Slaking behavior of saturated mudstones

Le délitement de la boue saturée

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ABSTRACT : The slaking behavior of the crushed mudstones, being saturated with water, can be idealized as the softening behavior of the heavily overconsolidated clays due to swelling above the critical state line in $q \sim p'$ space during shearing. To substantiate this, one-dimensional compression tests and triaxial compression tests for the crushed mudstones are carried out. The one-dimensional compression tests show that the normal consolidation line (N.C.L.) in e $\sim \sigma_v$ space, which is derived from the results of one-dimensional compression tests on remoulded mudstones represents a demarcation line between the states of loose and dense crushed mudstones. To avoid the occurrence of large settlements of an embankment made of mudstones, it is necessary to compact the mudstones to make their void ratio below the N.C.L. The crushed mudstones, packed densely, exhibit progressive failure due to slaking which is caused by the high contact shear stress between pebbles' boundaries.

RESUME : Le délitement de la boue broyée, étant saturée dans léeau, peut étre apperçu comme un comportement adouci de la boue sursolididifiée du au gonflement au dessus de la ligne d'état critique dans l'espace $q \sim p'$ lors de l'écoulement. Afin de confirmer cette hypothése, les examens de compression unie-dimensionnelle et de compression triaxiale de la boue broyée ont été réalisés. Les résultats des examens de compression unie-dimensionnelle montrent que la ligne normale de la consolidation (L.N.C.) dans léespace e $\sim \sigma_v$, représente la ligne de limite entre les états dispersés et denses de la boue broyée. Pour éviter la présence de précipités de la boue, il est nécessaire de compresser la boue pour avoir un ratio mort au dessous de L.N.C. La boue broyée, comprimée trés dense, exhibe une faillite progressive du au délitement qui est causé par un fort contact de la tension de surface des limites des cailloux.

1 INTRODUCTION

The most of the coastal areas and some mountainous areas in Japan are extensively covered by the Tertiary mudstone deposits. The major construction works, that are undertaken in those areas, involving deep excavations, cut and fills of mountains, bring forth the Tertiary mudstones as an excess material to be disposed. The piled up mudstones, when becomes saturated with water, easily lead to slaking and as a result the strength of them diminishes gradually. Due to this problematic nature, they are disposed as a waste material.

However, the environmental concerns suggest the possible usage of such mudstones as a potential construction material for earth structures. In order to use such material for constructions effectively, it is essential to understand the slaking mechanism based on established principles.

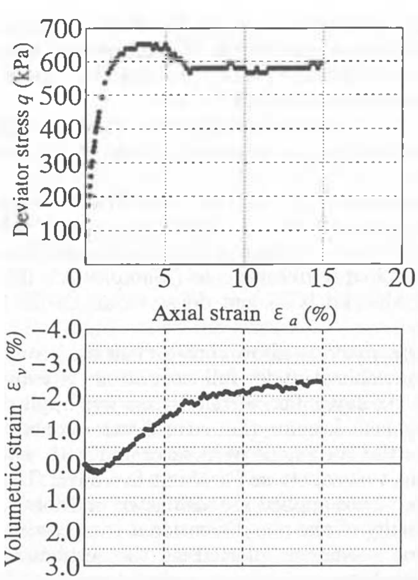


Fig.1 Deviator stress - volumetric strain - axial strain relationship of overconsolidated clay

The main objective of this study is, therefore, to explain the mechanism of slaking behavior, which particularly occur with water, of the crushed Tertiary mudstones based on the Critical State Soil Mechanics. In order to interpret the mechanism of slaking behavior in terms of elasto-plastic mechanics, saturated mudstone is idealized as the heavily overconsolidated clay.

Fig.1 shows the drained shear behavior of the heavily overconsolidated KAWASAKI clay in triaxial compression test. This clay shows the softening due to swelling during shearing after initial compression. During this softening process, clay reaches the normally consolidated state after the peak of deviator stress q. As the softening process progresses further with increase in water content, the clay become more soft like the remoulded clay. This softening process can be idealized as the slaking behavior.

