

The Rheological Significance of Consistency Limits of Soil

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Introduction

About 1911, the Swedish agriculturist Atterberg divided the entire cohesive range from the solid to the liquid state into five stages and set arbitrary limits for these divisions in terms of water content. The actual measurements of both the liquid and plastic limit are performed by standardized laboratory tests. The liquid limit is that water content at which a groove cut in a sample of soil in a standard liquid limit device is closed after 25 taps. The plastic limit is that water content at which a 1/8 in. diameter thread of soil begins to crack and crumble under continued rolling by hand. However, there are still problems how to explain the relation between two different tests, and why we can determine both upper and lower limits of plastic state by means of two quite unlike devices and procedures.

Consistency, in general, is that property of material which is manifested by its resistance to deformation and flow of soil, and therefore an indication of its rheological behavior¹⁾. The Atterberg consistency limits indicate the soil moisture content limits for various states of consistency. With reference to the Atterberg limits the author gives a rheological interpretation and then the actual standardized laboratory procedures are discussed on the basis of the rheological behavior of soil.

Soil as viscoelastic body

In the stress-strain diagram of a compression test of soil, the steepness of the curve will generally vary directly with the rate of deformation. Because the rate of deformation depends on the time scale of the compression test, shortening this time scale is equivalent to increasing the rate of deformation; the diagram will display a steeper curve. The ratio of viscosity to elasticity also has a time dimension. With a longer time scale, the behavior of the soil will be more viscous, while with a shorter time scale, the behavior will be more elastic. Thus a comprehensive understanding of soil behavior will require us to regard soil as a viscoelastic body. When we analyze the behavior of soil conceptualized as a set of N Voigt viscoelastic elements connected in series, each with a different retardation time, we usually obtain a bell-shaped retardation spectrum²⁾³⁾⁴⁾ as shown³⁾ Fig. 1. Another

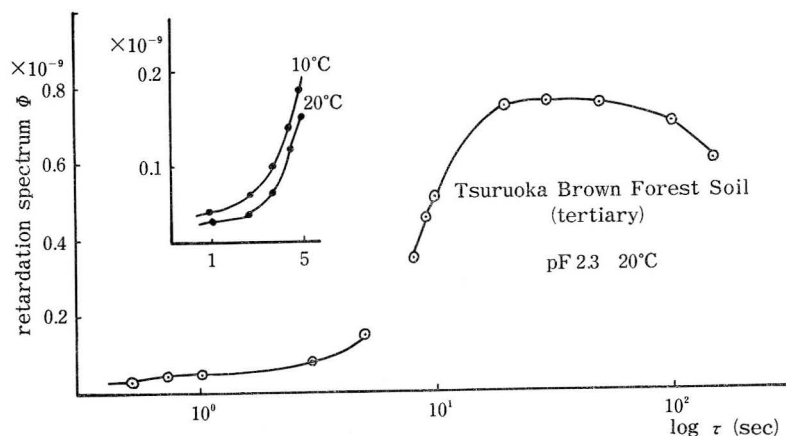


Fig. 1. Retardation spectrum of Tsuruoka Brown Forest Soil in tertiary mountains.

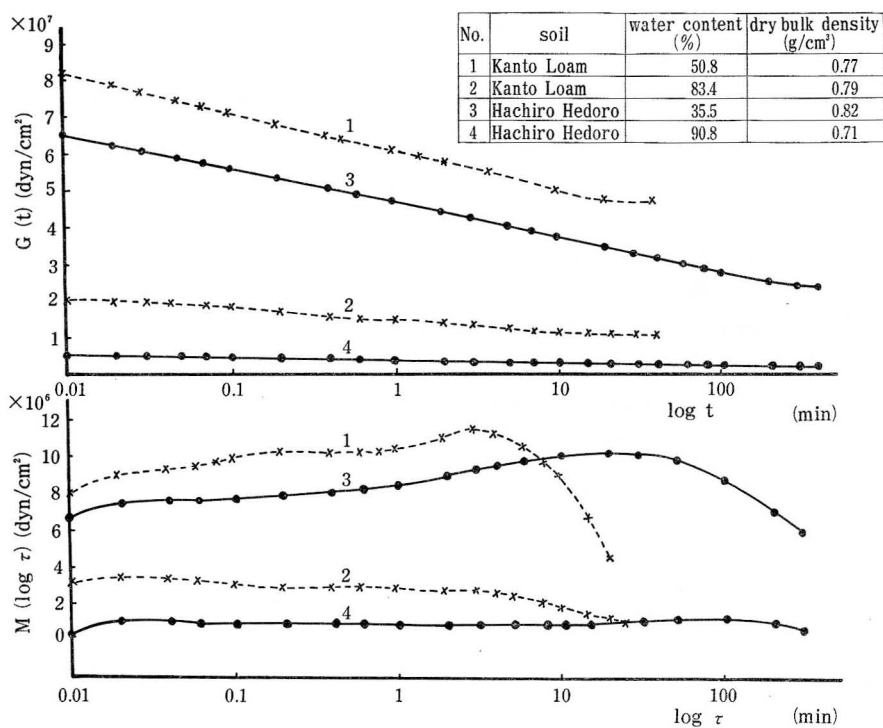


Fig. 2. Relaxation modulus $G(t)$ and relaxation spectrum $M(\log \tau)$.

