

建 設 省
土木研究所資料
第 2 7 9 号

PUBLIC WORKS RESEARCH INSTITUTE

MINISTRY OF CONSTRUCTION

RECORD No. 279

**Triaxial Compression Test and
Strength Characteristics of
Soft Rocks**

TOKYO 1967

TRIAXIAL COMPRESSION TEST AND STRENGTH CHARACTERISTICS OF SOFT ROCKS

Ryunoshin YOSHINAKA*

1. INTRODUCTION

In the construction of conventional structures, the problems such as bearing capacity, settlement of ground, etc. have been practically dismissed, when the bedrock turns out to be a soft rock.

Recently, however, with increased dimensions of structures, economic designing of foundation has become important and accordingly the necessity has arisen to make clear the engineering properties of soft rock which hitherto received little attention. Soft rocks have been neglected so far in soil mechanics or rock mechanics, and many things remain ambiguous about how to treat or consider them.

The Public Works Research Institute of the Ministry of Construction has taken up this kind of problem and conducted some tests in connection with the surveys for the inter-island highway project between Honshu and Shikoku Islands. This paper describes the triaxial compression test on soft rocks and the relation between the test results and the ground behaviour.

2. SAMPLES

2.1 *Soft rock*

The term "soft rock" is not a scientific name of rock, but it generally refers to the rock which is commonly deemed mechanically weak; it may be considered to lie in the boundary domain between rock mechanics and soil mechanics. Such rock is in the order of 1~100 kg/cm² in strength and 5~50% in water content. Soft rock deposits in Japan mostly belong to Pliocene or Pleistocene period and rarely occur in weathered rocks, fault zone or breccia zone.

Pliocene deposits of soft rocks are most widely distributed over Kanto, Tohoku and southern Hokkaido; in Tohoku district they are more often accompanied by greenish tuff. Their occurrence in the south-west of Japan is discontinuous; they occur in Kinki and Chugoku districts. Topographically, they constitute the bedrock of hills, low mountains or plains. Thus, they are distributed generally in urban and suburban areas in Japan.

* Public Works Research Institute, Ministry of Construction, Chiba, Japan.

2.2 *Sampling of soft rock*

If the foundation ground of structure to be surveyed is homogeneous, the sample may be taken at any spot. Usually, however, the ground is stratified, so it is necessary to take a typical sample or to sample every component. When it is impossible to take a sample of the ground just beneath the structure foundation, an appropriate site which is supposed to be of the same composition as the ground in consideration is located, and the sample may be taken at this site. Because the geological conditions under which the tertiary soft rock was deposited and the conditions which have been undergone by such deposits occur on an extensive scale. Exception must be made, however, for the case where a fault is involved.

The rock sample, which is supposed to be homogeneous, must be taken from the same spot and it must be big enough for obtaining more than five specimens. When it is directly cut out of the bedrock by hand, it should be 30 cm × 30 cm × 30 cm at least. In the case, however, when cracks develop in an interval less than 10 cm, the sample is not suitable for preparing specimens.

When specimens are to be prepared from a boring core, the core must be 40–50 cm long at least for the same species of rock.

Sometimes the ground is composed of thin layers of pelite. In such a case, if the information of strength anisotropy of the layer is necessary for the purpose of stability analysis, sampling should be made so as to be able to test the strength in both the normal and the parallel directions to the layer. The strength of such pelitic rocks sometimes happens to be markedly anisotropic.

In taking samples and carrying them back to the laboratory, a care must be fully taken not to dry or develop cracks. Routine methods to prevent drying are either to seal with paraffin wax or to store within wet wooden chips.

2.3 *Preparation of specimens*

Samples carried into the laboratory are usually trimmed into cylindrical shape. The dimensions depend upon the type of the triaxial compression chamber, the size of original sample and the number of specimens to be cut out of sample. Usual size of specimen is 3–10 cm in diameter and the height is more than two times the diameter.

Use of a diamond-bit laboratory sampler will be simple and efficient for preparing the cylindrical specimen. The spinning speed of the core bit and its tip pressure depend on the hardness of sample. For soft samples the speed should be as low as possible.

The above method can not be applied to soft sandy rocks which have little cohesion. In this case, a diamond-blade cutter is employed first to cut the sample to about double the proposed dimensions and then a trimmer and knife are used to trim it carefully.

