Swelling Characteristics of Bentonite

Takeshi ITO and Susumu ARAKAWA*

(Received on 31 Octorber, 1975)

1. Introduction

Foundation design[1] in expansive soils and rocks, such as bentonite, is complicated and impeded by the fact that they have remarkable swelling behavior. In a large number of tunnels and the works of resources development[2], many engineers have already been confronted with this difficulty for a long time. The problems on structural damages have included heavings, cracking and breakup of pavements, building foundations, slab-on-grade members, and channel and reservior linings. The main reason of them has been concluded as soils and rocks of the foundation containing much swelling clay minerals especially sodium montmorillonite. This clay mineral is contained in great quantities in bentonite, and which plays an important role in the behavior of such soils and rocks. To grasp the swelling behavior, several basic studies using bentonite powder had yet been carried out in the laboratory[3),4]. However, there is no report of swelling characteristics of natural bentonite rock up to date. Therefore, a basic study was made on a natural bentonite rock. This is a pioneering attempt to study the swelling behavior of natural bentonite rock. Through the experiments, numerical basic information on bentonite rock were obtained. The purpose of this Paper is to present the results of a study on the relationship of swelling properties of bentonite rock and its powder.

2. Basis Concepts of Swelling Behavior of Rock and Soil

According to a classification[5], swelling is explained as the phenomena of (1) Expansion, and (2) Plastic flow. Also mechanically it is explained as (1) Volume changes by water adosorption, (2) Chemical changes (clay forming), (3)Stress relaxation of ground pressure and its plastic deformation, and (4) Relaxation either residual stress (latent energy) or preconsolidation pressure stored in the soil when it deposited.

Schofield and Warkentin[6] suggest that swelling may be occurred when the condition mentioned below is kept.

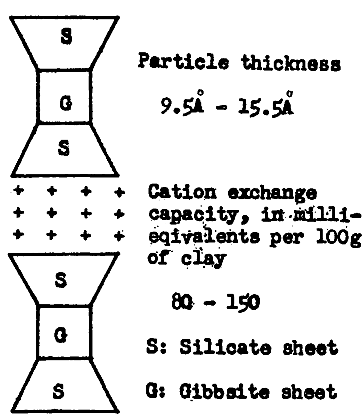

Fig 1 Schematic Structure of Montmorilllonite

Electrolitic energy of clay particles > Electro-

* Mining College, Akita University

litic energy overlapping of diffuse ion double layers between adjacent clay particles + External pressure.

There is no exact distinction among them, and there is also a standard lack of diffinition and measuring apparatus concerning the swelling pressure, for instance, like JIS.

However, in general, the degree of swelling is determined physically by the volume change of soil or rock based on the clay mineral conditions: cappillary and osmotic suction pressure, water fixation by polar adsorption, osmotic imbibition, and surface tension focres. The value of swell potential depends on the following factors that influence the volume change.

(1) Mineral type and Amount, (2) Initial density, (3) Load conditions (surcharge loads), (4) Structure (clay particle skeleton, disturbed or undisturbed), (5) Initial water content (degree of saturation), (6) Kind of pore fluid (pH etc.), (7) Time, and (8) Temperature.

It is difficult to satify all these factors in one test, thus the natural bentonite rock was selected for the present study because it keeps in general constant conditions.

3. Experimental

3.1 Samples

Samples used in this investigation were collected from deep working face of Kawamukai adit level, Kunimine Bentonite Mine, Yamagata prefecture. Two types of samples were given for tests as shown in the followings.

(1) Bentonite rock ; Taken by coring from the rock blocks.

(2) Powder ; Collected 74 micron under from crushed bentonite fractions.

For determining the physical and mechanical properties, raw bentonite rocks were tested. Analyses are summarized in Table－1.

Table－1 Properties of Bentonite rock

G_s	L.L.	P.I.	W_i	e_0	S_i	γ_t	S_c
2.71	584	541	27.4	0.75	99.0	1.98	15.0
g/cm³	%	%	%	—	%	g/cm³	kg/cm²

G_s: Specific gravity, L.L.: Liquid limit, P.I.: Plastic index, W_i: Initial water content, e_0: Initial void ratio, S_i: Initial saturation, γ_t: Wet density, and S_c: Uniaxial compressive strength.