useful model is the generalized Maxwell model for an infinite set of parallel Maxwell elements. Fig. 2 shows relaxation modulus and relaxation spectra for two soils at two water contents. One is Kanto Loam, volcanic ash soil (allophane) and the other is Hachiro Hedoro (sodium montmorillonite), which is the marine soil type

Hedoro in shallow coastal bays. Dynamic properties⁵⁾ of Tsuruoka Brown Forest Soil in tertiary mountains are shown in Fig. 3, where E' is dynamic Young's modulus, that is the real part of the complex elastic modulus E and η' is dynamic viscosity. Thus

$$E = E' + iE'' \quad \dots\dots\dots(1)$$

$$\eta = \eta' - i\eta'' = \eta' - iE''/\omega \quad \dots\dots\dots(2)$$

$$\tan \delta = E''/E' = \alpha_T/\pi = \Delta\nu/\nu_T = \omega\eta'/E' = 1/Q \quad \dots\dots\dots(3)$$

where $\tan \delta$ = loss tangent

E'' = imaginary part of a complex quantity

ν_T = resonance frequency

$\Delta\nu$ = band width

ω = circular frequency

α_T = logarithmic decrement

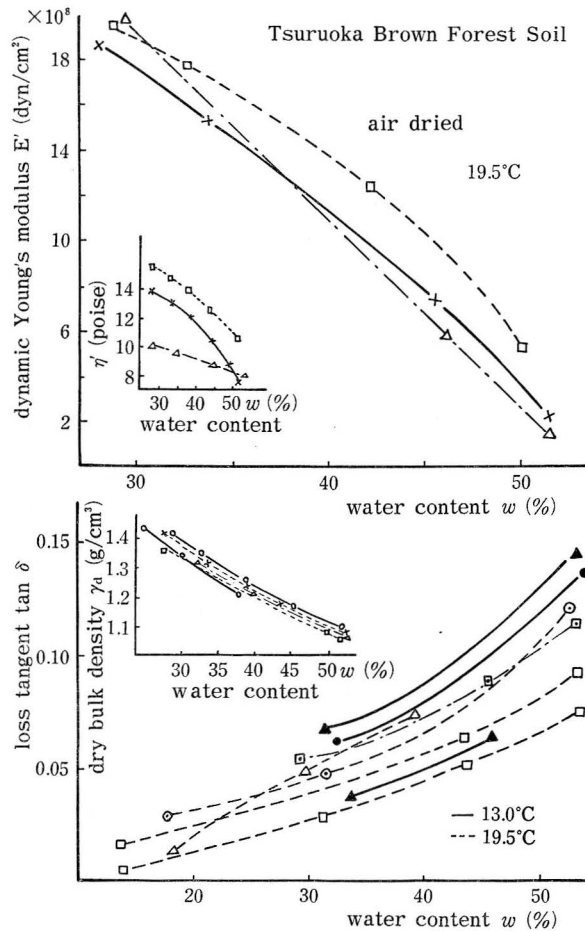


Fig. 3. Dynamic viscoelastic behavior.

As shown in Fig. 3 both the elastic modulus and the viscosity are the function of water content within the certain range.

Mechanical transition points

Soil has two different types of elasticity. One is energy elasticity, caused by crystalline material such as soil particles ; on account of its structure, soil always displays a pseudo-high elasticity. The other type of elasticity is entropy elasticity, which is caused by the thermal action of counter ions surrounding soil particles ; this is the same as the elasticity of rubber or gas. In high water content soil, entropy elasticity predominates, while in low water content soil above pF 3, energy elasticity predominates. The modulus of elasticity of materials such as metals and plastics as well as soil will increase steeply at certain temperatures we call transition points. But in the case of soil, elasticity will also increase steeply at certain water content levels, for which I would suggest the term, mechanical transition point⁶⁾. These mechanical transition points correspond closely to Atterberg limits, which describe the consistency of soil. Table 1 shows the correspondence of these two and their relationship to the pF scale. And the index of free energy pF is one of the state valuables in soil water system.

Table 1. Mechanical transition points and rheological models

water content (pF value)	<div><div>-2</div><div>-1</div><div>0</div><div>1</div><div>2</div><div>3</div><div>4</div><div>5</div><div>6</div><div>7</div></div>										
Atterberg limit (mechanical transition point)	<div><div>BL</div><div>LL</div><div>PL</div><div>SL</div><div>↑ air dried</div><div>↑ oven dried</div></div>										
viscoelastic behavior	sol Newtonian model		gel Voigt model		semi-solid Burgers model			solid state Hookean model			
plastic behavior			Bingham model		St. Venant model <div><div>ductile failure</div><div>brittle failure</div></div>						