Water poses a controversial problem often in the trimming of soft rock. Water is indispensable for cooling the cutting edge of a sampler and for washing off the cutting chips. Normally the preparation of specimen takes 10 minutes or so, therefore use of water will hardly affect the mechanical strength of sample. Underground soft rocks, with some exceptions, have generally high water content and are mostly saturated or

nearly saturated with water. Accordingly, only the water-contacting surface of sample will exhibit a swelling change in short time. Difficulty with water concerns the drying of sample. In the case of pelite, a dried surface of sample will soon develop fine cracks and in the case of stratified rocks, a spalling will occur along the stratified plane. Thus, maximum attention should be paid to the moisture conditions for preparing the specimen.

3. TEST PROCEDURE

The triaxial compression test of soft rock is essentially not different from that commonly performed in a soil test. But the high strength of soft rock requires a side compression mechanism and other parts to be mechanically rugged enough.

For investigating the deformation of rocks, the lateral pressure up to $1,000 \text{ kg/cm}^2$ is employed.

In the civil engineering practice, however, pressure of $50 \sim 100 \text{ kg/cm}^2$ will be sufficient. For axial compression, the loading tester of Amisler type is employed.

The following is an outline of the triaxial compression tester used in the Public Works Research Institute.

The Foundation Study Laboratory of the Institute devised a prototype triaxial compression tester for soft rocks in 1965 and ever since has been using it. This apparatus has been found easy in operation and working well.

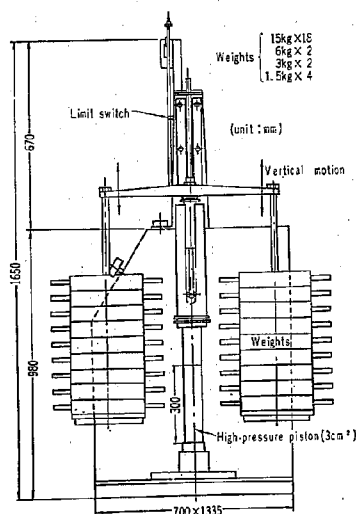


Fig. 1. Constant pressure device

For maintaining a constant fluid pressure within the triaxial cell, a weight type mechanism is employed. A weight is placed on the piston of a hydraulic cylinder which is filled with soft oil such as No. 44 oil. The oil pressure can be adjusted in the range of 2.5 kg/cm^2 to 100 kg/cm^2 by varying the weight. The piston drops as the cylinder oil flows out as a result of volume change of the specimen in the triaxial cell. When the falling piston reaches the lower limit, the microswitch acts for sucking oil and piston is pushed up to the upper limit, then a constant pressure is generated again. The piston stroke is 30 cm and its cross-sectional area is 3 cm^2 , which will be able to discharge 90 cm^3 of oil in one action of the stroke. In this manner, a constant pressure can be maintained in spite of a volume change of the specimen.

The sliding friction between piston and cylinder is reduced by the use of the rotating bush. Experimental results show that the pressure may vary more or less at the time of pushing up the piston. This trouble may be avoided by increasing a capacity of the hydraulic cylinder. A weight of the constant pressure device itself is about 300 kg. The apparatus is schematically shown in Fig. 1.

Fig. 2 shows the triaxial chamber which is 20 cm in inner diameter, 16 mm in wall thickness and about 100 kg in weight. The maximum diameter of the test specimen is 10 cm. For convenience of handling, the device is composed of separate units; mount, cell, top cover, piston and spherical seat. In addition, the followings are installed in the lower piston; a pore water pressure gauge for low pressure tests, 5 ton-, 30 ton- and 100 ton- load meters for axial compression and a pressure gauge for measuring the lateral pressure. Photo 1 shows the triaxial chamber mounted on the loading device.

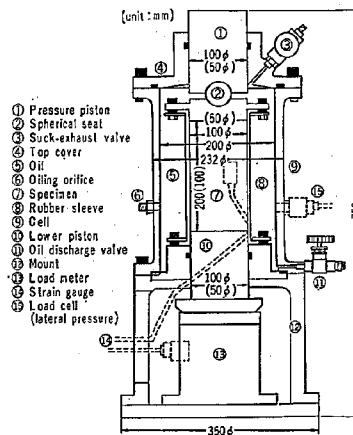


Fig. 2. Triaxial chamber

Prior to the compression test, measurements are made for each specimen on water content, unit weight and, if possible, on ultrasonic velocity. These measurements will be useful not only for giving information on general physical properties, but also for knowing the homogeneity of specimen, presence of cracks and calculating the dynamic modulus of elasticity. They will contribute to evaluation of the rheological properties of specimen, when compared with the static modulus of elasticity obtained from the triaxial compression test, hence they will help judging the properties of triaxial compression test data.