The samples are believed to contain over 70% montmorillonite and amurphous material, with the remainder mostly quartz. The montmorillonite in the samples is considered as sodium type which shows high swelling pressure. Swelling depends on the crystal lattice structure, the structure of the clay mass, and the cation exchange capacity of the materials. In general, swelling of montmorillonite is caused by the either adsorption of the pore fluid or formation of diffuse double layers as shown in Fig. 1. Such wide structure which is enable to adsorb the water is not seen in the other clay minerals.

3. 2 Apparatus and Testing Procedures

Fig. 2 shows the outline of an oedometer for measuring swelling properties.

One of the cells is 2 cm high and 6 cm in diameter, and the other is 3 cm high and 3 cm in diameter. The former was used only for rock and the latter was used for both rock and powder. The wall of the cell is rigid, so the swelling pressure and strain were measured in one dimensional. In this study, volume was not put back always to zero when swelling occurred. This is a main different point from other swelling tests. Because of the most of free faces in the mine are unconfined although the lateral is confined. Therefore, internal force of the sample, i. e., swelling pressure is always pushing the sample down as a surcharge load. Measuring was ended when the swelling rate got to 0.01 mm / 24 hrs.

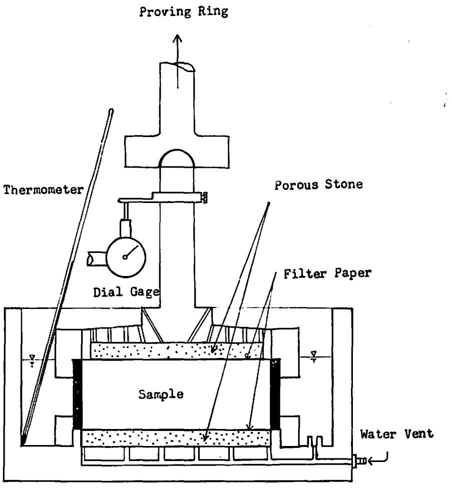

Fig. 2 Skech Digram of Oedometer

In the other test, bentonite rock was preconsolidated in the consolidometer with a weight of 12.8 kg / cm^2 for 24 hours. Then the weight was decreased in the same way of the soil consolidation test.

3. 3 Measurement Results

Fig. 3 shows an example of a representative relationship of swelling pressure vs. time of raw bentonite rock. It is seen that the rate of increase of swelling pressure is more rapidly in the initial stage, and then the curve becomes gradually slow rise as time passes. It takes a long time until the swelling pressure reaches to a certain steady state.

Fig. 4 shows the one demensional deformation vs. time. In the case of powder swelling pressure appeared highly more than that of rock. The curves of powder came out steps wisely in the vicinity of 10^4 minutes, while it is smooth to rock. To investigate this more clearly, after each test, the final water content of the sample was measured.

Fig. 5 shows the above main results. In the figure, dotted line shows the case of water vent only from the bottom of the sample. On the other hand, in case of water vent from the both ends, less water content appeared in the center part. From this, it is seen that the moiture of the sample would not distribute uniformly in it.

