 + H_2O \rightarrow H_2CO_3 \rightarrow H^+ + HCO_3^-$$

The acidic water will react with the rock surfaces and tend to dissolve the K ion and silica from the feldspar. Finally, the feldspar is transformed into kaolinite.

Feldspar + hydrogen ions+water → clay (kaolinite) + cations, dissolved silica

$$2KAlSi_3O_8 + 2H^+ + H_2O \rightarrow Al_2Si_2O_5(OH)_4 + 2K^+ + 4SiO_2$$

•Note that the hydrogen ion displaces the cations.

1.1 Origin of Clay Minerals (Cont.)

- The alternation of feldspar into kaolinite is very common in the decomposed granite.

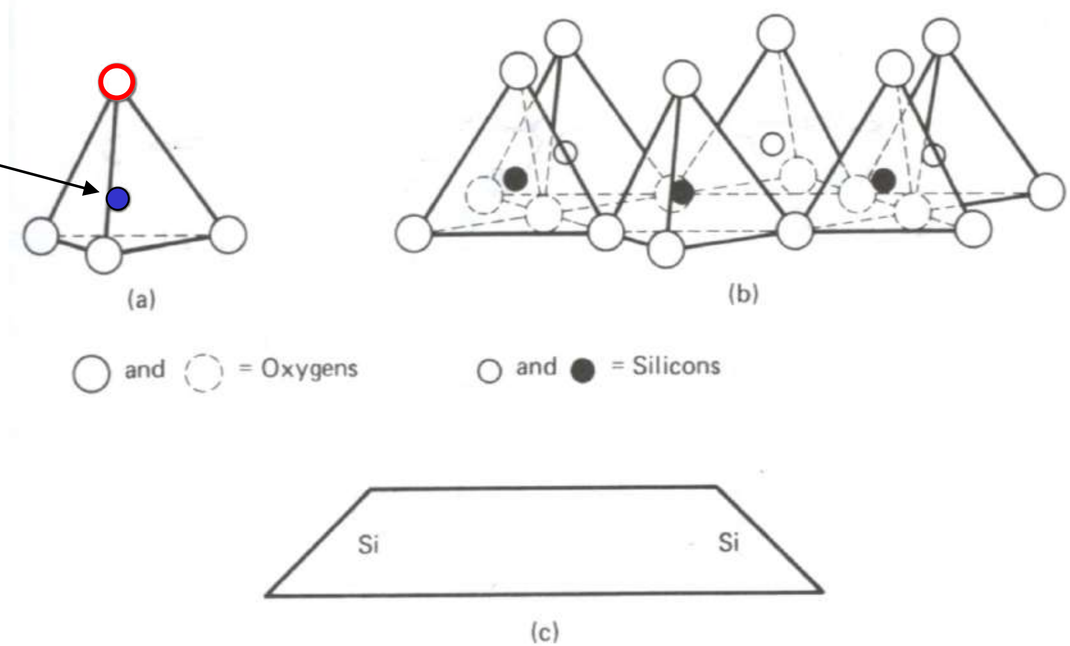
- The clay minerals are common in the filling materials of joints and faults (fault gouge, seam) in the rock mass.

  Weak plane!

1.2 Basic Unit-Silica Tetrahedra

1 Si

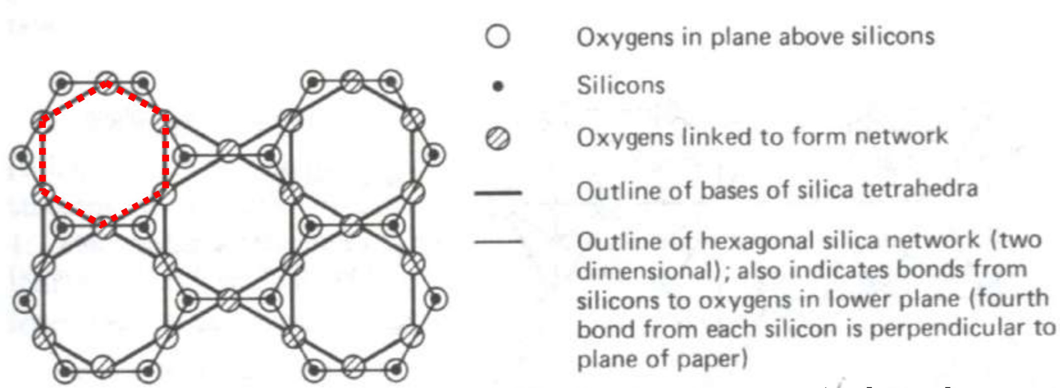
4 O



$(Si_2O_{10})^{-4}$

Replace four Oxygen with hydroxyls or combine with positive union

Hexagonal hole



Tetrahedron

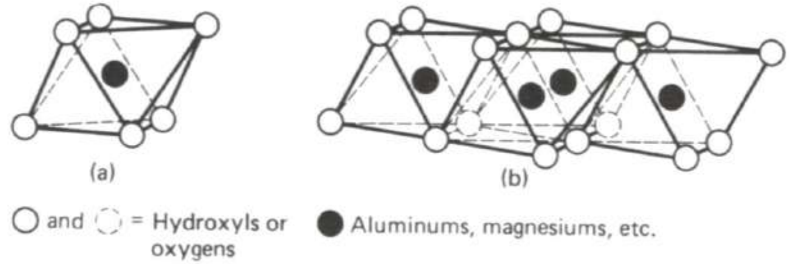
Plural: Tetrahedra

(Holtz and Kovacs, 1981)

1.2 Basic Unit-Octahedral Sheet

1 Cation

6 O or OH



Gibbsite sheet: Al^{3+}

$Al_2(OH)_6$, 2/3 cationic spaces are filled

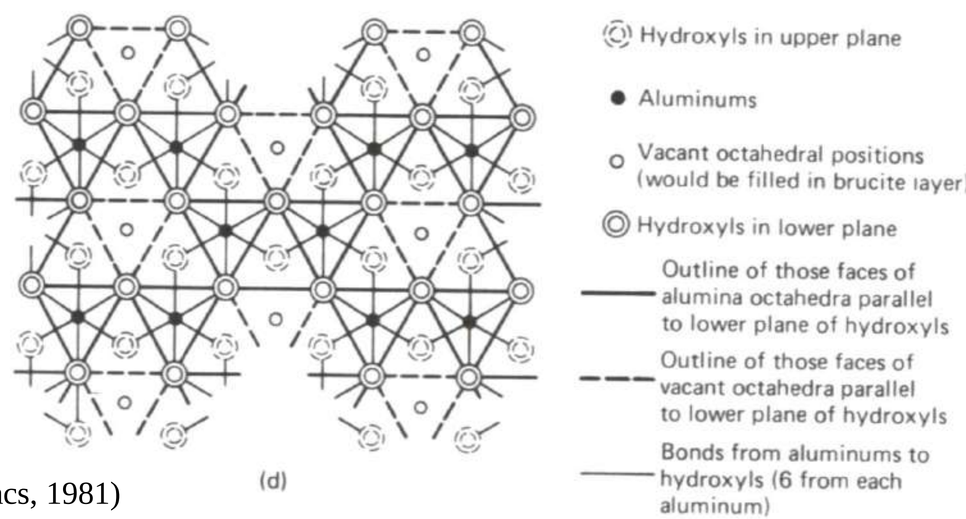
One OH is surrounded by 2 Al: Dioctahedral sheet

Different cations

Brucite sheet: Mg^{2+}

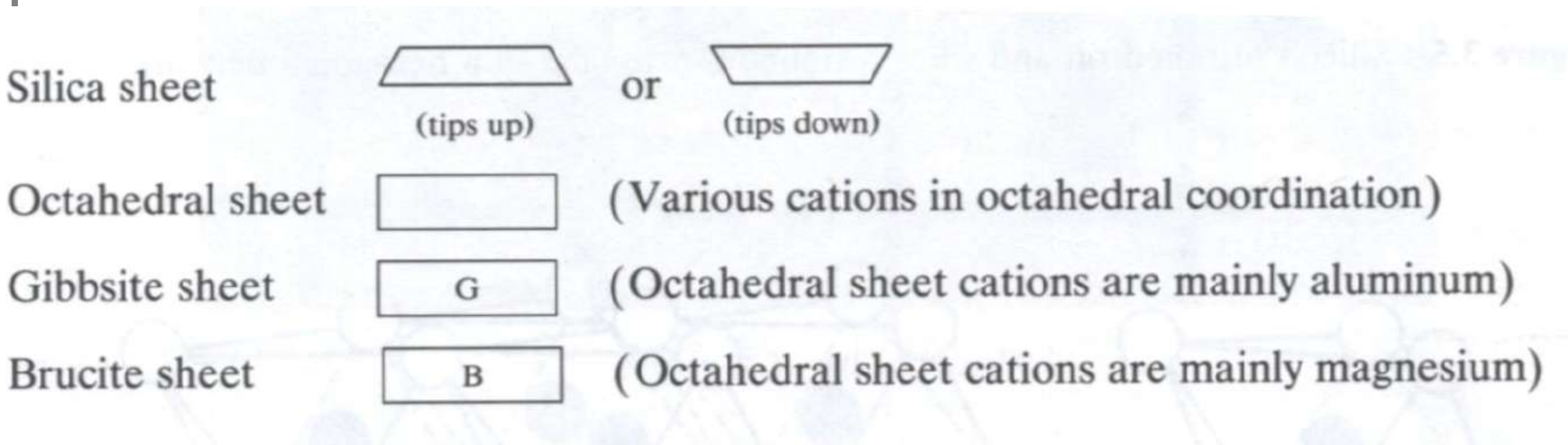
$Mg_3(OH)_6$, all cationic spaces are filled

One OH is surrounded by 3 Mg: Trioctahedral sheet



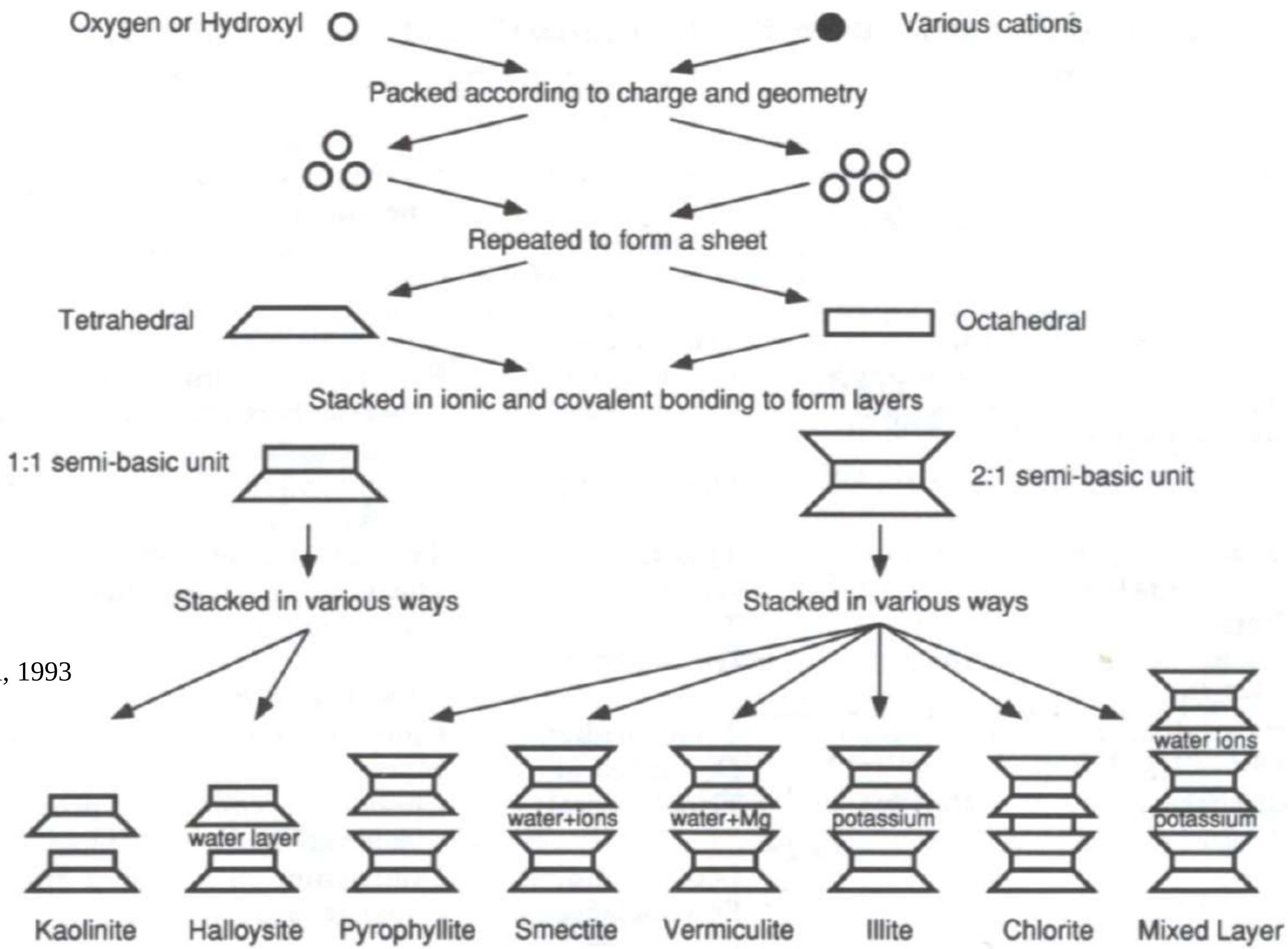
(Holtz and Kovacs, 1981)

1.2 Basic Unit-Summary



Mitchell, 1993

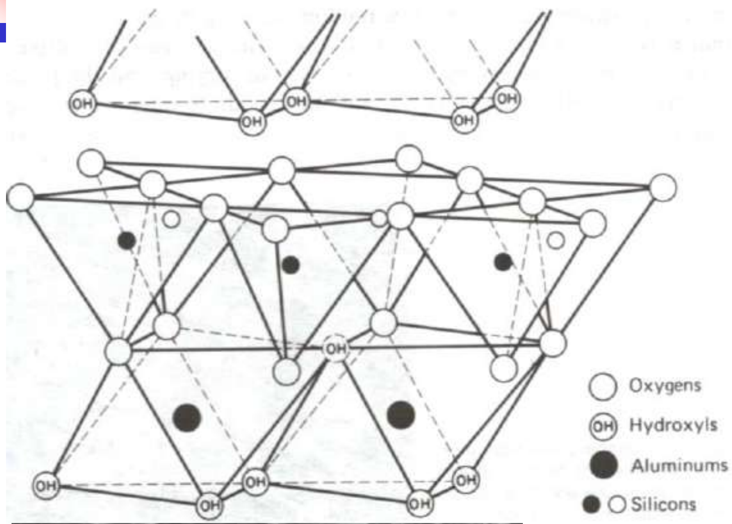
1.3 Synthesis

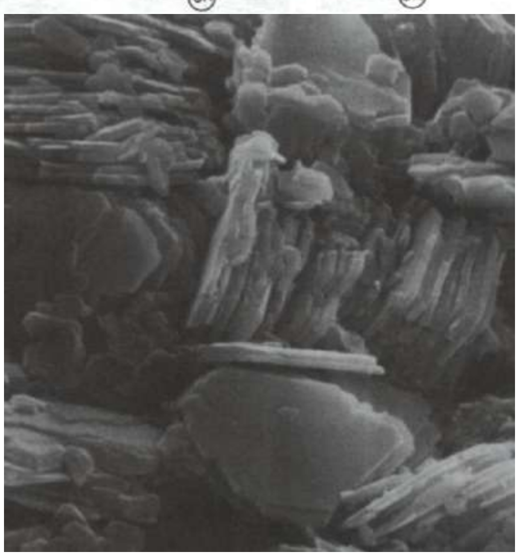


Mitchell, 1993

Noncrystalline clay - allophane

1.4 1:1 Minerals-Kaolinite

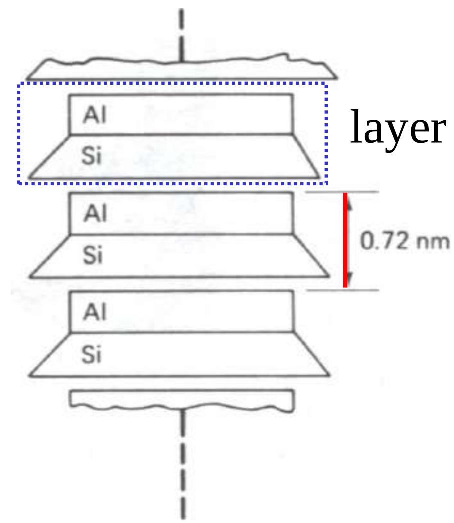




17 μm

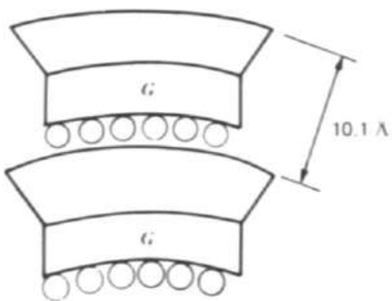
Trovey, 1971 (from Mitchell, 1993)

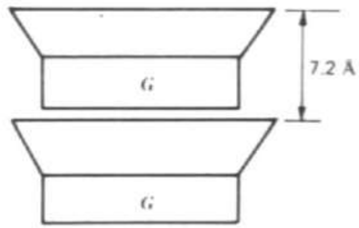
Basal spacing is 7.2 Å

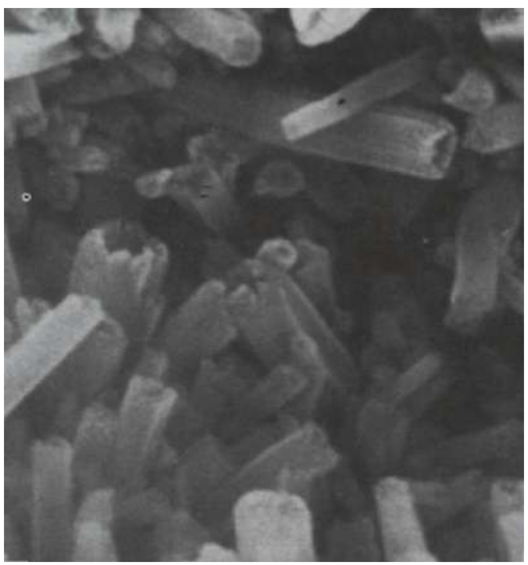


- $Si_4Al_4O_{10}(OH)_8$. Platy shape
- The bonding between layers are van der Waals forces and hydrogen bonds (strong bonding).
- There is no interlayer swelling
- Width: 0.1~ 4μm, Thickness: 0.05~2 μm

1.4 1:1 Minerals-Halloysite





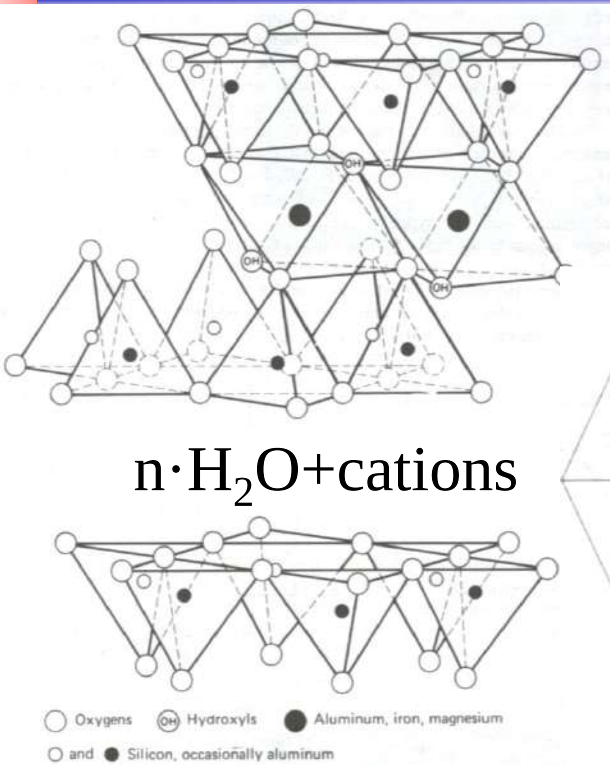


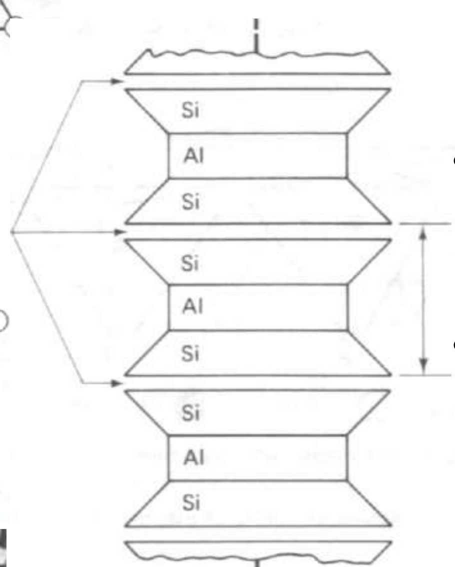
Trovey, 1971 (from Mitchell, 1993)

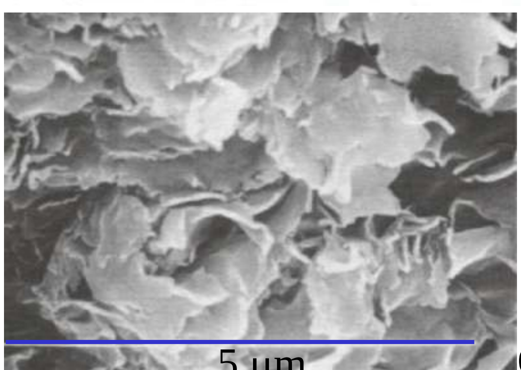
2 μm

- $Si_4Al_4O_{10}(OH)_8 \cdot 4H_2O$
- A single layer of water between unit layers.
- The basal spacing is 10.1 Å for hydrated halloysite and 7.2 Å for dehydrated halloysite.
- If the temperature is over 50 °C or the relative humidity is lower than 50%, the hydrated halloysite will lose its interlayer water (Irfan, 1966). Note that this process is **irreversible** and will affect the results of soil classifications (GSD and Atterberg limits) and compaction tests.
- There is no interlayer swelling.
- Tubular shape while it is hydrated.

1.5 2:1 Minerals-Montmorillonite



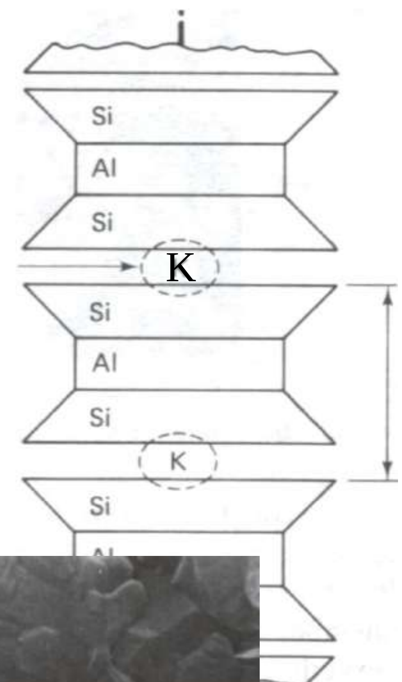


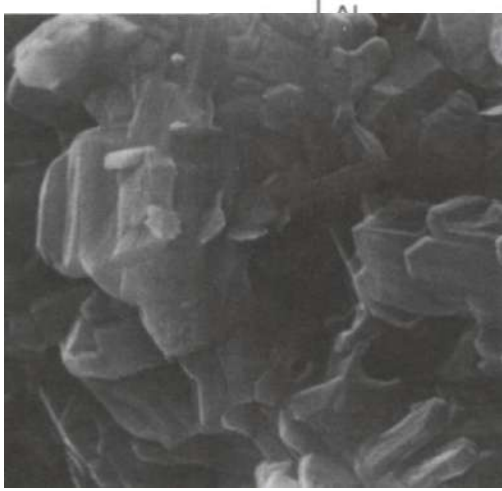


(Holtz and Kovacs, 1981)

- $Si_8Al_4O_{20}(OH)_4 \cdot nH_2O$ (Theoretical unsubstituted). Film-like shape.
- There is extensive isomorphous substitution for silicon and aluminum by other cations, which results in charge deficiencies of clay particles.
- $n \cdot H_2O$ and cations exist between unit layers, and the basal spacing is from 9.6 Å to ∞ (after swelling).
- The interlayer bonding is by van der Waals forces and by cations which balance charge deficiencies (weak bonding).
- There exists interlayer swelling, which is very important to engineering practice (**expansive clay**).
- Width: 1 or 2 μm, Thickness: 10 Å~1/100 width

1.5 2:1 Minerals-Illite (mica-like minerals)





7.5 μm

Trovey, 1971 (from Mitchell, 1993)

- $Si_8(Al,Mg, Fe)_{4\sim6}O_{20}(OH)_4 \cdot (K,H_2O)_2$. Flaky shape.
- The basic structure is very similar to the mica, so it is sometimes referred to as hydrous mica. Illite is the chief constituent in many shales.
- Some of the Si^{4+} in the tetrahedral sheet are replaced by the Al^{3+}, and some of the Al^{3+} in the octahedral sheet are substituted by the Mg^{2+} or Fe^{3+}. Those are the origins of charge deficiencies.
- The charge deficiency is balanced by the potassium ion between layers. Note that the potassium atom can exactly fit into the hexagonal hole in the tetrahedral sheet and form a strong interlayer bonding.
- The basal spacing is fixed at 10 Å in the presence of polar liquids (no interlayer swelling).
- Width: 0.1~ several μm, Thickness: ~ 30 Å

1. Soil Texture

1.1 Soil Texture

The texture of a soil is its appearance or “feel” and it depends on the relative sizes and shapes of the particles as well as the range or distribution of those sizes.

Coarse-grained soils: Fine-grained soils:

Gravel Sand Silt Clay

0.075 mm (USCS)

Sieve analysis Hydrometer analysis

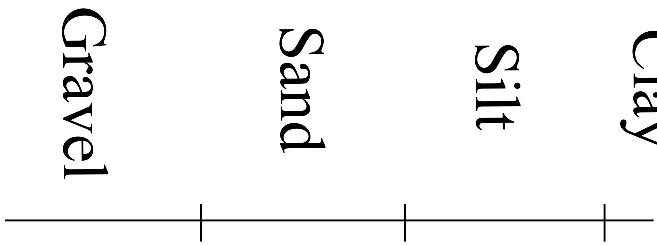
1.2 Characteristics

TABLE 2-2 Textural and Other Characteristics of Soils (Holtz and Kovacs, 1981)

Soil name:	Gravels, Sands	Silts	Clays
Grain size:	Coarse grained Can see individual grains by eye	Fine grained Cannot see individual grains	Fine grained Cannot see individual grains
Characteristics:	Nonplastic Granular	Nonplastic Granular	Plastic —
Effect of water on engineering behavior:	Relatively unimportant (exception: loose saturated granular materials and dynamic loadings)	Important	Very important
Effect of grain size distribution on engineering behavior:	Important	Relatively unimportant	Relatively unimportant

2. Grain Size and Grain Size Distribution

2.1 Grain Size

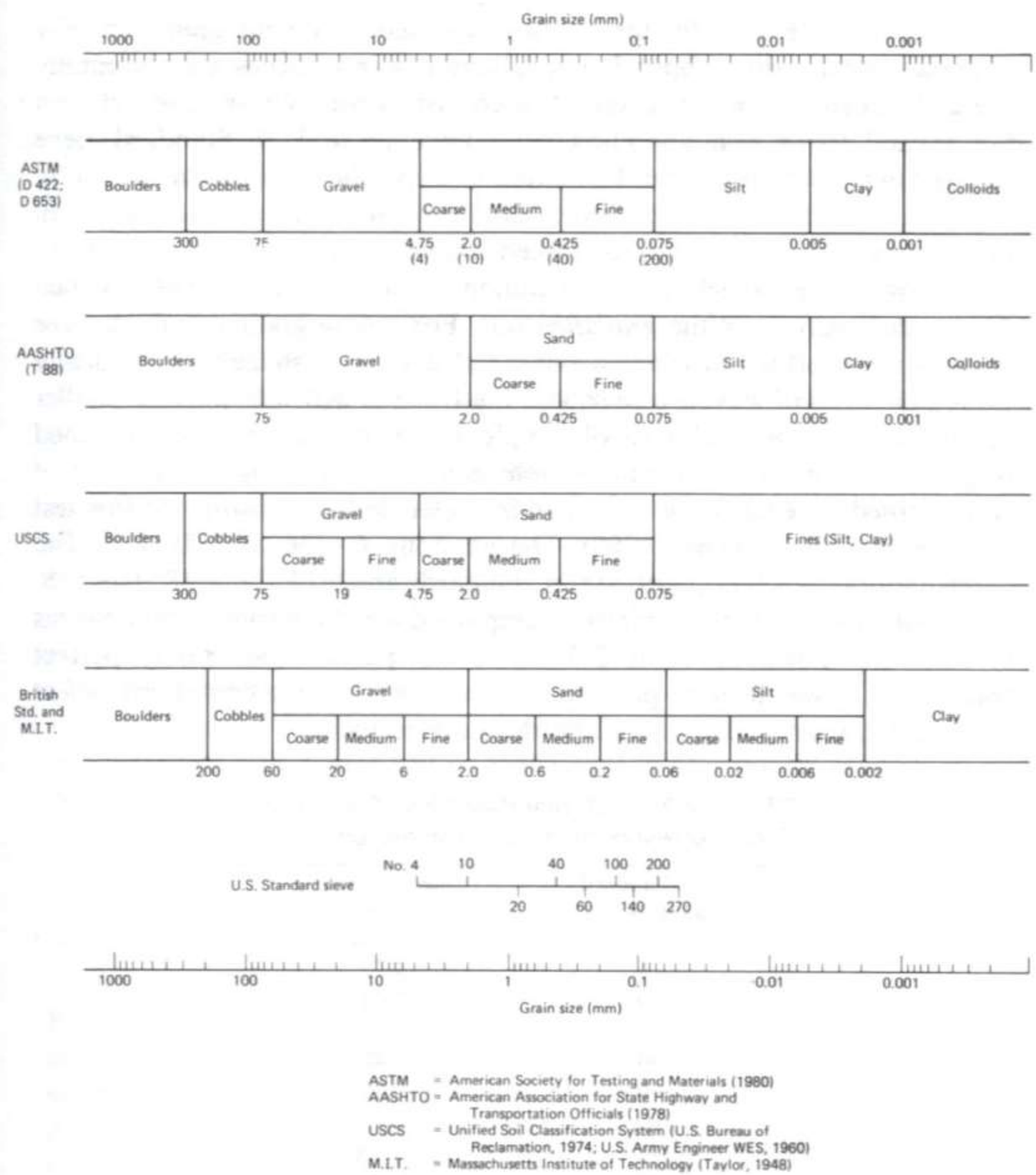


USCS	4.75	0.075	
BS	2.0	0.06	0.002

USCS: Unified Soil Classification

BS: British Standard

Unit: mm



(Holtz and Kovacs, 1981)

Fig. 2.3 Grain size ranges according to several engineering soil classification systems (modified after Al-Hussaini, 1977).

Note:

Clay-size particles

For example:

A small quartz particle may have the similar size of clay minerals.

Clay minerals

For example:

Kaolinite, Illite, etc.

2.2 Grain Size Distribution

- **Sieve size**

▼ **TABLE 1.5** U.S. Standard Sieve Sizes

Sieve no.	Opening (mm)
4	4.75
5	4.00
6	3.35
7	2.80
8	2.36
10	2.00
12	1.70
14	1.40
16	1.18
18	1.00
20	0.850
25	0.710
30	0.600
35	0.500
40	0.425
50	0.355
60	0.250
70	0.212
80	0.180
100	0.150
120	0.125
140	0.106
170	0.090
200	0.075
270	0.053

(Das, 1998)

Table 4.5(a). METRIC SIEVES (BS)

Construction	*Aperture size: Full Set (A)*	*'Standard' set (B)*	*'Short' set C)*
Perforated steel place (square hole)	75 mm	+	
	63	+	+
	50		
	37.5	+	
	28		
	20	+	+
	14		
	10	+	
	6.3	+	+
	5		
	3.35	+	
	2	+	+
	1.18	+	
	600 μm	+	+
	425		
	300	+	
	212		+
	150	+	
	63	+	+
Lid and receiver	+	+	+
	19 sieves	13 sieves	7 sieves

(Head, 1992)

2.2 Grain Size Distribution (Cont.)

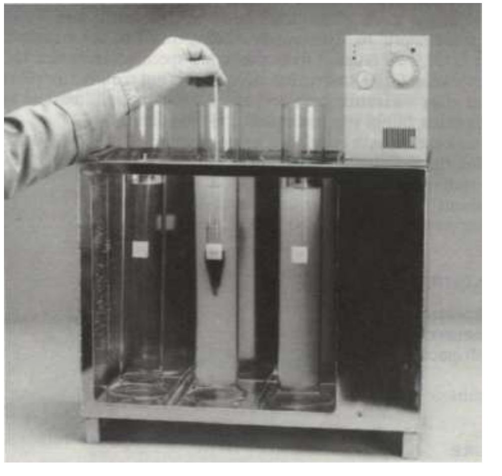
- **Experiment**

Coarse-grained soils: Fine-grained soils:

Gravel Sand Silt Clay

0.075 mm (USCS)

(Head, 1992)



Sieve analysis

Hydrometer analysis

2.2 Grain Size Distribution (Cont.)

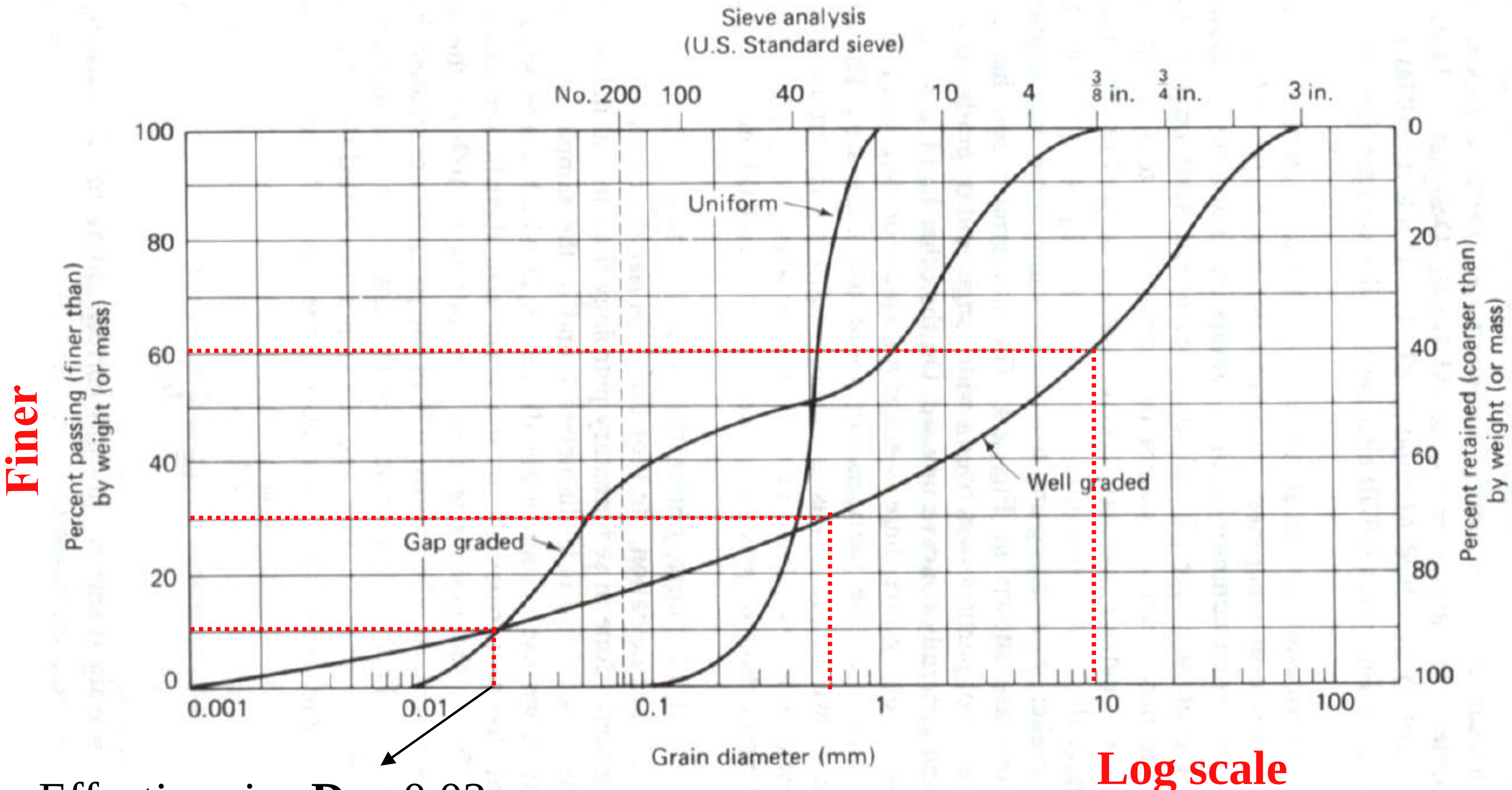


Fig. 2.4 Typical grain size distributions.

Effective size $\mathbf{D_{10}}$: 0.02 mm

$\mathbf{D_{30}}$: $\mathbf{D_{60}}$:

(Holtz and Kovacs, 1981)

2.2 Grain Size Distribution (Cont.)

- Describe the shape

Example: well graded

$$D_{10} = 0.02\,\text{mm (effective size)}$$

$$D_{30} = 0.6\,\text{mm}$$

$$D_{60} = 9\,\text{mm}$$

Coefficient of uniformity

$$C_u = \frac{D_{60}}{D_{10}} = \frac{9}{0.02} = 450$$

Coefficient of curvature

$$C_c = \frac{(D_{30})^2}{(D_{10})(D_{60})} = \frac{(0.6)^2}{(0.02)(9)} = 2$$

- Criteria

Well – graded soil

$$1 < C_c < 3 \text{ and } C_u \geq 4$$

(for gravels)

$$1 < C_c < 3 \text{ and } C_u \geq 6$$

(for sands)

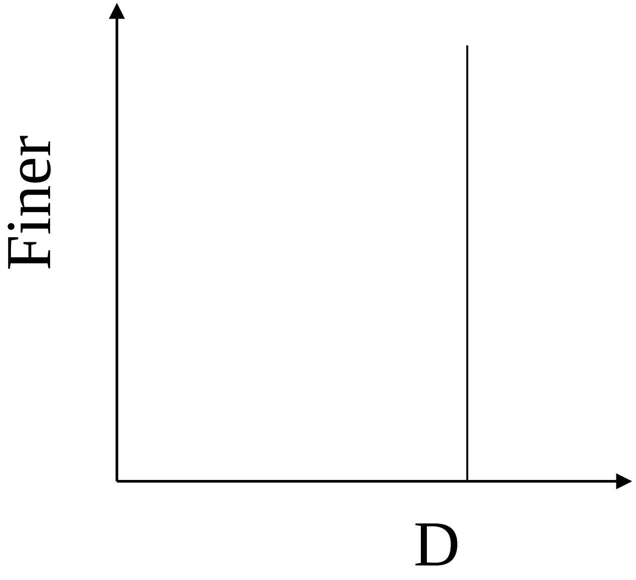
- Question

What is the C_u for a soil with only one grain size?

Answer

- Question

What is the C_u for a soil with only one grain size?



Grain size distribution

Coefficient of uniformity

$$C_u = \frac{D_{60}}{D_{10}} = 1$$

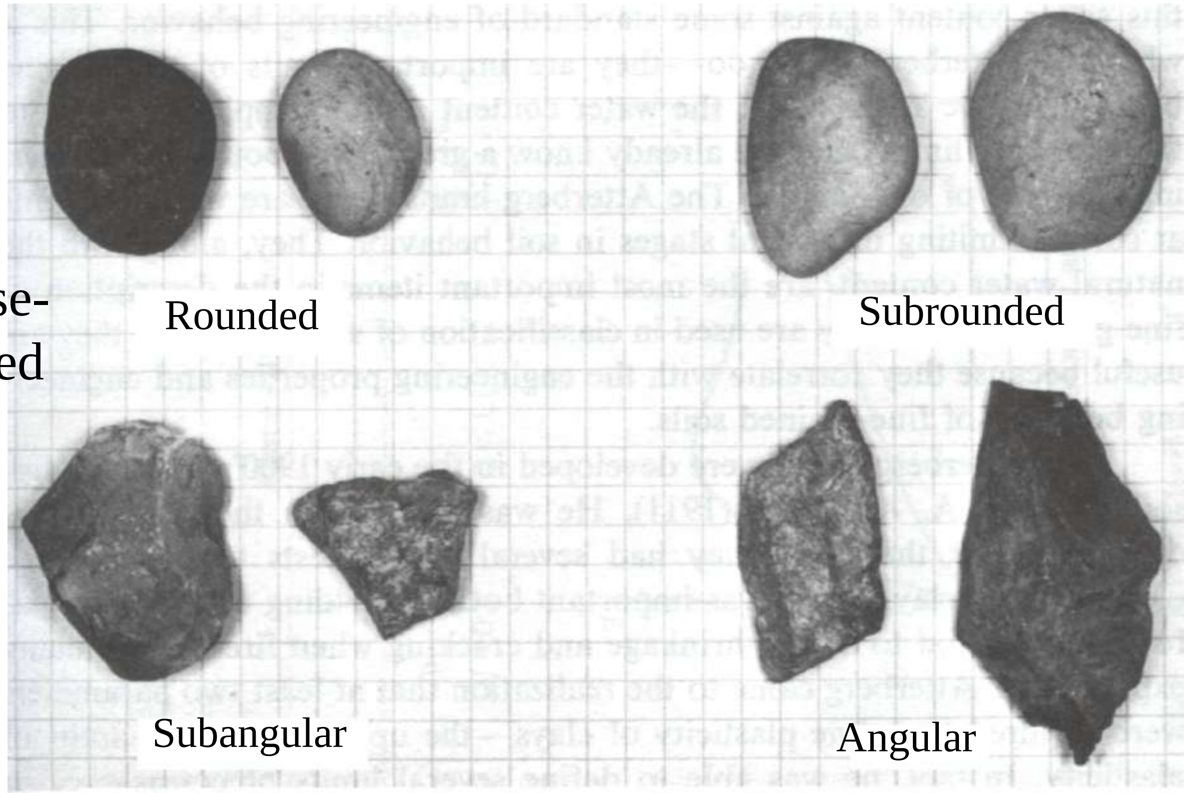
2.2 Grain Size Distribution (Cont.)

- **Engineering applications**
  - It will help us "feel" the soil texture (what the soil is) and it will also be used for the soil classification (next topic).
  - It can be used to define the grading specification of a drainage filter (clogging).
  - It can be a criterion for selecting fill materials of embankments and earth dams, road sub-base materials, and concrete aggregates.
  - It can be used to estimate the results of grouting and chemical injection, and dynamic compaction.
  - Effective Size, D_{10}, can be correlated with the hydraulic conductivity (describing the permeability of soils). (Hazen's Equation).(Note: controlled by small particles)

The grain size distribution is more important to ***coarse-grained*** *soils.*

3. Particle Shape

Coarse-grained soils



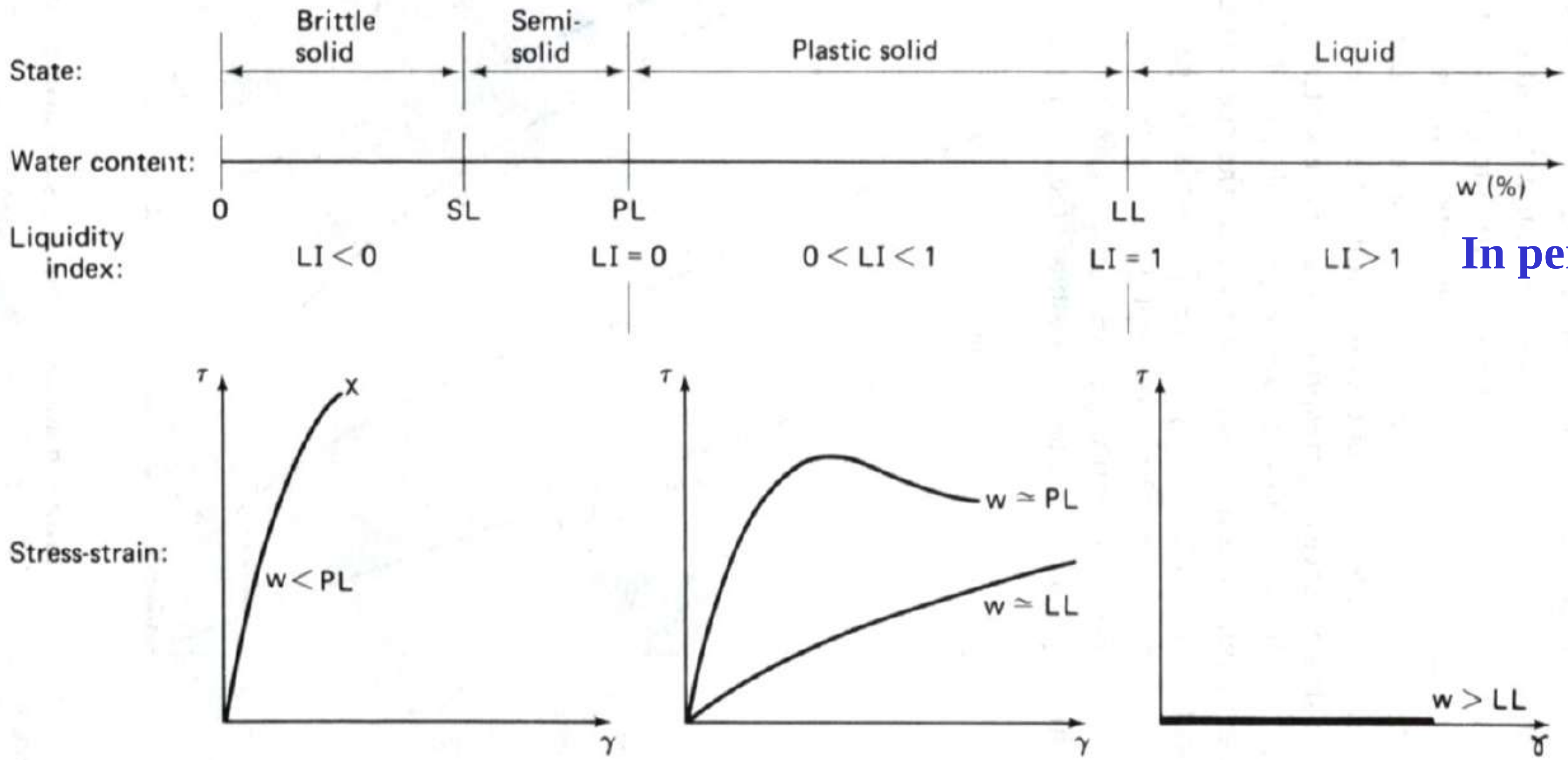
(Holtz and Kovacs, 1981)

- Important for granular soils
- Angular soil particle → higher friction
- Round soil particle → lower friction
- Note that clay particles are sheet-like.