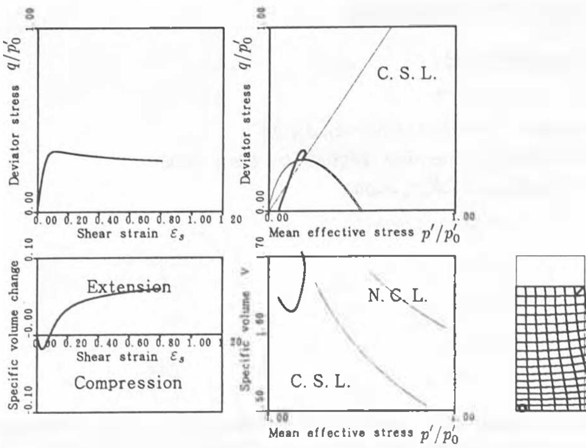
There are two load histories through which the overconsolidated clay can be brought to normally consolidated clay. One is to apply a large stress to the overconsolidated clay and the other is to shear the clay with supplying water. The slaking behavior is the latter process.

This paper basically consists of the following,

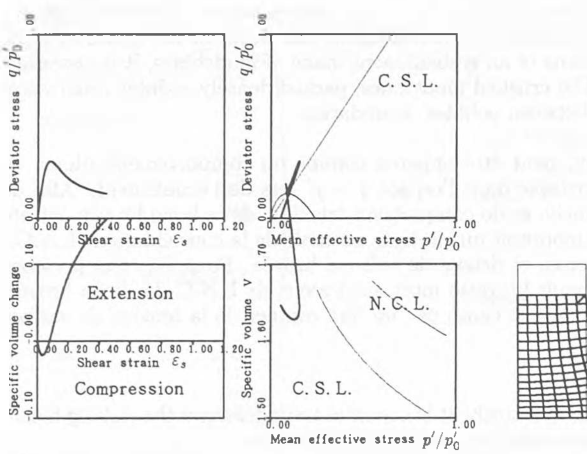
1. The proposed mechanism of slaking as stated above is verified through the numerical computation on the heavily overconsolidated clay.
2. The results of both one-dimensional compression tests and the triaxial compression tests for the crushed mudstones are explained based on the above mechanism.
3. Considering the worst case for an embankment made of the Tertiary mudstones, a concept for long term stability check is presented.

2 SOFTENING BEHAVIOR DUE TO SWELLING FOR HEAVILY OVERCONSOLIDATED CLAYS, A SIMULATION OF SLAKING

In order to grasp the slaking phenomenon based on elasto-plas-



(a) center of specimen



(b)outermost element from center of specimen

Fig.2 Element-wise behavior under drained conditions

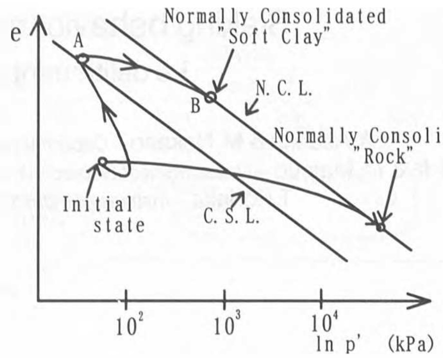


Fig.3 Two ways of reaching normally consolidated state from overconsolidated state

Table 1 Index properties of TOKONAME mudstone

Natural Water Content(%)	Specific Gravity	Water Limit(%)	Plastic Limit(%)	Dry Density (kgf/cm^3)
24.35	2.69	43.63	30.33	1.60

