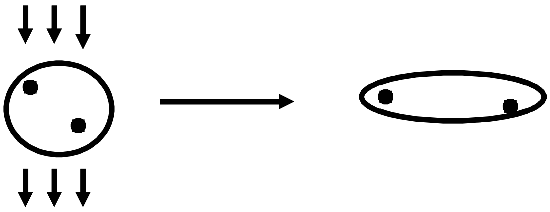
1. INTRODUCTION

▶ Soils ?

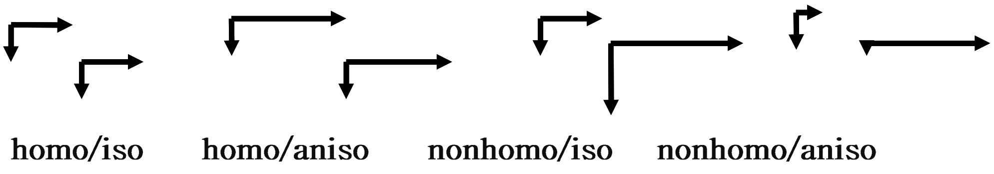
(1) Products of natural weathering (physical /chemical /biological) of rocks.

(2) Uncemented aggregate of minerals comprising 3 different phases (solid phase /aqueous phase /gaseous phase). Cemented Aggregate??
Thus, soil behavior is highly nonlinear/irreversible, and most of all hard to expect!!
Field/lab tests are must. Theoretical Evaluation??
Kansai Airport (Weak soil layer 200m< Settlement Problem, Power Plant-Netherlands-Continuous Pumping,
Need major remediation or reconstruction, What about Inchon Airport?)
Japan Visit (Taisei Corp. (English/Contact Method). Kyoto Univ. DPRI)

(3) Not a rigid body



(4) Some soils are electrically charged (clay surfaces are negatively charged)
(5) Typically, Nonhomogenous / Anisotropic material



※ Homogeneous soil is the one which the physical properties of all its parts are same regardless of its position (global concept)

※ Isotropic soil is the one which has the same physical properties in every direction at a point (directional concept) ex. k, Ko (pp. 67-69 + <u>HW #1 BOX CULVERT</u>)

(6) Time/Environment Dependent
examples : Consolidation, Frost heave (temp./gw/silt), Expansive clays (ex. bentonite-thick diffuse double layer-clay liners, slurry walls, Inchon culvert?? Japan Hojun Bentonite swell in a sea water condition??)

(7) Since 1925, when Karl Terzaghi introduced the concept of soil mechanics into the civil engineering field, the majority of the analysis of soil behavior is based on the physical-mechanical behavior of soil like loading condition. Since soil is very sensitive to local environmental conditions, it cannot be evaluated by a mechanical approach alone. Very little effort is made to study the environmental effects such as physical-chemical-biological on the properties of soil. Soil behavioral analysis is becoming more accurate when the environmental conditions are considered. (ASTM/AASHTO method, distilled water, room temperature, etc.)

Since soil is sensitive to local environments, some additional parameters must be evaluated.

ex. Comparison of Controlling Parameters for Soil Classifications.
Present controlling parameters : Grain-Size Dist., Soil Consistency (LL, PL, PI)
Proposed controlling parameters : pH ($=1/\log_{10}H^{+}$conc.) in pore fluid ($SiOH \rightarrow SiO^{+} + H^{+}$: the higher the pH, the higher the tendency for H^{+} goes into solution, surface gets more negatively charged, the effect of +charge at the edge of the clay soil particle decreases, thicker DDL, dispersed structure, lower k, lower S_u), Ion-exchange Capacity.

Paper Presentation + Writing IN ENGLISH

1. A paper will be given every week.

* Summary and WORD & PHRASE of the paper should be submitted every week.
Copy and verbatim will not be allowed. (10%)
* One student will have a presentation for the paper of the week.

Evaluation will be made by the fellow student and the professor (10%)

2. Writing (Pick one of the geotechnical papers written in Korean. Experimental papers preferred)

Style : ASCE J. of Geotechnical and Geoenvironmental Engineering

Why Study Soil Mineralogy??

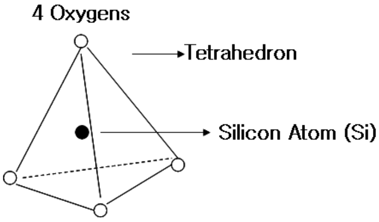
광물학적 성질 → 공학적 성질에 영향

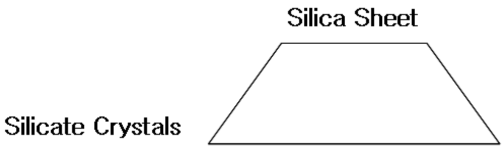
Soil mineralogy is essential to a fundamental understanding of soil behaviors.

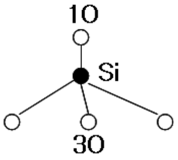
<u>Nonclay Minerals</u> : Sand / Silt (> 2㎛)

quartz (석영, most common)≫feldspar≫mica

Quartz($(SiO_2)_n$ is composed of *silica tetrahedra* grouped in such a way as to form *spirals* with all tetrahedral oxygens bonded to silicon (framework, round shaped).







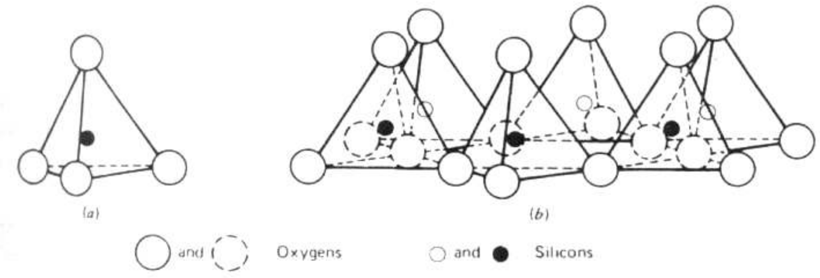


Figure 3.12 Silicon tetrahedron and silica tetrahedra arranged in a hexagonal network.

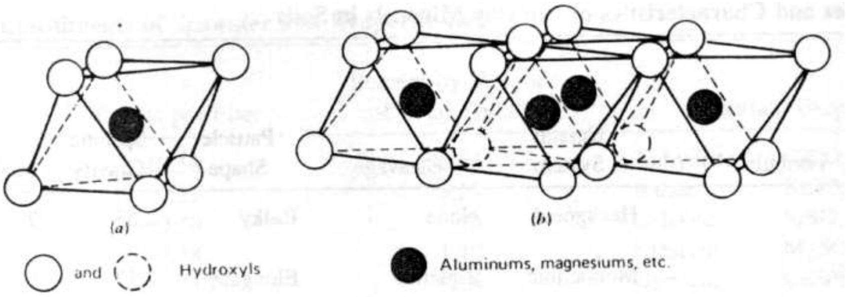


Figure 3.13 Octahedral unit and sheet structure of octahedral units.

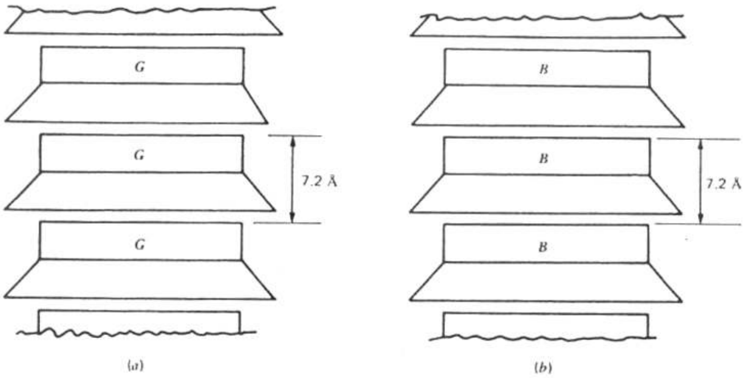


Figure 3.16 Schematic diagrams of the structures of kaolinite and serpentine: (*a*) kaolinite and (*b*) serpentine.

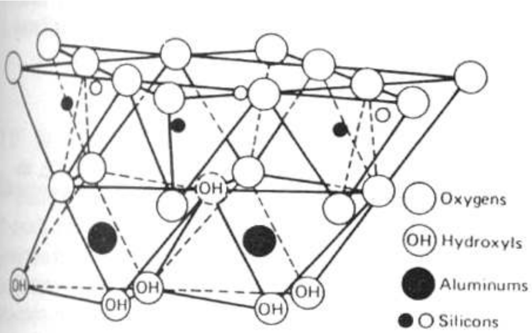


Figure 3.17 Diagrammatic sketch of the kaolinite structure.

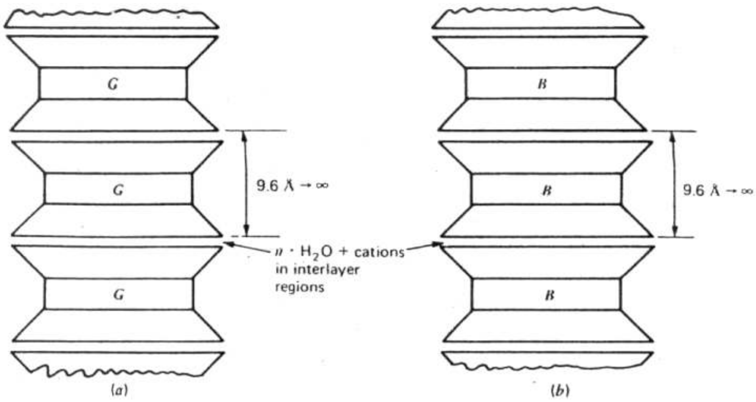


Figure 3.22 Schematic diagrams of the structures of the smectite minerals: (*a*) montmor lonite and (*b*) saponite.

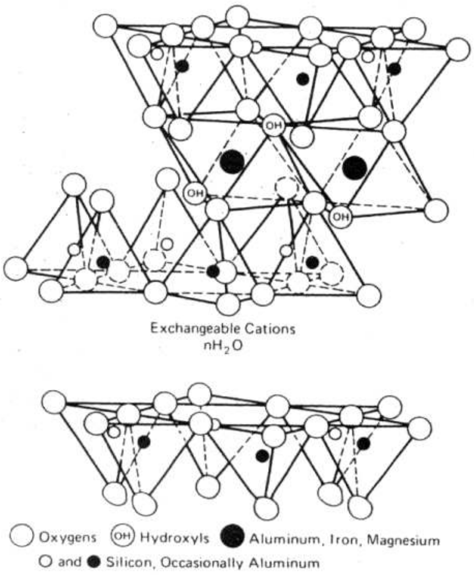


Figure 3.23 Diagrammatic sketch of the montmorillonite structure.

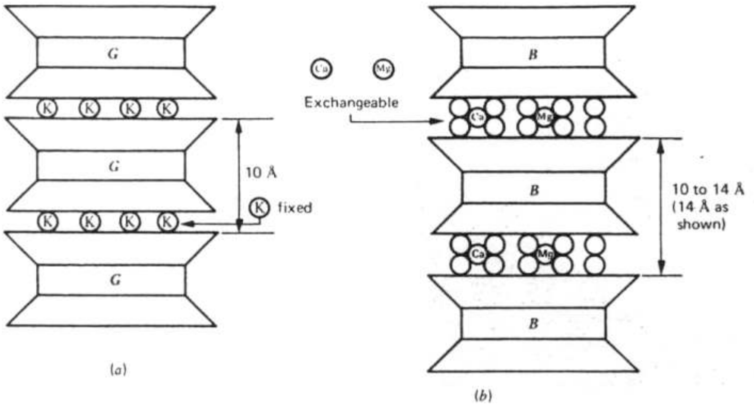


Figure 3.26 Schematic diagram of the structures of muscovite, illite, and vermiculite: (*a*) muscovite and illite and (*b*) vermiculite.

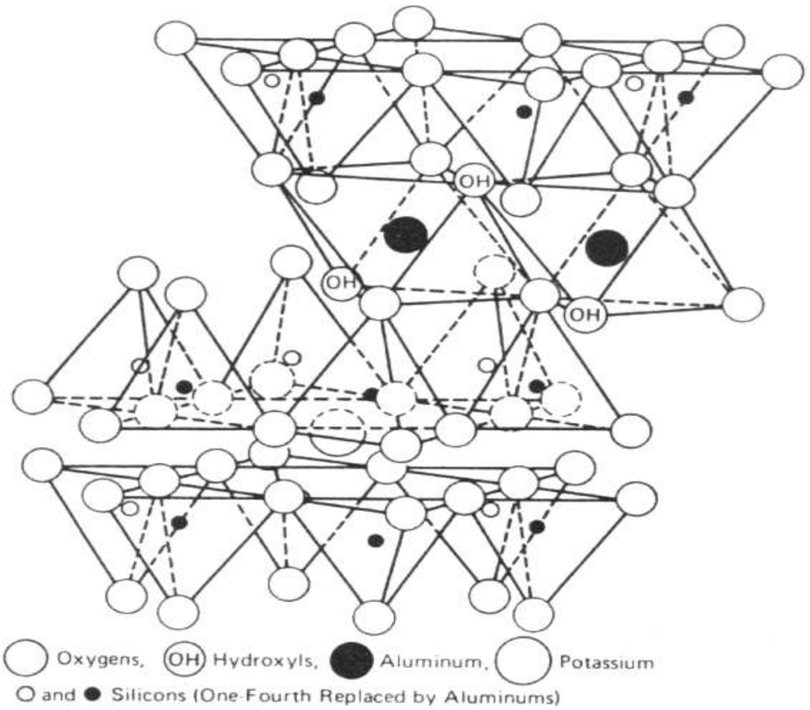


Figure 3.27 Diagrammatic sketch of the structure of muscovite.

The different silicates are classified according to how the Si-O associate with each other.

* *FIG. 2.7 MITCHELL* : The tetrahedral combinations increase in complexity from the top to the bottom, so does the stability (independent island < double < rings < chains < bands < sheet < round shaped framework (quartz))

Framework Silicates (quartz) : All four of the oxygens are shared with other tetrahedra, Already an oxide, No weakly bonded ions (high hardness), Electrically neutral (no interactions with other ions) → Very Stable Structure.

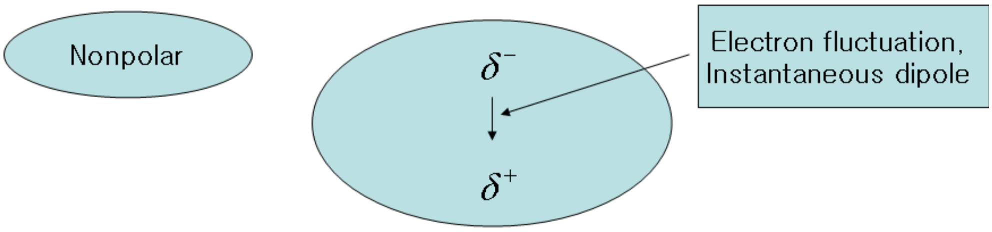
These factors account for the high persistance of quartz in soils.

※ Bonding in Soil Minerals

The interatomic bonds in soil particles are of the strong, primary valence type (ionic and covalent bondings). Quartz is half covalent and half ionic; whereas, usual interparticle bonds among particles are of the secondary valence, hydrogen, or van der waals type, which is relatively much weaker than primary bonds. That means particles are strong compared to groups of particles. Thus, in most cases, soil masses behave as assemblages of particles in which deformation processes are dominated by displacements between particles and not by deformations of particles themselves.

*Hydrogen Bonds : H^+ attracts the negatively charged(dominant for strongly electronegative atoms such as O and F) ion of the adjacent molecule. Ex. O-H for Alumina Octahedral Sheet and Silica Tetrahedral Sheet of Kaolinite. Water Molecule + B.P. Double Layer Water w/ surface of clays.

*Van der Waals Bonds : Weak electrostatic force b/w atoms, ions, and molecules that are caused by electron fluctuations that produce instantaneous dipoles.



O^- O^- O^- O^- O^- VDW bonds-Adsorption

The feldspars are silicate minerals with a 3-D framework structure in which part of the silicon is replaced by aluminum. The excessive negative charge resulting from this replacement is balanced by cations such as potassium(K^+), calcium(Ca^{++}), sodium(Na^+), strontium(Sr^{++}), and barium(Ba^{++}). As these cations are relatively large, their coordination number is also large. This results in an open structure with low bond strengths between units. Consequently, there are cleavage planes, the hardness is only moderate, and the feldspars are relatively easily broken down.

Mica has a sheet structure composed of tetrahedral and octahedral units. Sheets are stacked one on the other and held together primarily by potassium ions that provide an electrostatic bond of moderate strength. In comparison with the bonds within layers, however, this bond is weak, which accounts for the perfect basal cleavage of mica.

성분	상대치 (중량)
SiO_2	100
Al_2O_3	26
Fe_2O_3	12
MgO	6
CaO	9
Mg_2O	5
Na_2O	6
Cl	거의 없다.
SO_3	거의 없다.

주요 무기 성분의 지각중 존재량(토양오염개론, pp41)

Clay Minerals : Microscopic particles (size $< 2\mu m$), flake-shaped, having negative electrical charge which is balanced by the adsorption of cations from solution, showing plasticity when mixed with water.

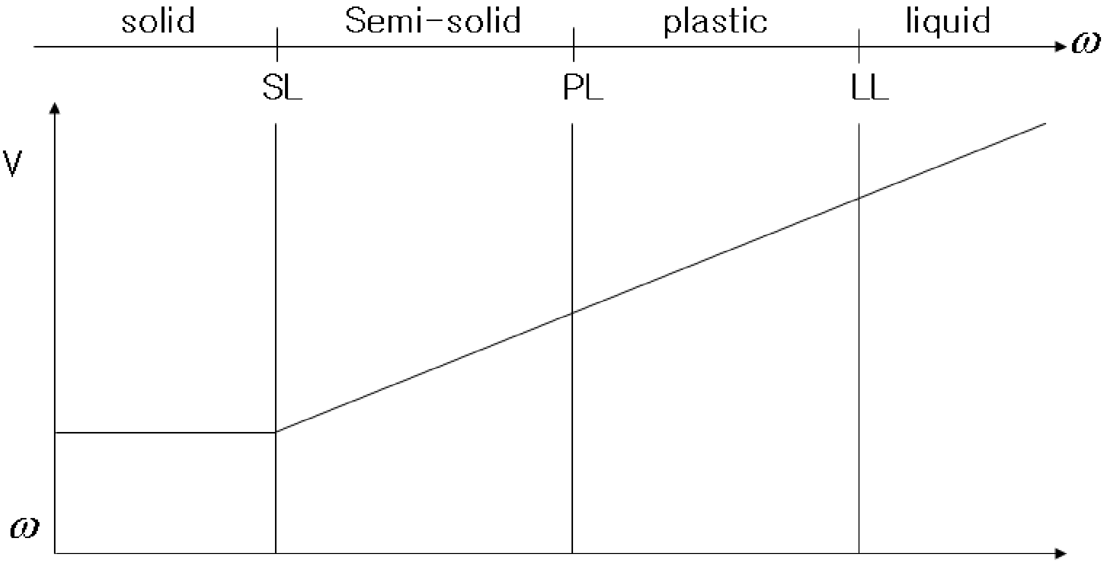
Clay minerals are formed by various combinations of the *tetrahedral*(silica) and *octahedral*(aluminium or magnesium) sheets. The different clay mineral groups are characterized by the stacking arrangement of sheets of these units and the manner in which two successive two- or three-sheet layers are held together. (Fig. 4.1 Fang, pp. 80)

※Plasticity of fine-grained soil is determined by Atterberg limits test, which is used for describing the consistency (stiffness/firmness) of the soils w/ varying degree of water content, then used for classifying the soil (w/ GSD) of engineering purposes (as pavement materials or CBR, frost vulnerability, shrink/swell, drainage, compaction characteristics, dry unit weight, and subgrade reaction coefficient)

Also used for,

* Swell potential, $S_p = 2.27 + 0.131LL - 0.27\omega$

Soils w/ random fabrics (generally at the dry side of optimum having greater deficiency of water) swell more than soil w/ oriented fabrics. Disturbance or remolding of oriented fabrics clay may increase the amount of swelling. $S_p > 25\%$ very high, $S_p < 1.5\%$ low.



* Relationship to stress history,
$LI = (\omega - PL)/PI$, $LI \geq 1$ (quick), $0 \leq LI < 1$ (NC), $LI < 0$ (OC)

Mineral	Activity, Mitchell, pp.186.
Smectite	1-7
Illite	0.5-1
Kaolinite	0.5

* Activity, A = PI/%clay, The higher A of a soil, the more important the influence of the clay fraction on properties and the more susceptible their values to changes in such factors as type of exchangeable cations and pore fluid composition. More sensitive to the environmental changes.

* C_c (compression index) = $0.009\,(LL-10)$

Soil Type(Mit.pp.184)	LL	e at LL	k (10^{-7}cm/sec)
Bentonite	330	9.240	1.28
Bentonite+sand	215	5.910	2.65
Natural marine soil	106	2.798	2.56
Air dried marine soil	84	2.234	2.42
Oven dried marine soil	60	1.644	2.63
Brown soil	62	1.674	2.83

*LL Test is analogous to a dynamic shear test. LL for all fine-grained soils corresponds to shearing resistance of about 1.7-2.0 kPa and a pore water suction of about 6 kPa. Values of hydraulic conductivity at the LL for several clays are given in Table (Mitchell pp.184).

Although the water contents and void ratios at the LL for the different clays vary over a very wide range, the k is very nearly the same for all of them. This means that the effective pore sizes controlling fluid flow must be about the same for all the clays. The approximately equal strength, pore water suctions, and k for all clays at the LL can be explained by the concepts that (1) the aggregates or clusters are the basic units that interact to develop the strength; i.e. the aggregates act somewhat like single particles, and the average size of intercluster pores is the same for all clays, *Fig.10.17 Mitchell*, (2) the average adsorbed water layer thickness is about the same for all particle surfaces as all clays have essentially the same surface structures. The greater the specific surface, the greater the total amount of water required to satisfy the conditions at the LL.

입자가 뭉쳐서 덩어리(cluster)로 작용한다는 가정 하에서(그림10.17 Mitchell), LL에서는 점토는 거의 유사한 구조(pore size, cluster arrangement 등)를 형성하고 있다. 따라서 투수계수가 유사하게 나타나고, 강도도 유사하게 나온다. 또, 점토의 입자형성구조가 비슷한 상황에서(aluminum, magnesium octahedral + silica tetrahedral), 같은 단위 표면적당 흡수하는 물의 양이 비슷하게 된다. 따라서 입자 비표면적이 큰 montmorillonite는 LL에서 더 많은 물을 흡수하게 되고 이는 큰 LL으로 나타나게 된다.

Soil Type (Mit.39)	Specific Surface (m^2/g)	LL(Das.pp.50)
Montmorillonite	700-840	100-900
Illite	65-100	60-120
Kaolinite	10-20	30-110

* The plastic limit has been interpreted as the water content below which the physical properties of the water no longer correspond to those of free water(Terzaghi, 1925) and as the lowest water content at which the cohesion between particles or groups of particles is sufficiently low to allow movement, but sufficiently high to allow particles to maintain the molded positions.

Clay Minerals Structures (Notes)

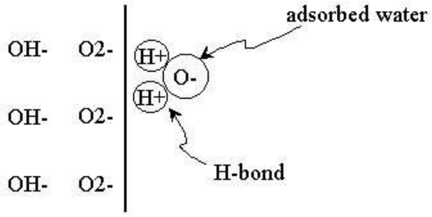
* Isomorphous Substitution (동형치환)
Substitution of one element for another w/ no change in the crystalline form. There is IS of Mg^{2+} and Fe^{2+} for Al^{3+} in the octahedral layer in montmorillonite (1 in 6) (Al^{3+} for Si^{4+} in illite (1 in 7)) to balance exchangeable cations. In kaolinite, there is very small amount of IS. One possibility is Al^{3+} repalcing Si^{4+} in the silica sheet (1 in 400).

만약 montmorillonite에 K^+를 제공하면 illite로 구조가 바뀔 수 있을까?? 즉 지반의 개량 (swelling 등)이 가능할까?

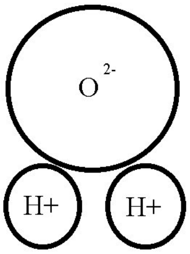
water in soil→interact with each other because water molecules are strongly attracted to and adsorbed on soil particle surfaces. These interactions influence the physical and physicochemical behavior of soil matrix

※ How is water adsorbed on the surface of the clay particle

①



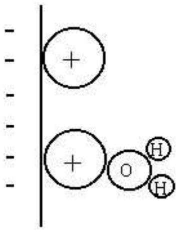
②



water ; dipole molecule two separate centers of charge.

electrostatically attracted to the clay surface with cation in clay

③ cations ; generally hydrated to some-extent important at low water contents



④ Attraction by Osmosis

IS(Isomorphous substitution) → crystal wants to be electrically neutral

→ cations in the water may be strongly attracted to the clay (different clay have different charge deficiencies and thus have different tendencies to attract the exchangeable cations[1])

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[1] one cation can be easily exchanged with other cations

Higher valence cations easily replace cations of lower valence. For ions of same valence, the larger ions the greater the replacement power.