4. Atterberg Limits
and
Consistency Indices

4.1 Atterberg Limits

- The presence of water in fine-grained soils can significantly affect associated engineering behavior, so we need a reference index to clarify the effects. (The reason will be discussed later in the topic of clay minerals)

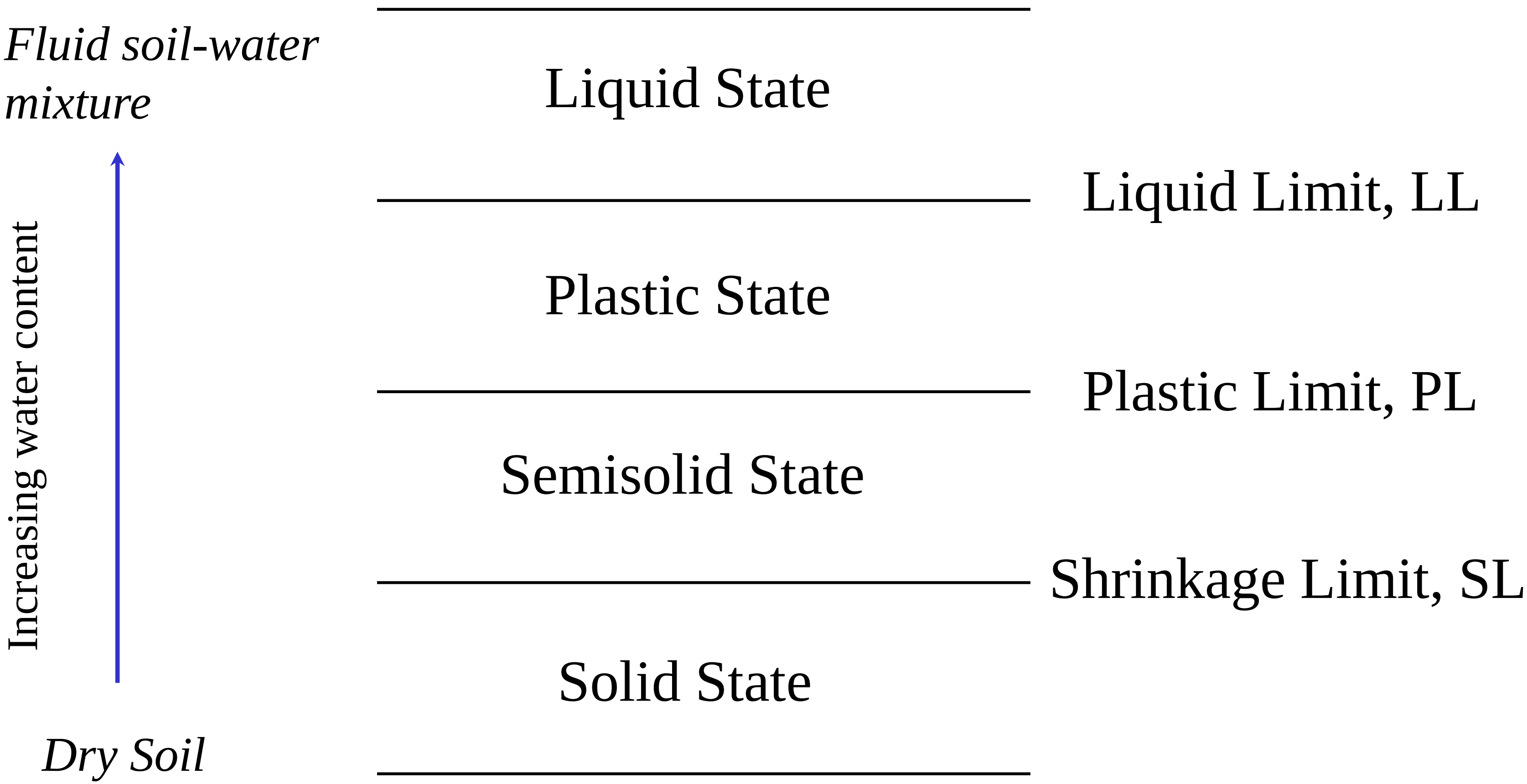


In percentage

Fig. 2.6 Water content continuum showing the various states of a soil as well as the generalized stress-strain response.

(Holtz and Kovacs, 1981)

4.1 Atterberg Limits (Cont.)



4.2 Liquid Limit-LL

Casagrande Method

(ASTM D4318-95a)

- Professor Casagrande standardized the test and developed the liquid limit device.
- Multipoint test
- One-point test

Cone Penetrometer Method

(BS 1377: Part 2: 1990:4.3)

- This method is developed by the Transport and Road Research Laboratory, UK.
- Multipoint test
- One-point test

4.2 Liquid Limit-LL (Cont.)

Dynamic shear test

- Shear strength is about 1.7 ~2.0 kPa.
- Pore water suction is about 6.0 kPa.

(review by Head, 1992; Mitchell, 1993).

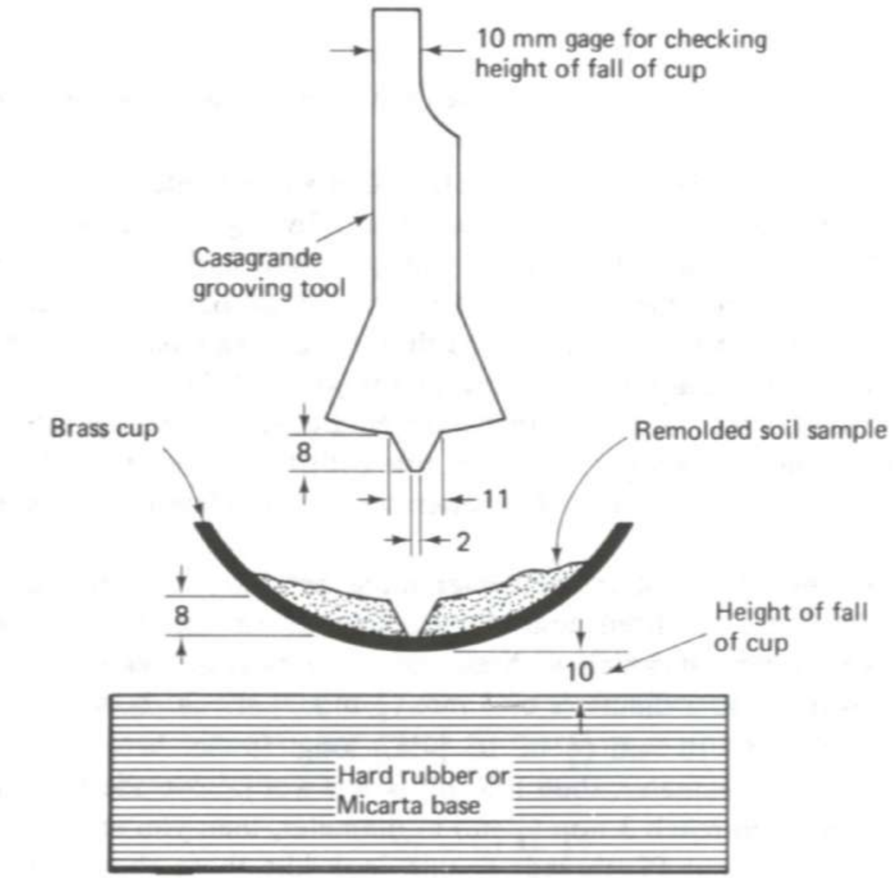
Particle sizes and water

- Passing No.40 Sieve (0.425 mm).
- Using deionized water.

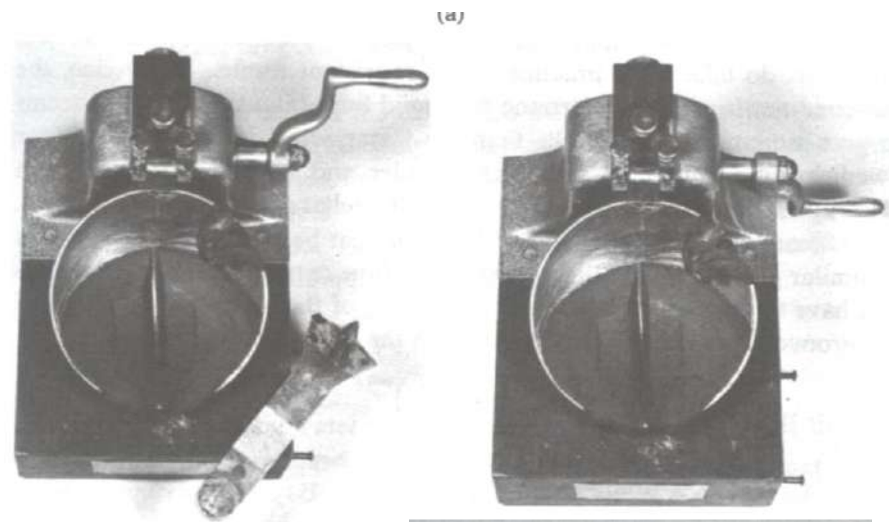
The type and amount of cations can significantly affect the measured results.

4.2.1 Casagrande Method

- **Device**

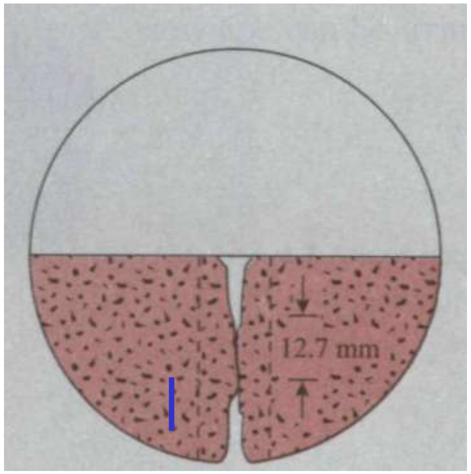


(Holtz and Kovacs, 1981)



N=25 blows

Closing distance = 12.7mm (0.5 in)



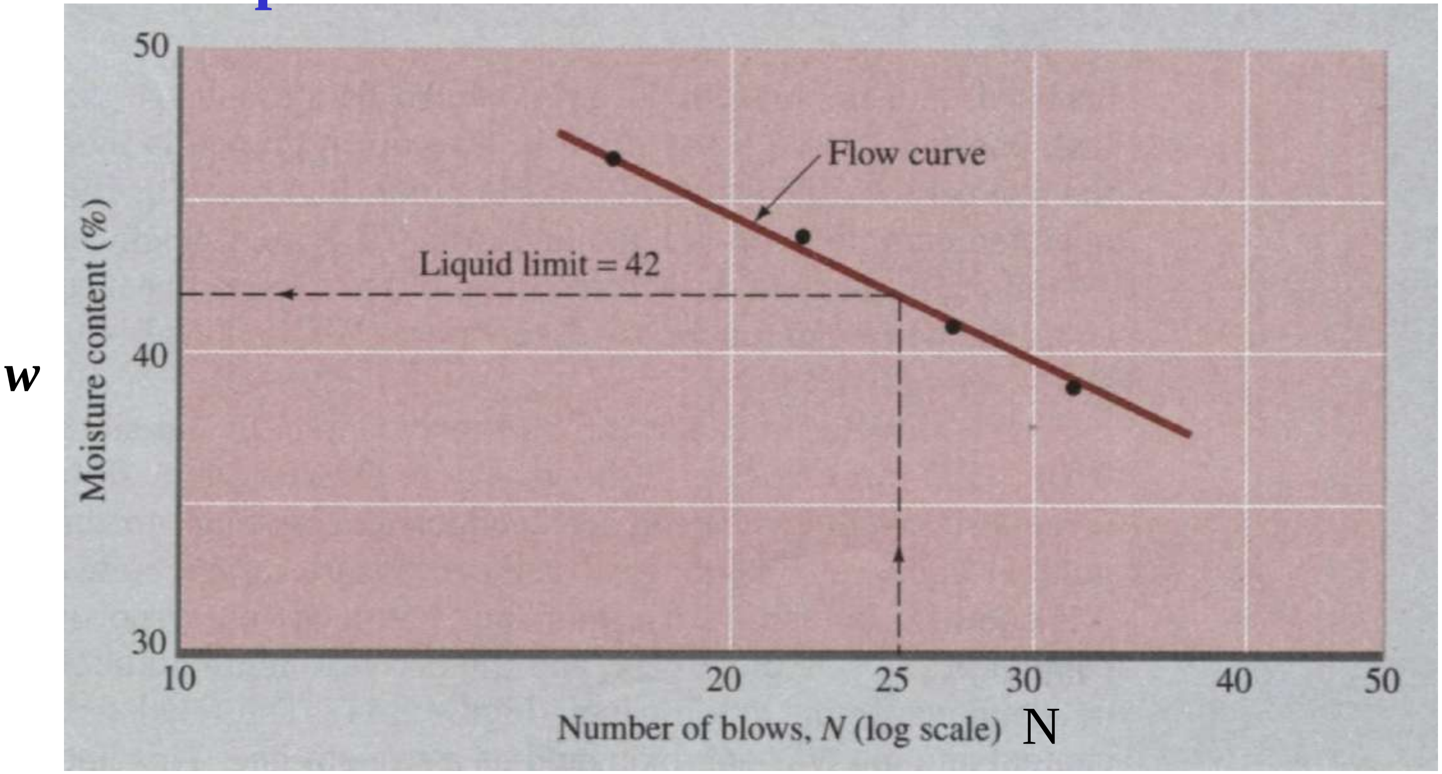
The water content, in percentage, required to close a distance of 0.5 in (12.7mm) along the bottom of the groove after 25 blows is defined as the liquid limit

4.2.1 Casagrande Method (Cont.)

Reference: Budhu: Soil Mechanics and Foundation

4.2.1 Casagrande Method (Cont.)

•Multipoint Method



Das, 1998

$$Flow\ index, I_F = \frac{w_1 - w_2}{\log(N_2 / N_1)} (choose\ a\ positive\ value)$$

$$w = -I_F \log N + cont.$$

4.2.1 Casagrande Method (Cont.)

•One-point Method

- Assume a constant slope of the flow curve.
- The slope is a statistical result of 767 liquid limit tests.

$$LL = w_n \left(\frac{N}{25} \right)^{\tan\beta}$$

$$N = number\ of\ blows$$

$$w_n = corresponding\ moisture\ content$$

$$\tan\beta = 0.121$$

Limitations:

- The β is an empirical coefficient, so it is not always 0.121.
- Good results can be obtained only for the blow number around 20 to 30.

4.2.2 Cone Penetrometer Method

- **Device**

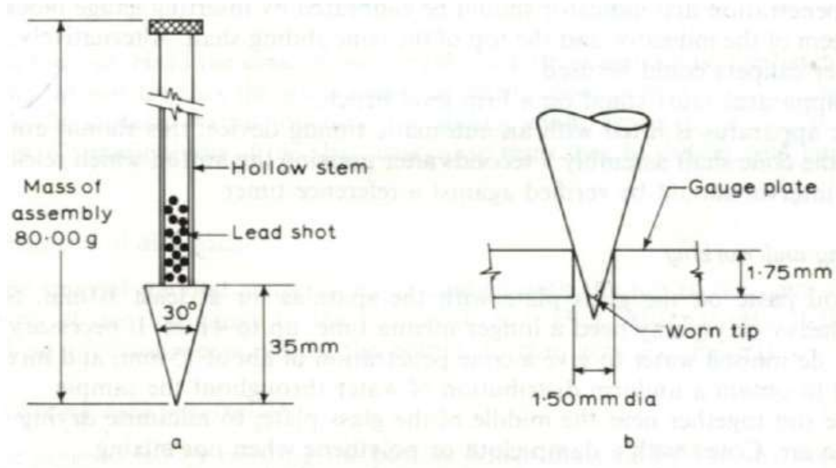


Fig. 2.12 Details of cone for penetrometer

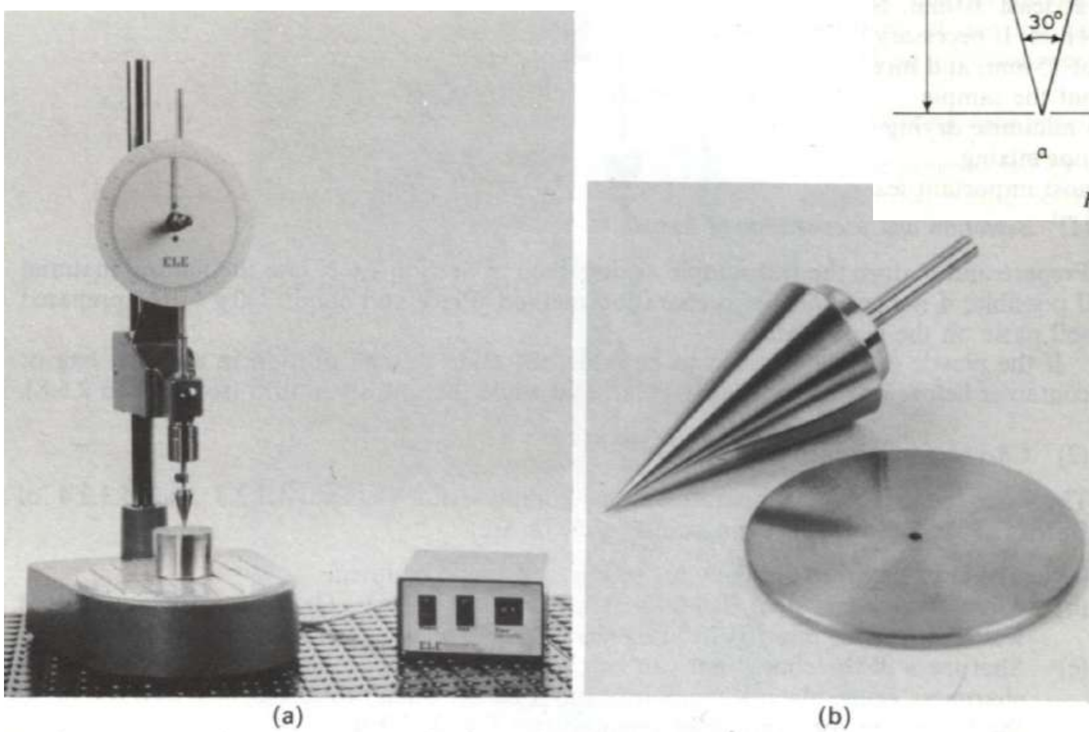


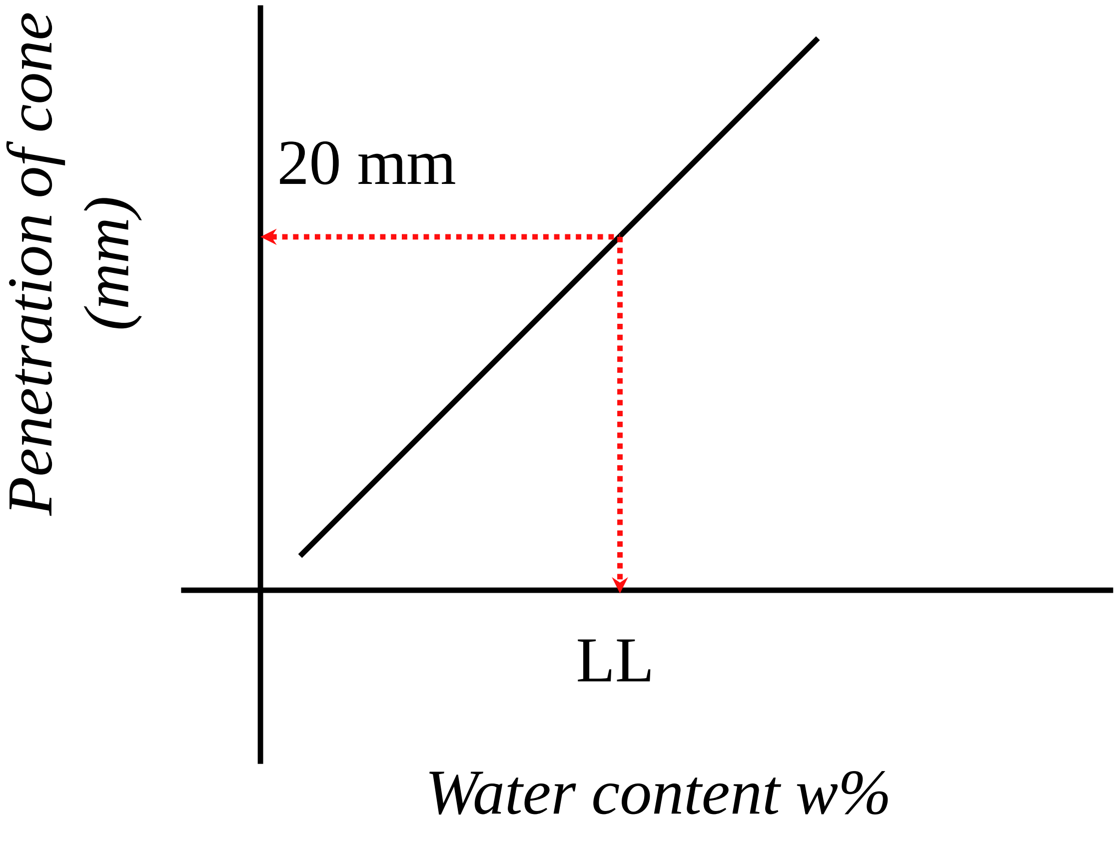
Fig. 2.11 Apparatus for cone penetrometer liquid test: (a) Cone penetrometer with automatic timing device, (b) cone and gauge plate

This method is developed by the Transport and Road Research Laboratory.

(Head, 1992)

4.2.2 Cone Penetrometer Method (Cont.)

- **Multipoint Method**



4.2.2 Cone Penetrometer Method (Cont.)

•One-point Method (an empirical relation)

Table 2.5. SUGGESTED FACTORS FOR CONE PENTRATION ONE-POINT LIQUID LIMIT TEST (from Clayton and Jukes, 1978)

Penetration (mm)	*Soil of high plasticity*	*Soil of intermediate plasticity*	*Soil of low plasticity*
15	1.098	1.094	1.057
16	1.075	1.076	1.052
17	1.055	1.058	1.042
18	1.036	1.039	1.030
19	1.018	1.020	1.015
20	1.001	1.001	1.000
21	0.984	0.984	0.984
22	0.967	0.968	0.971
23	0.949	0.954	0.961
24	0.929	0.943	0.955
25	0.909	0.934	0.954
Measured moisture content range	above 50%	35% to 50%	below 35%

(Review by Head, 1992)

Example: $\text{Penetration depth} = 15\,\text{mm},\ w = 40\%,$

$$\text{Factor} = 1.094,\ LL = 40 \cdot 1.094 \approx 44$$

4.2.3 Comparison

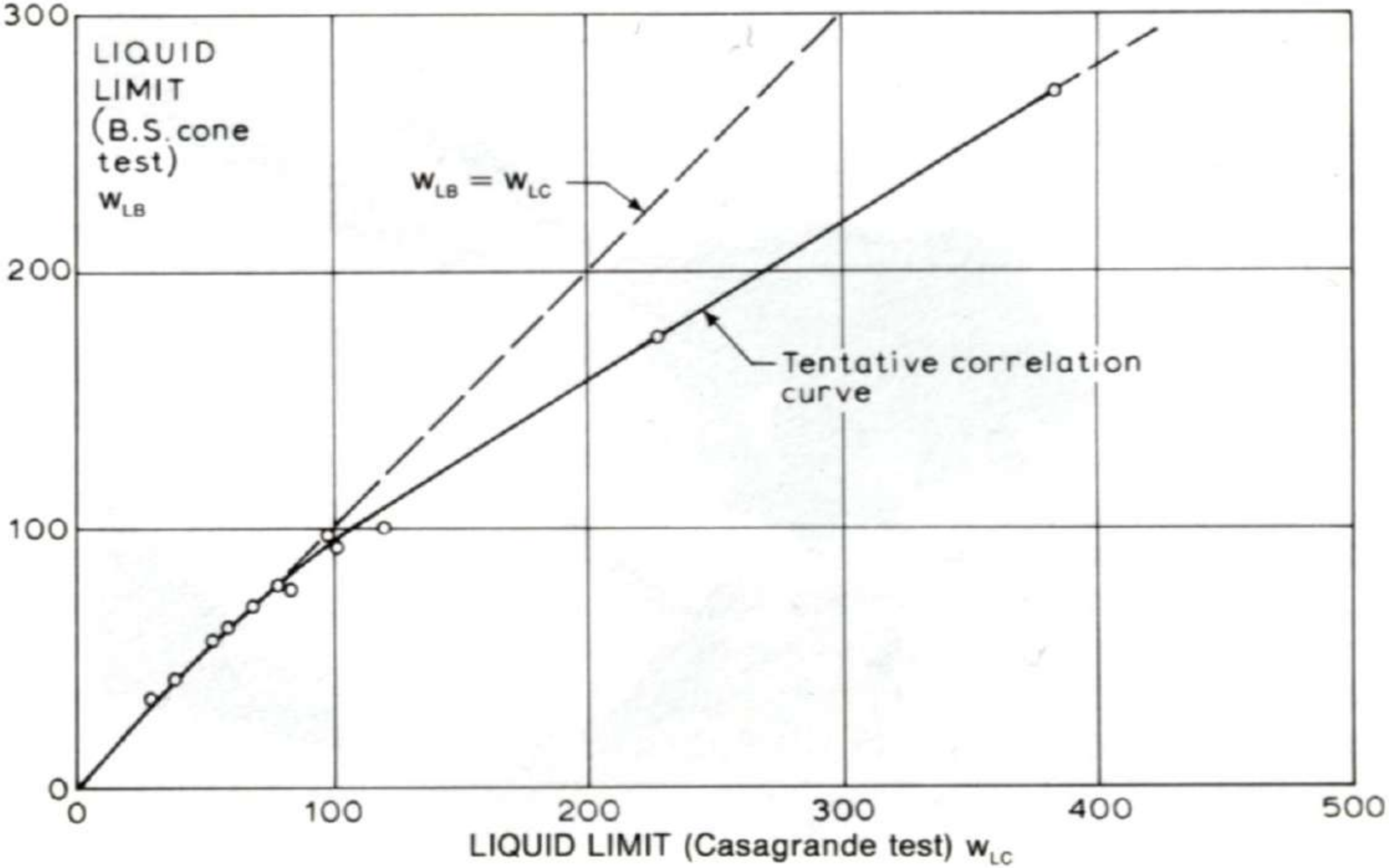


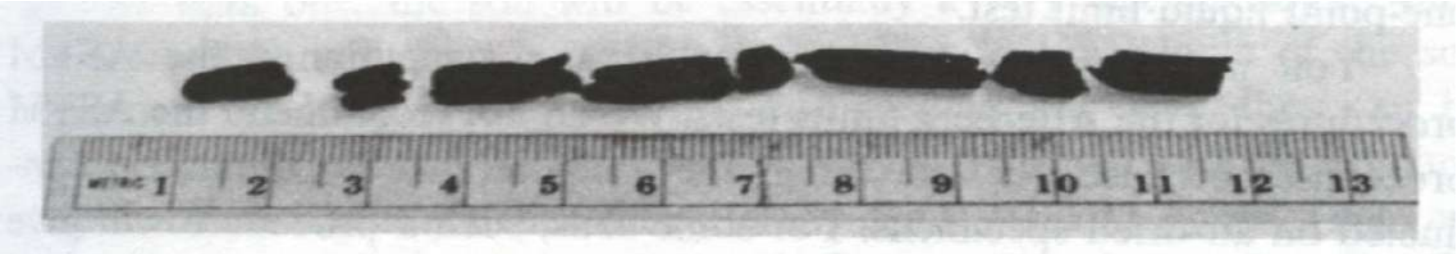
Fig. 2.9 Correlation of liquid limit results from two test methods

A good correlation between the two methods can be observed as the LL is less than 100.

Littleton and Farmilo, 1977 (from Head, 1992)

Question:
Which method will render more consistent results?

4.3 Plastic Limit-PL



(Holtz and Kovacs, 1981)

The plastic limit PL is defined as the water content at which a soil thread with ***3.2 mm diameter just*** crumbles.

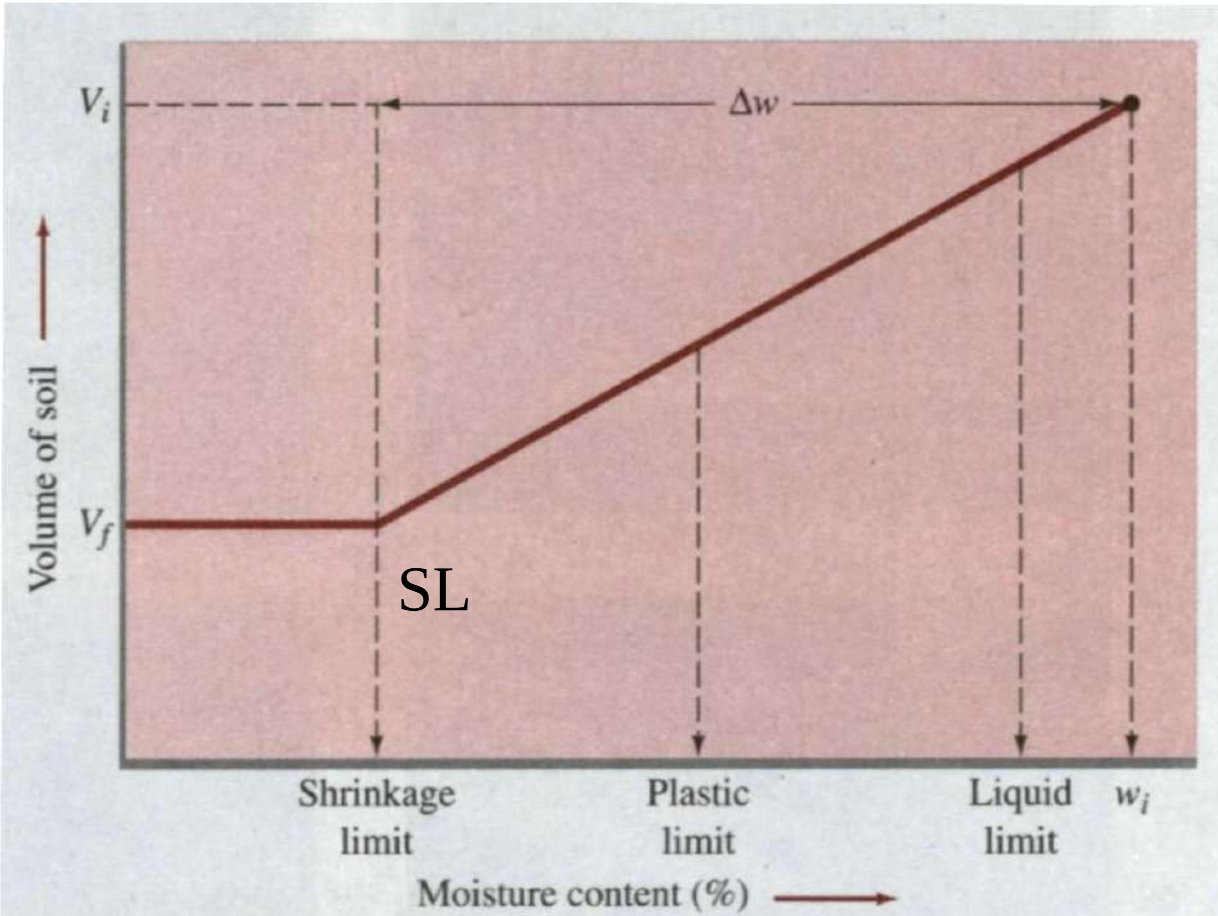
ASTM D4318-95a, BS1377: Part 2:1990:5.3

4.2.1 Casagrande Method (Cont.)

Liquid limit Test

Reference: Budhu: Soil Mechanics and Foundation

4.4 Shrinkage Limit-SL

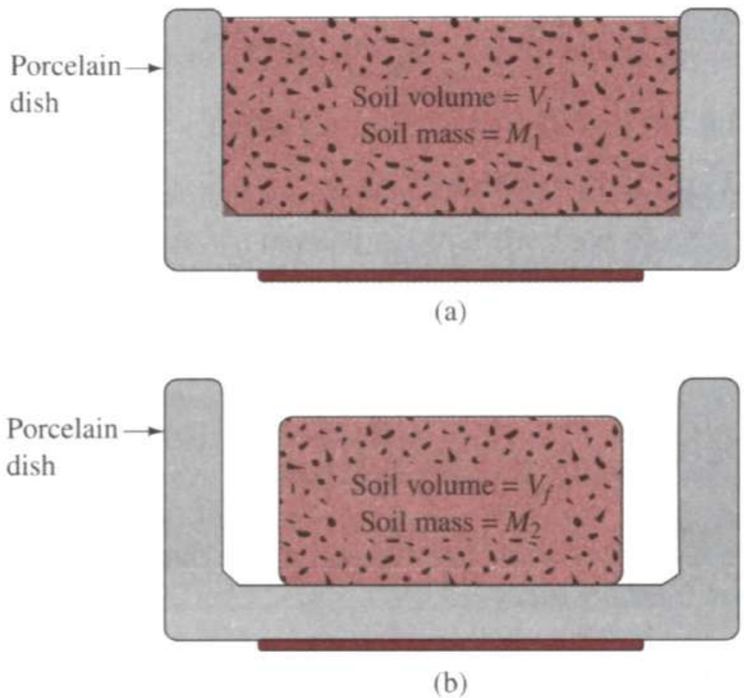


Definition of shrinkage limit:

The water content at which the soil volume ceases to change is defined as the shrinkage limit.

(Das, 1998)

4.4 Shrinkage Limit-SL (Cont.)



Soil volume: V_i

Soil mass: M_1

Soil volume: V_f

Soil mass: M_2

(Das, 1998)

$$SL = w_i(\%) - \Delta w(\%)$$

$$= \left(\frac{M_1 - M_2}{M_2} \right)(100) - \left(\frac{V_i - V_f}{M_2} \right)(\rho_w)(100)$$

4.4 Shrinkage Limit-SL (Cont.)

- “Although the shrinkage limit was a popular classification test during the 1920s, it is subject to considerable uncertainty and thus is no longer commonly conducted.”

- “One of the biggest problems with the shrinkage limit test is that the amount of shrinkage depends not only on the grain size but also on the initial fabric of the soil. The standard procedure is to start with the water content near the liquid limit. However, especially with sandy and silty clays, this often results in a shrinkage limit greater than the plastic limit, which is meaningless. Casagrande suggests that the initial water content be slightly greater than the PL, if possible, but admittedly it is difficult to avoid entrapping air bubbles.” (from Holtz and Kovacs, 1981)

4.5 Typical Values of Atterberg Limits

Table 10.1 Atterberg Limit Values for the Clay Minerals.

Mineral[a]	Liquid Limit (%)	Plastic Limit (%)	Shrinkage Limit
Montmorillonite	100–900	50–100	8.5–15
Nontronite	37–72	19–27	
Illite	60–120	35–60	15–17
Kaolinite	30–110	25–40	25–29
Hydrated Halloysite	50–70	47–60	
Dehydrated Halloysite	35–55	30–45	
Attapulgite	160–230	100–120	
Chlorite	44–47	36–40	
Allophane (undried)	200–250	130–140	

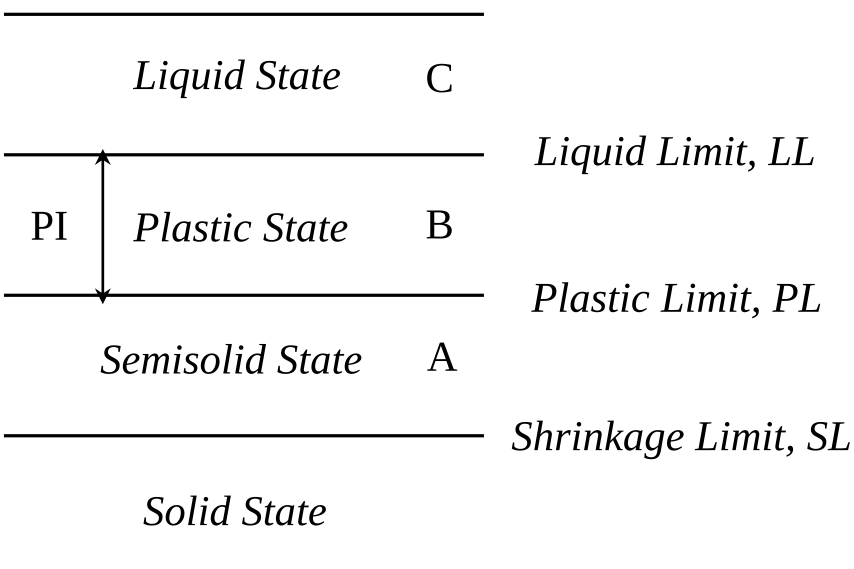
(Mitchell, 1993)

4.6 Indices

- **Plasticity index PI**

For describing the range of water content over which a soil was plastic

PI = LL – PL



- **Liquidity index LI**

For scaling the natural water content of a soil sample to the Limits.

$$LI = \frac{w - PL}{PI} = \frac{w - PL}{LL - PL}$$

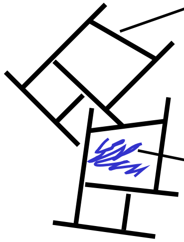
$$w \text{ is the water content}$$

LI <0 (A), brittle fracture if sheared

0<LI<1 (B), plastic solid if sheared

LI >1 (C), viscous liquid if sheared

4.6 Indices (Cont.)



$w > LL$

- **Sensitivity S_t** (for clays)

$$S_t = \frac{\text{Strength (undisturbed)}}{\text{Strength (disturbed)}}$$

Unconfined shear strength

TABLE 11-7 Typical Values of Sensitivity

Condition	Range of S_t U.S.	Range of S_t Sweden
Low sensitive	2–4	< 10
Medium sensitive	4–8	10–30
Highly sensitive	8–16	> 30
Quick	16	> 50
Extra quick	—	> 100
Greased lightning	—	

(Holtz and Kavocs, 1981)

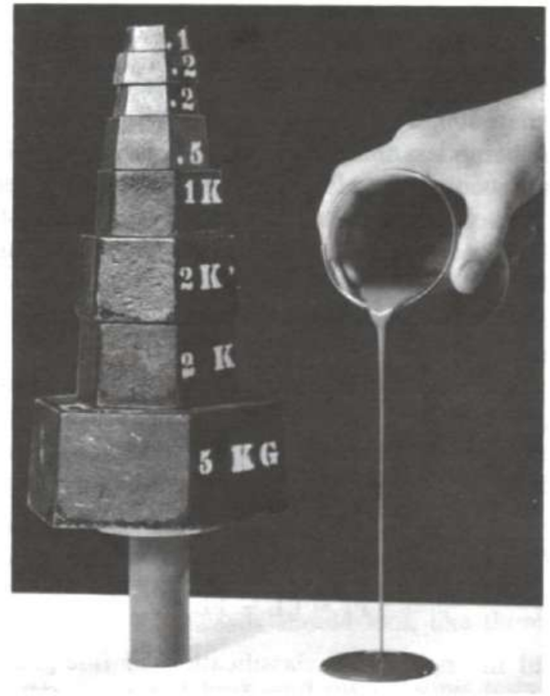


Fig. 2.9 (a) Undisturbed and (b) thoroughly remolded sample of Leda clay from Ottawa, Ontario. (Photograph courtesy of the Division of Building Research, National Research Council of Canada. Hand by D. C. MacMillan.)

4.6 Indices (Cont.)

- **Activity A**

(Skempton, 1953)

$$A = \frac{PI}{\%\,\text{clay fraction (weight)}}$$

$$\text{clay fraction} : < 0.002\,\text{mm}$$

Normal clays: 0.75<A<1.25

Inactive clays: A<0.75

Active clays: A> 1.25

High activity:

- large volume change when wetted
- Large shrinkage when dried
- Very reactive (chemically)

Mitchell, 1993

Table 10.4 Activities of Various Clay Minerals.

Mineral	Activity[a]
Smectites	1–7
Illite	0.5–1
Kaolinite	0.5
Halloysite ($2H_2O$)	0.5
Halloysite ($4H_2O$)	0.1
Attapulgite	0.5–1.2
Allophane	0.5–1.2

- **Purpose**

Both the *type* and *amount* of clay in soils will affect the Atterberg limits. This index is aimed to separate them.

4.7 Engineering Applications

- Soil classification (the next topic)

– The Atterberg limit enable clay soils to be classified.

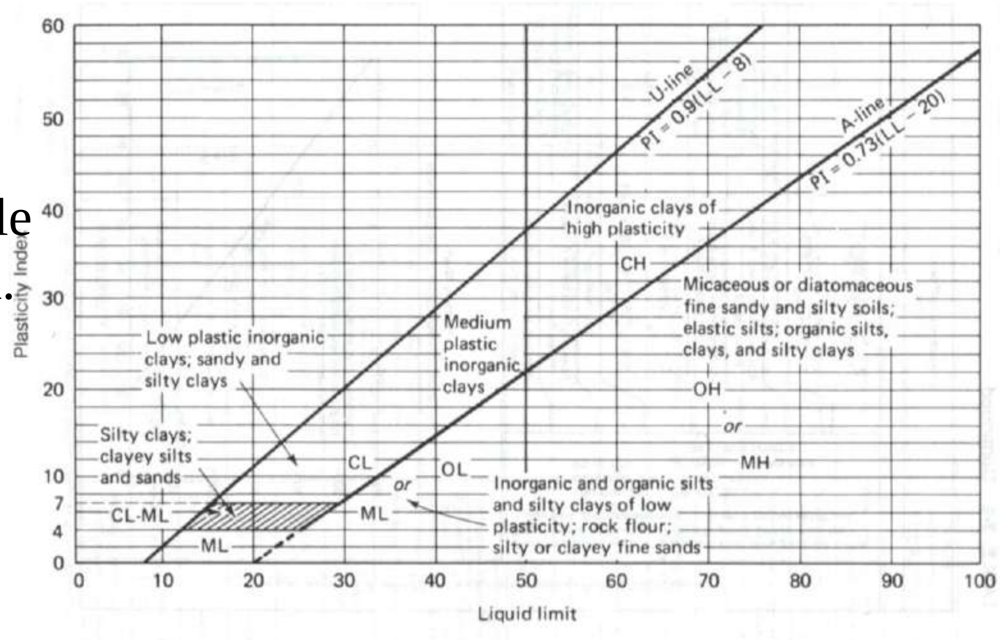


Fig. 3.2 Casagrande's plasticity chart, showing several representative soil types (developed from Casagrande, 1948, and Howard, 1977).

- The Atterberg limits are usually correlated with some engineering properties such as the permeability, compressibility, shear strength, and others.
  - In general, clays with high plasticity have lower permeability, and they are difficult to be compacted.
  - The values of SL can be used as a criterion to assess and prevent the excessive cracking of clay liners in the reservoir embankment or canal.

6. References

Main References:

Das, B.M. *Principles of Geotechnical Engineering*, 4th edition, PWS Publishing Company. (Chapter 2)

Budhu M. (2007)"Soil Mechanics and Foundations" Wiley, New York Holtz, R.D. and

Kovacs, W.D. (1981). *An Introduction to Geotechnical Engineering*, Prentice Hall. (Chapter 1 and 2)

Others:

Head, K. H. (1992). *Manual of Soil Laboratory Testing, Volume 1: Soil Classification and Compaction Test*, 2nd edition, John Wiley and Sons.

Lambe, T.W. (1991). Soil Testing for Engineers, BiTech Publishers Ltd.

Mitchell, J.K. (1993). *Fundamentals of Soil Behavior*, 2nd edition, John Wiley & Sons.

Hashemite University
Department of Civil Engineering

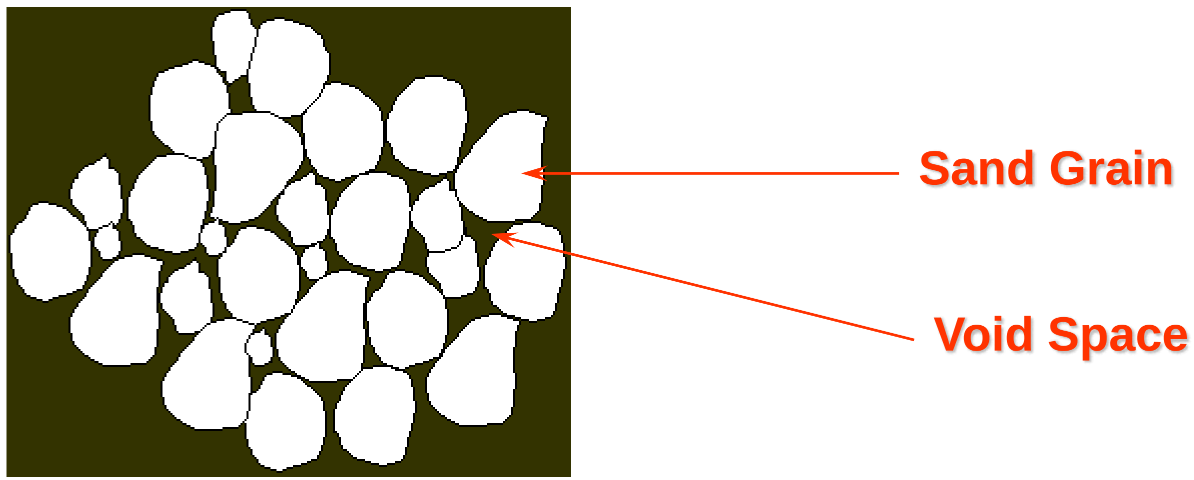
Geotechnical Engineering
Dr. Omar Al-Hattamleh

Part III: Phase relationships

Part I: Soil Mechanics

- Volume-Volume relation
- Mass-Mass relation
- Mass-Volume relation
- Derivative formulas

The nature of Soil

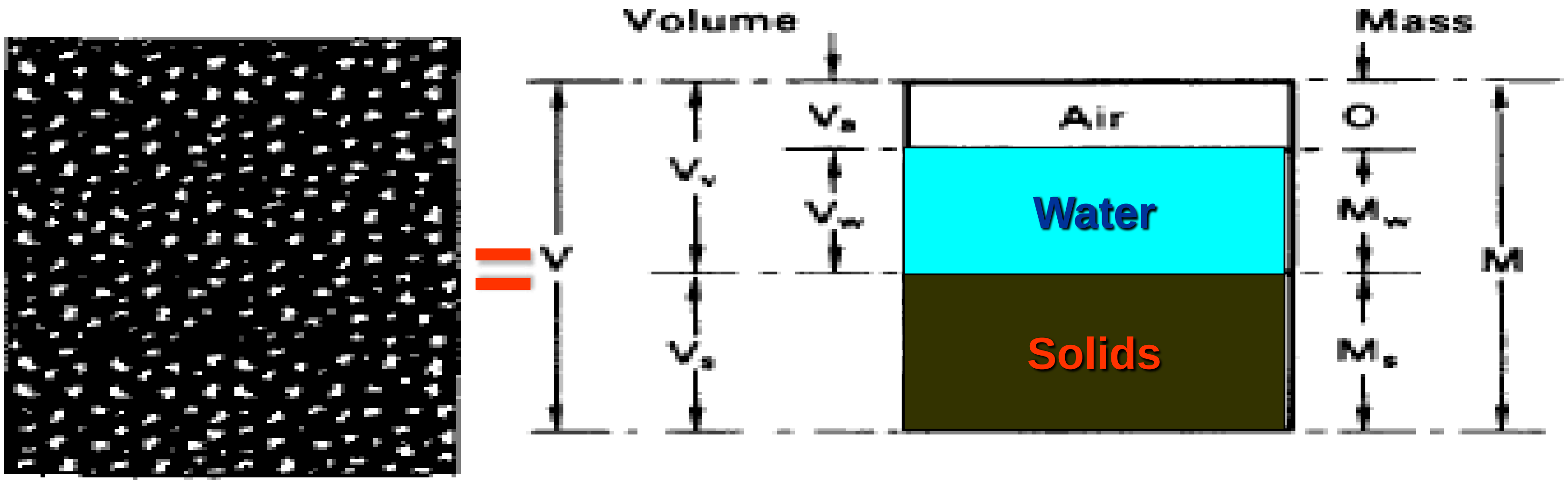


Soil is any uncemented or weakly cemented accumulation of mineral particles formed by the weathering of rocks.