Table 2 Test condition and initial condition of mudstone specimen

Test	Loading (kPa)	Dry Weight(g)	Initial Void Ratio	Initial Grain Size(mm)	Final Settlement (cm)
A3	296	2022.6	1.35	19.1～37.5	0.25
B3	296	1970.0	1.41	19.1～37.5	0.63
C3	296	2150.6	1.54	19.1～37.5	0.59
D3	296	1854.3	1.56	4.76～9.50	0.60
E3	296	2746.7	0.73	19.1～37.5	0.00
H5	490	1937.7	1.45	19.1～37.5	0.41
I5	490	2056.7	1.31	19.1～37.5	0.83
J5	490	1680.4	1.83	4.76～9.50	0.94
M6	637	1963.6	1.42	4.76～9.50	0.66
N7	686	2038.3	1.33	19.1～37.5	0.49

tic mechanics, firstly a numerical investigation is carried out for heavily overconsolidated clays.

To describe the behavior of heavily overconsolidated clay, the subloading surface concept first developed by Hashigushi and Ueno(1977) was applied to the original Cam-Clay model(Schofield and Wroth, 1968). The conventional drained triaxial compression test with the constant cell pressure of 98kPa for the overconsolidated KAWASAKI clay of OCR=18, was numerically simulated using the subloading surface Cam-Clay model. For details of the simulation, refer Asaoka, Nakano and Noda(1997).

Figs.2(a) and (b) show the results of the numerical simulation on the element-wise behavior, in which the locations of the soil elements are also indicated.

Both of the elements show the hardening behavior above the critical state line from the early stages of shearing. Subsequently, softening with swelling occurs above the critical state line. These plots(Figs.2(a),(b)) further show that the elements have already reached the normally consolidated state. Especially, the element shown in Fig.2(b) swells with water to a greater extent than the element shown in Fig.2(a) and make a "comeback" to normally consolidated state, of which preconsolidation pressure is about one fourth of its initial preconsolidation pressure of 1764kPa.

Within the context of swelling behavior of heavily overconsolidated clays as illustrated above, the slaking behavior of saturated mudstones can be idealized as the process of reaching normally consolidated states from heavily overconsolidated states by swelling during shearing as shown in Fig.3. If the clay specimen after failure (point A in Fig.3) is compressed, the clay can reach the normally consolidated line(point B in Fig.3) : *i.e.* annealable behavior of the clay (Asaoka, Nakano and Noda,1995). In fact, slaking phenomenon is an alternative process that the heavily overconsolidated clays can undergo to reach the normally consolidated state without being subject to large compressive stresses.

3 ONE-DIMENSIONAL COMPRESSION PROPERTIES OF CRUSHED TOKONAME MUDSTONES DUE TO "DOUBLE STRUCTURE"

The one-dimensional compression tests under full saturation with water were carried out to examine the compression characteristics of the crushed saturated TOKONAME mudstones. The TOKONAME mudstones, which lie in TOKONAME Tertiary deposits, were excavated from 50m below the sea level. The index properties of mudstones are shown in Table 1. The degree of saturation Sr was almost 100%. The mechanical stabilization method for the crushed mudstones and the experimental procedure adopted are as follows.

1. Firstly, the TOKONAME mudstones were crushed and their particle size was made to be between 19.1mm～37.5mm or 4.76mm～9.50mm.

2. The crushed mudstones were put into the CBR mold by compaction and then the specimens of 150mm in diameter and 100mm in height were made. As shown in Table 2, two methods of compaction were applied to obtain loose (Samples-A3, B3, C3, D3, H5, I5, J5, M6 and N7) and dense (Sample-E3) specimens.

3. For the specimens made as above, one-dimensional compression tests were carried out under full saturation. A complete test took about six days. The vertical stress was applied to the mudstone specimens keeping the natural water content for one day and then the specimens were saturated with water under the constant vertical stress for about five days. The magnitudes of vertical stress applied are also listed in Table 2.