Li+ < Na+ < K+ <<Mg++ < Ca++ << Al+++

※ Practical application

Na-Mont(shows high swell potential) stabilized by lime(Ca(OH)2)

Ca2+ replaces Na+ swelling of Na-Mont significantly reduced by the addition of lime.

Ion exchange reacition and Ion exchange capacity

A clay will adsorb cations of specific type and amount

cation ex - dominant Ca2+, Mg2+, H+, K+, NH4+, Na+

anion ex - SO42-, Cl-, PO43-, NO3-

Why exchange?

①IS ②Broken Bonds ③Replacement

Geotechnical point of view;

Ion exchange

① can cause changes in the soil-water structure from dispersive to flocculative structure or V.V.

② can change the water behavior, such as change hard water into soft water by the removal of Ca++ and Mg++ ions.

ex. Ion exchange

Na_2Z[2] + Ca[3]SO_4[4] → CaZ[5] + Na_2SO_4[6]

③ IE Capacity (IEC)

Maximum quantity of cations that the soil is capable of adsorbing from a neutral

---

[2] sodium zeolite

[3] very active displacing agent

[4] causing hard water

[5] calcium zeolite

[6] sodium sulfate

solution(meq/100g soil)

④ Why? happen?

ⅰ) IS

$Al^{3+} \rightarrow Si^{4+}$ silica sheet

$Mg^{2+} \rightarrow Al^{3+}$ octahedral sheet

Balancing cations are attracted to cleavage surface

Dominant process for illite + Mont

ⅱ) Broken Bonds

Exchange siltes may be present along particle edges and on non-cleavage surfaces. Major process for Kaolinite. For a given clay material the important of broken bond sites increases with decreasing particle size.

ⅲ) Replacement

The hydrogen of an exposed hydroxyl is replaced by another type of cation.

⑤ Factors Affecting Ion-Exchange Capacity.

ⅰ) Paricle size

CEC of clay minerals increases as the particle size decreases (not for Montmorillinite)

ⅱ) Specific Surface Area

The smaller the particle, the larger the specific surface, the larger the CEC[7]. Grinding(decrease particle size) causes an increase surface and increase the number of broken bonds → CEC increases

ⅲ) Temperature

CEC is reduced on heating[8]

Why? Thermal Energy make cations diffuse away from the clay surface

ⅳ) Ion size[9]

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[7] Fig.4.8 p96 Fang

[8] Fig.4.9, p97, Fang : The CEC is reduced or heating but the reduction is not uniform and various with the cation present

[9] Table 4.5 p97, Fang

If ions do not differ in size by more than 15% and in valency by no more than one unit, they can substitute for one another in the silicate minerals, Na^{+} and Ca^{2+} or Mg^{2+} and Fe^{2+} can replace on another completely.

ⅴ) PI

Linear relationship b/w PI & CEC is obtained fro most soils[10]

※ CEC↑ retardation for contaminant migration↑

Specific Surface (m^2/g)

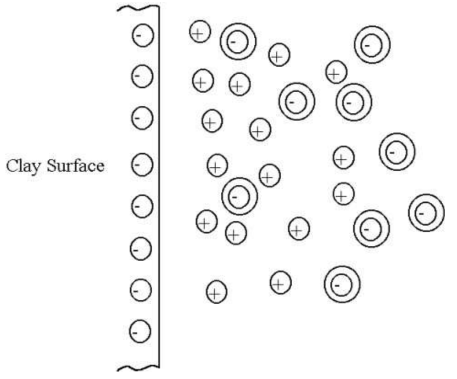
Kaolinite	10~20		
Illite	65~100		
Montmorillite	50~840	→	swell/shrink more

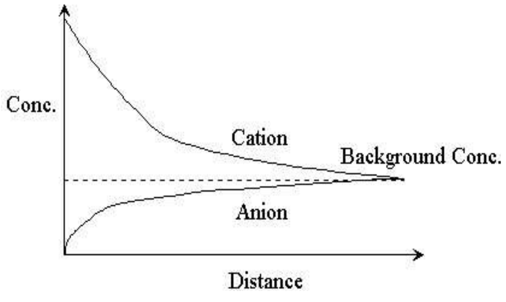
Diffusive Double Layer

In a dry clay, adsorbed cations as salt precipitates the negatively charged clay surfaces

In water, the cation salts go into solution. Because the adsorbed cations are responsible for a much higher concentration near the clay surface, there is a tendency for them to diffuse away in order to equalize concentration throughout[11]

The escaping tendency due to diffusion, and the opposing electrostatic attraction lead to a cation distribution adjacent to clay particle in suspension.





The negative surface nad the distributed charge in the adjacent phase are together termed as the "Diffuse Double Layer"

---

[10] Fig.4.10 p98, Fang

[11] thermal energy makes them diffusing away from the surface

Also water molecular are held by cations in the double layer and hydrogen bonding to the clay surface → double layer water.

Ion Distribution by Gouy-Chapman Theory

Assumption

① Ions in the double layer exist as having point charges, and no interactions among them

② Charge on the clay particle surface is uniformly distributed

③ The particle surface is a plate that is large relative to the thickness of the double layer(1D condition)

④ The static dielectric constant of the medium is independent of position→dielectric constant of the water in clay is 80(at 20℃) independent of position.

※ Dielectric constant : measure of the ease with which molecules can be polarized and oriented in an electric field

Quantitatively $F = \frac{QQ'}{Dd^2}$

or the ratio of the electrostatic capacity of plates separated by the given material to that of the some plates with vacuum b/w the plates.

From Boltzman equation

Ion concentration $n_i = n_{oi}(\coth(0.16\, v_i \sqrt{C_0}\, x))^2$

n_i ; ion concentration of ions of type i (ions/L)

n_{oi} ; bulk ion concentration of ion i conveniently taken at a large distance from the surface

v_i ; ionic valence of ion i

C_0 ; solution concentration (M)

x ; distance from the surface (Å)

Thickness of DDL

$$\frac{1}{k} = \left(\frac{DkT}{8\pi n_0 \varepsilon^2 v^2}\right)^{\frac{1}{2}}$$

where D ; Dielectric constant

k ; Boltzman constant (1.38×10-16 erg/。K)

T ; Temperature(。K)

n_0 ; ion concentration at bulk state (ions/㎤)

ε ; unit electronic charge (16×10^{-20} coulomb or 4.8×10^{-10} esu[12])

v ; ionic valence

ex. Compute the concentration of Na^+ at a given distance from the surface of the domain.

$n_0 = 6.02\times10^{16}$ ions/㎤

$v = +1$

$C_0 = 0.0001M$

$T = 20℃ = 293$。K

① $n : x$

$$n = 6.02\times10^{16}(\coth(0.16\sqrt{0.0001}\,x))^2 \text{ (ions/cm}^3\text{)}$$

x(Å)	n (ions/㎤)
5	9.4×10^{20}
10	2.4×10^{20}
30	2.5×10^{19}
60	1.6×10^{18}
91	2.8×10^{18}
200	6.3×10^{17}
305	2.9×10^{17}
500	1.4×10^{17}
1000	7.1×10^{16}

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[12] 1esu = 2.9979V

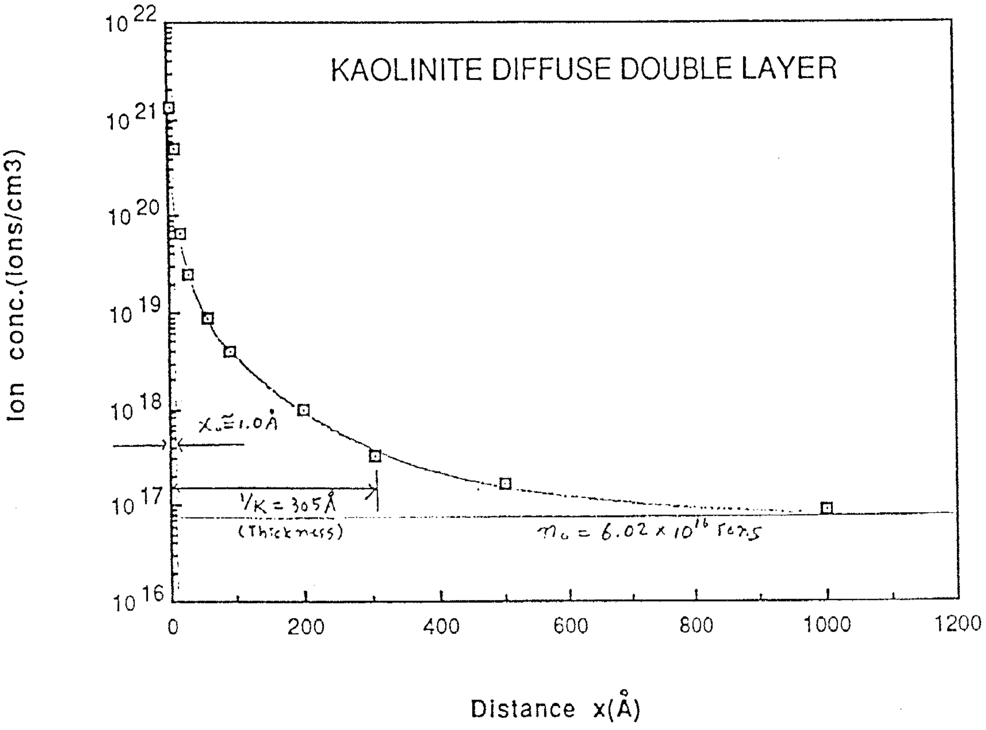


FIG.2 SODIUM ION DISTRIBUTION IN THE DOUBLE LAYER

② DDL thickness

$$K^2 = \frac{8\pi n_0 \varepsilon^2 v^2}{DkT} = \frac{8\pi \times 6.02 \times 10^{-16} \times (4.8\times10^{-2})^2\times1^2}{80\times1.38\times10^{-16}\times293}$$

$$K = 3.20 \times 10^5 \text{ cm}^{-1}$$

$\therefore$ 1/K = 3.05 × 10^{-6} cm = 305 Å

If there is only one kind of ion in the pore fluid, it v is doubled (+1 → +2). "v" becomes 2v, and if n_0 is also doubled. n_0 becomes $2n_0$, then the original thickness becomes

$$\frac{X_{new}}{X_{old}} = \left(\frac{\dfrac{DkT}{8\pi 2n_0 \varepsilon^2 4v^2}}{\dfrac{DkT}{8\pi n_0 \varepsilon^2 v^2}} \right)^{1/2} = \left(\frac{\frac{1}{8}}{1} \right)^{1/2}$$

$$= \left(\frac{1}{8}\right)^{1/2} = 0.354$$ (*DDL reduced*) → *k* ? ↑: 지반개량효과 증가

In general the thicker the DDL, particles in suspension to be dispersed

\+

get swell more

\+

k (hydraulic conductivity) decrease

※ Quick clay

Formed in sediment in marine environment. (high c), (thin DDL), (less repulsion).

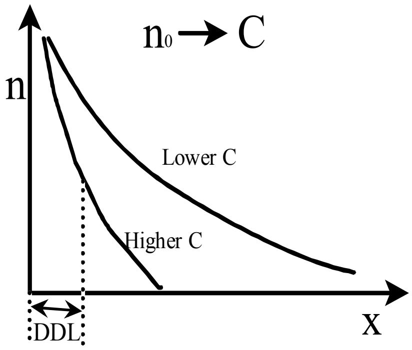
Flocculated structure(higher strength)

⇓

leached by freshwater(low c) (thicker DDL)

DDL 증가 → dispersed 구조로 바뀌려는 경향

leaching만으로 실제구조가 바뀌지는 않지만 disturbance에 의하여 바로 dispersed 구조로 바뀜



$$1/K = \left(\frac{DkT}{8\pi n_0 \varepsilon^2 v^2} \right)^{1/2}$$

The tendency towards flocculation (DDL 두께 얇다) will depend on increasing one or more of the following.

- **concentration of the electrolyte n_0**
- **valence of the ion v**
- **Temperature** ↑ 1/K ↑ ? but D also ↓, DT ↓ ∴ 1/K ↓

decreasing are or more of the following $1/K \propto D^{1/2}$, D ↓ 1/K ↓

- **dielectric constant of the pore fluid**

  $D_{water} = 80$, Organic comp. $D_{benzen} < 35$ → reduced DDL

  ⇒ increase hydraulic conductivity

- **size of the hydrated ion**

  $$\text{Li}^+ \overset{\text{size}}{>} \text{K}^+$$

  7.3 - 10Å 3.8 - 5.3Å : hydrated radians

  ↓ ↓

  DDL thicker DDL thinner floc.

  ↑

  the larger the ion size the thicker the layer required to accommodate the necessary number of cations have the greater the repulsion.

- **pH**

  $$\text{SiOH} \xrightarrow{H_2O} \text{SiO}^- + \text{H}^+$$

  The lower the pH the lower the tendency for H^+ go into soil.

  $$pH = \frac{1}{\log_{10} H^+ \text{conc.}}$$

  pH < 7 : acid, high H^+ conc. less OH dissociation → flocculated

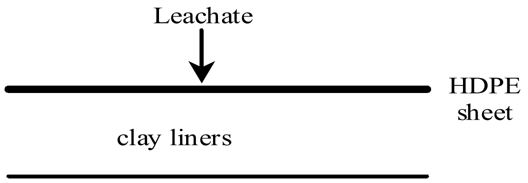
  pH > 7 : 이온화 더됨 → 더 표면이 negative charge → dispersion

- **anion adsorption**

  esp. phosphote (PO_4^{-3}) at the edge → disp. agent 적어야 flocc.

For Neutral Organics

That's why we install.

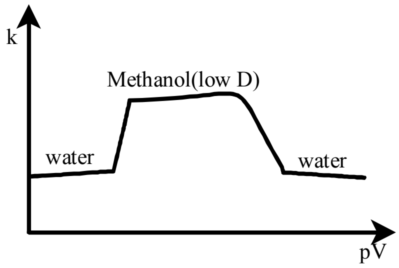


Low D

say Benzen, Aniline (D<35)

→ results in reduced DDL

→ increase k



※ Cohesive soil structure

When two clay particles in suspension come close to each other.

입자사이의 작용력은

rep > att

- DDL interpenetration (bumping) → repulsion

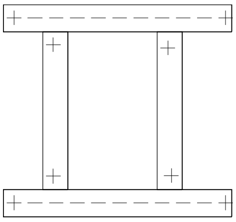
At the same time

- Van der Waal's forces → attraction

<u>very close</u> → attraction > repulsion → flocculated structure

while in suspension ↑

입자자체가 가진 전기적 성질에 의해 지배되어



In dilute suspension (clay is initially dispersed)

→ sedimentation 과정에서 salt가 들어가면 DDL가 얇아짐 → repulsion 감소 → flocc. 되려함

→ repulsion rules, gravity negligible → settle very slow or remain in suspension (Brownian motion) → From DISPERSED STRUCTURE

ex) 바다에서 sed. 된 marine clay는 주로 flocc. 구조

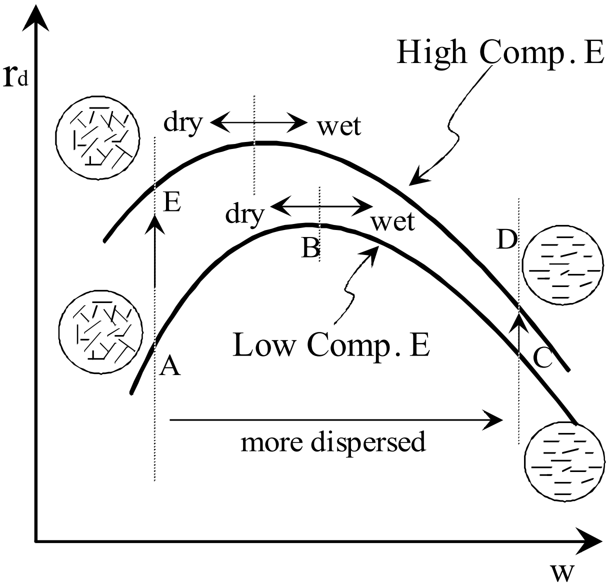
Flocc.	**Disp.**
- high e - marine clay deposit - k가 큼 - 강도 better	- low e - fresh water clay - k가 작다. - 강도 lower

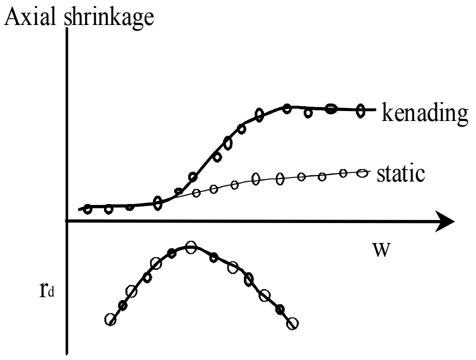
Properties and structures of compacted cohesive soils

Compaction of clay soil = fn (w, type of compaction, type of soil, comp. effort)

↑	↑	↑	↑
Dry & wet soil of optimum	kneading/vibratory/ static	plasticity different soil	compaction E

plasticity ↑, γ_{dmax} ↓





왜 Curve는 ∧ 모양 ?

w : usually water content of compacted soils is referred to the optimum water content for a given type of compaction.

Depending on their position, soils are called dry of optimum at optimum, or wet of optimum. When the soils are compacted dry of optimum, the structure of the soils is essentially independent of the type of compaction. Wet of optimum, however the type of compaction has a significant effect on the soil structure and thus on the strength compressibility etc.

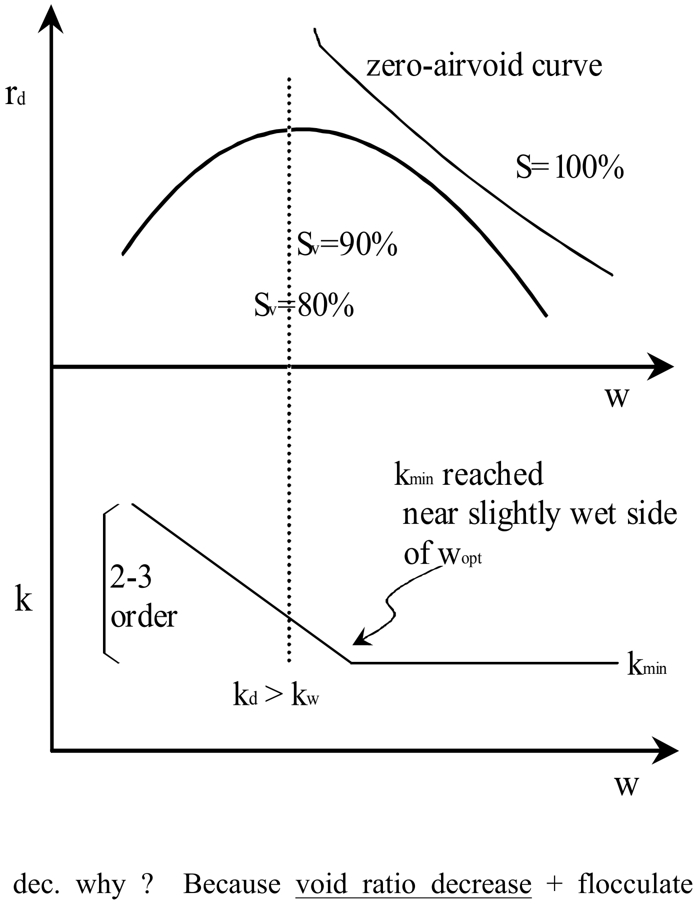
① **structures**

At low w of A , the DDL of ions surrounding the clay particles cannot be fully developed, hence the inter particle repulsion is reduced ⇒ more random particle (flocculated).

As A → C (w increases), the DDL more developed, repulsion increases ⇒ more dispersed.

As C → D or A → E, mechanical E causes the soil structure more dispersed orientation. However particles are closer together → higher γ_d

② **Permeability**

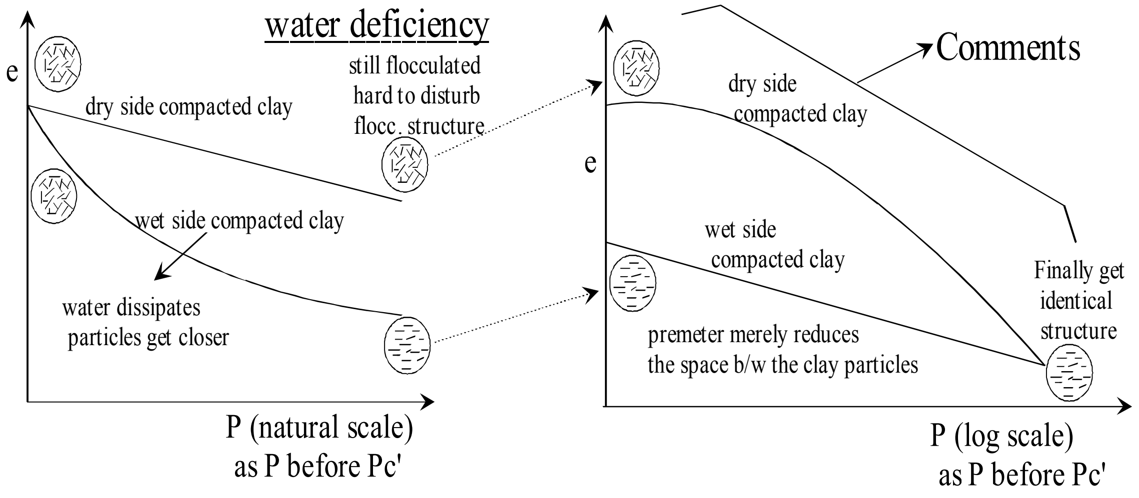


Sat. incr. but k dec. why ? Because void ratio decrease + flocculated (higher k)
(increasing γ_d) ↓
dispersed (lower k)

k_{min} : very important for k sensitive structure
like landfill liner, dam, embankment

③ **Compressibility**

Compressibility of compacted clay is a function of the stress level imposed on the soil mass. At relatively low stress clays compacted wet of optimum are more compressible. At high stress levels, the opposite is true.



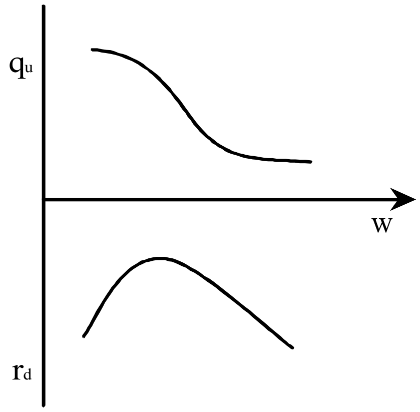
Comments : Pressure tend to orient the particle normal to the direction of application the space b/w the clay particles reduced at the same time structure collapsed to be dispersed structure.

④ **Swelling**

Swelling of compacted clays is greater for those compacted dry of optimum. They have greater deficiency of water, thus have a greater tendency to adsorb water (swell more). This is just opposite for shrinkage effect (wet side shrinks more). Soil at dry of optimum are in general more sensitive to environment to changes such as changes in w.

⑤ Strength

Samples compacted dry of optimum have higher strength than compacted wet of optimum.



pore water p. at failure : wet side higher

stress-strain modulus : dry side much greater

sensitivity : dry side more apt to be sensitive

flocc. $\xrightarrow[\text{remolding}]{}$ disp. : q_u ↓

* Water in Soil

water in soil → · fine grained soils much affected! (plasticity, k, etc.)
· important factors in geotechnical eng. problems
ex. capillarity, shrink/swell, frost action, seepage(through dams), consolidation/liquefaction, stability of slopes/foundation, <u>stability of excavation</u> → show ex.

미국의 경우 swelling의 피해가 Hurricane/Floods/Tornado and earthquake combined 보다 더 큰 경제적 손실을 가져온다.

