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ON THE STATE SURFACE OF SOILS*

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I. INTRODUCTION

Many engineering problems in respect to the stability of soil structures have been practically treated with the assumption that soils were rigid-plastic materials. This simple assumption and the subsequent simple analysis were very convenient to design soil structures upon or under complicated subgrade. This analysis is based on the following conditions;

(a) the equilibrium condition of mass
(b) the failure condition of a soil element (ordinarily Mohr-Coulomb failure condition is used)

and places no restrictions on the deformation of mass. Therefore it is the inherent defect that this analysis is useless for the practical problems with respect to the deformation of soil structures. For such problems the analyses based upon stress-strain relation of soils are necessary. One of them is elasticity. Elasticity is used very often to solve the stress or strain in the soil mass under loading, but it is not so reliable because of its assumptions that are not appropriate to mechanical properties of soils under shear. A new analysis based upon the stress-strain relation adequate to soils is necessary.

It might be more easy to establish a theoretical system of mechanical behaviors of soils including stress-strain relation to detect entire shearing process than to concentrate our attentions on the failure point whose definition is physically quite vague. Strength properties are the limited part of mechanical properties of clays under shear. This Paper is one of a series on mechanical behaviors of clays on the process to failure by Hata and Ohta[1),2)] (1968, 1969).

One of the most reasonable approaches to stress-strain relation of clays is to consider the clays as the work-hardening materials. Incremental stress-strain relation for work-hardening (stable) materials with convex yield surfaces are explained by Drucker[3)] (1959). But the concept of yield surface and normality are not necessarily applied to clays. Because it is still unknown whether the assumptions on which the theory is based are satisfied by clays. Many difficulties arising from the path dependent behaviors should be overcome before practically useful stress-strain relations of clays are found out. Some people tried to propose the reasonable yield surface of clays. Drucker, Gibson and Henkel[4)] (1957) proposed cone shaped yield surface with spherical cap for normally and over consolidated clays. The proposed yield surface itself could not throughly explain the plastic deformation of clays, but had eminent influence upon the following researches for stress-strain relation of soils.

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A series of Papers by Roscoe, Schofield and Wroth[5] (1958), Roscoe and Poorooshasb[6] (1963), Roscoe, Schofield and Thurairajah[7] (1963) proposed the general theory of the mechanical state of soils. Roscoe and Poorooshasb[6],[8] (1963) suggested implicitly the existence of reasonable yield surface of normally consolidated clays. Roscoe, Schofield and Thurairajah[7] (1963) proposed the logical yield surface of clays wetter than critical state. Throughout the series of Papers by Roscoe et al., the concept of critical state plays most important role. Palmer, Maier and Drucker[9] (1967) and Palmer[10] (1967) had advanced fundamental theory with respect to stress-strain relation of clays. Barden and Khayatt[11] (1966) showed the interesting yield surface of sands.

The subject of this Paper is to show the state surface (the assemblage of state paths of the soils which had been consolidated with the same hydrostatic pressure) of soils. The authors expect the concept of state surface to contain one of the most possible keys for yield surface.

II. PRELIMINARY CONSIDERATIONS OF MECHANICAL PROPERTIES OF CLAYS

(1) Definition of Mechanical State of Clays

In this Paper actual stress and natural strain are used, because nominal stress and conventional strain are not appropriate to the stage of large deformation. And the word "stress" in this Paper means the "effective stress". Effective stress is the macroscopic conventional concept, and is not equal to the interparticle stress. Therefore effective stress is not suitable to discuss the micrscopic mechanics of clay skelton under shear. Then our following discussion on the mechanical behaviors of clays has limited accuracy.

Many physical components come forward as the candidates for the essential variables (*state parameters*) expressing the mechanical state of clays. Most popular candidates are the three of shear stress τ and normal stress σ' applied on the potential failure plane and void ratio e. These three state parameters were used by Hvorslev[12] (1960). In this Paper the mechanical state of clays are expressed by the points in $(\tau_{oct}, \sigma_m', e)$ space. These points are called "*state points*" and this space is called "*state space*". It might not sufficient to define the mechanical state of clays by these three components, but this set of physical components is one of the most convenient set to define the mechanical state of clays. A transition of the mechanical state of a clay element is shown by a locus of the state point. This locus is called "*state path*". It will be shown later that a rule governing the movement of state points does exist. All state points should move in the state space under the rule described later with the change of shearing conditions.

(2) Time and Path Dependent Behaviors of Clays

a) Time dependent behaviors of clays

The deformation of clays (both isotropic and distortional deformation) causes the rearrangement of the structure of clay skelton. The deformation of clays means that the structure of skelton constructed of clay particles changes to keep equilibrium. The necessary duration for this rearrangement of clay skelton induced by the external agency is considerable. The experimental studies on consolidation and creep or relaxation show the existence of this time lag. We must not ignore the fact that this time lag is considerably long. The interval of loading in conventional stress controlled tests and strain rate in conventional strain controlled tests are decided unreasonably.

Fig. 1 is the stress path of a strain controlled triaxial test on a silty clay (P.L.=31.2%, L.L.=51.5%, P.I.=20.3%). The strain rate of the beginning part and the following part of the test were 20000 min/10 mm and 300 min/10 mm respectively (35 mm diameter and 80 mm height specimen). About 10 minutes were spent for the exchange of gears to change the shearing speed. This datum shows that the strain rate affects not only the development of pore water pressure but also the mobilization of shearing resistance. In this Paper only experimental data with sufficient duration for rearrangement of clay skelton are interesting.

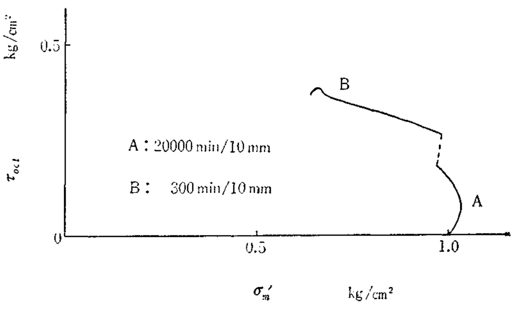

Fig. 1 Time dependent behavior of a clay under undrained shear.

b) Path dependent behaviors of clays

The mechanical behaviors of clays (for instance, state paths) are governed by the rearrangement of clay skelton caused by the external agency, and then depend not only on the present state but also on the stress history. The clays whose pre-consolidation pressures are same behave variously with various overconsolidation ratio. A theory on mechanical behaviors of clays should be adaptable not only to normally consolidated clays but also to overconsolidated clays.

Path dependent behaviors mentioned above are the mechanical behaviors of clays depending on stress history. It is noteworthy that there exist another sort of path dependent behaviors of clays. These are the mechanical behaviors of clays depending on strain history. The mechanical behaviors of a normally consolidated clays under shear with smoothly and monotonously increasing shear strain differ from these of the same normally consolidated clays under shear with cyclic increasing and decreasing shear strain. These path dependent behaviors relating to strain are shown experimentally by Murayama and Kurihara[13] (1968). The physical meaning of stress history is quite eqivalent to that of strain history in a stable ($\tau_{oct}\cdot\dot{\gamma}_{oct}>0$) clays, but not in an unstable ($\tau_{oct}\cdot\dot{\gamma}_{oct}<0$) clays, for instance, heavily overconsolidated clays.

In this Paper, only the mechanical behaviors of clays under shear with smoothly and monotously increasing shear strain are interesting.

III. STATE SURFACE

(1) Volume Change of Clays

It is convenient to consider the volume change of clays to be consisting of two components; one is the volume change induced by hydrostatic pressure and the other is that accompanied with distortional deformation. They are called consolidation and dilatancy component of volume change respectively. On the isotropic-volume change of clays, there is well ascertained experimental law; $e \sim \log \sigma_m'$ linear relation for normally consolidated clays. Fig. 2[14] is the typical $w \sim \log \sigma_m'$ relation for normally and overconsolidated clays. The linearity between e and $\log \sigma_m'$ for swelling curve and reloading curve is not recognized in strict sense, but is acceptable approximation.

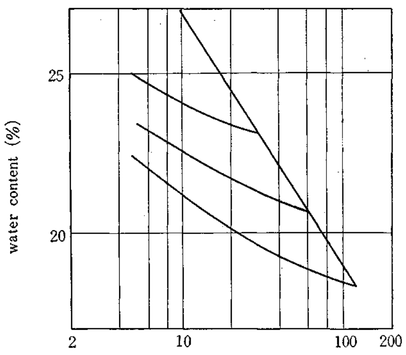

Fig. 2 $\omega \sim \log \sigma_m'$ relations for Weald Clay (after Henkel).