If we consider the mechanical transition points separately and in order from the lowest pF value, the first point is at the Bingham limit⁷⁾ (pF -1.5). This point is the water content of sediment which, under fixed conditions, settles out from suspended soil particles. In this state, the soil is hydrophyric gel, and has an entropy elasticity. Because the soil structure in this state is weak, even a small force such as remolding will cause it to yield and result in a Bingham flow. With structural yielding, the water which had been bound both in the soil structure and to the surface of soil particles is released and becomes free water. The result is the phenomenon of soil softening. The flow curve of a shearing force - shearing rate diagram assumes two different shapes in the return stage of decreasing shearing force. The first shape indicates a shear rate thixotropic flow in clay with a thin hydration layer, such as the hydrogen clay typical of tertiary mountains. The second

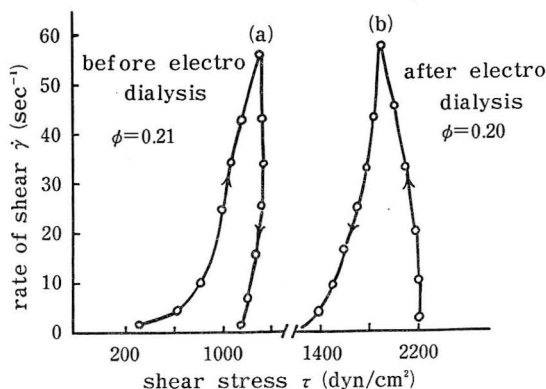


Fig. 4. Flow curves, (a) negative shear rate thixotropic flow, (b) shear rate thixotropic flow (Ariake Hedro, marine soil).

describes the negative shear rate thixotropic flow in clay with a thick hydration layer such as sodium clay and allophane⁹⁾. Fig. 4 illustrates an example⁹⁾. The negative shear rate thixotropic flow in natural marine clay, Ariake Hedoro (sodium montmorillonite), is shown as curve (a). After treating with same soil through electro dialysis it is turned into shear rate thixotropic flow, curve (b). Furthermore, yielding value increases seven times as much as before one with same concentration ϕ . In soil paste Yasutomi and Sudo⁸⁾ found that yielding value θ could be written as

$$\theta = A \exp (\phi - \phi_0) \quad \dots\dots\dots (4)$$

where A = constant

ϕ = volumetric concentration

ϕ_0 = volumetric concentration at the Bingham limit point (BL in Table 1)

Although a kinetic unit, or a mechanical structural unit is usually considered as an aggregation of soil particles, I will assume here an idealized model in which each mechanical unit consists of a single particle. This is known as a simple dispersion system. If we further assume that the thickness of the particle surface hydrate layer is equal to one-half the interparticle distance, d Å, the water content, w , per cent in such a system can be written¹⁰⁾¹¹⁾ as

$$w = W_w / W_s = Sd / 100 \quad \dots\dots\dots (5)$$

where W_w = weight of water

W_s = weight of soil particles

S = specific surface area (m²/g)

Liquid limit

The second mechanical transition point corresponds closely to the liquid limit (LL). In a liquid limit test, flow below pF 1.5 (that is, high water content) looks very similar to flow at the Bingham limit point. This is because in both cases yielding of gel releases previously bound water as free water. Many things are

known about the liquid limit. We know, for example, that the liquid limit of monmorillonite is ten to twenty times larger than that of kaolinite and that the liquid limit of clay with monovalent exchangeable cations is larger than that of clay with divalent cations. We also know the effects of salt concentration and of temperature on the liquid limit. Because all these depend on the soil surface S and the interparticle distance d , we can easily derive qualitative reasons for them from equation (5).

1) The significance of the 25-blow criterion

If we consider the liquid limit to be literally the boundary between the liquid state and the semi-solid state, we should use the Bingham limit because it is at this point that a yielding value first appears. However, soil structure at the Bingham limit is very weak and with every application of the smallest force, liquefaction occurs. If the liquid limit were thus defined, it would not be a useful index for practical application. Rather, it is more useful and appropriate to term the next mechanical transition point the liquid limit.

When results from a liquid limit test are graphed with a standard, arithmetic scale for blow numbers rather than a logarithmic scale, we see a clear difference between two groups of results; below 10 blows and above 40 blows. The type of flow in tests below 10 blows looks very similar to the type of flow observed at the Bingham limit point. This is because in both cases yielding of the gel releases previously bound water as free water. That is, too much remolding causes soil softening which results in requiring fewer blows to reach the test conclusion (i. e., closing of the middle groove for a distance of 1.5 cm).

On the other hand, in the case of samples requiring more than 40 blows, remolding has broken up soil aggregates giving rise to additional soil surfaces to which free water then attaches. The result of this is the phenomenon of soil hardening, and the soil will require more blows than average to reach the test conclusion.

The criterion of 25 blows as the liquid limit standard lies mid-way between these two groups, and its real significance is that it cancels out individual experimenter differences in remolding. That is, excess remolding at the low end will increase soil softening while excess remolding at the high end will increase soil hardening. Plotted on a logarithmic graph, such results will produce a flatter line of less slope, but the center pivot, the w value at 25 blows, will remain the same. Despite variations in remolding, the liquid limit indicator, 25 blows, remains constant.

