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# UNDRAINED SHEAR STRENGTH OF REMOULDED MARINE CLAYS

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## ABSTRACT

In order to investigate the change in undrained shear strength characteristics corresponding to the change in the test condition and soil type, four different types of consolidated undrained triaxial tests were performed on some natural marine clays in Japan, and the test result was compared with that previously obtained for artificially mixed soils. The following conclusions were obtained. i) Relationship between the undrained shear strength characteristics and the plasticity index obtained for the natural marine clays was practically identical to that obtained for the artificially mixed soils. ii) Angle of shear resistance and pore pressure coefficient at failure were found to be approximately in linear relation with the plasticity index corresponding to the type of triaxial tests. iii) Calculated value of the ratio of the undrained shear strength to consolidation pressure, based on the experimentally obtained relationship between the strength parameters and the plasticity index, compared well to the value measured in each of four different types of triaxial tests. iv) Relationship between undrained shear strength characteristics and the plasticity index obtained in the present study may be used, as the first approximation, for some of marine clays in coastal area of Japan.

**Key words** : anisotropy, cohesive soil, consolidated undrained shear, plasticity, shear strength, triaxial compression test (**IGC** : D 6)

## INTRODUCTION

Geotechnical engineers encounter a variety of soils in their construction sites and engineering properties of soils vary from those of clays to sands. In engineering practice, however, soils have been broadly classified into two types of cohesive soils and cohesionless soils. This classification of soil types stems from the recognition of the fact that the behaviour of soils largely depends on their time rate of change in effective stress due to loading. Geotechnical engineers benefit from this classification of soil type, since it provides a set of simplified design principles, i. e., the effective stress concept for cohesionless soils and the total stress concept for the cohesive soils. In practice, how-

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ever, it is not necessarily easy to judge which type a particular soil ought to be classified into. Natural soils involve a group of soils which seem to have intermediate properties between cohesive soils and cohesionless soils. Such soils have been named the "intermediate soils" (Osterberg, 1978).

In an attempt to find out the boundary between the cohesive soils and cohesionless soils, various soil tests had been made on artificially mixed soil samples having some range of consistency characteristics (Trollope and Zafar, 1956; Kurata and Fujishita, 1961). A similar sort of tests were carried out by the authors on reconstituted soil samples prepared by mixing the Kawasaki marine clay with the Toyoura sand and its crushed portion. In that series of tests, the plasticity index was selected for specifying the soil type, considering an importance of this index in geotechnical engineering, and four kinds of reconstituted soil samples having the plasticity indices of 10, 15, 20 and 30 were used. In what follows, these soil samples are named the Kawasaki clay-mixture series.

In the test on the Kawasaki clay-mixture series, consolidated undrained triaxial compression and extension tests were performed, where the soil specimens were subjected to either isotropic or $K_0$ consolidation. It is generally known that the undrained shear strength depends both on the in-situ conditions and on the test procedure, and the value of $c_u/p$ for a clay can vary widely. It was noted that both the external force and the structure of the soil in the failure zone influenced the shearing strength. In the isotropically consolidated specimen, i.e., the principal stresses are all equal, and the soil particles are randamly oriented. In the $K_0$-consolidated specimen, the soil particles are parallel to one another in groups of vary size.

The results of the Kawasaki clay-mixture series showed that the undrained shear strength anisotropy, in terms of the strength difference between the compression and extension tests, increased with the decrease in the plasticity index, and that the soils of the plasticity indices of 15 and 20 were considered the intermediate soils in view of the shape of stress paths and a degree of the strength anisotropy (Nakase and Kamei, 1983). It has been thought, however, necessary to verify the generality of the relationship between the strength characteristics and the plasticity index obtained for the Kawasaki clay-mixture series, since the soil samples of that series are artificially mixed soils. For this purpose, a similar series of triaxial tests were performed on some natural marine clays taken from various sites of coastal area in Japan, and the test result was compared with that of the Kawasaki clay-mixture series.

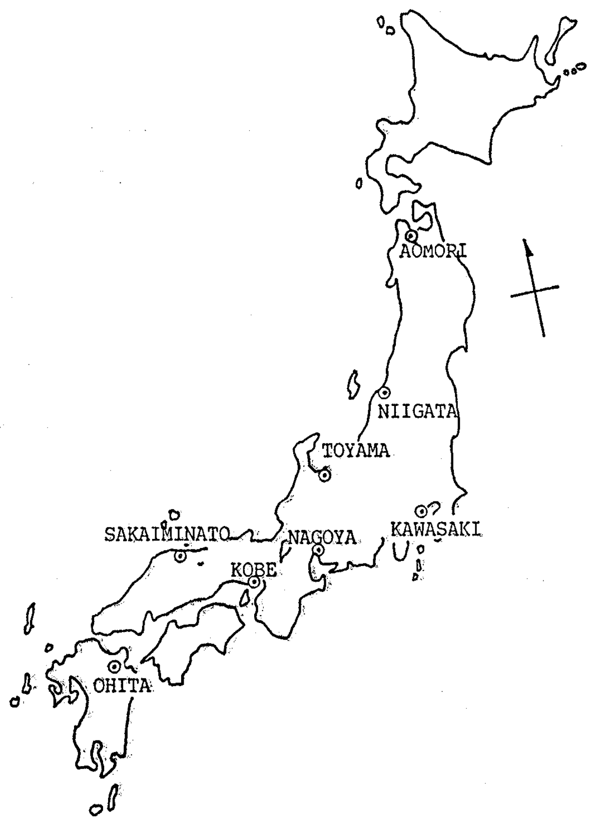


**Fig. 1. The location of the samples**

## EXPERIMENTS

*Samples*

Eight kinds of natural marine clays were used in the experiment described in this paper. These soils were dredged in different port sites in Japan as shown in Fig. 1. Index properties of these natural soils are

**Table 1. Index properties of the samples**

(a)

| | Kobe | Aomori | Toyama | Niigata | Sakai-minato | Nagoya | Ohita |
|---|---|---|---|---|---|---|---|
| $G_s$ | 2.73 | 2.71 | 2.66 | 2.73 | 2.62 | 2.66 | 2.72 |
| $w_L$ (%) | 86.1 | 81.8 | 78.4 | 70.9 | 54.2 | 45.5 | 41.8 |
| $w_p$ (%) | 30.3 | 34.2 | 35.2 | 33.2 | 25.8 | 21.8 | 29.2 |
| $I_p$ | 55.8 | 47.6 | 43.2 | 37.7 | 28.4 | 23.7 | 12.6 |
| Sand (%) | 4.5 | 20.5 | 24.5 | 19.0 | 11.4 | 28.7 | 47.5 |
| Silt (%) | 72.0 | 41.5 | 36.5 | 42.3 | 70.7 | 61.3 | 42.5 |
| Clay (%) | 23.5 | 28.0 | 39.0 | 38.7 | 17.9 | 10.0 | 10.0 |