The void space between the particles can be filled with

- ❑ **Liquid Water and/or**
- ❑ **Gas (Air)**

Volume -Volume Relation



Vs = volume of soil solids
Vw= volume of water
Va=volume of air
Vv = volume of voids
V = total volume of soil

Volume -Volume Relation

- The **void ratio (e)** is the ratio of the volume of voids to the volume of solid

$$e = \frac{V_v}{V_s}$$

- The **porosity (n)** is the ratio of the volume of voids to the total volume of the soil,

$$n = \frac{V_v}{V}$$

- The **degree of saturation (Sr)** is the ratio of the volume of water to the total volume of void space

The Sr can range between the limits of 0 for a dry soil and 1 (or 100%) for a saturated soil.

$$S_r = \frac{V_w}{V_v}$$

Volume -Volume Relation

$$n = \frac{V_v}{V} = \frac{V_v}{V_s + V_v} = \frac{\frac{V_v}{V_s}}{\frac{V_s}{V_s} + \frac{V_v}{V_s}} = \frac{e}{1 + e}$$

$$e = \frac{n}{1 - n}$$

specific volume (*v*) is the total volume of soil which contains unit volume of solids,

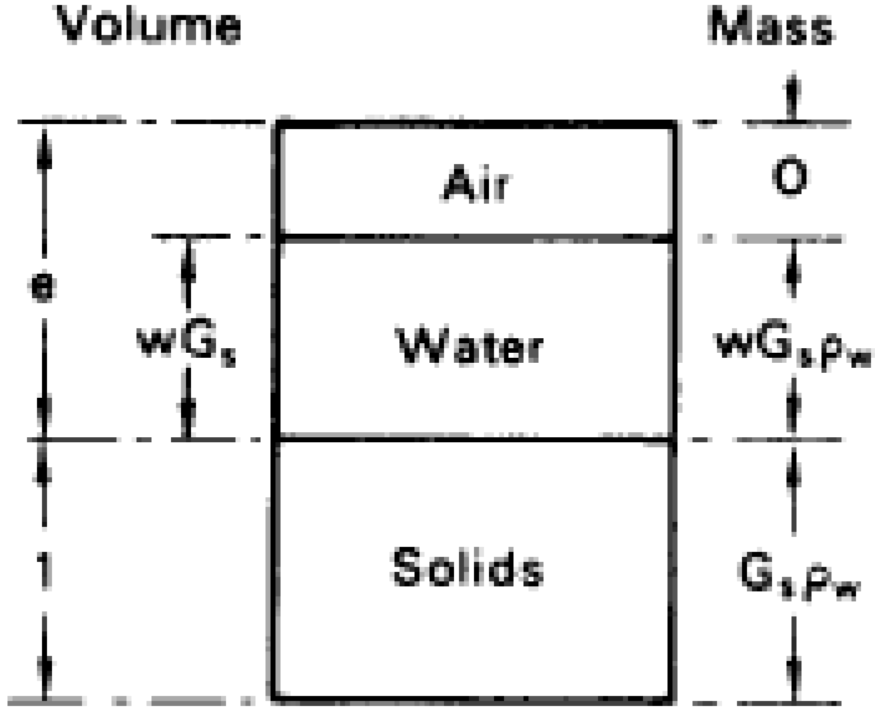
$$v = 1 + e$$

The **air content or air voids (A)** is the ratio of the volume of air to the total volume of the soil

$$A = \frac{V_a}{V}$$

Mass-Mass Relation

- The water content (w), or moisture content (m), is the ratio of the mass of water to the mass of solids in the soil,



$$w = \frac{M_w}{M_s}$$

Mass-Mass Relation

- The bulk density (ρ) of a soil is the ratio of the total mass to the total volume,

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

- ❑ Convenient units for density are kg/m^3 or Mg/m^3.
- ❑ The density of water (1000 kg/m^3 or 1.00 Mg/m^3) is denoted by ρ_w.

- The specific gravity of the soil particles (Gs) is given by

$$G_s = \frac{M_s}{V_s \rho_w} = \frac{\rho_s}{\rho_w}$$

where ρ_s is the particle density.

Derived Relation

- The degree of saturation can be expressed as

$$S_r = \frac{wG_s}{e}$$

- In the case of a fully saturated soil, Sr = 1; hence

$$e = wG_s$$

- The air content can be expressed as

$$A = \frac{e - wG_s}{1 + e} \quad \textbf{or} \quad A = n(1 - S_r)$$

Derived Relation

- The bulk density of a soil can be expressed as

$$\rho = \frac{G_s(1+w)}{1+e}\rho_w \quad \textbf{or} \quad \rho = \frac{G_s + S_r e}{1+e}\rho_w$$

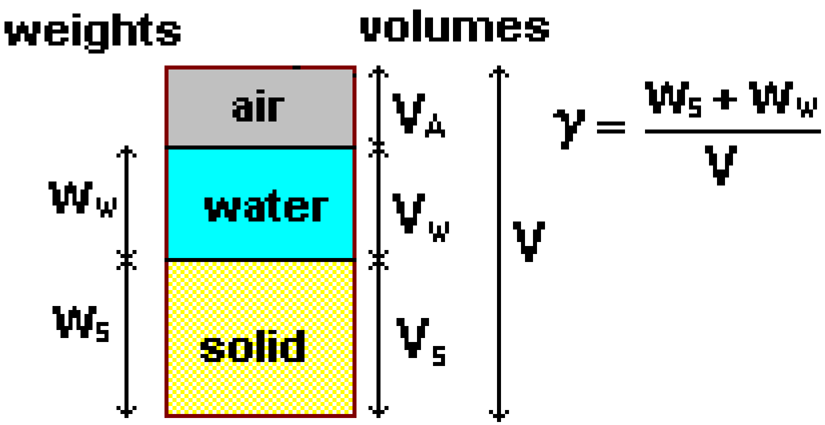
- For a fully saturated soil (Sr = 1)

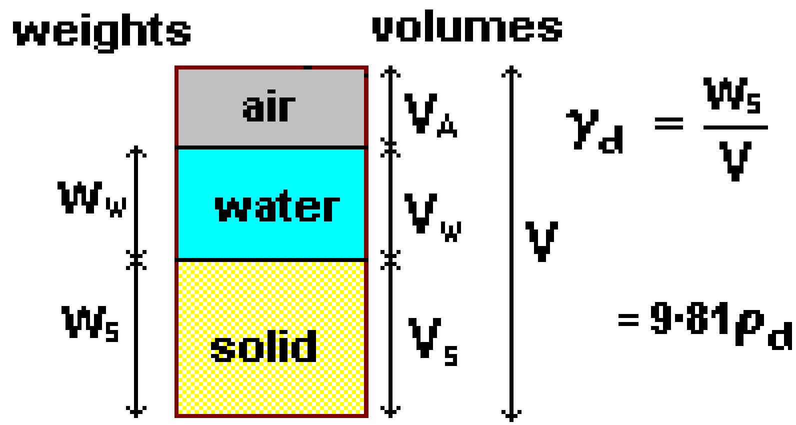
$$\rho_{sat} = \frac{G_s + e}{1+e}\rho_w$$

- For a completely dry soil (Sr = 0)

$$\rho_d = \frac{G_s}{1+e}\rho_w$$

Unit Weights





The unit weight (γ) of a soil is the ratio of the total weight (a force) to the total volume,

$$\gamma = \frac{W}{V} = \frac{Mg}{V}$$

or

$$\gamma = \frac{G_s(1+w)}{1+e}\gamma_w$$

$$\gamma = \frac{G_s + S_r e}{1+e}\gamma_w$$

Unit Weights

- where w is the unit weight of water. Convenient units are kN/m^3, the unit weight of water being 9.8 kN/m^3 (or 10.0 kN/m^3 in the case of sea water).
- When a soil in situ is fully saturated the solid soil particles (volume: 1 unit, weight: $G_s w$) are subjected to upthrust (γ_w). Hence, the buoyant unit weight (γ') is given by

$$\gamma' = \frac{G_s\gamma_w - \gamma_w}{1+e} = \frac{G_s - 1}{1+e}\gamma_w \quad \textbf{or} \quad \gamma' = \gamma_{sat} - \gamma_w$$

Other name for buoyant unit weight is effective unit weight or submerged unit weight

Relative density

- In the case of sands and gravels the density index (I_D) is used to express the relationship between the in-situ void ratio (e), or the void ratio of a sample, and the limiting values e_{max} and e_{min}. The density index (the term 'relative density, or Dr' is also used) is defined as

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

- The density index of a soil in its densest possible state ($e = e_{min}$) is 1 (or 100%)
- The density index in its loosest possible state ($e = e_{max}$) is 0.

EXAMPLE 1

In its natural condition a soil sample has a mass of 2290 g and a volume of 1.15×10^{-3} m^3. After being completely dried in an oven the mass of the sample is 2035 g. The value of Gs for the soil is 2.68. Determine

1. the bulk density,
2. unit weight
3. water content,
4. void ratio,
5. porosity,
6. degree of saturation and
7. air content.

EXAMPLE 2

- Given ρ=1.76Mg/m^3, ρ_s=2.7Mg/m^3 and w=10%

1. the dry density,
2. void ratio,
3. porosity,
4. degree of saturation and
5. air content
6. and Saturated density

Hint Use the total Volume =1m^3

V
Compaction

Outline

1. Compaction
2. Theory of Compaction
3. Properties and Structure of Compacted Fine-Grained Soils
4. Suggested Homework
5. References

2.1 Compaction and Objectives

Compaction

- Many types of earth construction, such as dams, retaining walls, highways, and airport, require man-placed soil, or fill. To compact a soil, that is, to place it in a dense state.
- The dense state is achieved through the reduction of the air voids in the soil, with little or no reduction in the water content. This process must not be confused with consolidation, in which water is squeezed out under the action of a continuous static load.

Objectives:

(1) Decrease future settlements

(2) Increase shear strength

(3) Decrease permeability

2.2 General Compaction Methods

	Coarse-grained soils	**Fine-grained soils**
Laboratory	•Vibrating hammer (BS)	•Falling weight and hammers •Kneading compactors •Static loading and press
Field	•Hand-operated vibration plates •Motorized vibratory rollers •Rubber-tired equipment •Free-falling weight; dynamic compaction (low frequency vibration, 4~10 Hz)	•Hand-operated tampers •Sheepsfoot rollers •Rubber-tired rollers
	Vibration	*Kneading*

(Holtz and Kovacs, 1981)

3.1 Laboratory Compaction

Origin

The fundamentals of compaction of fine-grained soils are relatively new. R.R. Proctor in the early 1930's was building dams for the old Bureau of Waterworks and Supply in Los Angeles, and he developed the principles of compaction in a series of articles in Engineering News-Record. In his honor, the standard laboratory compaction test which he developed is commonly called the *proctor test*.

Purpose

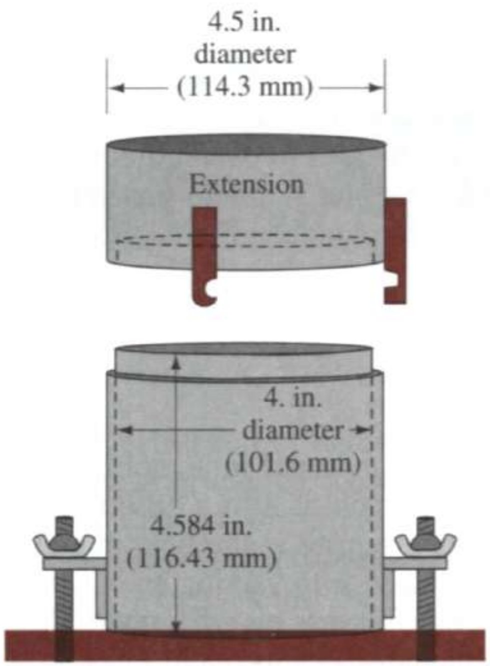
The purpose of a laboratory compaction test is to determine the *proper amount of mixing water* to use when compacting the soil in the field and the *resulting degree of denseness* which can be expected from compaction at this optimum water

Impact compaction

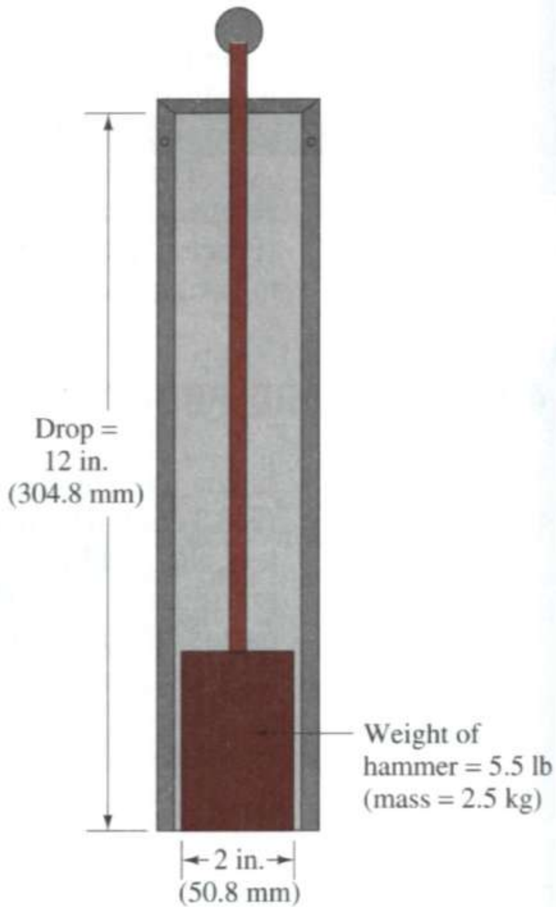
The proctor test is an *impact compaction*. A hammer is dropped several times on a soil sample in a mold. The mass of the hammer, height of drop, number of drops, number of layers of soil, and the volume of the mold are specified.

3.1.1 Test Equipment

Standard Proctor test equipment



(a)



Das, 1998

3.1.2 Comparison-

Standard and Modified Proctor Compaction Test

Standard Proctor Compaction Test Specifications (ASTM D-698)

Modified Proctor Compaction Test Specifications (ASTM D-698)

Standard Proctor Test	Modified Proctor Test
12 in height of drop	18 in height of drop
5.5 lb hammer	10 lb hammer
25 blows/layer	25 blows/layer
3 layers	5 layers
Mold size: 1/30 ft^3	Mold size: 1/30 ft^3
Energy 12,375 $ft \cdot lb/ft^3$	Energy 56,250 $ft \cdot lb/ft^3$

Higher compacting energy

3.1.3 Comparison-Why?

- In the early days of compaction, because construction equipment was small and gave relatively low compaction densities, a laboratory method that used a small amount of compacting energy was required. As construction equipment and procedures were developed which gave higher densities, it became necessary to increase the amount of compacting energy in the laboratory test.

- The modified test was developed during World War II by the U.S. Army Corps of Engineering to better represent the compaction required for airfield to support heavy aircraft. The point is that increasing the compactive effort tends to increase the maximum dry density, as expected, but also decrease the optimum water content.

(Holtz and Kovacs, 1981)

3.2 Variables of Compaction

Proctor established that compaction is a function of four variables:

(1)Dry density (ρ_d) or dry unit weight γ_d.

(2)Water content w

(3)Compactive effort (energy E)

(4)Soil type (gradation, presence of clay minerals, etc.)

Compactive effort (energy E)

$$E = \frac{\text{Weight of hammer} \times \text{Height of drop of hammer} \times \text{Number of blows per layer} \times \text{Number of layers}}{\text{Volume of mold}}$$

For standard Proctor test

$$E = \frac{2.495\,\text{kg}(9.81\,\text{m}/\text{s}^2)(0.3048\,\text{m})(3\text{ layers})(25\,\text{blows}/\text{layer})}{0.944\times 10^{-3}\,\text{m}^3}$$

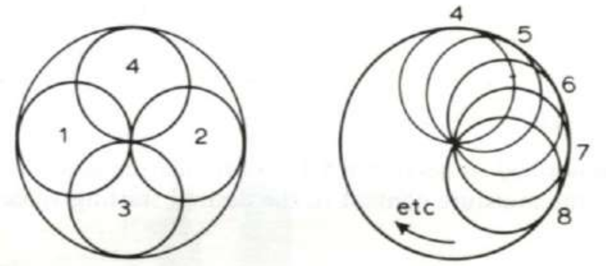
$$= 592.7\,\text{kJ}/\text{m}^3\,(12{,}375\,\text{ft}\cdot\text{lb}/\text{ft}^3)$$

3.3 Procedures and Results

Procedures

(1) Several samples of the same soil, but at different water contents, are compacted according to the compaction test specifications.

The first four blows



The successive blows

(2) The total or wet density and the actual water content of each compacted sample are measured.

$$\rho = \frac{M_t}{V_t}, \rho_d = \frac{\rho}{1+w}$$

Derive ρ_d from the known ρ and w

(3) Plot the dry densities ρ_d versus water contents w for each compacted sample. The curve is called as a *compaction curve*.

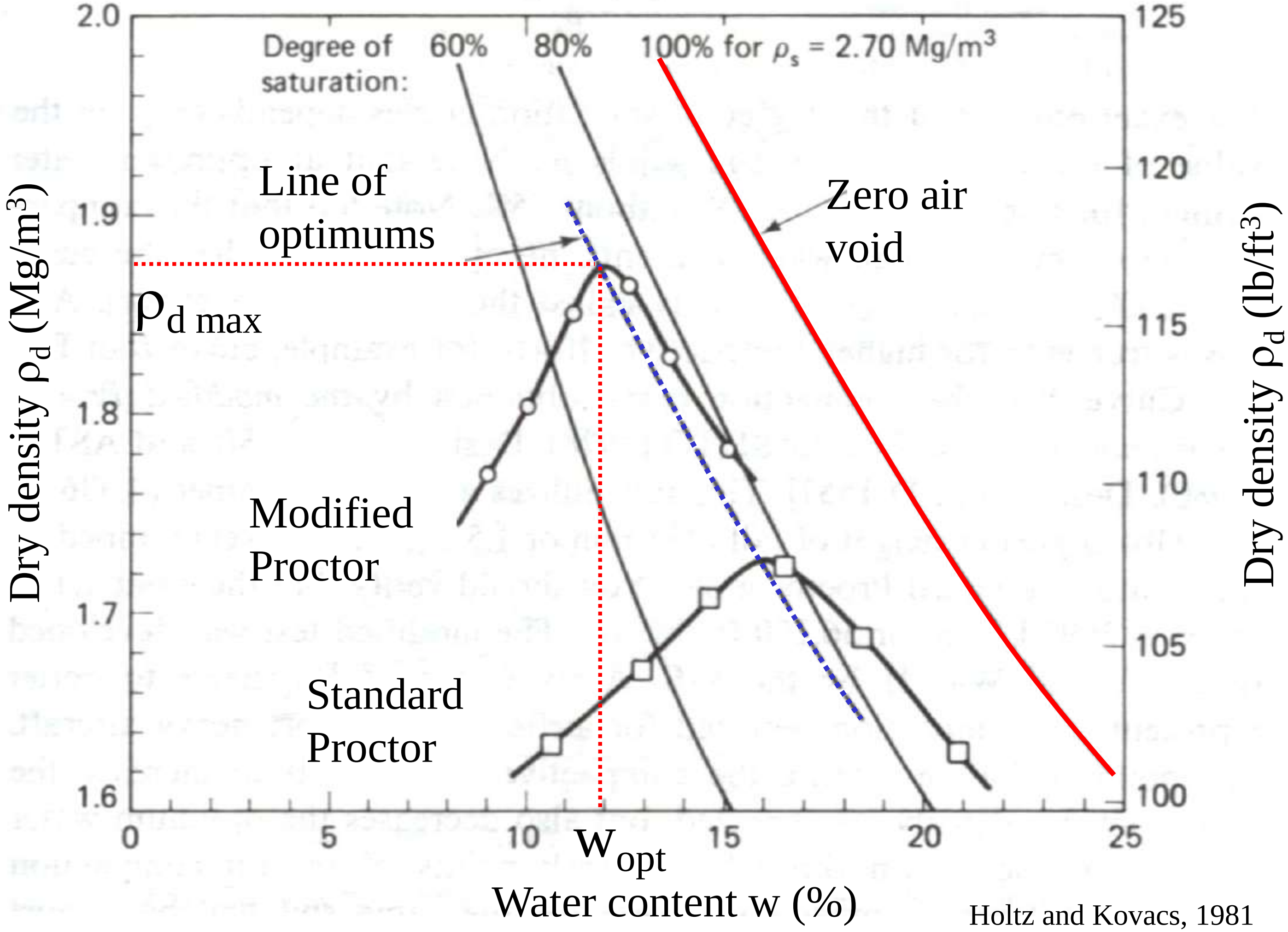
3.3 Procedures and Results (Cont.)

Results

Peak point

Line of optimum

Zero air void



Holtz and Kovacs, 1981

3.3 Procedures and Results (Cont.)

The peak point of the compaction curve

The peak point of the compaction curve is the point with the maximum dry density $\rho_{d\ max}$. Corresponding to the maximum dry density $\rho_{d\ max}$ is a water content known as the optimum water content w_{opt} (also known as the optimum moisture content, OMC). Note that the maximum dry density is only a maximum for a specific compactive effort and method of compaction. This does not necessarily reflect the maximum dry density that can be obtained in the field.

Zero air voids curve

The curve represents the fully saturated condition (S = 100 %). (*It cannot be reached by compaction*)

Line of optimums

A line drawn through the peak points of several compaction curves at different compactive efforts for the same soil will be almost parallel to a 100 % S curve, it is called the line of optimums

3.3 Procedures and Results (Cont.)

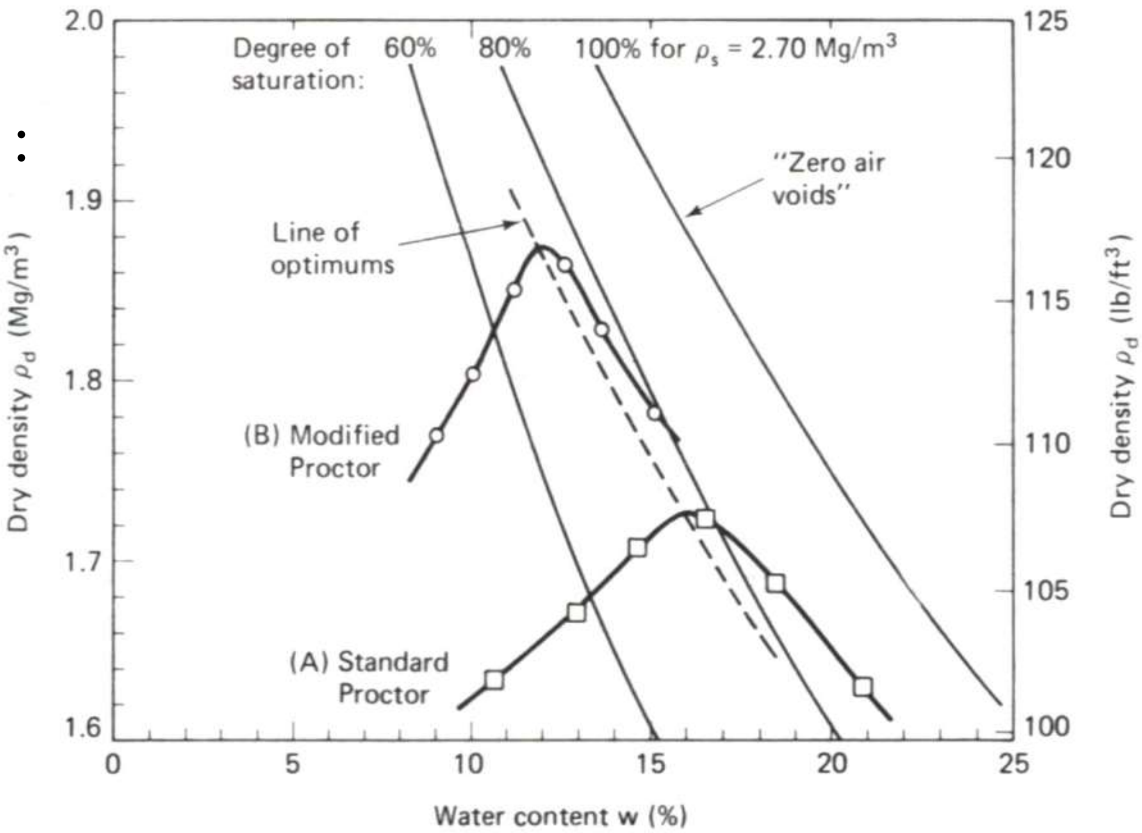
The Equation for the curves with different degree of saturation is :

$$\rho_d = \frac{\rho_w S}{w + \frac{\rho_w}{\rho_s} S} = \frac{\rho_w S}{w + \frac{S}{G_s}}$$

You can derive the equation by yourself

Hint: $$\rho_d = \frac{\rho_s}{1+e}$$

$$Se = wG_s$$



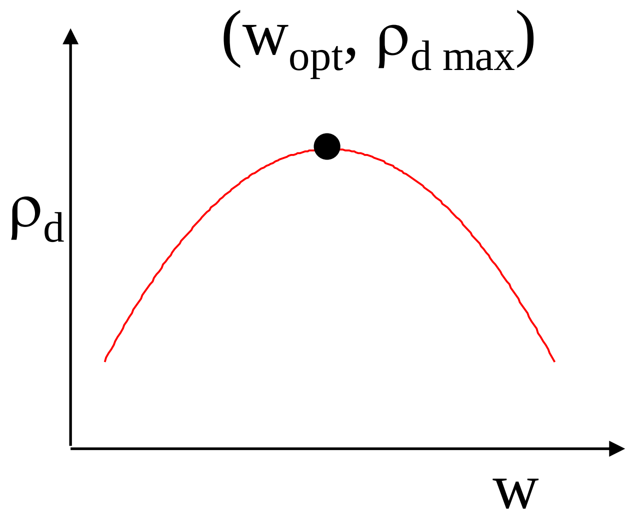
Holtz and Kovacs, 1981

3.3 Procedures and Results-Explanation

Lubrication or loss of suction??

Below w_{opt} (dry side of optimum):

As the water content increases, the particles develop larger and larger water films around them, which tend to "lubricate" the particles and make them easier to be moved about and reoriented into a denser configuration.



At w_{opt}:

The density is at the maximum, and it does not increase any further.

Above w_{opt} (wet side of optimum):

Water starts to replace soil particles in the mold, and since $\rho_w << \rho_s$ the dry density starts to decrease.

Holtz and Kovacs, 1981

3.3 Procedures and Results-Notes

- Each data point on the curve represents a single compaction test, and usually four or five individual compaction tests are required to completely determine the compaction curve.
- At least two specimens wet and two specimens dry of optimum, and water contents varying by about 2%.
- Optimum water content is typically slightly less than the plastic limit (ASTM suggestion).
- Typical values of maximum dry density are around 1.6 to 2.0 Mg/m^3 with the maximum range from about 1.3 to 2.4 Mg/m^3. Typical optimum water contents are between 10% and 20%, with an outside maximum range of about 5% to 40%.

Holtz and Kovacs, 1981

3.4 Effects of Soil Types on Compaction

The soil type-that is, grain-size distribution, shape of the soil grains, specific gravity of soil solids, and amount and type of clay minerals present.

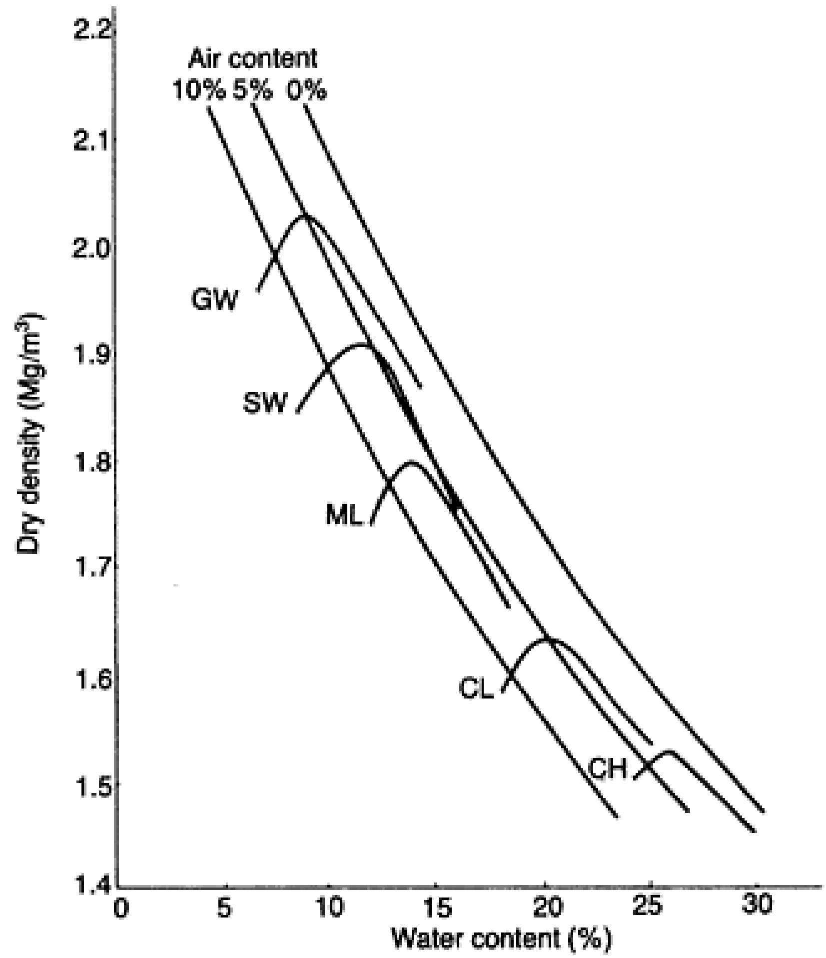


Figure 1.13 Dry density–water content curves for a range of soil types.

4.1 Structure of Compacted Clays

- For a given compactive effort and dry density, the soil tends to be more flocculated (random) for compaction on the dry side as compared on the wet side.
- For a given molding water content, increasing the compactive effort tends to disperse (parallel, oriented) the soil, especially on the dry side.

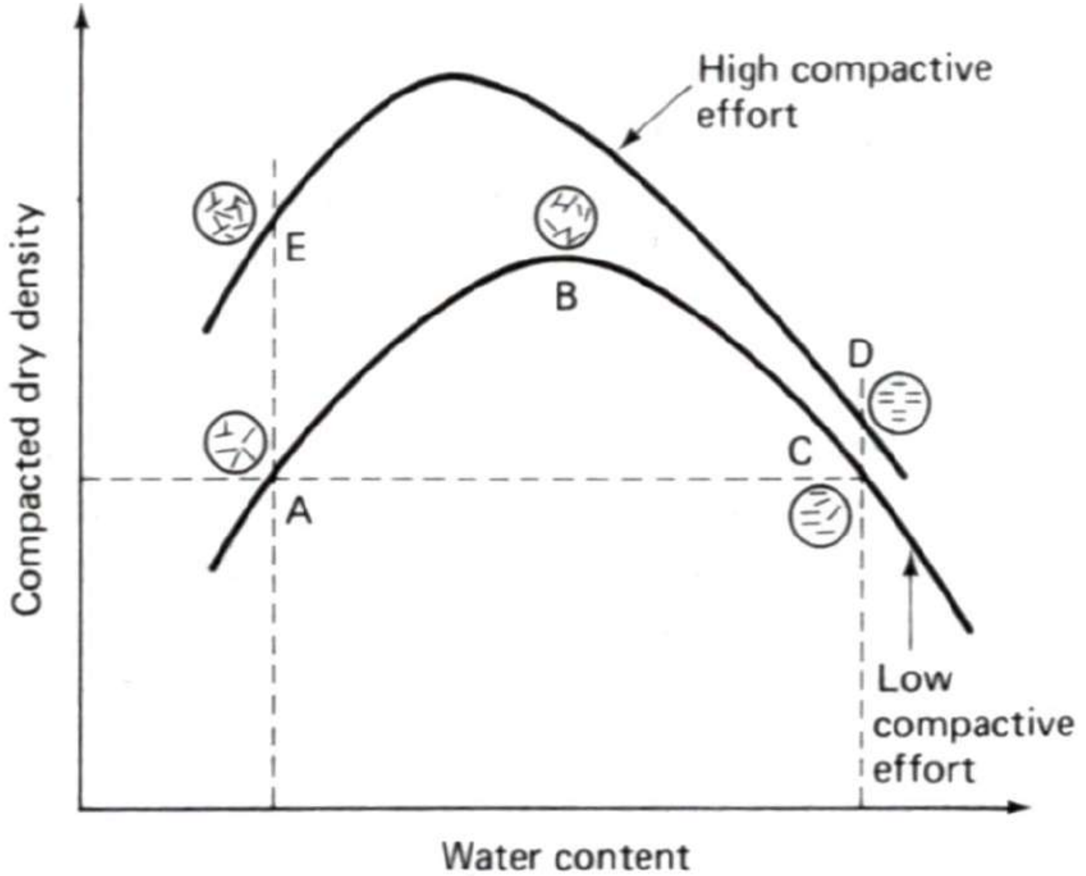
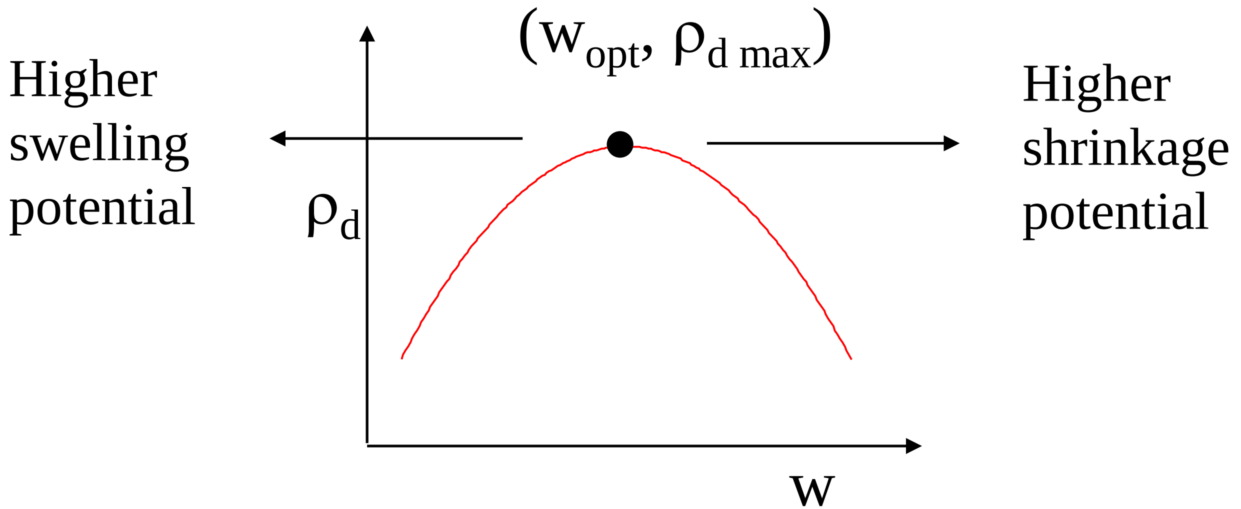


Fig. 5.5 Effect of compaction on soil structure (after Lambe, 1958a).

Lambe and Whitman, 1979

4.2 Engineering Properties-Swelling

- Swelling of compacted clays is greater for those compacted dry of optimum. They have a relatively greater deficiency of water and therefore have a greater tendency to adsorb water and thus swell more.



From Holtz and Kovacs, 1981

5.1 Control Parameters

- *Dry density* and *water content* correlate well with the engineering properties, and thus they are convenient construction control parameters.

- Since the objective of compaction is to stabilize soils and improve their engineering behavior, it is important to keep in mind the desired engineering properties of the fill, not just its dry density and water content. This point is often lost in the earthwork construction control.

From Holtz and Kovacs, 1981

5.2 Design-Construct Procedures

- Laboratory tests are conducted on samples of the proposed borrow materials to define the properties required for design.
- After the earth structure is designed, the compaction specifications are written. Field compaction *control tests* are specified, and the results of these become the standard for controlling the project.

From Holtz and Kovacs, 1981

5.3 Specifications

(1) End-product specifications

This specification is used for most highways and building foundation, as long as the contractor is able to obtain the specified *relative compaction* , how he obtains it doesn't matter, nor does the equipment he uses.

Care the results only !

(2) Method specifications

The type and weight of roller, the number of passes of that roller, as well as the lift thickness are specified. A maximum allowable size of material may also be specified.

It is typically used for large compaction project.

From Holtz and Kovacs, 1981

5.4 Relative Compaction (R.C.)

Relative compaction or percent compaction

$$\text{R.C.} = \frac{\rho_{d-\text{field}}}{\rho_{d\max-\text{laboratory}}} \times 100\%$$

Correlation between relative compaction (R.C.) and the relative density Dr

$$\text{R.C.} = 80 + 0.2D_r$$

It is a statistical result based on 47 soil samples.

As Dr = 0, R.C. is 80

Typical required R.C. = 90% ~ 95%

5.6 Determine the Relative Compaction in the Field

Where and When

- First, the test site is selected. It should be representative or typical of the compacted lift and borrow material. Typical specifications call for a new field test for every 1000 to 3000 m^2 or so, or when the borrow material changes significantly. It is also advisable to make the field test at least one or maybe two compacted lifts below the already compacted ground surface, especially when sheepsfoot rollers are used or in granular soils.

Method

- Field control tests, measuring the dry density and water content in the field can either be *destructive* or *nondestructive*.

Holtz and Kovacs, 1981

5.6.1 Destructive Methods

Methods

(a) Sand cone

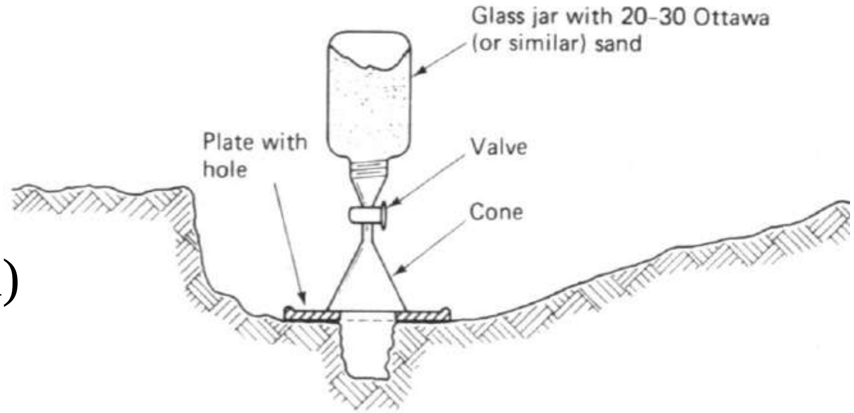
(b) Balloon

(c) Oil (or water) method

Calculations

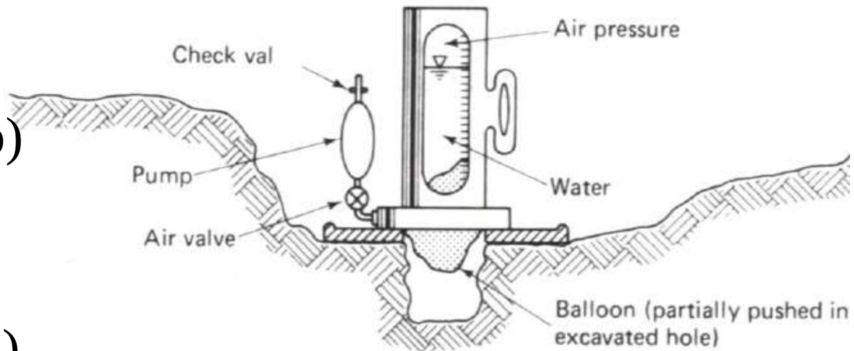
- Know M_s and V_t
- Get $\rho_{d\ field}$ and w (water content)
- Compare $\rho_{d\ field}$ with $\rho_{d\ max\text{-}lab}$ and calculate relative compaction R.C.

(a)



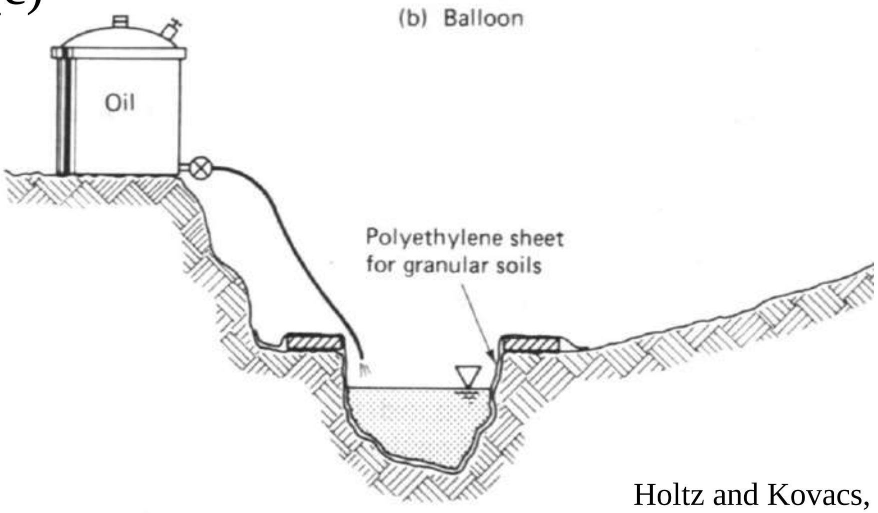
(a) Sand cone

(b)



(b) Balloon

(c)



Holtz and Kovacs, 1981

(c) Oil (or water) method

5.6.1 Destructive Methods (Cont.)

- The measuring error is mainly from the determination of the volume of the excavated material.

For example,

- For the sand cone method, the vibration from nearby working equipment will increase the density of the sand in the hole, which will gives a larger hole volume and a lower field density.

$$\rho_{d-\text{field}} = M_s / V_t$$

- If the compacted fill is gravel or contains large gravel particles. Any kind of unevenness in the walls of the hole causes a significant error in the balloon method.

- If the soil is coarse sand or gravel, none of the liquid methods works well, unless the hole is very large and a polyethylene sheet is used to contain the water or oil.

Holtz and Kovacs, 1981

5.6.2 Nondestructive Methods

Nuclear density meter

(a) Direct transmission

(b) Backscatter

(c) Air gap

Principles

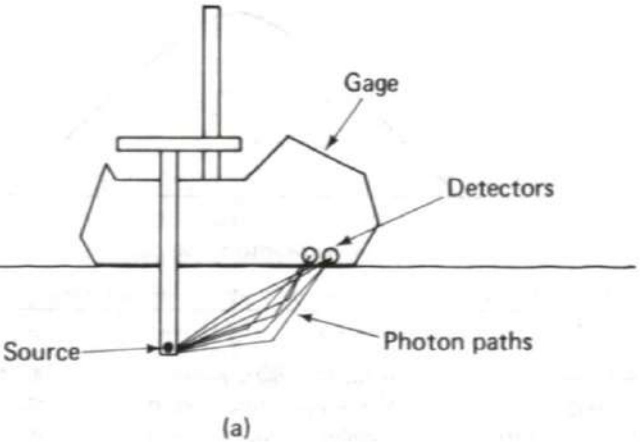
Density

The Gamma radiation is scattered by the soil particles and the amount of scatter is proportional to the total density of the material. The Gamma radiation is typically provided by the radium or a radioactive isotope of cesium.

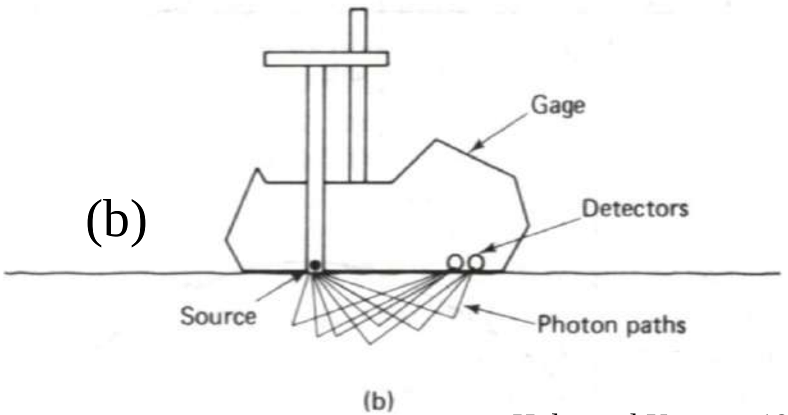
Water content

The water content can be determined based on the neutron scatter by hydrogen atoms. Typical neutron sources are americium-beryllium isotopes.

(a)

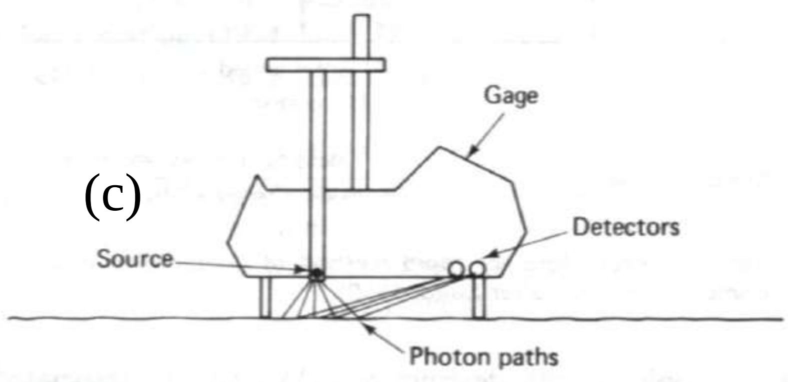


(b)



Holtz and Kovacs, 1981

(c)



9. References

Main References:

Holtz, R.D. and Kovacs, W.D. (1981). *An Introduction to Geotechnical Engineering*, Prentice Hall. (Chapter 5)

Others:

Das, B.M. (1998). *Principles of Geotechnical Engineering*, 4th edition, PWS Publishing Company.

Lambe, T.W. and Whitman, R.V. (1979). *Soil Mechanics*, SI Version, John Wiley & Sons.

Schaefer, V. R. (1997). *Ground Improvement, Ground Reinforcement, Ground Treatment*, Proceedings of Soil Improvement and Geosynthetics of The Geo-Institute of the American Society of Civil Engineers in conjunction with Geo-Logan’97. Edited by V.R. Schaefer.

III.
Soil Classification

Outline

1. Purpose
2. Classification Systems
3. The Unified Soil Classification System (USCS)
4. American Association of State Highway and Transportation Officials System (AASHTO)
5. Suggested Homework

1. Purpose

Classifying soils into groups with similar behavior, in terms of *simple* indices, can provide geotechnical engineers a general guidance about engineering properties of the soils through the *accumulated experience*.