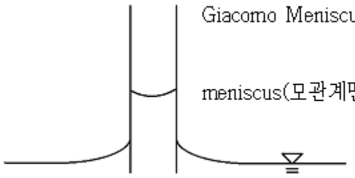
45＋Φ/2 해석법

* Capillarity

· comes from a fluid property known as "surface tension"(→occurs between surfaces of water/soil particle/air)

Giacomo Meniscus:베네치아 불리학자 레오나르도 다빈치 친구

meniscus(모관계면곡면-오목 or 볼록)



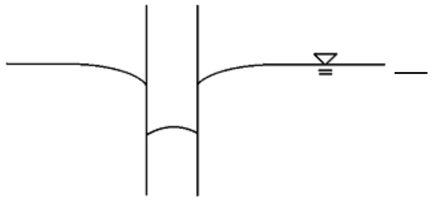
water

Adhesion > Cohesion

b/w glass wall and water

wetting

solid surface

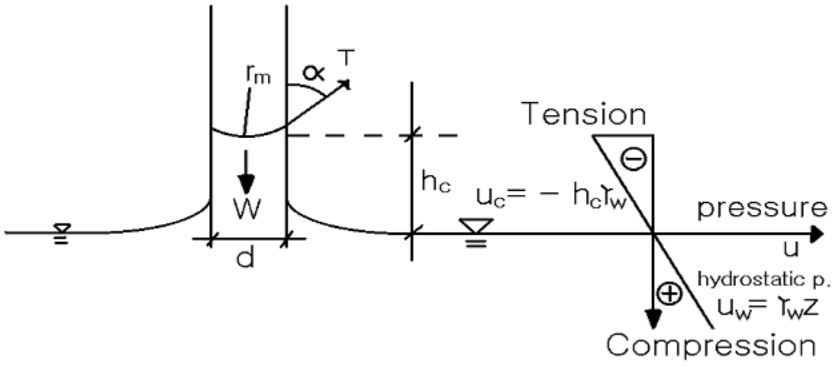


mercury

Adhesion < Cohesion

solid surface

non-wetting



For force equilibrium

$$\Sigma F_{down} = vol \cdot \gamma_w = h_c(\frac{\pi}{4}d^2)\gamma_w$$

$$\Sigma F_{up} = \pi d T \cos\alpha$$

$$\Sigma F_d = \Sigma F_u$$

$$\therefore \quad h_c = \frac{4T}{\gamma_w d} - \cos\alpha$$

For clean glass tube/ pure water

$\alpha \to 0$ $\qquad \therefore h_c = \dfrac{4T}{\gamma_w d}$

For, 20° C, T=73 dynes/cm = 73mN/m = 73×10^{-3}N/m

$$73\times10^{-5}N/cm = 73\times10^{-5}\frac{N}{cm}\times\frac{100cm}{m} = 73\times10^{-3}N/m$$

모관압 $u_c = -h_c\gamma_w = \dfrac{-4T}{d} = \dfrac{-2T}{\gamma_m}$

- Shape of meniscus; Spherical (a minimum energy condition)
- Effective pore diameter ≒ 20% of D_{10}(eff. grain size)

1 atm. abs = 101.325 kPa = 0.76 mHg = 14.696 psi

$$h_c = \frac{-4T}{\rho_w g d} = \frac{-4\times73\times10^{-3}N/m}{9800N/m^3\cdot d} = \frac{3\times10^{-5}m^2}{d(m)}$$

ex) ⓐ If D_{10} = 1μm clay h_c?

1 dyne: 질량1g의 물체에 매초 1cm의
가속도를 생기게 하는 힘 =10^{-5}N

$$h_c = \frac{3\times10^{-5}m^2}{0.2\times10^{-6}m} = 150m$$ (about 500ft.)

ⓑ Capillary pressure

$u_c = h_c\gamma_w = 150m\times1000kg/m^3\times9.81m/s^2 =$ 1500kPa = −15atm = 225psi (tension)

Intergranular (or effective stress σ')

$\sigma' = \sigma - u$ ($\leftarrow \sigma = \sigma' + u$)

At the capillary zone

At the top of the soil column

∴ the tension in the pore water pulls the grains of soil together. The greater the capillary tension, the greater the intergranular contact stress. → higher friction resistance

"Apparent cohesion" not true cohesion

*** Apparent cohesion**

resulting structure → loose but very stable honeycomb structure as long as the capillary meniscus are present

→ bulking

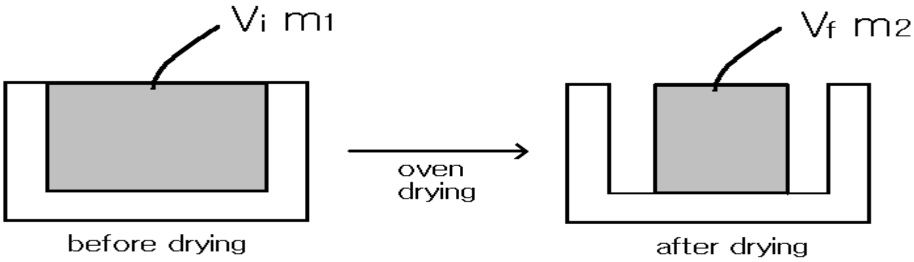
If flooded or disturbed structure destroyed → volume decreased!

If you buy moist sand, you may end up buying a lot of air

*** Shrinkage Limits (SL, w_s)**

A soil sample drying will form capillary menisci between the individual soil grains. As a result, the stresses between the grains(effective stresses) will increase and the soil will decrease in volume.

A point is finally reached where no further volume decrease occurs → Shrinkage limit



measuring V_f by displacing of mercury → non-wetting material → shape maintained
no swell

-문제점
entrrapped air bubble
cracking during drying
mercury poison

$$SL = w_i - \Delta w \quad ---(1)$$

$$w_i = \frac{m_1 - m_2}{m2} \times 100 \quad ---(2)$$

$$\Delta w = \frac{(V_i - V_f)\rho_w}{m_2} \times 100 \quad ---(3)$$

(2), (3)→ (1)

$$\therefore \ SL = (\frac{m_1 - m_2}{m_2}) \times 100 - (\frac{V_i - V_f}{m_2})\rho_w \times 100$$

Shrinkage cracks can occur locally when the capillary pressures exceeded the cohesion or the tensile strength of the soil

- quick clay의 특징 중 하나

If the soil is in a natural undisturbed state, then the shrinkage limit is often greater than the plastic limit(which is meaningless) due to the structure of the soil

SL test in undisturbed state → flocc. large e
PL test in disturbed state → disturbed small e

However for normal clays, soils at the SL will have a very low void ratio because the capillary pressures are so large, much greater than can be achieved by compaction.

ex) SL = 8 (for clays, for Montmorillonite SL≒6)

G = 2.7 (assume S=100%)

$S \cdot e = G \cdot w$

e= 2,7×0.08 = 0.22

$$\gamma_d = \frac{G}{1+e}\gamma_w = \frac{2.7}{1.22} \times 1t/m^3 = 2.21t/m^3$$

$$\gamma_{conc} = 2.4t/m^3$$

Capillary force must be very large to cause soil to become so dense → Very large dry strength

과거의 고문수단: 몸에 fat clay(CH)를 바르고 태양 아래에서 서서히 건조시킴 → shrink → pain

・Shrinkage cracks are zones of weakness which can significantly reduce the overall strength of a soil mass and affect the stability of clay slopes and the bearing capacity of foundation.

The soil mass volume changes resulting from both shrinkage and swell of fine-grained soils are often large enough to seriously damage buildings and highway pavements.

・Soft clays become OVERCONSOLIDATED and less compressible because of the increase in effective stress caused by capillary action

Dessication due to surface drying.

※Mechanisms causing preconsolidation of soils

1) Changes in total stress due to:

removal of overburden (geologic erosion or excavation by man)

past structures

glaciation

2) Changes in pore water pressure due to: σ′ 증가

changes in water table elevation (u감소)

desiccation due to surface drying ← capillary induced(-u 감소)

desiccation due to plant life ← capillary induced(-u 감소)

3) Changes in soil structure due to:

secondary compression (aging)

4) Environmental changes:

pH, temperature, salt concentration → DDL change →flocculated → 강도증가

ion exchange

*** Swelling**

- depends on the clay minerals present in soil

→soil structure, physico-chemical aspects(cationic valence, salt concentration, etc), organics

Montmorillonite > Illite > Kaolinite

(다짐시 건조측이 swell more(강도는 더 좋지만))

Flloc. > Disp

Mono Cat.(DDL thicker) > Di Cat.

(Sodium mon(느슨, 두꺼운 DDL) > Calcium Mon.)

- Swelling의 조건

① presence of Montmorillonite

② natural water content must be around PL

③ there must be a source of water for potentially swelling clay

- 방지책
  - soils compacte wet of optimum and at a lower density shows less tendency to swell (more oriented)
  - Moisture barriers and water proof membranes have been used to prevent water form reaching the swelling soil
  - Chemical stabilization

4. The porous Medium: Hydraulic Conductivity

* In GE →Flow through Porous Medium Important

=>Drainage Problems, Consolidation, Flow into Excavation, Dams, Embankments, GW Development and Management

=>Stability of soil elements depends upon the seepage

=>Need to solve flow problem

* Basic Assumptions (28-1)

* When the hydraulic boundary conditions remain constant with time, steady flow(seepage) condition occurs($\frac{\partial x}{\partial t}=0$). On the contrary transient flow(seepage) will develop as a consequence of apllied loads or any change of the hydraulic boundary conditions with time($\frac{\partial x}{\partial t}\neq 0$).

Flow through Porus Medium → Darcy's Law(1856)

$v_{discharge}=ki$ (k: hydraulic conductivity)

$v_a=nv_{seepage}$ (n: porosity)

For gravel and sand, k is so high that u_e (excess pore pressure), due to any external load, dissipates in a very short time. The consequence is that transient phase can be disregarded most of the time and soil behavior can be analyzed in DRAINED conditions by reffering to the steady state

G.S → u_e by any external load no building up (∵ k high) → Drained condition

steady state

On the contrary,

Silt & Clay → u_e by any load will change with time (∴k low)

(think consolidation)

→ (σ' will also change ($\sigma=\sigma'+u$))

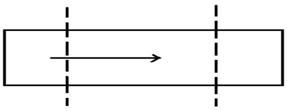
→ Transient state (Undrained Cond.)

Water in Soils

∗ General Assumptions for Flow through Porous Media (soil)

- 1-D flow

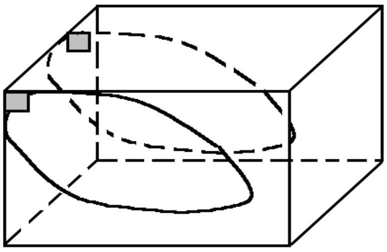
Flow in which all the fluid parameters such as pressure, velocity, temperature, etc., are constant in any cross-section perpendicular to the direction of flow



- 2-D flow

The fluid parameters are the same in parallel plane. whereas in 3-D flow, the fluid parameters vary in the three coordinate direction.

For GE, either 1D or 2D is adequate for most practical cases.



- Incompressible flow

As density changes can be neglected at ordinary stress levels/temperature levels for most GE applications, flow of water in soils can be considered incompressible.

otherwise, soil matrix 부피변화가 물이 빠져나가 흙입자가 재배열되는 것 말고도 물의 부피변화도 가능하므로 이론식이 맞지 않는다.

- Laminar flow

laminar flow, where the fluid flows in parralel layers without mixing or turbulent

($R = \frac{vd\rho}{\mu} < 10$)

For flow in most soils, v is so small that the flow can be considered laminar.

For water in motion in soil matrix

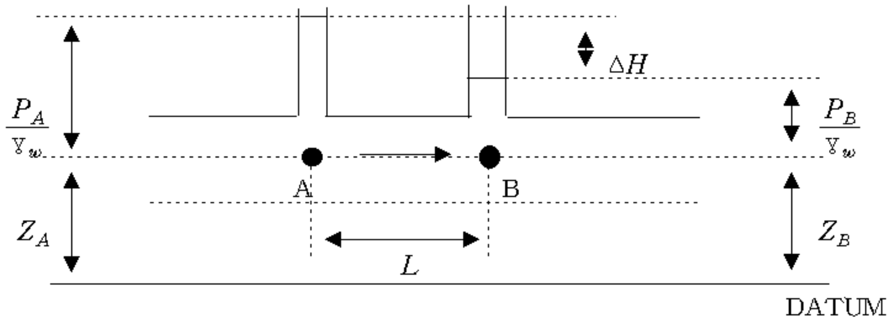
$$E_T = E_{kinetics} + E_{potential} = C$$

Total head $H_t = \frac{P}{\gamma_w} + \frac{v^2}{2g} + z$ (Bernoulli's eq.)

pressure head, velocity head, elevation head

In soil, v is almost negligible

$$\rightarrow H_t = \frac{P}{\gamma_w} + z$$



물은 T.H가 큰 곳으로부터 낮은 곳으로 흐른다.

For non-dissipative incompressible fluid, Head loss ΔH

$$\Delta H = (Z_A + \frac{P_A}{\gamma_w}) - (Z_B + \frac{P_B}{\gamma_w})$$

Head loss/L = i (hydraulic gradient)

$$i = \frac{\Delta H}{L} \quad \rightarrow \quad v_d \propto i \quad \rightarrow \quad v_d = ki \text{ (Darcy's law)}$$

q(flow rate) = vA = kiA

A = cross-sectional area of porous media

Darcy's law applied to the laminar flow($R = \frac{Vd\rho}{\mu} < 1$) condition in porous media applicable to most of the flow through soil media.

exceptions) at very low k → $v = ki^n$ (n=1.5 for swedish clay)

or at very high k, if turbulent flow, $v \neq ki$

▶ **k (hydraulic conductivity)**

- indicator of how well a porous media can transmit water
- k = fn(water properties, soil properties)

= fn(μ, γ_w, S_r, D, GSD, e, C)

For parameters with units

k = fn (μ, γ_w, D)

$$\gamma = [F/L^3] = [Ma/L^3] = [ML^{-2}T^{-2}]$$

$$\mu = [ML^{-1}T^{-1}], \quad D = [L]$$

Since we have n=4 physical variables and 3 basic dimensions of M, L, T.

n-3 = 4-3 = 1 dimensionless group from Buckingham π-theorem

$$k = C\gamma^{X1}\mu^{X2}D^{X3}$$

$$M^oL^1T^1 = c[ML^{-2}T^{-2}]^{X1}[ML^{-1}T^{-1}]^{X2}[L]^{X3}$$

$$\left.\begin{array}{l} [M] : 0 = X_1 + X_2 \\ [L] : 1 = -2X_1 - X_2 + X_3 \\ [T] : -1 = -2X_1 - X_2 \end{array}\right] \rightarrow X_1 = 1, X_2 = -1, X_3 = 2$$

$$\therefore k = C\gamma\mu^{-1}D^2 \rightarrow CD^2\frac{\gamma}{\mu}$$

$$k = ki(\frac{\gamma_w}{\mu})$$

k_i : intrinsic permeability for the properties of porous media alone

$k_i = cD_{10}^2$ → basically a function of the size of openings through which fluid moves

→ valid for $k \geq 10^{-3} cm/s$

c : mainly for describing the shape of the pore spaces (or particles-둥글수록 ↑)

$k_i = [L^2]$

Kozney-Carmen eq. → 유도과정 (31-1,2,3)

$$k = D_{10}^2\frac{\gamma_w}{\mu}\frac{e^3}{1+e}C$$

- Temperature↑ μ ↓ ∴ k ↑ (이 영향이 더 크다.)

  γ_w↓ ∴ k ↓

$$k_{20^\circ C} = (\frac{\mu_{T^\circ C}}{\mu_{20^\circ C}})k_{T^\circ C}$$

- S_r↑ k↑ (bubbles in air packet surface tension → water can flow well)
- D_r ↑ k↑ (for larger pore openings)

  (pore opening ∝ D_r, d ≒ 20% of D_{10})

  GS1) well graded k↓

  poorly graded k↑

- e↑ k↑
- C(shape factor) ↑(getting rounder) k↑

  1 ~ 1.5 (rough to round)

has more surface area → more surface tension to overcome for flow

※ Theoretical Equations for Permeability (K-C eq. derivation effect of S & e)

Poiseiulle's law for flow through a round capillary

$$v_{avg}=\frac{\gamma_p R^2}{8\mu} i_h \ (\mu: \frac{M}{LT,} \ \frac{g}{cm \cdot s}) \qquad ------ \ (1)$$

μ : viscosity, R : tube radius, γ_p : unit wt. of the per~

i_h : hydraulic gradient

Because the flow channels in a soil are of various shapes and sizes, a characteristic dimension is needed to describe average size.

The hydraulic radius is useful.

$$R_H=\frac{flow\ channel\ cross-\sec tion\ area}{wetted\ permeater}$$

For circular tube flowing full

$$R_H=\frac{\pi R^2}{2\pi R}=\frac{R}{2}$$

So Poiseiulle's equation becomes

$$q_{circle}=v_{avg}a=\frac{1}{2}\frac{\gamma_p}{\mu}R_H^2 i_h a \qquad ------- \ (2)$$

a : cross-sectional area of the tube

For other shapes of cross-section, an equation of the same form will hold, differing only in the value of a shape factor C_S, so that

$$q=C_s\frac{\gamma_p}{\mu}R_H^2 i_h a \qquad ----- \ (3)$$

For a bundle of parallel tubes of constant but irregular cross-section contributing to a total cross-sectional area, A(solid plus voids), the area of flow passages, A_f filled with water is

$$A_f=SnA \qquad ------ \ (4)$$

S : degree of saturation, $\frac{V_w}{V_v}$

n : porosity, $\frac{V_v}{V}$

$$e=\frac{V_v}{V_s} \quad \therefore \ V_v=eV_s \qquad V_vS=eV_sS=V\frac{V_w}{V_v}$$

$$\therefore \ V_w=eV_sS \ \rightarrow \ R_H=\frac{eV_sS}{S_o} \qquad ------ \ (6)$$

Then equation(3) becomes

$$q=C_s\frac{\gamma_p}{\mu}R_H^2 i_h SnA=C_s\frac{\gamma_p}{\mu}R_H^2 S(\frac{e}{1+e})i_h A \qquad ------ \ (7)$$

and substitution for R_H gives

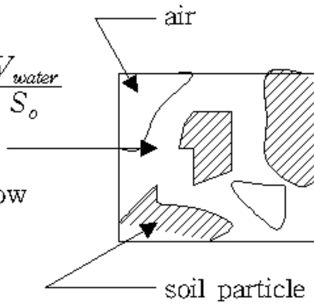
For this condition, R_H is given by

$$R_H = \frac{A_f}{P} = \frac{A_f L}{PL} = \frac{\text{Volume available for flow}}{\text{wetted area}} = \frac{V_{water}}{S_o}$$

P : wetted perimeter

L : Length of the flow channel in the direction of flow

S_o : specific surface per unit vol. of particles(L^2)



For void ratio, e and vol. of solids V_s, the vol. of water is

$$q = C_s \frac{\gamma_p}{\mu} V_s^2 \frac{1}{S_o^2} S^3 (\frac{e^3}{1_e}) i_h A$$

$$\therefore k = C_s V_s^2 \frac{\gamma_p}{\mu} \frac{S^3}{S_o^2} (\frac{e^3}{1+e}) \text{ ------(8)} \Rightarrow \text{Kozeny-Carman Eq.}$$

$$[\leftarrow k = C_s D_{10}^2 \frac{\gamma_w}{\mu} \frac{e^3}{1+e} S^3]$$

▶ K-C equation works well for k in uniformly graded sands and some silts, serious discrepaucies are found in clays because fabrics of clay materials do not contain uniform pore sizes.

Poiseiulle's law for flow is assumed through a round capillary which is not right for clay particles.

▶ K-C equation predicts that the hydraulic conductivity should vary directly with the cube of the degree of saturation(S) and experimental data support this.

※ Inherent Errors for estimating k in lab test(permeameter test)

① Soil is disturbed to somewhat extent

② Orientation of the in-situ stratum to the flow is not always duplicated

$$(\frac{k_h}{k_v} \geq 3 \text{ for } sand)$$

③ Boundary conditions are not the same. (fixed wall의 단점)

④ $i_{field} = 0.5 \sim 1.5$

$i_{lab} \geq 5$ (washing fine particles, k↑ turbulent flow made)

⑤ v is not always ki, sometimes $v = ki^n$ (non-linear)

⑥ The entrapped air bubbles on the lab sample will affect test results. Even for small bubbles since the lab sample is small.

⑦ Temperature Variations, especially for long-term test

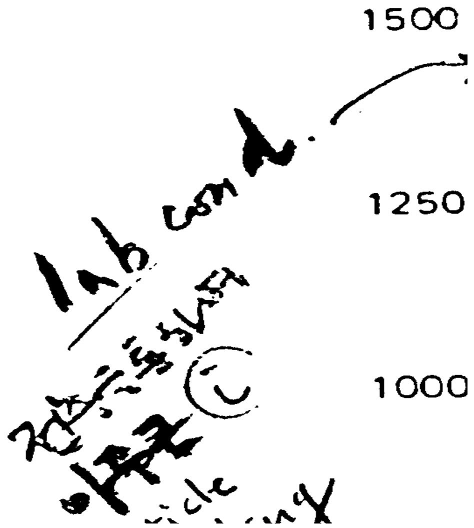
How to measure k?

- constant/variable head permeameter test
- consolidation test $k = c_v m_v \gamma_w$

- in-situ well tests

※ $i_{lab} > i_{field}$ (Fig 15.7 MIchell pp.352)

i vs T (time factor) during consolidation according the Terzaghi theory



pore water pressure by Terzaghi's 1D consolidation theory

$$u = \sum_{m=0}^{\infty} \frac{wu_o}{M}(\sin\frac{M_Z}{H})e^{-M^2T}$$

where M = $\pi\frac{(2m+1)}{2}$

Thus, i is

$$i = \frac{\delta}{\delta z}(\frac{u}{\gamma_w})^h = \frac{\delta u_o}{\gamma_w H}\sum_{m=0}^{\infty}\cos(\frac{MZ}{H})e^{-M^2T}$$

If a parameter p is defined by

$$p = 2\sum_{m=0}^{\infty}\cos(\frac{MZ}{H})e^{-M^2T} \rightarrow i = \frac{u_o}{\gamma_w H}p$$

For low values of $\frac{u_o}{\gamma_w H}$ such as would exist in the field, for example,

$u_o = 0.5kg/cm^2$, h=5m then $\frac{u_o}{\gamma_w H}$ = 1

the filed gradients are low throughout most of the layer thickness during the entire consolidation process.

For lab sample of 1cm thickness and the same stress, a case would be the cause of substantial discrepancie between lab and field values of coeff. of consolidation.

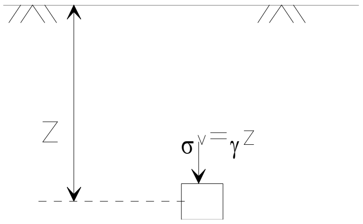
($\frac{u_o}{\gamma_w H}$ is 500, and i are very large)

If k is very much i-dependent, soil including small particles, not participating in the load-carrying function of a soil mass are likely to be moved under higher values of i, then the k_{lab} value is much higher than filed condition.

For soils composed of particles easily washed, constant rate of strain testing(i가 급작하게 변하지 않는 실험) would be preferable to the use of conventional load increment consolidation tests.

＊ Effective stress

· Vertical Stress



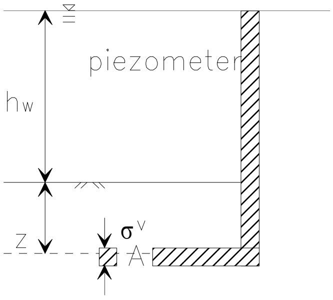
$\sigma_v = \gamma z$ γ : 흙의 단위중량

z : 고려하는 토층까지의 깊이

일반적으로는 각 토층의 성질이 다를 수 있으므로

$$\sigma_v = \sum_i^n \gamma_i z_i$$

※토층이 물 속에 존재하면 (no seepage의 경우)



총연직응력

$$\sigma_v = z\gamma_{sat} + h_w\gamma_w$$

γ_{sat} : 포화 단위중량

그러나 A는 hydrostatic의 상향력도 받으므로 이들의 piezometer에서 $z + h_w$만큼 물을 밀어올렸다. → 간극수압 u

($\gamma_w = 1000\,\text{kg중}/m^3 = 1\,t/m^3$)
$= 9.81\,kN/m^3 = 62.4\,lb/ft^3$, pcf

$$u = h_w\gamma_w + z\gamma_w$$

$1\,kg \cdot m/sec^2 = 1\,N$

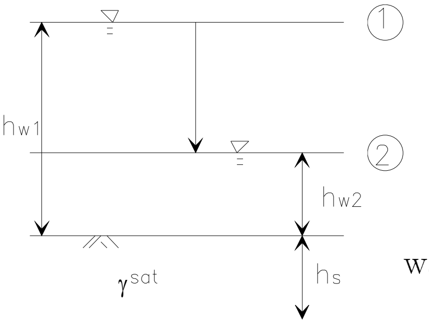
$9800\,N/m^2 = 1000\,\text{kg중}/m^2$
$9.81\,N = 1\,\text{kg중}$
$= 1\,kg \times 9.81\,m/sec^2$
$= 9.81\,kg\,m/sec^2$

따라서 A 점에서 흙의 한 입자가 실제 받는 응력은

→유효응력 (effective stress)

$$\overline{\sigma_v} = \sigma_v - u = z\gamma_{sat} + h_w\gamma_w - h_w\gamma_w - z\gamma_w = z(\gamma_{sat} - \gamma_w) = z\gamma_{s.ub}$$

전 토층이 물속에 존재하면(전부 부력을 받는 상태) 유효응력은 h_w와는 관계없다.



water level down from ① → ②

$$
\begin{aligned}
\sigma_{P\text{①}}' &= h_{w1}\gamma_w + \gamma_{sat}h_s - (h_{w1} + h_s)\gamma_w \\
&= (\gamma_{sat} - \gamma_w)h_s = \gamma_{s.ub}h_s \\
\sigma_{P\text{②}}' &= h_{w2}\gamma_w + h_w\gamma_{sat} - (h_{w2} + h_s)\gamma_w \\
&= (\gamma_{sat} - \gamma_w)\gamma_{s.ub}h_s
\end{aligned}
$$

$$\underline{\sigma_{P\text{①}}' = \sigma_{P\text{②}}'}$$

A change in the porewater pressure, u produces practically no volume change
→ no settlement

And has no influence on the stress condition for failure.