(b)

| | M-50 | M-30 | M-20 | M-15 | M-10 |
|---|---|---|---|---|---|
| $G_s$ | 2.69 | 2.69 | 2.68 | 2.67 | 2.67 |
| $w_L$ (%) | 84.5 | 55.3 | 42.7 | 34.8 | 27.6 |
| $w_p$ (%) | 33.4 | 25.9 | 23.3 | 20.2 | 16.9 |
| $I_p$ | 51.1 | 29.4 | 19.4 | 14.6 | 10.7 |
| Sand (%) | 4.7 | 16.1 | 34.2 | 47.4 | 60.4 |
| Silt (%) | 47.1 | 61.6 | 48.7 | 39.2 | 28.9 |
| Clay (%) | 48.2 | 22.3 | 17.2 | 13.4 | 9.7 |

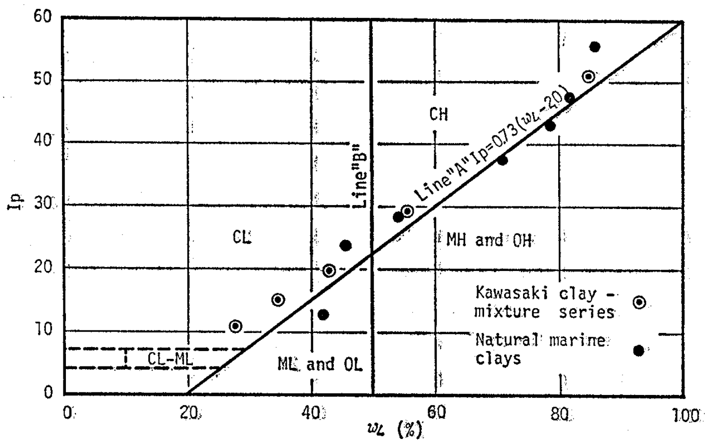


**Fig. 2. Plasticity chart**

shown in Table 1, together with those of the Kawasaki clay-mixture series. The Kawasaki clay-mixture series is composed of four kinds of samples of M-10, M-15, M-20 and M-30, after their plasticity index and the letter M stands for mixture. The M-30 is the Kawasaki clay. In the present series of test, the Kawasaki clay of the plasticity index of 50 is included, and this particular clay of M-50 is to be included in the Kawasaki clay-mixture series. Consistency characteristics of these soil samples are illustrated in the plasticity chart in Fig. 2. As seen in the figure, there seems no appreciable difference between the natural marine clays and the Kawasaki clay-mixture series.

The natural soil samples were thoroughly remoulded at a water content of approximately 1.3 times their liquid limit and passed through a 2000 $\mu$m sieve to get rid of fragments of shells and other undesirable stuff found in the soil sample. Thus prepared soil slurries were stored in a plastic container with distilled water on top of it.

Prior to preconsolidation process, the soil slurry was puddled in a soil mixer for about an hour and deaired under a pressure of 720 mmHg below atmospheric pressure. The soil slurry was carefully poured into a preconsolidation mould of 200 mm in diameter and 270 mm in height, then consolidated under a pressure of 68.9 kPa. After the completion of consolidation, a block of soil was extruded by pushing a bottom plate of the mould up and then cut into five equal pieces. Having coated with paraffin to prevent drying, the soil pieces were stored in a constant temperature room. Each test specimen was trimmed from the soil piece when needed. These procedures followed those employed for the Kawasaki clay-mixture series. The test specimen prepared through these procedures are considered a reconstituted, young and saturated. The authors have taken a view that finding some useful engineering relationships between plasticity index and the behaviour of soils with a group of artificially mixed soils may provide design engineers the first approximation to understand the complicated behaviour of natural marine clays. In the present study, therefore, the experiment was performed on clays in the normally consolidated young state which have no aging effect and it can be considered that the soil shows a fundamental mechanical behaviour in such state.

*Triaxial Tests*

Triaxial apparatus and the facilities have been described in detail elsewhere (Nakase and Kamei, 1983 ; Nakase et al., 1984). A series of consolidated undrained triaxial compression and extension tests were performed on eight kinds of natural soil samples. Size of the test specimen was 50 mm in diameter and 120 mm in height.

In the compression tests, specimens were consolidated either isotropically or under $K_0$ conditions and then subjected to undrained compression by increasing axial pressure, lateral pressure being kept constant during compression process. These triaxial tests are denoted as $\overline{\text{CIUC}}$ and $\overline{\text{CK}_0\text{UC}}$ tests respectively. In the extension tests, specimens were consolidated either isotropically or under $K_0$ conditions and then subjected to undrained extension by decreasing axial pressure, lateral pressure being kept constant during extension process. These triaxial tests are denoted as $\overline{\text{CIUE}}$ and $\overline{\text{CK}_0\text{UE}}$ tests respectively.

Two different values of vertical effective consolidation pressure were used in the consolidation process ; 196 kPa and 392 kPa in the

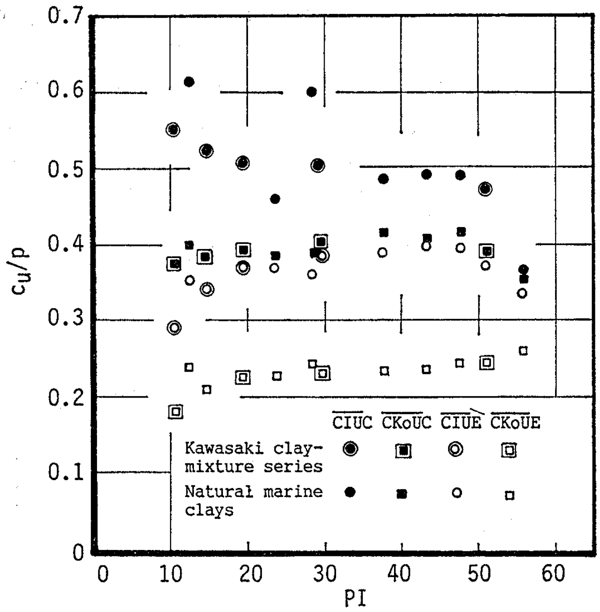


**Fig. 3 Relationship between $c_u/p$ and $I_p$**

case of compression tests, and 392 kPa and 588 kPa for extension tests. A back pressure of 196 kPa was applied to all the test specimens throughout the consolidation and shear processes. Compression and extension tests were carried out with a constant rate of axial strain of 0.07%/min.