The $e \sim \log \sigma_m'$ linear relation is represented by the following equation:

$$e = e_0 - C \ln \frac{\sigma_m'}{\sigma_{m0}'}, \tag{1}$$

where the coefficient C equals to λ and κ for normal consolidation curve and swelling curve respectively. The point (σ_{m0}', e_0) is an arbitrary point on the normal consolidation curve. The coefficient κ is not necessarily a constant for a clay, but a variable depending on the consolidation pressure applied to the clay before swelling.

The coefficient C in the anisotropic consolidation is also equal to λ for normally consolidated clays. The failure points ploted on $(e)_f \sim (\ln \sigma_m')_f$ diagram are also represented by equation (1), and the coefficient C equals to λ for normally consolidated clays as shown by Hankel and Sowa[15] (1963). The stress ratio τ_{oct}/σ_m' in anisotropic consolidation or at failure is constant judging from the experimental data up to date. These facts suggest that $e \sim \ln \sigma_m'$ linearity under the condition of τ_{oct}/σ_m': constant is one of the fundamental properties of clays.

Assumption 1

"*Equation (1) is always satisfied while the stress ratio* τ_{oct}/σ_m' *is kept constant.*"

Differentiating equation (1):

$$de = -C \frac{d\sigma_m'}{\sigma_m'} \tag{2}$$

is obtained.

Shibata[16] (1963) gave the relationship between applied stresses and volume change accompanied with distortional deformation by means of drained tests of normally consolidated clays keeping σ_m' constant as shown in Fig. 3. Karube and Kurihara[17] (1966) made further discussion on dilatancy. From the works carried by Shibata, Karube and Kurihara, volume change accompanied with distortional deformation is given by the following equation (2):

$$\frac{\Delta e}{1+e_0} = -\mu \Delta \left(\frac{\tau_{oct} - \tau_{nd}}{\sigma_m'} \right) \tag{3}$$

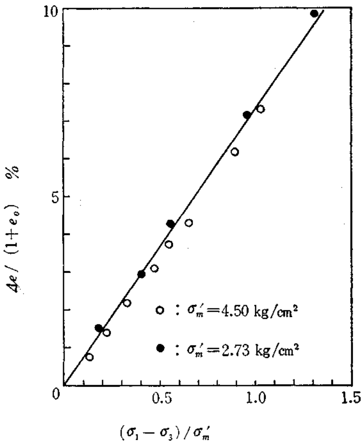

Fig. 3 Dilatancy-stress ratio relations for Amagasaki Clay (after Shibata).

where the coefficient μ is a constant for normally consolidated clay, and the symbol Δ means the total change. Until the magnitude of applied octahedral shear stress τ_{oct} exceeds the constant τ_{nd}, dilatancy does not take place. But non dilatant shear stress τ_{nd} might be essentially zero when the shearing speed is throughly slow so that the rearrangement of soil skelton can occur completely.

Equation (3) is based on the experimental facts in drained tests keeping σ_m' constant or in undrained tests of normally consolidated clays. Then it is not guaranteed directly by experiments that equation (3) is able to represent dilatancy when σ_m' increases in the process of shearing. This problem will be discussed later with drained test data.

Assumption 2

"*Equation (3) is always satisfied while σ_m' is kept constant*"

From equation (3) it is apparent that τ_{oct}/σ_m': constant means that the volume change of dilatancy does not take place and the assumption 1 is synonymous with the following statement.

"*Equation (1) is always satisfied while the volume change of dilatancy does not take place.*"

Differentiating equation (3):

$$de=\mu(1+e_0)\left(-\frac{d\tau_{oct}}{\sigma_m'}+\frac{\tau_{oct}}{\sigma_m'}\cdot\frac{d\sigma_m'}{\sigma_m'}\right) \tag{4}$$

is obtained, if $\tau_{nd}=0$.

General stress state of clays under shear consists of two components; one is the hydrostatic component and the other is deviatoric component. Therefore the volume change of clays under general stress state includes consolidation and dilatancy. But consolidation and dilatancy are not necessarily independent each other on the shearing process, and then the volume change of a clay might not equal to the simple summation of consolidation and dilatancy independent from each other.

However the infinitesimal volume change induced by the infinitesimal stress change may be nearly equal to the summation of infinitesimal consolidation and infinitesimal dilatancy.

Assumption 3

"*The infinitesimal volume change of clays is the summation of equation (2) and equation (4):*

$$de=-C\frac{d\sigma_m'}{\sigma_m'}+(1+e_0)\mu\left(\frac{\tau_{oct}}{\sigma_m'}\cdot\frac{d\sigma_m'}{\sigma_m'}-\frac{d\tau_{oct}}{\sigma_m'}\right) \tag{5}$$

Equation (5) is the approximate equation which the state parameters τ_{oct}, σ_m', e of a clay under shear should satisfy. Equation (5) may be altered judging from the more reliable experimental data and more general theoretical consideration in future.

(2) State Surface

The surface in the state space given by the solution of equation (5) is named the "*state surface*". It is necessary to discuss about the coefficients C and μ in equation (5) before solving it. C is the inclination of the projection of the section of the state surface by τ_{oct}/σ_m': constant plane to $e\sim\ln\sigma_m'$ diagram. The coefficient C is equal to λ and κ for normal consolidation curve ($\sigma_m'>\sigma_{m0}'$) and swelling curve ($\sigma_m'<\sigma_{m0}'$) respectively. But these curves are under $\tau_{oct}=0$ condition.

And it has not examined whether these relations are true or not for the general stress state.

The solution of equation (5) with the assumption that

$$C=\lambda \quad \text{for} \quad \sigma_m' > \sigma_{m0}'$$
$$C=\kappa \quad \text{for} \quad \sigma_m' < \sigma_{m0}'$$

for the fieneral stress state is not suitable to the experimental facts. Henkel[14] (1959) showed that the failure points of overconsolidated clays on $e \sim \ln \sigma_m'$ diagram supported this assumption. But the state surface derived from this assumption cannot reason out the mechanical behaviors of overconsolidated clays, for instance, positive dilatancy. This contradiction can be explained by the tendency of heavily overconsolidated clay specimens to make the local shear zone develop as mentioned by Roscoe, Schofield and Wroth[5] (1968). Murayama and Kurihara[13] (1968) pointed out this tendency too, and showed the contradictory experimental data against this assumption.

Another assumption that

$$C=\lambda \quad \text{for} \quad \sigma_m' > \sigma_{m0}'$$
$$C=f(\tau_{oct}/\sigma_m') \text{ and } f(0)=\kappa \quad \text{for} \quad \sigma_m' < \sigma_{m0}'$$

will lead too complicated solution of equation (5).

Now we would like to discuss the state surface with the following assumptions.

Assumption 4

" *The coefficient C equal to λ for all stress state except the stress state given by the state points on the vertical wall stand on the swelling curve.*"

The experimental back ground of this assumption will be discussed in the latter part of the Paper.

Assumption 5

" *The coefficient μ is constant for clays pre-consolidated with the pressure of* σ_{m0}'."

The solution of equation (5) is derived as follow with the boundary condition that the state point ($\tau_{oct}=0$, $\sigma_m'=\sigma_{m0}'$, $e=e_0$) is on the state surface given by the solution:

$$e-e_0+\lambda \ln \frac{\sigma_m'}{\sigma_{m0}'}+(1+e_0)\mu\frac{\tau_{oct}}{\sigma_m'}=0 \quad (6)$$

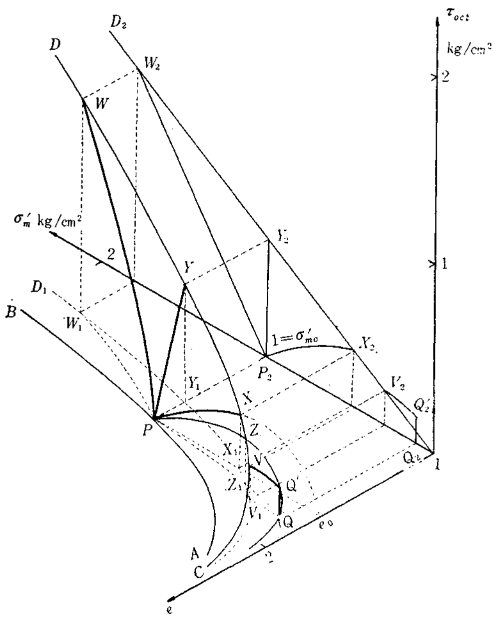

Fig. 4 Isometric view of state surface.