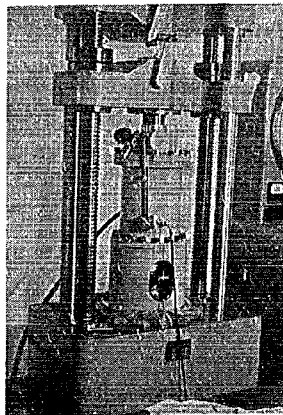


Photo. 1. Triaxial compression apparatus

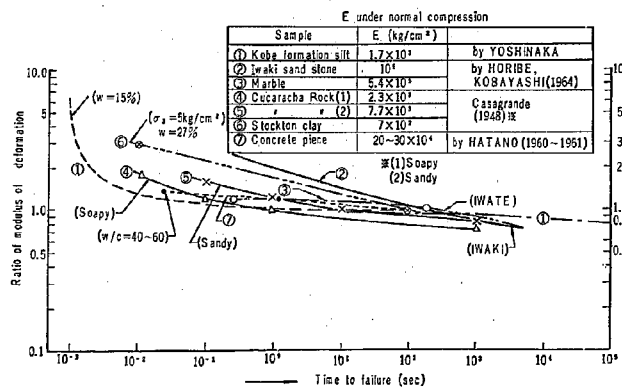


Fig. 3. Rate of loading vs ratio of modulus of deformation¹⁾

For the measurement of strain in the compression test, the dial gauge will be sufficient. As for the use of resistance wire strain gauges, no reliable bonding is available because the specimen contains much water. Moreover, if the bonded part is forced to dry, the strength uniformity of specimen may be lost.

The rate of compression should be selected so as not to affect the strength appreciably. Fig. 3 shows the influence of the rate of loading on the strength which is expressed in terms of a ratio of modulus of deformation, where the modulus corresponding to the rate of 10-10³ seconds is taken standard.

There is much ambiguity of assessing the in situ drainage condition of soft rocks, hence the drainage condition for laboratory tests. The rock masses constituting the bed are considered practically impermeable. And the preload is too large to expect any increase in strength of soft rock by consolidation*. Moreover, the design load becomes maximum under an earthquake shock, which is applied in very short time, and the information of strength parameters are required for this condition. Considering the above situation, the undrained shear test has been performed. Effect of pore water pressure set up in the specimen is considered important, and it is hoped to develop the measuring instrument.

4. TRIAXIAL TEST RESULTS

Species of soft rocks are very few as compared with those of ancient rocks. They are sandy rocks, pelitic rocks and tuffaceous rocks. Examples of test results on these rocks will be shown in the following.

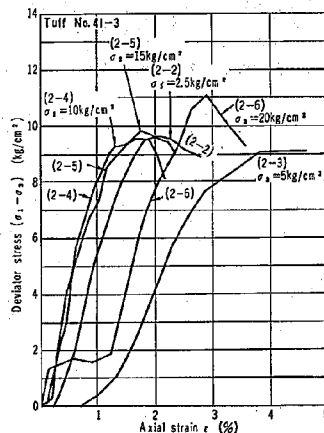


Fig. 4. Stress-strain Curves

* It is reported that the preload is about 1.5~5 times the unconfined compression strength.²⁾

4.1 Tuffaceous rocks

They are characterised by greenish colour and widely encountered in Tohoku district. They are deposits of volcanic ejecta and naturally comprise pumice-stones and much clay minerals. Some of them have small unit weight.

4.1.1 Tuff (tertiary), Onikubi, Miyagi Prefecture, greenish grey

A sample was dug out by hand from about 1 m depth below the top soil of river bed. The sampling site was below the flood level of river, however, it was supposed to be located normally above the ground water level.

Results of measurement are as follows; water content was $w=50\%$, unit weight was $\gamma_s=1.71 \text{ g/cm}^3$, specific gravity was $G_s=2.50$, degree of saturation was $S_r=100\%$ and unconfined compression strength was 10 kg/cm^2 on an average. Fig. 4 shows the stress-strain curves of this sample and Fig. 5 shows the Mohr's stress diagram. In the range of lateral pressure less than 20 kg/cm^2 , the strength parameters were found that $c=4.3 \text{ kg/cm}^2$ and $\phi=0^\circ$, where c is an apparent cohesion and ϕ is an angle of shear resistance.