Eff. st 변화 → Rearrangement of soil particles (부피 변화, 압밀)

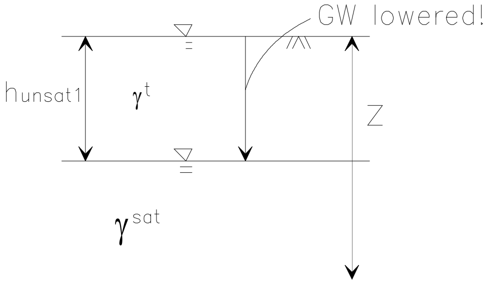
물 빠짐 → 입자가 응력 지탱. 평형조건 변화

However, when the gw table is lowered, the soil is subjected to an increase in effective stress.

This may lead to substantial areal subsidence as is occurring.

For example, in Mexico City and Las Vegas, GW is being pumped for municipal water supply, and the resulting settlements have caused substantial damage to streets, buildings, and underground utilities.

$$\begin{aligned}\sigma'_{initial} &= \gamma_s z - \gamma_w z \\ &= \gamma_{s.ub} z\end{aligned}$$



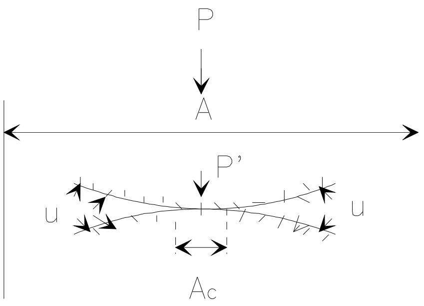
$$\sigma = h_{unsat1}\gamma_t + (z - h_{uns})\gamma_{sat}$$
$$u = (z - h_{uns})\gamma_w$$
$$\sigma' = h_{uns} \cdot \gamma_t + (z - h_{uns})\gamma_{s.ub}$$
$$= \underset{\text{이만큼 증가}}{\underline{h_{uns}(\gamma_t - \gamma_{s.ub})}} + z\gamma_{s.ub}$$

→유효응력 증가

→settlement may occur !

※ $\gamma_t = \frac{G + Se}{1 + e}\gamma_w$,	$\gamma_d = \frac{G}{1 + e}\gamma_w$,	$\gamma_{sat} = \frac{G + e}{1 + e}\gamma_w$,	$\gamma_{s.ub} = \frac{G - 1}{1 + e}\gamma_w$
습윤	건조	포화	수중단위
	(S=0)	(S=1)	

$\gamma_{sat} \geq \gamma_t \geq \gamma_d \geq \gamma_{s.ub}$



For granular particulate materials,

$$\text{Effective stress} = \frac{\Sigma\,\text{contact forces}}{\text{total area}}$$

$$P = P' + (A - A_c)u$$

Dividing by the total area A to obtain stresses,

$$\frac{P}{A} = \frac{P'}{A} + \frac{A - A_c}{A}u \qquad \text{or}$$

$\nearrow$ almost 0

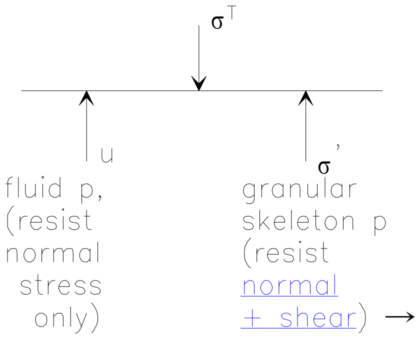
$$\sigma = \sigma' + \left(1 - \frac{A_c}{A}\right) \qquad \rightarrow \qquad \underline{\sigma = \sigma' + u}$$

For fine-grained cohesive soil,

it is doubtful that the mineral crystals are in actual physical contact since they are surrounded by a tightly bound water film

$\psi = 0$ 해석

However, for clays, the principle of effective stress is an excellent approximation to reality, though.



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

→ 강도문제, 부피변화와 직결

$\sigma = \sigma' + u$의 평형 상태

↓ <u>하중 가해짐</u>

간극 수압 소산

과잉 간극 수압 감소

↓

흙 입자가 받는 portion 증가 (σ' 증가)

↓

기존 soil structure matrix의 재배열 초래 (rearrangement)

↓

부피 변화 (압축, 압밀)

↓

stability 문제에 귀결

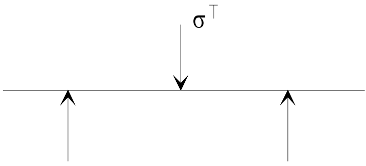
토층이 전체물에 잠긴 경우와 같이

Thus once soil mass is fully saturated with water

A change is the porewater pressure u produces practically no volume change and has practically no influence on the stress conditions for failure.

Effective stress의 변화 → Rearrangement of soil particvle (부피 변화, 압밀)

↓



평형 상태에 따라 stress 가해져서 평형이 깨지면 응력은 먼저 물이 받는다. 그 다음에 물이 빠져나가면 σ는 σ‘이 주로 받게 되어 σ’ 증가

“The response of soil mass to change in applied stresses (compatibility and shearing resistance) depend almost exclusively on the effective stresses in that soil mass.″

평형 상태 $\sigma = \sigma' + u_e$

↓

하중 가해짐

↓

간극 수압 빠져나가기 시작

u_e값 감소

↓

흙 입자가 받는 potion 증가 (σ'증가)

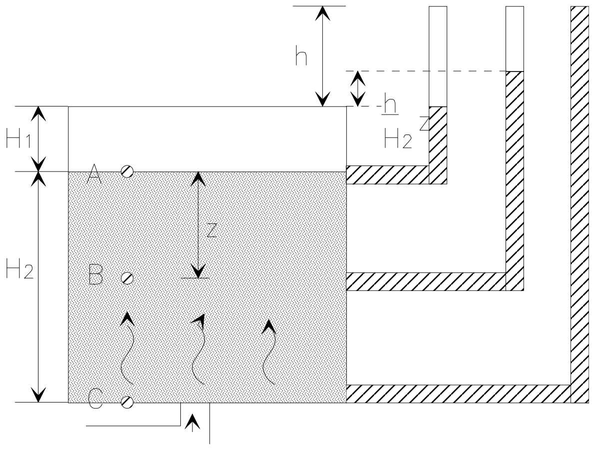
↓

기존 soil structure matrix의 재배열 초래 (Rearrangement)

→부피 변화 (압축, 압밀)

→Stability 문제에 귀결

Upward Seepage

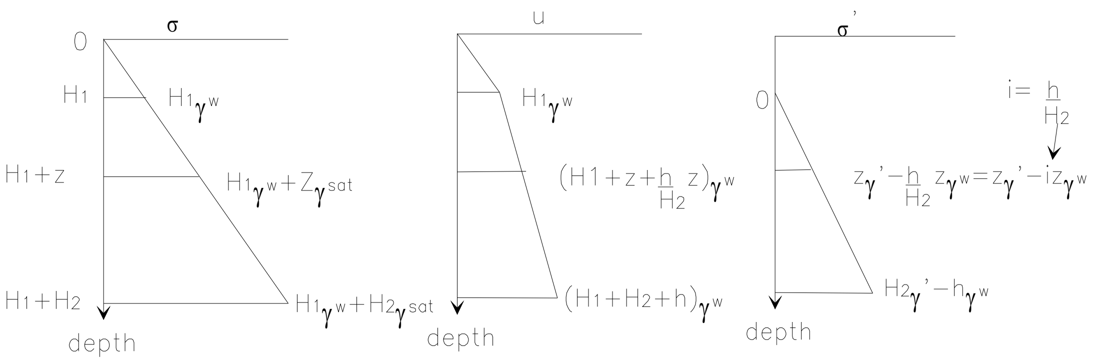


$H_2 : h = z : \square$

H_2가면서 h 상실한다면

z 가면서 얼마 상실!

$$\square = \frac{h}{H_2} z$$



No seepage 때와 비교. z에서 (B)

No : $z\gamma'$

upseep : $z\gamma' - iz\gamma_w$ 감소 $iz\gamma_w$만큼

만약 seepage 양이 많으면 h 증가 → i 증가

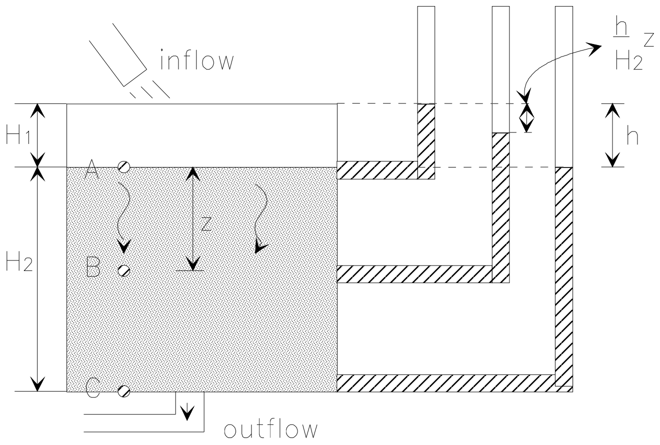
한계 조건에 도달

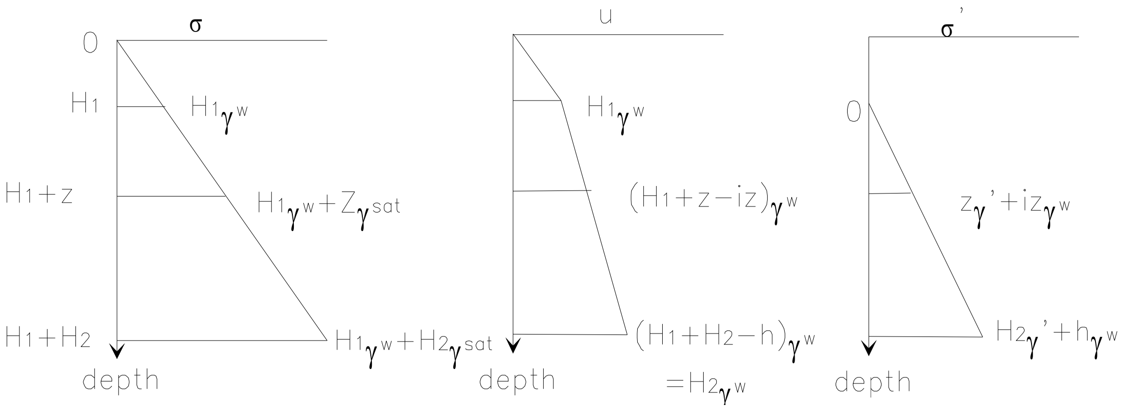
$$\sigma_{Bcrit}' = z\gamma' - iz\gamma_w = 0 \qquad i_{crit} = \frac{\gamma'}{\gamma_w}$$

입자 contact=0

이 상황에서는 유효응력이 0 안정성 잃음 → boiling, quick condition

Downward Seepage





$$\sigma_{Down}' > \sigma_{NC}' > \sigma_{up}'$$

* Seepage Force (침투력) --- $P = \sigma A$ $\sigma = P / A$

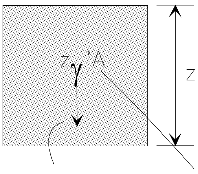
effect of seepage ↑↓ = 유효응력 effective stress의 증가 혹은 감소 초래

$\sigma' = \sigma - u$ ($\leftarrow \sigma = \sigma' + u$) 단위체적당 침투력

앞의 경우의 Pt.B에서의 침투력은

① No seepage의 경우

volume of soil=zA



$\sigma_z' = z\gamma'$ effective stress

여기 작용하는 effective force(흙만에 의한)

$$= \frac{P_0'}{A} = z\gamma'$$

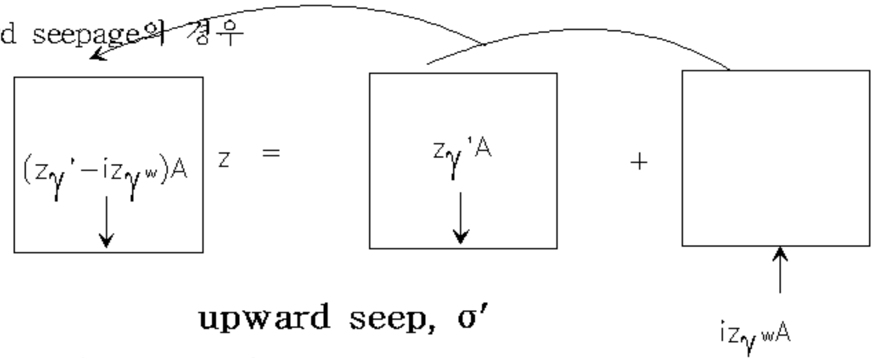
$\therefore$ $P_0' = z\gamma' A$ effective force on area A

$= zA\gamma'$

유효 무게 ; 부피 zA에 의해

② upward seepage의 경우

Vol=zA



upward seep, σ′

$P' = (z\gamma' - iz\gamma_w)A$ effective force on A at depth A

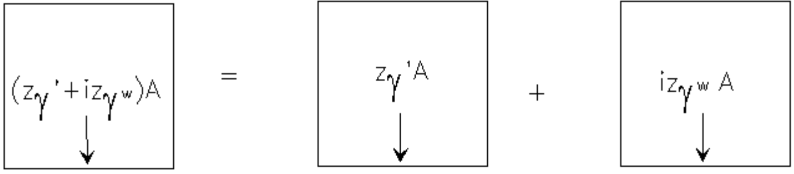
$\therefore$ seepage 에 의한 유효력 (effective force) 감소량

$$P_0' - P_0' = iz\gamma_w A$$

따라서 단위 부피당 침투력은

$$\frac{P_{0'} - P'}{\text{volume of soil}} = \frac{iz\gamma_w A}{zA} = i\gamma_w \uparrow$$

③ downward seepage



$P' = (z\gamma' + iz\gamma_w)A$

$$\frac{P_0' - P'}{\text{Vol}} = \frac{-iz\gamma_w A}{zA} = -i\gamma_w$$ 즉 방향이 ↓ 크기 $i\gamma_w$

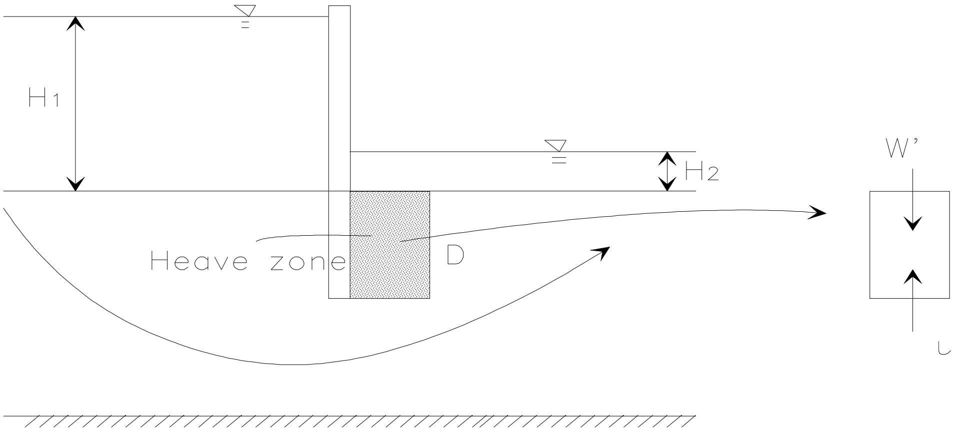
따라서 단위 체적당 침투력은 $i\gamma_w$

이 seepage force 개념은 hydraulic structure(댐, sheet pile)의 하류측(down stream, DS)에서 발생하는 heaving 현상에 대한 안전율 계산에 쓰인다.

Terzaghi는 sheet pile 주변의 heaving 현상 연구

→ D by D/2 zone에서 주로 발생. (실험적으로 발견)

↘가장 취약 지역



Factor of safety against heaving

$$F.S. = \frac{W'}{u}$$

w′ : submerged wt. of soil in the heave zone

u : uplifting force caused by seepage on the same volume of soil

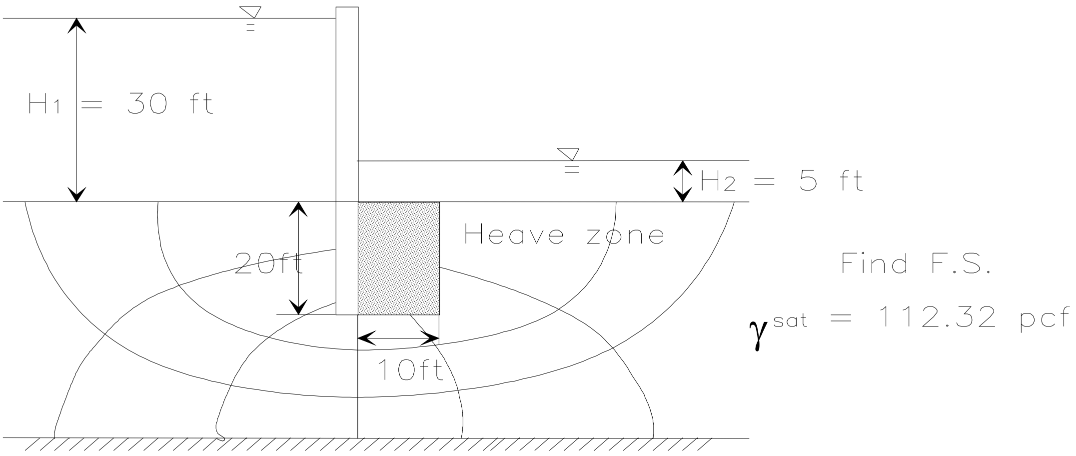
u = soil volume * $\underline{i_{avg} \cdot \gamma_w}$ 단위체적당 침투력

$= \frac{1}{2} D^2 \cdot i_{avg} \cdot \gamma_w$ ↑

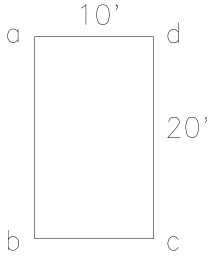
i_{avg} ; avg. hydraulic gradient at the bottom of the block of soil

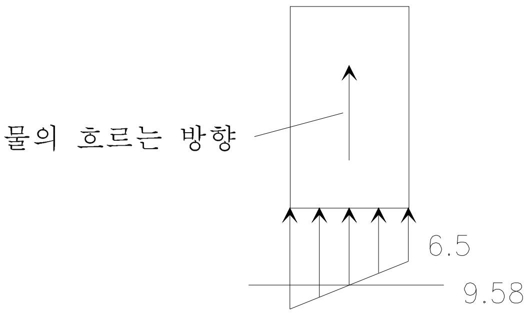
$$F.S = \frac{\frac{1}{2} D^2 \gamma'}{\frac{1}{2} D^2 i_{avg} \gamma_w} = \frac{\gamma'}{i_{avg}\gamma_w}$$

ex)



sol.





at b : $\frac{3}{6}(H_1 - H_2) = \frac{3}{6}\Delta H = 12.5$

avg = 9.58

at c : $\frac{1.6}{6}(H_1 - H_2) = \frac{1.6}{6}\Delta H = 6.7$

$$i_{avg} = \frac{9.58}{D = 20'} = 0.48$$

↘ 평균 수두가 soil column을 결쳐서 있으므로

h_2에 해당

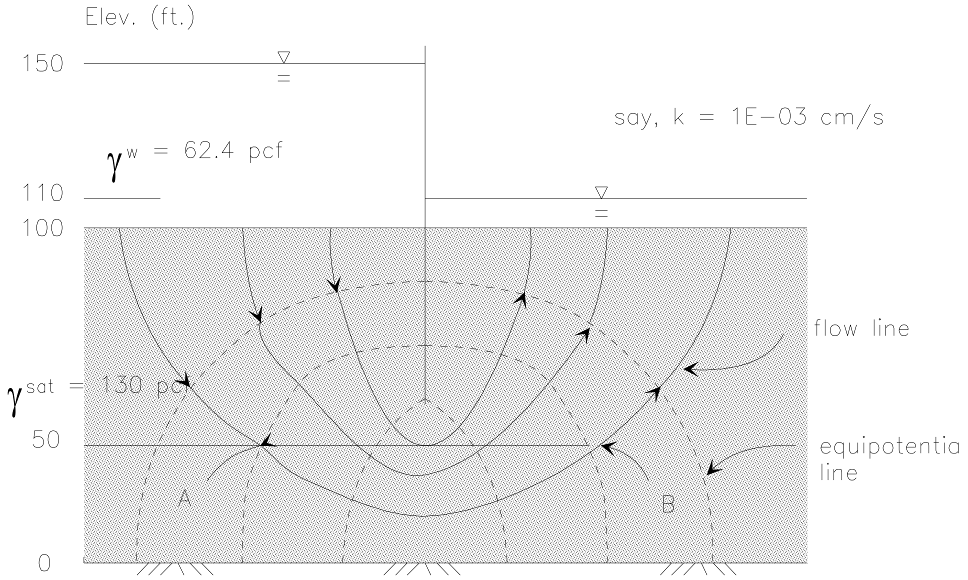
$$\therefore F.S = \frac{\gamma'}{i_{avg}\gamma_w} = \frac{112.32 - 62.4}{0.48 \times 62.4} = \frac{49.9}{29.9} \cong 1.7$$

Heaving 방지책 : ad에 weight을 더하여 W′의 무게를 늘인다.

30.

* Determination of Pressure heads from flow nets

$$h_{tx} = h_{initial} - n_{ex} \cdot \Delta h$$



$N_f = 4$
$N_d = 8$
$H = 150 - 110 = 40$
$q = k H \dfrac{N_f}{N_d}$

A점에서의 h_p구하기

$$\Delta h = \frac{\Delta H}{n_e} = \frac{150 - 110}{8} = 5'$$

$$h_{tA} = 150 - 2 \times 5 = 140'$$

$$\therefore h_{pA} = 140 - h_e = 140 - 30 = 110'$$

Required – Calculate effective stresses ar points A and B shown above.

Solution –

Point A에서 u, σ′,σ 구하기

$\sigma_T = (50')(62.4\text{pcf}) + (70')(130\text{ pcf}) = 12,220\text{ psf}$
$u \;= h_p \cdot \gamma_w = (h_t - h_e)\gamma_w = ((150' - (2)(5')) - 30')(62.4\text{pcf}) = 6,850\text{ psf}$
$\;\;= (140 - 30) \times 62.4$
$\sigma_E = 12,220\text{ psf} - 6,850\text{ psf} = 5,370\text{ psf}$

Point B에서 u, σ′,σ 구하기

$\sigma_T = (10')(62.4\text{pcf}) + (70')(130\text{ pcf}) = 9,720\text{ psf}$
$u \;= ((150' - (6)(5')) - 30')(62.4\text{pcf}) = 5,620\text{ psf}$
$\sigma_E = 9,720\text{ psf} - 5,620\text{ psf} = 4,100\text{ psf}$

※Dupuit's Assumptions

①Hydraulic gradient is equal to the slope of the water table,

②The flow lines are horizontal and the equipotential line is vertical.