Communicate between engineers

Simple indices

GSD, LL, PI

→ Classification system (Language) → Estimate engineering properties → Achieve engineering purposes

Use the accumulated experience

2. Classification Systems

Two commonly used systems:

- Unified Soil Classification System (USCS).
- American Association of State Highway and Transportation Officials (AASHTO) System
- Craig's Soil Mechanics use **BS**

3. Unified Soil Classification System (USCS)

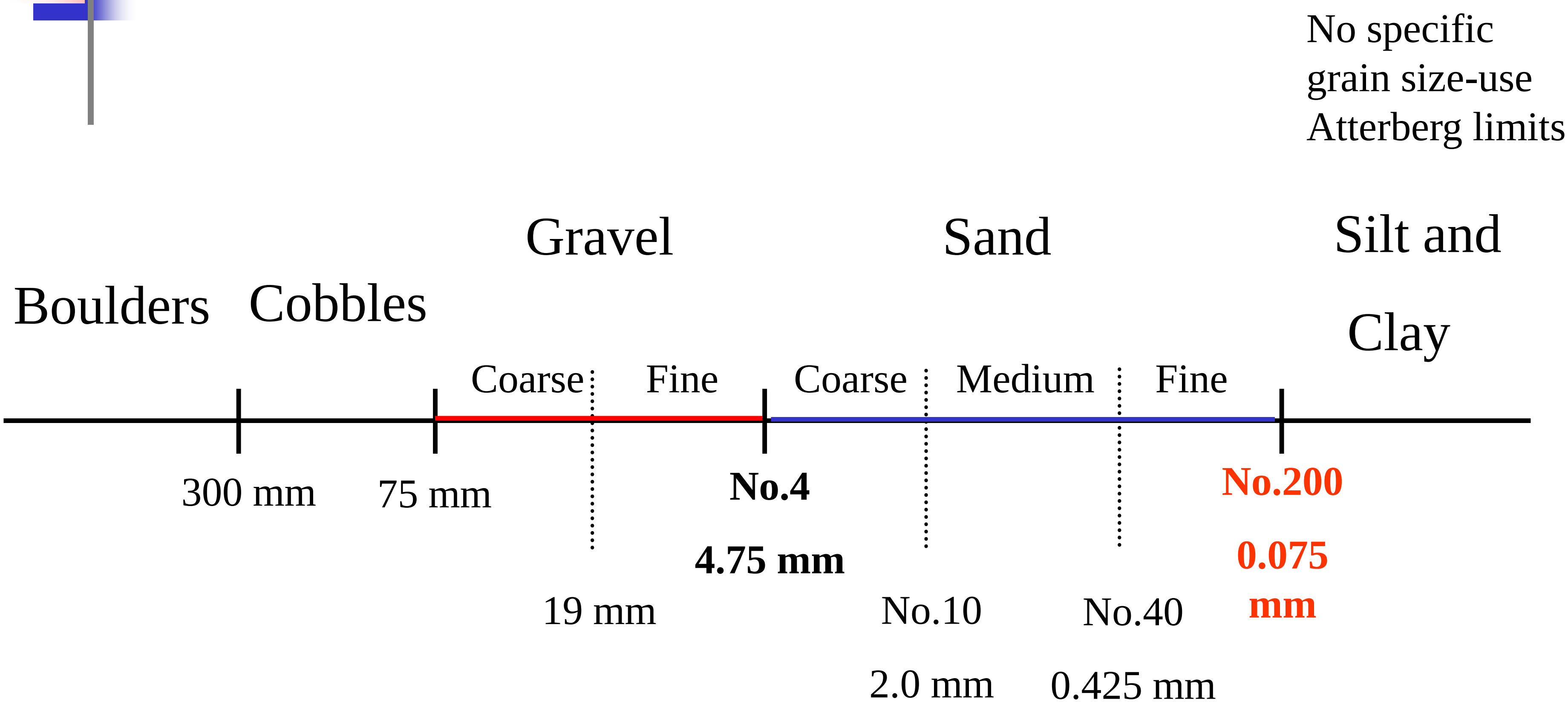
Origin of USCS:

This system was first developed by Professor A. Casagrande (1948) for the purpose of airfield construction during World War II. Afterwards, it was modified by Professor Casagrande, the U.S. Bureau of Reclamation, and the U.S. Army Corps of Engineers to enable the system to be applicable to dams, foundations, and other construction (Holtz and Kovacs, 1981).

Four major divisions:

(1) Coarse-grained
(2) Fine-grained
(3) Organic soils
(4) Peat

3.1 Definition of Grain Size



3.2 General Guidance

50 %

Coarse-grained soils: Fine-grained soils:

Gravel Sand Silt Clay

50% ***NO. 4*** ***NO.200***

4.75 mm ***0.075 mm***

•Grain size distribution

•C_u

•C_c

•PL, LL

•Plasticity chart

LL>50

LL <50

Required tests: Sieve analysis

Atterberg limit

3.3 Symbols

Soil symbols:

G: Gravel

S: Sand

M: Silt

C: Clay

O: Organic

Pt: Peat

Example: SW, Well-graded sand

SC, Clayey sand

SM, Silty sand,

MH, Elastic silt

Liquid limit symbols:

H: High LL (LL>50)

L: Low LL (LL<50)

Gradation symbols:

W: Well-graded

P: Poorly-graded

Well – graded soil

$$1 < C_c < 3 \text{ and } C_u \geq 4$$

(for gravels)

$$1 < C_c < 3 \text{ and } C_u \geq 6$$

(for sands)

3.4 Plasticity Chart

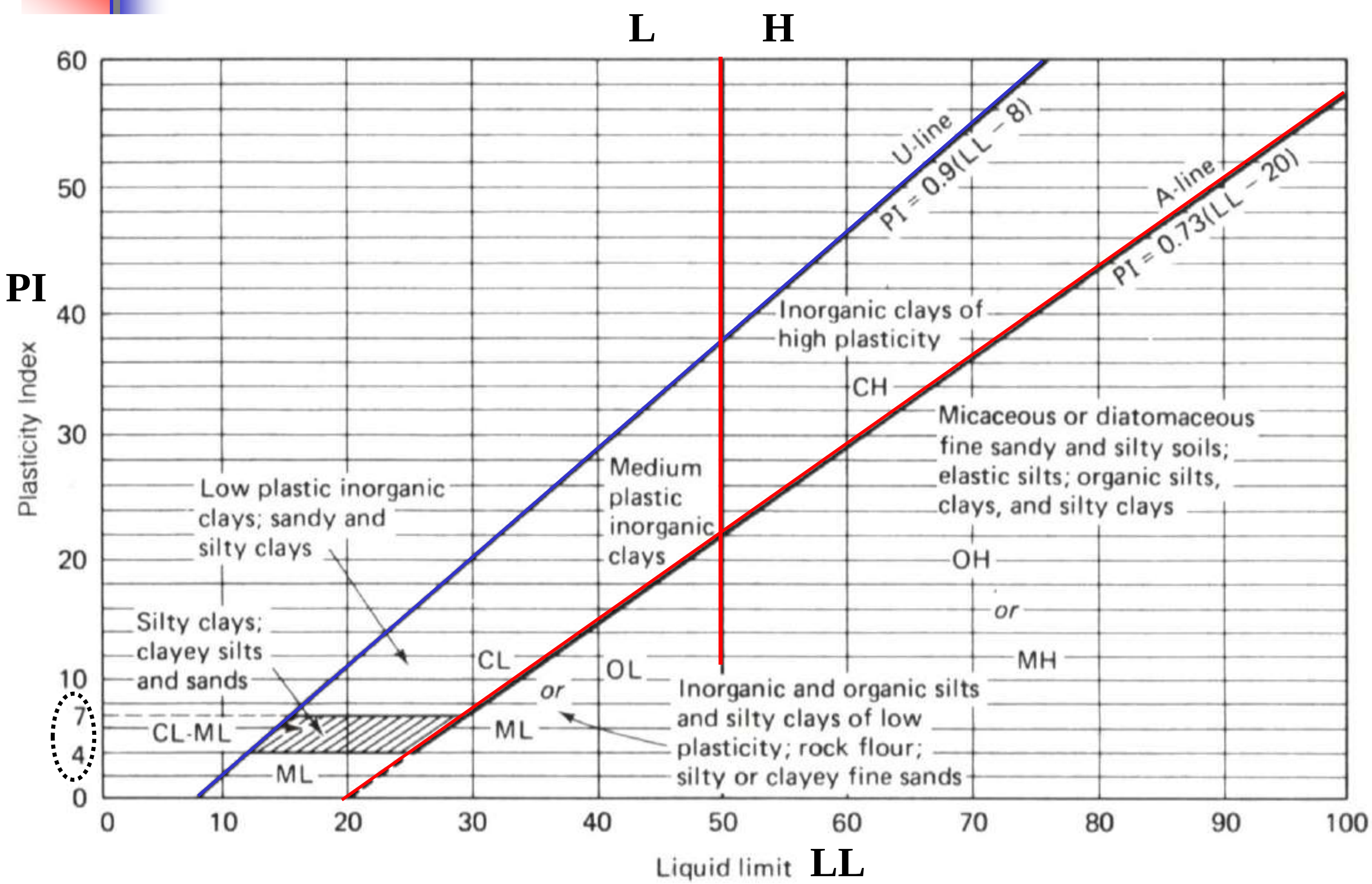


Fig. 3.2 Casagrande's plasticity chart, showing several representative soil types (developed from Casagrande, 1948, and Howard, 1977).

(Holtz and Kovacs, 1981)

- The A-line generally separates the more claylike materials from silty materials, and the organics from the inorganics.
- The U-line indicates the upper bound for general soils.

Note: If the measured limits of soils are on the left of U-line, they should be rechecked.

3.5 Procedures for Classification

Coarse-grained material

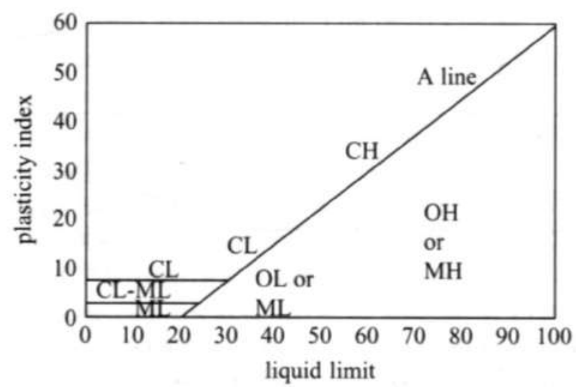
Grain size distribution

COARSE More than 50% retained sieve #200	Gravel: more than 50% coarse fraction retained on sieve #4	Less than 5% fines	$C_u > 4, 1 \le C_c \le 3$	→ GW
			Not satisfying GW	→ GP
		More than 12% fines	Below 'A' line	→ GM
			Above 'A' line	→ GC
	Sand: less than 50% coarse fraction retained on sieve #4	Less than 5% fines	$C_u > 6, 1 \le C_c \le 3$	→ SW
			Not satisfying SW	→ SP
		More than 12% fines	Below 'A' line	→ SM
			Above 'A' line	→ SC

Fine-grained material

LL, PI

FINE Less than 50% retained sieve #200	$LL < 50$		ML
			CL
			OL
	$LL > 50$		MH
			CH
			OH



Highly ORGANIC SOILS → Pt

Table 1.6 Unified Soil Classification Chart (after ASTM, 2005)

Criteria for assigning group symbols and group names using laboratory tests[a]				Soil classification: Group symbol	Soil classification: Group name[b]
Coarse-grained soils More than 50% retained on No. 200 sieve	Gravels More than 50% of coarse fraction retained on No. 4 sieve	Clean Gravels Less than 5% fines[c]	$C_u \ge 4$ and $1 \le C_c \le 3$[e]	GW	Well-graded gravel[f]
			$C_u < 4$ and/or $1 > C_c > 3$[e]	GP	Poorly graded gravel[f]
		Gravels with Fines More than 12% fines[c]	Fines classify as ML or MH	GM	Silty gravel[f,g,h]
			Fines classify as CL or CH	GC	Clayey gravel[f,g,h]
	Sands 50% or more of coarse fraction passes No. 4 sieve	Clean Sands Less than 5% fines[d]	$C_u \ge 6$ and $1 \le C_c \le 3$[e]	SW	Well-graded sand[i]
			$C_u < 6$ and/or $1 > C_c > 3$[e]	SP	Poorly graded sand[i]
		Sand with Fines More than 12% fines[d]	Fines classify as ML or MH	SM	Silty sand[g,h,i]
			Fines classify as CL or CH	SC	Clayey sand[g,h,i]
Fine-grained soils 50% or more passes the No. 200 sieve	Silts and Clays Liquid limit less than 50	Inorganic	PI > 7 and plots on or above "A" line[j]	CL	Lean clay[k,l,m]
			PI < 4 or plots below "A" line[j]	ML	Silt[k,l,m]
		Organic	$\dfrac{\text{Liquid limit—oven dried}}{\text{Liquid limit—not dried}} < 0.75$	OL	Organic clay[k,l,m,n] / Organic silt[k,l,m,o]
	Silts and Clays Liquid limit 50 or more	Inorganic	PI plots on or above "A" line	CH	Fat clay[k,l,m]
			PI plots below "A" line	MH	Elastic silt[k,l,m]
		Organic	$\dfrac{\text{Liquid limit—oven dried}}{\text{Liquid limit—not dried}} < 0.75$	OH	Organic clay[k,l,m,p] / Organic silt[k,l,m,q]
Highly organic soils	Primarily organic matter, dark in color, and organic odor			PT	Peat

[a]Based on the material passing the 75-mm (3-in) sieve.
[b]If field sample contained cobbles or boulders, or both, add "with cobbles or boulders, or both" to group name.
[c]Gravels with 5 to 12% fines require dual symbols: GW-GM well-graded gravel with silt; GW-GC well-graded gravel with clay; GP-GM poorly graded gravel with silt; GP-GC poorly graded gravel with clay.
[d]Sands with 5 to 12% fines require dual symbols: SW-SM well-graded sand with silt; SW-SC well-graded sand with clay; SP-SM poorly graded sand with silt; SP-SC poorly graded sand with clay.

[e]$C_u = D_{60}/D_{10}$ $\quad C_c = \dfrac{(D_{30})^2}{D_{10} \times D_{60}}$
[f]If soil contains ≥15% sand, add "with sand" to group name.
[g]If fines classify as CL-ML, use dual symbol GC-GM or SC-SM.
[h]If fines are organic, add "with organic fines" to group name.
[i]If soil contains ≥15% gravel, add "with gravel" to group name.
[j]If Atterberg limits plot in hatched area, soil is a CL-ML, silty clay.

[k]If soil contains 15 to 29% plus No. 200, add "with sand" or "with gravel," whichever is predominant.
[l]If soil contains ≥30% plus No. 200, predominantly sand, add "sandy" to group name.
[m]If soil contains ≥30% plus No. 200, predominantly gravel, add "gravelly" to group name.
[n]PI ≥ 4 and plots on or above "A" line.
[o]PI < 4 or plots below "A" line.
[p]PI plots on or above "A" line.
[q]PI plots below "A" line.

TABLE 3-2 Unified Soil Classification System*

Major Divisions			Group Symbols (†)	Typical Names	Field Identification Procedures (excluding particles larger than 75 mm and basing fractions on estimated weights)		
1	2		3	4	5		
Coarse-grained Soils More than half of material is larger than No. 200 (‡) (75 µm) sieve size. The No. 200 sieve size is about the smallest particle visible to the naked eye.	Gravels More than half of coarse fraction is larger than No. 4 sieve size. (4.75 mm) (For visual classification, 5 mm may be used as equivalent to the No. 4 sieve size)	Clean Gravels (little or no fines)	GW	Well-graded gravels, gravel sand mixtures, little or no fines.	Wide range in grain sizes and substantial amounts of all intermediate particle sizes.		
			GP	Poorly graded gravels, gravel-sand mixtures, little or no fines.	Predominantly one size or a range of sizes with some intermediate sizes missing.		
		Gravels with Fines (appreciable amount of fines)	GM	Silty gravels, gravel-sand-silt mixtures.	Nonplastic fines or fines with low plasticity (for identification procedures see ML below).		
			GC	Clayey gravels, gravel-sand-clay mixtures.	Plastic fines (for identification procedures see CL below).		
	Sands More than half of coarse fraction is smaller than No. 4 sieve size. (4.75 mm)	Clean Sands (little or no fines)	SW	Well-graded sands, gravelly sands, little or no fines.	Wide range in grain sizes and substantial amounts of all intermediate particle sizes.		
			SP	Poorly graded sands, gravelly sands, little or no fines.	Predominantly one size or a range of sizes with some intermediate sizes missing.		
		Sands with Fines (appreciable amount of fines)	SM	Silty sands, sand-silt mixtures.	Nonplastic fines or fines with low plasticity (for identification procedures see ML below).		
			SC	Clayey sands, sand-clay mixtures.	Plastic fines (for identification procedures see CL below).		
					Identification Procedures on Fraction Smaller than No. 40 Sieve Size		
					Dry Strength (crushing characteristics)	Dilatancy (reaction to shaking)	Toughness (consistency near PL)
Fine-grained Soils More than half of material is smaller than No. 200 (75 µm) sieve size.	Silts and Clays	Liquid limit less than 50	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity.	None to slight	Quick to slow	None
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.	Medium to high	None to very slow	Medium
			OL	Organic silts and organic silty clays of low plasticity.	Slight to medium	Slow	Slight
	Silts and Clays	Liquid limit greater than 50	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.	Slight to medium	Slow to none	Slight to medium
			CH	Inorganic clays of high plasticity, fat clays.	High to very high	None	High
			OH	Organic clays of medium to high plasticity, organic silts.	Medium to high	None to very slow	Slight to medium
Highly Organic Soils			Pt	Peat and other highly organic soils.	Readily identified by color, odor, spongy feel, and frequently by fibrous texture.		

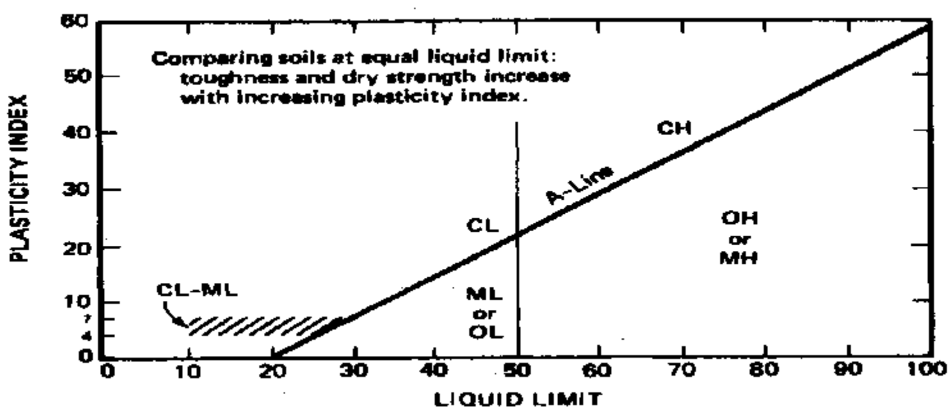
† Boundary classifications: soils possessing characteristics of two groups are designated by combinations of group symbols. For example GW-GC, well-graded gravel-sand mixture with clay binder.

‡ All sieve sizes on this chart are U.S. Standard.

TABLE 3-2 Continued

Laboratory Classification Criteria			
6			
Use grain size curve in identifying the fractions as given under field identification.	Determine percentages of gravel and sand from grain size curve. Depending on percentage of fines (fraction smaller than No. 200 sieve size) coarse-grained soils are classified as follows: Less than 5%: GW, GP, SW, SP. More than 12%: GM, GC, SM, SC. 5% to 12%: Borderline cases requiring use of dual symbols.	$C_u = \dfrac{D_{60}}{D_{10}}$ greater than 4; $C_c = \dfrac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3	(See Sec. 2-5)
		Not meeting all gradation requirements for GW	
		Atterberg limits below A-line, or PI less than 4	Above A-line with PI between 4 and 7 are borderline cases requiring use of dual symbols.
		Atterberg limits above A-line with PI greater than 7	
		$C_u = \dfrac{D_{60}}{D_{10}}$ greater than 6; $C_c = \dfrac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3	(See Sec. 2-5)
		Not meeting all gradation requirements for SW	
		Atterberg limits below A-line, or PI less than 4	Limits plotting in hatched zone with PI between 4 and 7 are borderline cases requiring use of dual symbols.
		Atterberg limits above A-line with PI greater than 7	

Plasticity Chart
For laboratory classification of fine-grained soils

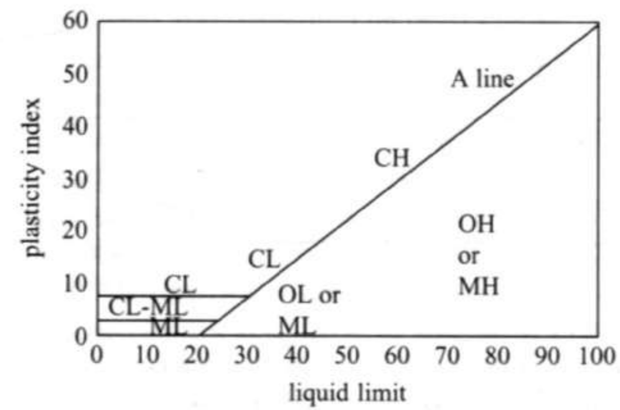


3.6 Example

Passing No.200 sieve 30 % **LL= 33**

Passing No.4 sieve 70 % **PI= 12**

COARSE More than 50% retained sieve #200	Gravel: more than 50% coarse fraction retained on sieve #4	Less than 5% fines	$C_u > 4$, $1 \le C_c \le 3$	→ GW
			Not satisfying GW	→ GP
		More than 12% fines	Below 'A' line	→ GM
			Above 'A' line	→ GC
	Sand: less than 50% coarse fraction retained on sieve #4	Less than 5% fines	$C_u > 6$, $1 \le C_c \le 3$	→ SW
			Not satisfying SW	→ SP
		More than 12% fines	Below 'A' line	→ SM
			Above 'A' line	→ SC
FINE Less than 50% retained sieve #200	$LL < 50$			ML
				CL
				OL
	$LL > 50$			MH
				CH
				OH
Highly ORGANIC SOILS				→ Pt



Passing No.200 sieve 30 %

Passing No.4 sieve 70 %

LL= 33

PI= 12

PI= 0.73(LL-20), A-line

PI=0.73(33-20)=9.49

SC

(≥15% gravel)

Clayey sand with gravel

3.7 Organic Soils

- **Highly organic soils- Peat (Group symbol PT)**
  - A sample composed primarily of vegetable tissue in various stages of decomposition and has a fibrous to amorphous texture, a dark-brown to black color, and an organic odor should be designated as a highly organic soil and shall be classified as peat, PT.

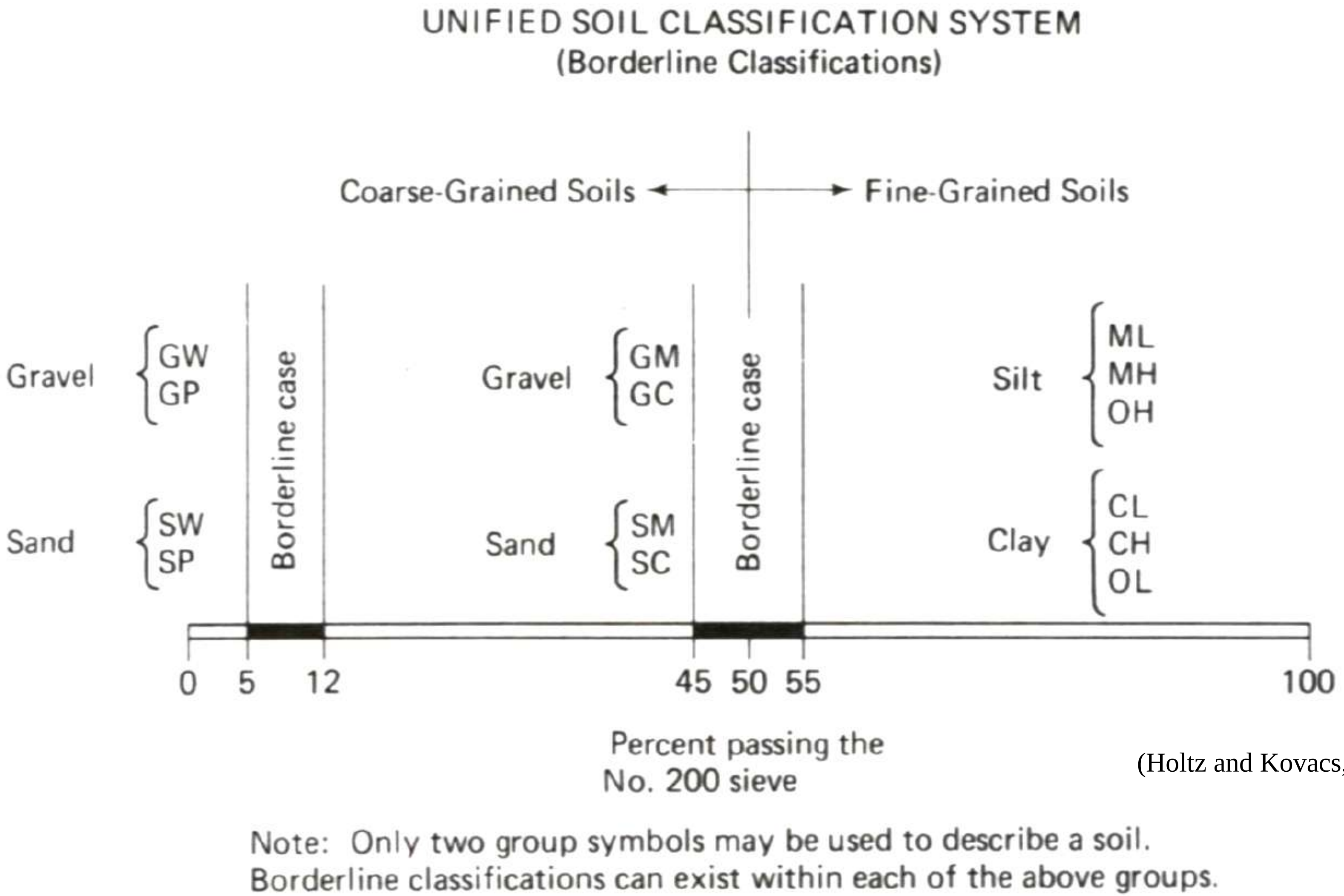
- **Organic clay or silt(group symbol OL or OH):**
  - "The soil's liquid limit (LL) after oven drying is less than 75 % of its liquid limit before oven drying." If the above statement is true, then the first symbol is O.
  - The second symbol is obtained by locating the values of PI and LL (not oven dried) in the plasticity chart.

3.8 Borderline Cases (Dual Symbols)

For the following three conditions, a dual symbol should be used.

- Coarse-grained soils with 5% - 12% fines.
  - **About 7 % fines can change the hydraulic conductivity of the coarse-grained media by orders of magnitude**.
  - The first symbol indicates whether the coarse fraction is well or poorly graded. The second symbol describe the contained fines. For example: SP-SM, poorly graded sand with silt.
- Fine-grained soils with limits within the shaded zone. (PI between 4 and 7 and LL between about 12 and 25).
  - It is hard to distinguish between the silty and more claylike materials.
  - CL-ML: Silty clay, SC-SM: Silty, clayed sand.
- Soil contain similar fines and coarse-grained fractions.
  - possible dual symbols GM-ML

3.8 Borderline Cases (Summary)



(Holtz and Kovacs, 1981)

6. References

Main References:

Holtz, R.D. and Kovacs, W.D. (1981). *An Introduction to Geotechnical Engineering*, Prentice Hall. (Chapter 3)

Das, B.M. (1998). *Principles of Geotechnical Engineering*, 4th edition, PWS Publishing Company. (Chapter 3)

Water in Soil

Omar H. Al-Hattamleh
The Hashemite University,
Zarqa, Jordan

Outlines

- Introduction
- Darcy's Law
- Volume of water flowing per unit time
- Measuring K in laboratory
- Seepage Theory
- Flow Net

Introduction

- All soils are permeable materials, water being free to flow through the interconnected pores between the solid particles.
- You must know how much water is flowing through a soil per unit time.
- This knowledge is required to
  - Design earth dams.
  - Determine the quantity of seepage under hydraulic structures.
  - and dewater foundations before and during their construction.
- The pressure of the pore water is measured relative to atmospheric pressure and the level at which the pressure is atmospheric (i.e. zero) is defined as the water table (WT) or the phreatic surface.
- Below the water table the soil is assumed to be fully saturated,
- Below the water table the pore water may be static, the hydrostatic pressure depending on the depth below the water table, or may be seeping through the soil under hydraulic gradient: this PPT is concerned with the second case.

Introduction

- Bernoulli's theorem applies to the pore water but seepage velocities in soils are normally so small that velocity head can be neglected

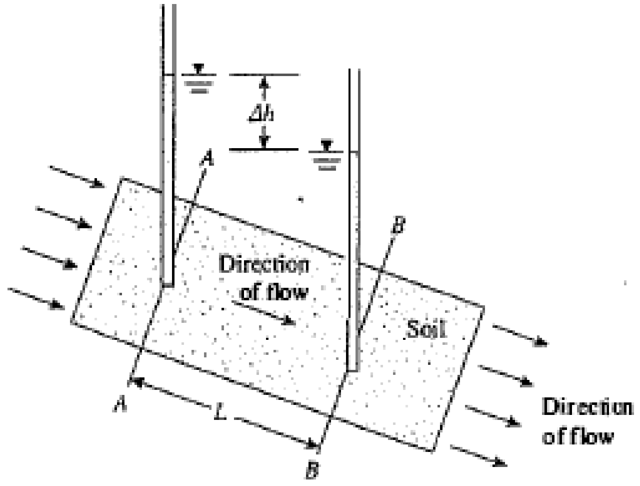
$$h = \frac{u}{\gamma_w} + z$$

where h is the total head, u the pore water pressure, γ_w the unit weight of water (9.8 kN/m^3) and z the elevation head above a chosen datum.

Darcy's law

- Darcy (1856) proposed the following equation for calculating the velocity of flow of water through a soil:

$$v = ki$$



In this equation,
v = Darcy velocity (unit: cm/sec)
k = hydraulic conductivity of soil (unit: cm/sec)
i = hydraulic gradient

The hydraulic gradient is defined as

$$i = \frac{\Delta h}{L}$$

where
Δh = piezometric head difference between the sections at AA and B B
L = distance between the sections at AA and BB
(Note: Sections AA and BB are perpendicular to the direction of flow.)

Volume of water flowing per unit time

$$q = Aki$$

where q is the volume of water flowing per unit time, A the cross-sectional area of soil corresponding to the flow q,

The K also varies with temperature, upon which the viscosity of the water depends. If the value of k measured at 20 C is taken as 100% then the values at 10 and 0 C are 77 and 56%, respectively. The coefficient of permeability can also be represented by the equation:

$$k = \frac{\gamma_w}{\eta} K$$

where γ_w is the unit weight of water, the viscosity of water η and K (units m2) an absolute coefficient depending only on the characteristics of the soil skeleton.

The values of k for different types of soil are typically within the ranges shown in Table

Table 2.1 Coefficient of permeability (m/s) (BS 8004: 1986)

1	10^{-1}	10^{-2}	10^{-3}	10^{-4}	10^{-5}	10^{-6}	10^{-7}	10^{-8}	10^{-9}	10^{-10}
Clean gravels	Clean sands and sand–gravel mixtures			Very fine sands, silts and clay-silt laminate			Unfissured clays and clay-silts (>20% clay)			
	Desiccated and fissured clays									

seepage velocity

On the microscopic scale the water seeping through a soil follows a very tortuous path between the solid particles but macroscopically the flow path

The seepage velocity

$$v' = \frac{q}{A_v}$$

$$v' = \frac{ki}{n}$$

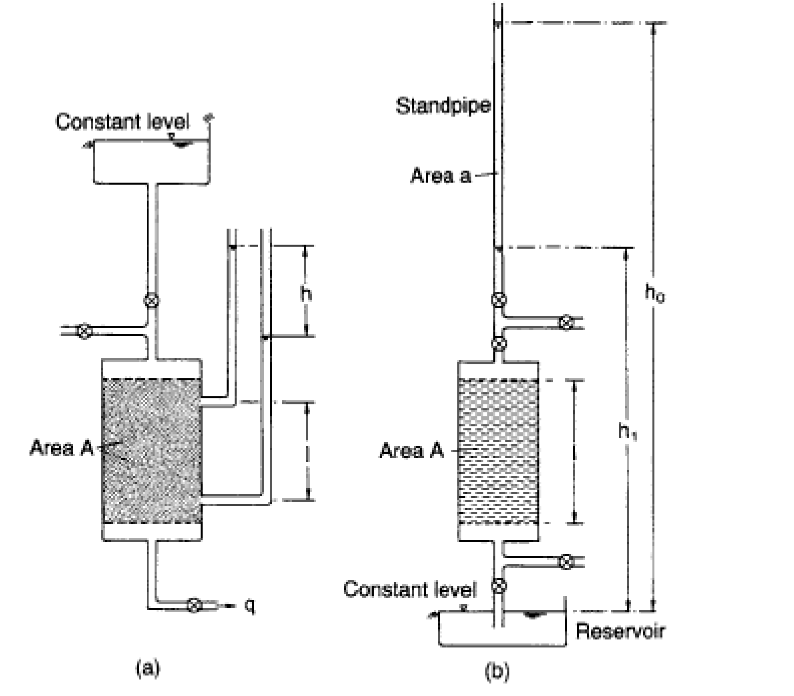
Av: the average area of voids

The porosity, n, can also be expressed as

$$n = \frac{A_v}{A}$$

Measuring K in laboratory

- Two Main Method
  - The coefficient of permeability for coarse soils can be determined by means of the constant-head permeability test
  - For fine soils (clays and silt) the falling-head test should be used



Laboratory permeability tests:
(a) constant head and
(b) falling head.

Measuring K in laboratory

(a) constant head and

$$k = \frac{ql}{Ah}$$

(b) falling head.

$$k = \frac{al}{At_1}\ln\frac{h_0}{h_1} = 2.3\frac{al}{At_1}\log\frac{h_0}{h_1}$$

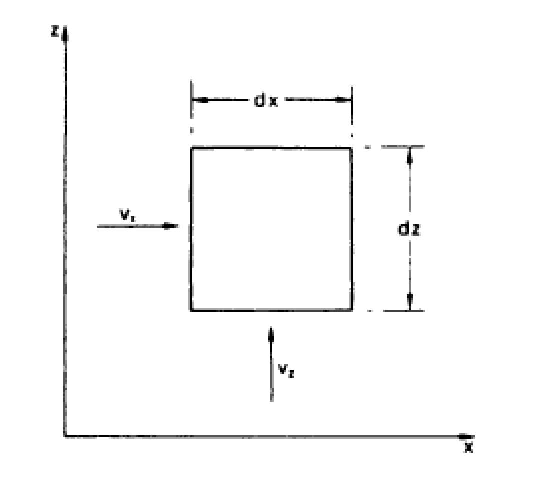
For Clean Uniform sands, Hazen showed that the approximate value of k is given by

$$k = 10^{-2} D_{10}^2 \quad (\mathrm{m/s})$$

where D_{10} is the effective size in mm.

Seepage Theory

- The general case of seepage in two dimensions will now be considered
- Assumption
  - soil is homogeneous and isotropic
  - Generalized Darcy Law will be used



Derivation

$$v_x = k i_x = -k \frac{\partial h}{\partial x}$$ **(1)**

$$v_z = k i_z = -k \frac{\partial h}{\partial z}$$

total head h decreasing in the directions of v_x and v_z

$q_{in} = q_{out}$ (2)

$$v_x\,dy\,dz + v_z\,dx\,dy = \left(v_x + \frac{\partial v_x}{\partial x}dx\right)dy\,dz + \left(v_z + \frac{\partial v_z}{\partial z}dz\right)dx\,dy$$

$$\frac{\partial v_x}{\partial x} + \frac{\partial v_z}{\partial z} = 0$$ (3) **equation of continuity in two dimensions.**

Solution

- Two ⊥ Functions satisfy Laplace Equation
  - First function $\phi(x, z)$, called the potential function,
  - Second function $\psi(x, z)$, called the flow function,

$$\frac{\partial \phi}{\partial x} = v_x = -k\frac{\partial h}{\partial x}$$

$$\frac{\partial \phi}{\partial z} = v_z = -k\frac{\partial h}{\partial z}$$

⟹ **In Eq 3**

$$\frac{\partial^2 \phi}{\partial x^2} + \frac{\partial^2 \phi}{\partial z^2} = 0$$

$$-\frac{\partial \psi}{\partial x} = v_z = -k\frac{\partial h}{\partial z}$$

$$\frac{\partial \psi}{\partial z} = v_x = -k\frac{\partial h}{\partial x}$$

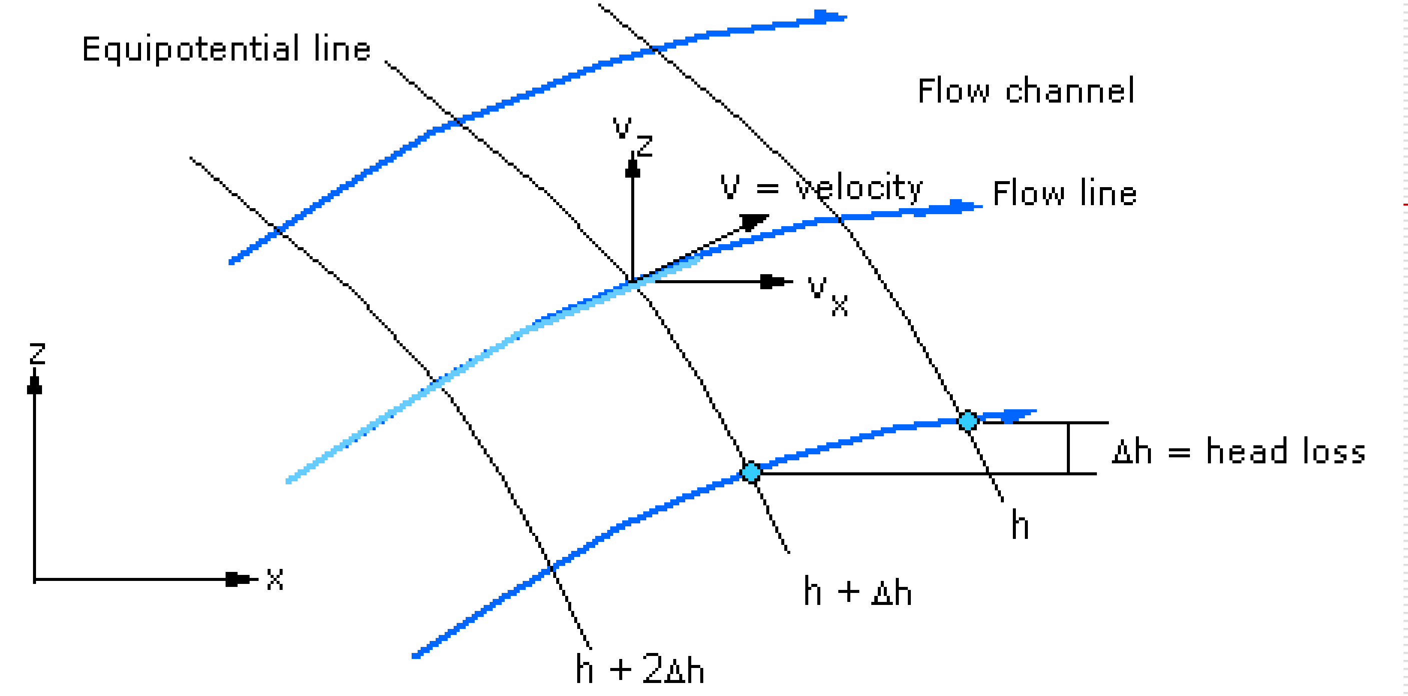
If the function $\psi(x, z)$ is given a constant value ψ_1 then $d\psi=0$ and

$$\frac{dz}{dx} = \frac{v_z}{v_x}$$

If $\phi(x, z)$ is constant then $d\phi = 0$ and

$$\frac{dz}{dx} = -\frac{v_x}{v_z}$$

Flow Net



A flow net is a graphical representation of a flow field and comprises a family of flow lines and equipotential lines. The flow terms are:

1. Flow lines or streamlines represent flow paths of particles of water.
2. The area between two flow lines is called a flow channel.
3. The rate of flow in a flow channel is constant.
4. Flow cannot occur across flow lines.
5. An equipotential line is a line joining points with the same head.
6. The velocity of flow is normal to the equipotential line.
7. The difference in head between two equipotential lines is called the potential drop or head loss.

Constrained for Flow Net

CONSTRAINTS FOR SKETCHING FLOW NET

A flow net must satisfy the following criteria.

1. Flow conditions at entrances and exits.

2. Flow lines must intersect equipotential lines at right angles.

3. The area between flow lines and equipotential lines must be curvilinear squares. A curvilinear square has the property that an inscribed circle can be drawn to touch each side of the square and continuous bisection results, in the limit, to a point.

4. The quantity of flow through each flow channel is constant.

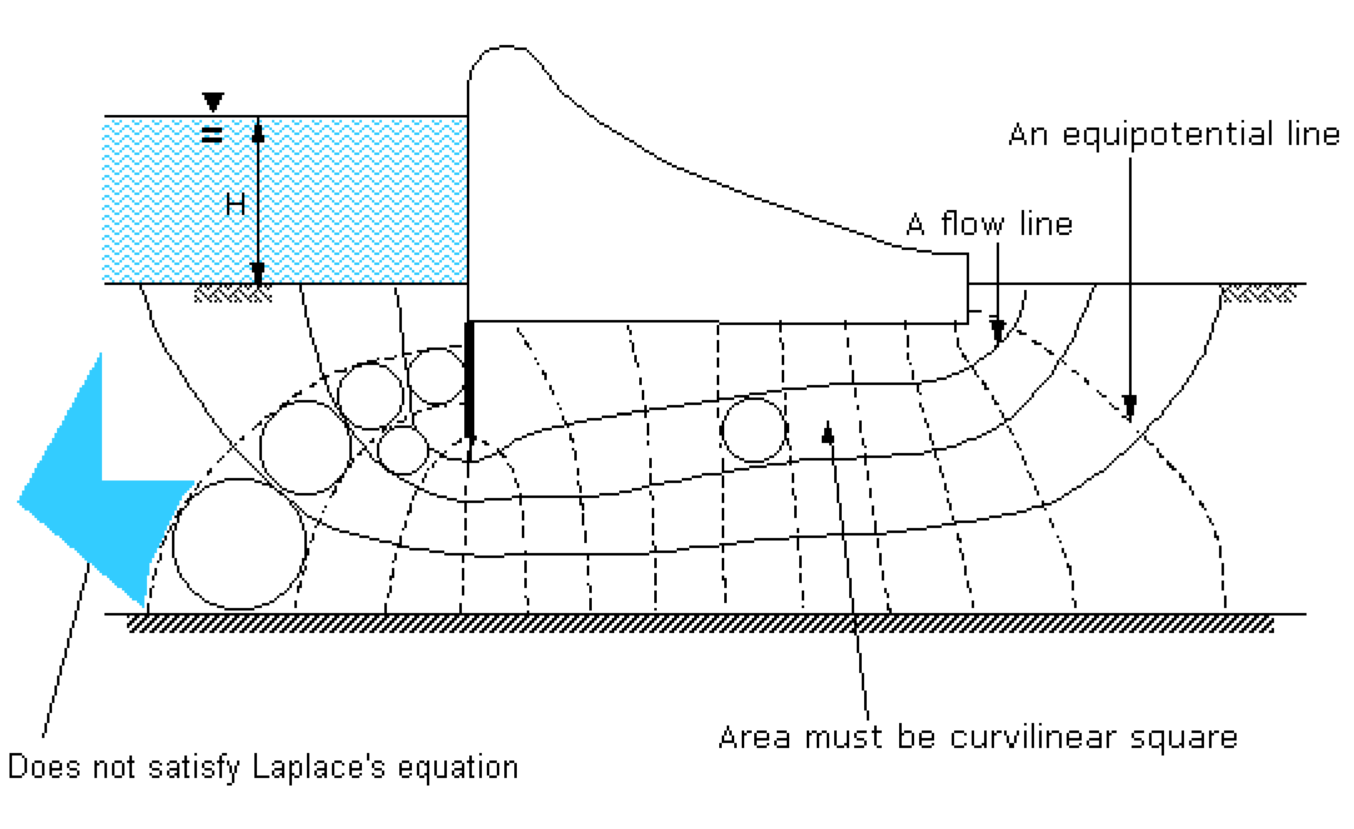
5. The head loss between each consecutive equipotential line is constant.

6. A flow line cannot intersect another flow line.

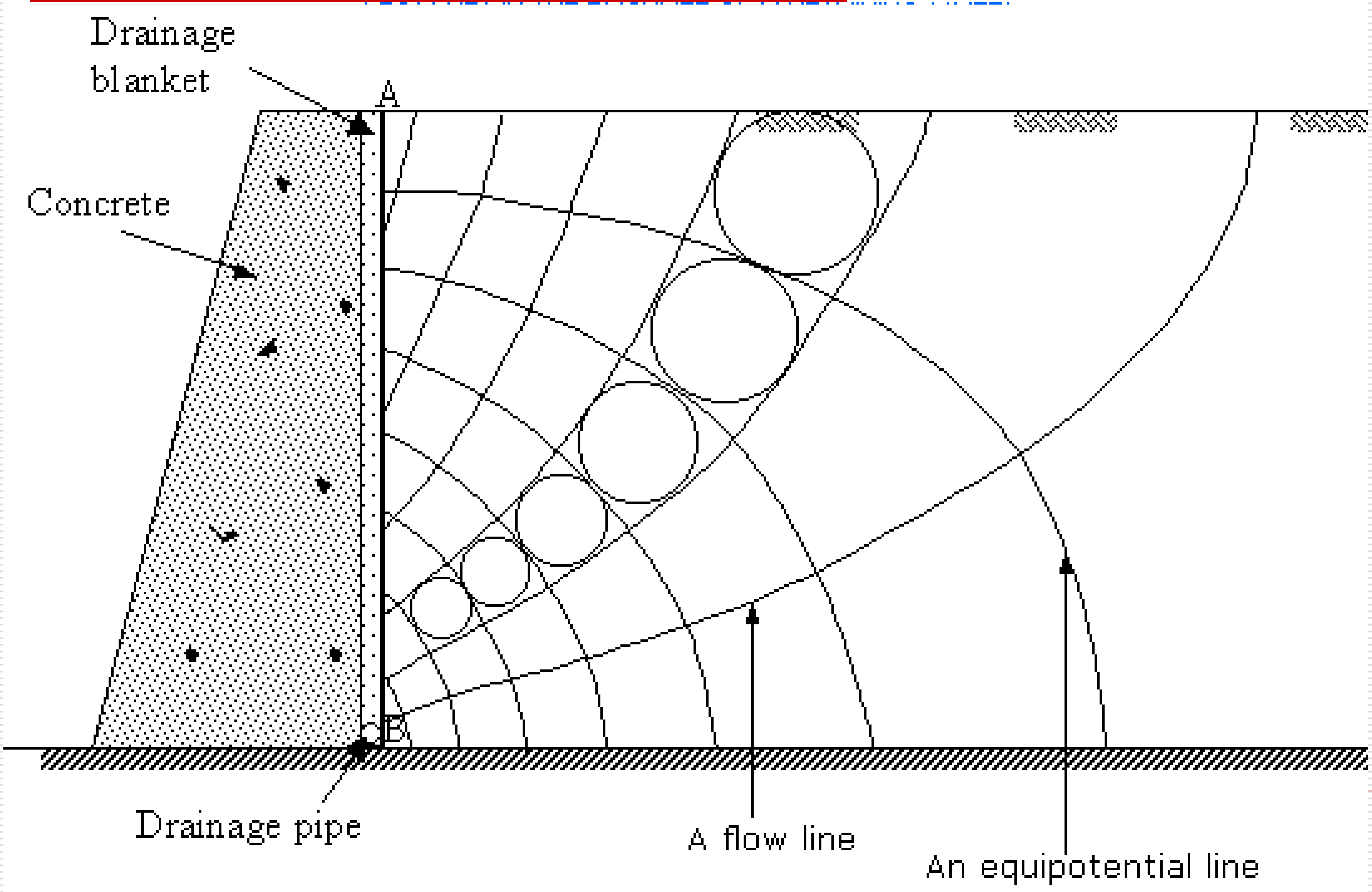
7. An equipotential line cannot intersect another equipotential line.

An infinite number of flow lines and equipotential lines can be drawn to satisfy Laplace's equation. However, only a few are required to obtain an accurate solution. Thus, a very fine mesh may not result in a significant increase in accuracy.