2) The meaning of the flow curve in the liquid limit test

We often see the common example of repeated blows to the bottom of a container filled with flour producing closer packing of the flour. In the liquid limit test, free water released from both inside and the surface of the mechanical structural units

causes a change in the curvatures of the micro-meniscus between units; these changing curvatures result in a Bingham flow as the units pack more closely together.

In the liquid limit test, a cup 180 g in weight is dropped 1 cm. In order to observe more effectively the packing phenomenon, experiments were conducted using larger dimensions: a mold 1400 grams in weight was dropped 5 cm. Fig. 5 describes the relation between the bulk dry density γ_d and the number of blows, N ; the former is the index of packing.

Then, the same soil was used in a mold compression test in which samples in a mold were compressed by means of an oil jack. Fig. 6 shows the relation between bulk dry density γ_d and compression force per unit area p .

Now, if we compare Fig. 5 and Fig. 6, we can see that increasing the number of blows corresponds to increasing the compression force and that in terms of mechanical effect on soil, the number of blows N is equivalent to the compression

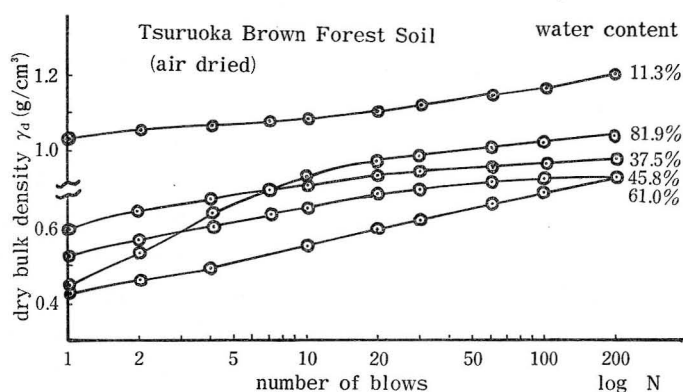


Fig. 5. Relation between packing and tapping.

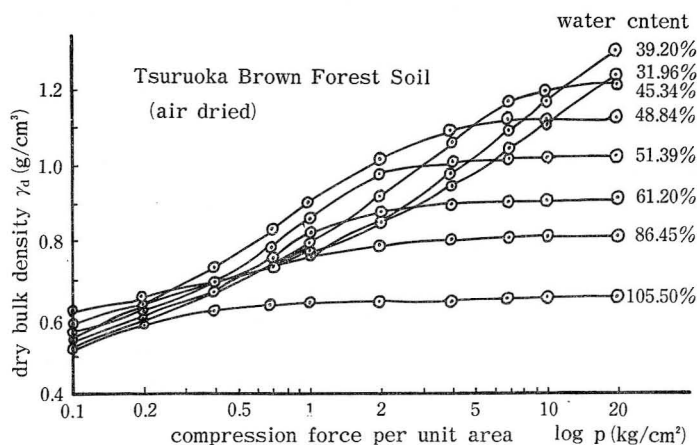


Fig. 6. Relation between packing and one dimensional compression force.

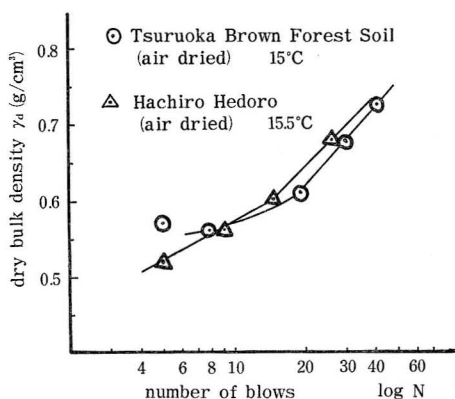


Fig. 7. Relation between packing and tapping (liquid limit test).

force p . The bulk dry density γ_d is a function of the void ratio e , and it is also a function of water content w assuming 100% saturation (see Eq.(7)). This leads us to the conclusion that the flow curve w -log N in the liquid limit test is similar to the e -log p curve in the consolidation test.

It is this correspondence between the flow curve and the e -log p curve which explains the observation of Skempton of the relation between the compression index C_c and the liquid limit

w_L . The flow curve γ_d -log N in place of w -log N in the liquid limit test is shown in Fig. 7. In this experiment one should pay special attention to take a small amount of sample from the groove closed. Because only the portion of the groove closed would reach to the state of closest packing and also 100 per cent saturation.

3) The meaning of the 1.5 cm closure criterion of the trapezoidal shaped groove

Let us consider the meaning of the 1.5 cm closure of the groove in the liquid limit test. If at the start of the test we take mini scale samples from both sides of the groove and measure for bulk dry density, water content, and degree of saturation, the values vary widely and randomly. However, as the groove closure approaches 1.5 cm with successive blows, we can easily understand that there is a levelling of these values as the bottom layer of soil becomes packed uniformly. The test is ingeniously designed so that the 1.5 cm closure criterion requires a substantial number of blows to reach; that is, it requires more than just localized packing. In the same sense, because the cone in the fall cone test is large, it induces yielding over a wide area of the sample soil in reaching penetration equilibrium.