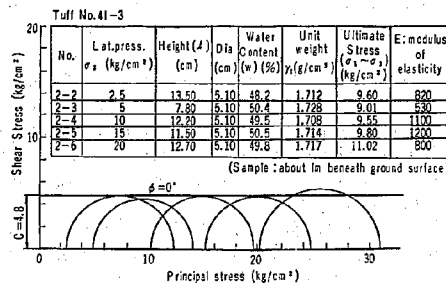


Fig. 5. Mohr's stress diagram

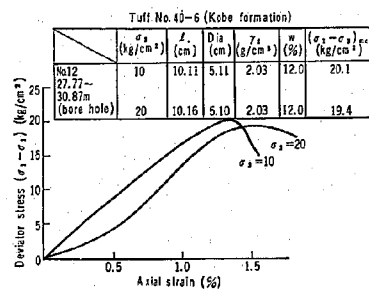


Fig. 6. Stress-strain curves

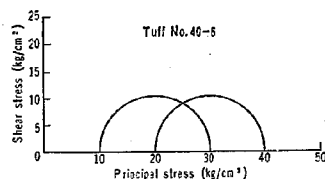


Fig. 7. Mohr's stress diagram

4.1.2. Tuff (tertiary), Kobe formation, dark green

Samples were taken out of a boring core from the Akashi Straits. Water content was 12% and unit weight was 2.03 g/cm³. Results of triaxial compression tests are shown in Figs. 6 and 7, where $c=10$ kg/cm² and $\phi=0^\circ$.

The condition of $\phi=0$ is common to these two cases. Though $\phi=13^\circ$ has been observed for some of the Kobe samples, this kind of rock may be characterised by very small value of ϕ . The breaking strain was found to be about 1.5% in the range of lateral pressure of 0~20 kg/cm².

4.2. Pelitic rocks

These rocks are as widely distributed as sandstones. Thickness of single layer of them is in the range from several tens of meters to several centimeters. The rocks belonging to the later tertiary period, if not weathered, are homogeneous and quite free from cracks. The rocks belonging to the earlier stage of tertiary period, or dating further back, develop secondary joints in the process of turning into rocks and become shale or slate which is liable to spall.

Water content is generally high, i.e., 10~30%. The constituents include clay minerals, heavy minerals, quartz, which govern their mechanical properties.

4.2.1. Pelitic rock, Miura formation (tertiary), dark gray.

Samples were taken by hand from the vicinity of Yokohama. Similar formations constitute the bedrocks surrounding the Tokyo Bay. Sampling depth was about 5.5 m beneath the ground surface, where alluvial deposit extended down to 4 m and soft rock existed further down. Accordingly, the pre-alluvial cover was 1.5 m deep. The underground water level was at about 3 m beneath the ground surface. Water content was 42.1%, unit weight 1.84 g/cm³, degree of saturation 100%, unconfined compression strength averaged 37 kg/cm². The results of triaxial tests are shown in Fig. 8 and 9, where $c=16.5$ kg/cm² and $\phi=6^\circ$. Unconfined static modulus of elasticity was 5,000 kg/cm² and that in triaxial test was 6,360 kg/cm² on an average.

4.2.2. Pelitic rock, Kobe formation (tertiary), dark bluish gray. No. 41-12.

Samples were taken by hand on land near Tarumi from about 1 m depth beneath ground level but above the underground water level. Water content was 17%, unit weight 2.14 g/cm³, degree of saturation 100%, unconfined compression strength averaged 30 kg/cm². The triaxial compression tests were carried out with the lateral pressure of 0~79 kg/cm².