Fig. 4 is the isometric view of state surface for clays which have the pre-consolidation pressure of $\sigma_{m0}'=1$ kg/cm². CVZXYWD is the critical is the critical state line defined later. APB is the normal consolidation curve and the state point P is represented as ($\tau_{oct}=0$, $\sigma_m'=\sigma_{m0}'$, $e=e_0$). PZ₁Q on the $\tau_{oct}=0$ plane is the swelling curve and the vertical wall on the swelling curve

QZ_1PZQ' is named as the "swelling wall." PZQ' is on the state surface, and the swelling curve PZ_1Q is under the state surface given by equation (6). The swelling wall may have special physical meanings. The state surface given by equation (6) should be limited by $e=e_{max}$ plane and $e=e_{min}$ plane.

The state paths starting from the initial state points on the swelling line with the increasing shearing strain creep the swelling wall and then creep over the state surface given by equation (6), and they cannot enter the domain under the state surface but on the swelling wall. The critical state line does not allow the state paths to cross it. Then the state paths starting from the initial state points on PZ_1 with the increasing shearing strain can be on the swelling wall PZZ_1 or on the state surface BPZXYW, and those starting from the initial state points on Z_1Q can be on the swelling wall $Q'ZZ_1Q$ or on the state surface $Q'ZVC$. PW, PY, PX, $QQ'V$ are cited as instances of the state paths. When the restraint of increasing shearing strain is elliminated, for instance, loading and unloading cyclic shear, the state paths can enter the domain under the state surface as mentioned by Murayama and Kurihara[13] (1968).

If e=constant, the following equation is derived from equation (6):

$$\lim_{\sigma_m'\to 0}\tau_{oct}=\lim_{\sigma_m'\to 0}\frac{\sigma_m'}{(1+e_0)\mu}\left(e_0-e-\lambda\ln\frac{\sigma_m'}{\sigma_{m0}'}\right)=0 \quad (7)$$

Equation (7) means that e-axis is contained within the state surface. The fact that equation (6) consists of consolidation term and dilatancy term suggests that consolidation and dilatancy may be essentially independent phenomena.

(3) State Paths

The state paths of clays whose initial state points are on the swelling curve including normally consolidated state point ($\tau_{oct}=0$, $\sigma_m'=\sigma_{m0}$, $e=e_0$) creep the swelling wall and creep over the state surface.

When a saturated clay was sheared under undrained condition, void ratio e is kept constant:

$$e=e_i\geqq e_0 \quad (8)$$

where:

$$e_i=e_0-\kappa\ln\frac{\sigma_{mi}'}{\sigma_{m0}'} \quad (9)$$

Then the state path starting from the initial state point ($\tau_{oct}=0$, $\sigma_m'=\sigma_{mi}'\leqq\sigma_{m0}'$, $e=e_i\geqq e_0$) is given by the intersection of the state surface or the swelling wall with the $e=e_i$ plane. The intersection of the state surface with the $e=e_i$ plane is given by substituting equations (8), (9) into equation (6):

$$\tau_{oct}=-\frac{\sigma_m'}{(1+e_0)\mu}\left\{\lambda\ln\frac{\sigma_m'}{\sigma_{m0}'}-\kappa\ln\frac{\sigma_{mi}'}{\sigma_{m0}'}\right\} \quad (10)$$

If the initial state is the normally consolidated state, $\sigma_{mi}'=\sigma_{m0}'$, and then equation (10) is reduced to:

$$\tau_{oct}=-\frac{\lambda}{(1+e_0)\mu}\sigma_m'\ln\frac{\sigma_m'}{\sigma_{m0}'} \quad (11)$$

This is the same equation derived by Roscoe, Schofield and Thurairajah[7] (1963) for normally consolidated clays. Hata and Ohta[2] (1969) discussed on the stress paths of normally consolidated clays under undrained shear represented by equation (11).

Fig. 5 shows the stress paths of normally consolidated clays for various value of $\lambda/((1+e_0)\mu)$, under $e=e_0$ condition. The state paths represented by equations (10), (11) are shown in Fig. 6. All these state paths pass through the e-axis. The state paths of overconsolidated clays under undrained shear consist of two parts; one is $\sigma_m'=\sigma_{mi}'$ line on the swelling wall, RR′ or Z_1Z or QQ′ in Fig. 6 and the other is the curve from R′ or Q′ to the critical state point U or V (mentioned later) along the intersection of the state surface with $e=e_i$ plane. The state point R′ or Z or Q′ in Fig. 6 is represented by:

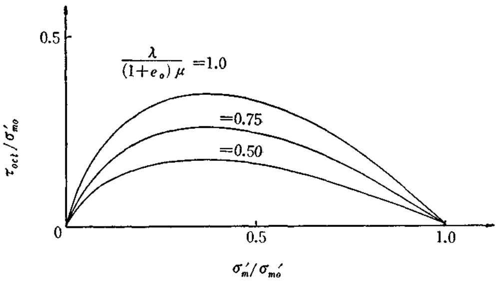

Fig. 5 Stress paths of normally consolidated clay under undrained shear.

$$e=e_i, \qquad \sigma_m'=\sigma_{mi}', \qquad \tau_{oct}=-\frac{\lambda-\kappa}{(1+e_0)\mu}\sigma_{mi}'\ln\frac{\sigma_{mi}'}{\sigma_{m0}'} \tag{12}$$

Fig. 7 shows the stress paths of clays which have the same pre-consolidation

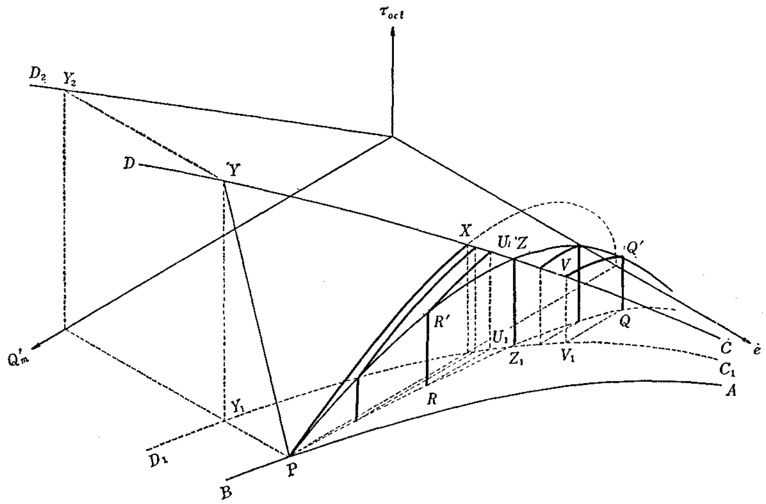

Fig. 6 State paths of the clays under undrained shear.

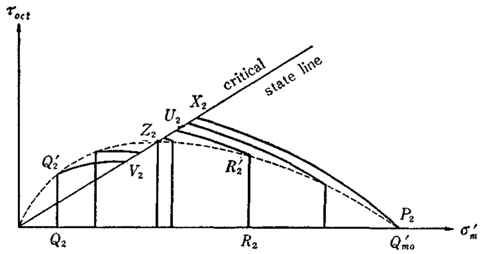

Fig. 7 Stress paths in undrained shear tests of the clays preconsolidated with the pressure of σ_{m0}'.

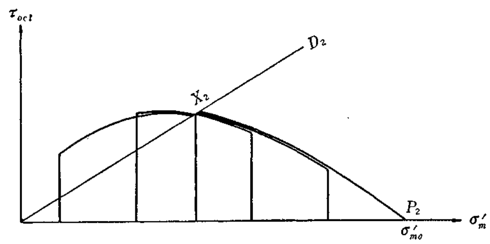

Fig. 8 Stress paths in undrained shear tests of the clays whose void ratio are equal to each other.

pressure σ_{m0}'. Fig. 8 shows the stress paths of clays whose void ratios are the same of e_i.