4) The principle behind the fall cone test

The above explanation also serves to shed light on the question of why the fall cone test, a very different test from the standard liquid limit test, also yields the liquid limit. In the standard test, an increase in number of blows to the cup represents an increase in the external force exerted on the soil, while in the fall cone test, because the weight of the cone is fixed, external force on the soil decreases as the cone enters the soil (that is, as the surface area of the penetrated section increases). The cone penetrates until the stress produced by the penetrated section is in equilibrium with the yielding value of the soil¹²⁾. Thus, the depth of cone penetration becomes a standard for external force, like the number of blows in the liquid limit test.

Plastic limit

The third mechanical transition point is at the plastic limit. The plastic qualities of soil in its clay state have long made it a valuable material for figurines, statuary, masks, etc. There are two conditions that such a material must satisfy. One is that its yielding value must be small enough that it can be shaped by a small force such as the press of a finger and then remain permanently set. A second important condition regards the type of failure after yielding. We know there are two types of failure: brittle failure (or brittle fracture) and ductile failure. The former is when yielding causes sudden surface cracking and fracturing, while the latter is when yielding leads to a gradual deformation of shape. It is this latter property of ductile failure which allows us to shape an image from clay without it fissuring under finger pressure.

The present plastic limit test is without real theoretical underpinnings and is rather based more on experience and customary practice. I would suggest that it is important to conduct the plastic limit test as a test of material failure. This involves rolling a sample back and forth on a flat, frosted glass under constant palm pressure; this creates a repetition of alternate tensile and compressive stress on the material. This is repeated with samples of the same soil at increasing water content levels to determine the minimum water content at which ductile failure occurs. This would represent the plastic limit; soil with water content above this plastic limit point would always display ductile failure under the palm press and rolling test while brittle fracture would occur in soil with water content below this point.

In this sense, the third mechanical transition point, which separates ductile failure from brittle fracture under the palm press and rolling test, corresponds closely in numerical value and is linked theoretically to the plastic limit (PL). This transition point is generally just above pF 3 although it may vary up to about pF 4 in volcanic ash soil, organic soil, and soil with high concentrated cations.

1) The rationale for the 3 mm diameter criterion

The rough surface of the frosted glass used in the plastic limit test both hastens the packing of the soil and allows excess free water to drain off which might otherwise obstruct the packing of the inter-structural unit pore spaces. In determining a criterion for the test diameter of the rolled, thread-like soil sample, as small a diameter as possible would be desirable to minimize any differences of water content, temperature, and packing between the surface and the center of the soil sample thread; perhaps as small as 1-2 mm. Such a small diameter would also contribute to shortening the time of consolidation.

However, unfortunately, the smaller the test diameter, the greater the number of times the thread is rolled during one pass of the hand, and thus the rate of

Table 2. Results of plastic limit test with various rolling speeds

speed rolled by hand	number of times rolled per minute	rate of strain percent/min	water content (%) (plastic limit)
very low	45	1.3	60.39
standard	80	2.2	54.12
extremely high	150	5.1	46.07

samples : Hachiro Hedro (air dried soils), temperature : 13°C controlled

deformation will increase. When this happens, the type of failure will shift to ductile failure while brittle failure would occur at slower rates of deformation. Under fast rates of deformation, the rolling of the soil thread will continue without breaking to a much lower water content level¹³⁾ (see Table 2). This will be especially true in samples where hardening develops. These factors then recommend a rather thick diameter, for example, 5–6 mm.

With these considerations in mind, 3 mm is an appropriate test diameter because it is roughly mid-way between these diameters. In the above discussion we use the 3 mm diameter according to the Japanese criterion, which is 1/8 in. approximately.

2) The mechanism of brittle fracturing

In the plastic limit test, rolling the soil sample thread causes a continual repetition of stress that is alternately tensile and compressive in nature and which forces out free water that drains away on the frosted glass. It is easy to imagine that as the thread gets narrower, soil packing will be promoted. Bulk dry density will gradually increase as the number of times rolled increases, reaching a maximum

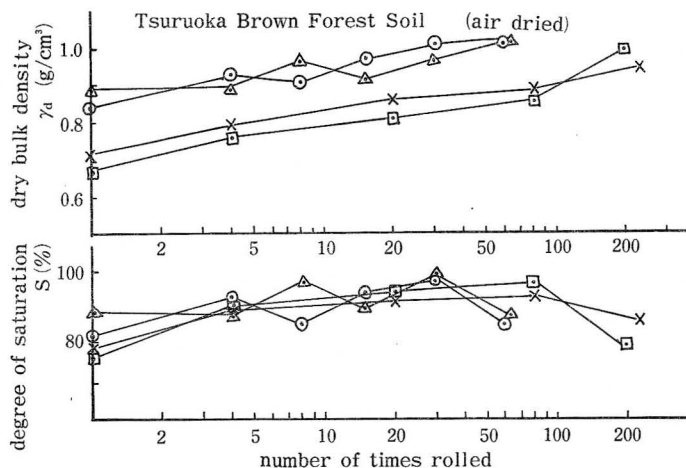


Fig. 8. Relation between packing and rolling (plastic limit test).

value as shown in Fig. 8. At the same time, the degree of saturation will approach 100%, and then, as Fig. 8 indicates, will decrease after surface cracking and fissuring.