22. Direction / Gradient of GW Flow

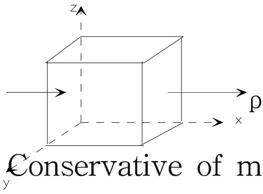
▶ OHPs are shown here

▶ Examples

23. Steady-State Saturated Flow

steady-state occurs when at any point in a flow field the mag. and direction of the flow velocity are constant w/ time. $\frac{\partial V}{\partial t} = 0$ Does this condition always satisfy $\frac{\partial V}{\partial s} = 0$? uniform flow? No not always. In S-S condition, velocity may vary from pt. to pt., but it won't vary w/ time at any given pt.

$$\searrow \frac{\partial V}{\partial t} = 0$$



$$\rho v_x \qquad \rho v_x - \frac{\partial}{\partial x}(\rho v_x) \qquad \rho V = \frac{\text{mass / area}}{\text{time}}$$

Conservative of mass ; $M_{i.n} - M_{out}$=change in storage/time

= 0 for S-S condition

Darcy's law valid , $k_x = k_y = k_z$

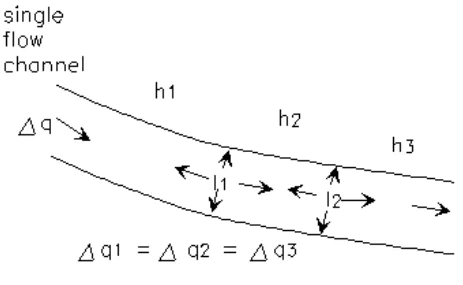
$$-\frac{\partial}{\partial x}(\rho v_x) - \frac{\partial}{\partial y}(\rho v_y) - \frac{\partial}{\partial v_z}(\rho v_z) = 0 \quad \Rightarrow \quad \frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial y^2} + \frac{\partial^2 h}{\partial z^2} = 0$$

Governing eq. for ss sat. flow for cnf/uncnf cond ↙

The solution os h=h(x,y,z) , the hydraulic head at any point in the flow domain. In two D, the solution is equivalent to the graphical Flow Net.

(FL⊥EL, square,Boundary FL,EL)

· Seepage Calculation from Flow Net



$$\Delta q = k\left(\frac{h_1 - h_2}{l_1}\right)l_1 \times 1 = k\left(\frac{h_2 - h_3}{l_2}\right)l_2 \times 1$$

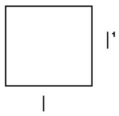
$$h_1 - h_2 = h_2 - h_3 = \cdots = \frac{H}{N_d}$$

$$\therefore \Delta q = k\frac{H}{N_d} \quad \leftarrow \text{diff. of head b/w u.s \& D.S}$$

↖No. of potential drops

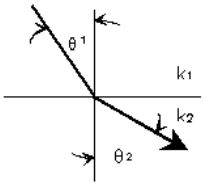
$$\therefore q = \Delta q \cdot N_f \leftarrow \text{No. of flow channels} \quad \therefore q = kH\frac{N_f}{N_d} \text{ (flow rate per unit width)}$$

$$\therefore Q = q \cdot w \text{ (w: width of the flow zone)}$$

if  $n=l'/l$, $q = kH(\frac{N_f}{N_d})\cdot n$

if $k_x \neq k_z$, $q = \sqrt{k_x k_z}\,H(\frac{N_f}{N_d})$

※Refraction of Flow Line



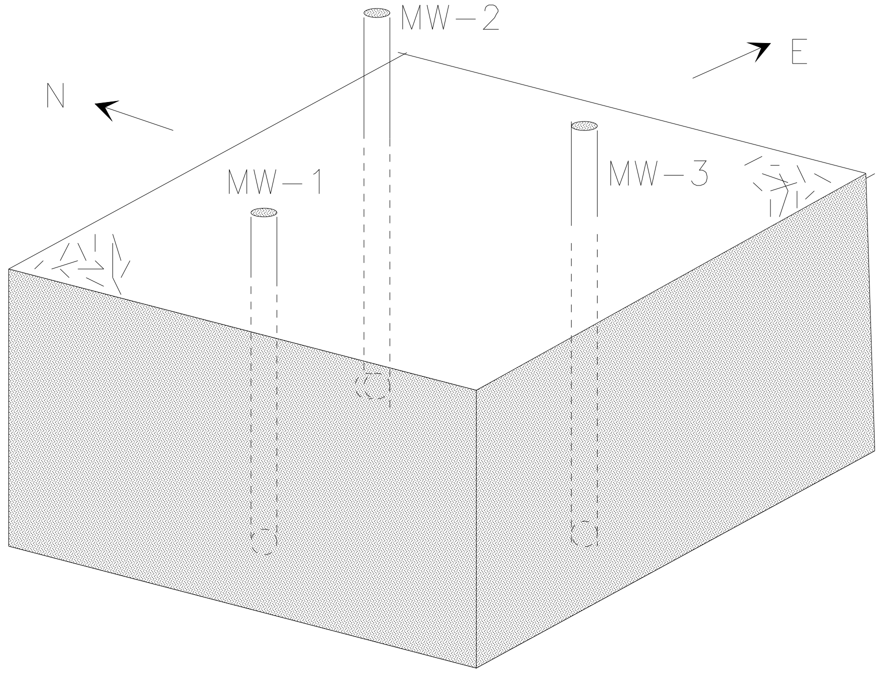
$$\frac{k_1}{k_2} = \frac{\tan\Theta_1}{\tan\Theta_2}$$

▶ OHPs are shown here

for Determination of pressure head / effective stress / flow rate unit width

-DETERMINING HYDRAULIC GRADIENT BY TRIANGULATION

CONCEPTUAL MODEL

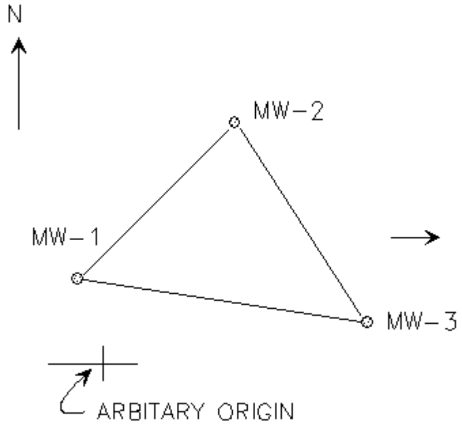


ASSUMPTIONS

- CONFINED OR UNCONFINED AQUIFER UNIT, DUPUIT CONDITIONS VALID.
- ALL WELLS SAMPLE SAME AQUIFER UNIT AND ARE SCREENED SO THAT VERTICALLY AVERAGED HEAD IS MEASURED
- PIEZOMETRIC SURFACE IN REGION OF WELLS CAN BE APPROXIMATED BY A PLANE

OBJECTIVE

DETERMINE DIRECTION & MAGNITUDE OF HYDRAULIC GRADIENT (OF -GRAD/H)

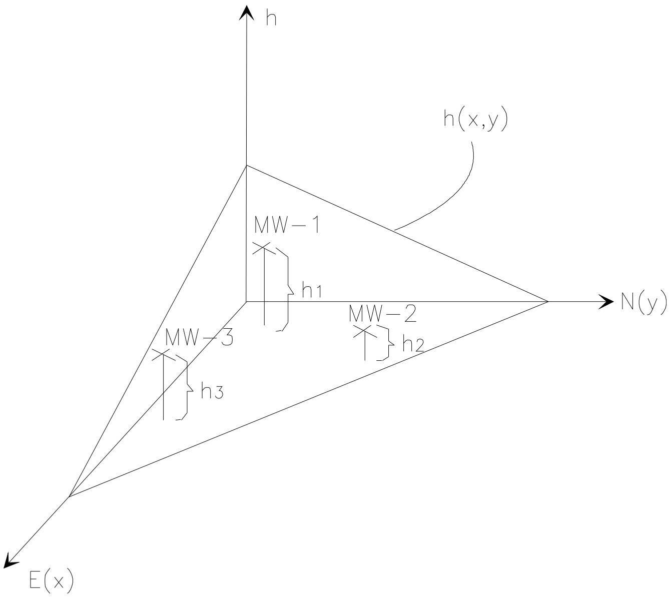


THEORY

--LET HEAD IN EACH WELL BE h_1, h_2 AND h_3, RESPECTIVELY

--ASSUME THAT PIEZOMETRIC SURFACE IS A PLANE

--IF WE KNOW THE EQUATION OF THE PLANE $h(x,y) = f(x,y)$, THEN TAKE GRADIENT TO FIND DIRECTION AND MAGNITUDE OF HYDRAULIC GRADIENT (-GRAD(h))



h(x,y) = ax + by + c (PARAMETER EQUATION OF PLANE IN 3-D)

DATA FROM WELLS

$h_1 = a x_1 + b y_1 + c$ →3 EQUATION,3 UNKNOWNS

$h_2 = a x_2 + b y_2 + c$ FULLY DETERMINED SYSTEM

$h_3 = a x_3 + b y_3 + c$ WANT TO SOLVE FOR a, b, c

IN VECTOR MATRIX FORM

$$\underbrace{\begin{pmatrix} x_1 & y_1 & 1 \\ x_2 & y_2 & 1 \\ x_3 & y_3 & 1 \end{pmatrix}}_{A} \underbrace{\begin{pmatrix} a \\ b \\ c \end{pmatrix}}_{x} = \underbrace{\begin{pmatrix} h_1 \\ h_2 \\ h_3 \end{pmatrix}}_{b} \qquad Ax=b$$

SOLUTION

$x = A^{-1}b$

NOW ONCE x IS DETERMINED, CAN FIND GRADIENT

$$h(x,y)=ax + by + c$$

$$g.rad(h) = \frac{d}{dx}(ax + by + c)i^{\wedge} + \frac{d}{dy}(ax + by + c)j^{\wedge}$$
$$= a\,i^{\wedge} + b\,j^{\wedge}$$

HUDRAULIC GRADIENT = - grad(h)

$$-g.rad(h) = -a\,i^{\wedge} - b\,j^{\wedge}$$

SUMMARY

--DATA FROM THREE WELLS IN TERMS OF HEAD FROM A KNOWN VERTICAL DATUM, AND CARTESIAN LOCATION IN THE HORIZONTAL FROM AN ARBRITARY ORIGIN

--ASSUMING PIEZOMETRIC SURFACE IS REPRESENTED BY A PLANE FIND PARTICULAR EQUATION OF THE PLANE :

$$h(x,y) = ax + by + c$$

BY SOLVING

$$\begin{pmatrix} x_1 & y_1 & 1 \\ x_2 & y_2 & 1 \\ x_3 & y_3 & 1 \end{pmatrix} \begin{pmatrix} a \\ b \\ c \end{pmatrix} = \begin{pmatrix} h_1 \\ h_2 \\ h_3 \end{pmatrix}$$

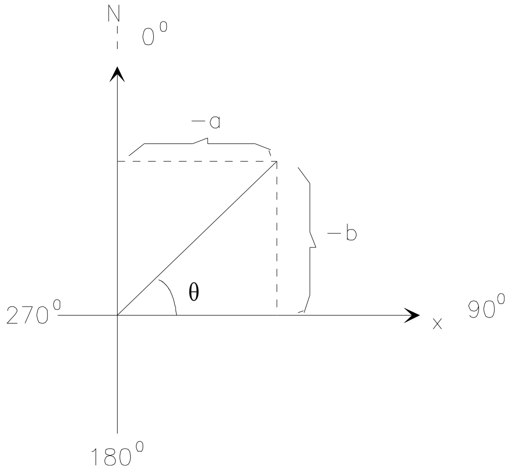
--HYDRAULIC GRADIENT IS THEN :

$$-g.rad(h) = -a\,i^{\wedge} - b\,j^{\wedge}$$

TO FIND MAGNITUDE JUST FIND THE l_2-NORM OF THE VECTOR

$|-\text{g.rad}(h)| = (a^2 + b^2)^{1/2}$

TO FIND DIRECTION, CONVERT x-y COORDINATES INTO POLAR COORDINATES, WITH 0° = NORTH



$\Theta = \arctan\left(\frac{b}{a}\right)$

FIND Θ, CONVERT INTO AZMIUTH-REMEMBER TO FIND CORRECT QUADRAUT.

SEE READINGS FOR A GRAPHICAL APPROACH-OBSERVE GRAPHICAL APPROACH USES IDENTICAL THEORY ∴ HAS SAME ASSUMPTIONS & LIMITATIONS.

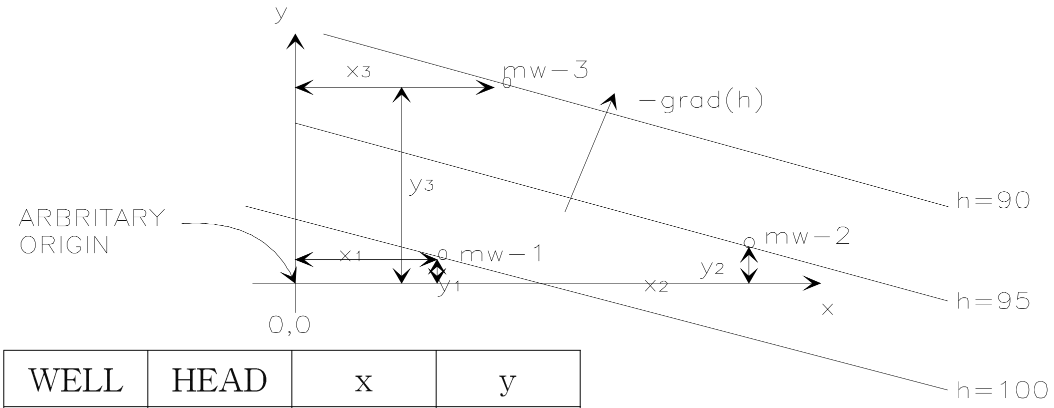
ALSO, HEATH pg 11 - VALID FOR HOMOGENEOUS-ISOTROPIC SYSTEM (GRADIENT PROCEDURE IS CORRECT REGARDLESS)

--FOR CONVIENCE A COMPUTER PROGRAM NAMED "GRADIENT"IS AVAILABLE ON THE FILE SERVER.

--THE PROGRAM PROMPTS THE USER FOR x,y COORDINATES AND HEADS FOR THREE WELLS.

--THE PROGRAM SOLVES Ax=b AND COMPUTES THE HYDRAULIC GRADIENT

EXAMPLE



WELL	HEAD	x	y
mw-1	100	100m	20m
mw-2	95	280m	20m
mw-3	90	100m	180m

→TYPICAL DATA SET

--COMPUTER PROGRAM OUTPUT SHOWS

-grad(h) = 0.027i^ + 0.0625 j^

COMPUTER OUTPUT

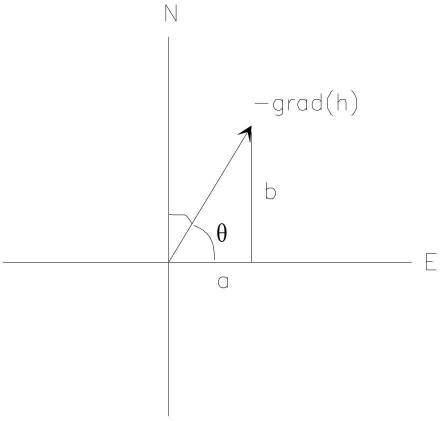
```
GWGRAD estimates regional gradient from heads in 3 wells using planar
distribution
        head(x,y) = ax + by + c
        grad(head) = ai + bj
GWGRAD procedure -- Data for three wells
        X position for data point #        1        ?
        → 100
        Y position for data point #        1        ?        →mw-1
        → 20
        Head value for data point #        1        ?
        → 100
        X position for data point #        2        ?
        → 280
        Y position for data point #        2        ?        →mw-2
        → 20
        Head value for data point #        2        ?
```

```
→ 95
X position for data point #        3       ?
→ 100
Y position for data point #        3       ?        →mw-3
→ 180
Head value for data point #        3       ?
→ 90
Approximate regional head distribution
H(X,Y)= -0.278E-01 x + -0.625E-01 y + 104.
              a                b          c
Hydraulic Gradient = 0.277778E-01 i + 0.625000E-01 j
                           a              b

STOP
```

ENTER THESE DATA, PROGRAM COMPUTES GRADIENT.
SOURCE CODE, APPLICATION, AND MAKEFILE ON SEVER

DIRECTION & MAGNITUDE CALCULATIONS.



$$\tan \Theta = \frac{0.0625^{b}}{0.027\,a} = 3.3148$$

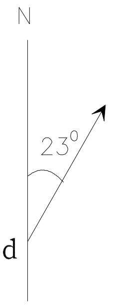
$$\Theta = \tan^{-1}(2.3148) = 66.6°$$

∴ DIRECTION = 90°-66.6°= 23.36°(NORTH=0°)

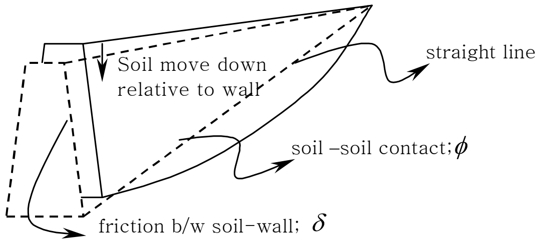
$$\text{MAGNITUDE} = \sqrt{(0.0625)^2 + (0.027)^2} = 0.068$$

ASSUMING THE AQUIFER IS HOMOGENEOUS, AND ISOTROPIC ONE CAN ESTIMATE SPECIFIC DISCHARGE GIVEN K.
I.E SUPPOSE K = 10 m/d

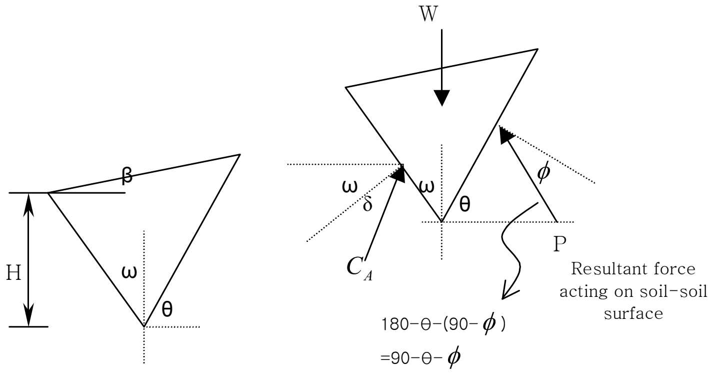
$\vec{q} = -K\nabla(h) = (10m/d)(0.068) \equiv = 0.68m/d$



Coulomb's Earth Pressure Theory
(벽마찰력을 고려 Rankine보다 더 경제적인 설계가 가능해짐)
A.Active case
All soil resistance is mobilized along soil-soil and soil-wall contacts
Plane strain condition assumed.



(i), **c=0, dry.**



*Coulomb's active earth pressure

$C_A = fn$(geometry of H, ω, Θ, β)
(strength paraϕ)
(interface wall friction δ)

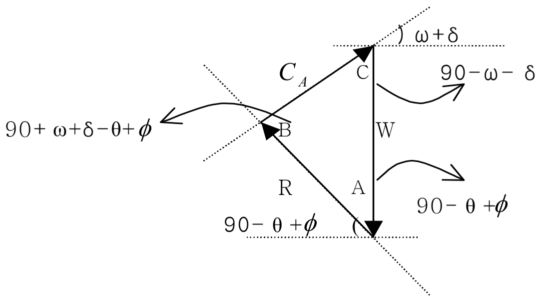
*To find C_A

1.assume Θ, and calculate C_A from vector addition

knowns W: magnitude +orientation

R: orientation

C_A: orientation



sin 법칙으로(closed form 해법)

또는 $\frac{C_A}{\sin A} = \frac{W}{\sin B} = \frac{R}{\sin C}$

$\sum F_x = 0, \sum F_y = 0$

- C_A가 구해진다.

2.Repeat 1. for various Θ (Θ 를 모르므로 다양하게 가정한다.)

3.Select $C_{A,\max}$

*Closed form solution(no Θ assumption)

$$\frac{C_A}{\sin(\theta-\phi)} = \frac{W}{\sin(90+\omega+\delta-\theta+\phi)}$$

$$C_A = \frac{\sin(\theta-\phi)}{\sin(90+\omega+\delta-\theta+\phi)}\cdot W$$

$$= \frac{1}{2}\gamma\cdot H^2\left[\frac{\cos(\omega-\theta)\cos(\omega-\beta)\sin(\theta-\phi)}{\cos^2\omega\cdot in(\theta-\beta)\sin(90+\omega+\delta-\theta+\phi)}\right]$$

*To determine the critical value of Θ for maximum

$$\frac{dC_A}{d\theta} = 0$$

$$C_A = \frac{1}{2}\gamma\cdot H^2 K_A$$

$$K_A = \frac{\cos^2(\phi-\omega)}{\cos^2\omega\cdot\cos(\delta+\omega)\left[1+\sqrt{\frac{\sin(\delta+\phi)\sin(\phi-\beta)}{\cos(\delta+\omega)\cos(\omega-\beta)}}\right]^2}$$

if $\omega = 0, \delta = 0, \beta = 0$ (like Rankine case)

R $K_A = \frac{\cos^2\phi}{\left(1+\sqrt{\sin^2\phi}\right)^2} = \frac{1-\sin^2\phi}{(1+\sin\phi)^2} = \frac{1-\sin\phi}{1+\sin\phi}$ - Rankine Active state coeff.와 동일

if $\omega \neq 0, \delta \neq 0, \beta \neq 0$

$\sigma_A = K_A \gamma \cdot H$

$C_A = \frac{1}{2}\gamma \cdot H^2 K_A$ K_A는 표5.3 이용 (Das F.E.4th ed. P354)

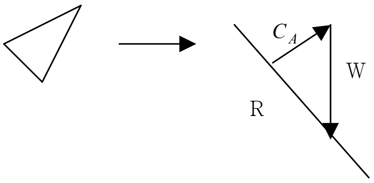
Coulomb's Active Case by Trial Wedge Method.

1) ϕ , c = 0

1) $C_A = \frac{1}{2}\gamma H^2 K_A$

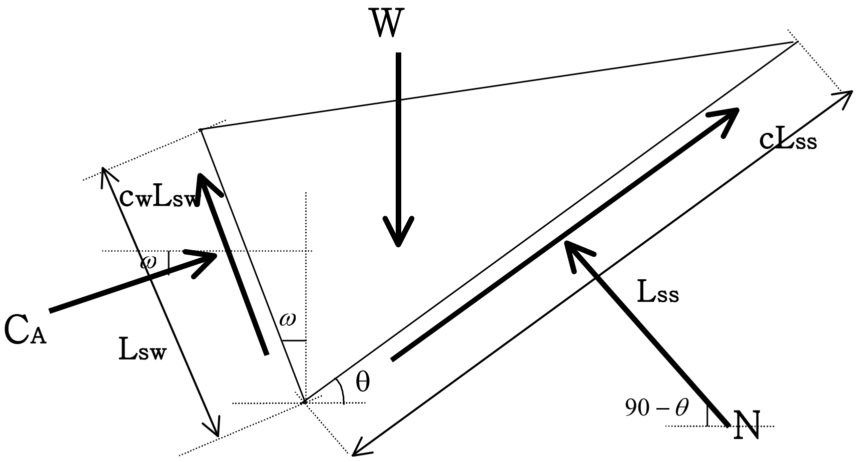
- K_A by chart (closed solution)

2)



W/ Θ assumption Max C_A

2) ϕ = 0, c ($\delta = \frac{2}{3}\Phi = 0$)

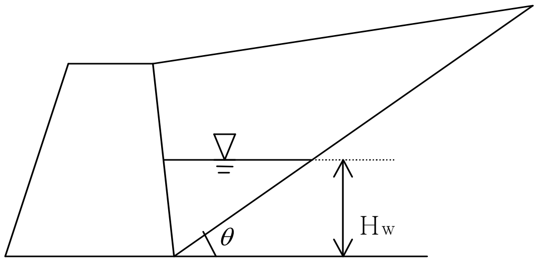


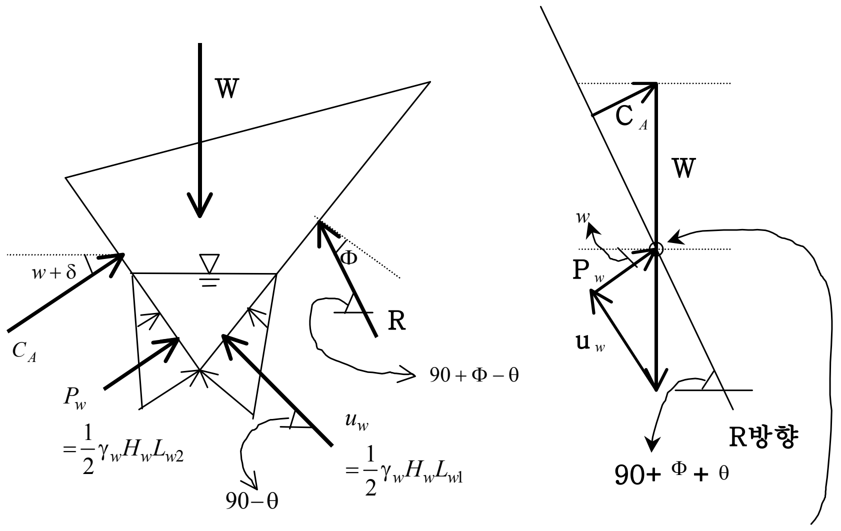
C_A : adhesive b/w soil and wall
W : magnitude + orientation
CwLsw : magnitude + orientation
CLss : magnitude + orientation
N : orientation
C_A : orientation

3) 만약 φ , c, dry 이면

N → R의 각도 : $90+\Phi-\theta$
CA → CA의 각도 : $90+\Phi-\theta$
CA를 구할 수 있다.