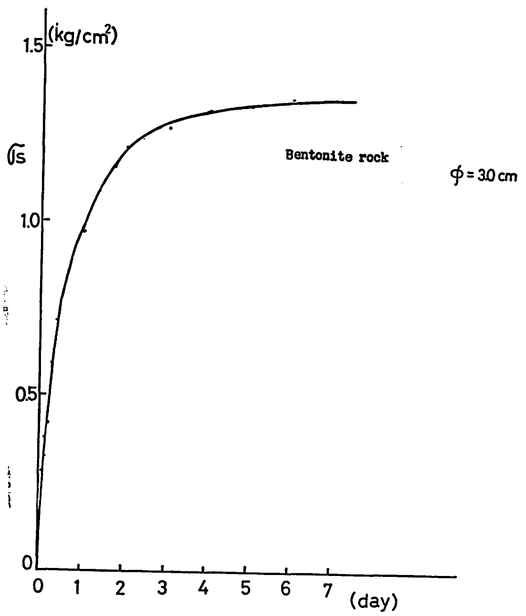

Fig. 3 Swelling pressure Vs. Time

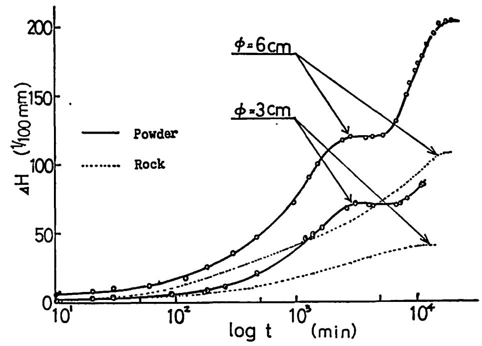

Fig. 4 Swelling deformation Vs. Time

Fig. 6 shows the relationship of swelling pressure vs. strain in log-log. graph. In swell tests, swelling pressure is very much dependent in magnitude on the initial water content of the sample. That is, low moisture brings about higher swelling activity.

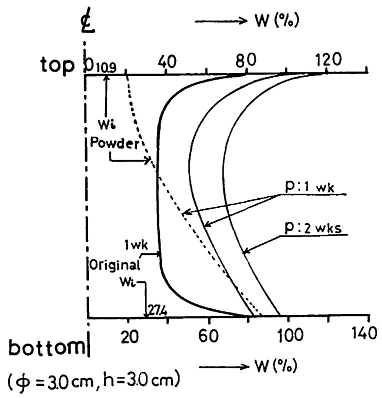

Fig. 5 Water content distribution

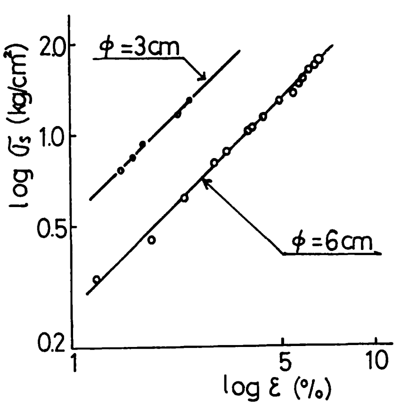

Fig 6. $\sigma_s \sim \varepsilon$ Relationships

Fig. 7 shows the swelling process by unloading. From the figure, rebound behavior of bentonite rock is gradually increased as surcharge load is decreased. The volume of the sample becomes finally numerical times over than the original's one. In such a condition, free face of the sample in water is similar to a paste state.

Fig. 8 shows the representative curves under several surcharge loads.

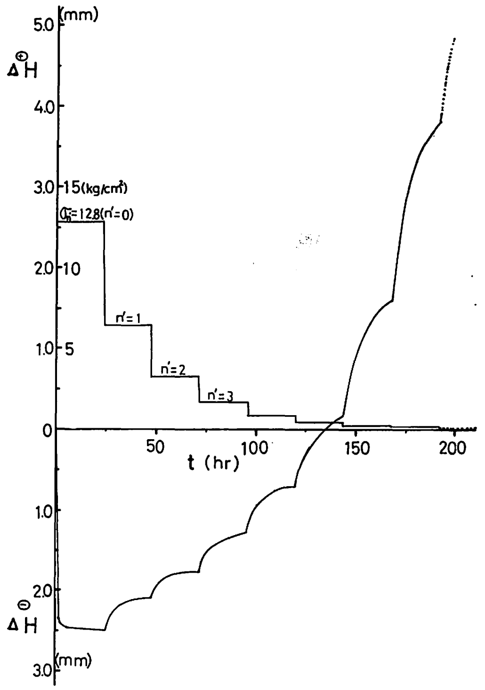

Fig. 7 Swelling proces by unloading

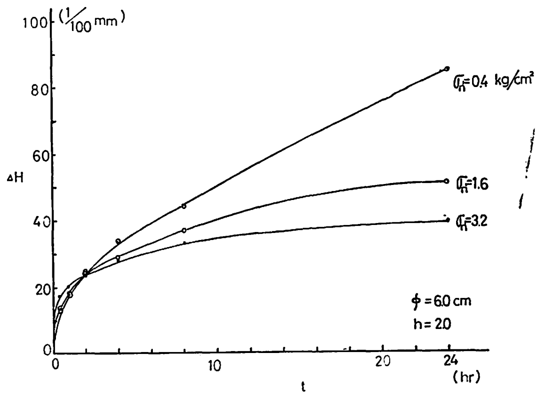

Fig. 8 Swelling deformation

4. Discussion of Results

4. 1 Effect of wall Friction

All the experimental results of swelling tests by the oedometer were obtained in one dimensional. Bentonite is very cohesive, thus friction force would act to the cell wall as the swelling is developped. At the same time, swelling behavior is restrained by its lateral reaction and friction.