Flow Net under Dam



Flow Net in Backfill of Retaining Wall



INTERPRETATION OF FLOW NET

- Head loss between each consecutive pair of equipotential lines

$$\Delta h = \frac{\Delta H}{N_d}$$

- Flow through each flow channel for an isotropic soil from Darcy's Law

$$\Delta q = Aki = \left(b \times 1\right) k \frac{\Delta h}{L} = k\Delta h \frac{b}{L} = k \frac{\Delta H}{N_d} \frac{b}{L}$$

- Total flow

$$q = k \sum_{i=1}^{N_f} \left(\frac{\Delta H}{N_d}\right)_i = k\Delta H \frac{N_f}{N_d}$$

Hydraulic Gradient

- Hydraulic gradient over each square

$$i = \frac{\Delta h}{L}$$

- Maximum hydraulic gradient

$$i_{max} = \frac{\Delta h}{L_{min}}$$

- Critical Hydraulic Gradient
  - Critical hydraulic gradient that brings a soil mass to static liquefaction, , Heaving, Boiling, and Piping

$$i = i_{cr} = \frac{\gamma'}{\gamma_w} = \left(\frac{G_s - 1}{1 + e}\right)\frac{\gamma_w}{\gamma_w} = \frac{G_s - 1}{1 + e}$$

- **Safe if $i < i_{critical} \Rightarrow F.S = i_{critical}/i_{exit} \geq 1.0$**

Pore Water Pressure Distribution

- **Pressure head**

$$\left(h_p\right)_j = \Delta H - (N_d)_j \Delta h - h_z$$

- **Pore water pressure**

$$u_j = \left(h_p\right)_j \gamma_w$$

- **Uplift Forces**

$$P_w = \sum_{j=1}^{n} u_j \Delta x_j$$

- **Calculating the uplift force per unit length using Simpson's rule**

$$P_w = \frac{\Delta x}{3}\left(u_1 + u_n + 2\sum_{\substack{i=3 \\ \text{odd}}}^{n} u_i + 4\sum_{\substack{i=2 \\ \text{even}}}^{n} u_i \right)$$

ANISOTROPIC SOIL CONDITIONS

- Most natural soil deposits are anisotropic, with the coefficient of permeability having a maximum value in the direction of stratification and a minimum value in the direction normal to that of stratification; these directions are denoted by x and z, respectively, i.e.

$$k_x = k_{\max} \quad \text{and} \quad k_z = k_{\min}$$

- Same solution but you need to have x_t instead of x and K' instead of K in flow equation.

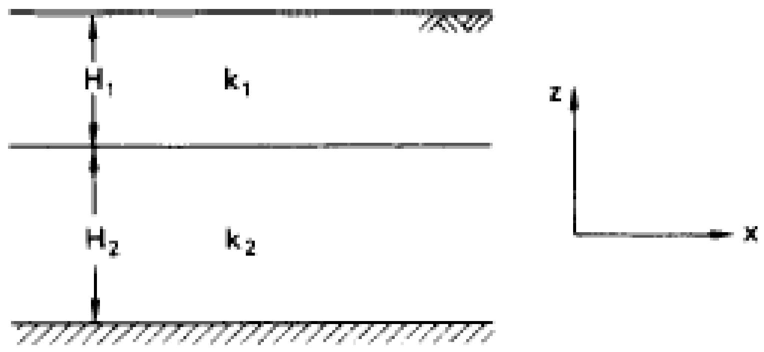
$$x_t = x\sqrt{\frac{k_z}{k_x}}$$

$$k' = k_x\sqrt{\frac{k_z}{k_x}} = \sqrt{(k_x k_z)}$$

Non-homogeneous Soil Conditions

- For horizontal flow, the head drop Dh over the same flow path length H1+H2 will be the same for each layer.

$$\bar{k}_x = \frac{H_1 k_1 + H_2 k_2}{H_1 + H_2}$$



- For vertical flow, the flow rate q through area A of each layer is the same.

$$\therefore \ \bar{k}_z = \frac{H_1 + H_2}{\left(\frac{H_1}{k_1}\right) + \left(\frac{H_2}{k_2}\right)}$$

Flow Nets Example

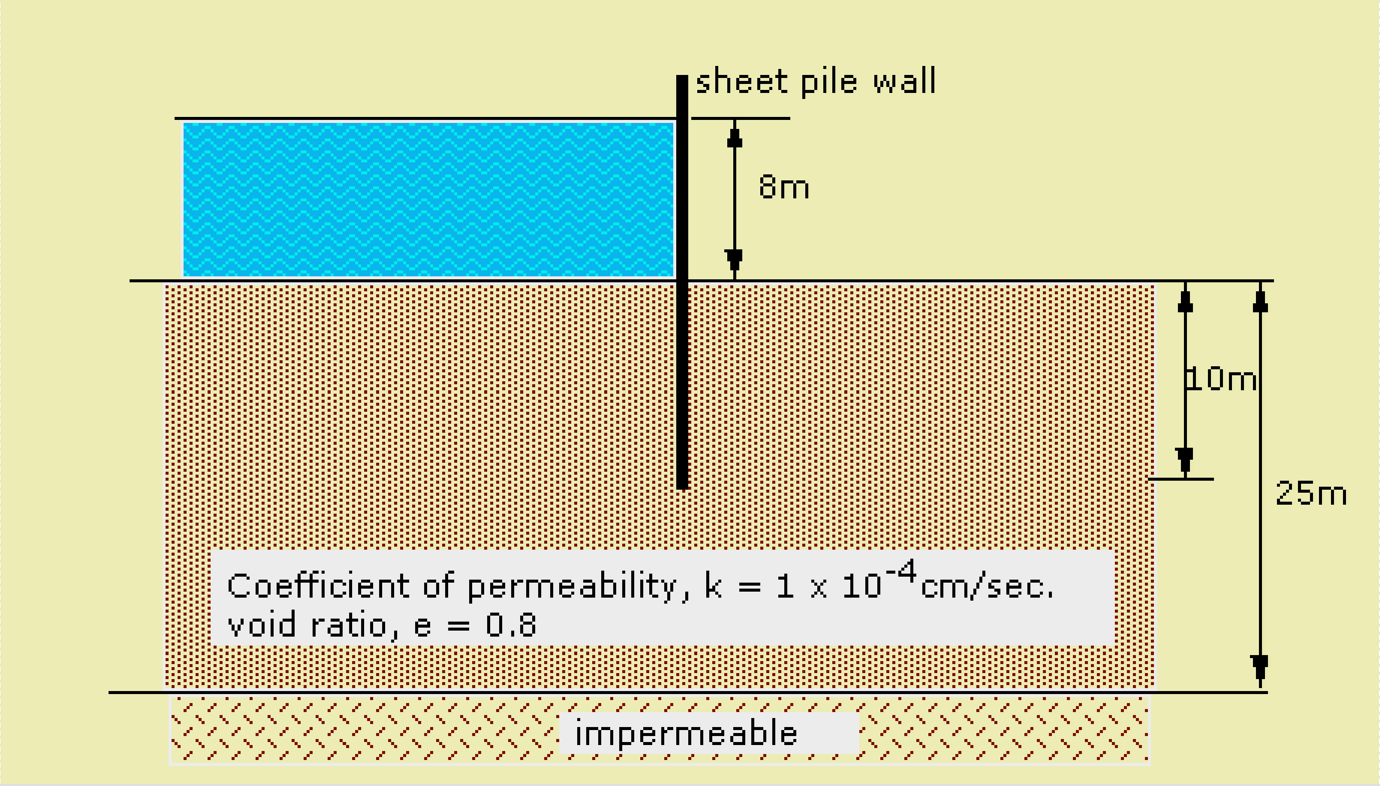
Omar H. Al-Hattamleh
The Hashemite University,
Zarqa, Jordan

Outlines

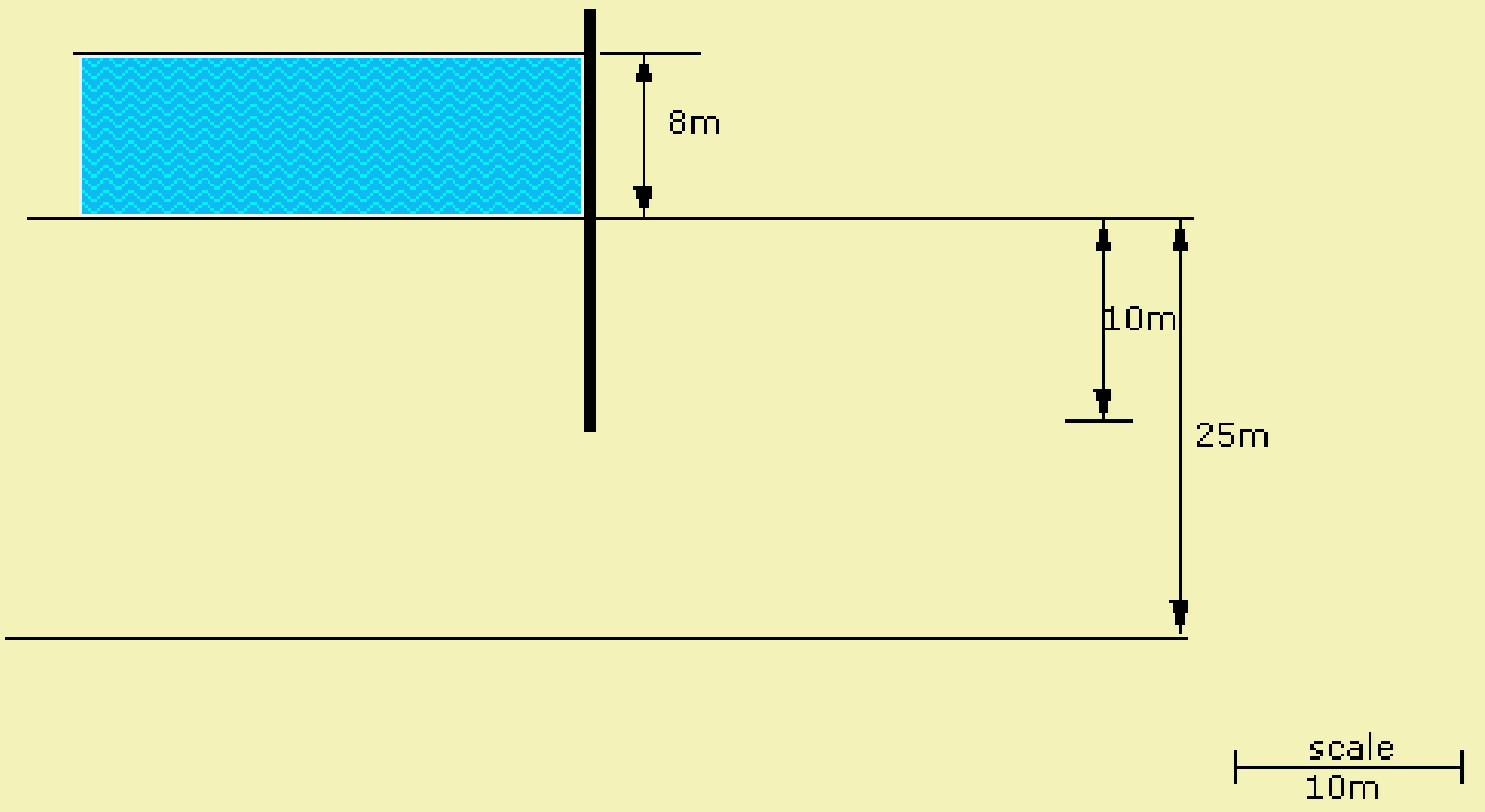
- Sheet pile Problem
- Procedure to get the drawing
- Compute no of flow and drops
- Compute the uplift pressure and uplift force

Problem

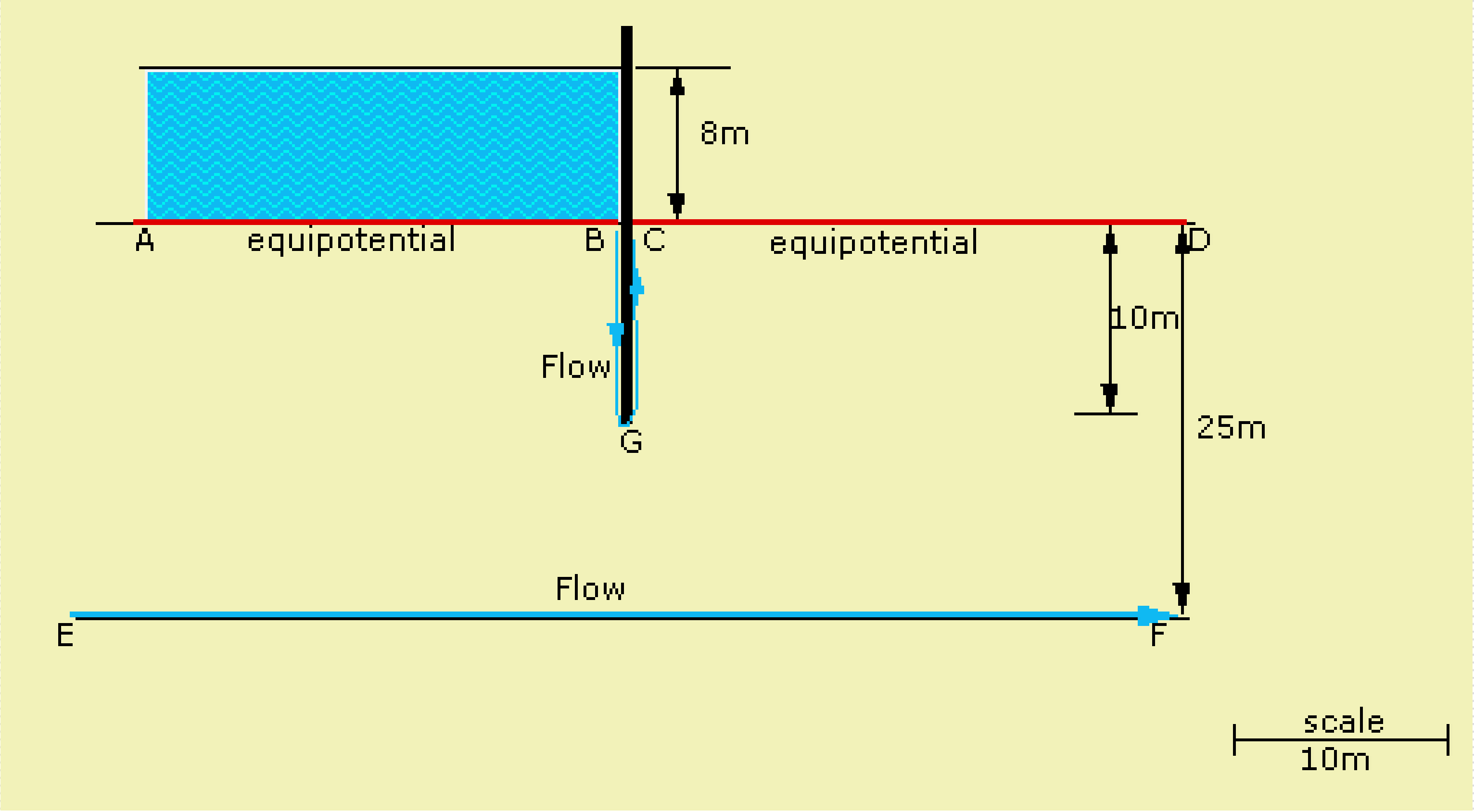
TWO DIMENSIONAL FLOW OF WATER THROUGH A SHEET PILE WALL



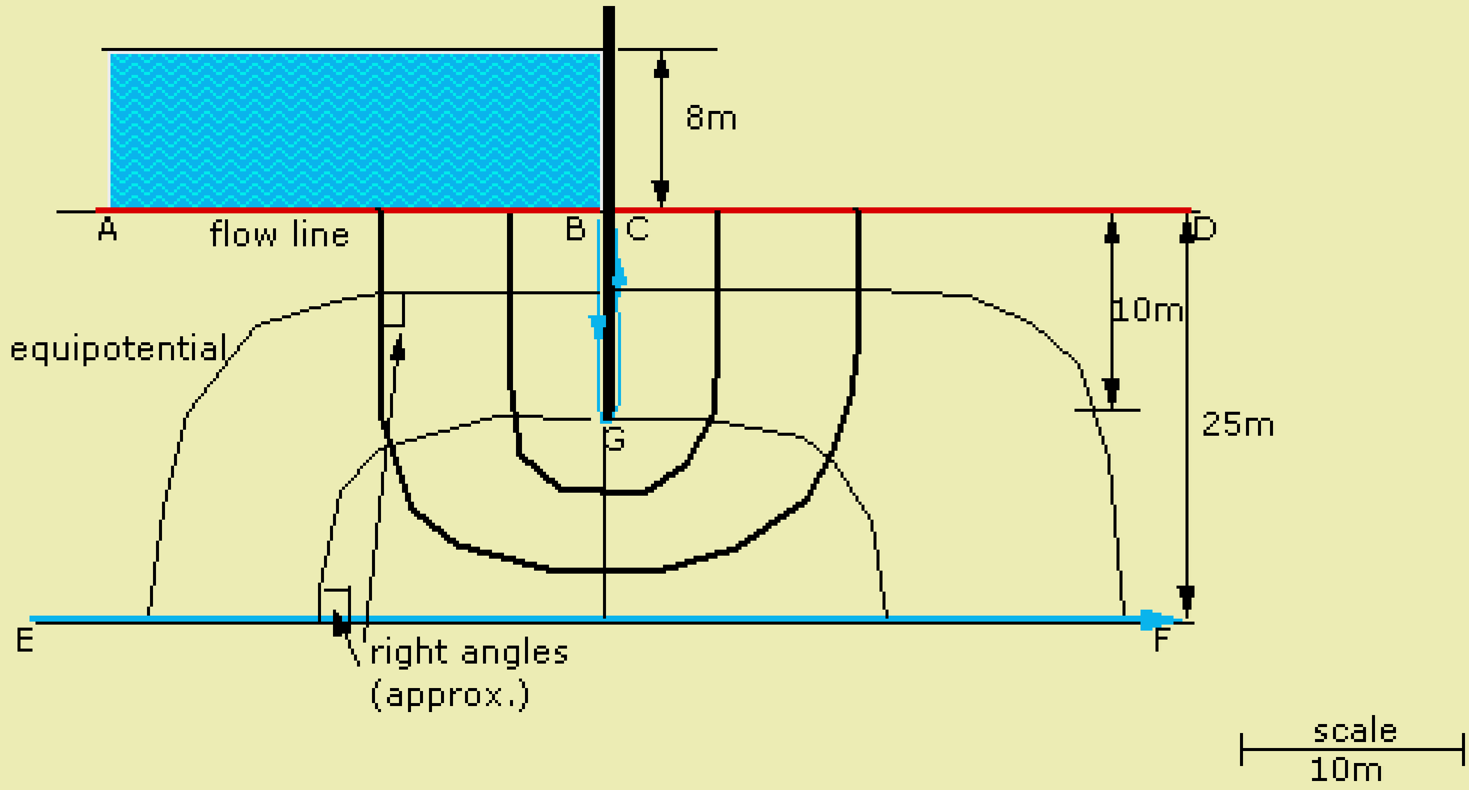
Step 1: Draw the sheet pile wall and soil mass to a suitable scale as shown.



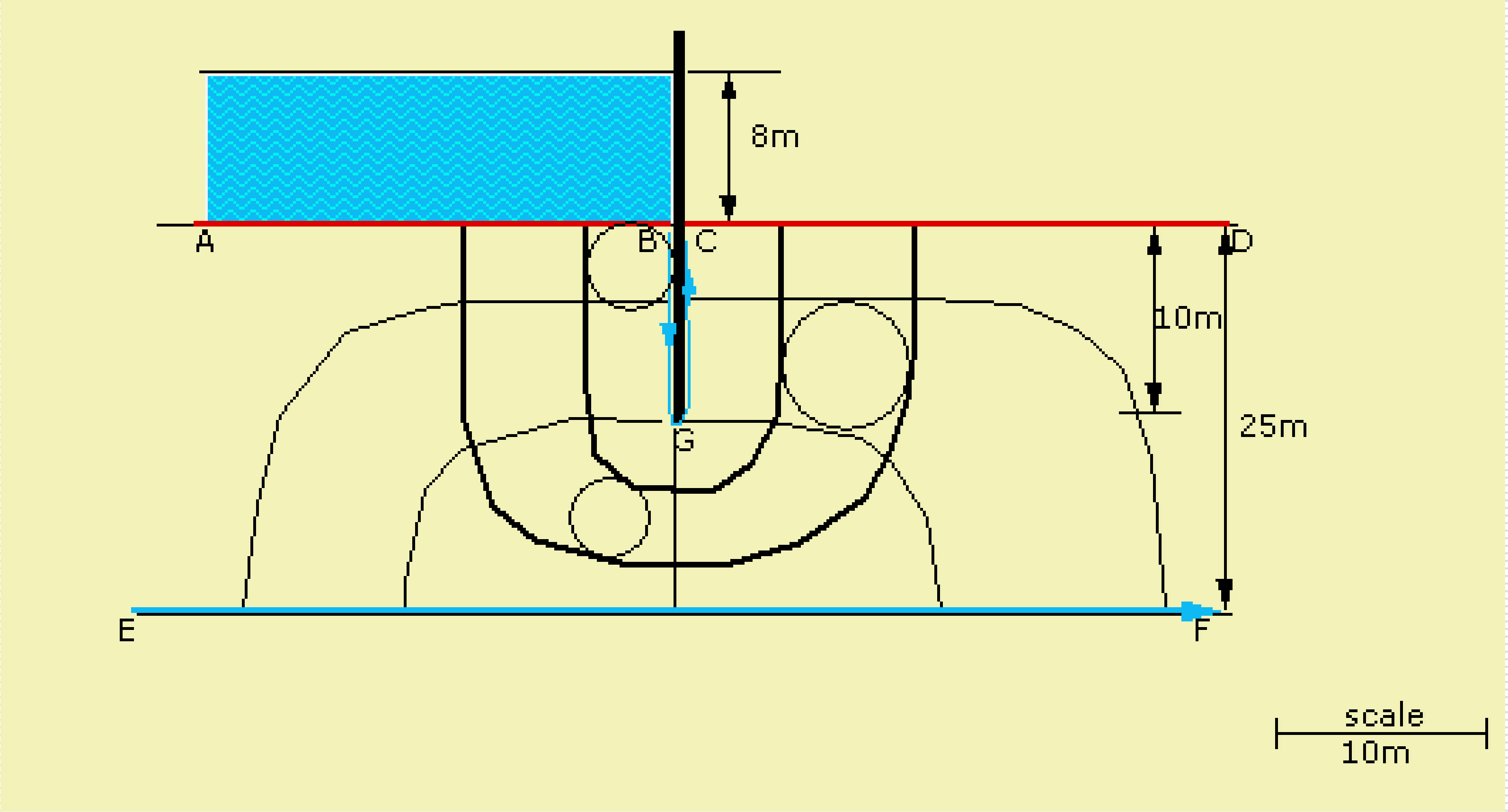
Step 2: Identify equipotential and flow boundaries. AB and CD are equipotential boundaries because the heads at each point on each of these boundaries are equal; the head along AB is 8m and the head along CD is 0m. EF and BGC are flow lines because particles of water can take these paths as flow takes place from the upstream to the downstream side of the wall. Flow lines follow impervious boundaries.



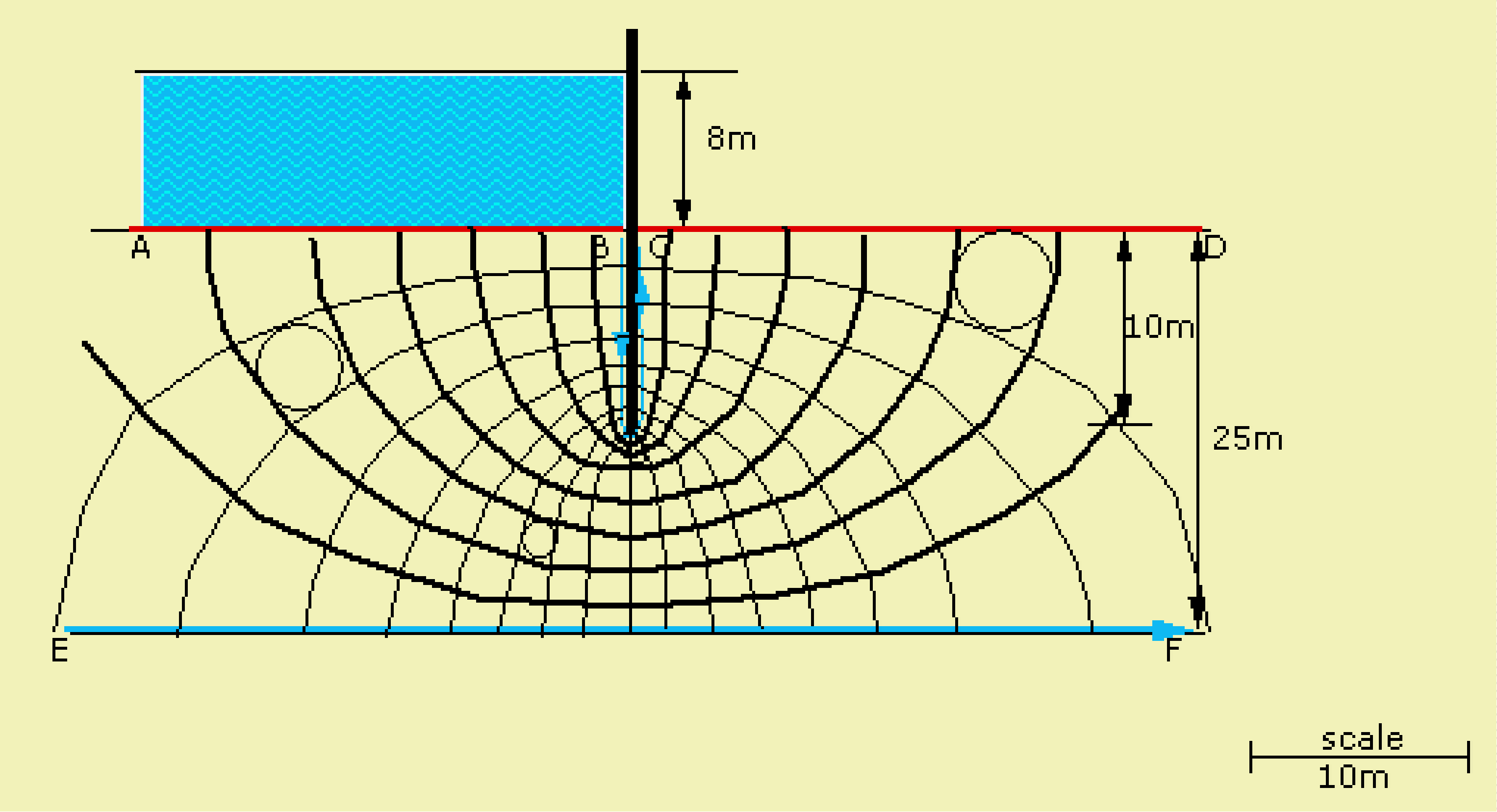
Step 3: Sketch in a few flow lines and equipotential lines subjected to the following constraints. (1) Flow lines cannot intersect each other (2) Equipotential lines cannot intersect each other (3) Flow lines and equipotential lines intersect at right angles (approximately) and (4) The flow net must be comprised of curvilinear squares.



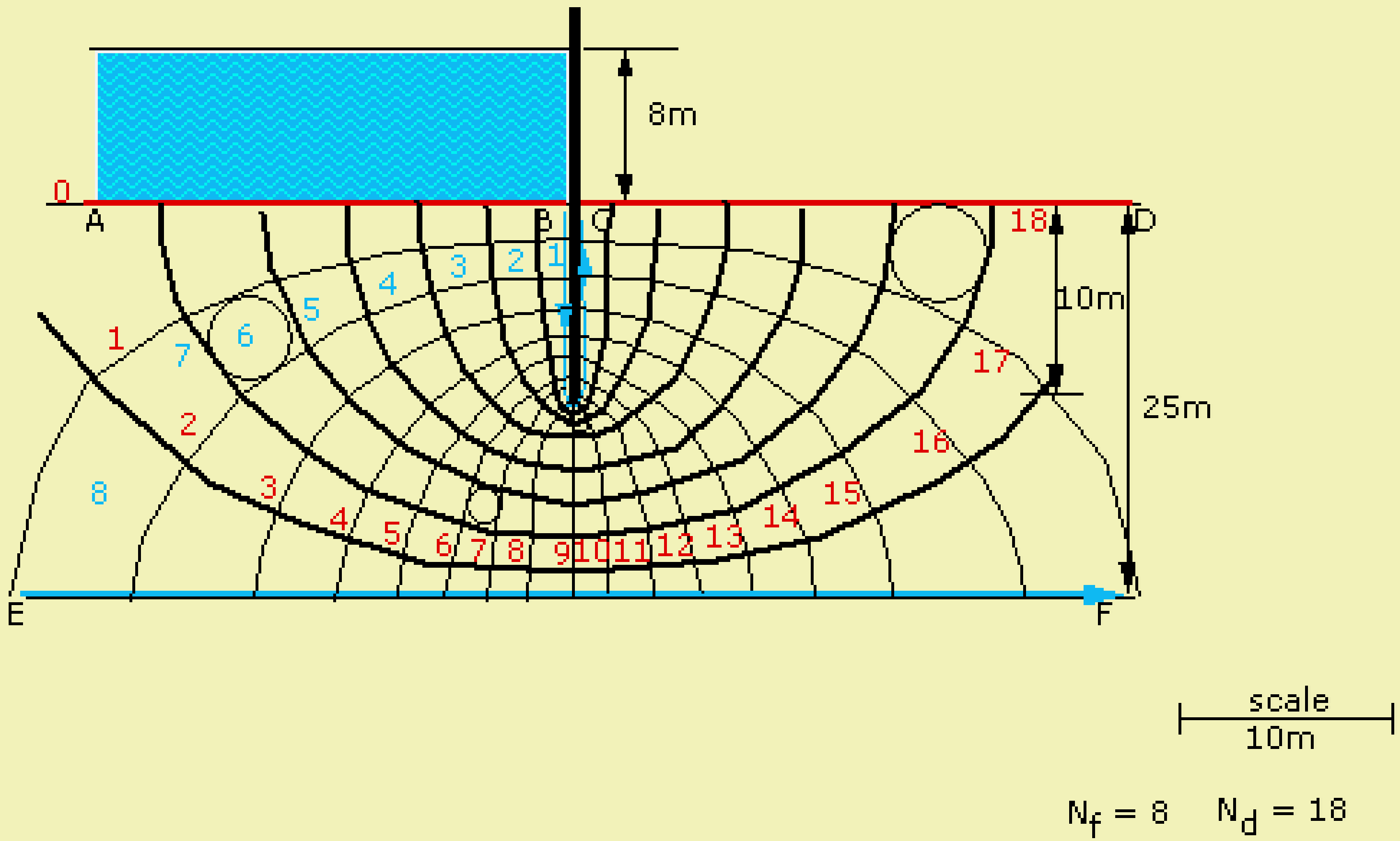
Step 4: Check for curvilinear squares by inscribing a circle in each square. Obviously, this is a poorly drawn flow net. You must now manipulate or redraw the flow net by adding or erasing flow and equipotential lines to satisfy the flow net constraints. In this case, we will have to add more flow lines and consequently we will get more equipotential lines. You will have to erase your flow net and start over.



After repeated trials, you should get a reasonably accurate flow net as shown. This flow net could be refined further. We could have exploited vertical symmetry in this problem and drawn the flow net for only the left half of the flow domain. You should consider this in problems where symmetry exists.

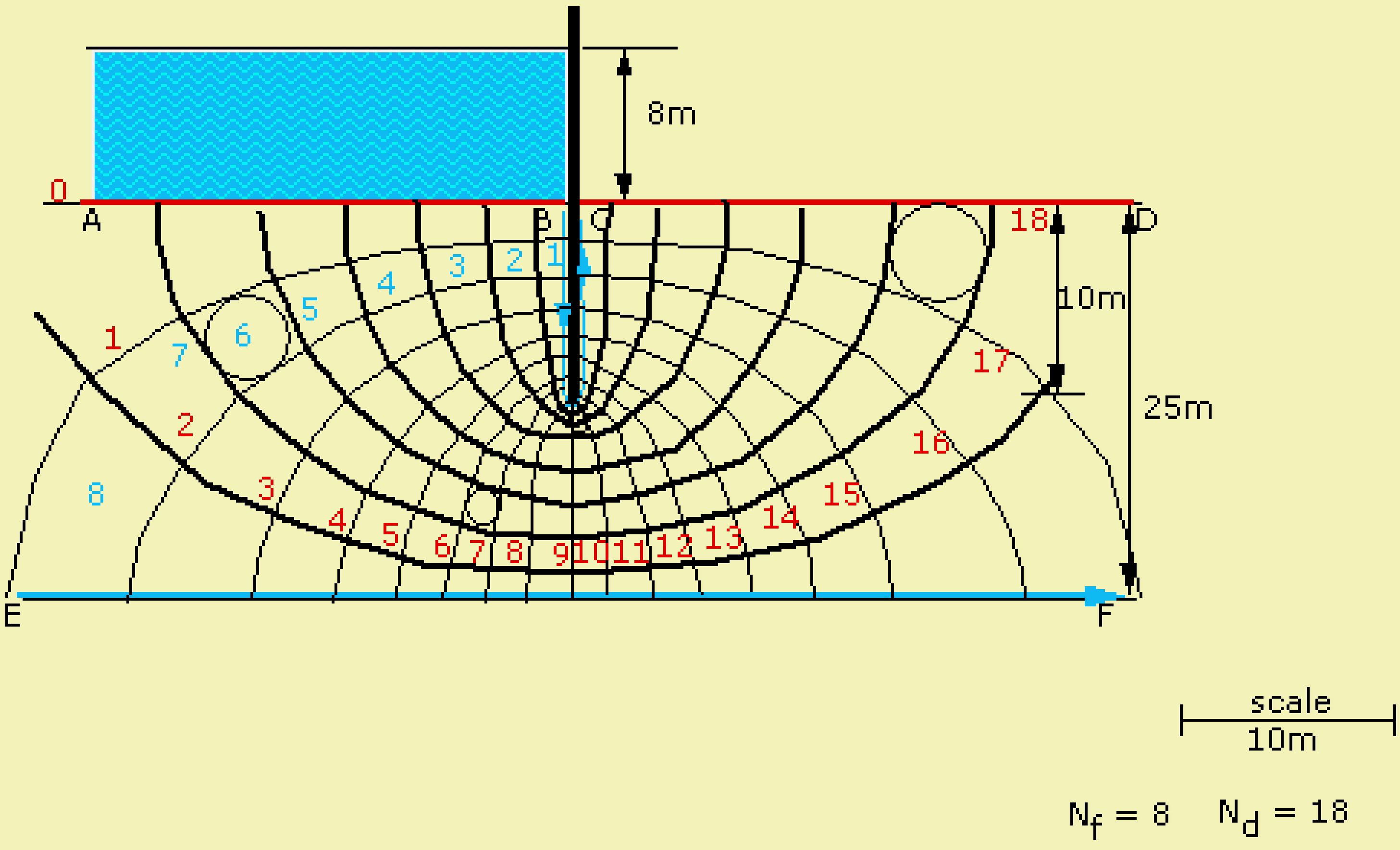


DETERMINATION OF FLOW CHANNELS AND EQUIPOTENTIAL DROPS



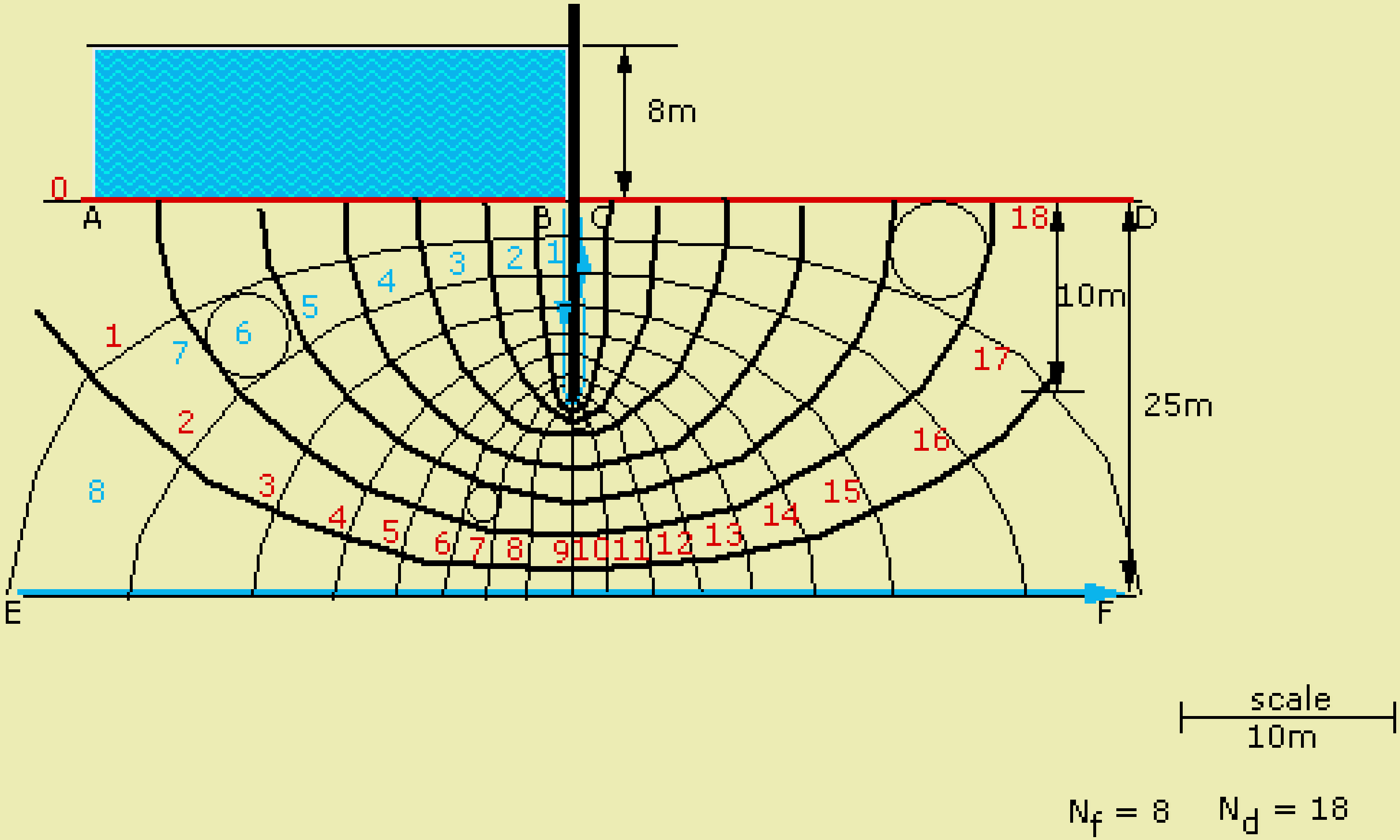
CALCULATION OF HEAD LOSS

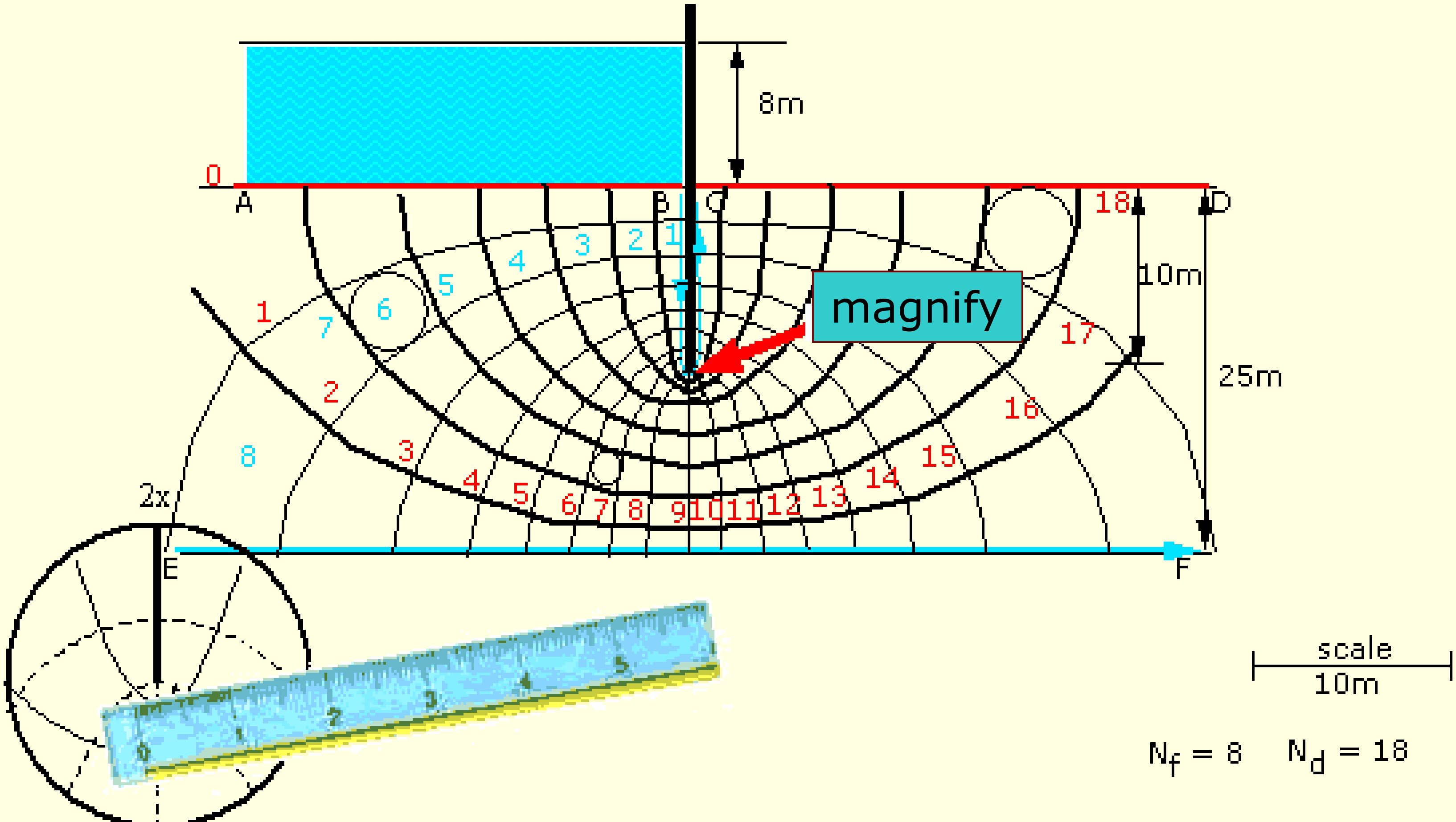
The amount of head loss from the upstream to the downstrean end is $\Delta H = 8m$. Our flow net construction is based on a uniform head loss along each flow channel. That is, the head loss over each equipotential drop is $\Delta h = \Delta H/N_d = 8/18 = 4/9$



CALCULATION OF FLOW

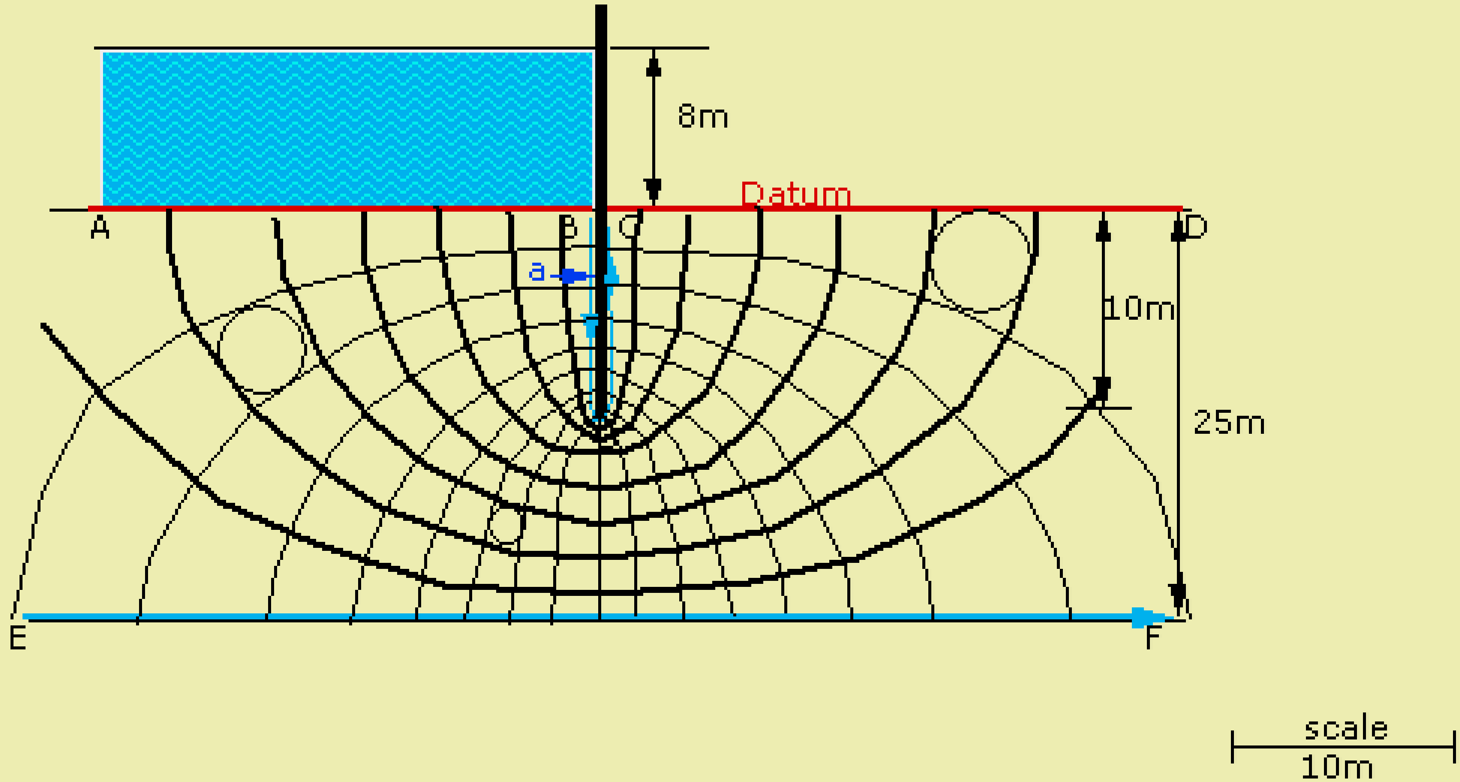
The flow through the soil is $q = k\Delta H N_f/N_d = k\Delta h N_f =$ $1x10^{-4}x8x(8/18)=3.56x10^{-4}cm^3/sec/m$





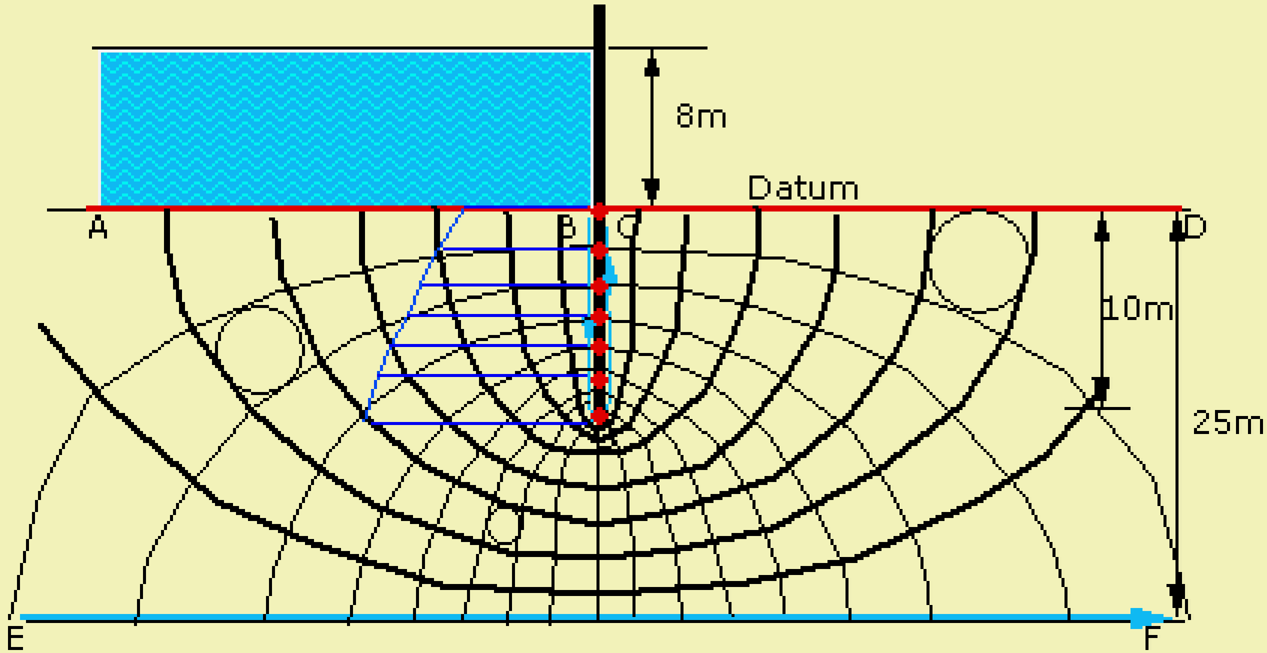
DETERMINATION OF CRITICAL HYDRAULIC GRADIENT

The maximum hydraulic gradient is $i_{max} = \Delta h/L_{min}$. We will now have to determine the minimum length of an equipotential drop (L_{min}). Usually, L_{min} occurs at exits. In our case, L_{min} occurs at the exit at the base of the sheet pile. Click at the base to expand the region near it by a factor of 2. By measurement using a scale, L_{min} = 0.6m; this is the average distance between the equipotential lines in cell near the base. Therefore, i_{max} = (4/9)/ 0.6 = 0.74



CALCULATION OF PORE WATER PRESSURES

Let us now calculate the pore water pressures at any point, say a, near the sheet pile wall. Firstly, we need to define a datum. Let us take the downstream end as datum. The elevation head, measured from the flow net, at a is h_z = -3.4m. The head loss at a =1.75Δh = 1.75 * 4/9 = 0.78m. Note that you can have frational head loss. The pore water pressure head is $\Delta H - 1.75\Delta h - h_z = 8 - 0.78 - (-3.4) = 10.62$m. The pore water pressure is $10.62\gamma_w = 10.62 \times 9.8 = 104.1$kPa



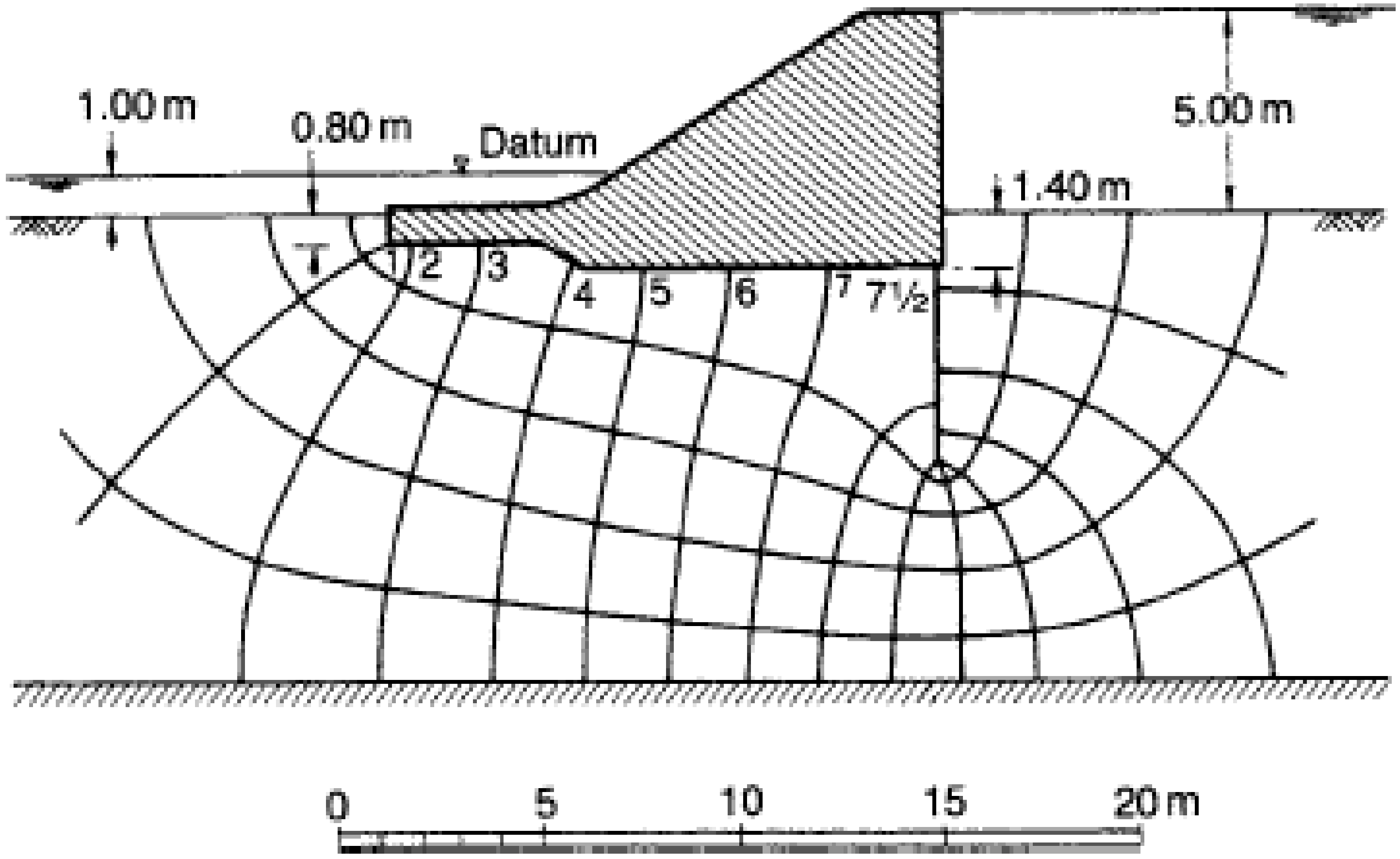
h_z	N_d	$N_d\Delta h$	$h_p = \Delta H - N_d\Delta h - h_z$	$u = h_p\gamma_w$ (kPa)
0	0	0	8	78.4
-2	1	.44	9.56	93.6
-3.8	2	.89	10.91	106.9
-5.2	3	1.33	11.87	116.3
-7	4	1.78	13.22	129.6
-8.4	5.5	2.44	13.96	136.8
-10	8	3.56	14.44	141.6

scale
10m

Let us plot the distribution of pore water pressures on the upstream face of the retaining wall.