Fig.4 shows the results of the one-dimensional compression test on the mudstone specimens plotted as the settlement against time. The mudstone specimens packed loosely with large void ratio; Samples-A3 to D3, when saturated with water, showed an immediate compression. On the other hand

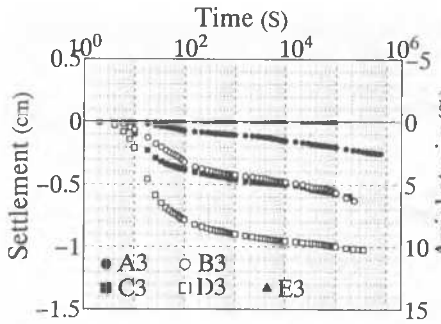


Fig.4 Settlement behavior of crushed mudstones during compression tests (σ_v =296kPa)

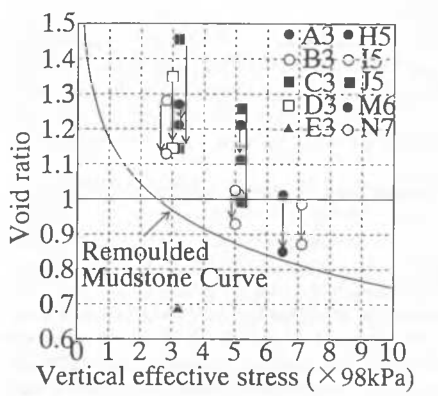


Fig.5 Change of void ratio during compression tests

Sample-E3 packed densely with small void ratio, swelled instead of compression.

Fig.5 shows the same results replotted as the relationship between void ratio e and vertical compressive stress σ_v for all mudstone specimens. The initial states of loose mudstone specimens lie above the normal consolidation line (N.C.L.). The dense mudstone specimen E3 lies below the N.C.L. The N.C.L. in this figure is plotted from the results of one-dimensional compression test on fully remoulded mudstone specimen. As shown in Fig.5, the state of loose mudstones which originally lying above the N.C.L. moves toward the N.C.L. during testing. In contrast, the state of the dense mudstones originally lying below the N.C.L. does not move. Fig.5 further shows that the N.C.L. is a demarcation line of the states mudstone specimens for compression to be possible or not. If the applied loading for loose mudstones is known, the maximum settlement can be predicted from the void ratio on the N.C.L. This settlement behavior in one-dimensional compression test further illustrates that how dense the crushed mudstones to be packed in the field to reduce its compression.

The N.C.L. in e $\sim$ σ_v plot generally represents the boundary between possible states and impossible states. In contrary to this, as shown in Fig.5, the state of loose mudstones lie above the N.C.L. The reason why the state can lie above the N.C.L. is that the assembly of the mudstone pebbles form of a "**double structure**" , *i.e.*: mudstone specimen consists of pebbles and void spaces. The pebble itself consists of soil grains and voids. Fig.6 shows the existence of mudstones can be of three states; the stiff solid state, loosely or densely packed state of the crushed mudstones and the remoulded normally consolidated state. The state of crushed mudstones is the assembly of solid mudstone pebbles in fully saturated condition. Even though the crushed mudstone specimen is loaded one-dimensionally, the excess concentrated shear stresses occur at the contact surfaces between pebbles' boundaries and the pebbles themselves can soften due to swelling by shearing as observed in overconsolidated clays. The specimen, however, compresses as a whole due to porewater squeezing from voids among pebbles, which is higher than swelling of pebbles themselves.