## TEST RESULTS AND DISCUSSIONS

*Undrained Shear Strength*

Fig. 3 shows relationship between $c_u/p$ values and the plasticity index (PI) for each test condition, where $c_u$ is the undrained shear strength and $p$ is the vertical effective consolidation stress in the consolidation process. As seen in the figure, the result obtained from the natural marine clays are very close to those obtained from the Kawasaki clay-mixture series. Values of $c_u/p$ are found to decrease in order of the $\overline{\text{CIUC}}$, $\overline{\text{CK}_0\text{UC}}$, $\overline{\text{CIUE}}$ and $\overline{\text{CK}_0\text{UE}}$ tests, irrespective of the PI of the samples. Looking over the test results, it may be said that the difference in $c_u/p$ value due to a difference in the test condition is more marked than that due to a difference in the PI.

In order to examine the fundamental concepts of shear strength, the analytical and experimental values of the undrained shear strength will be compared.

For a given initial state of stress expressed in terms of $K_0$, a linear Mohr's failure envelope is defined by a set of strength parameter of $c'$ *and* $\phi'$, and change in excess pore pressure during undrained shear is expressed in terms of the pore pressure coefficient $A$. Pore pressure coefficient at failure $A_f$ in conventional triaxial compression and extension tests has been given by Parry and Nadarajah (1974). The following expression for the $c_u/p$ in compression tests has been given (Skempton and Bishop, 1954; Leonards, 1962).

$$\frac{c_u}{p}=\frac{c'\cos\phi'/p+\sin\phi'[K_0+A_f(1-K_0)]}{1+(2A_f-1)\sin\phi'} \quad (1)$$

In a similar manner, the expression for the $c_u/p$ in extension tests has been presented (Leonards et al., 1984),

$$\frac{c_u}{p}=\frac{c'\cos\phi'/p+\sin\phi'[1-A_f(1-K_0)]}{1+(2A_f-1)\sin\phi'} \quad (2)$$

For normally consolidated clays, the cohesion $c'$ is taken zero, then Eqs. (1) and (2) reduce to

$$\frac{c_u}{p}=\frac{\sin\phi'[K_0+A_f(1-K_0)]}{1+(2A_f-1)\sin\phi'} \quad (3)$$

$$\frac{c_u}{p}=\frac{\sin\phi'[1-A_f(1-K_0)]}{1+(2A_f-1)\sin\phi'} \quad (4)$$

In addition, the ratio of lateral to vertical effective normal stresses is equal to unity for isotropically consolidated undrained triaxial test, Eqs. (3) and (4) reduce to the same expression.

$$\frac{c_u}{p}=\frac{\sin\phi'}{1+(2A_f-1)\sin\phi'} \quad (5)$$

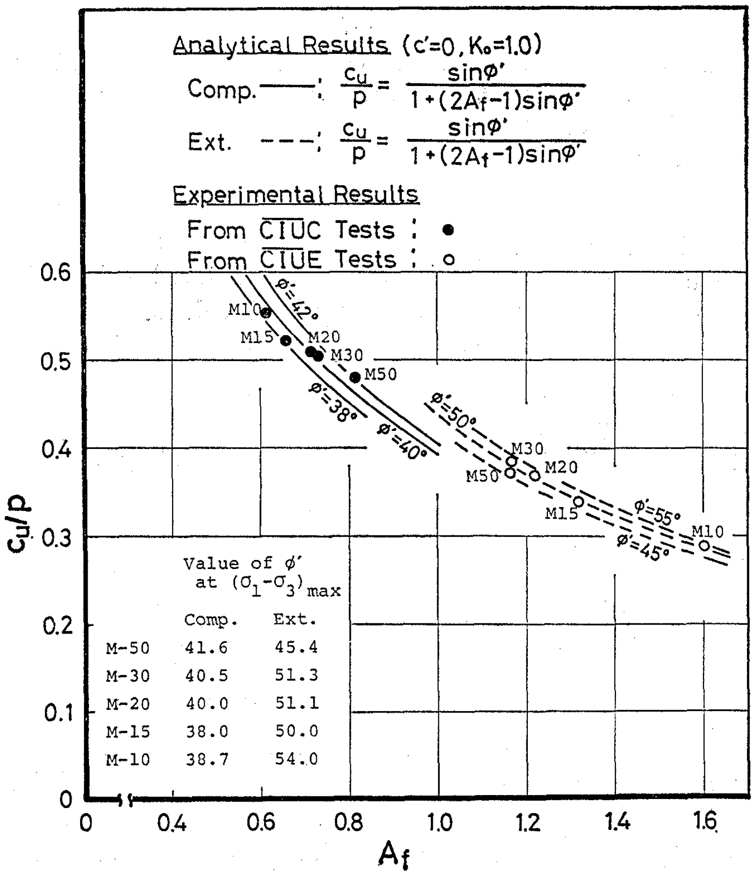


**Fig. 4. Comparison of the variation of analytical and experimental $c_u/p$ values with $A_f$ values in $\overline{\text{CIU}}$ tests for the case of the Kawasaki clay-mixture series**