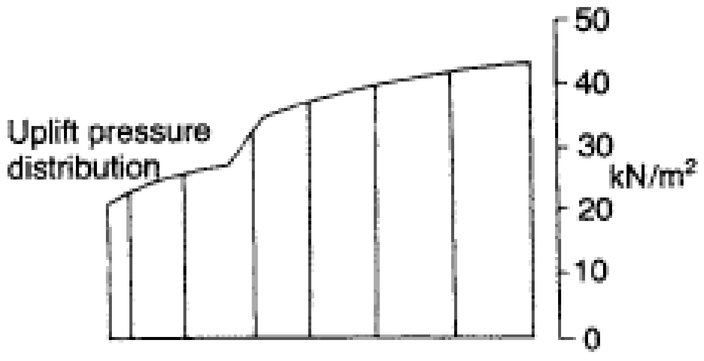
Example 2.2 (text)

- The section through a dam is shown in Figure. Determine the quantity of seepage under the dam and plot the distribution of uplift pressure on the base of the dam. The coefficient of permeability of the foundation soil is 2.5×10^{-5} m/s.



$$q = kh\frac{N_f}{N_d} = 2.5 \times 10^{-5} \times 4.00 \times \frac{4.7}{15}$$
$$= 3.1 \times 10^{-5}\ \mathrm{m^3/s} \quad \text{(per m)}$$

Point	h (m)	z (m)	$h - z$ (m)	$u = \gamma_w(h-z)$ (kN/m²)
1	0.27	−1.80	2.07	20.3
2	0.53	−1.80	2.33	22.9
3	0.80	−1.80	2.60	25.5
4	1.07	−2.10	3.17	31.1
5	1.33	−2.40	3.73	36.6
6	1.60	−2.40	4.00	39.2
7	1.87	−2.40	4.27	41.9
$7\frac{1}{2}$	2.00	−2.40	4.40	43.1



Recommendation problem

- Do the following problem in Chapter Two
  - 2.1
  - 2.2
  - 2.3
  - 2.4
  - 2.5
  - 2.6
  - 2.9
- References Craig Chapter Two Sections 2.1-2.6

Effective Stresses

Omar H. Al-Hattamleh
The Hashemite University,
Zarqa, Jordan

Outlines

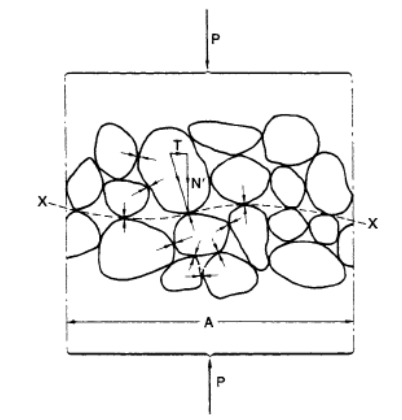
- Introduction
- The principle of effective stress
- Response of effective stress to a change in total stress
- Partially saturated soils
- Influence of seepage on effective stress

Introduction

- A soil can be visualized as a skeleton of solid particles enclosing continuous voids which contain water and/or air.
- For the range of stresses usually encountered in practice the individual solid particles and water can be considered incompressible; air, on the other hand, is highly compressible.
- The volume of the soil skeleton as a whole can change due to rearrangement of the soil particles into new positions, mainly by rolling and sliding, with a corresponding change in the forces acting between particles.
- The actual compressibility of the soil skeleton will depend on the structural arrangement of the solid particles.
- In a fully saturated soil, since water is considered to be incompressible, a reduction in volume is possible only if some of the water can escape from the voids.
- In a dry or a partially saturated soil a reduction in volume is always possible due to compression of the air in the voids, provided there is scope for particle rearrangement.

THE PRINCIPLE OF EFFECTIVE STRESS

- Effective stress: the forces transmitted through the soil skeleton from particle to particle was recognized in 1923 By Terzaghi
- The the principle applies only to fully saturated soils and relates the following three stresses:
  1. The total normal stress (σ) on a plane within the soil mass, being the force per unit area transmitted in a normal direction across the plane
  2. The pore water pressure (u), being the pressure of the water filling the void space between the solid particles;
  3. The effective normal stress (σ') on the plane, representing the stress transmitted through the soil skeleton only.



$$\sigma = \sigma' + u$$

Effective vertical stress due to self-weight of soil (overburden pressure)

- The total vertical stress (i.e. the total normal stress on a horizontal plane) at depth z is equal to the weight of all material (solids + water) per unit area above that depth, i.e.

$$\sigma_v = \gamma_{sat} z$$

- The pore water pressure (static or hydrostatic) at any depth will be hydrostatic since the void space between the solid particles is continuous, so at depth z

$$u = \gamma_w z$$

- The effective vertical stress at depth z will be

$$\sigma'_v = \sigma_v - u = (\gamma_{sat} - \gamma_w)z = \gamma' z$$

where γ' is the buoyant unit weight of the soil.

Component of Pore water pressure

- If the soil subjected to seepage or to the load the pore water pressure (pwp). At any time during drainage the overall pore water pressure (u) is equal to the sum of the static and excess components, i.e.

$$u = u_s + u_e$$

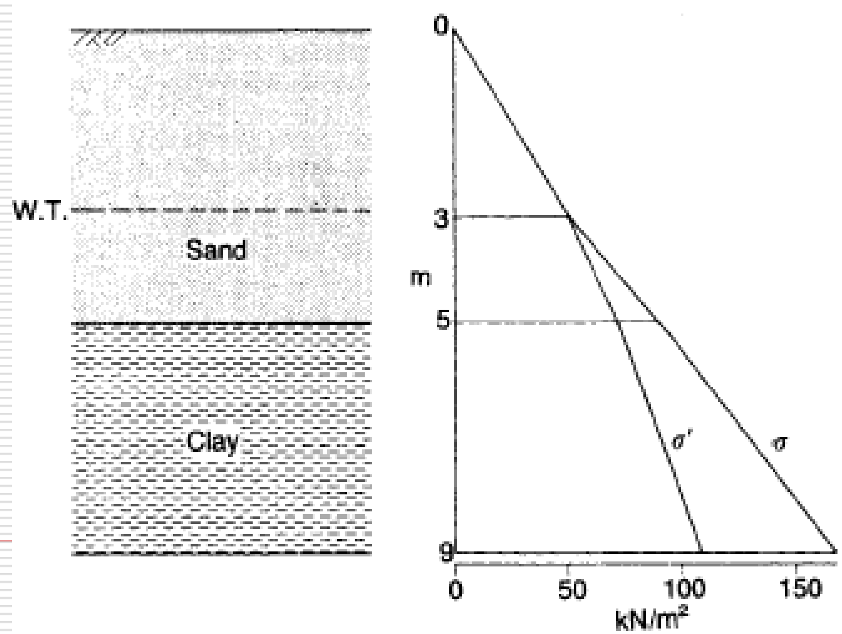
- The reduction of excess pore water pressure as drainage takes place is described as dissipation and when this has been completed (i.e. when ue= 0) the soil is said to be in the drained condition.
- Prior to dissipation, with the excess pore water pressure at its initial value, the soil is said to be in the undrained condition.
- It should be noted that the term ‘drained’ does not mean that all water has flowed out of the soil pores: it means that there is no stress-induced pressure in the pore water. The soil remains fully saturated throughout the process of dissipation.

Example

- A layer of saturated clay 4m thick is overlain by sand 5m deep, the water table being 3m below the surface. The saturated unit weights of the clay and sand are 19 and 20 kN/m^3, respectively; above the water table the unit weight of the sand is 17 kN/m^3. Plot the values of total vertical stress and effective vertical stress against depth. If sand to a height of 1m above the water table is saturated with capillary water, how are the above stresses affected?

Depth (m)	σ_v (kN/m^2)	
3	3×17	$= 51.0$
5	$(3 \times 17) + (2 \times 20)$	$= 91.0$
9	$(3 \times 17) + (2 \times 20) + (4 \times 19)$	$= 167.0$

u (kN/m^2)	$\sigma'_v = \sigma_v - u$ (kN/m^2)
0	51.0
$2 \times 9.8 = 19.6$	71.4
$6 \times 9.8 = 58.8$	108.2



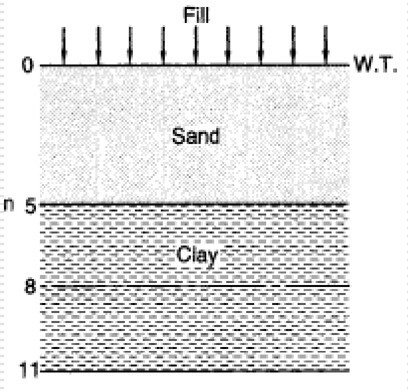
Effect of capillary rise

- What Would happen to σ' If sand to a height of 1m above the water table is saturated with capillary water, how are the above stresses affected?

- Effect of capillary rise The water table is the level at which pore water pressure is atmospheric (i.e. u = 0). Above the water table, water is held under negative pressure and, even if the soil is saturated above the water table, does not contribute to hydrostatic pressure below the water table. The only effect of the 1m capillary rise, therefore, is to increase the total unit weight of the sand between 2 and 3m depth from 17 to 20 kN/m^3, an increase of 3 kN/m^3. Both total and effective vertical stresses below 3m depth are therefore increased by the constant amount 3 x 1 = 3.0 kN/m^2, pore water pressures being unchanged.

Example (from here)

- A 5m depth of sand overlies a 6m layer of clay, the water table being at the surface; the permeability of the clay is very low. The saturated unit weight of the sand is 19 kN/m3 and that of the clay is 20 kN/m3. A 4m depth of fill material of unit weight 20 kN/m3 is placed on the surface over an extensive area. Determine the effective vertical stress at the centre of the clay layer (a) immediately after the fill has been placed, assuming this to take place rapidly and (b) many years after the fill has been placed.



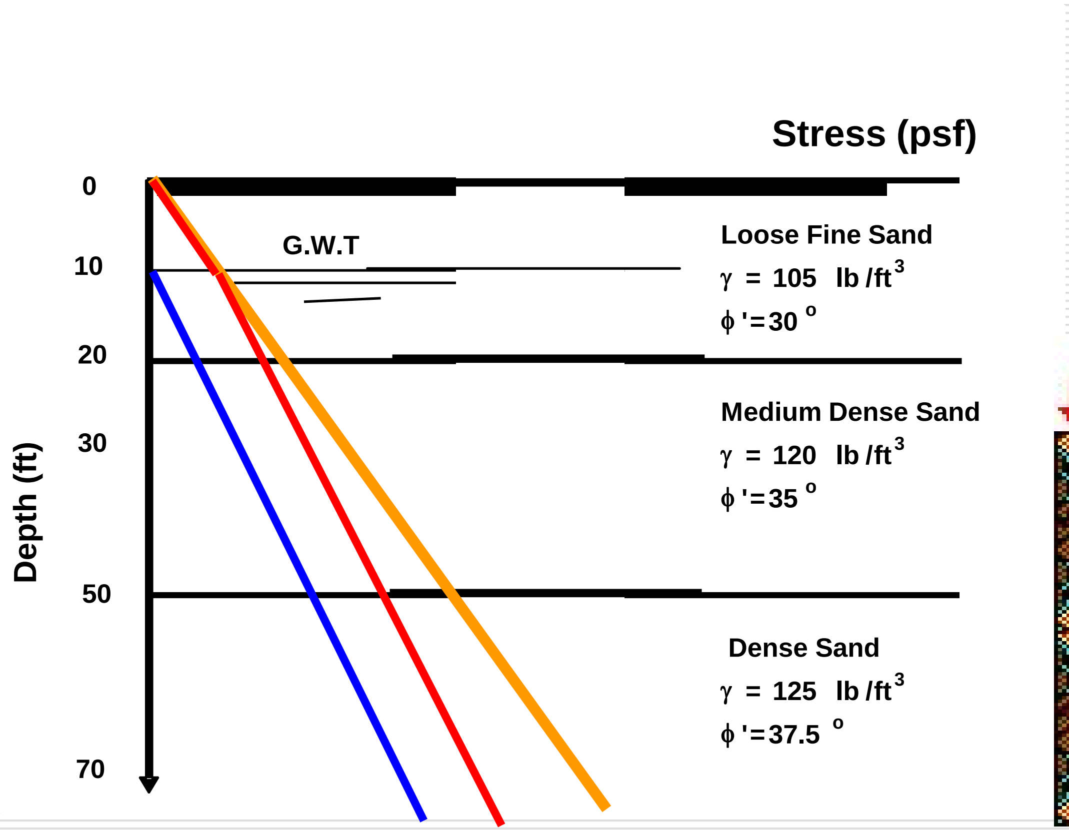
a.

$$\sigma'_v = (5 \times 9.2) + (3 \times 10.2) = 76.6\,\mathrm{kN/m^2}$$

b.

$$\sigma'_v = (4 \times 20) + (5 \times 9.2) + (3 \times 10.2) = 156.6\,\mathrm{kN/m^2}$$

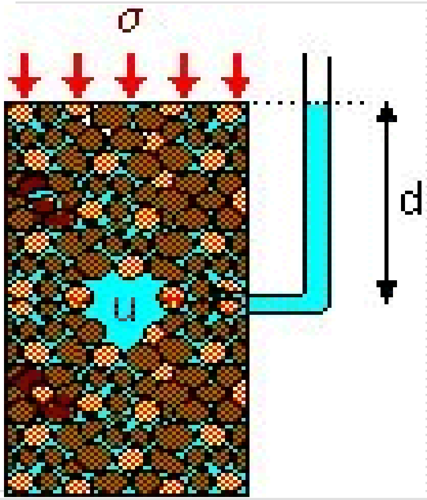
Imperial (B.S.) Unit example



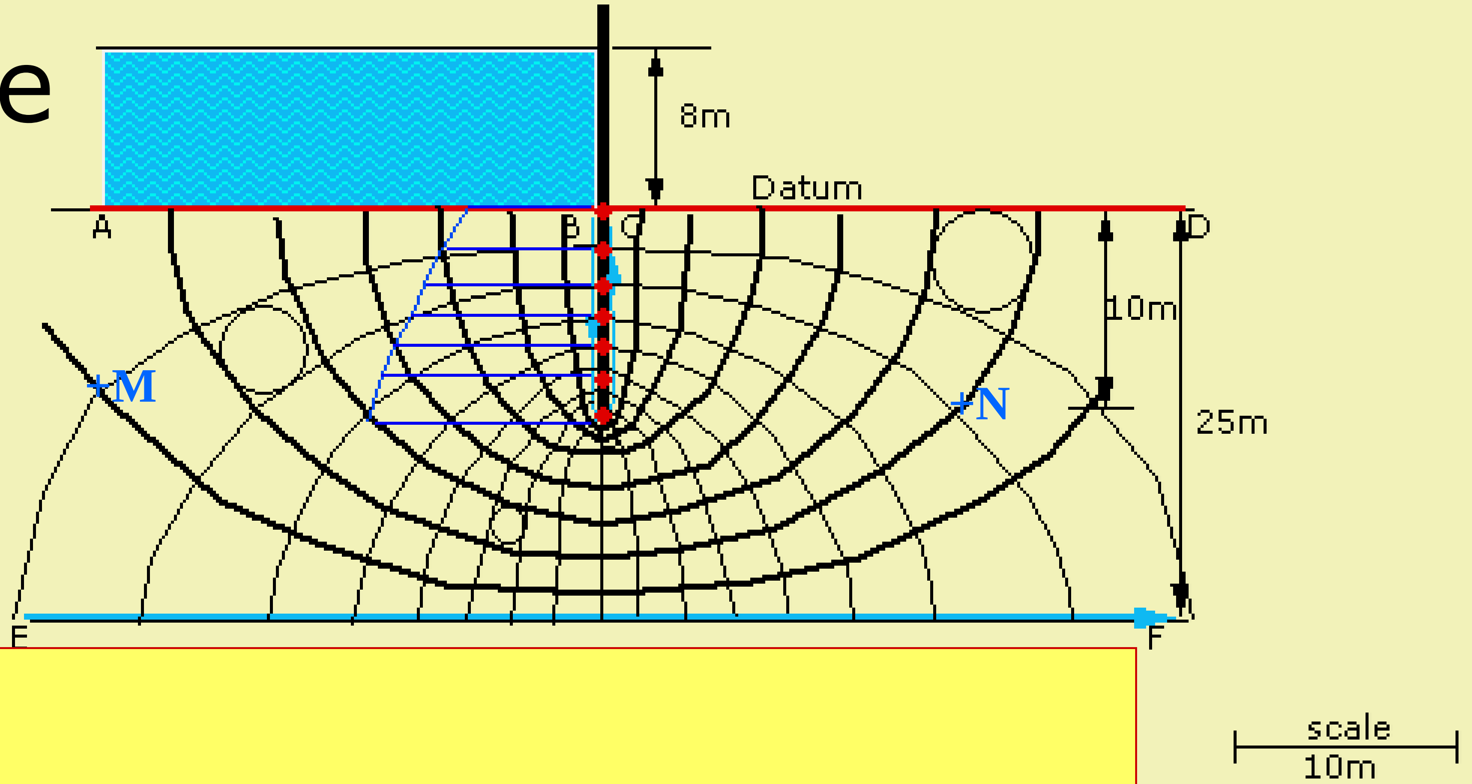
Total Stress

Hydrostatic Stress

Effective Stress



Example



For the sheet pile shown find the σ’in point M and N shown in the figure use γ for soil 20kN/m^3

Stress Increment From Elastic Solution

Omar H. Al-Hattamleh
Hashemite University,
Zarqa, Jo

Class of Year 2017-2018

Outlines

- Point load
- Line load
- Strip Load
- Circular loaded area
- Rectangular loaded area
- Approximate Method 2:1 method

Stresses Caused by a Point Load

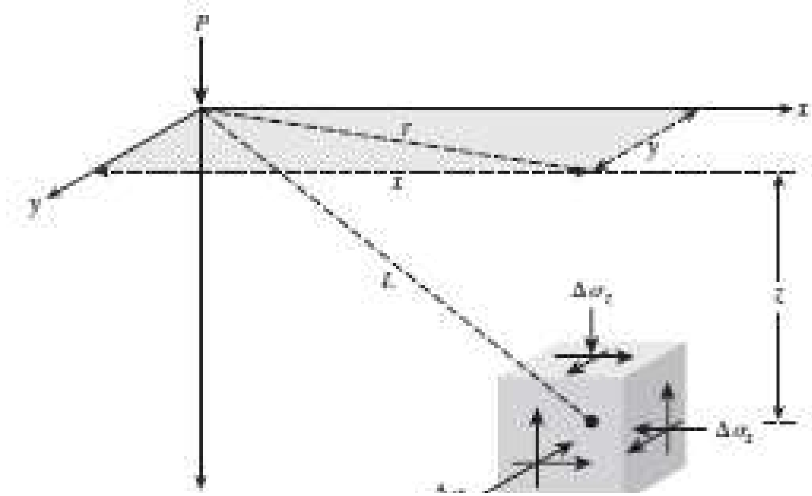


Figure 10.7 Stresses in an elastic medium caused by a

$$\Delta\sigma_z = \frac{3P}{2\pi}\frac{z^3}{L^5} = \frac{3P}{2\pi}\frac{z^3}{(r^2+z^2)^{5/2}}$$

$$\Delta\sigma_z = \frac{P}{z^2}\left\{\frac{3}{2\pi}\frac{1}{[(r/z)^2+1]^{5/2}}\right\} = \frac{P}{z^2}I_1$$

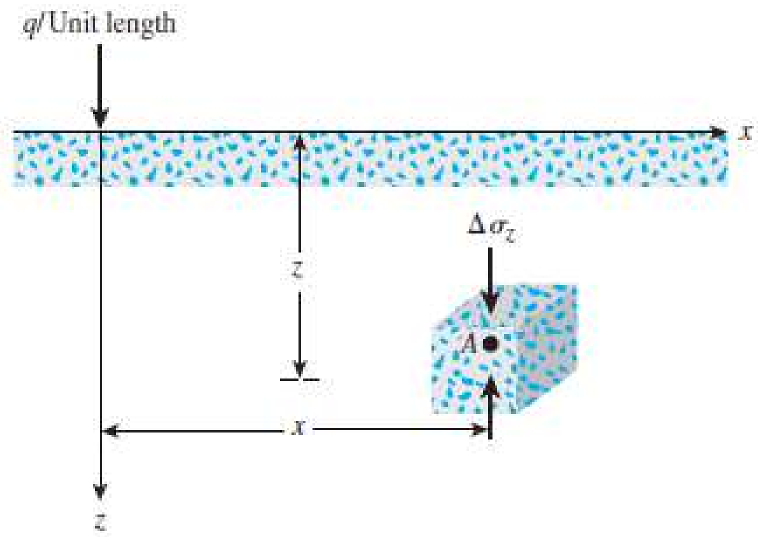
$$I_1 = \frac{3}{2\pi}\frac{1}{[(r/z)^2+1]^{5/2}}$$

Table 10.1 Variation of I_1 for Various Values of r/z [Eq. (10.14)]

r/z	I_1	r/z	I_1	r/z	I_1
0	0.4775	0.36	0.3521	1.80	0.0129
0.02	0.4770	0.38	0.3408	2.00	0.0085
0.04	0.4765	0.40	0.3294	2.20	0.0058
0.06	0.4723	0.45	0.3011	2.40	0.0040
0.08	0.4699	0.50	0.2733	2.60	0.0029
0.10	0.4657	0.55	0.2466	2.80	0.0021
0.12	0.4607	0.60	0.2214	3.00	0.0015
0.14	0.4548	0.65	0.1978	3.20	0.0011
0.16	0.4482	0.70	0.1762	3.40	0.00085
0.18	0.4409	0.75	0.1565	3.60	0.00066
0.20	0.4329	0.80	0.1386	3.80	0.00051
0.22	0.4242	0.85	0.1226	4.00	0.00040
0.24	0.4151	0.90	0.1083	4.20	0.00032
0.26	0.4050	0.95	0.0956	4.40	0.00026
0.28	0.3954	1.00	0.0844	4.60	0.00021
0.30	0.3849	1.20	0.0513	4.80	0.00017
0.32	0.3742	1.40	0.0317	5.00	0.00014
0.34	0.3632	1.60	0.0200		

Vertical Stress Caused by a Vertical Line Load

$$\Delta\sigma_z = \frac{2qz^3}{\pi(x^2 + z^2)^2}$$



$$\frac{\Delta\sigma_z}{(q/z)} = \frac{2}{\pi[(x/z)^2 + 1]^2}$$

Table 10.2 Variation of $\Delta\sigma_z/(q/z)$ with x/z [Eq. (10.16)]

x/z	$\Delta\sigma_z/(q/z)$	x/z	$\Delta\sigma_z/(q/z)$
0	0.637	1.3	0.088
0.1	0.624	1.4	0.073
0.2	0.589	1.5	0.060
0.3	0.536	1.6	0.050
0.4	0.473	1.7	0.042
0.5	0.407	1.8	0.035
0.6	0.344	1.9	0.030
0.7	0.287	2.0	0.025
0.8	0.237	2.2	0.019
0.9	0.194	2.4	0.014
1.0	0.159	2.6	0.011
1.1	0.130	2.8	0.008
1.2	0.107	3.0	0.006

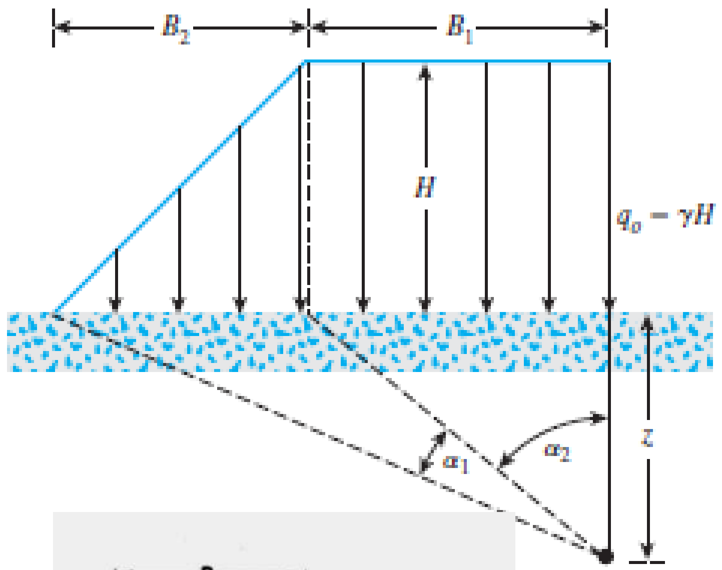
Vertical Stress Due to Embankment Loading

$$\Delta\sigma_z = \frac{q_o}{\pi}\left[\left(\frac{B_1 + B_2}{B_2}\right)(\alpha_1 + \alpha_2) - \frac{B_1}{B_2}(\alpha_2)\right]$$

where $q_o = \gamma H$
γ = unit weight of the embankment soil
H = height of the embankment

$$\alpha_1 \text{ (radians)} = \tan^{-1}\left(\frac{B_1 + B_2}{z}\right) - \tan^{-1}\left(\frac{B_1}{z}\right)$$

$$\alpha_2 = \tan^{-1}\left(\frac{B_1}{z}\right)$$



$$\sigma_z = \frac{q}{\pi}\{\alpha + \sin\alpha\cos(\alpha + 2\beta)\}$$

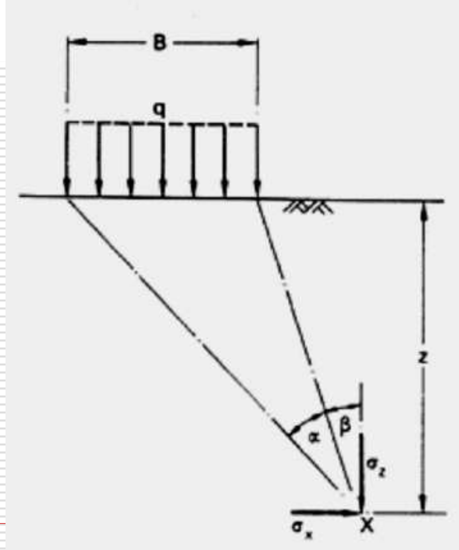
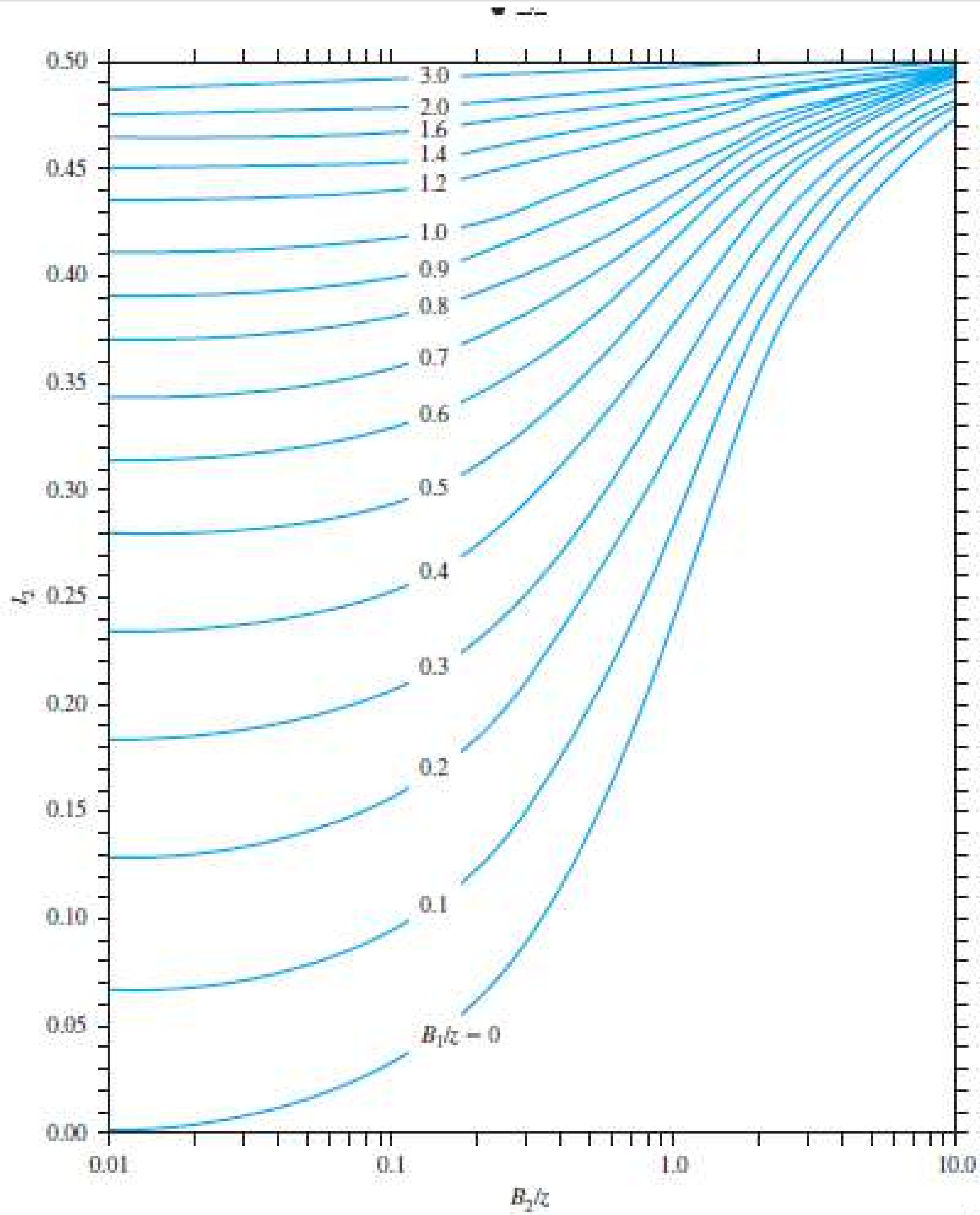


Figure 10.15 Osterberg's chart for determination of vertical stress due to embankment loading



Circular area carrying uniform pressure

$$\sigma_z = q\left[1 - \left\{\frac{1}{1+(R/z)^2}\right\}^{3/2}\right] = qI_c$$

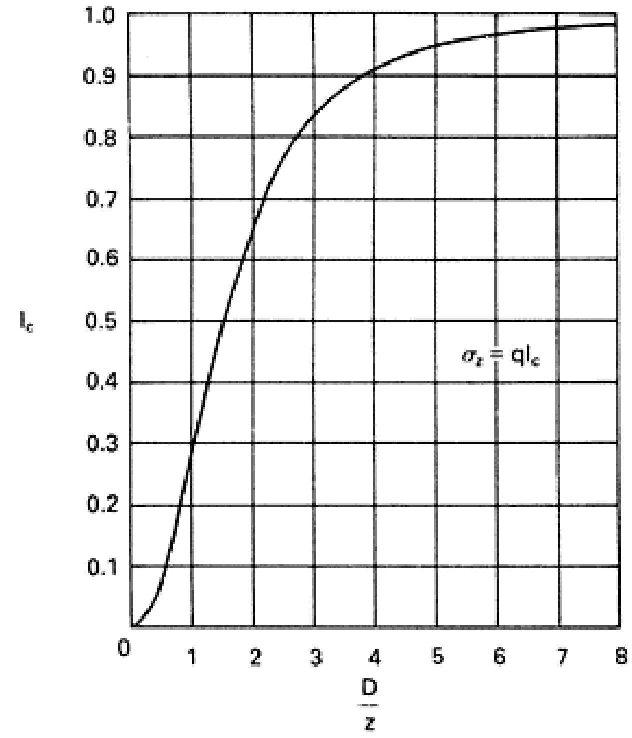


Figure 5.9 Vertical stress under the centre of a circular area carrying a uniform pressure.

Rectangular area carrying uniform pressure

$$\sigma_z = qI_r$$

$$\Delta\sigma_z = \int d\sigma_z = \int_{y=0}^{B}\int_{x=0}^{L} \frac{3qz^3(dx\,dy)}{2\pi(x^2+y^2+z^2)^{5/2}} = qI_3$$

$$I_3 = \frac{1}{4\pi}\left[\frac{2mn\sqrt{m^2+n^2+1}}{m^2+n^2+m^2n^2+1}\left(\frac{m^2+n^2+2}{m^2+n^2+1}\right) + \tan^{-1}\left(\frac{2mn\sqrt{m^2+n^2+1}}{m^2+n^2-m^2n^2+1}\right)\right]$$

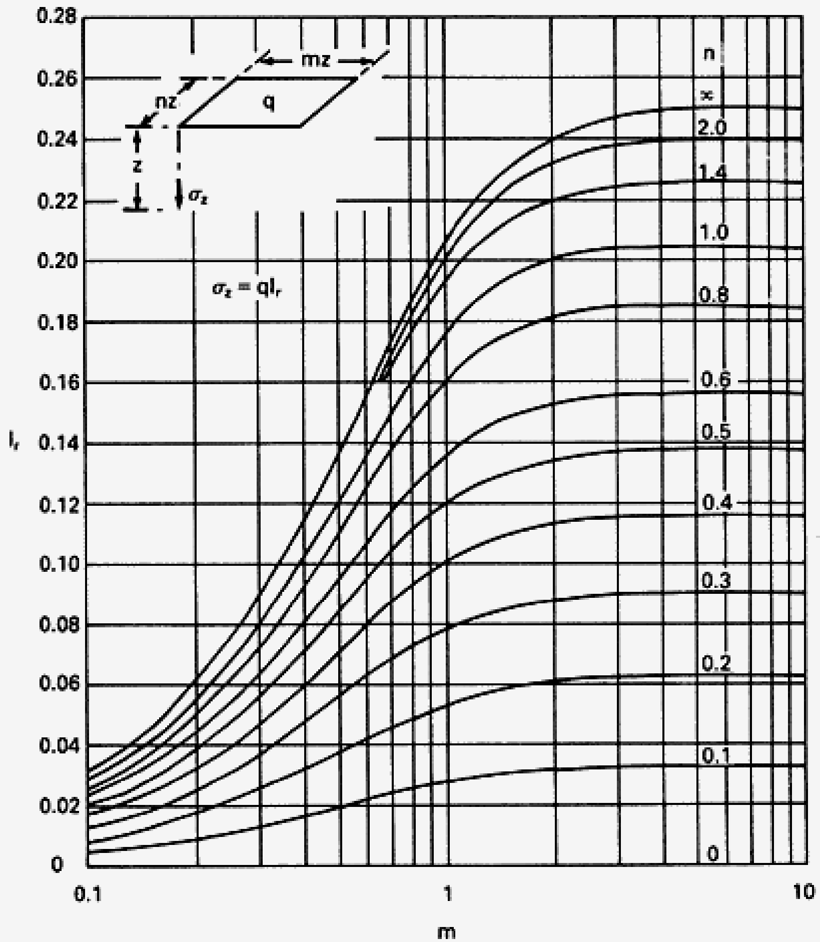


Figure 5.10 Vertical stress under a corner of a rectangular area carrying a uniform pressure. (Reproduced from

Approximate Method 2:1 method

Strip Load Width B

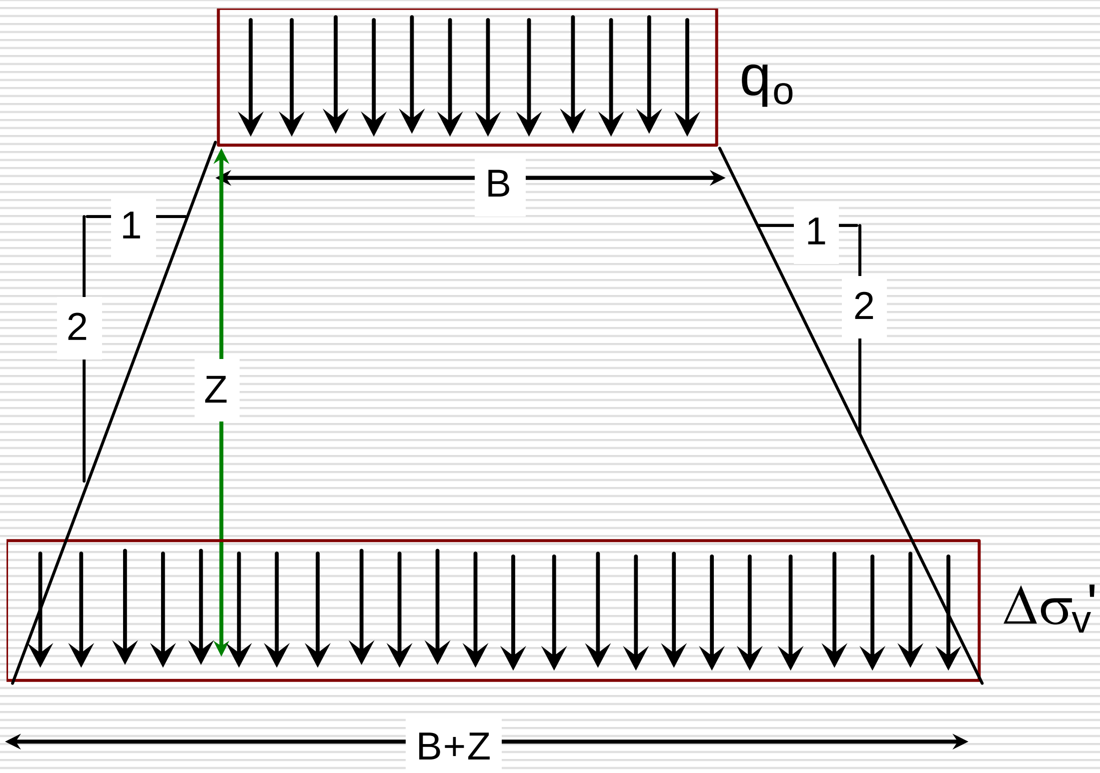
$$\Delta\sigma'_v = \frac{q_o(Bx1)}{(B+Z)}$$

Square Load Width B

$$\Delta\sigma'_v = \frac{q_o(BxB)}{(B+Z)(B+Z)}$$

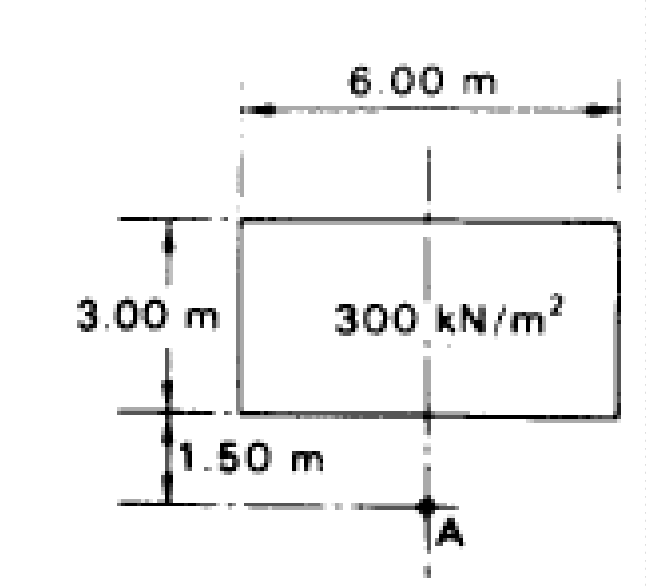
Rectangular Load B X L

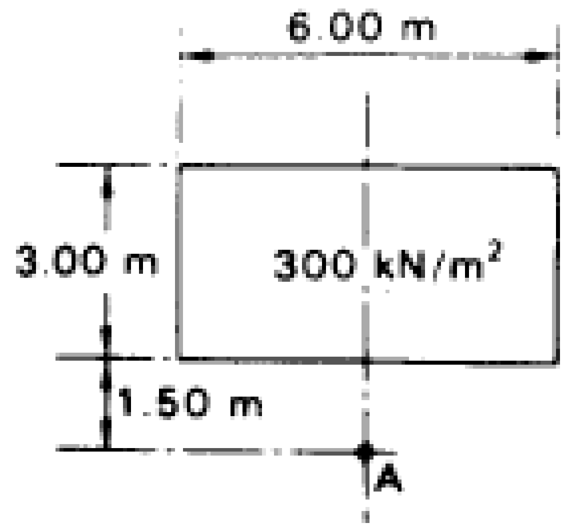
$$\Delta\sigma'_v = \frac{q_o(BxL)}{(B+Z)(L+Z)}$$

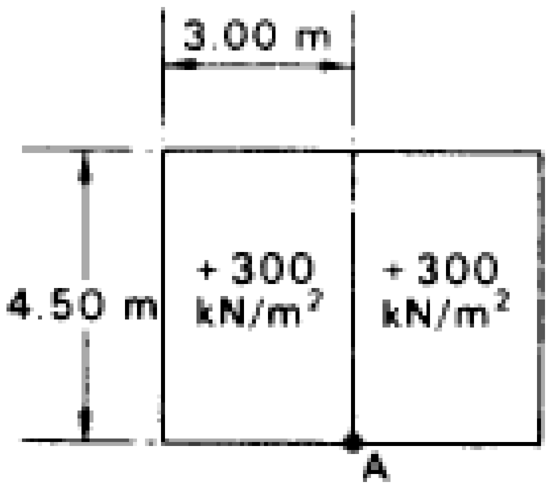


EXAMPLE

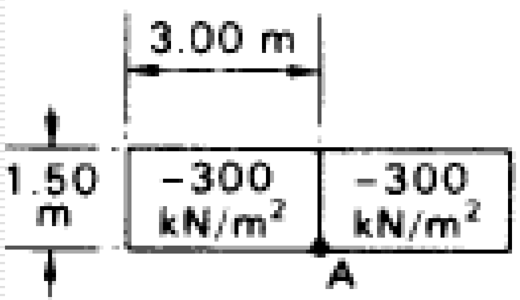
- A rectangular foundation 6 X 3m carries a uniform pressure of 300 kN/m2 near the surface of a soil mass. Determine the vertical stress at a depth of 3m below a point (A) on the centre line 1.5m outside a long edge of the foundation (a) using influence factors




6.00 m
3.00 m
300 kN/m²
1.50 m
A


3.00 m
4.50 m
+300 kN/m²
+300 kN/m²
A

(1)


3.00 m
1.50 m
−300 kN/m²
−300 kN/m²
A

(2)

Soil Shear Strength

Hashemite University

CE 336 Geotechnical Engineering

Class 2017-2018

Definition of Shear Strength

The **shear strength** of a soil mass is the internal resistance per unit area that the soil mass can offer to resist failure and sliding along any plane inside it.