On the theoretical basis of consolidation, the deformation of the sample is expressed as :

$$\varDelta H = m_v \cdot \varDelta\sigma s \cdot h \qquad (1)$$

where $\varDelta H$ is swelling deformation, m_v is swellability of sample, σs is defference of swelling pressure and h is height of sample.

The sample does not expand to lateral direction when the swelling is occurred because the cell wall is rigid. In that condition, if we take $\beta = \frac{1-\nu-2\nu^2}{1-\nu}$ then,

$$\varDelta H = \frac{\beta}{E}\int_0^h \varDelta\sigma s \cdot dx \qquad (2)$$

where ν is poisson's ratio and E is Young's modulus. But in this σs, various values are contained. That is, available factors to be considered for them are a weight of the top platten (σw), bulk density of the sample (ρ), up lift pore pressure (U) by water supply, and shear force on the cell wall (τ). True swelling pressure (ps) appears to be governed by these factors in the term of

$$ps - \sigma_S - \sigma_W - \rho \cdot h + U - \tau = 0 \qquad (3)$$

in which,

$$\tau = C' + \sigma' \tan\phi' = C' + \mu \cdot K \cdot \sigma_S \qquad (4)$$

C' is cohesion, μ is coefficient of wall friction and K is Rankine's constant to be expressed as $K = (1 - \sin\phi')/(1 + \sin\phi')$, ϕ' is angle of internal friction when the sample is saturated. In general, $\rho \cdot h$ and U are negligible, thus from (3) and (4)

$$\sigma_S = \frac{ps - \sigma_W - C'}{1 + \mu \cdot K} \qquad (5)$$

It is difficult to determine ϕ' and C', and ϕ' is often assumed finally as zero. Making a rough estimation, $\tau \approx 1/5\,\sigma_S$. Consequently, it is understood that swelling pressure is restrained about 20% in the uniaxial cell.

4. 2 Stress - Strain Diagram

As was already indicated in Fig. 6, σ_S-ε relationships were similar to each other. The relation may be stated in the form of

$$\sigma_S = C \cdot \varepsilon^n \qquad (6)$$

where C and n are material parameters to be determined from the chart. C is equivalent to Young's modulus, and which was smaller to rock in this test. n is nearly equal to unit. Therefore, the behavioris now still within a Hookian body in a sense. In such a condition, the material is generally considered as an elastic body because yield point does not yet appear within this test range.

Fig. 9 (a) shows the diagram of uniform stress distribution on the top platten. For reference, in making an estimation of inner stress condition for the present by Boussinesque's theory in accordance with Fig. 9 (a), it becomes like Fig. 10. Fig. 9 (b) shows a model of reaction of viscous soil. In conjunction with this, if it is allowed to consider the sample in the cell as a short column as shown in Fig. 11, it may be possible to estimate the maximum bearing force (Ru) using Dörr's theory, that is,