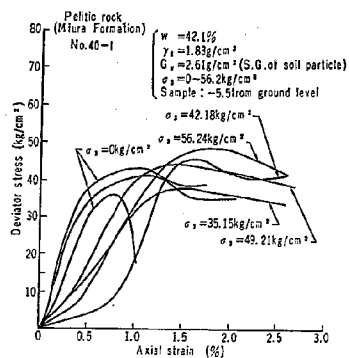


Fig. 8. Stress-strain curves

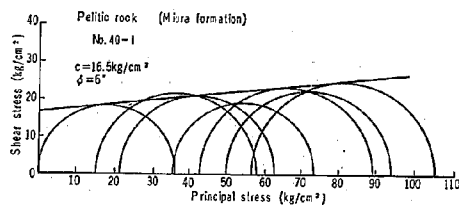


Fig. 9. Mohr's stress diagram

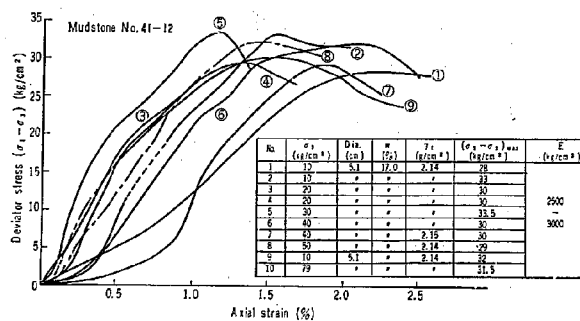


Fig. 10. Stress-strain curves

The results are shown in Figs. 10 and 11, where $c=14.5 \text{ kg/cm}^2$, and $\phi=1^\circ$. Static modulus of elasticity was $2,500\sim 3,000 \text{ kg/cm}^2$.

Looking at these examples and data on other samples, it may be said that, for pelitic rocks with higher water contents, the angle of shear resistance is $\phi=0\sim 20^\circ$ and the failure strain is in the range of $0.5\sim 1.0\%$, and the stress-strain curve, after passing the maximum stress point, tends to show a pattern of flow deformation as the lateral pressure increases.

4.3 Sandy Rocks

Tertiary sandy rocks, in general, have little cohesion and accordingly it is difficult

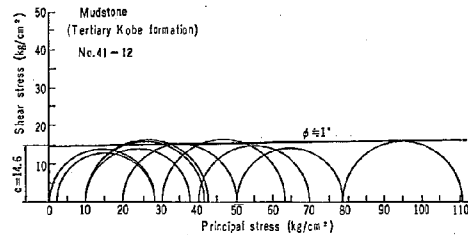


Fig. 11. Mohr's stress diagram

No.	σ_1 (kg/cm^2)	σ_2 (cm)	σ_3 (cm)	ν (%)	τ_1 (kg/cm^2)	$(\sigma_1 - \sigma_2)$ (kg/cm^2)	E (kg/cm^2)
9	6	5.6	5.3	10.4	2.03	12.0	1660
7	3	10.5	5.3	10.4	2.07	30	1000
9	10	10.6	5.3	10.4	2.19	45	1120
10	11	5.7	4.8	10.4	2.13	27.5	3180
1	20	10.6	5.1	10.4	2.97	149	4580
2	19	10.5	5.1	10.4	2.37	141	4120
3	40	10.6	5.3	10.4	2.52	180	2220
6	50	11.9	5.2	10.4	1.91	166	4580

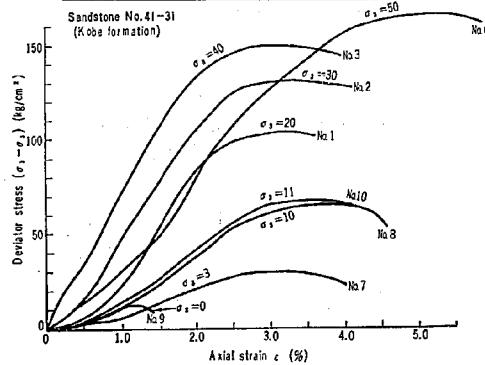


Fig. 12. Stress-strain curves

to take their boring core, hence their data are not many. Two examples, one of which is taken by hand and the other from core boring, will be shown here.

4.3.1 Sandstone, Kobe formation (tertiary)

The sample was taken on land by hand. Water content was 10.4%, unit weight 2.11 g/cm³ and unconfined compression strength 12 kg/cm². The results of triaxial tests are given in Figs. 12 and 13.

4.3.2 Fine particle sandstone (tertiary), Fukushima formation, dark gray

This is a formation of Kitamatsuura-gun, Saga Prefecture, which is supposed to correspond to Sasebo group of formation. Fukushima formation nearby seems to have been exposed more or less to the thermal action of basalt lava which directly covered it after deposition. It is exceptionally hard as compared with common tertiary rocks. Unit weight of sample was 2.32 g/cm³, water content 6~7%. The results of triaxial tests are given in Figs. 14 and 15, where $c=57.9$ kg/cm² and $\phi=53^\circ$.