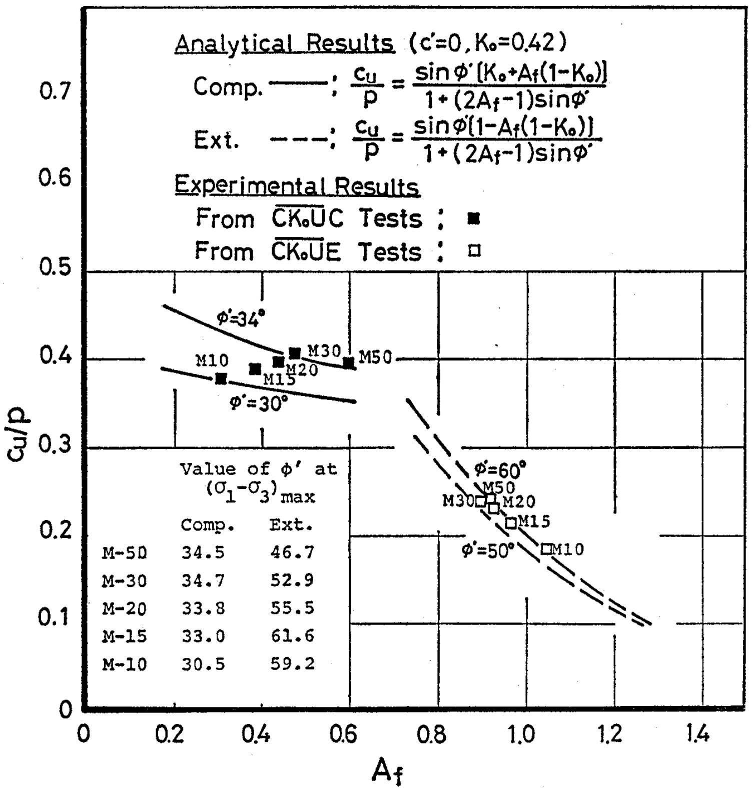


**Fig. 5. Comparison of the variation of analytical and experimental $c_u/p$ values with $A_f$ values in $\overline{CK_0U}$ tests for the case of the Kawasaki clay-mixture series**

In the above expressions, $c'$ and $\phi'$ are constants for a particular soil, but $A_f$ is dependent on $p$. In practical problems the range of $p$ is small, so it may be permissible to assume that the $A_f$ is a constant and equal to the average value over a particular pressure range (Skempton and Bishop, 1954).

Figs. 4 and 5 compare the variation of analytical and experimental values of $c_u/p$ in the $\overline{CIU}$ and $\overline{CK_0U}$ tests with $A_f$ values, for the case of the Kawasaki clay-mixture series. Figs. 6 and 7 show a similar comparison for the case of the natural marine clays. In calculating the $c_u/p$ value by the above equations, it seems logical to use the $\phi'$ and $A_f$ values at a condition of $(\sigma_1-\sigma_3)_{max}$ in view of the definition of the $c_u$. Values of $\phi'$ and $A_f$ at $(\sigma_1-\sigma_3)_{max}$, therefore, are used with $c'=0$ and the $K_0$ value is taken as the average of observed values, i. e., 0.42 for Kawasaki clay-mixture series and 0.44 for the natural marine clays. The results shown in Figs. 4 through 7 confirm a closeness of the calculated value of $c_u/p$ and the experimental values.

Leonards et al. (1984) emphasized that the strength parameters and the effect of stress path on the $c_u$, in general, could not be correlated solely to the plasticity index. It is worth investigating, therefore, the dependence of the $A_f$ and $\phi'$ at a condition of $(\sigma_1-\sigma_3)_{max}$ on the PI for each test type. Kenney (1959) reported a good indication of typical values of sin $\phi'$ for soil, i. e., there was a definite trend toward decreasing sin $\phi'$ with increasing PI, although there was considerable scatter. The $\phi'$, however, is changeable for the initial state of soil, failure criteria, consolidation and shear conditions etc.. As shown in Figs. 8 and 9, the $A_f$ and sin $\phi'$ at $(\sigma_1-\sigma_3)_{max}$ are found to be approximately in linear relationship with the PI. Table 2 shows expressions for the linear relationships illustrated in the Figs. 8

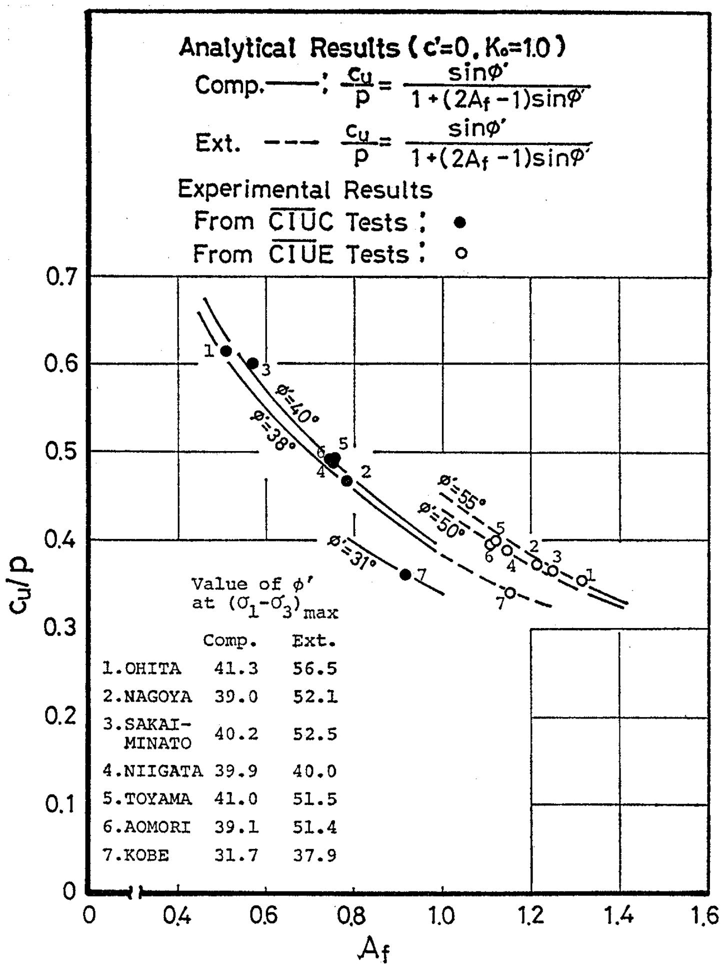


**Fig. 6. Comparison of the variation of analytical and experimental $c_u/p$ values with $A_f$ values in $\overline{CIU}$ tests for the case of the various kinds of natural marine calys**

and 9. By substituting the $A_f$ vs. PI and sin $\phi'$ vs. PI relationships of Table 2 into Eqs. (3) and (4), the $c_u/p$ values for each of the four test types can be worked out. Thus calculated values of $c_u/p$ are plotted against the PI in Fig. 10. In this calculation, a constant value of $K_0$ of 0.42 based on the Kawasaki clay-mixture series is employed irrespective of the soil type and test condition, since no appreciable relationship between the $K_0$ and PI is found.