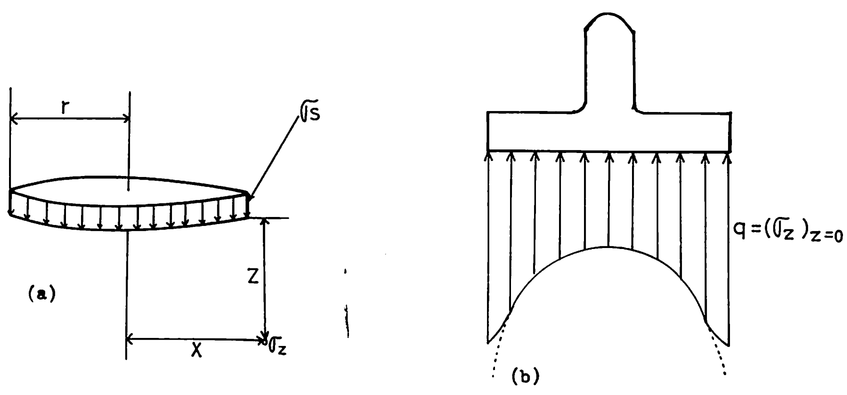

Fig. 9 (a) Stress distribution (b) Reaction

$$\left.\begin{aligned} Ru &= \pi r^2 \gamma_t h \tan^2(45° + \phi'/2) + \mu(1 + \tan^2\phi')\pi r h^2 \gamma_t + 2\pi r h c' \\ 3/4 \tan\phi' &\leqq \mu \leqq \tan\phi' \end{aligned}\right\} \quad (7)$$

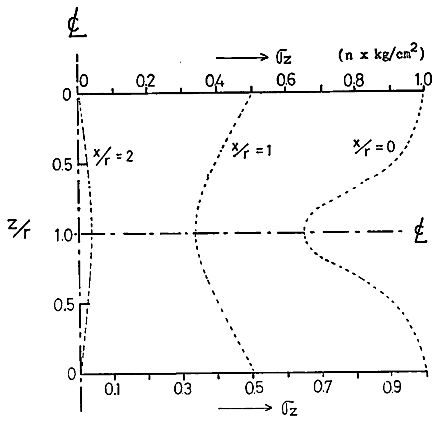

Fig. 10 Inner Stress in oue half specemen

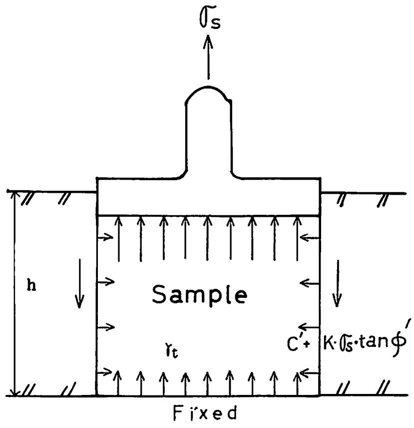

Fig. 11 A column in a rigid cell

From a tentative uniaxial compressin test results, maximum swelling stress proved to be about 4.12 kg/cm². Therefore, it should be taken such a stress into account for supporting the shaft wall.

4. 3 Swelling Behavior by Unloading

After the bentonite rock, was preconsolidated surcharge load was decreased by

$$\sigma_n' = \frac{1}{2^{n'}}, \; (n = 0, 1, 2, 3 \cdots\cdots) \quad (8)$$

σ_n' is n'-th surcharge. If n' becomes large, swelling deformation becomes large. In general, the swelling deformation is also considered to be based on the change of void ratio.

$$\left(\frac{\Delta V}{V}\right)_s = \frac{C_s}{1 + e_0} \log \frac{\sigma_n'}{\sigma} \quad (9)$$

where e_0 is the initial void ratio, V and ΔV are the original volume and deformed volume respectively. C_s is the swelling index defined by

$$C_s = -\frac{\Delta e}{\Delta \log \sigma} \quad (10)$$

Thus Fig. 12 was drawn in term of e and σ. A hyperbolic relationship is appeared by a continious decrease of surcharge. Because of this, following is a modified equation.

$$\log e = \log \alpha - C_s \cdot \log \sigma \quad (11)$$

The swelling line will be expressed for this swell test by the equation of

$$e = 0.91\,\sigma^{-0.118} \quad (12)$$

Swellability by unloading, i. e., the coefficient of volume swellability is defined as

$$m_s = \Delta e / \Delta \sigma \quad (13)$$

Similarly,

$$\log m_s = \log \delta - k \cdot \log \sigma \tag{14}$$

or

$$\sigma = \left(\frac{\delta}{m_s}\right)^{\frac{1}{k}} \tag{15}$$

δ and k are the material constants depending on the type of clay. Fig. 13 shows the above relationship. Still further study will be needed to clarify the physical meanings.