Needed **to analyze soil stability problems such as :**

1. Bearing capacity, 2) slope stability, and 3) lateral pressure on earth-retaining structures.

Shear strength for all of the stability analyses is represented by a Mohr-Coulomb failure envelope that relates shear strength to either total or effective normal stress on the failure plane

a. Total stresses

Type of Shear Strength Parameters

In general there are two types:

1. **Total normal stress**
2. **Effective normal stress**

In the case of total stresses, the shear strength is expressed as:

$$\boxed{s=\tau=c+\sigma \tan \phi}$$

Where

c = cohesion intercept

ϕ = friction angle

σ = total normal stress on the failure plane

Effective Shear Strength

For effective stresses the shear strength is expresses as:

$$\boxed{s=\tau=c'+(\sigma-u)\tan\phi'}$$

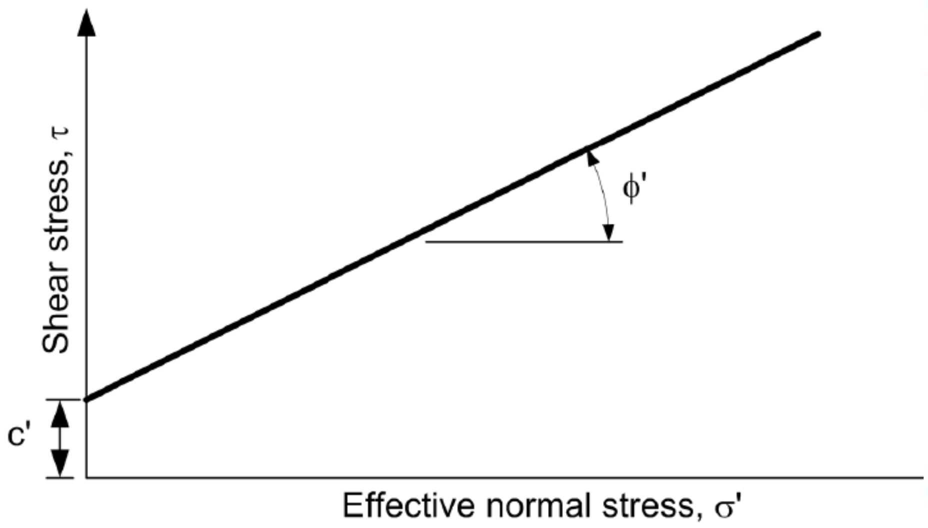


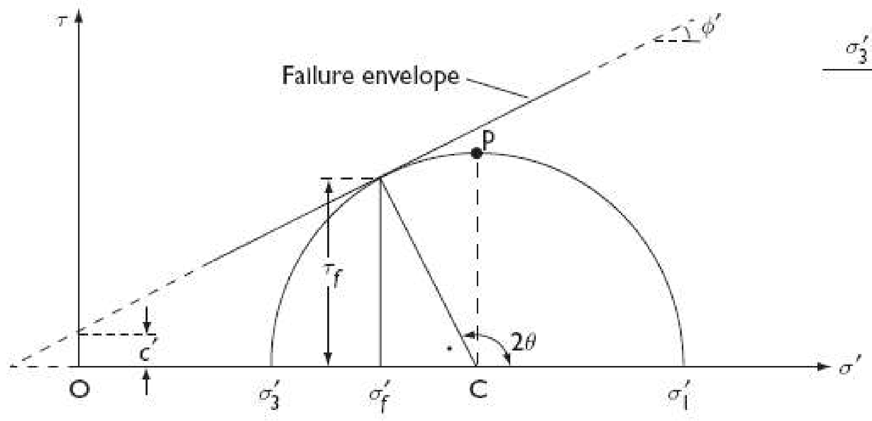
Table 12.1 Typical Values of Drained Angle of Friction for Sands and Silts

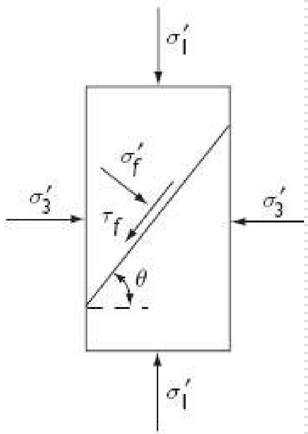
Soil type	ϕ' (deg)
Sand: Rounded grains	
Loose	27–30
Medium	30–35
Dense	35–38
Sand: Angular grains	
Loose	30–35
Medium	35–40
Dense	40–45
Gravel with some sand	34–48
Silts	26–35

c' and ϕ' = intercept and slope angle for the failure envelope plotted in terms of effective stresses

σ and u = total normal stress and pore water pressure, respectively, on the failure plane

Mohr–Coulomb failure criterion





$$\sigma'_f = \frac{1}{2}(\sigma'_1 + \sigma'_3) + \frac{1}{2}(\sigma'_1 - \sigma'_3)\cos 2\theta$$

$$\tau_f = \frac{1}{2}(\sigma'_1 - \sigma'_3)\sin 2\theta$$

$$\theta = 45^\circ + \frac{\phi'}{2}$$

θ is the theoretical angle between the major principal plane and the plane of failure

$$\sin\phi' = \frac{\frac{1}{2}(\sigma'_1 - \sigma'_3)}{c'\cot\phi' + \frac{1}{2}(\sigma'_1 + \sigma'_3)}$$

$$\sigma'_1 = \sigma'_3\tan^2\left(45^\circ + \frac{\phi'}{2}\right) + 2c'\tan\left(45^\circ + \frac{\phi'}{2}\right)$$

Laboratory Strength Test

- **The shear strength parameters, c and ϕ or c' and ϕ', are determined from laboratory shear test data.**

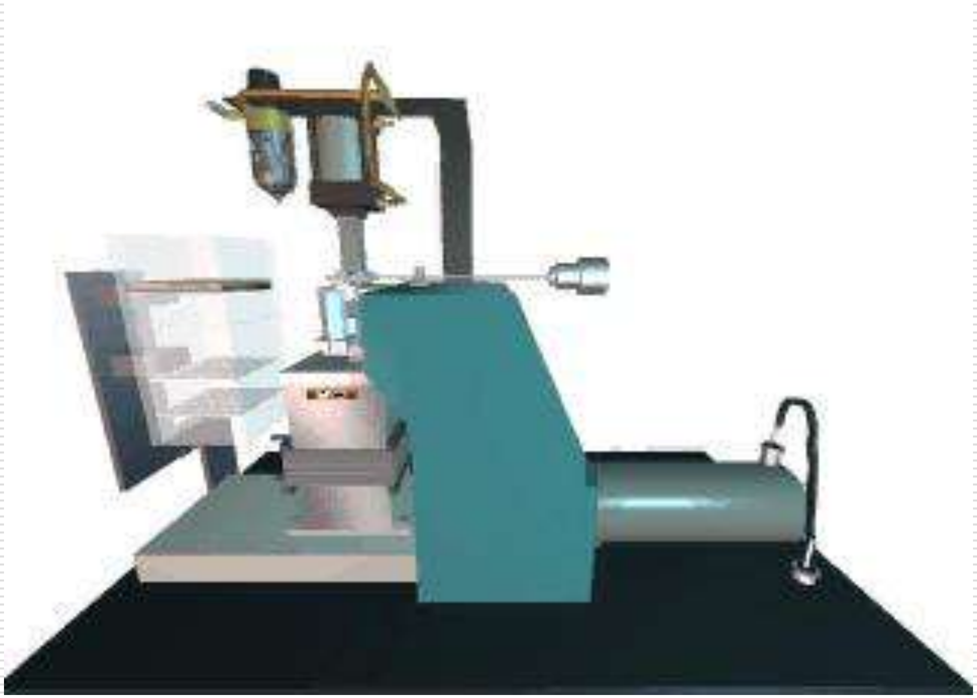
Two common tester in the lab:

1. **Direct shear test**
2. **Triaxial test**
3. **Direct simple shear test**
4. **Plane strain triaxial test**
5. **Torsional ring shear test**

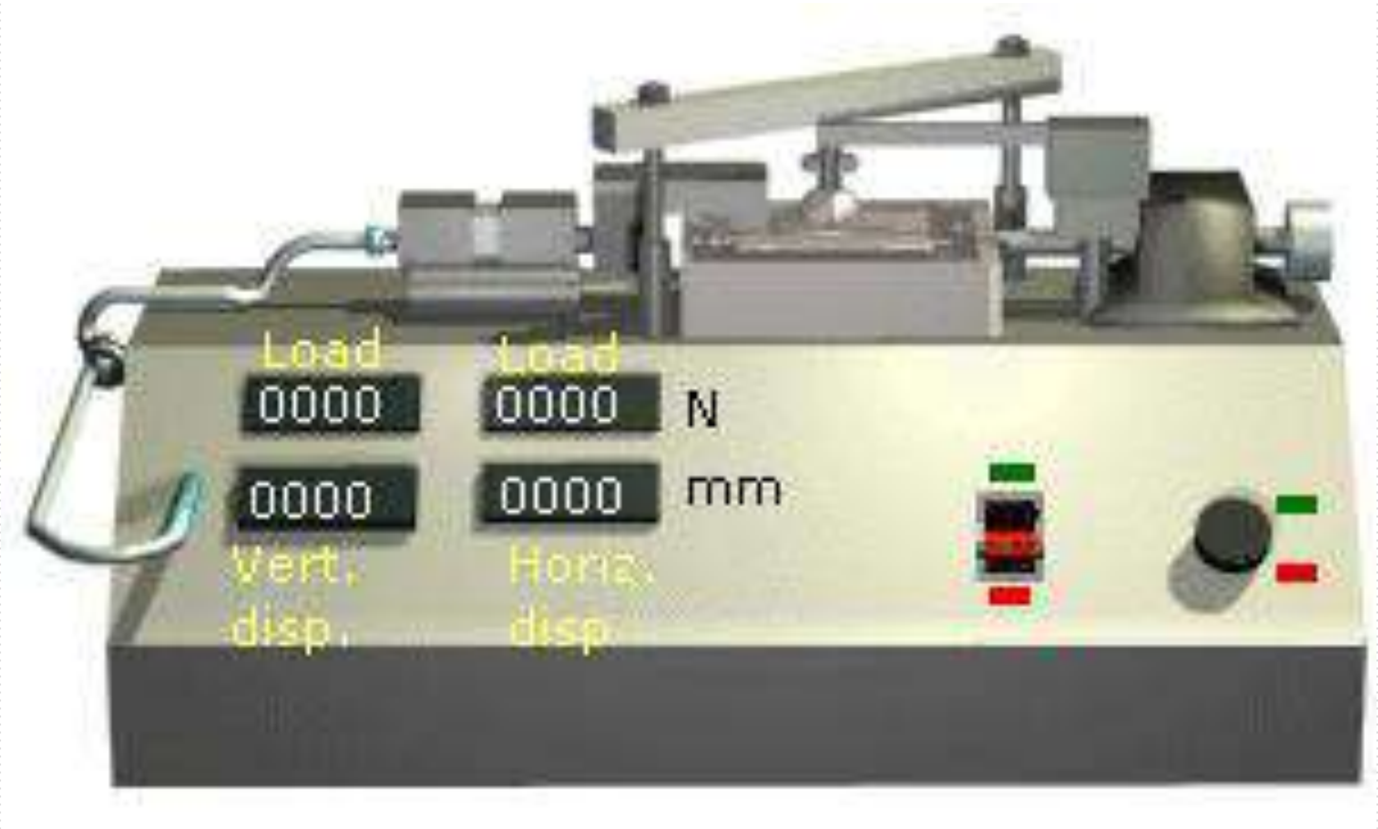
A two-stage loading procedure is used in each of these tests

- **First stage, a confining stress is applied**
- **Second involves shearing the specimen**

Direct Simple Shear

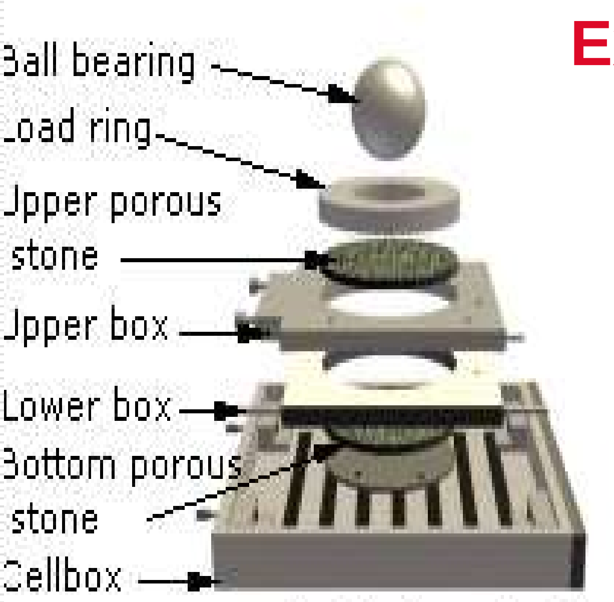


Direct Shear I



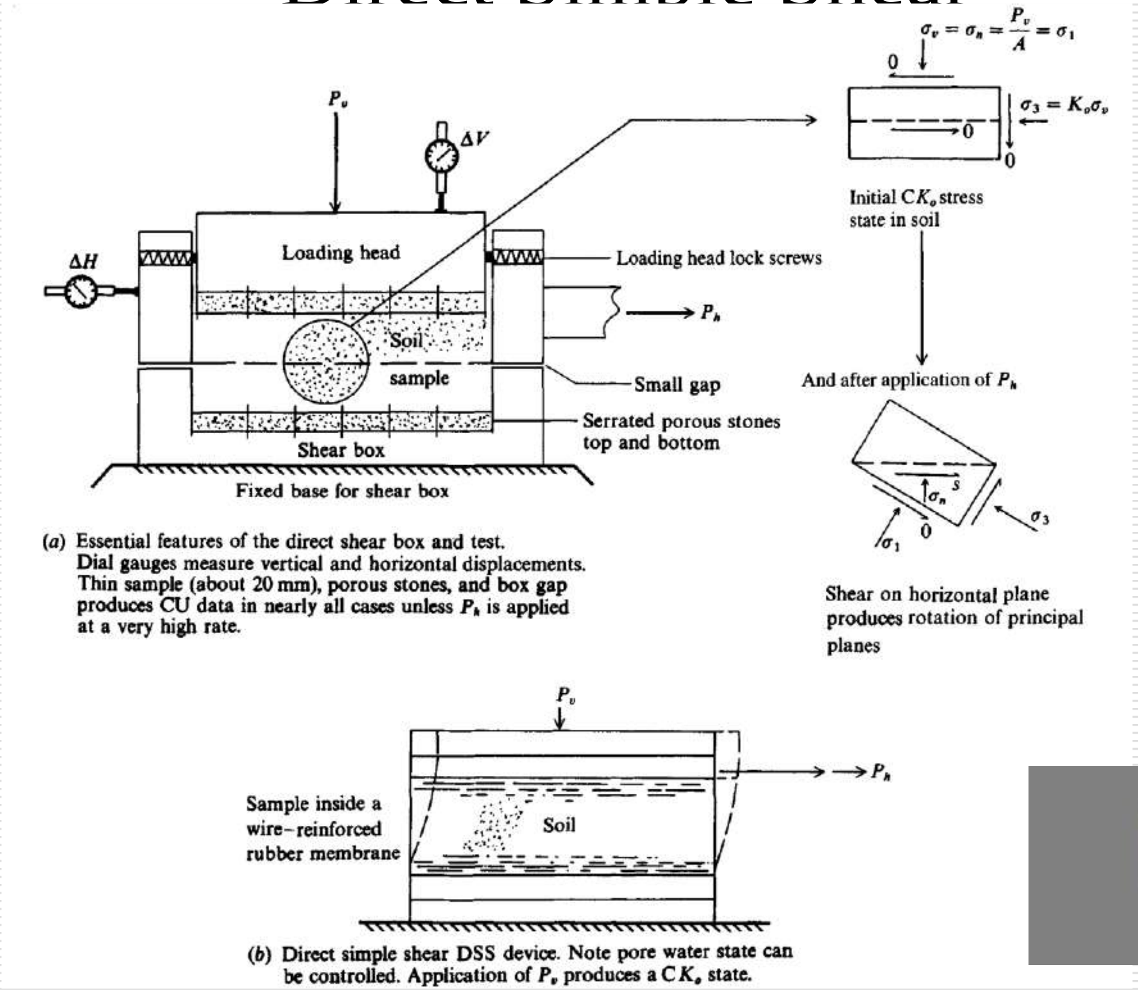
Direct Shear II

Explode View Of Direct Shear Cell



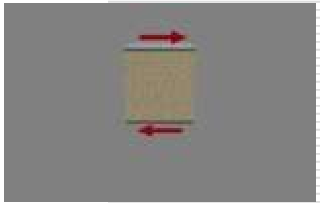
Exploded view of cell

Direct Simple Shear

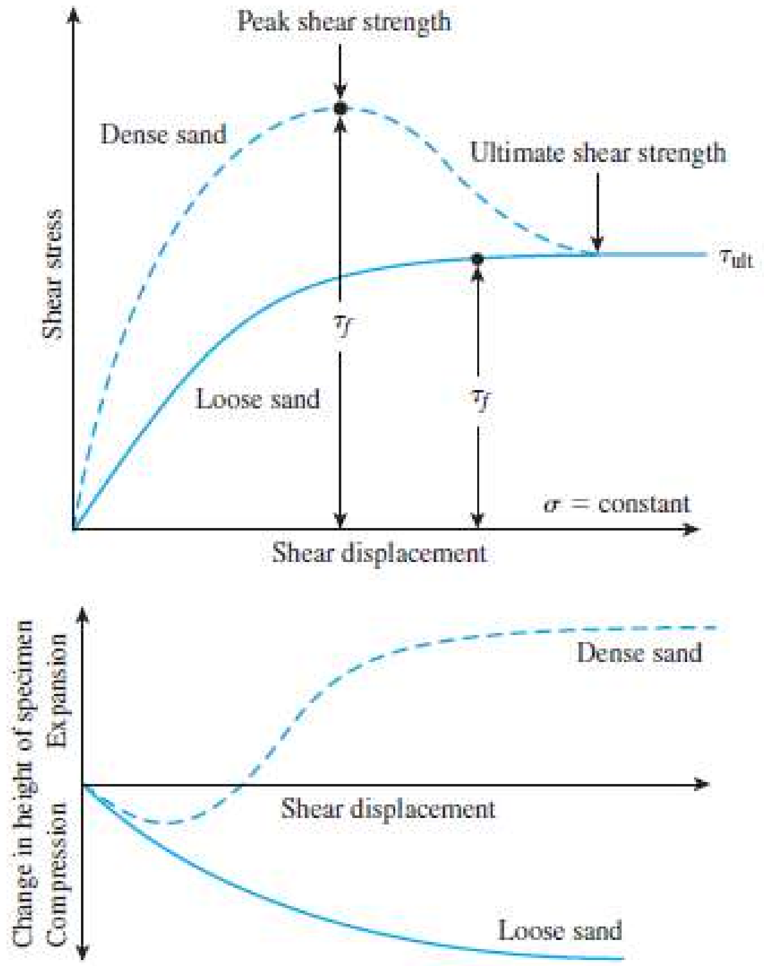


(a) Essential features of the direct shear box and test. Dial gauges measure vertical and horizontal displacements. Thin sample (about 20 mm), porous stones, and box gap produces CU data in nearly all cases unless P_h is applied at a very high rate.

(b) Direct simple shear DSS device. Note pore water state can be controlled. Application of P_v produces a CK_o state.



Typical Shear Stress – shear Displacement



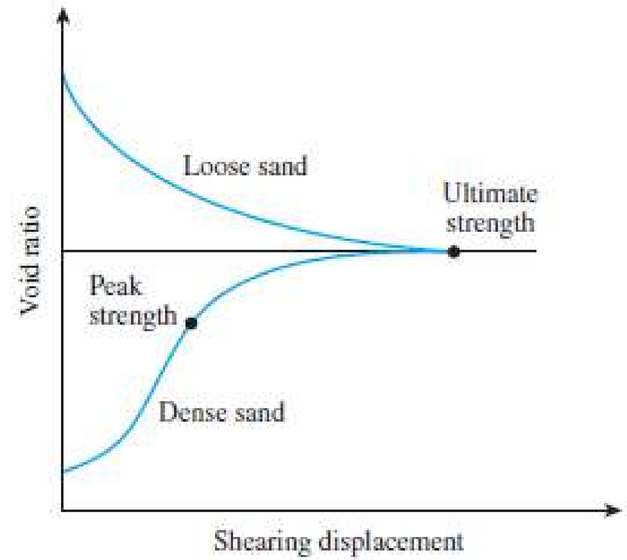
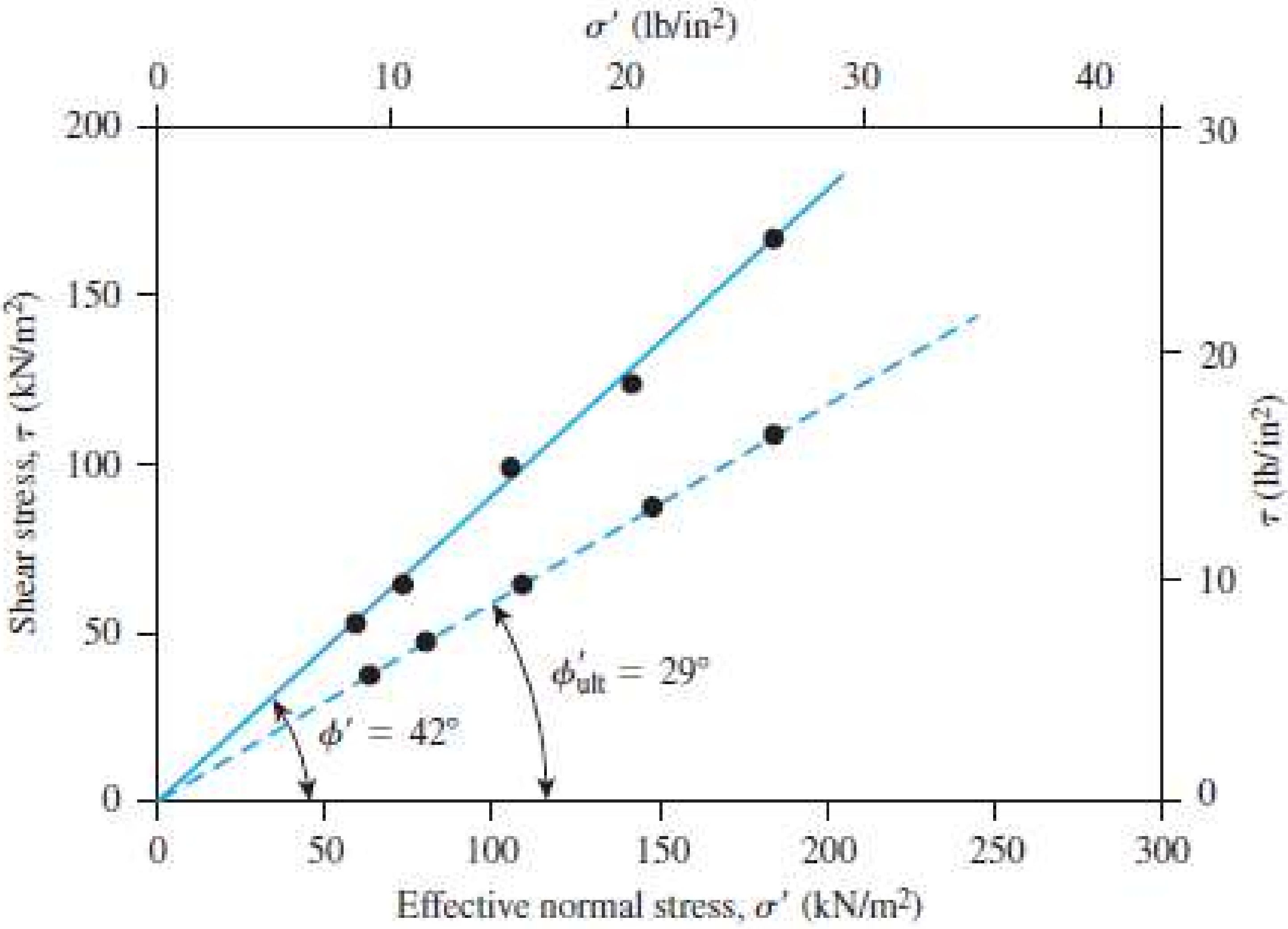


Figure 12.8 Nature of variation of void ratio with shearing displacement

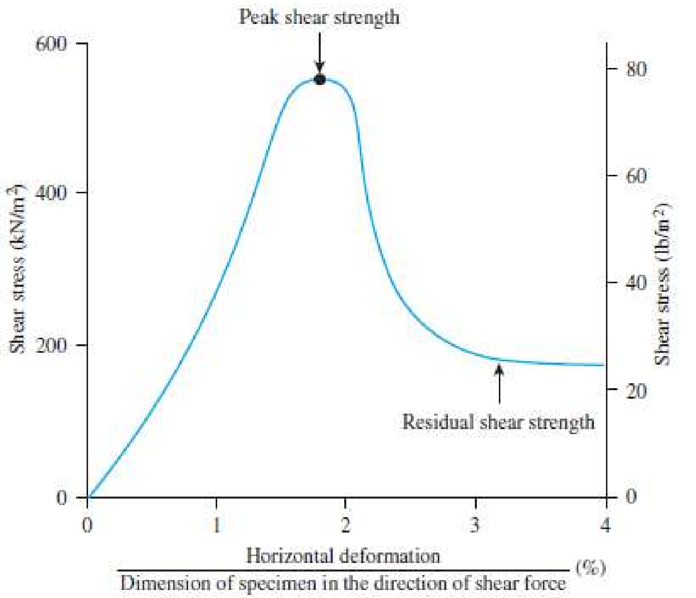
Figure 12.7 Plot of shear stress and change in height of specimen against shear displacement for loose and dense dry sand (direct shear test)

Data Reduction For Shear Strength Parameters



Note that the value of **c′ ≈ 0.** for a normally consolidated clay and sand.

Direct Shear Test In Over Consolidated Soil



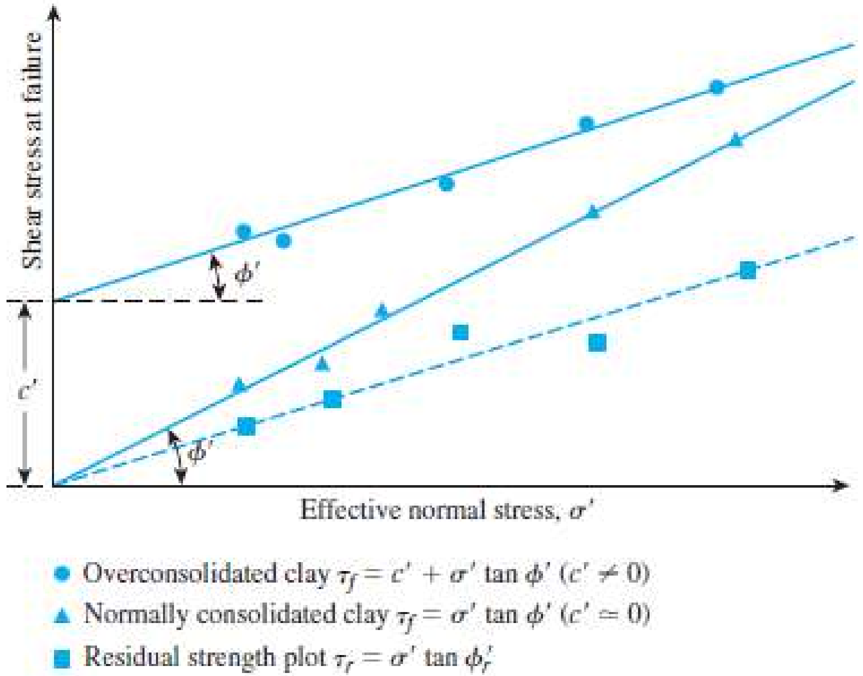
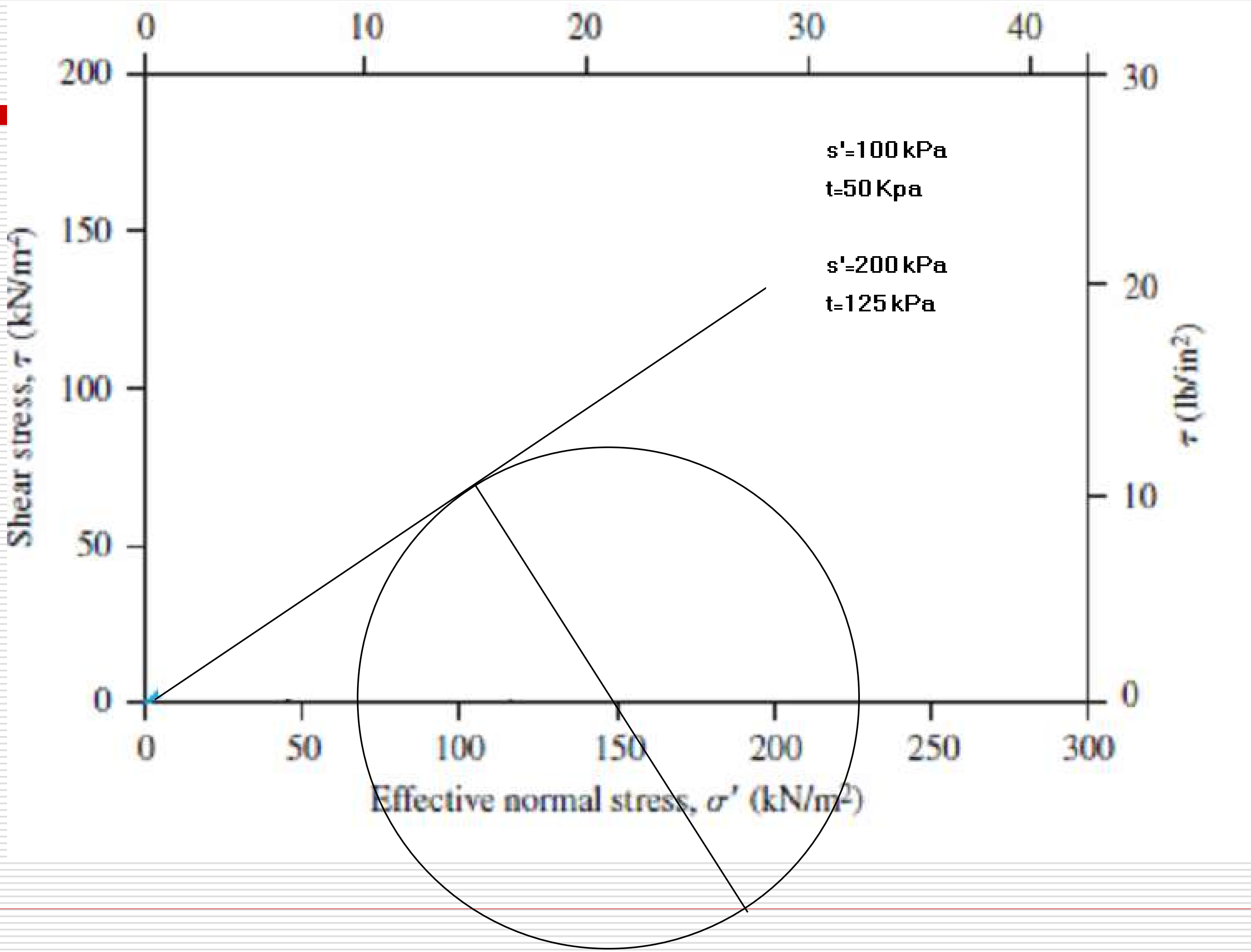
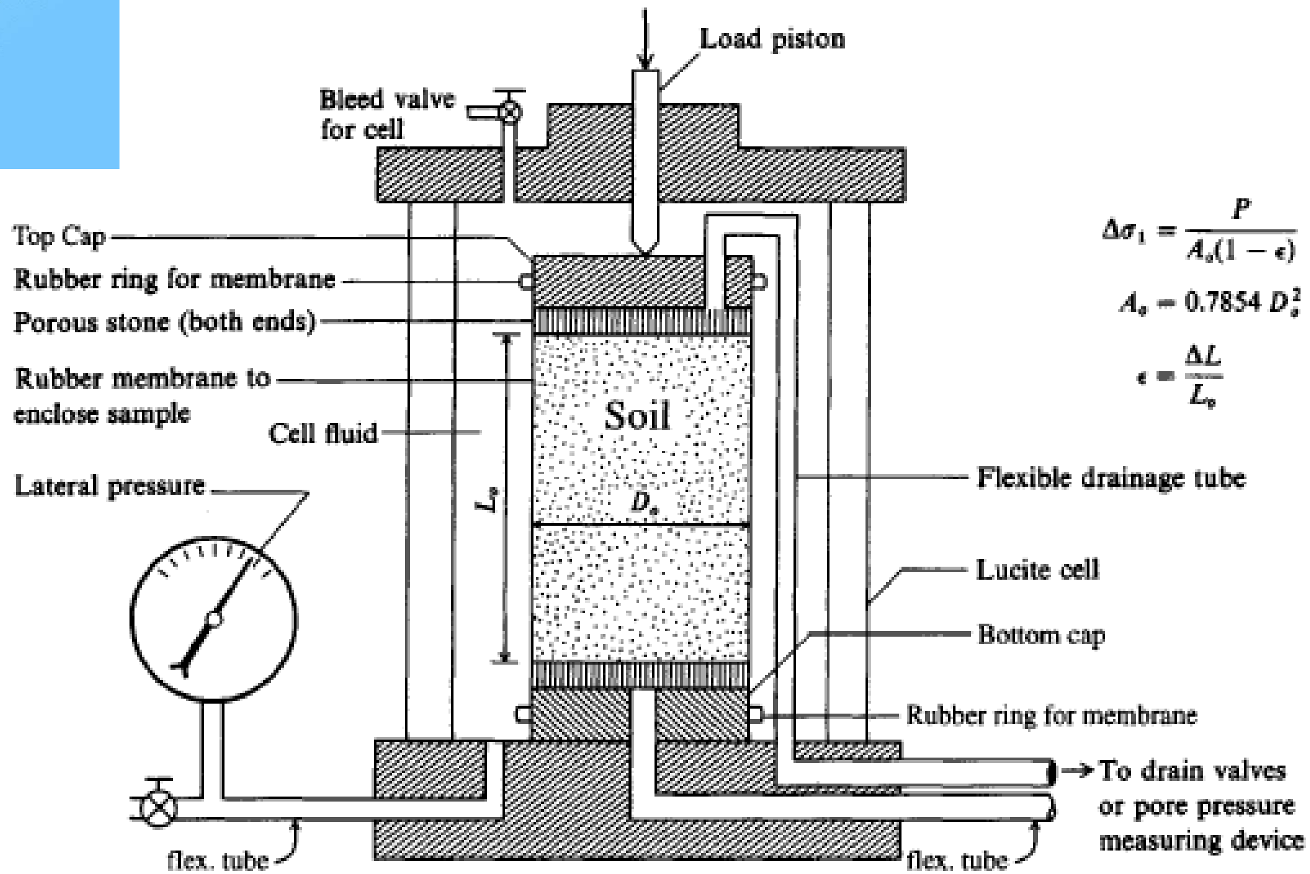


Figure 12.11: Failure envelope for clay obtained from drained direct shear tests

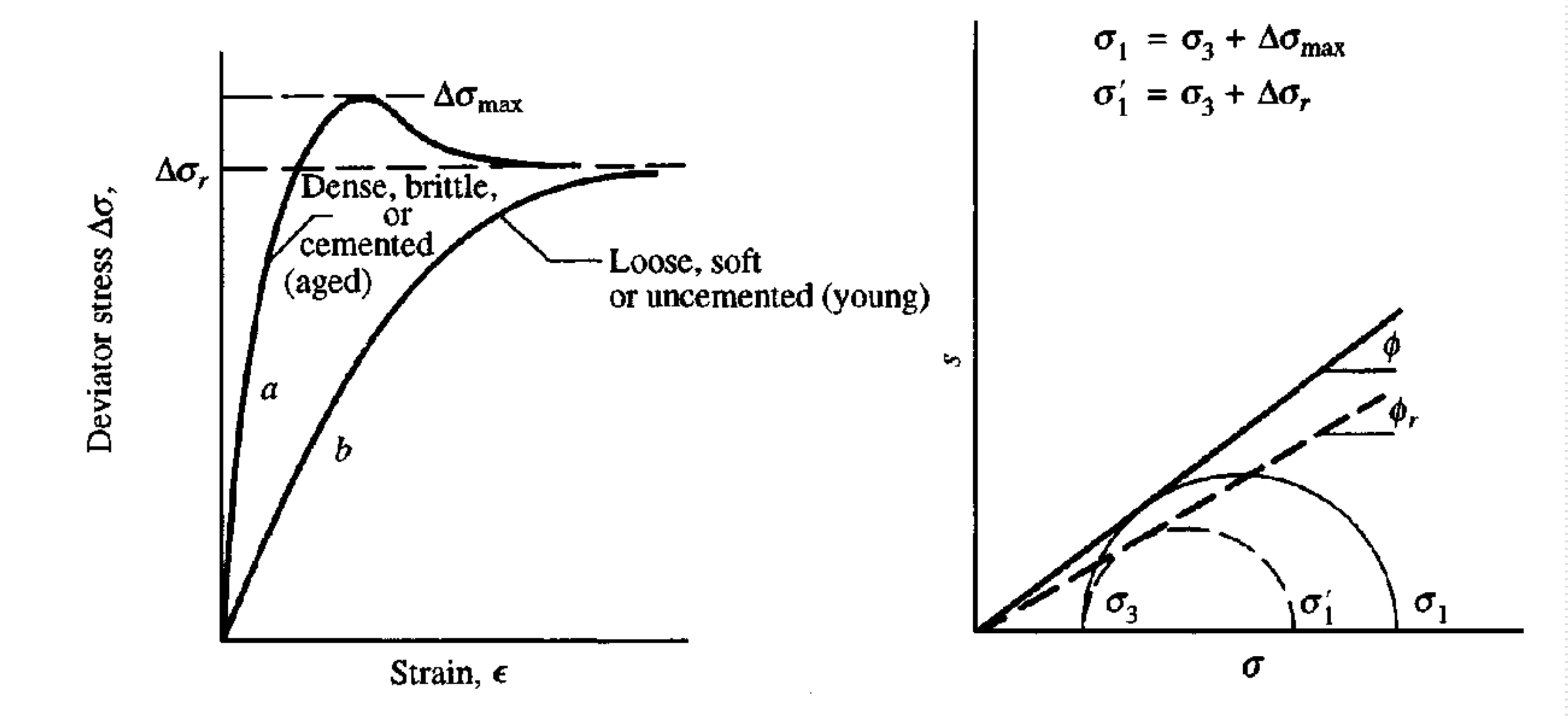

s'=100 kPa
t=50 Kpa
s'=200 kPa
t=125 kPa
0
10
20
30
40
200
150
100
50
0
0
50
100
150
200
250
300
30
20
10
0
Shear stress, τ (kN/m²)
τ (lb/in²)
Effective normal stress, σ' (kN/m²)

Triaxial Tester



<u>Triaxial Test on Soil Sample in Laboratory</u>

Residual Soil Strength



(a) Stress-strain plot applicable for any soil,
(b) Mohr's circle qualitatively shown for a dense sand.

Advantages and Disadvantages of Direct Shear Tester

The advantages of the direct shear test are:

1. **Cheap, fast and simple - especially for sands.**
2. **Failure occurs along a single surface, which approximates observed slips or shear type failures in natural soils.**

Disadvantages of the test include:

1. **Difficult or impossible to control drainage, especially for fine-grained soils.**
2. **Failure plane is forced--may not be the weakest or most critical plane in the field**

3. Non-uniform stress conditions exist in the specimen.

4. The principal stresses rotate during shear, and the rotation cannot be controlled.

Principal stresses are not directly measured.

Comparison Of Triaxial with Direct Shear Test

The advantages of the triaxial test over the direct shear test are:

- **Progressive effects are less in the triaxial.**
- **The measurement of specimen volume changes are more accurate in the triaxial.**
- **The complete state of stress is assumed to be known at all stages during the triaxial test, whereas only the stresses at failure are known in the direct shear test.**
- **The triaxial machine is more adaptable to special requirements.**

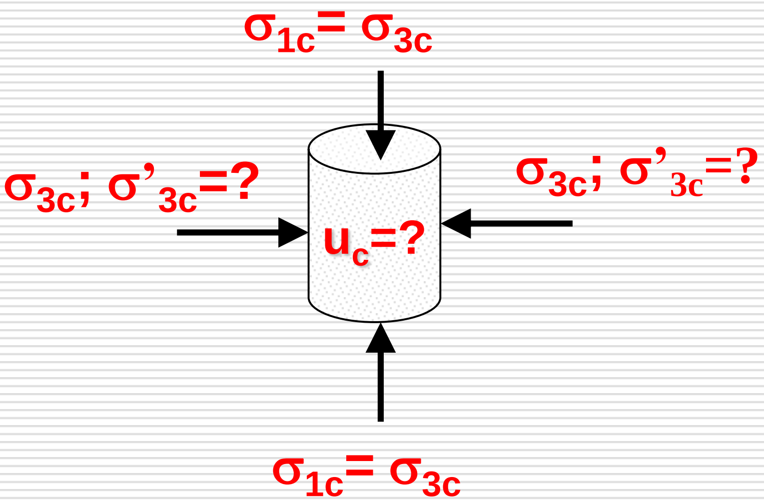
Types of Tests

There are 3 types of tests

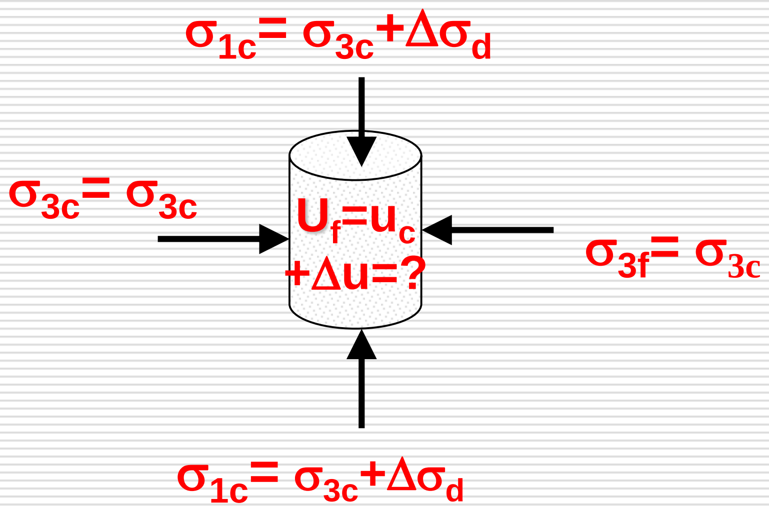
1. UU Quick Q Test

- **Unconsolidated-Undrained (UU) test which is also called the quick test (abbreviations commonly used are UU and Q test).**
- **This test is performed with the drain valve closed for all phases of the test.**
- **Axial loading is commenced immediately after the chamber pressure $\sigma 3$ is stabilized.**

UU – Q Test



Stage 1 Confinement Stage



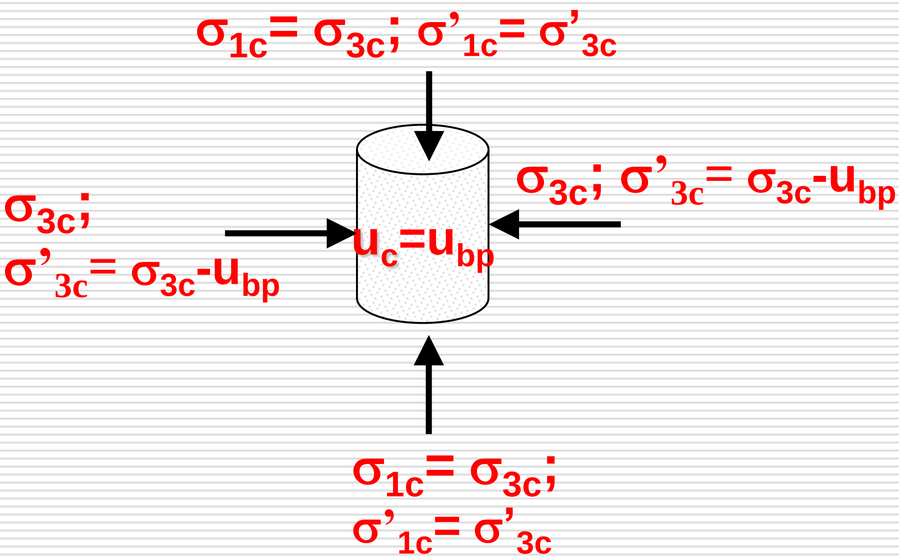
Stage 2 Shear Stage

Types of Tests

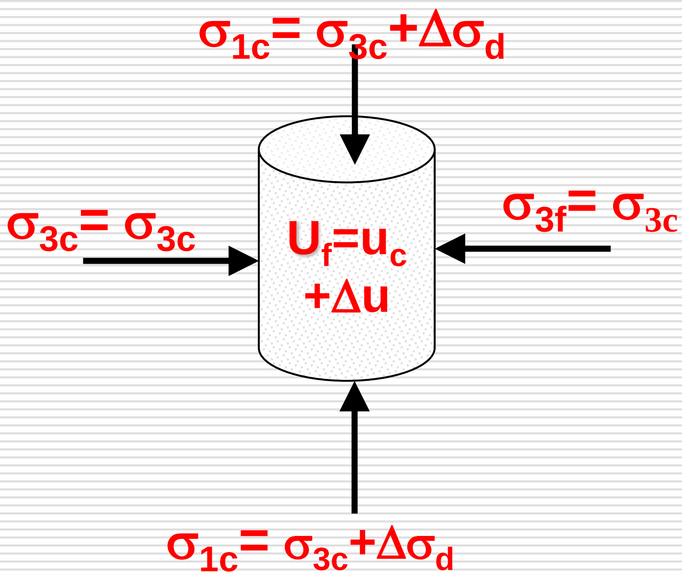
2. CU or R Test

- **Consolidated-Undrained (CU)test, also termed consolidated-quick test or R test (abbreviated CU or R).**
- **In this test, drainage or consolidation is allowed to take place during the application of the confining pressure σ3.**
- **Loading does not commence until the sample ceases to drain (or consolidate).**
- **The axial load is then applied to the specimen, with no attempt made to control the formation of excess pore pressure.**
- **For this test, the drain valve is closed during axial loading, and excess pore pressures can be measured.**

CU – R Test



Stage 1 Confinement Stage



Stage 2 Shear Stage

Types of Tests

3. CD or S Test

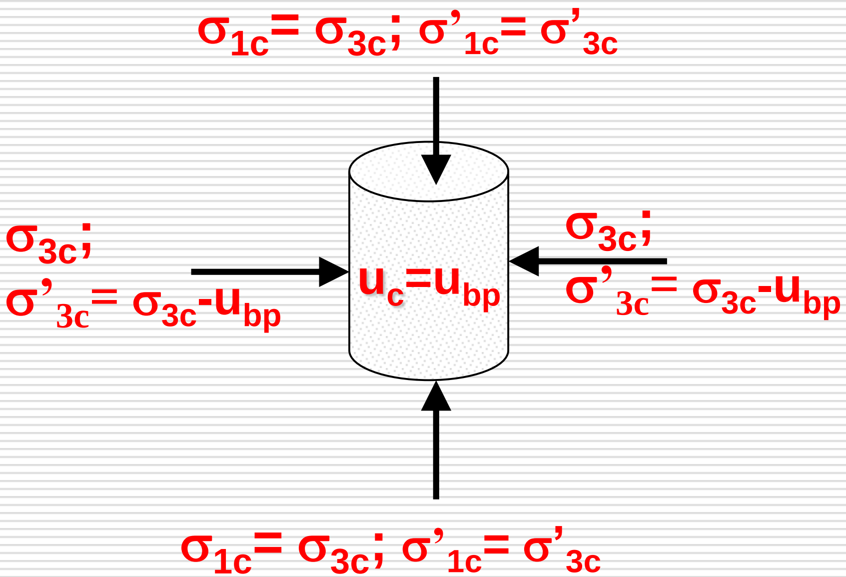
- Consolidated-Drained (CD) test, also called slow test (abbreviated CD or S).