As compared with the experimental results it may be said that the calculated value of $c_u/p$ based on the $A_f$ vs. PI and the sin $\phi'$ vs. PI relationships could be readily employed for predicting the undrained shear strength of natural marine clays as a first approximation. This result, in turn, implies the generality of the test results obtained for the Kawasaki clay-mixture series. It should be noted, however, that the present result is confined to the maine clays and also confined to the soils of the PI of less than 50. In addition, it should be stated that the present result is obtained from triaxial tests, where no sample disturbance has been given, then the result is to correspond to that for the ideal samples (Ladd and Lambe, 1963).

*Undrained Shear Strength Anisotropy*

In the present paper, anisotropy in undrained shear strength is defined as a ratio of the extensive strength to the compressive

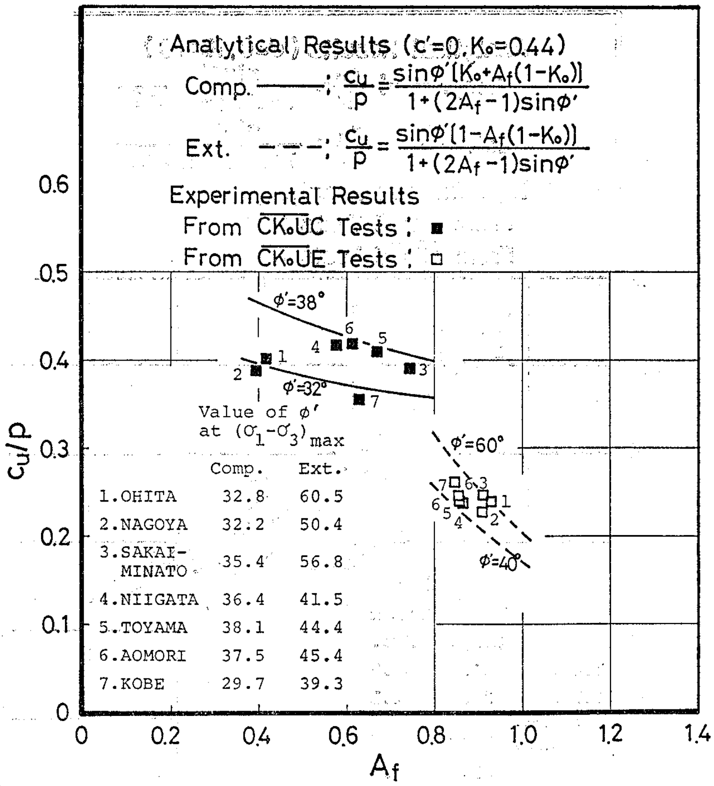


**Fig. 7. Comparison of the variation of analytical and experimental $c_u/p$ values with $A_f$ values in $\overline{CK_0U}$ tests for the case of the various kinds of natural marine clays**

**Table 2. The expressions for the linear relationships illustrated in the Figs. 8 and 9**

| Test type | PI<50 | $r$ |
|---|---|---|
| $\overline{CIUC}$ | $A_f=0.679+2.0\times10^{-3}I_p$<br>$\sin\phi'=0.629+4.2\times10^{-4}I_p$ | 0.83<br>0.92 |
| $\overline{CIUE}$ | $A_f=1.347-4.9\times10^{-3}I_p$<br>$\sin\phi'=0.789-1.6\times10^{-4}I_p$ | 0.86<br>0.85 |
| $\overline{CK_0UC}$ | $A_f=0.289+6.4\times10^{-3}I_p$<br>$\sin\phi'=0.495+2.7\times10^{-3}I_p$ | 0.93<br>0.93 |
| $\overline{CK_0UE}$ | $A_f=1.013-3.4\times10^{-3}I_p$<br>$\sin\phi'=0.929-4.8\times10^{-3}I_p$ | 0.88<br>0.91 |

$r$ : coefficient of correlation

strength. This definition has been widely used (Duncan and Seed, 1966; Berre and Bjerrum, 1973; Kinner and Ladd, 1973; Ladd et al., 1977).

Fig. 11 shows the relationship between the plasticity index and the ratio of the $c_u/p$ value of the extension test, $(c_u/p)_E$, to the $c_u/p$ value of the compression test, $(c_u/p)_C$. As shown in the figure, the strength anisotropy in terms of the ratio of extensive strength to compressive strength, decreases as the PI of soil decreases. And no appreciable difference in the tendency is found between the Kawasaki clay-mixture series and the natural marine clays. The undrained shear strength anisotropy is more marked in the $\overline{CK_0U}$ tests. This figure shows that $(c_u/p)_E$ is smaller than $(c_u/p)_C$, and the ratio of $(c_u/p)_E$ to $(c_u/p)_C$ decreases with a decrease in PI. The undrained shear strength of the $\overline{CK_0UE}$ test is about 0.6 times that of the $\overline{CK_0UC}$ test, but for soil with lower plasticity index this ratio drops to about 0.5.

Considering the state of stress in the consolidation process, the strength anisotropy of the $K_0$-consolidated specimens is to be considered more pronounced than that of the isotropically consolidated specimens. El-Sohby (1969) presented a simple measure of the degree of inherent anisotropy of soils by applying an isotropic stress to a sample in

