

Experimental and Theoretical Consideration of Liquid Limit

액성한계에 대한 실험적 및 이론적 고찰

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요 지

많은 경우 액성한계 이상의 함수비 상태를 갖는 지반을 액체상태로 표현한다. 액체상태의 공학적 의미는 유효응력이 존재하지 않는 상태로 볼 수 있으며, 전단강도가 0 임을 의미한다. 그러나 실제로는 액성한계 이상의 함수비를 갖는 지반에서도 유효응력이 존재하며, 따라서 전단강도가 존재한다. 연약한 해성점토의 경우는 이러한 경우가 비교적 흔하다. 일반적으로 액체상태란 용어에서 연상되는 지반상태는 물과 같은 상태로서 기술자로 하여금 지반의 상태에 대한 올바른 감을 가지는 것을 어렵게 한다. 본고는 Bolt의 electrical double layer 이론을 이용한 이론적 해석 및 실험적 자료들을 이용하여 액성한계란 용어의 올바른 재조명에 초점을 맞추었다.

Abstract

The condition soils that has higher water content than the liquid limit is interpreted as liquid state. In engineering sense, the liquid state may be interpreted as the condition of zero effective stress, and the soil does not have shear strength. However, the effective stress is not really zero for the soils that have higher water content than the liquid limit. This condition is not rare for soft marine clays. The conventional concepts of liquid limit tend to give the wrong idea of soil condition by giving the impression that the soil is in liquid condition when the water content is higher than the liquid limit. This study presents the correct interpretation of liquid limit by Bolt's double layer theory and experimental results.

Keywords : Liquid limit, Fluid limit, Water content, Double layer, Bolt equation

1. Introduction

Atterberg limits - one of the most widely used terminology in geotechnical engineering, are first proposed by Swedish agricultural soil scientist Albert Mauritz Atterberg (1846-1916), and Atterberg limits are obtained from the water content of the disturbed soil, and many soil properties can be predicted easily from these index properties. However, there are some instances that this Atterberg limits are misunderstood and result in the wrong idea of the ground conditions. This study focuses especially on the correct interpretation of the liquid limit which is one of the most widely used index parameters of the soft soil.

Typically, many of the soil mechanics textbooks define

the liquid limit of the soil as the water content at which the soil is liquid state. This expression induces the image of water as the soil condition, even though it is not correct in reality. In the nature, one can find many cases that the soil has higher water content than the liquid limit, though the soil is not a liquid state. In the laboratory, one may determine the liquid limit using the ASTM procedure; which is the water content when the bottom one centimeter of the soil groove meet each other at twenty five dish drops. At the water content which is higher than the liquid limit, the bottom one-centimeter of the soil meet each other before twenty five dish drops, maybe twenty dish drops. However, one cannot say that the behavior of the soil whose bottom one centimeter groove meets at 20

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dish drops is similar to the liquid.

This example shows that the typical concept of the liquid limit may give the wrong idea of the soil condition. Thus this study tried to clarify the meaning of the liquid limit using the theoretical consideration and the experimental results.

2. Traditional Concepts of Liquid Limit

Except K. Terzaghi's (1943) book "Theoretical Soil Mechanics", most of the soil mechanics textbooks deal with the Atterberg limits. Atterberg proposed these limits in his article "Die Plastizität und Kohärenz der Tone und Lehme (Vorläufige Mitteilung)", *Chemische Zeitschrift*, 34(42): 369-371, 34(43): 379-380, and started to use it from 1910s. However, the application was limited until when A. Casagrande (1932) proposed the standard experimental procedure for Atterberg limits and K. Terzaghi (1925) adapted it in geotechnical engineering.

The first man who realized the importance of Atterberg limit and adapted in geotechnical engineering is Terzaghi (Brand and Brenner, 1981). Terzaghi mentioned the Atterberg limit as follows. The consistency of the remolded clay can be changed by increasing or decreasing the water content. For example, if one decreases the water content of clay slurry, the state of clay slurry will be changed from liquid state to plastic state and finally to solid state. The water content at the state change is clearly distinguished, and these properties can be used to distinguish the clay properties. But the state change of clay material occurs gradually, and there is not clear boundary between them. Atterberg limits are one of the most reasonable method to determine the above mentioned boundary water content.