When clays are sheared under drained condition, the stress state is defined directly as the applied stress state. In the conventional traxial drained tests, tests, stress paths are represented by:

$$\tau_{oct}=\sqrt{2}\,(\sigma_m'-\sigma_{mi}') \tag{13}$$

Then the state paths are given by substituting equation (13) into equation (6). For normally consolidated clays $\sigma_{mi}'=\sigma_{mi}'$, then the state paths of conventional traxial drained tests on normally consolidated clays are given by:

$$e=e_0-\lambda\ln\frac{\sigma_m'}{\sigma_{m0}'}-\sqrt{2}\,(1+e_0)\mu\left(1-\frac{\sigma_{m0}'}{\sigma_m'}\right) \tag{14}$$

and equation (13). Equation (14) shows the projection of the state paths of normally consolidated clays under drained shear on $\tau_{oct}=0$ plane.

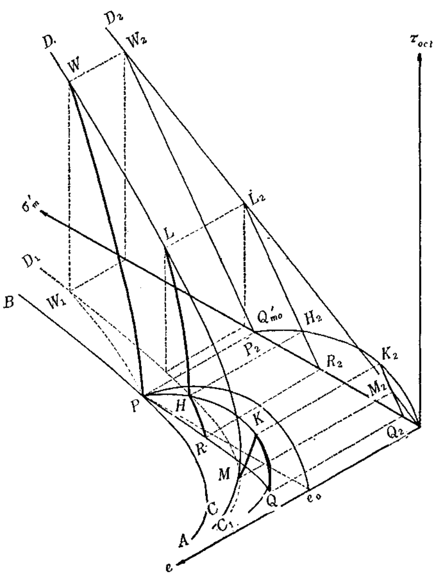

Fig. 9 State paths of clays under drained shear.

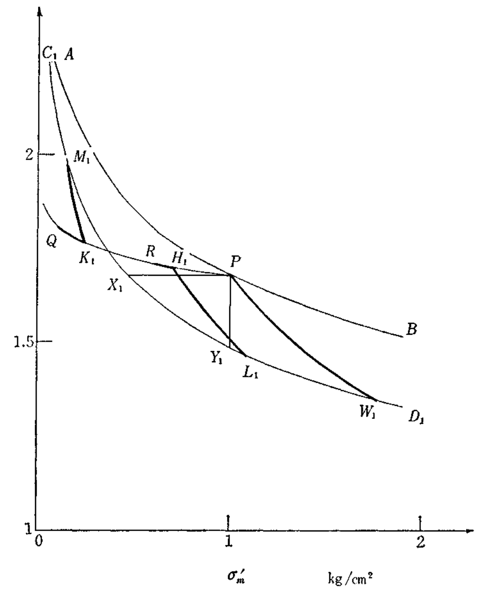

Fig. 10 State paths of clays under drained shear.

In Fig. 9, the plane given by equation (13) is PWW_2P_2, and the surface given by equation (14) is PWW_1. Then the state paths of a normally consolidated clay under drained shear is given as the curve PW. The state paths of overconsolidated clays are shown as RHL and QKM. The consist of two parts; one is the intersection of the plane given by equation (13) with the swelling wall, RH or QK and the other is the intersection of the plane given by equation (13) with the state surface, HL or KM. The projection of HL or KM on $\tau_{oct}=0$ plane is given by:

$$e=e_0-\lambda\ln\frac{\sigma_m'}{\sigma_{m0}'}-\sqrt{2}\,(1+e_0)\mu\left(1-\frac{\sigma_{mi}'}{\sigma_m'}\right) \tag{15}$$

the projections of these state paths on $\tau_{oct}=0$ plane are shown in Fig. 10. Fig. 10

reasons out the tendency of dilatancy of normally, slightly and heavily overconsolidated clays.

Substituting $\sigma_m'=\sigma_{m0}'$ into equation (6) the term of consolidation is elliminated:

$$\tau_{\mathrm{oct}}=-\frac{\sigma_{m0}'}{(1+e_0)\mu}(e-e_0) \tag{16}$$

Equation (16) represents the projection of the state path of σ_m': constant test on a normally consolidated clay on $\sigma_m'=0$ plane. Fig. 11 shows this state path PY as the intersection of the state surface with $\sigma_m'=\sigma_{m0}'$ plane. Substituting $\sigma_m'=\sigma_{mi}'\neq\sigma_{m0}'$ into equation (6), $\sigma_m'=\sigma_{mi}'$: constant state paths are derived. σ_m': constant state paths for overconsolidated clays are shown in Fig. 11 as RR'S, Z_1Z, QQ'T.

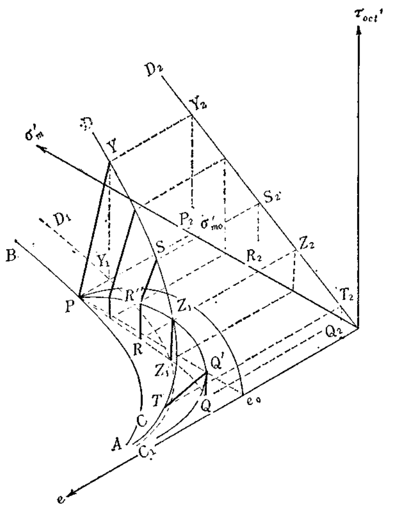

Fig. 11 State paths of clays under σ_m': constant shear.

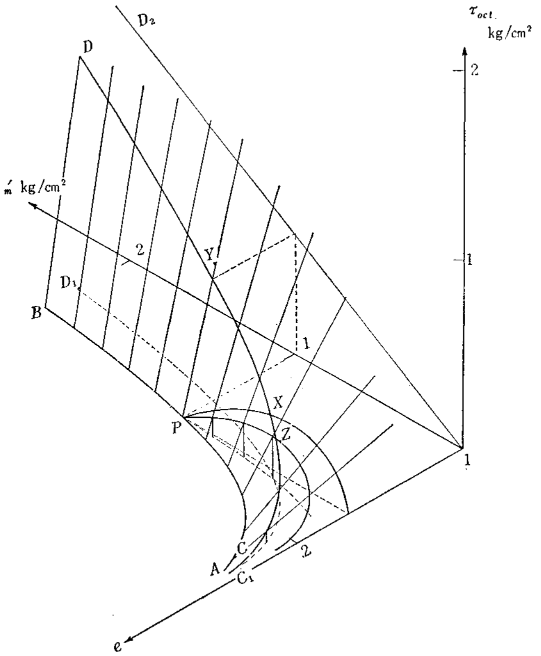

Fig. 12 Construction of state surface.

The state paths on the state surface, R'S, Q'T are given by:

$$\tau_{\mathrm{oct}}=-\frac{\sigma_{mi}'}{(1+e_0)\mu}\left(e-e_0+\lambda\ln\frac{\sigma_{mi}'}{\sigma_{m0}'}\right) \tag{17}$$

Equation (17) shows that the intersections of the state paths with $\tau_{\mathrm{oct}}=0$ plane are on the normal consolidation line which is given by:

$$e=e_0-\lambda\ln\frac{\sigma_{mi}'}{\sigma_{m0}'} \tag{18}$$

as shown in Fig. 12. The obliquity of the state path is $-\sigma_{mi}'/((1+e_0)\mu)$. From these consideration, it is apparent that the state surface is the assemblage of inclined straight lines shown in Fig. 12. $\sigma_m'=\sigma_{mi}'<\sigma_{m0}'$ state paths of overconsolidated clays climb up vertically along the swelling wall and then go straight ahead to the critical state line, along the inlined lines given by equation (17).

When $\sigma_{mi}'=0$, the obliquity of the state path becomes zero and then it is apparent that e-axis is included in the state surface. Substituting $\sigma_{mi}'=0$ into equation (18), e becomes infinite and this is not seemed to be supported by the experiments. But true $\sigma_{mi}'=0$ condition can hardly exist for clays, then this problem should be discussed in future with more fundamental considerations. From the consideration of e: constant state paths and σ_m': constant state paths, it is apparent that the mechanism of dilatancy of normally consolidated clays is essentially same to that of overconsolidated clays. The difference of positive and negative dilatancy is derived from only the difference of the initial condition.

Substituting $\tau_{oct}/\sigma_m'=k$ into equation (6), straight lines parallel to the normal consolidation curve in $e\sim\ln\sigma_m'$ diagram are derived as follow:

$$e=-\lambda\ln\frac{\sigma_m'}{\sigma_{m0}'}+\{e_0+(1+e_0)\mu k\} \tag{19}$$

One of these lines shown in Fig. 13 and Fig. 14 is the critical state line.