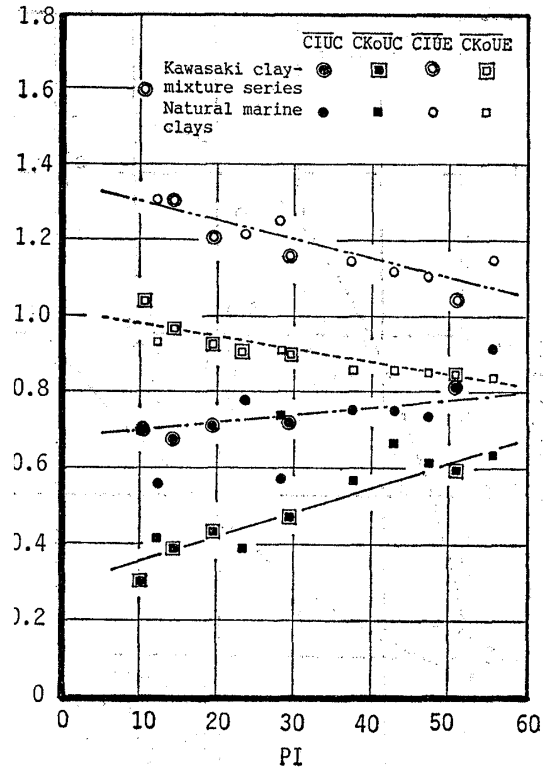


**Fig. 8. Relationship between $A_f$ values and PI**

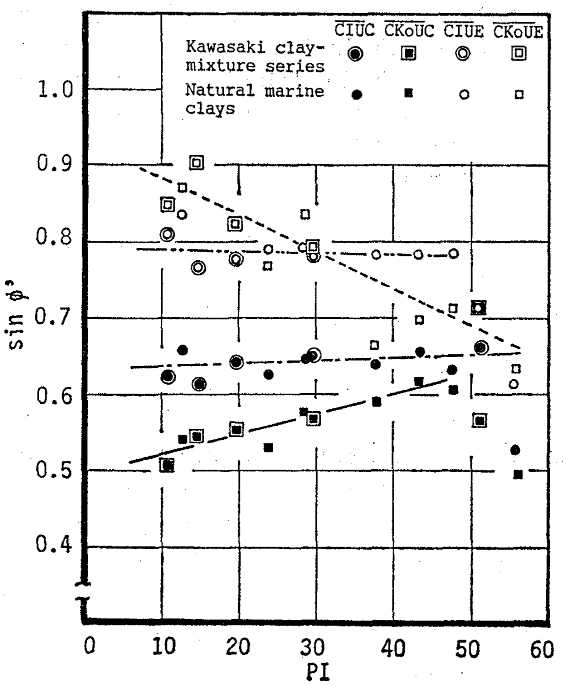


**Fig. 9. Relationship between $\sin\phi'$ values and PI**

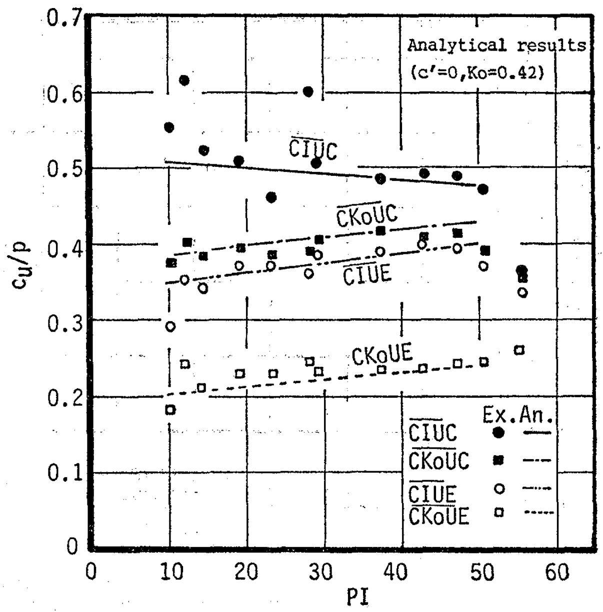


**Fig. 10. The calculated value of $c_u/p$ based on the $A_f$ vs. PI and the $\sin\phi'$ vs. PI relationships are illustrated against the PI**

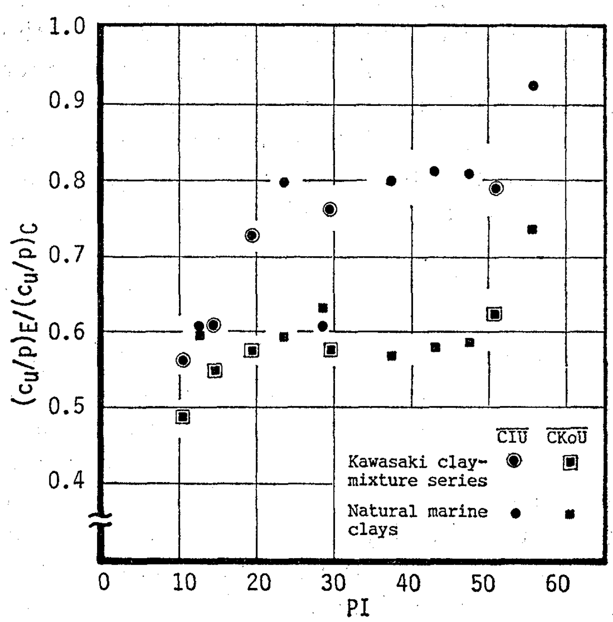


**Fig. 11. Relationship between $(c_u/p)_E/(c_u/p)_C$ and PI**

triaxial cell and measuring the axial ($\varepsilon_{ca}$) and volumetric ($\varepsilon_{cv}$) strains. For isotropic materials, $\varepsilon_{cv}/3$ must be equal to $\varepsilon_{ca}$, and any deviation from this relationship therefore implies the inherent anisotropy. The values of $\varepsilon_{ca}/(\varepsilon_{cv}/3)$ measured on either the Kawasaki clay-mixture series and the natural marine clays are plotted against the PI in Fig. 12. As seen in the figure, the $\varepsilon_{ca}/(\varepsilon_{cv}/3)$

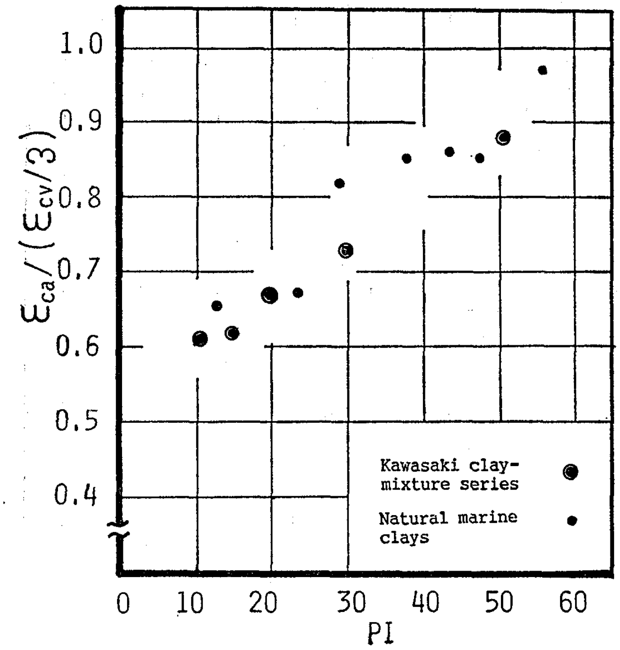


**Fig. 12. Relationship between $(\varepsilon_{ca})/(\varepsilon_{cv}/3)$ and PI**

values increase approximately linearly with an increase in the PI. This means that the soil with lower plasticity index has higher degree of inherent anisotropy. Again in that figure, no substantial difference is found between the test results of the Kawasaki clay-mixture series and the natural marine clays.