Terzaghi's brief statement about Atterberg limits can be interpreted as the water content at which the clay slurry changes the state from slurry to plastic state. The slurry itself is very close to the liquid, thus Terzaghi's liquid limit can be interpreted as the water content at which the soil changes the state from the liquid state to the plastic state. However, Terzaghi also indirectly mentioned that the soil at liquid state is not really the liquid by stating that the undrained strength of the natural soil at the liquid limit is

approximately 1.5 to 5 t/m².

Croney and Coleman (1954), from the results of the measurements of the pore-water tension in clay soils, reported that the soil has the negative pore water pressure to the order of 0.04 t/m² even at the liquid limit. It can be inferred that these results back up the fact that there exists the effective stress even at the liquid state.

Norman (1958) reported that the undrained shear strength of the clayey soil is approximately 0.25 t/m² at the liquid limit. Casagrande (1958) supported Norman by adding that the test results at Harvard showed the range of shear strengths of clayey soil to be 0.14 to 0.2 t/m² at the liquid limit. These old works indicate that the soil may have shear strength at the liquid limit.

Scott (1963) stated, "Cohesive soils have the undrained shear strength 0.2 to 0.3 t/m² at the liquid limit; also even at some range of the water content which is higher than the liquid limit, the shear strength is not completely zero, it just gradually decreases." Thus one can know that Scott's understanding of the liquid limit is contrast to the common understanding that the shear strength is zero at the liquid limit.

Seed et al. (1964) carried out the broad experimental research about the Atterberg limits. From their experimental results, they proposed that the soils have shear strength 0.2 to 0.25 t/m² at the water content of liquid limit. This result also backs up the hypothesis that the soil state is not liquid state even after the liquid limit.

Monte and Krizek (1976) stated the followings on the liquid limit and liquid state. Typically, the soil is said to be a liquid state at the water content higher than the liquid limit. However, there are evidences that clayey soils have some shear strength, thus the soil is not truly liquid state even at the higher water content than liquid limit. If the water content of the clayey soils is continuously increased, the soils become truly liquid state, and the minimum water content at this state can be defined as a fluid limit. At the fluid limit, the soil particles do not directly contact each other and the effective stresses do not exist. Typically, the fluid limit is 4 to 5 times the water content at liquid limit. When Monte and Krizek (1976)'s fluid limit is interpreted more easily, it can be said that the liquid limit is the

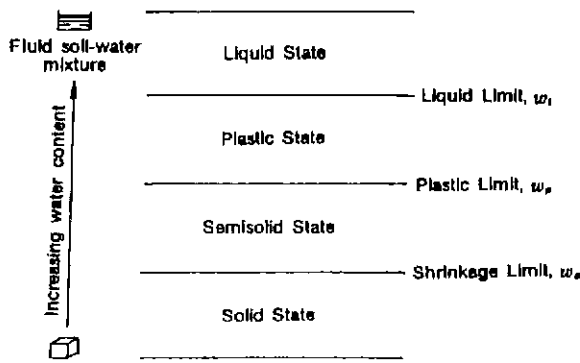


Fig. 1 Definition of Atterberg limits (after Lambe and Whitman, 1979)

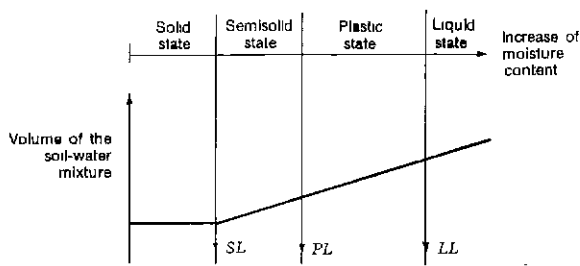


Fig. 2 Conceptual diagram of Atterberg limits (after Das, 1984)

boundary water content whether the soil is in the water or whether the water is in the soil. It can also be interpreted that the effective stress and shear strength exist below the fluid limit. Thus it can be inferred that the fluid limit by Monte Krizek (1976) has qualitatively closer meaning to the dictionary meaning of liquid limit, while the quantitative liquid limit is obtained from the Casagrande's test method.