4) φ , c = 0, water (no flow)

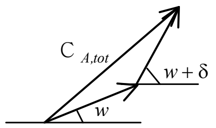




Flow가 생기면 힘의 평형이 이루어지지 않으므로 이점에서 만나지 못함

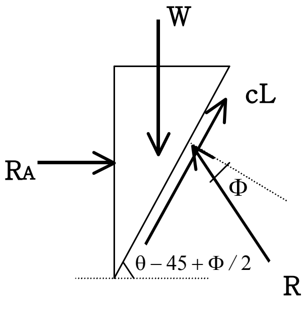
Therefore, total pressure on wall $C_{A,tot}$

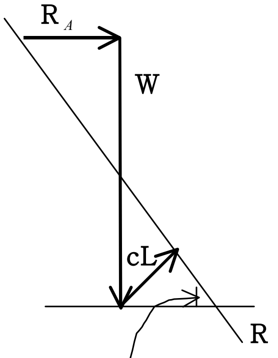
$$C_{A,tot} = C_A + p_w$$



Trial wedge for Rankine case

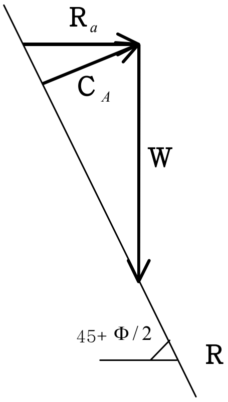
$w=0,\ \delta=0,\quad \beta=0$ (Φ, c clay)





$$180-\theta-(90-\Phi)$$
$$=90-\theta+\Phi=90+\theta-\Phi$$
$$=90+\Phi-45-\Phi/2=45+\Phi/2$$

Comparison b/w R and C for , c=0 dry condition



R*a*>*a* C

: The effect of wall friction is to reduce total force against wall. Therefore, Rankine gives conservative design forces relative to Coulomb.

$C_A = \frac{1}{2}\gamma H^2 K_a$ $\sigma_A = K_a \gamma H$

-**Coulomb's active earth pressure**

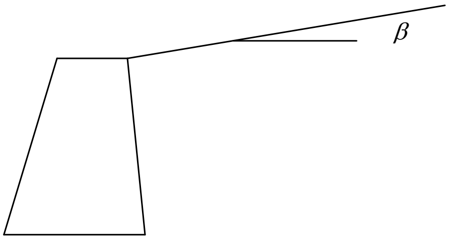
Coulomb's Pctive Earth Pressure

$P_p = \frac{1}{2} K_p \gamma H^2$, $\sigma_p = K_p \gamma H$

-use minimum of C_p for θ when using trial wedge method. K_p use Table 5.5 / Table 5.7 / Figure 5.15

*** Special Case**

-Rankine Active and Passive Earth Pressure for inclined Granular Backfill



$$K_a = \cos\beta \frac{\cos\beta - \sqrt{\cos^2\beta - \cos^2\Phi}}{\cos\beta + \sqrt{\cos^2\beta - \cos^2\Phi}}$$ -----table 5.8

$$K_p = \cos\beta \frac{\cos\beta + \sqrt{\cos^2\beta - \cos^2\Phi}}{\cos\beta - \sqrt{\cos^2\beta - \cos^2\Phi}}$$ ------table 5.9

$P_a = \frac{1}{2}K_a\gamma H^2$, $\sigma_a = K_a\gamma H$ $P_p = \frac{1}{2}K_p\gamma H^2$, $\sigma_p = K_p\gamma H$

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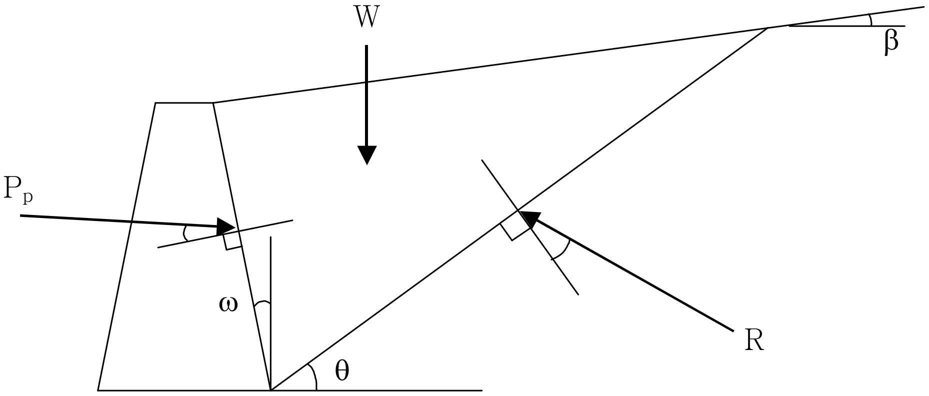

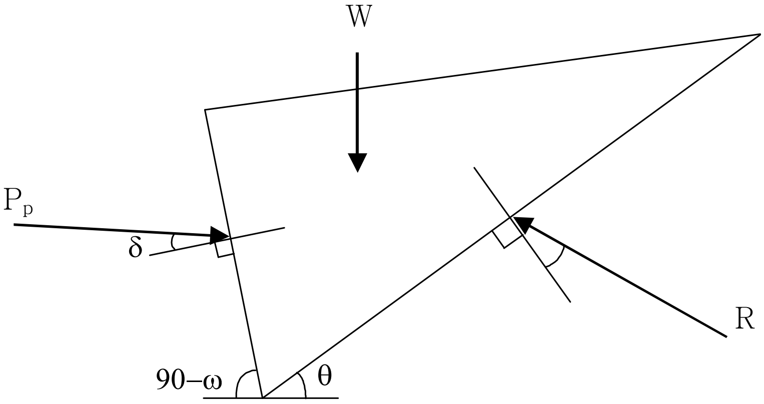

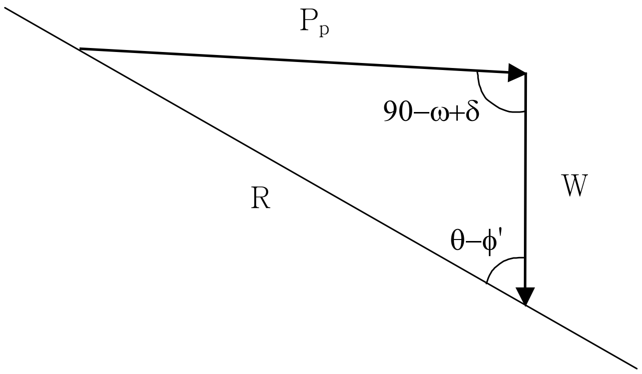

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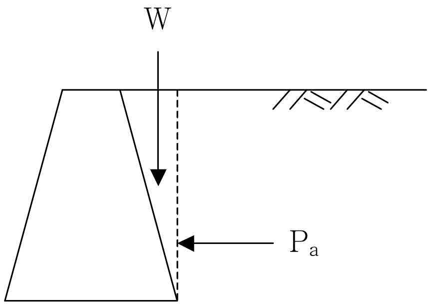
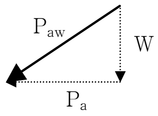
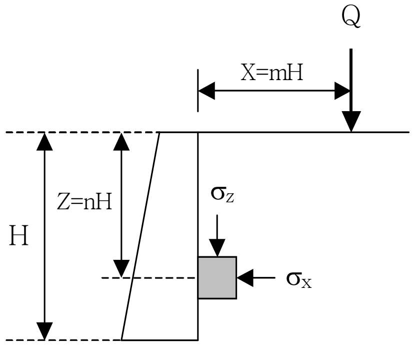
Rankine with W (inclined wall)



Weight of soil wedge, W is vectorally added to Pa
따라서 벽체는 기울어진 토압을 받는다.



*Surcharge Load (Based on the theory of elasticity)
1)Point load surcharge



σz : 수직방향 토압 증가량 방법이용

σx m>0.4

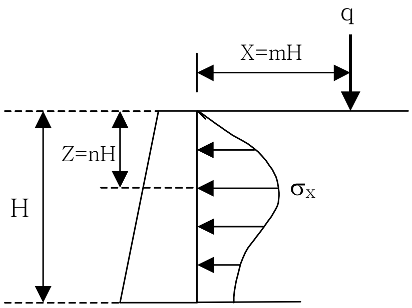
$$\sigma_x = \frac{1.77Q}{H^2}\frac{m^2n^2}{(m^2+n^2)^3}$$

m≤0.4

$$\sigma_x = \frac{0.28Q}{H^2}\frac{n^2}{(0.16+n^2)^3}$$

$$P = \int_0^H \sigma_x dz$$

2)Line load surcharge



m>0.4

$$\sigma_x = \frac{4q}{\pi H}\frac{m^2n}{(m^2+n^2)}$$

m≤0.4

$$\sigma_x = \frac{0.203q}{H}\frac{n}{(0.16+n^2)^2}$$

$$P = \int_0^H \sigma_x dz$$

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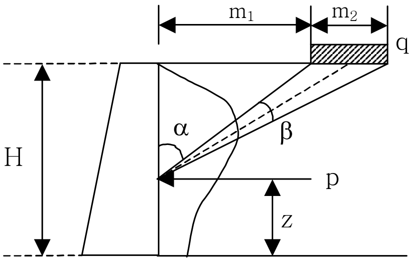
3)Strip load surcharge



$$\sigma_x = \frac{2q}{H}(\beta - \sin\beta\cos 2\alpha)$$

$$p = \frac{q}{90}(H(\theta_2 - \theta_1))$$

$$\theta_1(\text{deg}) = \tan^{-1}(\frac{m_1}{H})$$

$$\theta_2(\text{deg}) = \tan^{-1}(\frac{m_1 + m_2}{H})$$

Ex) q=1000 psf, m1=10ft, m2=5ft, H=10ft

Sol) $P = \int_0^H \sigma_x dz = \frac{q}{90}(H(\theta_2 - \theta_1)) = \frac{1000}{90}(10 \times (\theta_2 - \theta_1))$

$$\theta_2 = \tan^{-1}(\frac{10+5}{10}) = 56.31(\text{deg})$$

$$\theta_1 = \tan^{-1}(\frac{10}{10}) = 45(\text{deg})$$

$p = \frac{1000}{90}(10(56.31 - 45)) = 1256$ (lb/ft)

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Table 5.5 Values of Kp for ω=0 and β=0

Φ (deg)	δ(deg)				
	0	5	10	15	20
15	1.698	1.900	2.130	2.405	2.735
20	2.040	2.313	2.636	3.030	3.525
25	2.464	2.830	3.286	3.855	4.597
30	3.000	3.506	4.143	4.977	6.105
35	3.690	4.390	5.310	6.854	8.324
40	4.600	5.590	6.946	8.870	11.772

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Table 5.6 General Range of Wall Friction Angles for masonry or Mass Concrete Walls

Backfill Material	Range of (deg)
Gravel	27~30
Ciarse Sand	20~28
Fine Sand	15~25
Stiff Clay	15~20
Silty Clay	12~16

Table 5.7 Values of Kp for a Vertical Wall with Horizontal Cohesionless Soil as Backfill

φ	δ					
	20	25	30	35	40	45
0	2.04	2.46	3.00	3.69	4.60	5.83
5	2.27	2.78	3.44	4.31	5.46	4.09
10	2.47	3.08	3.68	4.92	6.36	8.43
15	2.64	3.34	4.28	5.53	7.30	9.89
20	2.87	3.61	4.58	6.17	8.30	11.49
25		4.00	5.12	6.85	9.39	13.20
30			5.81	7.61	10.60	15.31
35				8.85	12.00	17.65
40					14.40	20.48
45						25.47

Figure 5.15 Variation of Kp obtained from Terzaghi and Peck' s methid [vertical backface of wall(?=0) and horizontal granular backfill(?=0)

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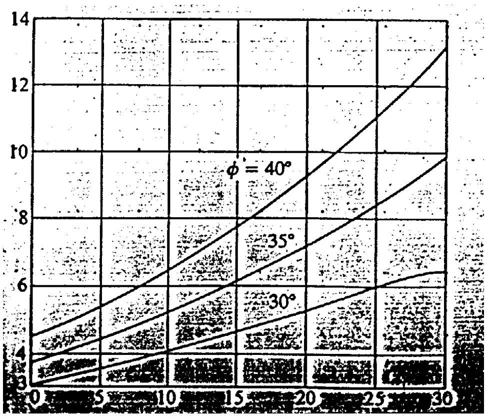
Table 5.8 Active Earth Pressure Coefficient, Ka

β(Deg)	φ(deg)						
	28	30	32	34	36	38	40
0	0.361	0.333	0.307	0.283	0.260	0.238	0.217
5	0.366	0.337	0.311	0.286	0.262	0.240	0.219
10	0.380	0.350	0.321	0.294	0.270	0.246	0.225
15	0.409	0.373	0.341	0.311	0.283	0.258	0.235
20	0.461	0.414	0.374	0.338	0.306	0.277	0.250
25	0.573	0.494	0.434	0.385	0.343	0.307	0.275

β(Deg)	φ(deg)						
	28	30	32	34	36	38	40
0	0.361	0.333	0.307	0.283	0.260	0.238	0.217
5	0.366	0.337	0.311	0.286	0.262	0.240	0.219
10	0.380	0.350	0.321	0.294	0.270	0.246	0.225

Table 5.9 Passive Earth Pressure Coefficient, Ka

β(Deg)	φ(deg)						
	28	30	32	34	36	38	40
0	2.770	3.000	3.255	3.537	3.852	4.204	4.599
5	2.715	2.943	3.196	3.476	3.788	4.136	4.527
10	2.551	2.775	3.022	3.295	3.598	3.937	4.316
15	2.284	2.502	2.740	3.003	3.293	3.615	3.977
20	1.918	2.132	2.362	2.612	2.886	3.189	3.526
25	1.434	1.664	1.894	2.135	2.394	2.676	2.987

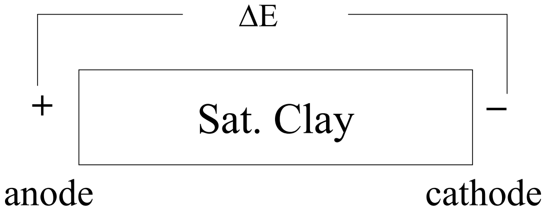


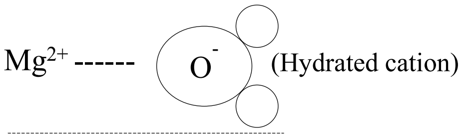
* Coupling b/w electrical and hydraulic flows

4 electrokinetic phenomena in *fine-grained soils*

① Electro-Osmosis

There are more cations than anions in soils

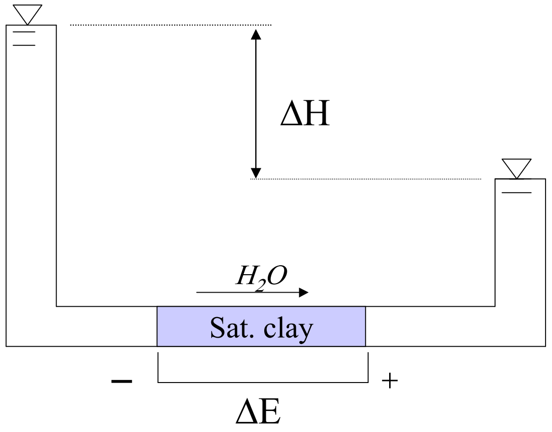




attracted to cathode(-)

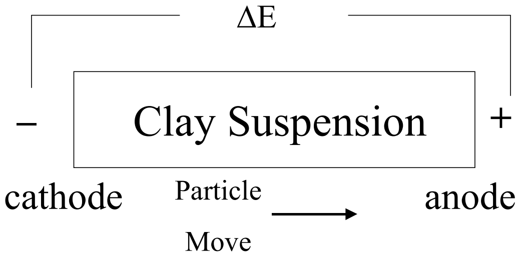
∴ There is a net water flow induced toward (-) : *electro-osmosis*
→ E-O can be applicable for the acceleration of drainage
(Electro-Osmotic Consolidation)

② Streaming Potential



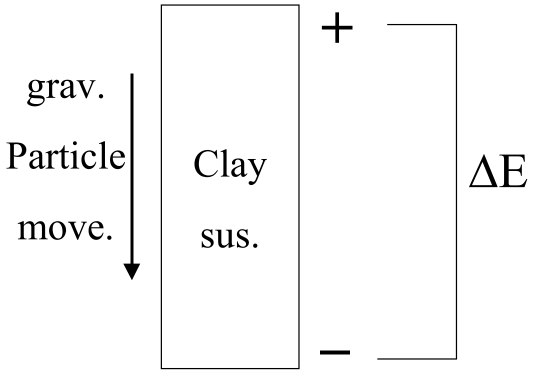
H_2O movement induced by △H cations hydrated.
The result is an electrical potential difference proportional to the hydraulic flow rate, called 'streaming potential b/w the opposite ends of the soil mass. s.p. of several tens of mVs have been measured.

③ Electro phoresis



Negatively charged clay particles move toward the anode. Discrete particle transport through water. ⇒ E. gradient induces particle movement. (지반개량)

④ Migration or Sedimentation Potential



The movement of charged clay particles during gravitational settling generates an E. potential difference. Negatively charged clay particles settle down at the bottom

→ More negatively charged at the bottom

※ Theories for Electro-osmosis

k_e : coefficient of electro-osmosis hydraulic conductivity.

Hydraulic flow velocity under a unit electrical gradient. $1\times10^{-9}\sim1\times10^{-8}$ ㎡/s/volt

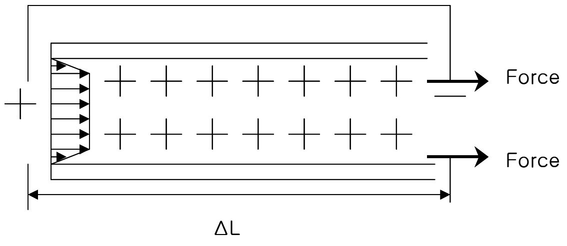
No.	Material	Water Contents(%)^a	k_e in 10^{-5} (cm^2/sec-V)^a	Approximate k_h (cm/sec)
1	London clay	52.3	5.8	10^{-8}
2	Boston blue clay	50.8	5.1	10^{-8}
3	Kaolin	67.7	5.7	10^{-7}
4	Clayey silt	31.7	5.0	10^{-6}
5	Rock flour	27.2	4.5	10^{-7}
6	Na-Montmorillonite	170	2.0	10^{-9}
7	Na-Montmorillonite	2000	12.0	10^{-8}
8	Mica powder	49.7	6.9	10^{-5}
9	Fine sand	26.0	4.1	10^{-4}
10	Quartz powder	23.5	4.3	10^{-4}
11	Ås quick clay	31.0	20.0-2.5	2.0×10^{-8}
12	Bootlegger Cove clay	30.0	2.4-5.0	2.0×10^{-8}
13	Silty clay, West Branch Dam	32.0	3.0-6.0	1.2×10^{-8}-6.5×10^{-8}
14	Clayey silt, Little Pic River, Ontario	26.0	1.5	2×10^{-5}

k_e, same order of magnitude for most soil types.

Measurement is made by determination of the flow rate of water through a soil sample of known length and cross-section under a known electrical gradient.

Electro-osmosis theories providing a basis for quantitative prediction of flow rate.

Helmholtz and Smouchowski Theory



v : flow velocity

δ : distance from the wall between the wall and the center of the plane of mobile charge

Model : *A liquid-filled capillary is treated as an electrical condenser with charges of one sign on and countercharges concentrated in a layer in the liquid a small distance from the wall.*
The mobile cations are assumed to drag water through the capillary by flow.

The rate of water flow is controlled by the balance between the electrical force causing water movement and friction between the liquid and the wall.

At equilibrium,

$\mu \frac{v}{\delta} = \sigma \frac{\Delta E}{\Delta L}$ ------- ①

μ : viscosity

σ : surface charge density

or

$\sigma\delta = \mu v \frac{\Delta L}{\Delta E}$ ------- ②

From electrostatics, the potential across a condenser ζ is

$\zeta = \frac{\sigma\ \delta}{D}$ ------- ③ ⇒ zeta potential

(D : dielectric constant of pore fluid)

$\sigma\ \delta = \zeta\ D$ substitute to ②

$\zeta\ D = \mu v \frac{\Delta L}{\Delta E}$ → $v = (\frac{\zeta D}{\mu}) \frac{\Delta L}{\Delta E}$ ------ ④

For single capillary of area a, the flow rate is

$q_a = va = \zeta D \mu \frac{\Delta L}{\Delta E} a$ ------- ⑤

and for a bundle of N capillaries within total cross section area A normal to the flow direction

$q_A = Nq_a = \frac{\zeta D}{\mu} \frac{\Delta L}{\Delta E} Na$ ------- ⑥

If the porosity is n , then the cross-sectional area of voids is nA, which must equal Na, thus,

$q_A = \frac{\zeta D}{\mu} \frac{\Delta L}{\Delta E} nA$ ------- ⑦

By analogy with Darcy's law,

$q_A = k_e i_e A$

i_e : *electrical potential grad* $\frac{\Delta L}{\Delta E}$

k_e : *coefficient of electro-osmotic hydraulic conductivity*

$k_e = \frac{\zeta D}{\mu} n$ ------- ⑧

⇒ independent of pore size. this explain Table 12. 7

This is contrast to the hydraulic conductivity, k_h, which varies as the square of some effective pore size. Because of this independence of pore size, electro-osmosis can be effective for water movement in fine-grained soils compared to flow under or i_h

ex. $k_{h,s}$ = 1×10^{-5} m/s (sand)

$k_{h,c}$ = 1×10^{-10} m/s (clay)

$k_{e,clay\ \text{and}\ sand} = 5\times10^{-9}$ ㎡/s volt

For equal flow rates $k_h i_h = k_e i_e$, so

$$i_h = \frac{k_e}{k_h} i_e$$

If $\frac{\Delta E}{\Delta L}$ = 20 v/m (elect. pot. grad) → i_e

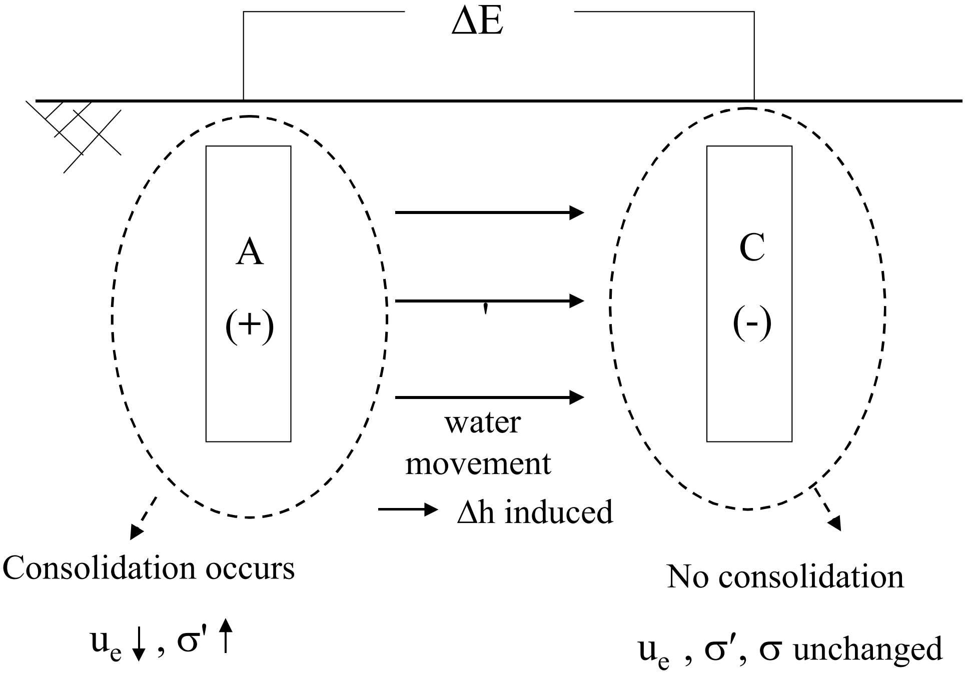
then, $i_{h,sand}$ = 0.01

$i_{h,clay}$ = 1000

$i_{h,sand}$ of 0.01 can move water as effectively as an electrical grad. of 20v/m in fine sand. However, for clay, $i_{h,clay}$ = 1000 would be needed to offset the electro-osmotic flow. 즉 20v/m을 걸어준 것이 i = 1000 준 것과 같은 효과

* Consolidation by Electro-Osmosis

Electro-osmosis draws water to cathode where it is drained away, then consolidation of the soil between the electrodes occurs in an amount equal to the volume of water removed.



Consolidation continues until hydraulic force that drives water back toward the anode exactly balances the electro-osmosis force driving water toward the cathode. Then,
i) How much consolidation will there be?
ii) How long will it take?