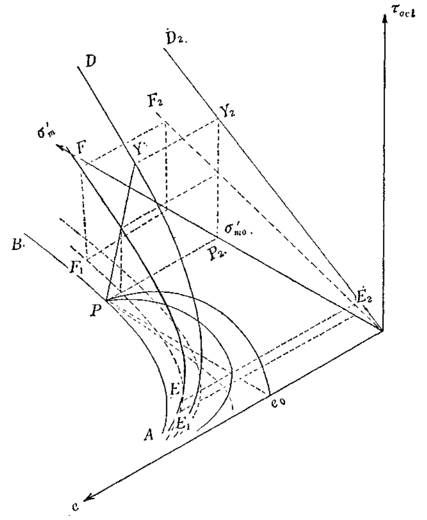

Fig. 13 τ_{oct}/σ_m': constant lines.

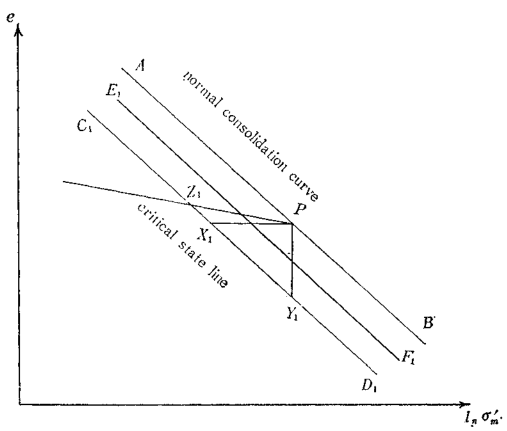

Fig. 14 τ_{oct}/σ_m': constant lines.

IV. CRITICAL STATE

(1) Definition of Critical State

Roscoe, Schofield and Wroth[5] (1958) proposed the concept of crititical state as the state represented by the following equation:

$$\frac{de}{d\gamma_{oct}}=\frac{d\sigma_m'}{d\gamma_{oct}}=\frac{d\tau_{oct}}{d\gamma_{oct}}=0 \tag{20}$$

when the state paths arrived at the critical state points, they cannot move further and τ_{oct}, σ_m', e are kept constant in spite of increasing shear strain. In strict sense, the critical state does not equal to the failure point of clays. There are two definitions of the concept of failure point; the first is $(\sigma_1'-\sigma_3')_{max}$ point and the other is $(\sigma_1'/\sigma_3')_{max}$ point. These two failure points are neighboring for ordinary normally consolidated or slightly overconsolidated clays, but not for heavily overconsolidated clays. The concept of critical state seems to be more convenient than that of failure point for the authors theory.

The interest of many researchers has been concentrated on the failure point or the critical state. The critical state is a special state on the shearing process. However, it is not a discontinuous point of the state path, but a consequent result of the smooth continuous change of state during shear. It might be one of the most possible approaches for finding out the general system of mechanical behaviors of granular materials to detect the principle ruling the stress-strain* relations of clays.

The definition of critical state by Roscoe et al. has relations with the concept of normality closely. It is in question whether the concept of normality can be applied to soils. But if normality is adaptable to clays, the critical state defined by equation (20) might be represented as the stationary point of the yield locus on $\tau_{oct}-\sigma_m'$ diagram.

Stress ratio τ_{oct}/σ_m' of the critical state point is a constant value for the clays pre-consolidated with the pressure of σ_{m0}' as shown later. Substituting $\tau_{oct}/\sigma_m'=k$ into equation (6), parallel lines to the normal consolidation curve on $e-\ln\sigma_m'$ diagram are given and this is supported by the experimental data. The difference of void ratio of the normal consolidation curve and of the critical state line for same σ_m' is constant Δe and then the projection of the critical state line on $e-\sigma_m'$ diagram is given by:

$$e=e_0-\Delta e-\lambda\ln\frac{\sigma_m'}{\sigma_{m0}'} \tag{21}$$

Substituting equation (21) into equation (6), the stress ratio τ_{oct}/σ_m' of the critical state line is given by the following equation:

$$\left(\frac{\tau_{oct}}{\sigma_m'}\right)_{crit.}=\frac{\Delta e}{(1+e_0)\mu} \tag{22}$$

This critical state line is shown in Figs. 4, 6, 7, 8, 9, 10, 11, 12, 13, 14 as CMVZUXYLWD.

If the yield locus of clays is shown, the value of Δe should be determined, but we have few reliable informations about the yield locus of clays. Roscoe, Schofield and Thurairajah[7] (1963) proposed the yield locus as the intersection of the state surface with the swelling wall. The acceptance of their proposal seems to be in question. But now we should like to continue our discussion about the the critical state with the following assumption.

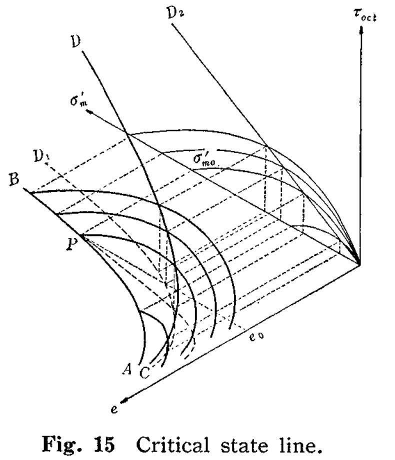

Fig. 15 Critical state line.

Assumption 6

" The yield locus of clays is the intersection of the state surface with the vertical walls parallel to the swelling wall on $e-\ln\sigma_m'$ diagram as proposed by Roscoe et al."

Fig. 15 shows that the critical state line is constructed by the stationary points of the yield loci.

The projection of the yield locus on $\tau_{oct}=0$ plane is given by:

$$e=e_Y-\kappa \ln \frac{\sigma_m'}{\sigma_{mY}'} \tag{23}$$

if the intersecting point of the yield locus with the normal consolidation curve is given as (σ_{mY}', e_Y). Substituting equation (23) into equation (6), the projection of the yield locus on $e=0$ plane is given by:

$$\tau_{oct}=-\frac{\lambda-\kappa}{(1+e_0)\mu}\sigma_m' \ln \frac{\sigma_m'}{\sigma_{mY}'} \tag{24}$$

The critical state point defined as the stationary point of the yield locus is derived as follow by differentiating equation (24):

$$\tau_{oct}=\frac{\lambda-\kappa}{(1+e_0)\mu}\sigma_{mY}' \exp(-1) \qquad \text{(a)}$$

$$\sigma_m'=\sigma_{mY}' \exp(-1) \qquad \text{(b)} \qquad (25)$$

$$e=e_Y+\kappa=e_0+\lambda \ln \frac{\sigma_{m0}'}{\sigma_{mY}'}+\kappa \qquad \text{(c)}$$

The projection of the critical state line on $e-\sigma_m'$ diagram is derived from equation (25)(c) as:

$$e=e_0-(\lambda-\kappa)-\lambda \ln \frac{\sigma_m'}{\sigma_{m0}'} \tag{26}$$

Comparing equation (26) with equation (21), it is apparent that:

$$\Delta e=\lambda-\kappa \tag{27}$$

Substituting equation (27) into equation (22):

$$\left(\frac{\tau_{oct}}{\sigma_m'}\right)_{crit.}=\frac{\lambda-\kappa}{(1+e_0)\mu} \tag{28}$$

Comparing equation (11) and equation (24), the coefficient of the stress path $-\lambda/((1+e_0)\mu)$ is given by deviding the coefficient of the assumed yield locus $-(\lambda-\kappa)/((1+e_0)\mu)$ by $(1-(\kappa/\lambda))$. This result coincides with the energy theory of Roscoe, Schofield and Thurairajah[7] (1963) for normally consolidated clays.

(2) Critical State Line

The critical state line is closely neighboring the ordinary $(\sigma_1'-\sigma_3')_{max}$ failure envelope on $\tau_{oct}-\sigma_m'$ diagram for clays wetter than critical state, but for heavily overconsolidated clays dryer than critical state. The intersection of τ_{oct} axis and the critical state line of the clays whose initial state points are on the same swelling curve is zero. If the critical state line is represented as follows:

$$\tau_{oct}=c'+\sigma_m' \tan \varphi' \tag{29}$$

c' is to be zero and $\tan \varphi'$ is equal to equation (22) or equation (28).