The static modulus of elasticity increased from 37,000 kg/cm² to 56,000 kg/cm² as

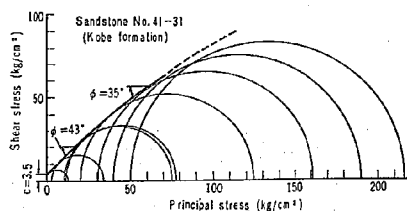


Fig. 13. Mohr's stress diagram

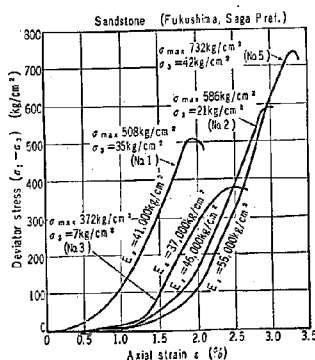


Fig. 14. Stress-strain curves

a lateral pressure increased from 7 kg/cm^2 to 42 kg/cm^2 .

From the above examples and the test results on other samples, it may be pointed out that, for the common soft sandstone with unconfined compression strength of $10 \sim 20 \text{ kg/cm}^2$, the angle of shear resistance is observed to be $\phi = 25^\circ \sim 30^\circ$, though higher values of $\phi = 40^\circ \sim 60^\circ$ are encountered in one case.

Meanwhile, the static modulus of elasticity depends largely on the lateral pressure and is not specific to each sample. The elastic limit strain increases with an increasing lateral pressure.

4.4 Modulus of deformation

The above series of triaxial compression tests has revealed that the shear character-

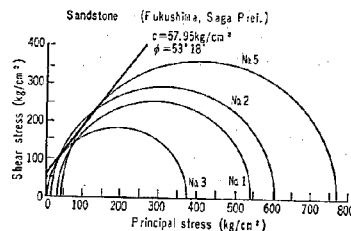


Fig. 15. Mohr's stress diagram

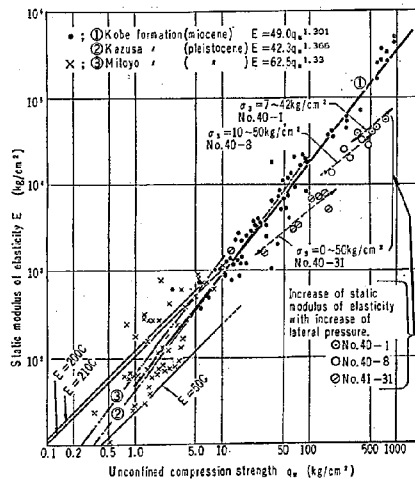


Fig. 16. Compression strength vs Static modulus of elasticity

istics of sandy rocks are heavily influenced by the lateral pressure. In the ϕ -material, the maximum deviator stress $(\sigma_1 - \sigma_3)_{\max}$ is proportional to the confining pressure, and there exists a correlation between this deviator stress and modulus of deformation (static modulus of elasticity). On the other hand, there is a certain relation between compression strength and static modulus of elasticity at lateral pressure $\sigma_3 = 0$. These two relations will be compared.

Fig. 16 shows the relation between the unconfined compression strength and the static modulus of elasticity in unconfined compression tests of tertiary deposits. Samples were taken from Kobe formation, Mitoyo formation near Kagawa Prefecture, Miura formation and Kazusa formation distributed around the Tokyo Bay, respectively. It should be noted that the data corresponding to the compression strength of more than 500 kg/cm² include those on the granite taken from about 200 m depth in the Akashi Straits. From the figure, linear relations between logarithm of the unconfined compression strength q_u and the static modulus of elasticity E are determined by the method of least square as follows,

- (1) Kobe formation $E = 49.0 q_u^{1.301}$ (from 70 samples)
- (2) Miura and Kazusa formations $E = 42.3 q_u^{1.366}$ (from 150 samples)
- (3) Mitoyo formation $E = 42.3 q_u^{1.33}$ (from 50 samples)

The samples comprised sandstone, mudstone, tuff and granite, however, the above relations did not seem to depend on rock species.

Compression strength at $\sigma_3 = 0$ can be expressed, from Coulomb's formula, as $\sigma_1 = q_u = 2c \cdot \tan(45^\circ + \phi/2)$. Substituting into the above relations, we have the following expression.