Mayne (1983) compared the undrained shear strength anisotropy of $K_0$-consolidated cohesive soils between Kawasaki clay-mixture series and data collected from other sources. He concluded that the same trend was observed for a wide range of soils and supported a relevanse of the test results on the Kawasaki clay-mixture series as shown in Fig. 13, where the present result for the natural marine clays are added.

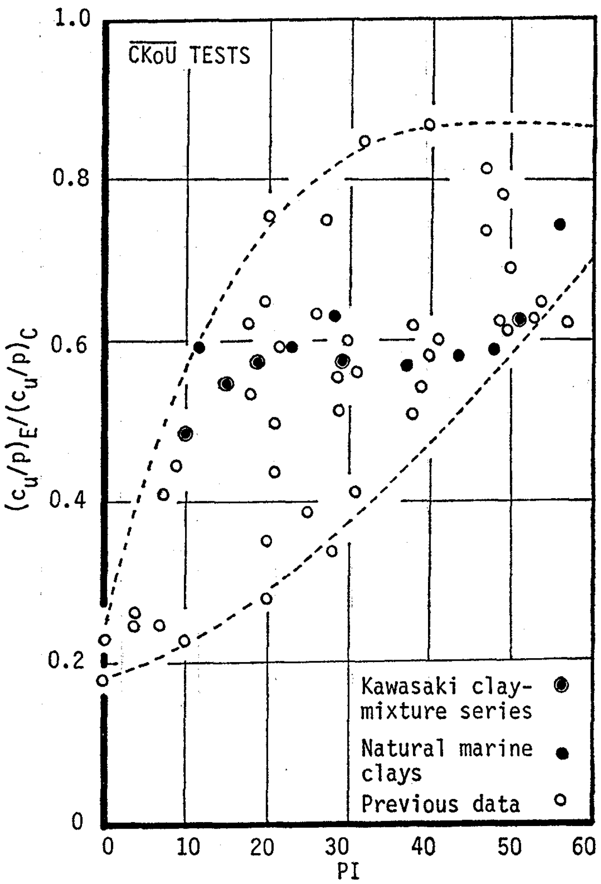


**Fig. 13. Observed relationship between strength in triaxial extension to strength in triaxial compression with PI for $K_0$-consolidated soils reported in the geotechnical literatures**

### *Summary of the Triaxial Test Results*

Triaxial test results obtained in the present experiments are summarized in Table 3, together with the results on the Kawasaki clay-mixture series. In the table, the angle of shearing resistance $\phi'$ is the value at the condition of $(\sigma_1'/\sigma_3')_{max}$, but all other parameters such as $A_f$ and $\varepsilon_f$ correspond to the condition of $(\sigma_1-\sigma_3)_{max}$, since the main topic of the present paper is the undrained shear strength.

Parry (1971) reported that localised strains often develop well before peak in extension tests and for this reason the values of $\phi_E'$ relative to $\phi_C'$ are often not observed experimentally. Where pronounced localised strains, or rupture planes, do develop before peak, the experimental observations may show $\phi_E'=\phi_C'$. Most likely explanation for the discrepancies in observed $\phi_E'$ values relative to $\phi_C'$ values is that different patterns of strains have developed in the test specimens. The strain pattern will in fact depend on factors such as sample preparation, soil type, boundary restraints and the nature of the testing equipment. In most practical problems, however, the soil is such that