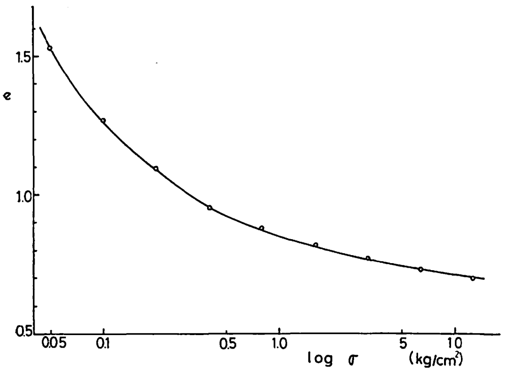

Fig. 12 e ~ σ Relationship

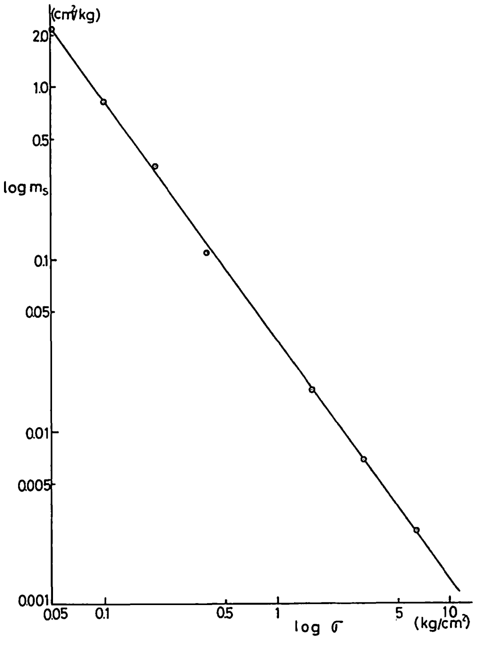

Fig. 13 m_s ~ σ Relationship

5. Conclusions

The study described herein was conducted to investigate the swelling characteristics of bentonite powder and bentonite rock which is unparalleled up to date. The following conclusions may be drawn from the present test results.

(1) For significant statistical analyses of swelling, it takes a long measuring time until swelling behavior stabled.

(2) The swell line curve of powder is steps wisely. On the other hand such a phenomenon is not appeared in rock. The main reason is probably due to the soil particle array.

(3) The swelling pressure of powder is superior to rock's one. This depends on the initial high dry density and low moisture content which tend to produce high swelling pressure.

(4) No special defference was found between powder and rock except above comment (2).

(5) Moisture does not distribute uniformly in the swelling specimen. This leads

the fact that stress distribution is not also uniform.

(6) As is one demensional test, swelling pressure is affected by lateral reaction and friction. By a tentative calculation the influence of laterally confined swelling pressure is about 20% decrease.

(7) Good correlation was found between the swelling pressure and the strain irrespective of rock and powder in the form of eq. (6).

(8) Influence of preloading and unloading are of importance in swelling. Applying a low surcharge resulted high swelling deformation.

(9) The main cause of this appears to be a change of void ratio by unloading. Thus $e \sim \sigma$ relation can be stated in eq. (12).

(10) The wide range of swelling parameters mentioned in section **2** is significant to evaluate the activity of swelling.

(11) Data and the method of measuring of swelling pressure described herein are insufficient to establish a consistent relationship. It is necessary to take an another swell test[7] and to take more influence factors of swelling into consideration.

Acknowledgements

The Paper is based on the Papers presented to the symposiums of the Mining and Metallurgical Institute of Japan, and the Japan Society of Civil Engineers.

The Authors are indebed to Professor T. Horibe and Professor R. Kobayashi, Tohoku University, and Professor S. Honda, Akita University for making valuable suggestions.

References

1) R. L. Lytton, Proc. Symp. on Soils and Earth Structures in Arid Climates, 29—37, 1971.
2) R. Krishna & N. Whittaker, Colliery Guardian, 396—402, 1973.
3) T. Ito, Annual Symp. of JSEC-Tohoku branch, 64—65, 1974.
4) T. Ito & S. Arakawa, Symp. of MMIJ, 196—197, 1975.
5) H. Takahashi, Yusui to Jiatsu, Sankaido, 1963.
6) B. P. Warkentin & R. K. Schofied, Jr Soil Sci., V. 13, 1962.
7) H. B. Seed., R. J. Woodward & R. Lundgren, Jr ASCE 88, SM3, 80, 1962.