The mechanism of brittle fracturing that occurs at the maximum bulk dry density value can be explained as a dilatancy effect. That is, when the structural unit at maximum bulk dry density is subjected to external force, its volume expands; we are all familiar with the sudden absorption of water by beach sand that has just been walked on. This same dilatancy acts in the plastic limit test and explains the brittle fracturing of the soil sample at the plastic limit point.

3) The relation of the plastic limit to the optimum moisture content value

It has been observed by several researchers that the value of the plastic limit is near to that of the optimum moisture content in the compaction test¹⁴⁾. As I suggested above, in the plastic limit test, soil reaches the state of maximum closed packing just before surface cracking. Thus, it is natural that the value of the plastic limit approximates the optimum moisture content, provided the external force per unit area is equal. This is in fact the case.

In the compaction test, a 25 kg-weight drop hammer is dropped from a height of 30 cm and impacts the soil surface while in the plastic limit test, a sample is rolled on a flat glass under palm pressure. It may seem difficult to believe that the stress is in both cases the same, but in fact, as Fig. 8 shows, the bulk dry density value is nearly equal in both tests. Whichever way the force is applied, the rheological result is the same.

This result may appear all the more surprising when we consider that there are individual experimenter differences in the application of palm pressure in the plastic limit test; compared to the fixed weight of the drop hammer. When we investigated these differences by actually measuring palm pressure during rolling, we obtained values in the range of 200–250 grams. Dividing by the surface area of the rolled samples just after initial surface cracking, we obtained a minimum value of 1.1 kg/cm², a maximum value of 8.3 kg/cm², and a majority of the values in the 2–5 kg/cm² range.

This wide range of 2–5 kg/cm² would appear to upset the claim of equal force in the two tests, but Fig. 6 provides us with an explanation. We see in this figure that bulk dry density at first increases with increasing force but when p reaches the 2–5 kg/cm² range, it rises very little at all, and the curve flattens out. This is because after the soil reaches 100% saturation, the external force will be almost entirely borne by pore water pressure and additional force will not cause further compaction (that is, will not increase bulk density). Thus, individual experimenter differences in palm pressure do not result in significant variation in compaction.

4) The plasticity chart

The plasticity chart is employed in soil classification for engineering. As we

know, on this chart the longitudinal axis is the plasticity index, and the latitudinal axis is the liquid limit. Research in Japan, for example, has demonstrated that soil samples from a soil layer that lies at varying depths from the surface will lie along a line parallel to Casagrande's A-line and that samples of a soil that is progressively air-dried will also descend along a line parallel to the Casagrande's A-line in proportion to the length of air-drying, with fresh soil having the highest P. I. value¹⁵⁾.

Because the plasticity index is the difference of the liquid limit and the plastic limit, we may deduce that this index is the amount of water which contributes to plastic behavior per one gram of soil. Casagrande's A-line has been determined experimentally, and I would suggest that it has several theoretical meanings. First, it may be understood to show that the plasticity index increases in proportion to increases in the liquid limit; this is natural in light of the above discussion of the plasticity index. Second, let us consider the meaning of the case of w_L-20 in the P. I. equation.

Although equation (5) is based on an idealized model in which each mechanical structural unit consists of a single particle, in fact of course, a structural unit is an aggregates of soil particles; between which is water. Thus, the water content value determined to be the liquid limit will include this structural dead volume, but this dead volume of water does not contribute to the flow behavior of soil.

Soil typically has a shrinkage limit of about 20, and I would suggest that the dead volume corresponds closely to this shrinkage limit. If this is true, then w_L-20 represents the effective quantity of water in the soil.

This value 20 is not reflected in the plasticity index because in subtracting the plastic limit value from the liquid limit value, the dead volume in each is cancelled out.

In Japan, a D-line¹⁶⁾, which employs w_L-50 , has been experimentally proposed for volcanic ash soil. It is not surprising then to realize that the shrinkage limit of volcanic ash soil in Japan is approximately 50.

Shrinkage limit

Let us turn for a moment to the shrinkage limit. If the plastic limit test were conducted using the hand of a giant with enormous force on the soil, the point of transition from a saturated state to an unsaturated state would shift to a lower water content level. If this force were so great that it equalled the shrinkage stress of evaporation of soil water, this transition point would approximate the shrinkage level.

The shrinkage stress brought on by the evaporation of soil water causes a breaking up of part of the soil particle aggregations or of the orientation of the

particles* ; it results in a larger bulk dry density and in closer packing than at the plastic limit.