The $(\sigma_1'-\sigma_3')_{max}$ failure envelope of clays wetter than critical coincides approximately with the critical state line, but the peak points of heavily overconsolidated clays do not necessarily coincide with the critical state points. From Figs. 7, 8, it is apparent that the stress paths of overconsolidated clays under undrained shear can pass through the upside of $(\sigma_1'-\sigma_3')_{max}$ failure envelope. This is supported experimentally by Murayama and Kurihara[18] (1968). These results show that the stress state of clays can be outside the failure envelope substituted by the critical state line as shown in Fig. 16.

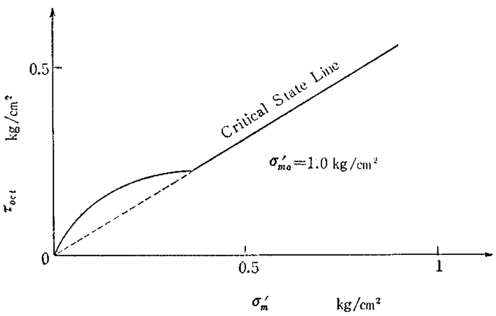

Fig. 16 Envelope of undrained stress paths of the clays preconsolidated with the pressure of 1.0 kg/cm².

The inherent physical meanings of Mohr-Coulomb failure criterion for clays are made vague by these results. Then supposing Mohr-Coulomb failure criterion to be true for clays, it may be mere accident. The theory mentioned above is based on the concept of octahedral stresses without clear reasons, but it can reason out many mechanical behaviors of clays systematically. These considerations do not support that Mohr-Coulomb criterion is more adaptable for clays than extended von Mises criterion.

V. ANALYSIS ON UNDRAINED AND DRAINED TESTS

(1) The Clay Used in the Investigations

The clay used in the investigations is Osaka alluvial clay containing shells deposited in the sea water. The index properties of the clay, percentage of clay particles ($\leq 2\mu$), activity and specific gravity are shown in Table 1. The water was added to the clay and then this clay paste was made to pass the 0.074 mm screen to elliminate the shells and was left still in a bucket for about two months.

Table 1 Physical properties of clay specimen.

L.L.	P.L.	P.I.	$\leq 2\mu$	Activity	Specific gravity
69.2%	32.5%	36.7%	25%	1.47	2.64

Then the clay paste was consolidated with its own weight. This half consolidated clay paste was mixed again and consolidated in a cylinder (30 cm diameter, 15 cm height) with about 0.3 kg/cm² for four weeks. The consolidated clay was coated with parchment paper and wax and then was preserved in the machine oil for several months. The water content of the clay in this state was about 88%. This high water content of the clay seemed to be derived from the high water content of the clay paste before consolidation.

(2) Test Procedure

a) Triaxial test

The clay specimens (35 mm diameter, 80 mm height) covered with drainage filter and thin rubber membranes were mounted directly on the pedestal in the triaxial cell. Porous stone was not used because of the possibility of residual air bubbles. Cell pressure and pore pressure were measured by the height difference of mercury columns. Consolidation time was about 5 days and swelling time was about 15 days. Back pressure was applied for the undrained tests. Strain rate was selected quite slow. Loading rate of the stress controlled drained tests were about a few weeks for one loading step. Area correction was put in with natural strain.

b) Shear box test

Constant volume tests were carried. In constant volume test, the volume of the specimen (60 mm diameter and about 10 mm thickness) is maintained constant during the test by increasing or decreasing the normal pressure keeping free drainage condition. Area correction was not put in because of the reasons given by Matsuo and Karube[18] (1966). Loading rate of the test was selected quite slow.

(3) Analysis on Undrained Tests

a) Normally consolidated clay

A normally consolidated clay (initial $w=88.34\%$) was consolidated with the cell pressure of about 0.998 kg/cm^2 (the cell pressure was changed from 0.967 kg/cm^2 to 1.029 kg/cm^2 by the friction of the constant pressure apparatus) for 5 days. The void ratio e_0 of the specimen after consolidation was 1.594. The coefficient λ and κ for the clay measured by consolidation and swelling tests in triaxial cell were 0.2789 and 0.0275 respectively. The stress path of the clay under strain controlled shear with the strain rate of 4.40×10^{-4} mm/min (22719.98 min/10 mm) is given in Fig. 17. The selection of the critical state point on the stress path is quite difficult practically. In this case the authors would like to select the ○ point as the critical state point.

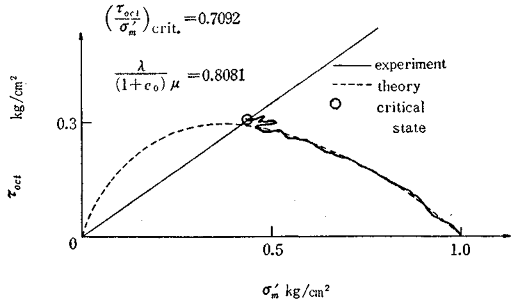

Fig. 17 Stress path of the normally consolidated clay under undrained shear.

The theoretical stress path of a normally consolidated clay under undrained shear is given by equation (11) and the theoretical stress path for $\lambda/\mu(1+e_0)=0.8081$ is suitable with the experimental stress path. At the experimental critical state point, $\tau_{oct}=0.3058$ kg/cm^2 and $\sigma_m'=0.4312$ kg/cm^2, then the stress ratio $(\tau_{oct}/\sigma_m')_{crit.}=0.7092$. The theoretical stress ratio at the critical state derived from equation (28):

$$\left(\frac{\tau_{oct}}{\sigma_m'}\right)_{crit.}=\frac{\lambda-\kappa}{(1+e_0)\mu} \tag{28}$$

substituting the values of $e_0=1.594$, $\lambda=0.02789$, $\kappa=0.0275$ and $\lambda/(1+e_0)\mu=0.8081$ is 0.7287 which is differ from the experimental value of 0.7092 to some extent. This difference may be the error of the measurement of the value of λ. If the theoretical stress ratio $(\tau_{oct}/\sigma_m')_{crit.}$ coincides with the experimental value of 0.7092, the values λ and μ derived by solving the following simultaneous equations:

$$\frac{\lambda}{(1+e_0)\mu}=0.8081, \qquad \frac{\lambda-\kappa}{(1+e_0)\mu}=0.7092$$

subsituting the values of $e_0=1.594$ and $\kappa=0.0275$ are $\lambda=0.2522$ and $\mu=0.1203$. All following calculations for the theoretical analysis will be carried with $\lambda=0.2522$, $\kappa=0.0275$ and $\mu=0.1203$ shown in Fig. 18.

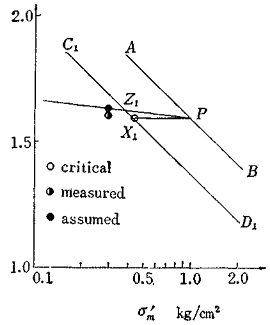

Fig. 18 Normal consolidation curve and swelling curve.

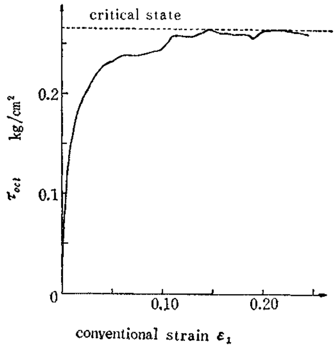

Fig. 19 Stress-strain relation of the overconsolidated clay.

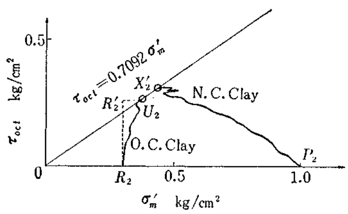

Fig. 20 Stress paths of the clays under undrained shear.

b) Overconsolidated clay

An overconsolidated clay (initial water content was 88.44%) was consolidated with the cell pressure of 1.010 ± 0.001 kg/cm² for 5 days and rebound to the pressure of 0.296 ± 0.031 kg/cm² for 16 days. This specimen was sheared under strain controlled undrained condition. The strain rate was selected as 3.049×10^{-4} mm/min (32800 min/10 mm). Judging from the stress strain relation shown in Fig. 19, the authors would like to select the ○ point in Fig. 20 as the critical state point. The stress path of this overconsolidated clay is shown in Fig. 20, and the stress ratio (τ_{oct}/τ_m') of the ○ point of the overconsolidated clay is 0.7049.