Fig.7 shows the results of grain size analysis after the test

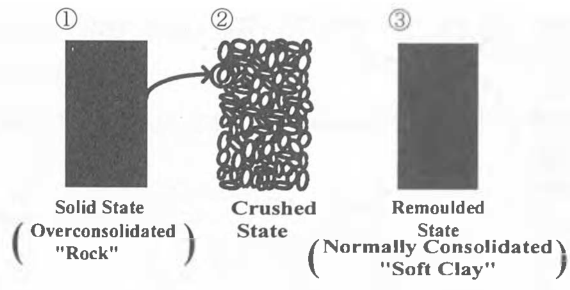


Fig.6 Three states of mudstones

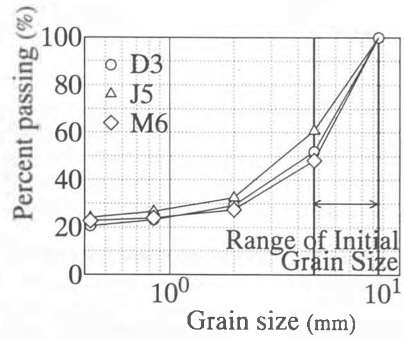


Fig.7 Grain size analysis after compression tests

plotted as a grading curve. For the samples D3, J5 and M6, which had initial grain sizes of 19.1 to 37.5mm, the percent passing of the mudstone pebbles whose grain size is less than 2mm, is more than 20 % due to slaking during one-dimensional compression test.

4 PROGRESSIVE FAILURE OF TOKONAME MUDSTONE WITH SLAKING

In order to examine the shear characteristics of crushed mudstone specimen which forms a **"double structure"** , both the strain controlled and the load controlled drained triaxial compression tests for the crushed mudstone specimen, which are packed densely, are carried out.

4.1 *Preparation of mudstone specimens*

The mudstones were crushed to reach the particle diameter between 2.0 $\sim$ 4.75mm. These were then compacted by 9.8 N weight rammer, with a drop height of 150mm and 55 drops per layer. After compacting thirteen layers, the specimens of 50mm in diameter and 100mm in height were recovered. The specimens were filled with water and de-aired by vacuum pump to make full saturation and then frozen to make easy for setting. The frozen specimen was set in the triaxial test apparatus. The specimens had its void ratio less than 0.96. The specimen was packed dense enough such that no compression if it was in one-dimensional compression test as shown in Fig.5.

4.2 *Strain controlled drained triaxial compression test*

The mudstone specimens were isotropically compressed to 98kPa, in which the cell pressure was 294kPa, and the back pressure was 196kPa. The specimens were then subjected to the strain controlled drained compression test with the strain rate of 7.8×10^{-3}%/min, in which the cell pressure was held constant.

The relationships between q $\sim$ ε_a and ε_v $\sim$ ε_a for both the crushed mudstone specimen and the overconsolidated KAWASAKI clay specimen are shown in Fig.8. The symbols ○,● in Fig.8 stand for mudstone and overconsolidated clay, respectively. The $q \sim \varepsilon_a$ curve, which shows a clear peak of q

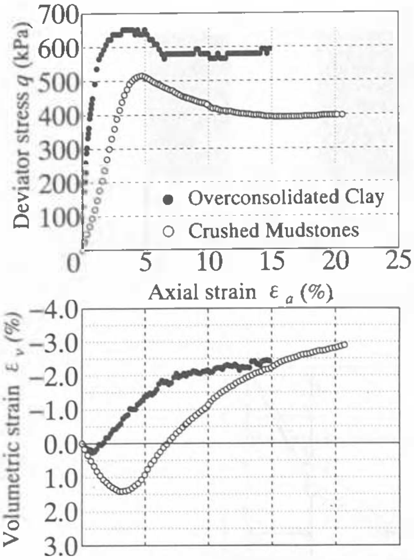


Fig.8 Drained triaxial tests of crushed mudstones and remoulded overconsolidated clays