- In this test, the drain valve is opened and is left open for the duration of the test, with complete sample drainage prior to application of the vertical load.

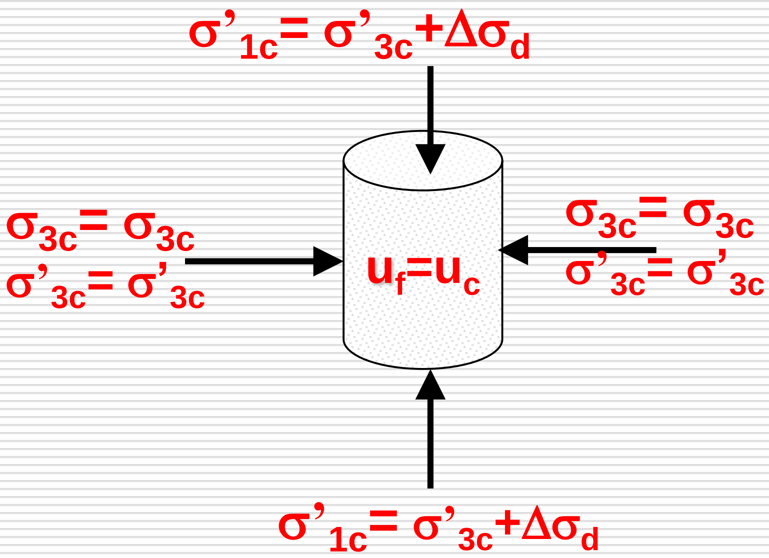
- The load is applied at such a slow strain rate that particle readjustments in the specimen do not induce any excess pore pressure.

- Since there is no excess pore pressure total stresses will equal effective stresses.

CD – S Test



Stage 1 Confinement Stage



Stage 2 Shear Stage

Use of Data

Unconsolidated-Undrained (UU) or Q test

- Results from UU are always plotted using total stresses.
- Thus, the shear strength is expressed in terms of total stress, using c and ϕ.
- Pore water pressures are not measured and are unknown

Use of Data

Consolidated-Undrained (CU) or R test

- Shear strength data from Consolidated-Undrained tests are used in four different ways for slope stability computations:

1. To determine the effective stress shear strength parameters for long-term, steady-state seepage analyses.
2. To determine the relationship between undrained shear strength and effective consolidation pressure (τ_{ff} vs. σ'_{fc}) for analyses of rapid drawdown.
3. To estimate undrained-shear strengths and reduce effects of sample disturbance for end-of construction stability analyses.
4. To estimate undrained shear strength for analyses of staged construction of embankments.

Use of Data

Consolidated-Drained (CD or S)

- CD test used to determine the effective stress shear (ESA) strength parameters of freely draining soils.

- These soils will drain with relatively short testing times and the consolidated-drained loading procedure comes closest to representing the loading for long-term, drained conditions in the field.

- Consolidated-Drained tests procedures are also used to measure the residual shear strength of clays using direct shear or torsional shear equipment.

Summary of Use of Data

Design Condition	Shear Strength
During Construction and End-of-Construction	Free draining soils – use drained shear strengths related to effective stresses Low-permeability soils – use undrained strengths related to total stresses
Steady-State Seepage Conditions	Use drained shear strengths related to effective stresses.
Sudden Drawdown Conditions	Free draining soils – use drained shear strengths related to effective stresses.

Summary of Use of Data

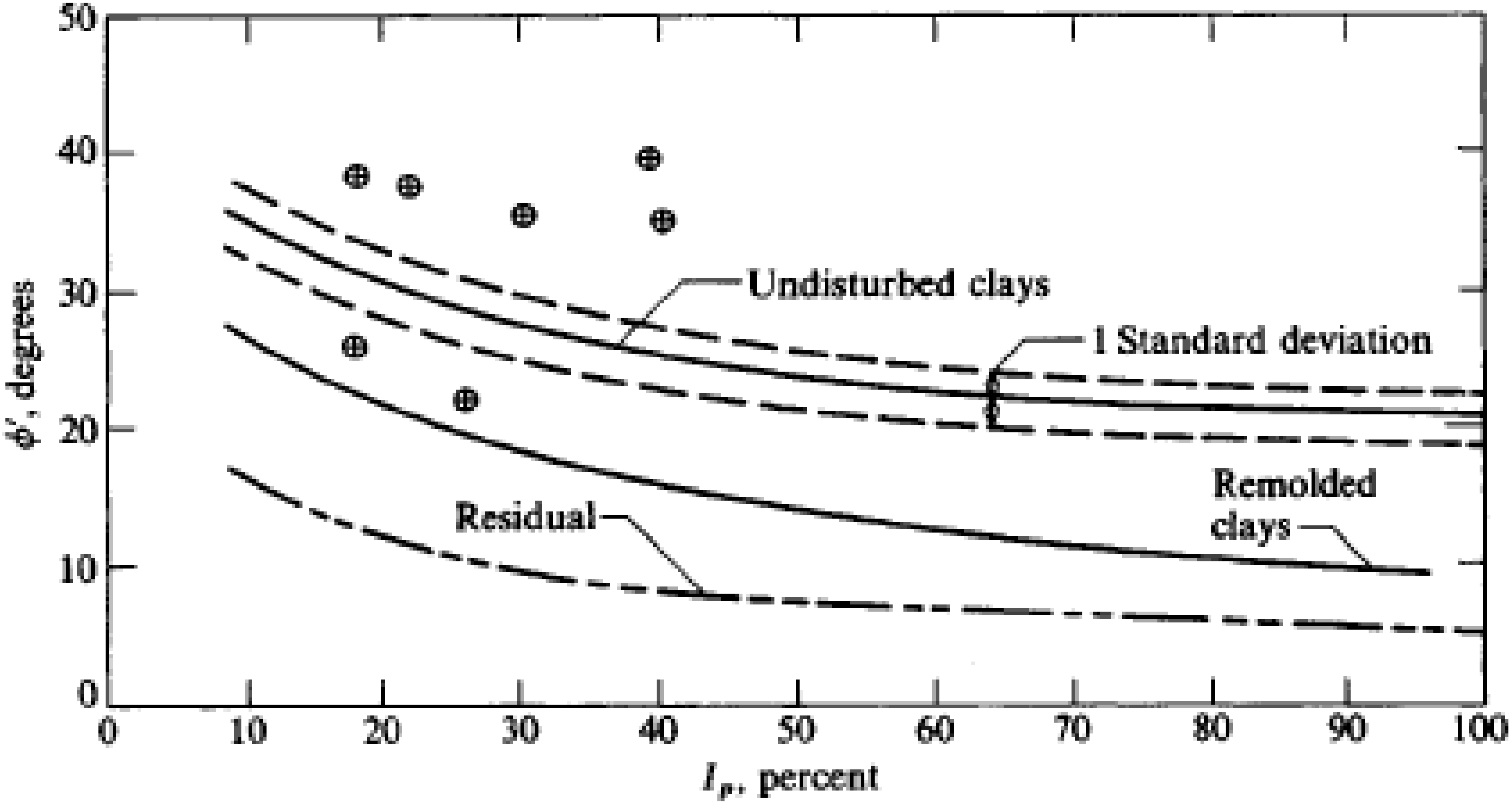
Design Condition	Shear Strength
Sudden Drawdown Conditions	Low-permeability soils – Three-stage computations: First stage-use drained shear strength related to effective stresses; second stage-use undrained shear strengths related to consolidation pressures from the first stage; Third stage-use drained strengths related to effective stresses, or undrained strengths related to consolidation pressures from the first stage, depending on which strength is lower – this will vary along the assumed shear surface.

Typical Shear Strength Values

Representative values for angle of internal friction ϕ

	Type of test*		
Soil	**Unconsolidated-undrained, U**	**Consolidated-undrained, CU**	**Consolidated-drained, CD**
Gravel			
Medium size	40–55°		40–55°
Sandy	35–50°		35–50°
Sand			
Loose dry	28–34°		
Loose saturated	28–34°		
Dense dry	35–46°		43–50°
Dense saturated	1–2° less than dense dry		43–50°
Silt or silty sand			
Loose	20–22°		27–30°
Dense	25–30°		30–35°
Clay	0° if saturated	3–20°	20–42°

Correlation between ϕ' and plasticity index Ip for normally consolidated



Unconfined Compression Test

- In this test, the confining pressure σ3 is 0. An axial load is rapidly applied to the specimen to cause failure.
- At failure, the total minor principal stress is zero and the total major principal stress is σ1

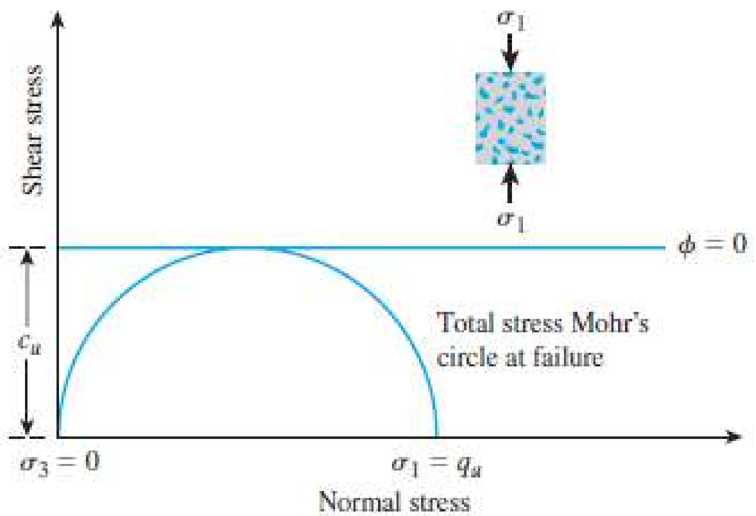


Figure 12.33 Unconfined compression test

Table 12.4 General Relationship of Consistency and Unconfined Compression Strength of Clays

	q_u	
Consistency	kN/m²	ton/ft²
Very soft	0–25	0–0.25
Soft	25–50	0.25–0.5
Medium	50–100	0.5–1
Stiff	100–200	1–2
Very stiff	200–400	2–4
Hard	>400	>4

Su for NC

- **Undrained Shear Strength for Normally consolidated Clay (NC)**

$$\frac{S_u}{P_o'} = 0.45(I_p)^{1/2}$$

Ip in decimal and > 0.5

$$\frac{S_u}{P_o'} = 0.11 + 0.0037 I_p$$

Ip in percent

S_u = Undrained Shear Strength

P_o'= In Situ overburden stress

I_p = plasticity index

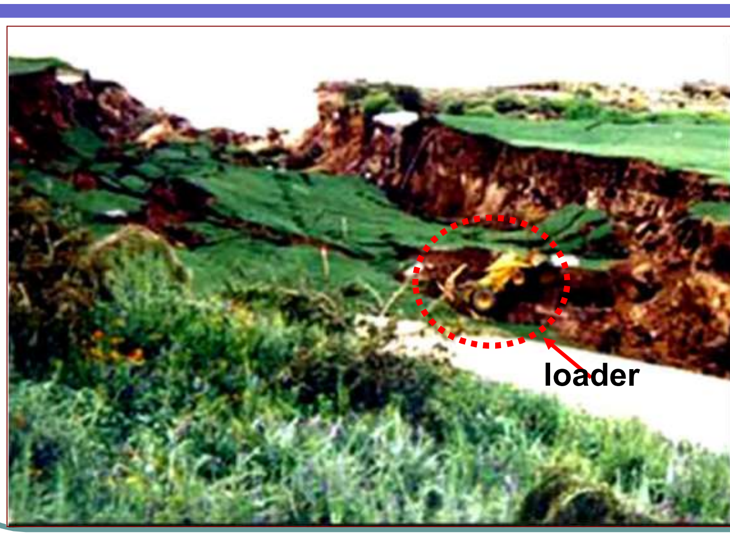
HW

- Try solve questions as much as possible from problems at the end of Chapter 12 in your text book

Slope Stability

Dr. Omar Al Hattamleh
Hashemite University

Slope Stability



Lower San Fernando Dam Failure, 1971

Outlines

- Introduction
- Definition of key terms
- Some types of slope failure
- Some causes of slope failure
- Shear Strength of Soils
- Infinite slope
- Two dimensional slope stability analysis

Introduction I

- ❑ Slopes in soils and rocks are ubiquitous in nature and in man-made structures.
- ❑ Highways, dams, levees, bund-walls and stockpiles are constructed by sloping the lateral faces of the soil
  - ❑ Slopes are general less expensive than constructing a walls.
- ❑ Natural forces (Wind, water, snow, etc.) change the topography on Earth often creating unstable slopes.
- ❑ Failure of such slopes resulted in human loss and destruction.
- ❑ Failure may be sudden and catastrophic; others are insidious;
- ❑ Failure wither wide spread or localized.

Introduction II

- ❑ In this session we will discuss a few methods of analysis from which you should be able to :
  1) Estimate the stability of slopes with simple geometry and geological features
  2) Understand the forces and activities that provoke slope failures
  3) Understand the effects of geology, seepage and pore water pressures on the stability of slopes

Definitions of Key Terms

- Slip or Failure Zone: A thin zone of soil that reaches the critical state or **residual state** and results in movement of the upper soil mass
- Slip plane; failure plane; Slip surface; failure surface: Surface of sliding
- Sliding mass: mass of soil within the slip plane and the ground surface
- Slope angle: Angle of inclination of a slope to the horizontal
- Pore water pressure ratio (r_u): The ratio of pore water force on a slip surface to the total weight of the soil and any external loading.

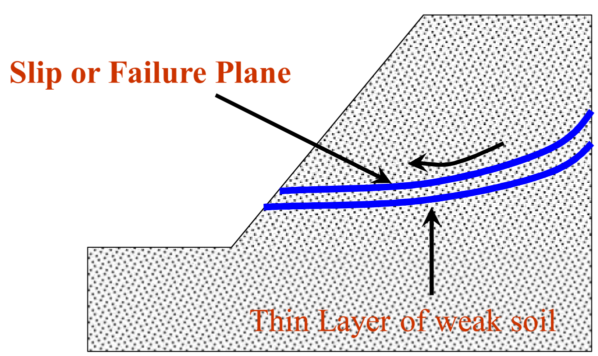
Common Type of Slope Failure

- **Slope failures depends on**
  - The Soil Type,
  - Soil Stratification,
  - Ground Water,
  - Seepage and
  - Geometry.

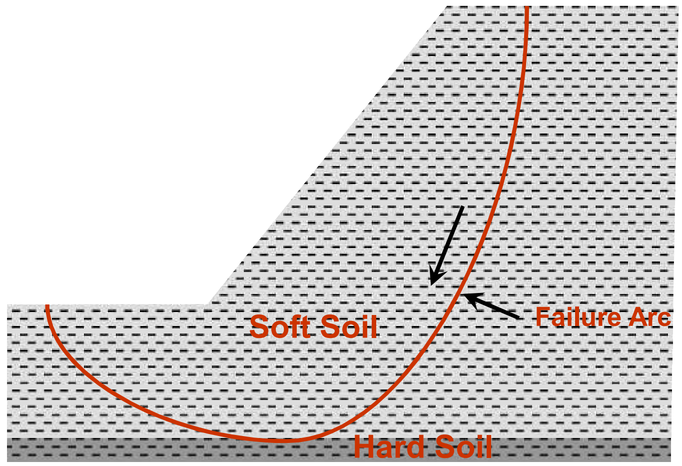
Common Type of Slope failures

- **Common Type**
  - Movement of Soil Mass Along a Thin Layer of Weak Soil
  - Base Slide
  - Toe Slide
  - Slope Slide
  - Flow Slide
  - Block Slide

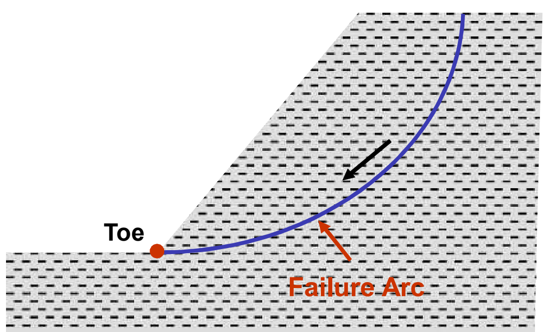
Movement of soil mass along a thin layer of weak soil



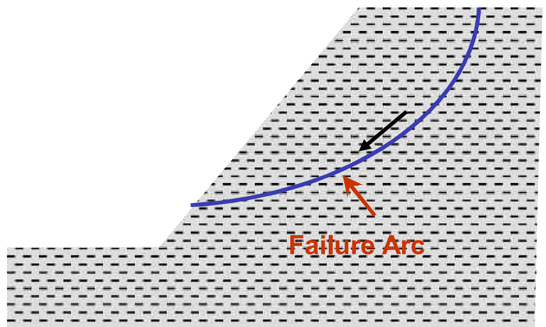
Base Slide



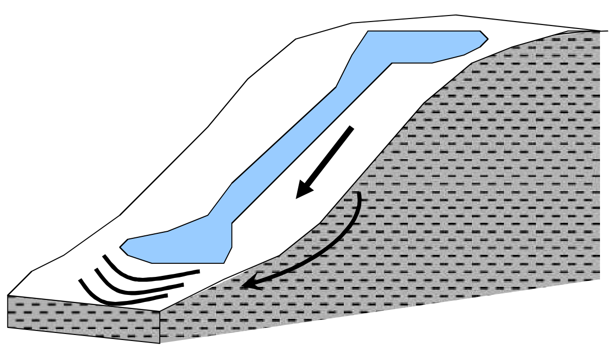
Toe Slide



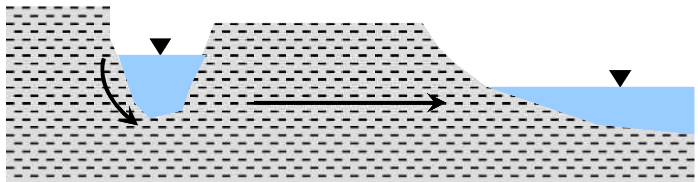
Slope Slide



Flow Slide



Block Slide

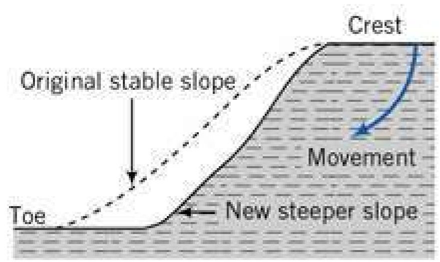


Some causes of slope failure

- Erosion
- Rainfall
- Earthquake
- Geological factures
- External loading
- Construction activity
- Excavated slope
- Fill Slope
- Rapid draw Down

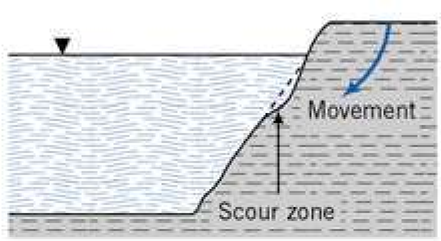
Steepening by Erosion

- Water and wind continuously erode natural and man made slopes.
- Erosion changes the geometry of the slope, ultimately resulting in slope failures or, more aptly, landslide.



Water Scouring

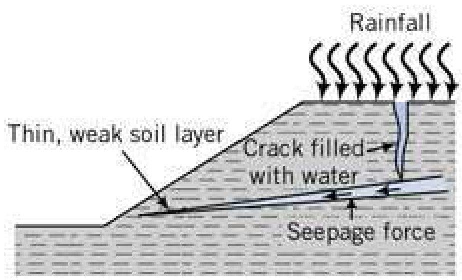
- Rivers and stream continuously scour their banks undermining their natural or man made slopes



Scouring by water movement

Rainfall

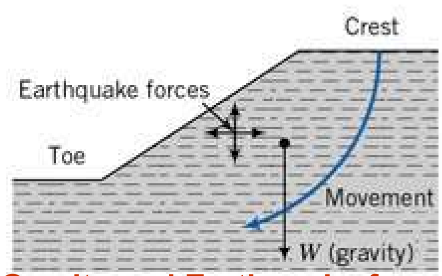
Long period of rainfall saturate, soften and erode soils. Water enter into exiting crack and may weaken underlying soil layers leading to failure e.g. mudslides



Rainfall fills crack and introduces seepage forces in the thin, weak soil layer

Earthquake

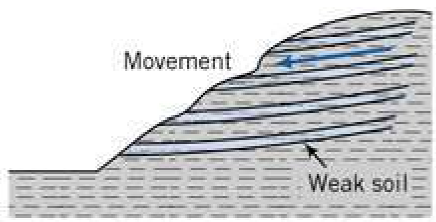
- Earthquake introduced dynamic forces. Especially dynamic shear forces that reduce the shear strength and stiffness of the soil. Pore water pressures rise and lead to liquefaction



Gravity and Earthquake forces

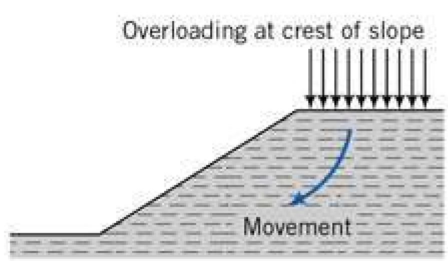
Geological factures

- Sloping stratified soils are prone to translational slide a long weak layer



External loading

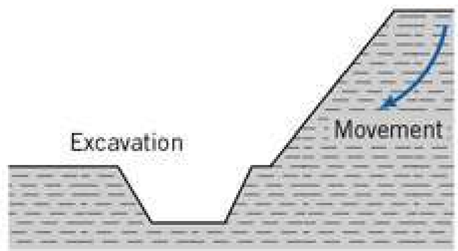
- ❑ Loads placed on the crest of a slope add to the gravitational load and may cause slope failures.



- ❑ Load places at the toe called a berm, will increase the stability of the slope. Berms are often used to the remediate problem slopes.

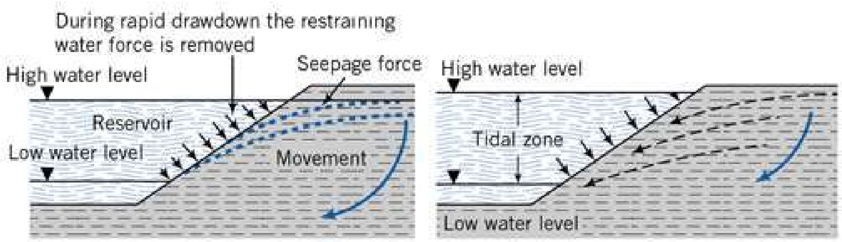
Construction Activity

- Excavated slopes: If the slope failures were to occur, they would take place after construction is completed.
- Fill slopes: failure occur during construction or immediately after construction.



Rapid Draw Down

- Later force provided by water removed and excess p.w.p does not have enough time to dissipated



Infinite slope I

Analysis of a Plane Translational Slip

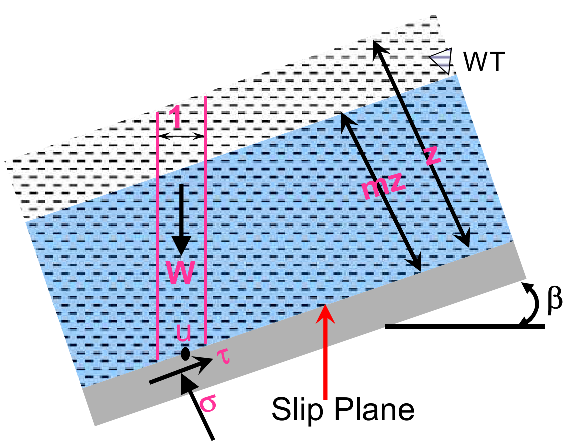
Infinite slope I

- Definition:
  - Infinite Slope: a slope that have dimension extended over great distance.
- Assumption:
  - The potential Failure surface is parallel to the surface of the Slope
  - Failure surface depth << the length of slope
  - End effects are ignored

Infinite Slope II

- ❑ Assumption Continued:
  - ❑ The failure mass moves as an essentially rigid body, the deformation of which do not influence the problem
  - ❑ The shearing resistance of the soil mass at various point along the slide of the failure surface is independent of orientation
  - ❑ The Factor of safety is defined in term of the average shear strength along this surface.

Infinite Slope III



Infinite Slope IV

Stress in the soil mass and Available Shear Strength

$$\sigma = [(1-m)\gamma + m\gamma_{sat}]z\cos^2\beta$$

$$\tau = [(1-m)\gamma + m\gamma_{sat}]z\sin\beta\cos\beta$$

$$u = mz\gamma_w\cos^2\beta$$

$$\tau_f = c' + (\sigma - u)\tan\phi'$$

Infinite Slope V

Effective stresses (Three Scenarios)

1) 0<m<1 $$\boxed{F.S = \frac{\tau_f}{\tau_m} = \frac{c' + (\sigma - u)\tan\phi'}{[(1-m)\gamma + m\gamma_{sat}]z\sin\beta\cos\beta}}$$

2) m=0 & c'=0. $$\boxed{F.S = \frac{\tan\phi'}{\tan\beta}}$$

3) m=1 & c'=0. $$\boxed{F.S = \frac{\gamma'}{\gamma_{sat}}\frac{\tan\phi'}{\tan\beta}}$$

Total stresses: c' $\rightarrow c_u$ and ϕ' $\rightarrow \phi_u$ and u=0

Infinite Slope VI

- Summary:
  1) The maximum stable slope in a coarse grained soil, in the absence of seepage is equal to the friction angle
  2) The maximum stable slopes in coarse grained soil, in the presence of seepage parallel to the slope, is approximately one half the friction angle
  3) The critical slip angle in fine grained soil is 45^o for an infinite slope mechanisms

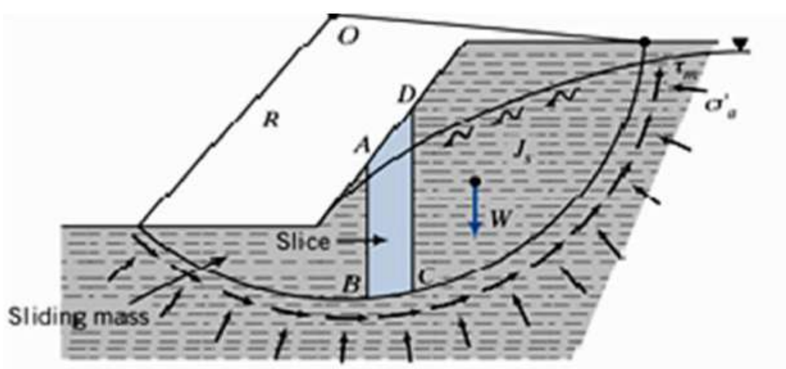
Finite Slopes

Analysis of a Finite Slip Surface

Two Dimensional Slope Stability Analysis

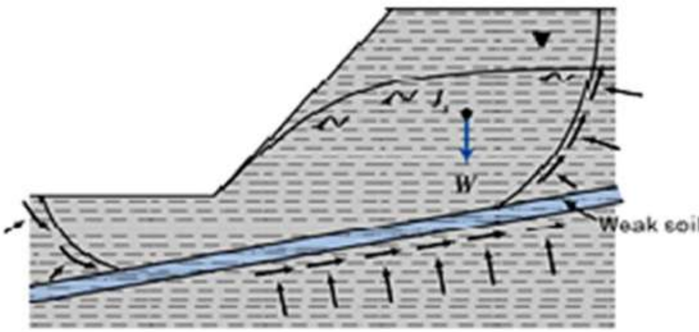
- Slope stability can be analyzed on different method
  - Limit equilibrium (most used)
    - Assume on arc of circle (Fellenius, Bishop)
    - Non circular slope failure (Janbu)
  - Limit analysis
  - Finite difference
  - Finite element (more flexible)

Rotational Failure



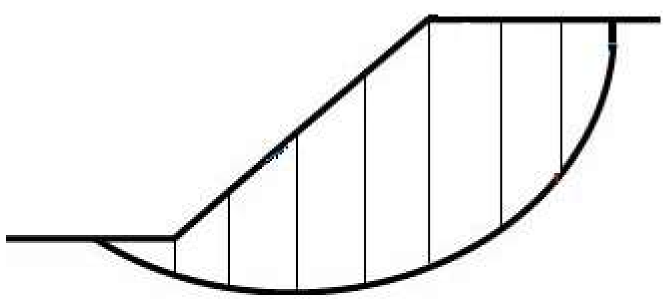
Circular Failure Surface

Rotational Failure

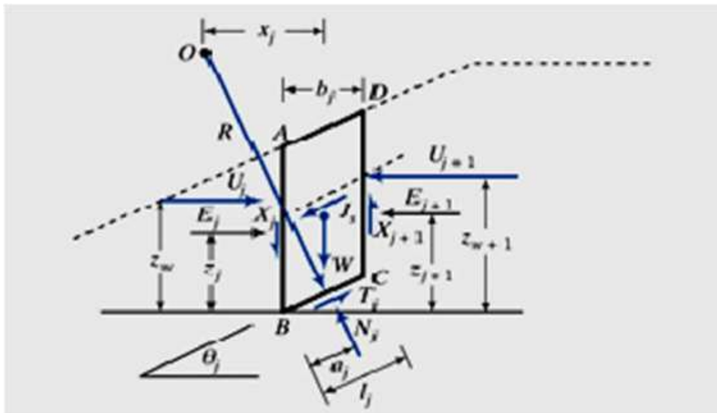


Noncircular Failure Surface

Method of Slices



Forces on Single slice



Forces On Single Slice

- W_j =total weight of a slice including any external load
- E_j = the interslices lateral effective force
- $(Js)_j$ = seepage force on the slice
- N_j = normal force along the slip surface
- X_j = interslices shear forces
- U_j = forces form pore water pressure
- Z_j =Location of the interslices lateral effective force
- Z_w=Location of the pore water force
- a_j = location of normal effective force along the slip surface
- b_j= width of slice
- l_j= length of slip surface along the slice
- θ_j = inclination of slip surface within the slice with respect to horizontal

Equilibrium Assumption and Unknown

- Factors in Equilibrium Formulation of Slope Stability for n slices

Unknown	Number
Ei	n-1
Xi	n-1
Bi	n-1
Ni	n
Ti	n
θi	n
Total Unknown	6n-3

❖**The available Equation is 3n**

Bishop Simplified Method I

❑Bishop assumed

- ❑ a circular slip surface
- ❑ E_j and E_{j+1} are collinear
- ❑ U_j and U_{j+1} are collinear
- ❑ N_j acts on center of the arc length
- ❑ Ignore X_j and X_{j+1}

Bishop Simplified Method II
Factor of Safety

- Factor of safety for an ESA

$$F.S = \frac{\sum c'_j l_j + \sum (W_j (1 - r_u)(\tan\phi')_j m_j)}{\sum W_j \sin\theta_j}$$

$$m_j = \frac{1}{\cos\theta_j + \dfrac{\tan(\phi')_j \sin\theta_j}{FS}}$$

- Factor of safety when groundwater is below the slip surface, ru = 0

$$F.S = \frac{\sum c'_j l_j + \sum (W_j (\tan\phi')_j m_j)}{\sum W_j \sin\theta_j}$$

Bishop Simplified Method III
Factor of Safety

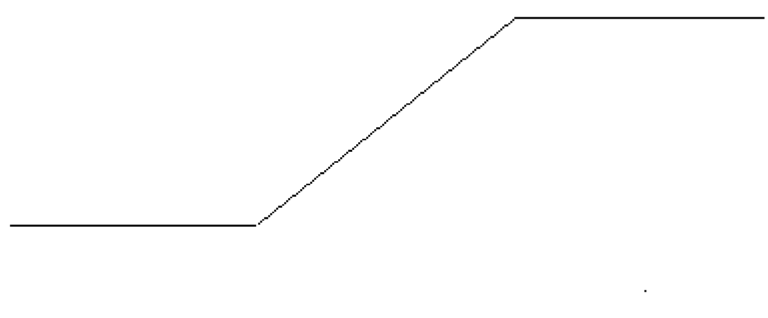
- Factor of safety equation based on TSA

$$FS = \frac{\sum (s_u)_j \dfrac{b_j}{\cos\theta_j}}{\sum W_j \sin\theta_j}$$

- If m=1 the method become Fellenius method of slices

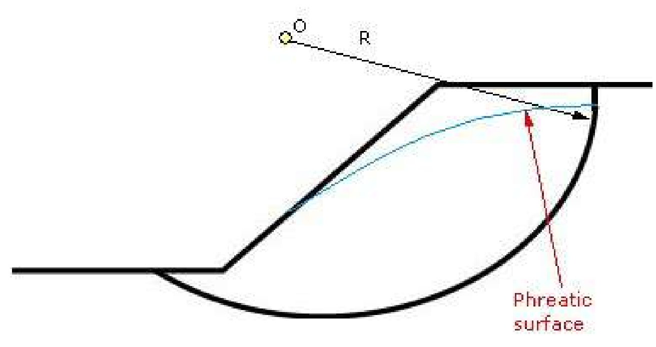
Procedure of analysis Method of slices

- Draw the slope to scale including soil layer



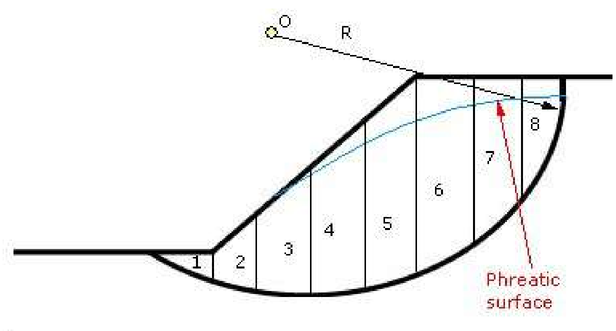
Procedure of Analysis Method of slices

Step 2: Arbitrarily draw a possible slip circle (actually on arc) of a radius R and locate the phreatic surface



Procedure of analysis Method of slices

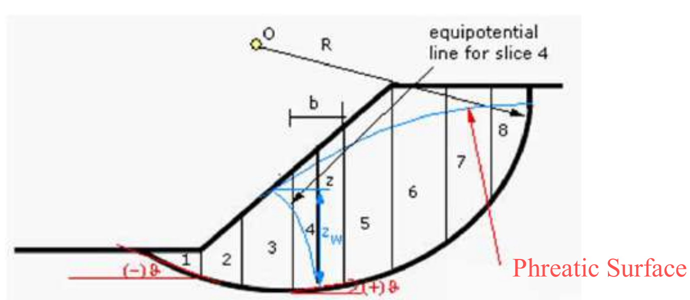
- Step three: divide the circle into slices; try to make them of equal width and 10 slices will be enough for hand calculation



Procedure of analysis Method of slices

- Step four: make table as shown and record b, z, z_w, and θ for each slice

Slice	b	z	W	Zw	ru	θ	mj	l=bcosθ	Cl	Wsinθ	W(1-ru)tanϕ'mj



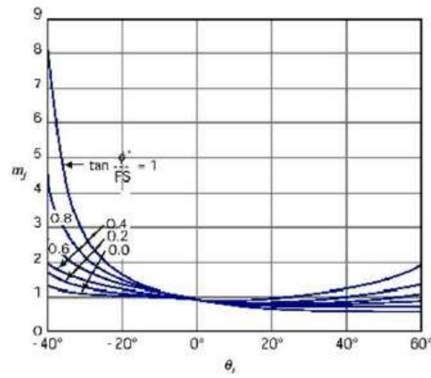
Procedure of analysis Method of slices

- Step five: calculate $W = \gamma bz$, $r_u = z_w \gamma_w / gh$,

assume FS and calculate mj

$$m_j = \frac{1}{\cos\theta_j + \dfrac{\tan(\phi')_j \sin\theta_j}{FS}}$$

complete rest of column

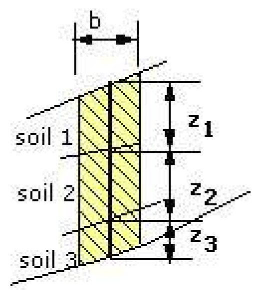


Procedure of analysis Method of slices

- Step Six: Divide the sum of column 10 by the sum of column 9 to get FS.
- If FS is not equal to the assumed value , reiterate until FS calculated and FS are approximately equal

Procedure of analysis Method of slices

- ❑ Multiple soil layer within the slice
  - ❑ Find mean height of each soil layer
  - ❑ $W=b(\gamma_1 z_1+\gamma_2 z_2+\gamma_3 z_3)$
  - ❑ The ϕ' will be for soil layer # three (in this case)



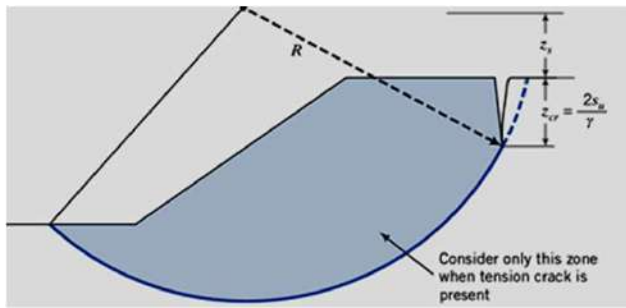
Friction Angle

- For Effective Stress Analysis
  - Use ϕ'_{cs} for most soil
  - Use ϕ'_{res} for fissured over consolidated clay
- For Total Stress Analysis use conservative value of S_u

Tension Crack

- Tension crack developed in fined grain soil.

1. Modify failure surface: failure surface stop at the base of tension crack



2. May Filled with water: reducing FS since the disturbing moment increase

Simplified Janbu's Method I

- Janbu assumed a noncircular slip surface
- Assumed equilibrium of horizontal forces
- Simplified form of Janbu's equation for an ESA

$$F.S = f_o \frac{\sum c'_j l_j + \sum (W_j (1 - r_u)(\tan\phi')_j m_j \cos\theta_j)}{\sum W_j \sin\theta_j}$$

f_o= correction factor for the depth of slope (BTW 1.0 and 1.06)

Simplified Janbu's Method II

- Factor of safety when groundwater is below the slip surface, $r_u = 0$

$$\boxed{F.S = f_o \frac{\sum(c'_j l_j) + \sum(W_j \tan\phi'_j m_j \cos\theta_j)}{\sum(W_j \sin\theta_j)}}$$

- Simplified form of Janbu's equation for a TSA

$$\boxed{F.S = f_o \frac{\sum(Su_j b_j)}{\sum(W_j \tan\theta_j)}}$$

f_o= correction factor for the depth of slope (BTW 1.0 and 1.12)

Summary For Bishop and Janbu

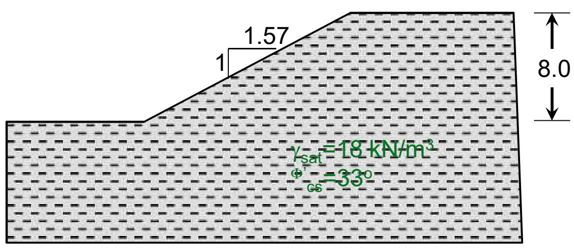
- Bishop (1955) assumes a circular slip plane and consider only moment equilibrium. He neglect seepage force and assumed that lateral normal forces are collinear. In Bishop's simplified, the resultant interface shear is assumed to be zero
- Janbu (1973) assumed a noncircular failure and consider equilibrium of horizontal forces. He made similar assumptions to bishop except that a correct force is applied to replace interface shear
- For slopes in fine grained soils, you should conduct both an ESA and TSA for a long term loading and short term loading condition respectively. For slopes in course grained soil, only ESA is necessary for short term and long term loading provided the loading is static

Microsoft Excel Sheet Solution

Examples of Bishop's and Janbu's method by utilizing excel worksheets

Examples # 1

Slope satiability by Bishop's Method using excel sheets



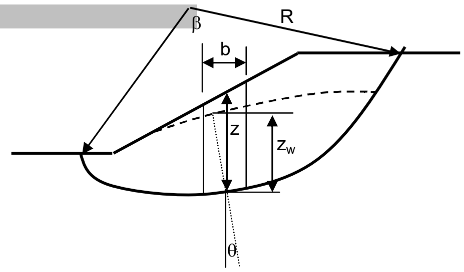
- ❑ **Using Bishop's method determine FS**

1. **If there is no tension crack**

Examples # 1 Solution

Bishop's simplified method
Homogenous soil

s_u	30	kPa
ϕ'	33	deg.
γ_w	9.8	kN/m^3
γ_{sat}	18	kN/m^3
z_{cr}	3.33	m
z_s	4	m
FS	1.06	assumed



No tension crack

								ESA	ESA	TSA
Slice	b	z	$W=\gamma bz$	z_w	r_u	θ	m_j	$W\sin\theta$	$W(1-r_u)\tan\phi'\, m_j$	$s_u\, b/\cos\theta$
	m	m	kN	m		deg				
1	4.9	1	88.2	1	0.54	-23	1.47	-34.5	38.3	159.7
2	2.5	3.6	162.0	3.6	0.54	-10	1.14	-28.1	54.6	76.2
3	2	4.6	165.6	4.6	0.54	0	1.00	0.0	49.0	60.0
4	2	5.6	201.6	5	0.49	9	0.92	31.5	62.1	60.7
5	2	6.5	234.0	5.5	0.46	17	0.88	68.4	72.2	62.7
6	2	6.9	248.4	5.3	0.42	29	0.85	120.4	80.1	68.6
7	2	6.8	244.8	4.5	0.36	39.5	0.86	155.7	87.6	77.8
8	2.5	5.3	238.5	2.9	0.30	49.5	0.90	181.4	97.5	115.5
9	1.6	1.6	46.1	0.1	0.03	65	1.02	41.8	29.6	113.6
							Sum	536.6	570.9	794.8
								FS	**1.06**	**1.48**

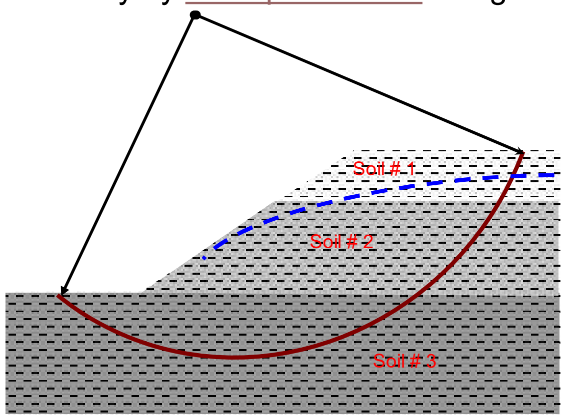
Examples # 1 Solution

No tension crack

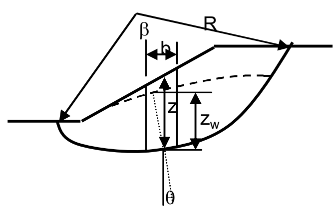
								ESA	ESA	TSA
Slice	b	z	$W=\gamma bz$	z_w	r_u	θ	m_j	$W\sin\theta$	$W(1-r_u)\tan\phi'\, m_j$	$s_u\, b/\cos\theta$
	m	m	kN	m		deg				
1	4.9	1	88.2	1	0.54	-23	1.47	-34.5	38.3	159.7
2	2.5	3.6	162.0	3.6	0.54	-10	1.14	-28.1	54.6	76.2
3	2	4.6	165.6	4.6	0.54	0	1.00	0.0	49.0	60.0
4	2	5.6	201.6	5	0.49	9	0.92	31.5	62.1	60.7
5	2	6.5	234.0	5.5	0.46	17	0.88	68.4	72.2	62.7
6	2	6.9	248.4	5.3	0.42	29	0.85	120.4	80.1	68.6
7	2	6.8	244.8	4.5	0.36	39.5	0.86	155.7	87.6	77.8
8	2.5	5.3	238.5	2.9	0.30	49.5	0.90	181.4	97.5	115.5
9	1.6	1.6	46.1	0.1	0.03	65	1.02	41.8	29.6	113.6
							Sum	536.6	570.9	794.8
								FS	**1.06**	**1.48**

Examples # 2

Slope satiability by Bishop's Method using excel sheet



Examples # 2 Solution



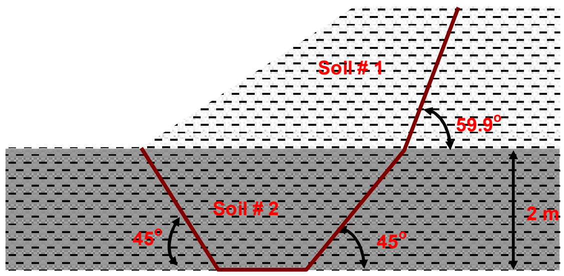
Three soil layers

	Soil 1	Soil 2	Soil 3	
s_u	30	42	58	kPa
ϕ'	33	29	25	deg.
γ_w	9.8			kN/m^3
γ_{sat}	18	17.5	17	kN/m^3
FS	1.01	assumed		

											ESA	TSA
Slice	b	z_1	z_2	z_3	$W=\gamma bz$	z_w	r_u	θ	m_j	$W\sin\theta$	$W(1 - r_u)\tan\phi' \, m_j$	$s_u \, b/\cos\theta$
	m	m	m	m	kN	m		deg				
1	4.9	1	0	0	88.2	1	0.54	-23	1.49	-34.5	39.0	159.7
2	2.5	2.3	1.3	0	160.4	3.6	0.55	-10	1.15	-27.8	53.7	76.2
3	2	2.4	2.2	0	163.4	4.6	0.55	0	1.00	0.0	47.6	60.0
4	2	2	3.6	0	198.0	5	0.49	9	0.92	31.0	59.7	60.7
5	2	0.9	4.1	1.5	226.9	5.5	0.48	17	0.87	66.3	67.6	62.7
6	2	0.8	4.1	2	240.3	5.3	0.43	29	0.84	116.5	74.7	68.6
7	2	0	3.7	3.1	234.9	4.5	0.38	39.5	0.89	149.4	72.6	108.9
8	2.5	0	1.5	3.8	227.1	2.9	0.31	49.5	0.94	172.7	81.1	161.7
9	1.6	0	0	1.6	43.5	0.1	0.04	65	1.19	39.4	23.3	219.6
									Sum	513.1	519.1	978.1
										FS	**1.01**	**1.91**

Examples # 3

Slope satiability by Janbu's Method using excel sheets

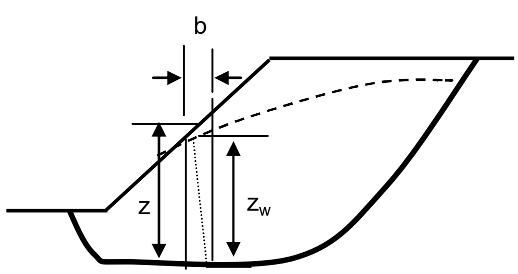


A coarse grained fill was placed on saturated clay. Determine FS if the noncircular slip shown was a failure surface

Examples # 3 Solution

Janbu's method

	Soil 1	Soil 2	
ϕ'	29	33.5	deg.
γ_w	9.8		kN/m^3
γ_{sat}	18	17	kN/m^3
d	4.5		m
l	11.5		
d/l	0.39	f_o	1.06
FS	1.04	ssumed	



Slice	b	z_1	z_2	$W=\gamma bz$	θ	m_j	$W\tan\theta$	$W\tan\phi' \cos\theta\, m_j$
	m	m	m	kN	deg			
1	2	1	0.7	59.8	-45	3.03	-59.8	71.0
2	3.5	2	2.5	274.8	0	1.00	0.0	152.3
3	2	1	4.3	182.2	45	0.92	182.2	65.9
4	2.9	0	2.5	123.3	59.9	0.95	212.6	38.9
						Sum	335.0	328.0
							FS	**1.04**