Lambe and Whitman (1979)'s Soil Mechanics is regarded as the classical of soil mechanics textbook. They mentioned the liquid limit as shown Fig. 1. From Fig. 1, it can be inferred that the soil at the water content higher than liquid limit can be regarded as liquid state and it is conceptually different from that of Croney and Coleman (1954), Norman (1958), Scott (1963), Monte and Krizek (1976)'s concept.

Craig (1983) explained the liquid limit in a slightly different way considering that the soil becomes stiff at the decreased water content due to the close distance between the soil particles. Plastic state can be defined as the state at which the soil particles can dislocate freely without losing inter particle force. The maximum water content at which

the soil can keep plastic state is the liquid limit and the minimum water content at which the soil can keep plastic state is the plastic limit. This statement can be interpreted as that the soil particles can dislocate freely but lose inter-particle force at the water content higher than the liquid limit. Thus the interpretation of Craig (1983) liquid limit and liquid state is essentially same as Lambe and Whitman (1969).

Das (1984) mentioned the Atterberg limits as follows. Clayey soils can flow like a liquid when it is mixed with enough amount of water. Then as we decrease the water content, the state changes from liquid state to plastic state, from plastic state to semi solid state, and from semi solid state to solid state. The boundary water content between liquid state and plastic state is called liquid limit, and the boundary water content between plastic state and semi solid state is called plastic limit, etc. (Fig. 2)

Thus one can infer that the concept of liquid limit in Das (1984) is essentially same to Lambe and Whitman (1969) and Craig (1983).

Locat et al. (1984) reported the liquid limits of the fifteen Canadian clayey soils. He reported that the natural water content of most of the investigated clayey soils were higher than liquid limit and the undisturbed undrained shear strength was on the range from 2.3 to 24.5 t/m². Locat et al. (1984) results showed that the soil can have a significant shear strength depending of the fabric textures, phyllosilicates and amorphous contents.

Song (1992) reported that the natural water content (85%) of Yeochon Bay Mud Clay of Korea is substantially higher than the liquid limit (60%) but it has the shear strength approximately 1 t/m². Song (1992) also reported the high fluid limit (322%) and the shear strength 0.1 t/m² even at the water content 135% which is more than two times of the liquid limit. Additionally, Song (1994) reported the worldwide cases of which the natural water content is higher than the liquid limit or nearly equal. Therefore it can be inferred that Song (1992, 1995)'s results clearly support the idea that the clayey soil does not behave like a liquid even at the high water content. Also Song's results agree with to Scott (1963), Monte and Krizek (1976) but disagree with Lambe and Whitman (1979),

Craig (1983), Das (1984), etc.

Mitchell (1993) clearly reported that the shear strength of soils is approximately 0.17 to 0.2 t/m^2 and pore water suction is approximately 0.6 t/m^2 based on other researches (Russell and Mickie, 1970; Wroth and Wood, 1978; Whyte, 1982).

From these discussions, one can see that there are different interpretations of liquid limit. Especially, one can see that most of the well known textbooks define the liquid limit as the threshold water content for liquid state even though the real condition of the soil is not the liquid state.

3. Analysis

3.1 Conceptual Approach

From the literature survey, it is quite clear that the general meaning of liquid limit is the boundary water content between the liquid state and plastic state while the soil is liquid state at the water content higher than the liquid limit in reality. However, the test method which is we use currently does not present the result which is faithful to its original meaning. Also, from the literature it was clearly known that the soil condition is not truly liquid state even after the liquid limit because it has effective stresses and shear strength.

In many cases, this low shear strength is not really valuable and can be neglected without arising severe

engineering problems. However, when one deals with the really soft ground, such as hydraulically filled ground, mine tailing deposit, or tailing dam of fossil fuel power plant, this small strength is very important. Therefore the clear understanding of the soil behavior at the water content higher than the liquid limit is essential.