The critical state line in Fig. 20 derived from $(\tau_{oct}/\sigma_m'{}_{crit.})=0.7092$ is suitable with the experimental critical state point of the overconsolidated clay. The theoretical stress path $R_2R_2'U_2$ for the overconsolidated clay in Fig. 20 calculated with the assumption that the normally consolidated clay mentioned above is sheared after rebounding to the ● point in Fig. 18 is suitable with the experimental fact that the void ratio of rebounded clay was 1.608 shown by ◑ point in Fig. 18.

c) Shear box test

Stress paths of normally consolidated clays under shear in shear box are shown in Fig. 21. Strain rate are given in Table 2. Assuming that the stress state (τ, σ_N') is synonymous with that on the φ' maximum mobilized plane as shown in Fig. 22, the theoretical stress paths are calculated[2] as shown in Fig. 23. The stress state in the shear box seems to be approximately plane strain. Karube and Harada[19] (1967) showed that the value of $N=$

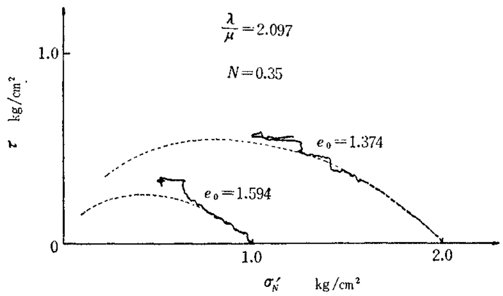

Fig. 21 Stress paths of normally consolidated clays under shear in shear box.

Table 2 Strain rate of the shear box test.

Displacement ($\times 10^{-2}$ mm)	Strain rate mm/min
Consolidation Pressure 2 kg/cm^2	
0~70	0.01
70~140	0.02
140~300	0.04
300~500	0.05
500~600	0.10
Consolidation Pressure 1 kg/cm^2	
0~20	0.005
20~50	0.01
50~100	0.02
100~200	0.04
200~600	0.05

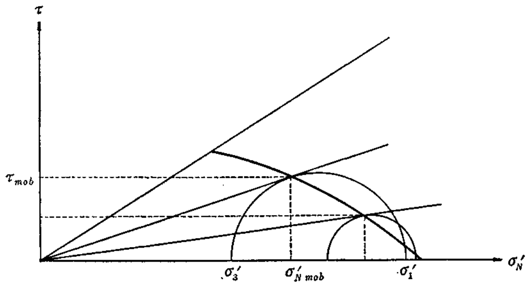

Fig. 22 Stress state on the φ' maximum mobilized plane.

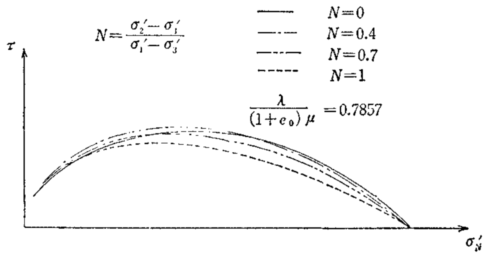

Fig. 23 Theoretical stress paths of clays in shear box.

$(\sigma_2'-\sigma_3')/(\sigma_1'-\sigma_3')$ varied from 0.2–0.4. Hata and Ohta[2] (1969) showed that the theoretical stress paths normally consolidated clays under plane strain shear were substituted approximately by the theoretical curves for $N=0.35$. The value of λ/μ of the theoretical stress paths in Fig. 21 is derived from $\lambda=0.2522$ and $\mu=0.1203$ given by the triaxial test. The theoretical curves agree with the experimental stress paths. And this agreement suggests the appropriateness of the assumption about the stress state in the shear box.

(4) Analysis on Drained Tests

a) Normally consolidated clay

A normally consolidated clay (initial water content was 86.18%) was consolidated with the cell pressure of 0.967 ± 0.050 kg/cm^2 for 2 days and then sheared under stress controlled drained condition. The void ratio after consolidation was 1.606. The axial stress was increased with 5 steps remaining the radial stress constant. The expelled water is shown in Fig. 24.

Fig. 25 shows the experimental data comparing with the theoretical curve derived from the data of $\lambda=0.2522$, $\kappa=0.0275$ and $\mu=0.1203$ by means of equation (14) and it suggests the possibility of the incomplete drainage in spite of the very long duration for each step of loading. The clay specimen could not arrived at the critical state point, because the deformation of the specimen was too large to continue the test with the next step of loading. The stress paths of the undrained and drained tests on the normally consolidated clay and the theoretical critical state line are shown in Fig. 26. Fig. 27 shows the projection of the state paths of the undrained and drained test to the $\tau_{oct}=0$ plane, and the two points

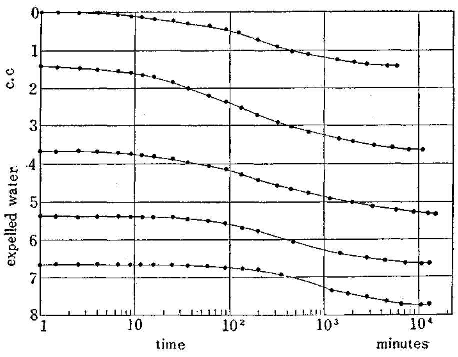

Fig. 24 Expelled water～time relation of consolidated clay.

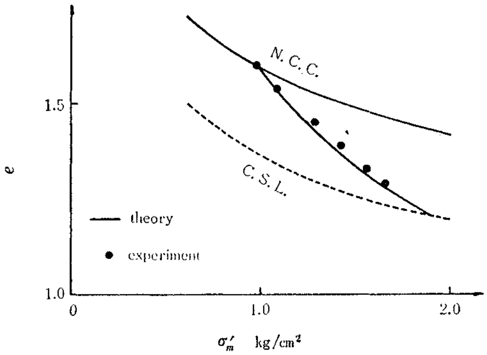

Fig. 25 $e-\sigma_m'$ relation of the normmally the norally consolidated clay under drained shear.

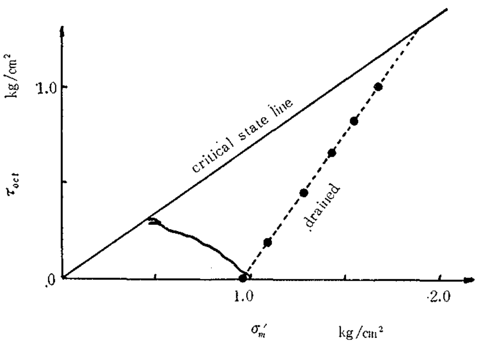

Fig. 26 Stress paths of the normally consolidated clays.

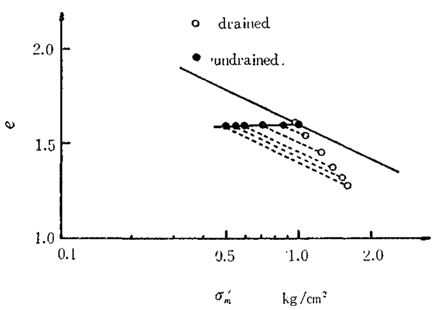

Fig. 27 τ_{oct}/σ_m': constant lines for the normally consolidated clays.

combined with the dotted line is the pair of the same stress ratio τ_{oct}/σ_m'. The assumption 4 states that these dotted lines are parallel to the normal consolidation curve. Judging from the experimental data shown in Fig. 27 (including the possibility of incomplete drainage in drained test), the assumption 4 is generally acceptable.

b) Overconsolidated clay

An overconsolidated clay (initial water content was 87.46%) was consolidated with the pressure of 1.892 ± 0.051 kg/cm² for 16 days ($e_0=1.3738$) and rebounded to the pressure of 0.9189 ± 0.0364 kg/cm² for 9 days. This specimen was sheared under stress controlled drained condition with 3 steps of loading. The effective mean stress σ_m' was kept nearly constant because the σ_m': constant test in strict sense cannot be carried on by the difficulties in the exact expection of the shear strain and of the radial stress. The shear strain increased suddenly about 5 minutes after the last step of loading and it was apparent that the last load was over the critical load.

The theoretical state path of an overconsolidated clay in σ_m': constant test is given in Fig. 11 as RR′S. Fig. 28 shows the state path of this test. The last step of loading was carried rather under the undrained condition than under σ_m':

constant condition, because the pore pressure could not dissipate. The point 0, 1, 2, 3 are the experimental data and the point S_0 and S_2 are the critical points for $\sigma_m'=0.9189$: constant and $\sigma_m'=1.0232$: constant test respectively. The section of the swelling wall and the state surface are shown in Fig. 28 as RR′ and R′S_0 respectively for $\sigma_m'=0.9189$. Fig. 28 shows that the swelling wall is standing vertically on the swelling curve.