Answers obtained by using the coupled flow equations in place of Darcy's law in consolidation theory.

→ Mitchell pp. 277-281

■ Assumptions

1. Homogeneous and saturated soil
2. The physical and physicochemical properties of the soil are uniform and constant with time
3. No soil particles are moved by electrophoresis,
4. The velocity of water flow by electroosmosis is directly proportional to the voltage gradient,
5. All the applied voltage is effective in moving water,

6. The electrical field is constant with time
7. The coupling of hydraulic and electrical flows can be formulated by equations[13]
8. There are no electrochemical reactions

■ Governing Equations

For one dimensional flow between plate electrodes equations becomes,

$$q_h = -\frac{k_h}{\gamma_w}\frac{\partial u}{\partial x} - k_e\frac{\partial V}{\partial x} \quad \cdots \ 12.135$$

for the flow rate per unit area. For radial flow for the conditions show in Fig 12.37*b* and a layer of unit thickness

$$q_h = \frac{-k_h}{\gamma_w}\frac{\partial u}{\partial r} \cdot 2\pi r - k_e\frac{\partial V}{\partial r} \cdot 2\pi r \quad \cdots \ 12.136$$

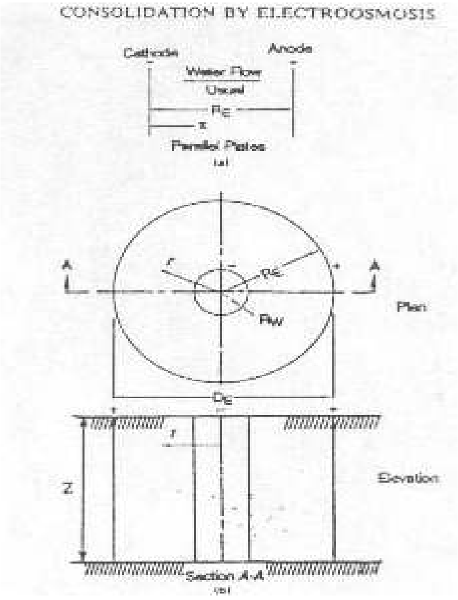


Fig. 12.37 Electrode geometrics for analysis of consolidation by electroosmosis (a) One-dimensional flow (b) Radial flow

Introduction of equation(12.135) in place of Darcy's law in the derivation of the diffusion equation governing consolidation in one dimension leads to

$$\frac{k_h}{\gamma_w}\frac{\partial^{2u}}{\partial x^2} + k_e\frac{\partial^{2V}}{\partial x^2} = m_v\frac{\partial u}{\partial t} \quad \cdots \ 12.137$$

and

$$\frac{\partial^{u}}{\partial x^2} + \frac{k_e}{k_h}\gamma_w\frac{\partial^{2V}}{\partial x^2} = \frac{1}{c_v}\frac{\partial u}{\partial t} \quad \cdots \ 12.138$$

where m_v is the compressibility and c_v is the coefficient of consolidation. For radial

---

[13] $q_w=k_h i_h+k_e i_e$, $I=\sigma_h i_h+\sigma_e i_e$

flow, the use of equation(12.136) gives

$$\frac{\partial^{2u}}{\partial r^2}+\frac{k_e}{k_h}\gamma_w\partial^2\frac{V}{\partial r^2}+\frac{1}{r}\left(\frac{\partial u}{\partial r}+\frac{k_e}{k_h}\gamma_w\frac{\partial V}{\partial r}\right)=\frac{1}{c_v}\frac{\partial u}{\partial t}\quad\cdots\ 12.139$$

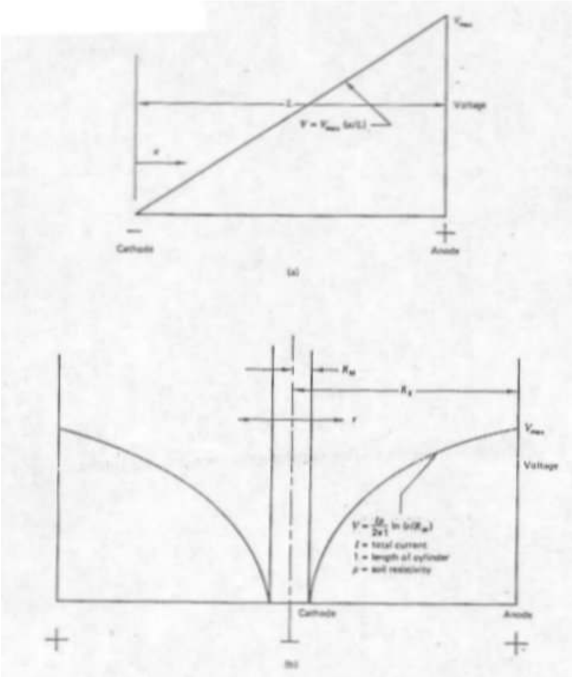


Fig. 12.38 Assumed variation of voltage with distance during electroosmosis (a) One-dimensional flow (b) Radial flow.

Both V and u are functions of position, as shown in Fig.12.38 V is assumed constant with time; whereas, u varies.

■ Amount of Consolidation

When the hydraulic gradient that develops in response to the differing amounts of consolidation between the anode and cathode produces a counterflow $(k_h/\gamma_w)\cdot(\partial u/\partial x)$ that exactly balances the electroosmotic counter flow $k_e(\partial V/\partial x)$, consolidation is complete. As there is no flow, q_h in equation (12.135) and (12.136) is zero. Thus equation (12.135) is

$$\frac{k_h}{\gamma_w}\frac{\partial u}{\partial x}=-k_e\frac{\partial V}{\partial x}\quad\cdots\ 12.140$$

or $du=-\frac{k_e}{k_h}\gamma_w dV\quad\cdots\ 12.141$

The solution of this equation is

$$u=\frac{-k_e}{k_h}\gamma_w V+C\quad\cdots\ 12.142$$

At the cathode, V=0 u=0; therefore, C=0, and the pore pressure at equilibrium at any point is given by

$$u = -\frac{k_e}{k_h}\gamma_w V \cdots (12.143)$$

where the values of u and V correspond to any point of interest. A similar result is obtained from equation (12.136) for radial flow.

Equation (12.143) indicates that electroosmotic consolidation continues at a point until a negative pore pressure, relative to the initial value, has been developed that depends on the ratio k_e/k_h and on the voltage at the point. For consolidation of constant total stress, there must be an equal and opposite increase in the effective stress. It is this increase in effective stress that is responsible for consolidation. For the one-dimensional case, consolidation by electroosmosis is analogous to the loading shown in Fig.12.39.

For a given voltage, the magnitude of effective stress increase that develops depends on k_e/k_h. As k_e only varies within narrow limits for different soils, the amount of consolidation that can be achieved depends largely on k_h. However, the amount of consolidation in any case depends on the soil compressibility as well as on the change in effective stress. For a linear soil, the coefficient of compressibility a_v is

$$a_v = -\frac{de}{dp} = \frac{de}{du} \cdots 12.144$$

or

$$de = a_v du = -a_v d\sigma' \cdots 12.145$$

in which $d\sigma'$ is the increase in effective stress.

It follows, therefore, that electroosmosis will be of little value in an oversonsolidated clay unless the effective stress increase are large enough to bring the material back into the virgin compression range.

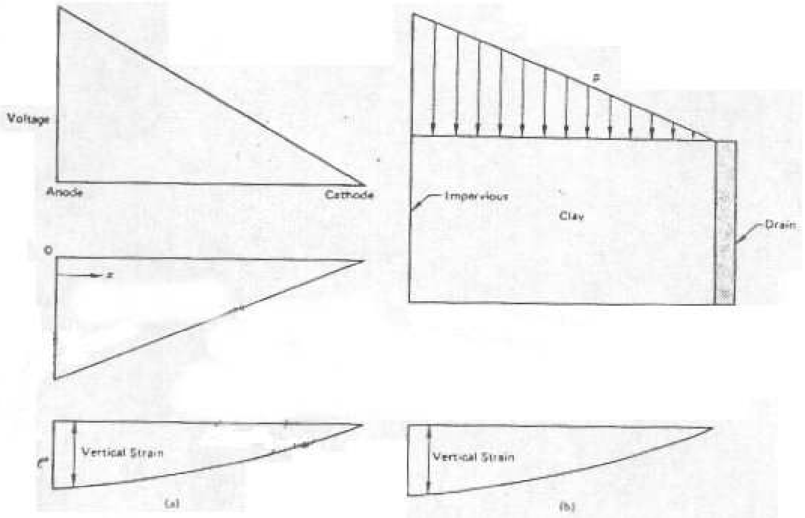


Fig. 12.39 Consolidation by electroosmosis and by direct loading. one-dimensional case. (a) Electroosmosis. (b) Direct loading.

■ Rate of Consolidation

Solutions for equations (12.138) and (12.139) have been obtained for several cases (Esrig, 1968,1971). For the one-dimensional case, and assuming a freely draining (open) cathode and a closed anode(no flow), the pore pressure is

$$u = \frac{k_e}{k_h}\gamma_w V(x) + \frac{2k_e\gamma_w V_m}{k_h\pi^2} \quad \cdots \ 12.146$$

$$\cdot \sum_{n=0}^{\infty}\frac{(-1)^n}{(n+1/2)^2}\sin\left[\frac{(n+1/2)\pi x}{L}\right]$$

$$\cdot \exp\left[-(n+1/2)^2\pi^2 T_v\right]$$

where V(x) is the voltage at x, V_m is the maximum voltage, and Tv is the times factor, defined in terms of the distance between electrodes L and real time t as

$$T_v = \frac{c_v t}{L^2} \quad \cdots \ (12.147)$$

$$C_v = \frac{k_h}{m_v\gamma_w} \quad \cdots \ (12.148)$$

The average degree of consolidation U as a function of time is

$$U = 1 - \frac{4}{\pi^3}\sum_{n=0}^{\infty}\frac{(-1)^n}{(n+1/2}\Big)^3\exp\left[-(n+1/2)^2\pi^2 T_v\right] \quad \cdots \ 12.149$$

Solutions for equations (12.146) and (12.149) are shown in Fig.12.40 and 12.41. They are applied in the same way as the theoretical solution for classical consolidation theory.

A numerical solution to equation (12.149) has been obtained, with the results shown in Fig. 12.42 (Esrig, 1968, 1971). For the case of two pipe electrodes, a more realistic field condition than the radial geometry of Fig. 12.37b, Fig. 12.42 cannot be expected to apply exactly. Along a straight line between two pipe electrodes, however, the flow pattern is approximately the same as for the radial case for a considerable distance from each electrode.

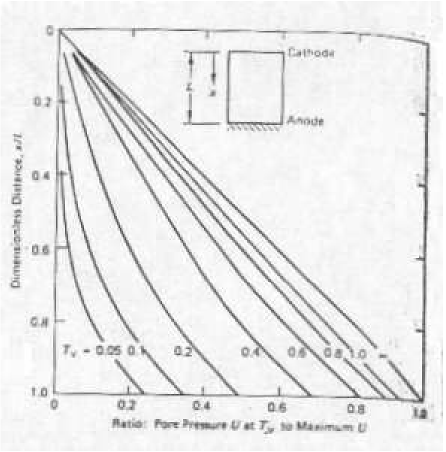


Fig. 12.40 Dimensional pore pressure as a function of dimensionless time and distance for one-dimensional consolidation by electroosmosis

A solution for the rate of pore pressure buildup at the cathode for the case of no drainage(closed cathode) is shown Fig. 12.43. This condition is relevant to pile driving, pile pulling, reduction of negative skin friction, and recovery of buried objects. Special solutions for in situ determination of soil consolidation properties by electroosmosis measurements have also been developed(Banerjee and Mitchell, 1980)

One of the most important points to be noted from these solutions is that the rate of consolidation depends completely on the coefficient of consolidation, which varies directly as k_h, but is completely independent of k_e. Low values of k_h, as is the case in highly

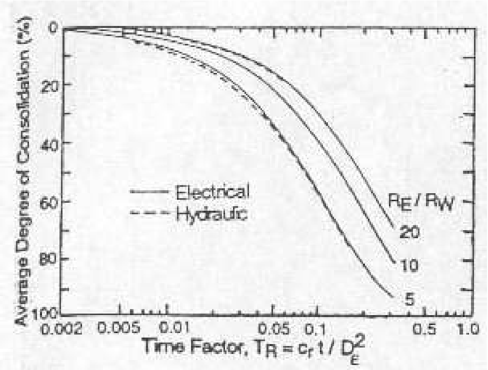


Fig. 12.42 Average degree of consolidation as a function of dimensionless time for radial consolidation by electroosmosis. (From Esrig, 1968). Reprinted from the Journal of the Soil Mechanics and Foundation Division, ASCE, Vol. 94, No.4, pp899-921. Copyright 198. With Permission of ASCE.

plastic clays, mean long consolidation times. Thus, whereas a low value of k_h, means a high value of k_e/k_h and the potential for a high effective consolidation pressure, it also means longer required consolidation times for a given electrode spacing. The optimum situation is one in which k_e/k_h is high enough to generate a large pore water tension of reasonable electrode spacing (2m～3m) and maximum voltage(50V～150V DC), but k_h is high enough to enable consolidation in a reasonable time. The soil types that best satisfy these conditions are silts, clayey silts, and silty clays. Most successful field applications for consolidation have been in these types of materials. As noted earlier, the electrical conductivity of the soil is also important; if it is too high, as in the case of high salinity pore water, the economics for application of electroosmosis may be unfavorable.

Electroosmosis was used for the consolidation of a quick clay at As in Norway(Bjerum et al., 1967). Water contents, strengths, and plasticity values as a function of position between electrodes are shown in Fig.12.44. Values of these properties before treatment are also shown for comparison. The variations in strength and water content after treatment are consistent with the patterns to be expected based on the predicted variation of pore pressure decrease and vertical strain stress increase with voltage and position shown in Fig. 12.39

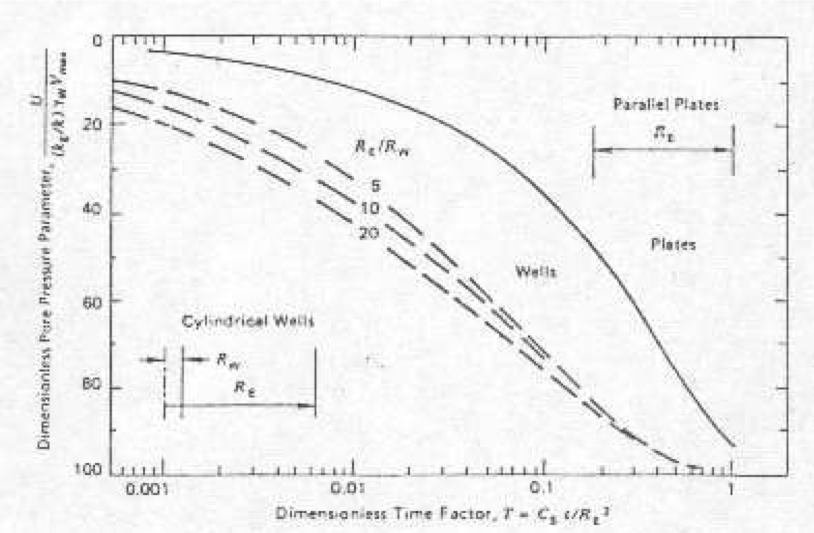


Fig. 12.43 Dimensionless pore pressure at the face of a cylindrical electrode as a function of dimensionless time for the case of a closed cathode (A swelling condition) (from Esrig and Henkel, 1968)

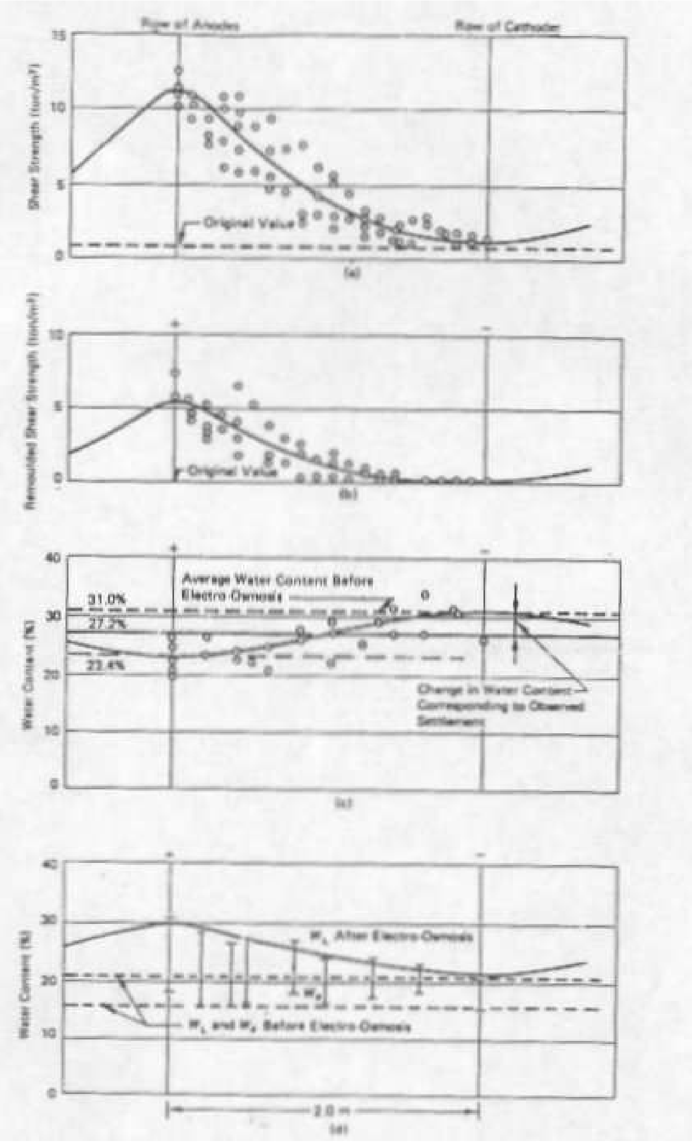


Fig. 12.44 Effect of electroosmosis treatment on properties of quick clay at Ås, Norway (from Bjerrum et al., 1967) (a) Undrained shear strength. (b) Remolded shear strength. (c) Water content. (d) Atterberg limits.

Thermal Properties of Soils

*Importance

ex. ① Loss and recovery of bearing capacity under the effect of freezing-thawing

ex) frost heaving

② Moisture movements in any type of pavement components, embankments and earth dams and clay liners T → μ에 영향 → k 변화

③ Volatilization of VOCs in subsurface

T → volatil. enhanced →

Sources

variation of T. comes from

① atmospheric, solar, climate, geothermal - nature

② heat dissipation from buried structure - man made

* Characteristics of Heat

Heat is a form of energy

Heat Transfer Process

①Conduction

Conduction is the process in which heat energy is transferred between molecules within a substance, between two substances in physical contact, or by direct molecular interaction

-Rate of heat flow or the thermal conductivity (k_T) of a substance is dependent on the capacity of its molecules to give and adsorb heat.

ex) air → poor conductor, but excellent insulator

dry soil with high porosity → increasing thermal resistivity

* Thermal conductivity

$k_T = q/iAt$

* Thermal resistivity

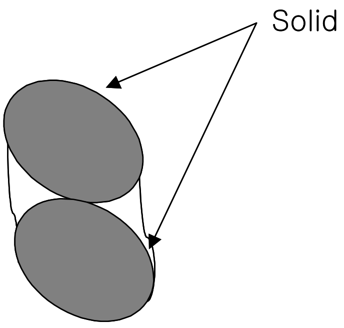
$R_T = 1/k_T$

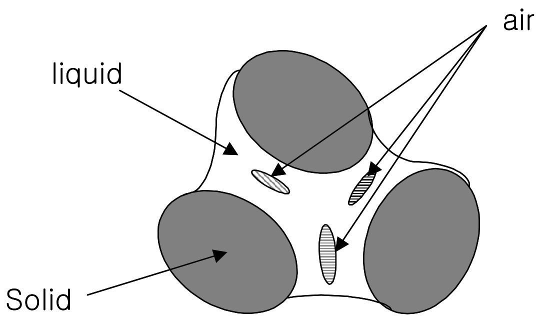
② Convection

Convection is defined as the heat transfer between a surface and moving liquid or gas or the transfer of heat by the movement of molecules from one point to another.

In convection processes, heat always moves from warm to cool.

In most soils, the heat transfer process within the soil mass is conduction or convection or both.





Conduction
Heat transfer by direct
contact between two soil particles

Convection
Heat transfer by moving liquid or air

For soil, in addition to its density and composition the moisture content and texture of the soil will also greatly affect the thermal behavior.

Mass Heat Capacity(C_m) 질량 열용량

Defined as the quantity of heat (Q) required to raise a unit weight of material 1℃
단위 무게의 물질을 1도 올리는데 필요한 열량

$$C_m = Q/\text{Temp.}\ (\text{J/g}\cdot{}^\circ\text{C})$$

Volumetric Heat Capacity(C_m) 체적 열용량

Defined as the quantity of heat (Q) required to raise a unit volume of material 1℃

$$C_v = C_m\gamma(\text{J/mm}^3\cdot{}^\circ\text{C})$$

Specific Heat

$$C_{sp} = \frac{C_{m,\,\text{substance}}}{C_{m,\,\text{water}}}\ \text{, dimensionless}$$

ex. Kaolinite 0.227, Na-Montmorillonite 0.191

*Soil-Heat Interaction

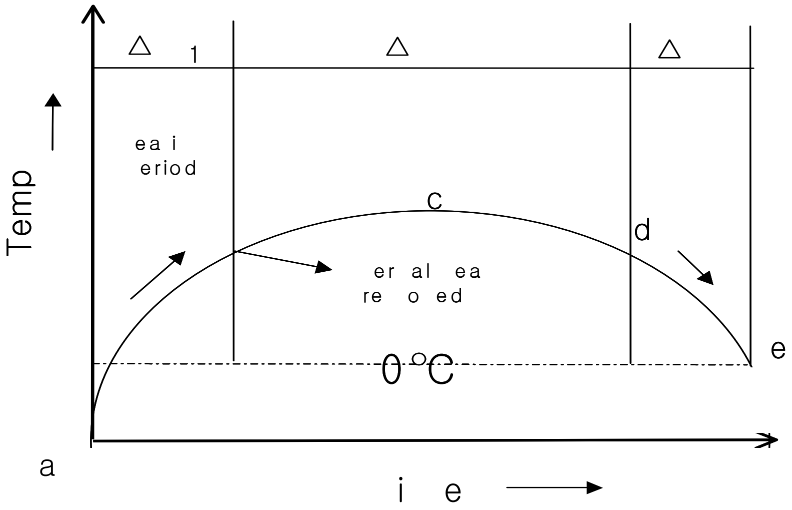
water movement in the vapor phase
lightness of water molecule

$$k_{T,\ \text{vertically upward}} > k_{T,\ \text{vertically downward}}$$

The ability of soil to retain or dissipate heat is dependent on its heat capacity and thermal conduction. C_m , C_v

k_T

Soil Thermal Storage Capacity (TSC)



ΔT_1 : heating period, heat energy being absorbed at the depth being monitored

ΔT_2 : b-d no more external heat provided, but energy still being received

TSC : Area under b, c, d, and e

ΔT_3 : dissipation stage, d-e

Ideal soil for thermal storage application?

small ΔT_1 and large ΔT_2 (b, c, d, high)

*Thermal Conductivity, k_T (Q/iAt)

k_T of a particular material is defined as the quantity of heat, *Q*, which flows normally across a surface of unit area, *A*, of the material per unit time, *t*, per unit temperature gradient, *i*, normal to the surface.

R_T(thermal resistivity) = 1/ k_T

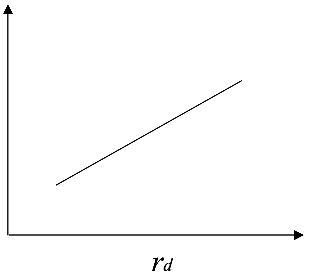
k_T=fn(k_T of solid, liquid and gaseous compo., their volume proportions, arrangement of particles, size, shape, gradation)

$k_{T,\ solid} > k_{T,\ liquid} > k_{T,\ gas}$

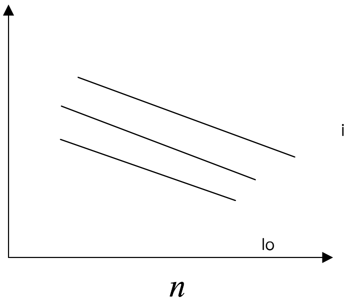
∴ To describe the thermal conductivity of an aggregate of soil particles, it is necessary to describe the manner in which the solid particles either make contact with each other or are separated by intervening liquid and gas.