Here, too, the rate of deformation is very important. Ununiform stress caused by fast evaporation or sudden changes of temperature will result in surface cracking or local brittle fracturing. In the shrinkage limit test, we usually pay much attention to eliminating bubbles in the sample and to remolding it thoroughly. I would suggest that this is done in order to obtain a normal shrinkage pattern and to encourage ductile deformation without brittle fracturing. In other words, as the rate of deformation depends on the time scale, the deformation with a shorter time scale than the retardation time of strain results failure in dilatancy, that is brittle fracturing. And that leads same conclusion suggested by Takenaka¹⁷⁾ that shrinkage should be understood as the behavior of rate process.

Properties as a continuous body

1) index of packing

In a constrained compression test on unsaturated soil in a mold, bulk dry density γ_d which is the index of packing in the mechanical structural unit, can be written experimentally as

$$\gamma_d = a + b \log P \quad \dots\dots\dots(6)$$

where $a, b = \text{constants}$

$P = \text{compression force per unit area}$

Equation (6) can be applicable carefully within a range between two transition points. For example, when increasing of compression force P results the reaching to the fully saturation, since at which the transition from one state to another one takes place, therefore, we can not employ same constants a, b through all the case.

For a fully saturated soil-water system, bulk dry density γ_d divided by G_s , the specific gravity of solids, is described by definition as follows.

$$\frac{\gamma_d}{G_s} = \frac{1}{1 + wG_s} = \frac{1}{1 + e} = \frac{V_s}{V} = \phi \quad \dots\dots\dots(7)$$

where $w = \text{water content}$

$e = \text{void ratio}$

$V_s = \text{volume of solids}$

$V = \text{total volume}$

$\phi = \text{volumetric concentration}$

Furthermore, because $\gamma_d/G_s = \phi$, we can see that equation (6) may be reduced to

* In the above discussion on the plastic limit, I have not touched on the effect of particle orientation although, of course, where sample soil material consists of plate-like particles, this effect is considerable. Soil physics texts in fact usually explain the plastic limit in terms of particle orientation. However, even soils whose particles are not plate-like, such as the Japanese type Kanto Loam, have a plastic limit, so this explanation is not entirely satisfactory.

equation (4).

2) Swelling and slaking

When a specimen of soil is immersed in water, the phenomenon by which it collapses is known as slaking¹⁸⁾. The explanation for slaking has long been based on the supposed action of the air pressure within the soil clod in breaking it up. However, because our experiments indicate that air pressure within the clod is actually seldom high enough to cause clod collapse, we would propose an alternative, which accounts for slaking in terms of the infiltration of the water in which it is immersed and subsequent swelling¹⁹⁾²⁰⁾²¹⁾ of the soil.

When the soil clod is immersed in water, water first infiltrates the macro-pores between structural units; after a while, it infiltrates into the insides of the units. It has been shown that during initial infiltration, the time required is a function of the square of the distance of infiltration²²⁾.

The infiltration will cause a swelling of the soil, and it is the swelling pressure that seems to bring about the slaking of the soil. In measuring this swelling pressure we usually observe three stages of increasing pressure²³⁾. The first two stages correspond to water infiltration, but in the third and last stage, swelling will increase for a long period, often more than several months. This third stage must be related to the hydration of water attached to the surface of the soil particles, which requires a long time to complete. However, much less time is required for the first two stages, and remolding can promote the swelling.

Slaking and swelling are of course fundamental to the transition point tests because we begin by mixing water with a dry powder soil²⁴⁾. One reason why the remolding time and manner of applying the water can affect test results is because often the test is started before the second stage of swelling is completed. Also we never initially pour water into the dry powder soil but rather spray it carefully. This is because a large volume of water poured on would cause swelling and slaking near the surface; that is, the increased volume from the swelling and the collapsed units from slaking would tend to stop up and interrupt further infiltration. The result would be local disequilibrium of water which causes hardening during remolding and eventually affects test results.

3) Mechanical structural units

If the liquid limit test is to be done quickly and there is not enough time for the second stage of swelling to be completed, test results will be greatly affected by the mixing and remolding time. Fig.9 describes the experimentally determined relationship of the unit diameters and remolding and slaking²⁵⁾; we can see that structural unit diameters are much smaller with increased remolding and slaking. The average diameter of 80% of mechanical structural units of soil that has been left 10 minutes after spraying is 200μ or less (determined by sedimentation analysis), while the diameter of 80% of units at the liquid limit point is 50μ or less (range

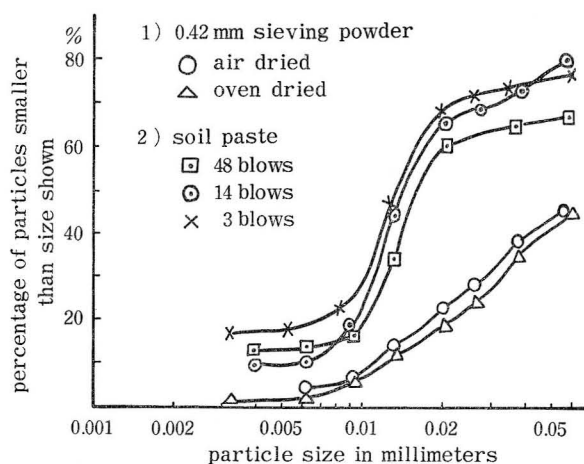


Fig. 9. Particle size distribution curves of Hachiro Hedoro.

is $5\text{--}50\mu$).