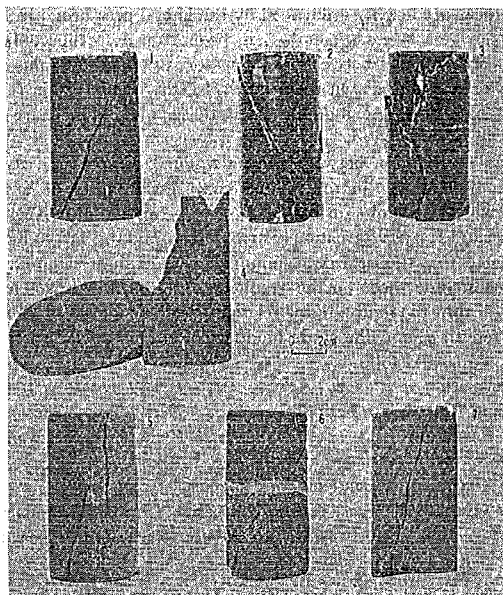
$$E = (103 \sim 154) \cdot c^{1.3} \cdot \tan^{1.3} \left(45 + \frac{\phi}{2} \right).$$

For $\phi = 0$, the above expression becomes $E = (103 \sim 154) \cdot c^{1.3}$, and on the average $E = 128 \cdot c^{1.3}$. It should be noted that similar relations of $E = 210 \cdot c^{0.9}$ and $E = (50 \sim 200) \cdot c^{0.4}$ for undisturbed clays have been reported. The present relation is shown in Fig. 16.

The relation between the maximum deviator stress $(\sigma_1 - \sigma_3)_{\max}$ and E of sandy rocks in triaxial compression tests is also shown in Fig. 16. In the figure the number of samples is not large. But the trend of $E \propto (\sigma_1 - \sigma_3)^{0.65}$ will be seen. Since $(\sigma_1 - \sigma_3)_{\max} = f(c, \phi, \sigma_3)$, E is a function of σ_3 alone for the samples which have the same values of c and ϕ , and a relation of $E = A \cdot f(\sigma_3)^{0.65}$ seems to hold, where A is a constant. Therefore, it will be understood that, in the case when the deviator stress of rocks is same at $\sigma_3 = 0$, E will be constant regardless of rock species. But the rate of increase in E with lateral pressure is different corresponding to rock species.

This problem has an important bearing on the solidification mechanism of rocks and a further investigation is desired.

Photo 2 shows the specimen after the triaxial compression test.



- | | | |
|----------------------------------|-----------------------------|-----------------------|
| 1. Miura formation mudstone | No. 40-1, Lateral pressure | 35 kg/cm ² |
| 2. Kobe " " | No. 41-12, " " | 40 kg/cm ² |
| 3. " " " | No. 41-12, " " | 10 kg/cm ² |
| 4. Miura " " | No. 41-1, " " | 56 kg/cm ² |
| 5. Kobe " sandstone | No. 41-31, Lateral pressure | 0 kg/cm ² |
| 6. Kobe formation sandstone | No. 41-31, " " | 50 kg/cm ² |
| 7. Fukushima formation sandstone | " " | 2 kg/cm ² |

Photo 2. Test specimens after triaxial compression test

5. COMPARISON BETWEEN STRENGTH CHARACTERISTICS OF TEST SPECIMEN AND GROUND

One-to-one correspondence of strength characteristic between test specimen and ground occurs only when the ground is a uniform elastic body. In reality, however, it becomes necessary to find some corrective method to relate the test specimen to the ground. In the case of a hard bedrock of dam, no way of correlating these two conditions has been established yet, because the strength characteristics of the ground with developed cracks and weathered surface vastly differ from that of the test specimen. In contrast, the soft rocks are geologically younger and relatively free from disturbance after deposition. They did not develop any conspicuous cracks on account of great

flexibility (even when subjected to disturbance). In soft rocks, therefore, it may be assumed that there is a close relation between the strength characteristics of specimen and ground.

In the following, comparisons will be made between the strength characteristic of specimen and ground for sandy and pelitic grounds respectively.

5.1 Sandy rocks (*Tarumi site of Kobe formation*)

A test pit, 2.5 m deep, 1.5 m wide and 7.5 m long, was cut in sandstone and therein a horizontal loading test was performed using a rigid circular loading plate of 30 cm diameter. The ground was a light-yellow homogeneous mass of coarse particle sandstone with a few cracks of 1 m spacing.

Samples for triaxial compression tests were taken from this ground. Test results were as follows; water content 10.9%, unit weight 2.15 g/cm^3 , unconfined compression strength averaging 15 kg/cm^2 , apparent cohesion $c=3 \text{ kg/cm}^2$, angle of shear resistance $\phi=23^\circ$, and static modulus of elasticity $E=1,870 \text{ kg/cm}^2$ (unconfined). The average of the loading test results were as follows; static modulus of elasticity 1470 kg/cm^2 (average of three points), ultimate bearing capacity 93.6 kg/cm^2 . It will be seen that in this case there is no great difference in modulus of elasticity between specimen and ground. Fig. 17 shows the load-deformation curves.