The process of soil formation from the slurry is as shown in Fig. 3. Fig. 3, shows that the deposition and consolidation process of mine tailings which was flown into a pond as a water mixture state. In this figure each zone represents the following conditions.

Zone A : The soil particles are suspending around and the concentration is very low.

Zone B : The transition zone where the concentration of soil particles increase quickly

Zone C : The stagnant zone where the concentration of soil particles is almost constant. At the bottom, consolidation can take place.

Zone D : The concentration of soil particles is very high and consolidation can take place.

For this sedimentation and consolidation process, Yong (1984) suggested that only zone A is close to liquid state, which does not have any effective stress. Below zone A, however, he reported that there is some effective stress. From Fig. 3, one can infer that the water content at the boundary between zone A and B is fluid limit and the water

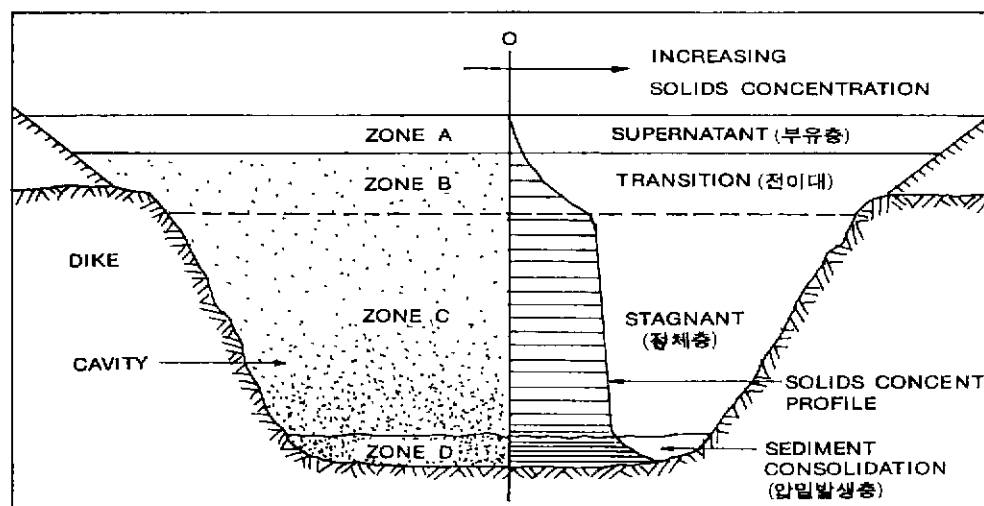


Fig. 3 Sedimentation and consolidation of slurry (after Yong, 1984)

content somewhere in zone B or C is the liquid limit. Pane and Schiffman (1985) expressed the effective stress of the soils which have the very high water content as equation (1)

$$\sigma = \beta(e) \sigma' + u_w \quad (1)$$

where, σ = total stress

β = coefficient of interaction which is a function of void ratio

σ' = effective stress

u_w = pore water pressure

Equation (1) shows that β equals 1 when effective stress theory can be applied perfectly, and β equals 0 when there is not any effective stress. Thus one can infer that the medium is perfectly liquid or fluid when β equals 0. The transition between β equals 1 and 0 is the soil state when the soil changes state from slurry to solid state; in this state $0 < \beta < 1$ and the soil is not truly liquid nor truly solid.

In Fig. 4, one can know that the β equals 1 is equivalent to $e \leq e_s$ (e represents the void ratio), and the complete effective stress concept can be applied. Then one can infer that e_s is the maximum void ratio from which effective stress concept is valid. From the point of $e > e_s$, the medium start to deviate from the effective stress concept and completely deviates from the effective stress concept when e equals e_m . The medium is completely liquid when $e \geq e_m$.

When one compare Fig. 3, Fig. 4 and equation (1), one can know that the state $e \geq e_m$ in Fig. 4 is equivalent to Zone A in Fig.3 and the state when β equals 0 in equation (1). Also, the state at which $e_s < e < e_m$, correspond to Zone

B and C in Fig. 3, and $0 < \beta < 1$ in equation (1). The state at which $e \leq e_s$ corresponds to Zone D in Fig. 3 and β equals 1 in equation (1).