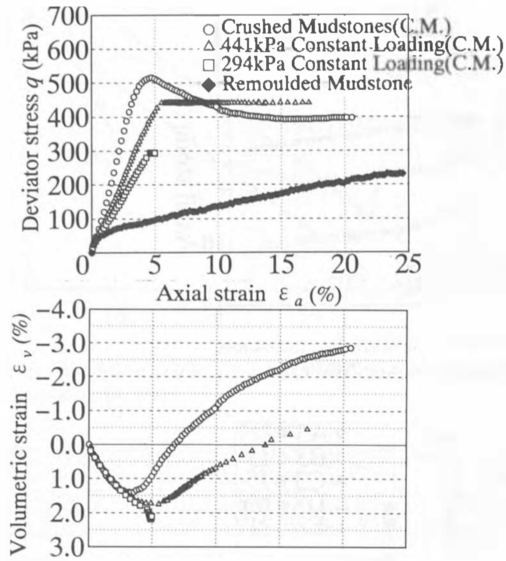


Fig.9 Drained triaxial tests of crushed and remoulded mudstones

for the mudstone specimen, is similar to the curve for the overconsolidated clay specimen. The $\varepsilon_v \sim \varepsilon_a$ curve for the mudstone, which initially compresses and subsequently dilates, is also similar to the overconsolidated clay. However, the significant difference between the behavior of two specimens is that for the mudstone hardening occurs with drainage and softening occurs with swelling, while for the overconsolidated clay hardening still occurs even with swelling. The KAWASAKI clay shows the typical behavior of heavily overconsolidated clays. The reason for this difference is that the mudstone specimen forms a **"double structure"** which is constituted of the mudstone pebbles. Since the pebbles are densely packed, the dilatancy characteristics seem to be the same as the overconsolidated clay or dense sand. However, since the mudstone pebbles swell with water and then transform from overconsolidated state to normally consolidated state due to slaking during shearing, the mudstone does not show hardening with swelling.

4.3 *Load controlled drained test*

The load was applied to the specimen at a constant rate and then held constant under a constant cell pressure. The test was carried out with two constant load magnitudes, one was 441kPa between "peak strength" and "residual strength" of the strain controlled drained test for the crushed mudstones, and the other was 294kPa between "residual strength" for the crushed mudstones and the drained strength of the strain controlled drained test for remoulded mudstone.

Fig.9 shows the results of the load controlled drained compression tests and the strain controlled drained compression tests, on mudstone specimens plotted as the deviator stress q against axial strain ε_a and volumetric strain ε_v against axial strain ε_a. Under the constant load of 441kPa, the axial strain continued to increase with swelling and the specimen failed like creep failure for natural clays. Under the constant load of 294kPa, neither the axial strain nor the volumetric strain changed and the specimen did not reach failure. Such a failure behavior is different from that observed for the saturated dense sand; *i.e.* if the dense sand specimen is kept under the constant loading which is between "peak strength" and "residual strength" , the sand specimen will not reach failure.

The reason for the failure of mudstone specimen under the constant load of 441kPa is that large shear stress concentration at contact surfaces between pebbles' boundaries cause slaking.

This suggests that for the long term stability of an embankment made of crushed mudstones, the use of "peak strength" or "residual strength" given by the triaxial test on crushed mudstones will not guarantee the safety of embankment.

5 CONCLUSIONS

A series of one-dimensional compression tests and triaxial compression tests on the crushed TOKONAME mudstones are carried out to investigate the slaking phenomenon. The experimental findings are used to suggest some design guidelines when the mudstones are used as construction materials for embankments. The following conclusions are drawn through this study:

1. The slaking behavior of mudstones, being saturated with water, can be idealized as the softening behavior of the heavily overconsolidated clays, in which the heavily overconsolidated clays change to normally consolidated clays by swelling with shearing.

2. The N.C.L., which is determined from the remoulded mudstone, represents a demarcation line between the states of loose and dense crushed mudstones in e $\sim \sigma_v$ space. The states of loose mudstone specimens, which lie above the N.C.L., continue to move down towards the N.C.L. To the contrary, the states of dense mudstone specimens show no change. This is due to the fact that the crushed mudstones form a **"double structure"** . To avoid the occurrence of large settlements of an embankment made of mudstones, it is necessary to compact the mudstones to make their void ratio below the N.C.L.

3. Densely packed crushed mudstones still exhibit progressive failure. The reason for this is that the high shear stresses at contact surfaces between pebbles' boundaries give rise to softening with swelling; *i.e.* slaking. Therefore, for long term stability of an embankment it is unsafe to use the "peak strength" or the "residual strength" of the crushed mudstones under drained condition observed in laboratory.

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