These experimental data can be reasoned out approximately by the theory mentioned above.

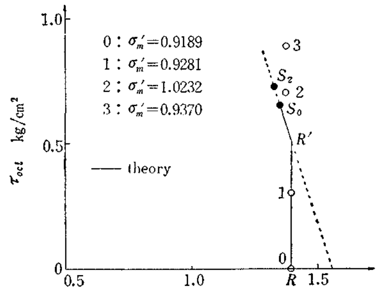

Fig. 28 σ_m': constant test on the overconsolidated clay.

VI. EXTENTION OF THE THEORY TO SANDS AND SILTS

Clays, sands and silts are all the granular materials and then should behave mechanically ruled by the similar principles. The theory for clays mentioned above is based on the experimental laws about consolidation and dilatancy. If these assumptions are adaptable to sands and silts, the discussion for clays can be extended to all sorts of soils.

The linear relation between e and $\ln \sigma_m'$ is valid for normally consolidated sands as shown in Fig. 29 given by Vesić and Clough[20] (1968). Normally consolidated state may be defined as the state that the maximum void ratio is maintained for the applied hydrostatic pressure. In this sense, normally consolidated state is quite sensitive state to disturbance. The skelton of soil particles under normally consolidated state is prone to decrease the void ratio with small disturbances.

Usually sands may be in overconsolidated state and this is supported by the undrained stress paths of sands as shown in Fig. 30. Silts may be in similar state judging from the data given by Schultze and Horn[21] (1965). It is quite

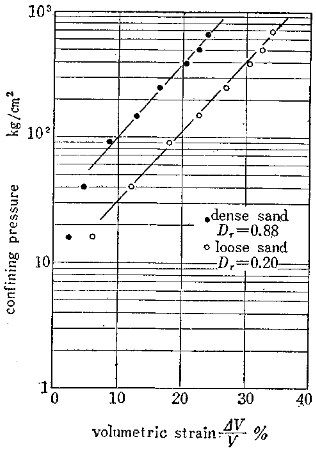

Fig. 29 Consolidation of sands (after Vesić and Clough).

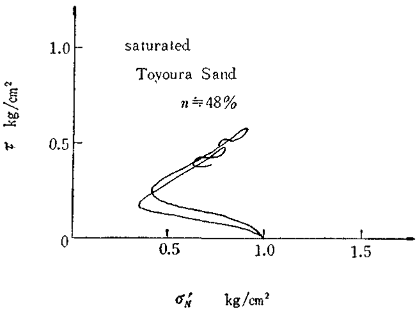

Fig. 30 Stress paths of sands under constant volume shear in shear box.

difficult to make sands be in normally consolidated state under the usual cell pressure, and therefore shear tests of normally consolidated sands are hardly carried on, but a few cases. The stress paths of normally consolidated sands under undrained shear given by Bishop, Webb and Skinner[22] (1965) are quite similar to those of normally consolidated clays. These facts suggest that the assumption on dilatancy equation (31) is adaptable to sands. Because of experimental difficulty, it can hardly be proved to be so directly by σ_m': constant tests of normally consolidated sands. If the theory is adaptable to sands, the shearing properties of ordinary sands under σ_m': constant condition should be similar to those of overconsolidated clays shown in Fig. 11 as QQ'T. σ_m': constant test carried by Yagi[23] (1968) is quite similar to QQ'T in Fig. 11.

From those facts, it can be concluded that the theory is adaptable to sands and silts. It is noteworthy that the theoretical state paths of soils in overconsolidated state are quite similar to those given by Rowe[24] (1962).

VII. CONCLUSION

The necessary soil constants in the theory are κ, λ, μ and Δe. It should be discussed further whether $\Delta e=\lambda-\kappa$. The three soil constants κ, λ, μ might be the substantial soil constants. Many mechanical behaviors of soils (saturated or unsaturated) in any state (normally consolidated or overconsolidated) are derived from the theory based on κ, λ, μ. The theory shows that the differences of mechanical behaviors of the soils which have the same soil constants κ, λ, μ are caused only by the different initial conditions and then soils in all states are ruled by the unique principle. The theory will be discussed critically with the more reliable experimental data and more basic considerations. But at present, the theory can reason out the mechanical behaviors of soils systematically.

ACKNOWLEDGEMENTS

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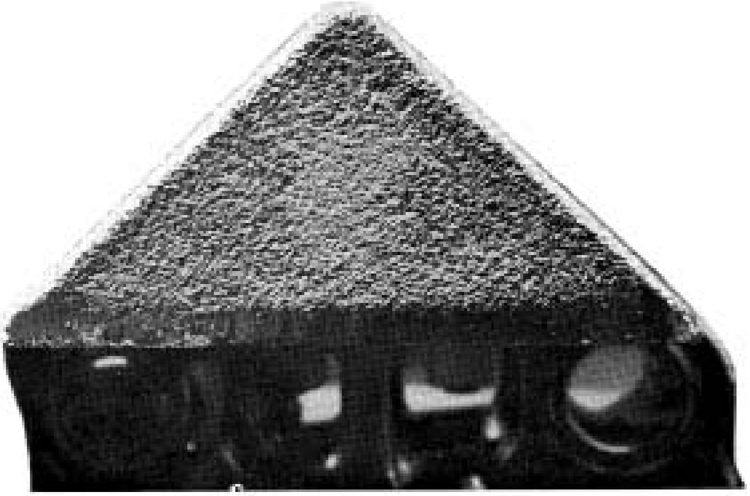
(*Received May 31, 1969*)

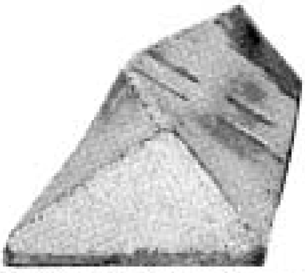
NTKが正統派です



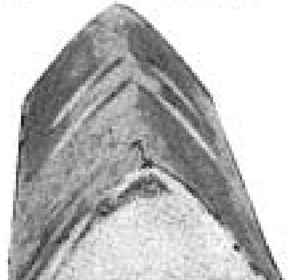
二等辺直角三角形の純正ＮＴＫ三角シュー

悪質地盤の処理に抜群の働きを示す湿地ブルドーザは、日本のユーザーにとって欠かせない機動力であることはご承知の通りですが、その普及の目覚ましさにつれて、あるメーカーが「日本No.1の技術が開発した…」といえば、一方は「世界No.1の技術が…」とうたう。しかし、一寸待って下さい。湿地ブルドーザの生命である三角シューは、北海道の泥炭地と取組んだ日特金属が発明したもの。それまでシューは、グローサしかなかったのです。日特の開発した三角シューは〝土がつかず、土を乱さず、転圧力が大きく、傾斜地にも強い〟という画期的な性能を備えており、日本のユーザーの要望に完全に応えたもの。そして湿地研究から生れた頑強な足廻り、理想の車体バランス等正統派ＮＴＫの湿地ブルドーザは断然ピカ一です。超々湿地ブルまで種類も豊富。湿地ブルについては専門家のＮＴＫにまずご相談下さい。

三角シューのいろいろ

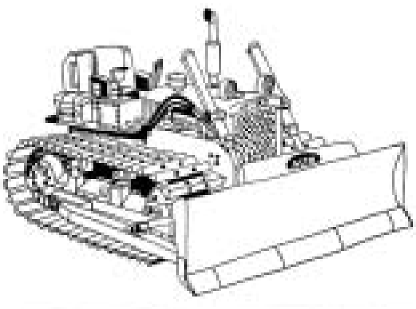


稜線にカーブをつけた三角シュー



側面に丸味をつけた三角シュー

豊富な湿地シリーズ	接地圧	定格出力	総重量
NTK－6C 湿地	0.29kg／cm²	120ps	15,000kg
NTK－5 湿地	0.27kg／cm²	76ps	9,200kg
NTK－5 超湿地	0.19kg／cm²	76ps	10,300kg
NTK－5 超々湿地	0.13kg／cm²	76ps	10,000kg
NTK－4 湿地	0.25kg／cm²	61ps	8,100kg



NTK
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