When one apply the concept of liquid limit to equation (1) and Fig. 4, one can get interesting results as follows. If one define the liquid limit as the highest water content of plastic state, it corresponds to $\beta \leq 1$ in equation (1) and $e \geq e_s$ in Fig.4. If one define the liquid limit as the lowest water content of liquid state, it corresponds to $\beta \geq 0$ in equation (1) and $e \leq e_m$ in Fig. 4. Thus it can be inferred that the liquid limit can be interpreted as either the condition of $\beta=0$ and $e=e_m$ or the condition of $\beta=1$ and $e=e_s$ depending on the interpretation. Also, it can inferred that current way of definition of liquid limit is not a unique state, it can be interpreted differently; in other words, liquid limit is a range of water content at which $e_s \leq e \leq e_m$ and $0 \leq \beta \leq 1$.

Now let us apply the liquid limit obtained from current test method and the fluid limit of Monte Krizek (1976) and Song (1992) to equation (1) and Fig. 4. During the test procedure of liquid limit one can deduce that the sample is

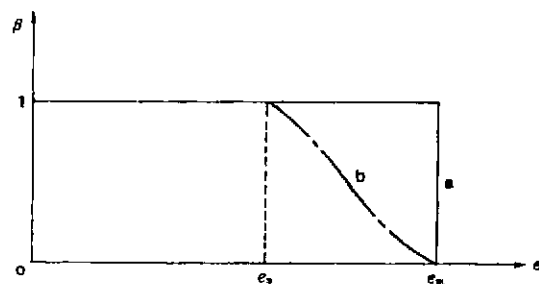


Fig. 4 Range of β (after Pane and Schiffman, 1985)

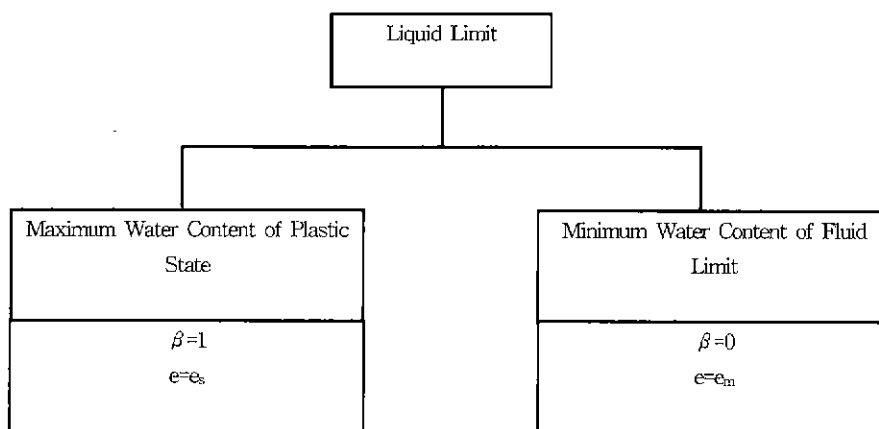


Fig. 5 Two way interpretations of liquid limit

within the effective stress condition. Then one can say that the liquid limit corresponds to $\beta=1$ in equation(1) and $e=e_s$ in Fig. 4. At the fluid limit, there is no effective stress, thus one can say that the fluid limit corresponds to $\beta=0$ in equation (1) and $e=e_m$ in Fig. 4. Thus, critically speaking, one can say that the soil condition after liquid limit is pseudo liquid, not truly liquid. The soil condition is truly liquid after the fluid limit not after the liquid limit.

If one reorganizes the above discussions, one can construct Fig. 5 for the two way interpretations of liquid limit.

3.2 Theoretical Approach

The concept of the pseudo liquid and the true liquid at liquid limit and at fluid limit was further investigated theoretically. Throughout the literature study, it was found that most of the research about the Atterberg limits was performed experimentally. Due to the very complex nature of clay minerals, theoretical approaches were very limited. In this study, theoretical approach using electrical double layer theory was tried and resulted in the good agreement with experimental results.