If the above-mentioned values of c and ϕ are substituted into the Terzaghi's formula of bearing capacity, the ultimate bearing capacity of about 90 kg/cm^2 is obtained, which is close to the above value of ultimate bearing capacity. Thus, in this example, sufficient correspondence of static modulus of elasticity, c and ϕ may be noted between the results of triaxial compression test and those of in-situ loading test.

5.2 Pelitic rocks, *Miura formation (near Yokohama)*

The sample was of a typical soft rock. Test results were as follows; water content 40%, unit weight 1.84 g/cm^3 , $c=16.5 \text{ kg/cm}^2$, $\phi=6^\circ$, unconfined compression strength 37 kg/cm^2 (average). Average Static modulus of elasticity in unconfined and triaxial compression tests was $5,000 \text{ kg/cm}^2$ and $6,000 \text{ kg/cm}^2$ respectively. The results of horizontal loading test at the ground showed that the static modulus of elasticity (between 0 and yield point) was about $6,000 \text{ kg/cm}^2$, which is the same as that measured in the compression test. Ultimate bearing capacity of about 100 kg/cm^2 (equal to $3/2$ times the yield point load) corresponded to about 6 times the apparent cohesion obtained from the triaxial test, which is little different from the bearing capacity factor for cohesive soils.

According to the Terzaghi's formula, the ultimate bearing capacity is to be about 150 kg/cm^2 , which is larger than the value of loading test.

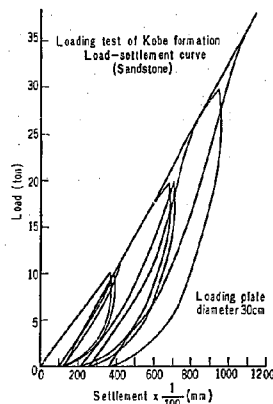


Fig. 17. Loading test results

6. CONCLUSIONS

As the results of the present experiments, it may be concluded as follows :

- 1) In the case of both sandy rocks and pelitic rocks, there is practically no difference between the static modulus of elasticity measured in the triaxial compression test and in the in-situ loading test.
- 2) For pelitic rocks, which are almost saturated, the ultimate bearing capacity obtained in the in-situ loading test is equal to 6~7 times the apparent cohesion c obtained from the undrained triaxial compression test. The above values of 6~7 are very close to the bearing capacity factor for cohesive soils.
- 3) In the case of sandy rocks, the ultimate bearing capacity obtained in the in-situ loading test may be explained by considering strength parameters c and ϕ obtained from the undrained triaxial compression test.

7. ACKNOWLEDGEMENTS

The Author wishes to express his gratitude to Mr. Yoshida, Head of the Foundation Study Laboratory and Mr. Komada, research member of the Foundation Study Laboratory, Public Works Research Institute, Ministry of Construction, for their encouragement and cooperation in the present research work.

REFERENCES

- 1a) R. Yoshinaka & I. Yoshida; Geotechnical Properties of Kobe and Akashi Formations, Report of the Public Works Research Inst. Ministry of Const. No. 129, 1966 (in Japanese)
- 1b) T. Horibe & R. Kobayashi; On Mechanical Behaviour of Rocks under Various Loading-Rates, Pro. of Japan Rock Mechanics Symposium, 1964 (in Japanese)
- 1c) A. Casagrande & W. L. Shannon; Research on Stress-Deformation and Strength Characteristics of Soil and Soft Rocks under Transient Loading, Harvard Univ. Soil Mechanics Series No. 31, 1948.
- 1d) T. Hatano & K. Tsutsumi; Dynamical Compressive Deformation and Failure of Concrete under Earthquake Load, Transaction of the Japan Society of Civil Engineers, No. 67, 1960 (in Japanese)
- 1e) T. Hatano; Relation Between Strength of Failure, Strain Ability, Elastic Modulus and Failure Time of Concrete, Transaction of the Japan Society of Civil Engineers, No. 73, 1961 (in Japanese)
- 2) A. Mori; Some Mechanical Properties of Soft Rocks (Dotan), Annual Convention, Japanese Soc. Civil Engineers, 1964 (in Japanese)
- 3) J. Takenaka; Sampling and its Reliability of Clay, Soil Mechanics and Some Recent Problem and Soil Testing Method, Japan Society for Testing Materials, 1962 (in Japanese)
- 4) Meyerhof, G. G.; Some Recent Foundation Research and Its Application to Design, Structural Engineer 31, 151, 1963

(Received: January 17, 1967)