Fig 8.5 (Fang. p.233)

*Dry density increases the thermal conductivity



*Decreasing the w and increasing the porosity will decrease k_T



*Degradation, well-graded → fills void better → better contact between particles → increasing density → $k_T \uparrow$

*Time Sand containing high % of silica, carbonates → cementation with time → increase of k_T with time

* Thermal Resistivity

$$R_T = \frac{1}{k_T} = fn\ (density,\ w,\ D,\ claycontent,\ etc.)$$

① particle Aspects --> *show Table 8. 3 & 8. 4*

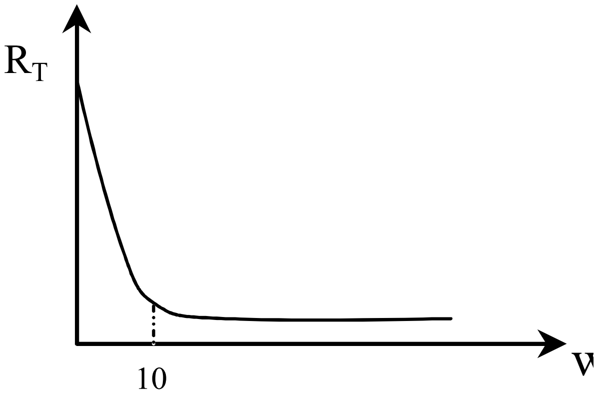
Quartz(solid) < Rocks < soil

(granite<sandstone ;more nonaqeuous) (dense < loose , wet soil < dry soil)

Table 8.3 & 8.4

Material	R_t (∘ c - cm/watt)
Air	4000
Granite	25~58
Limestone	45
Mica	170
Organic Matter(wet)	400
Organic Matter(dry)	700
Sandstone	58
water	165

② Moisture content and density --> *show fig. 8.8*



i) Air(high R) increase the overall R_T of soil

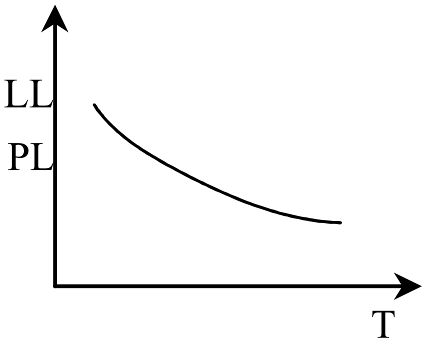
ii) Compaction reduce R_T, both by reducing V_v and improving contact between the solid particles

iii) very small amount of water added → thin film (solid solution)
→ increase the particle contact area → reduce R_T

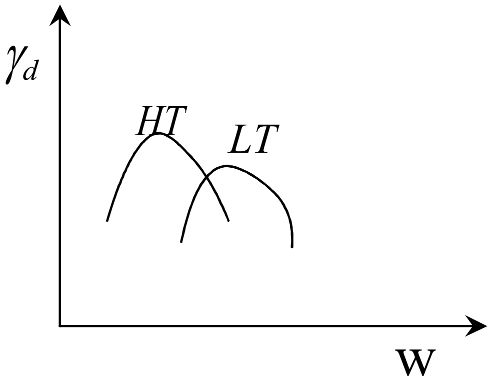
* Effects of Heat on Soil Behavior

T ↑ increase in kinetic energy of exchange ions → movement of hydrated ions
→ loosen (restrained water bound) → increase free water (while decreasing μ)
→ increase hydraulic conductivity, k

i) LL and PL of fine-grained soils decreased with increase in T
T↑, lubricating effect↑ 낮은 함수비에서도 액체거동



ii) Unit weight increased and OMC decreased when the temperature of the soil increased(high in clay content) --> T↑, lubrication↑ 더 쉽게 wetting(OMC↓)
향상된 다집효과



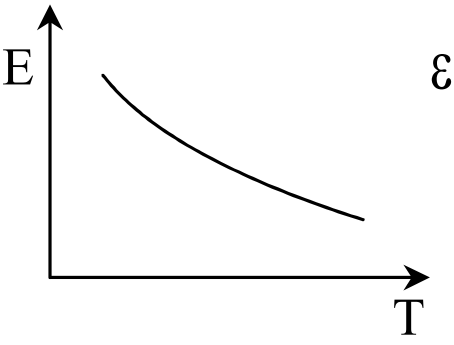
iii) Increase in compressibility with increase in temperature, the greatest effects being observed in the range of secondary consolidation.

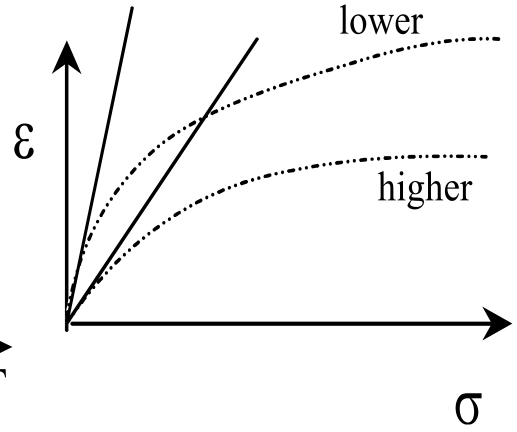
London clay 5。C → 15。C , C_v(압밀계수) increased(Lewes, 1950)
Lambe(1958) T↑ , decrease in soil volume
Habibagahi(1976) 25。C → 50。C , void ratio decreased under constant consolidation pressure.

iv) T↑, porewater pressure↑ (thermal expansion of water)
For less compressible materials, the porewater prerssure change was considerably greater

v) Elastic modulus of soil decreased with increase in temperature (Mitchell 1969)

Field plate load rests yield higher values for moduli and bearing values during the colder months





summary

soil properties	Temperature	
	Low	High
LL	High	Low
PL	High	Low
γ_{dmax}	Decrease	Increase
OMC	Increase	Decrease
e	High	Low
volume change	Swelling	shrinkage
C_v , secondary compression	Decrease	Increase
u	Decrease	Increase
shear strength	High	Low
E	High	Low
k	Decrease	Increase

* Frozen and Thawing Soils

○ Depth – Analytical approach ① Stefan eq. (Jumikis, 1955)

② Berggren eq. (Aldrich, 1956) → Terzaghi
압밀식과 유사

- in-situ measurement of <u>electrical resistivity</u>

Diff. clear for solid & liquid phase of wet

○ Frost heaving is affected by soil types (% finer than 0.074mm)
Increase in % fines will increase heaving
Rate of heave increases as % fines increased (Fig. 8. 15)
why? 모관현상 only? No. 흡착수는 고농도의 양이온 때문에 0。C보다 훨씬 낮은 온도에서 언다. 따라서 흡착수가 존재하는 세립토의 경우 물의 유동이 얼지 않고 계속 가능하여 Heaving을 확장시킨다.

○ Thawing → decreasing strength of soil (Table 8. 9) →Frost susceptibility

[illegible] will strength of soil be like

[illegible] vice)

[illegible] levels.

[illegible]

stand.

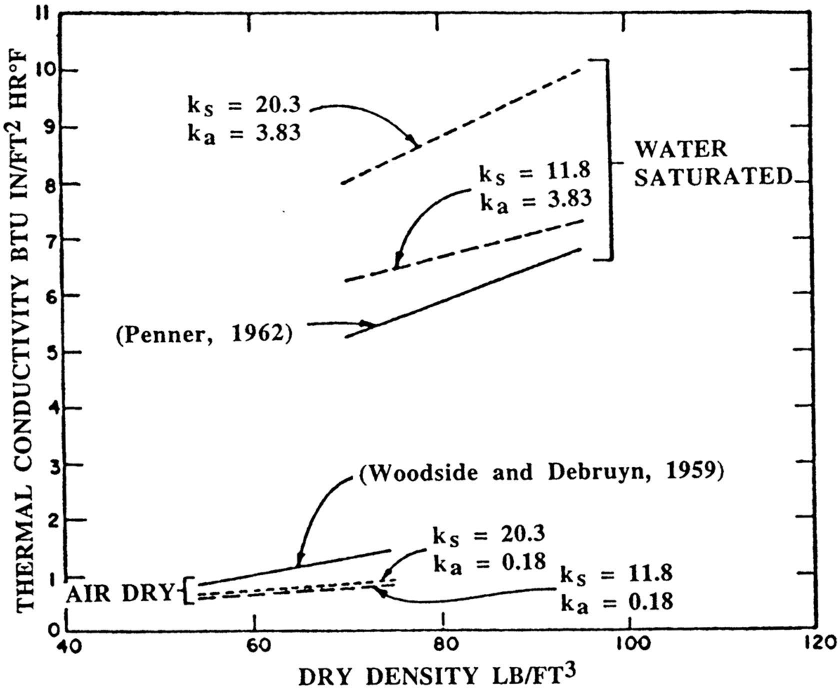


Figure 8.5 Thermal conductivity vs. dry density for Canadian Leda clay (After Penner, [1963], Proc. Int. Conf. of Permafrost, NAS-NRC Publ. 1287.)

BTU : British Thermal Unit
물을 1기압하에서 공기가 제거된물
1 lb를 60°F에서 61°F로 올리는데
필요한 열량 (= 1054.5 joules)

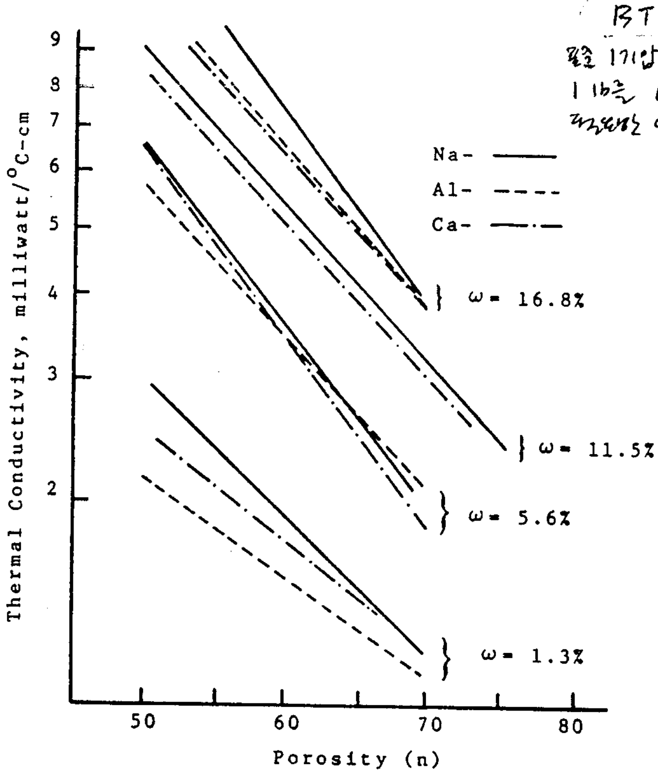


Figure 8.6 Thermal conductivity vs. porosity of natural kaolinite clay (After Reno, W.H. and Winterkorn, H.F. [1967], Thermal Conductivity of Kaolinite Clay as a Function of Type of Exchange Ion, Density and Moisture Content, HRR no. 209, pp. 79–85.)

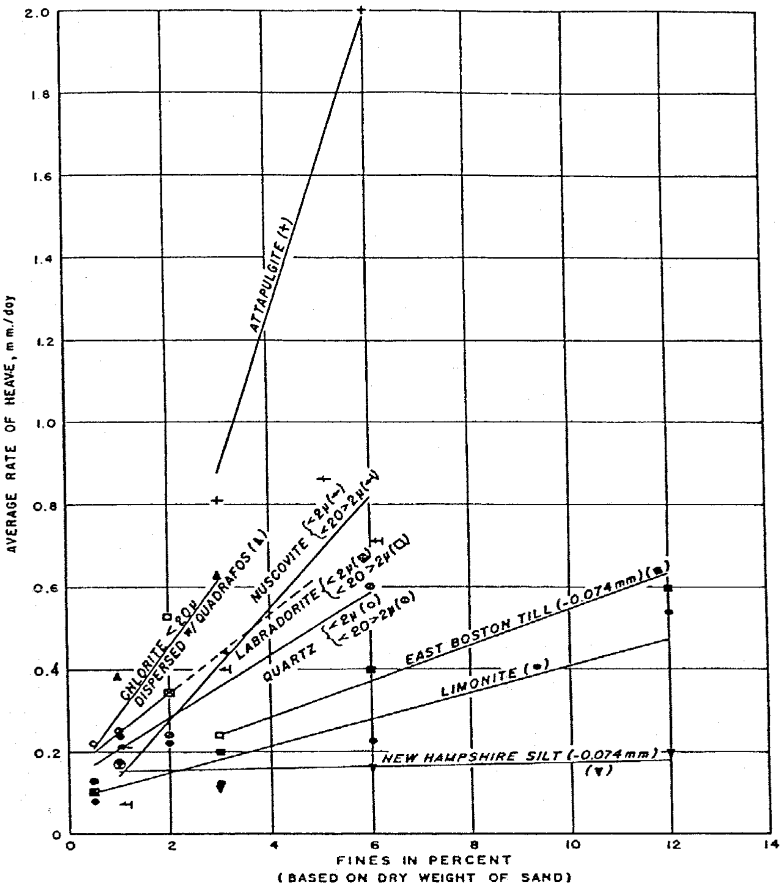


Figure 8.15 Effect of rate of heave on sand added with various amounts of clay minerals and clays. (After Linell, K.A. and Kaplar, C.W. [1959], The Factor of Soil and Material Type in Frost Action, HRB Bulletin 225, pp. 23–44.)

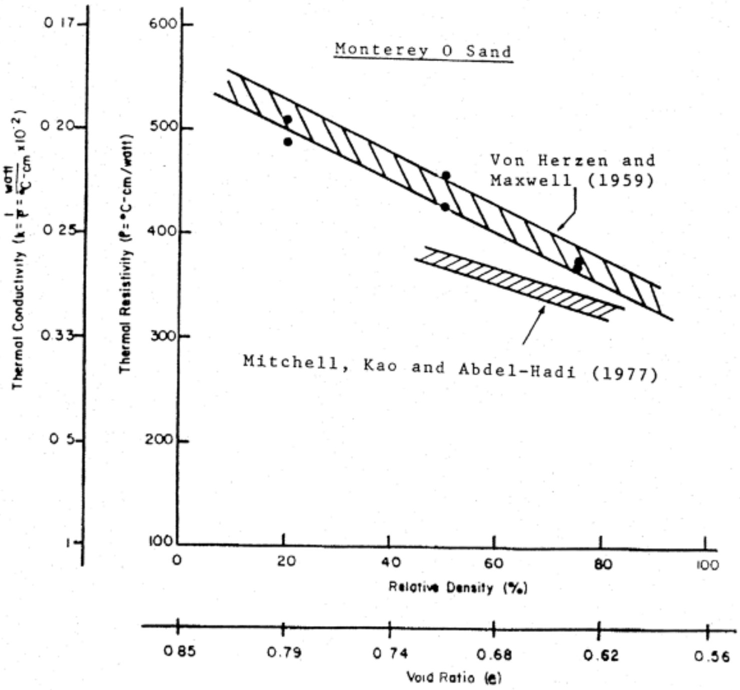


Figure 8.7 Thermal conductivity vs. relative density and void ratio of dry sand. (After Chaney, R.C., Ramavjaneya, G., Kanchanastit, P., and Fang, H.Y. [1983], *ASTM Geotech. Testing J.*, v. 6, no. 4, pp. 220–225.)

Table 8.9 Guidelines for Selection of Heave Rate or Frost Susceptibility Class

Unified classification (soil type)	Symbol	Percent (<0.02 mm)	Heave rate (mm/day)	Frost suscept. classification
Gravels and sandy Gravels	GP	0.4	3.0	Medium
	GW	0.7–1.0	0.3–1.0	Neg. to low
		1.0–1.5	1.0–3.5	Low to medium
		1.5–4.0	3.5–2.0	Medium
Silty and sandy Gravels	GP-GM	2.0–3.0	1.0–3.0	Low to medium
	GW-GM	3.0–7.0	3.0–4.5	Medium to high
	GM			
Clay and silty Gravels	GW-GC	4.2	2.5	Medium
	GM-GC	15.0	5.0	High
	GF	15.0–30.0	2.5–5.0	Medium to high
Sands and gravelly Sands	SP	1.0–2.0	0.8	Very low
	SW	2.0	3.0	Medium
Silty and gravelly Sands	SP-SM,	1.5–2.0	0.2–1.5	Neg. to low
	SW-SM,	2.0–5.0	1.5–6.0	Low to high
	SM	5.0–9.0	6.0–9.0	High to very high
		9.0–22.0	9.0–5.5	
Clay and silty Sands	SM-SC	9.5–35.0	5.0–7.0	High
	SC			
Silts and organic Silts	ML-OL,	23.0–33.0	1.1–14.0	Low to very high
	ML	33.0–45.0	14.0–25.0	Very high
		45.0–65.0	25.0	Very high
Clay silts	ML-CL	60.0–75.0	13.0	Very high
Gravelly and sandy Clays	CL	38.0–65.0	7.0–10.0	High to very high
Lean clays	CL	65.0	5.0	High
	CL-OL	30.0–70.0	4.0	High
Fat clays	CH	60.0	0.8	Very low

After Moulton (1991).

content of the soil play an important role. The amount of unfrozen water increases in the direction of sand > silt > clay.

ELECTRICAL CONDUCTIVITY

*Introduction

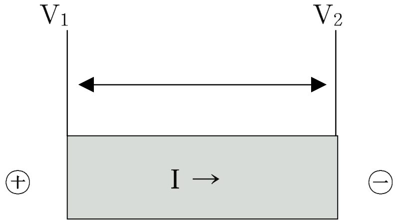
Ohm's law

$I = \sigma\, i_e A = -\sigma \frac{\partial V}{\partial X} A = \frac{V}{R}$ $(q_H = k\, i_H A)$

i_e= gradient = $-\frac{\partial V}{\partial X}$ (V/m) $\left(i_H = -\frac{\partial h}{\partial X} \right)$

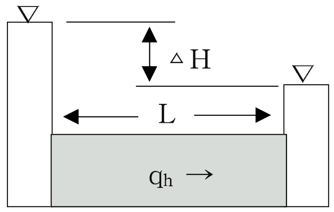
I = current (amp) (q_h = flow rate, m^3/s)

σ= electrical conductivity (siemens/m) (k = hydraulic conductivity, m/s)



Ohm's law

$$I = \sigma \frac{\triangle V}{L} A$$



Darcy's law

$$q_h = k \frac{\triangle H}{L} A$$

$\sigma = \frac{1}{R} \cdot \frac{L}{A}$ (siemens/m) (E. conductivity equals the inverse of the E. resistivity)

R : resistance (ohm)

L : length of sample (m)

A : cross-sectional area (m^2)

σ: 0.01~1.0 siemens/m for a saturated soil

σ= f_n(n, S, conductivity of pore water, particle size, shape, surface conductance, soil structure fabric, cementation, temperature)

E. measurement → applicable for petroleum Eng. geophysical mapping, contaminant subsurface plume delineation.

No theoretical equations expressing E. conductivity as a function of all the variables are not available

→ ∵ inherent complexity of soil-water system

However, emperical and/or theoretical eq's based on simplified models are available depending on particular soil and conditions.

* Nonconductive Particle Models

: distinguish b/w approaches of nonconductive solids and those that attempt to include the contribution of clay's surface conductance

- Formation factor

Archie (1942) : σ is directly proportional to that of the pore water based on the experiments on clean saturated sands and sandstones.

For S=1 ; F(formation factor)= $\dfrac{\rho_T}{\rho_W} = \dfrac{\sigma_T}{\sigma_W}$

ρ_T : resistivity of soil(saturated)

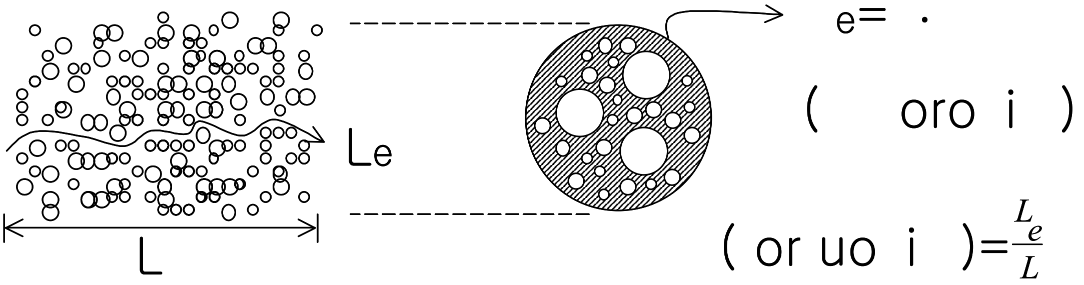
ρ_W : resistivity of pore water

F = n^{-m} (for clean sands and sandstones)

m = 1.3 for loose sand
= 2.0 for highly cemented sandstones
n = porosity

For S≤1 ; F = $S^2 \dfrac{\sigma_w}{\sigma_T}$

- Capillary Model



F = $\dfrac{T^2}{n}$ (for saturated soil, S=1)

F = $\dfrac{T^2}{nS}$ (for S<1)

- Two-Parallel resistor model

(considering the contribution of the ions concentrated at the surface of negatively charged particles)

System is equivalent to two electrical resistors in parallel

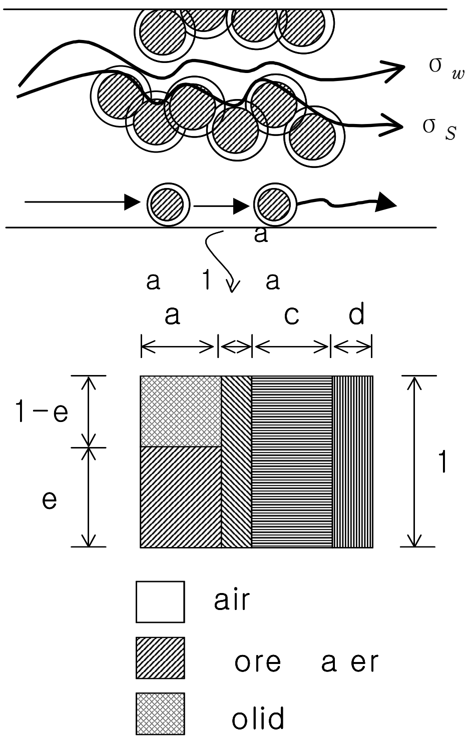
$\sigma_T = X(\sigma_T + \sigma_S)$

σ_s : surface conductivity term

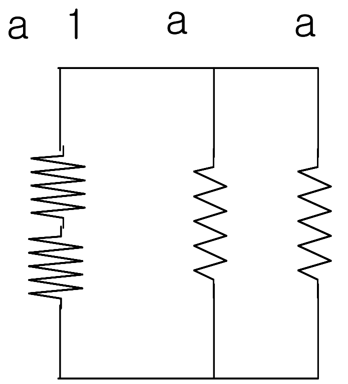
X : constant analogous to the reciprocal of the formation factor that represents the internal geometry

Good for clay-bearing soils.

It fails to include the contribution of the surface conductance and pore water conductance in a series path assumed constant but dependent on the concentrations of electrolyte in the pore water.



· Three element network model



$$\sigma_T = \frac{a\sigma_W\sigma_S}{(1-e)\sigma_W + e\sigma_S} + b\sigma_S + c\sigma_W$$

$f_n(n, s)$ a, b, c, d, e : geometric parameter

Flow along particle surfaces and through pore water in series in addition to the paths included in the two-parallel resistor model

For sands(Archie, 1942) , $\sigma_T = Constant \times \sigma_W$ $(\sigma_S = 0)$

(Mitchell, pp,250. Fig.12.21)

Smith and Arulanaudan(1981) modified three-element model by adding a capacitor in parallel with each capacitor. The resulting equations can be fit to experimental frequency dispersions of the conductivity and apparent dielectric constant.

* Practical Range of Flow Parameters for Fine-Grained Soils (fully saturated)

* Simultaneous Flows of Water, Current, and Chemicals(Salts) through a Soil

Yeung and Mitchell(1992)

The analysis applies to an initially homogeneous soil mays that separates solutions of different concentrations of anions and cations, at different electrical potentials, and under different hydraulic heads.

Driving Forces : $\nabla(-P)$ hydraulic gradient
$\nabla(-E)$ electrical gradient
$\nabla(\mu_c^c)$, $\nabla(\mu_a^c)$ chemical potential grad. of cations and anions respectively.

Flux : J_v volume flow rate of solution/unit area (hyd.)
I electrical current (elect.)
J_c^d, J_a^d diffusional flow rate of cations and anions per unit area (chem.)

The set of equations that relates the four flows and driving forces is ;

The flow equations can be incorporated into numerical models for the solution of transient flow problems.

Conservation of mass requires that ;

$$\frac{\partial C_i}{\partial t} = -\nabla J_i - \nabla G_i$$

G: in a sink-source term describing the addition or removal of species I from solution.
ex. first-order decay such as biodeg. , half-life decay etc.