Due to the electrical double layer, clayey soil has some osmotic pressure naturally. The mechanism of osmotic pressure was well described in Mitchell (1993). Due to this osmotic pressure there is some negative excessive pore pressure in the clayey soil and it resulted in the effective stress in the soil. However, this osmotic pressure in the clayey soil can be changed by the concentration of the cations around the clay particle (Bolt, 1956). By Bolt (1956), the relationship between the cation concentration and the osmotic pore pressure is expressed as follows.

$$P_s = RTC_o (C_o/C_o + C_o/C_c - 2) \quad (2)$$

P_s = negative pore pressure = osmotic pressure

R = gas constant

T = absolute temperature

C_o = equilibrium solution concentration

C_c = mid-plane cation concentration

Also, the mid-plane concentration C_o can be correlated with void ratio e by equation (3) (Bolt, 1956).

$$\nu(\beta C_o)^{1/2} (x_o + (e/G_s A_s)) = 2(C_o/C_c)^{1/2} \times \int_0^{\pi/2} [1 - (C_o/C_c)^2 \sin^2 \phi]^{-1/2} d\phi \quad (3)$$

where,

ν = valence

$\beta = 10^{16}$ m/mole for normal temperature

C_o = equilibrium solution concentration

C_c = mid-plane cation concentration

$x_o = 4/\nu \beta \Gamma$ (Γ = surface density of charge)

e = void ratio

G_s = density of clay

A_s = specific surface area of clay

ϕ = angle for elliptical integration

Equation (2) is the relationship between osmotic pressure and cation concentration, and equation (3) is the relationship between void ratio and cation concentration. Thus the direct relationship between void ratio and osmotic pressure can be derived as follows. From equation (3), let $C_o/C_c = A$, and equation (3) becomes,

$$\begin{aligned} &\nu(\beta C_o)^{1/2} (x_o + e/G_s A_s) \\ &= 2A^{1/2} \times \int_0^{\pi/2} (1 - A^2 \sin^2 \phi)^{-1/2} d\phi \quad (0 < A < 1) \end{aligned} \quad (4)$$

Right sides of equation (4) are elliptic integration, and the solution is possible only by numerical approximation. By numerically solving the right term of equation (4), one can obtain equation (5).

$$\begin{aligned} &\nu(\beta C_o)^{1/2} (x_o + e/G_s A_s) \\ &= 2(A)^{1/2} \times (\pi/2) (1 + (1/2)^2 A^2 \\ &+ (1 \times 3/2 \times 4)^2 A^4 + (1 \times 3 \times 5/2 \times 4 \times 6)^2 A^6 + \dots) \end{aligned} \quad (5)$$

In equation (5) $0 < A < 1$, therefore when one disregards the

terms which has high order of A, we get equation (6).

$$\nu(\beta C_o)^{1/2}(x_o + (e/G_s A_s)) = A^{1/2} \times \pi \quad (6)$$

If one substitute equation (6) into equation (3), one obtains equation (7):

$$e = [(A^{1/2} \times \pi)/\nu(\beta C_o)^{1/2} - x_o]G_s A_s \quad (7)$$

Also, equation (2) becomes, equation (8)

$$P_s = RTC_o(1/A + A - 2) \quad (8)$$

If one lets $B = P_s/RTC_o$, the roots of equation (8) becomes,

$$\begin{aligned} A &= [(2 + B) \pm \{(2 + B)^2 - 4\}^{1/2}]/2 \\ &= [(2 + B) - \{(2 + B)^2 - 4\}^{1/2}]/2 \\ &\quad (\text{since } 0 < A < 1) \\ &= 2/[(2 + B) + \{(2 + B)^2 - 4\}^{1/2}] \end{aligned} \quad (9)$$