**Table 3. Summary of triaxial test results**

| Materials | Test type | $c_u/p$ | $\varepsilon_f$(%) | $A_f$ | $\phi'$(°)* | $K_0$ |
|---|---|---|---|---|---|---|
| M-50 | $\overline{\mathrm{CIU}}\mathrm{C}$ | 0.472 | 14.3 | 0.807 | 41.6 | 1.00 |
| | $\overline{\mathrm{CK_0U}}\mathrm{C}$ | 0.393 | 1.3 | 0.597 | 40.5 | 0.43 |
| | $\overline{\mathrm{CIU}}\mathrm{E}$ | 0.373 | −12.4 | 1.142 | 45.2 | 1.00 |
| | $\overline{\mathrm{CK_0U}}\mathrm{E}$ | 0.246 | −22.5 | 0.856 | 46.7 | 0.43 |
| M-30 | $\overline{\mathrm{CIU}}\mathrm{C}$ | 0.505 | 14.7 | 0.720 | 40.8 | 1.00 |
| | $\overline{\mathrm{CK_0U}}\mathrm{C}$ | 0.405 | 0.87 | 0.480 | 41.0 | 0.41 |
| | $\overline{\mathrm{CIU}}\mathrm{E}$ | 0.385 | −12.6 | 1.160 | 51.4 | 1.00 |
| | $\overline{\mathrm{CK_0U}}\mathrm{E}$ | 0.233 | −18.9 | 0.900 | 52.9 | 0.43 |
| M-20 | $\overline{\mathrm{CIU}}\mathrm{C}$ | 0.507 | 16.1 | 0.710 | 40.6 | 1.00 |
| | $\overline{\mathrm{CK_0U}}\mathrm{C}$ | 0.394 | 0.62 | 0.440 | 40.1 | 0.41 |
| | $\overline{\mathrm{CIU}}\mathrm{E}$ | 0.369 | −12.1 | 1.210 | 51.9 | 1.00 |
| | $\overline{\mathrm{CK_0U}}\mathrm{E}$ | 0.227 | −18.9 | 0.930 | 55.5 | 0.43 |
| M-15 | $\overline{\mathrm{CIU}}\mathrm{C}$ | 0.522 | 17.2 | 0.650 | 38.7 | 1.00 |
| | $\overline{\mathrm{CK_0U}}\mathrm{C}$ | 0.385 | 0.44 | 0.390 | 40.1 | 0.40 |
| | $\overline{\mathrm{CIU}}\mathrm{E}$ | 0.341 | −13.8 | 1.310 | 50.6 | 1.00 |
| | $\overline{\mathrm{CK_0U}}\mathrm{E}$ | 0.211 | −19.0 | 0.970 | 61.6 | 0.42 |
| M-10 | $\overline{\mathrm{CIU}}\mathrm{C}$ | 0.553 | 19.6 | 0.610 | 39.2 | 1.00 |
| | $\overline{\mathrm{CK_0U}}\mathrm{C}$ | 0.375 | 0.26 | 0.310 | 39.3 | 0.42 |
| | $\overline{\mathrm{CIU}}\mathrm{E}$ | 0.291 | −12.9 | 1.600 | 55.3 | 1.00 |
| | $\overline{\mathrm{CK_0U}}\mathrm{E}$ | 0.182 | −20.6 | 1.050 | 59.2 | 0.44 |
| Ohita Clay | $\overline{\mathrm{CIU}}\mathrm{C}$ | 0.615 | 13.6 | 0.51 | 41.2 | 1.00 |
| | $\overline{\mathrm{CK_0U}}\mathrm{C}$ | 0.401 | 0.43 | 0.42 | 41.0 | 0.43 |
| | $\overline{\mathrm{CIU}}\mathrm{E}$ | 0.353 | −14.3 | 1.32 | 56.5 | 1.00 |
| | $\overline{\mathrm{CK_0U}}\mathrm{E}$ | 0.238 | −18.3 | 0.93 | 62.2 | 0.44 |
| Nagoya Clay | $\overline{\mathrm{CIU}}\mathrm{C}$ | 0.465 | 14.7 | 0.79 | 39.3 | 1.00 |
| | $\overline{\mathrm{CK_0U}}\mathrm{C}$ | 0.386 | 0.39 | 0.40 | 38.9 | 0.41 |
| | $\overline{\mathrm{CIU}}\mathrm{E}$ | 0.370 | −12.5 | 1.22 | 52.1 | 1.00 |
| | $\overline{\mathrm{CK_0U}}\mathrm{E}$ | 0.229 | −20.0 | 0.91 | 50.5 | 0.43 |
| Sakaiminato Clay | $\overline{\mathrm{CIU}}\mathrm{C}$ | 0.599 | 14.5 | 0.57 | 40.5 | 1.00 |
| | $\overline{\mathrm{CK_0U}}\mathrm{C}$ | 0.388 | 1.38 | 0.75 | 41.3 | 0.44 |
| | $\overline{\mathrm{CIU}}\mathrm{E}$ | 0.365 | −14.2 | 1.25 | 52.7 | 1.00 |
| | $\overline{\mathrm{CK_0U}}\mathrm{E}$ | 0.245 | −22.0 | 0.92 | 57.5 | 0.45 |
| Niigata Clay | $\overline{\mathrm{CIU}}\mathrm{C}$ | 0.486 | 17.3 | 0.75 | 40.0 | 1.00 |
| | $\overline{\mathrm{CK_0U}}\mathrm{C}$ | 0.416 | 3.20 | 0.58 | 40.4 | 0.46 |
| | $\overline{\mathrm{CIU}}\mathrm{E}$ | 0.388 | −13.3 | 1.15 | 51.7 | 1.00 |
| | $\overline{\mathrm{CK_0U}}\mathrm{E}$ | 0.236 | −19.1 | 0.87 | 41.0 | 0.46 |
| Toyama Clay | $\overline{\mathrm{CIU}}\mathrm{C}$ | 0.492 | 16.0 | 0.76 | 41.1 | 1.00 |
| | $\overline{\mathrm{CK_0U}}\mathrm{C}$ | 0.408 | 4.52 | 0.67 | 40.9 | 0.42 |
| | $\overline{\mathrm{CIU}}\mathrm{E}$ | 0.399 | −13.2 | 1.12 | 51.5 | 1.00 |
| | $\overline{\mathrm{CK_0U}}\mathrm{E}$ | 0.236 | −19.0 | 0.86 | 45.2 | 0.43 |
| Aomori Clay | $\overline{\mathrm{CIU}}\mathrm{C}$ | 0.489 | 15.1 | 0.75 | 40.2 | 1.00 |
| | $\overline{\mathrm{CK_0U}}\mathrm{C}$ | 0.416 | 2.23 | 0.62 | 40.5 | 0.43 |
| | $\overline{\mathrm{CIU}}\mathrm{E}$ | 0.395 | −11.2 | 1.11 | 51.7 | 1.00 |
| | $\overline{\mathrm{CK_0U}}\mathrm{E}$ | 0.243 | −22.6 | 0.86 | 45.8 | 0.43 |
| Kobe Clay | $\overline{\mathrm{CIU}}\mathrm{C}$ | 0.364 | 13.0 | 0.92 | 31.9 | 1.00 |
| | $\overline{\mathrm{CK_0U}}\mathrm{C}$ | 0.354 | 1.61 | 0.63 | 33.1 | 0.48 |
| | $\overline{\mathrm{CIU}}\mathrm{E}$ | 0.336 | −14.4 | 1.15 | 37.8 | 1.00 |
| | $\overline{\mathrm{CK_0U}}\mathrm{E}$ | 0.260 | −24.3 | 0.84 | 39.9 | 0.51 |

*: values at the condition of $(\sigma_1'/\sigma_3')_{max}$

pronounced localised strains or rupture zones will develop and the higher $\phi_E'$ values could not be relied upon in design studies.

## CONCLUSIONS

The following conclusions were obtained :

i) Relationship between the undrained shear strength characteristics and the PI obtained for a series of triaxial tests on eight kinds on natural marine clays in Japan was found practically identical to that previously obtained for the artificially mixed soils of the Kawasaki clay-mixture series.

ii) Angle of shearing resistance $\phi'$ and pore pressure coefficient at failure $A_f$ were found to be approximately in linear relation with the PI, corresponding to the type of triaxial tests.

iii) Calculated values of $c_u/p$ based on the $A_f$ vs. PI and the sin $\phi'$ vs. PI relationship compared well to the value measured in each of four different types of triaxial tests.

iv) Relationship between the $c_u/p$ and the PI, and that between the undrained shear strength anisotropy and the PI presented in this paper may be used, as the first approximation, for some of marine clays in coastal area of Japan.

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## NOTATION

$A_f$=pore pressure coefficient at failure

$c_u/p$=ratio of the undrained shear strength to the vertical effective consolidation pressure

$\overline{\text{CIU}}$C=isotropically consolidated undrained compression test

$\overline{\text{CK}_0\text{U}}$C=$K_0$-consolidated undrained compression test

$\overline{\text{CIU}}$E=isotropically consolidated undrained extension test

$\overline{\text{CK}_0\text{U}}$E=$K_0$-consolidated undrained extension test

$K_0$=coefficient of earth pressure at rest

$M$=letter $M$ is named after their plasticity index in Kawasaki clay-mixture series

$\varepsilon_f$=axial strain at failure

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