Therefore, when we explain the theoretical meaning of Atterberg's consistency limits, we must consider not only the microscopic interparticle surfaces and their interactions in the soil-water system but also macroscopic behavior of mechanical structural units.

Discussion

1) The mechanism of the liquid limit and plastic limit

It is important to realize that the mechanisms of the liquid limit test and the plastic limit test are similar. In the liquid limit test, because the soil is almost saturate, the micro-meniscus between the mechanical structural units is necessary for the groove to maintain its shape. The force of the impact when the cup drops causes yielding which in turn induces flow; what happens is that some of the micro-menisci disappear while the curvature of others is altered as free water is released from its bounded state. The result in terms of distance between units is a packing of the units. In terms of the seepage of water from inside to outside of the units under the external impaction force, the result is the phenomenon of soil consolidation.

The change of state from saturated to unsaturated will tend to increase the yielding value, and the soil-water system will begin to display properties of a coarse particle system at this point, although the soil-water system must still be considered as a continuous body.

The above description also fits the mechanism at the plastic limit; provided we omit the word "micro" and replace "impaction" with "repetition"¹¹⁾.

2) The rheological meaning of Atterberg limits

The fourth mechanical transition point is very near to the shrinkage limit (SL).

The shrinkage limit is the water content of a soil sample at the point where shrinkage ceases. We can however consider the process of normal shrinkage of a sample during the shrinkage limit test as a type of ductile deformation caused by the shrinkage stress brought on by the evaporation of soil water.

Furthermore, given the conditions of the shrinkage limit test (i. e., the gradual drying of a saturated soil sample), the soil sample even at the shrinkage limit should be properly considered as a single continuous body. Table 1 has been constructed on the basis of the following theoretical proposition: that the state variable⁽²⁶⁾⁽²⁷⁾⁽²⁸⁾ of a water-soil particle system in a state of equilibrium and in the absence of external mechanical forces can be defined by the free energy index pF ; in other words, it is equivalent to the state variable of soil water. Yielding values vary directly with the state of packing. Therefore, yielding values correspond to the state variables of soil-water system. In other words, the rheological meaning of Atterberg limits is measuring such yielding values affected by state variable instead of measuring rheological constants as elasticity and viscosity at the mechanical transition points.

3) The future of rheological analysis

The above discussion has been limited to certain rheological properties and behavior of soil as summarized in Table 1. There are many problem areas in which rheological analysis is relevant and promising⁽²⁹⁾; for example, in reclamation of shallow coastal bays with the marine soil type Hedoro, in earthwork and highway construction in area of volcanic ash soil like Kanto Loam, and in landslide prevention in Tertiary mountain zones. Unfortunately, it is at present only applied to a few special problems such as soil consolidation, but the future of rheological analysis is promising. Recent textbooks on soil mechanics have begun to include chapters on rheology, and rheological analysis itself has been greatly aided by the rapidly expanding use of computers, which facilitate handling of large deformation problems and non-linear problems.

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土のコンシステンシー限界のレオロジー的意義

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摘 要

コンシステンシーという語は、一般にやわらかい物体が、外力をうけたときの変形または流動に対して抵抗する性質を評価するもので、材料のレオロジー的性質を示すものである。土のコンシステンシー測定については、力学的な試験法のかなりの部分をこれに含めて考えることもできるが、普通はコンシステンシー状態を区分するアッターベルグ限界が中心となろう。ここでは特にその中で、塑性領域の上下限を決める液性限界と塑性限界に焦点をしばり、一見、全然似ていないこの二つの試験法が、何故に上下限を決めうるか、試験法の意義や、力学的内容について、レオロジー的な考察と検討を加えた。

はじめに、粘弾性体としての土の理解が必要であることにふれ、力学的スペクトル、弾性の機構、シキソトロピー流動などに特徴づけられる土のレオロジー的挙動を把握するためには、状態量としての pF スケールで表-1のように整理されることを示した。そこで用いられた力学的転移点 (mechanical transition point) は、弾性率、粘性率などのレオロジー一定数の測定によって決められるべきものである。しかし実用的な液性、塑性の両限界試験法のレオロジー的意味は、実は充填状態の変化、すなわち状態量 (state variable) の変化が降伏値に影響することを巧みに利用したものであることを明らかにした。したがってこれらの試験でえられるコンシステンシー限界は実用的には力学的転移点に近いものとして考えられることを示した。

このような観点から公定法の意味を整理すると、一見、似ていない25回皿たたき法とフオールコーン法のいずれでも液性限界が測れること。液性限界によって圧密試験の圧縮指数が推定できること。塑性限界によって締固め試験の最適含水比が推定できることなどの、これまでの数々の疑問に対し、統一的解釈のもとにすべて説明することができる。そしてこれまた、レオロジー的挙動を土の状態量の反映としてとらえ、整理していく方向から導かれた成果でもある。