From equation (7) and (9), the coupled equation of osmotic pressure and void ratio is obtained. Also, from the experimental results, it can be assumed that the osmotic pressure at liquid limit is approximately 6 kPa. Thus the void ratio at liquid limit can be calculated by set $P_s = 6$ kPa in B of equation (9). The osmotic pressure at fluid limit can be assumed zero, because there is no inter-particle contact, thus the void ratio at fluid limit can be calculated by set $P_s = 0$ in equation (7) and (9). The water content at liquid limit was calculated by equation (10)

$$G_s = (e/w) \gamma_w \cdot S_r \quad (10)$$

For saturated soil, $S_r = 1$, thus equation (11) can be obtained.

$$W = e/G_s \cdot \gamma_w \quad (11)$$

Finally, the complete coupled equation of osmotic pressure, equilibrium solution concentration and water

content was obtained in equation (12).

$$w = [(A^{1/2} \times \pi)/\nu(\beta C_o)^{1/2} - x_o]A_s \gamma_w \quad (12)$$

where

$$A = 2/[(2 + B) + \{(2 + B)^2 - 4\}^{1/2}]$$

$$B = P_s/RTC_o$$

3.3 Comparison between Experimental Result and Theoretical Results

Comparison was made for kaolinite for $P_s = 6$ kPa (for liquid limit) and 0 kPa (for fluid limit) using equation (12) using material properties as follows: $T = 293$ K, $\nu = 1$, $\beta = 10^{16}$ m/mole, $x_o = 2/\nu \lambda = 2 \times 10^{-10}$ m, $A_s = 18$ m²/g, and $G_s = 2.7 \gamma_w$. Input parameters are shown in Table 1. The computed water content of kaolinite at liquid limit is shown in Table 1 and is approximately 77%. The variation of the liquid limit was not very sensitive to the equilibrium concentration of cations, especially when the equilibrium concentration is low. For the plain water, the equilibrium concentration of cation is very low but one can expect that the liquid limit will be approximately 77%.

However, the fluid limit from equation (12) was turned out to be extremely high (more than 1000 %), and it seems that the applicability of equation (12) is not good for very low osmotic pressure. The experimental results from Seed et al. (1964), showed that the liquid limit of kaolinite is approximately 50%.

The computed results is not exactly same as the experimental value, however they are approximately similar.

Table 1. Calculation results of liquid limit for Kaolinite at 6 kPa osmotic pressure

C_o (mole/m ³)	Liquid Limit(%)
0.0099999998	73.6925812
0.0010000000	77.5195999
0.0001000000	77.9323883
0 0000100000	77 9740067
0 0000010000	77 9781647
0.0000001000	77 9785843
0 0000000100	77 9786224
0.0000000010	77.9786224
0 0000000001	77 9786224

Truly Liquid State	Fluid Limit
Pseudo Liquid State	Liquid Limit
Plastic State	Plastic Limit
Semi Solid State	Shrinkage Limit
Solid State	

Fig. 6 New definition of Atterberg limits

This tendency was predicted from the fact, the equation (7) and (9) assumed the ideal conditions such as pure kaolinite, parallel orientation of clay particles, and disregard the van der Waals force and etc. as discussed by Lu (1990). However, it can be concluded that the agreement between the computed results and the experimental results are fairly good.

From these results, it can be inferred that the soils with the higher water content than the liquid limit will have smaller values of osmotic pressure, thus have smaller value of effective stress but not zero effective stress. These results coincide with the concepts of equation (1), Fig. 3, and Fig.4. The previously mentioned concepts that the soil after the liquid limit is pseudo liquid and the soil after the fluid limit is truly liquid is valid. This concept can be reorganized as Fig. 6.

4. Conclusion

From the experimental and theoretical consideration of the liquid limit, the following conclusions could be made.

- 1) Soils of higher water content than the liquid limit have effective stress and shear strength, and are not truly liquid. This statement could be verified theoretically and experimentally.
- 2) Soils of higher water content than the liquid limit can be regarded as pseudo liquid state, while soils of higher water content than the fluid limit can be regarded as true liquid state.
- 3) Bolt (1956)'s electrical double layer theory showed good agreement with the experimental data at the water constant of liquid limit level. But the agreement

was poor at the water content of fluid limit level.

- 4) For the very soft soils such as mine tailing dams, ash ponds of fossil power plants, the understanding of liquid limit and fluid limit is helpful.

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