Method for Unconsolidated-Undrained (UU) Triaxial Compression Test on Rocks

1. FOREWORD

This official standard presented by the Japanese Geotechnical Society (JGS) is for the unconsolidated-undrained (UU) triaxial compression test methods on rocks. The original draft was prepared by the Standardizing Committee of the Unconsolidated-Undrained (UU) Triaxial Compression Test on Rocks (refer to Table I for the committee member composition). Brief history and significance of this standard, and notes are described in the following sections.

1.1 Brief history of the standard

To select laboratory and in-situ test methods to be standardized, the Study Committee for the Rock Testing Methods of the Japanese Geotechnical Society (chaired by Professor Ryunoshin Yoshinaka of Saitama University, founded in 1993), conducted surveys on specifications and guidelines officially used by both domestic and foreign academic societies and organizations. The Committee studied practicability and necessity of the new standards. Two-year study showed that although the uniaxial and triaxial compression tests are widely used and considered important, there are no standards covering rock characteristics varying widely from soft to hard rocks. It was therefore concluded that the establishment of a new standard would be extremely important.

Under these circumstances, the Study Committee for the Uniaxial and Triaxial Compression Test Methods on Rocks (hereafter called the Study Committee), chaired by Professor Yoshinaka, carried out a three-year preliminary investigation since 1995 to collect and classify necessary information for standardization of the test methods. There were five main activities involved: (1) questionnaire survey; (2) nationwide round robin tests; (3) literature research; (4) sorting out of major points for standardization of test methods; and (5) organizing symposiums focusing on the uniaxial and triaxial compression test methods.

The unconsolidated-undrained (UU) triaxial compression test method on rocks presented here is based on the original plan by the Standardizing Committee for the Triaxial Compression Test Method on Rocks (founded in 1998). The draft standard was prepared by the Standardizing Committee, taking the results of the study by the aforementioned Study Committee into consideration, and presented on the monthly journal of JGS, *Tsuchi to Kiso*, Vol.47, No.9. This standard was then finalized, incorporating the decisions reached through in-depth discussions by the Study Committee for Standards Test and Survey Methods on Rocks, and the Standardizing Department.

1.2 Significance of the specification

This standard for unconsolidated-undrained (UU) triaxial compression test method is mainly for saturated rocks from soft to hard rocks, and rock like geomaterials. Namely, the draft standard is for test methods where a rock specimen is subjected to a constant cell pressure in a triaxial cell, blocking the

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drainage path for pore water, while the axial stress is monotonically increased until the specimen fails.

The aforementioned Study Committee conducted a nationwide questionnaire to learn what sort of uniaxial and triaxial compression tests on rocks have been conducted for what purposes, and how the test results have been used. Seventy-nine domestic organizations responded (response rate: 65%) and multiple analyses on the survey results were conducted. Main results obtained so far are, as follows:

1) Next to the uniaxial compression test, the triaxial compression test is the second most important and frequently used test method on soft and hard rocks.

2) Some of the commonly used triaxial compression test methods for rocks are UU, CU (consolidated-undrained test), CU (consolidated-undrained test with pore pressure measurement), and CD (consolidated-drained test) methods.

For soft rocks UU, CU, and CD tests are used with a similar frequency, whereas UU tests are much more frequently used for hard rocks than CU, and CD tests.

The main reason to choose UU as the test method for standardization was because this UU method is the most essential and popular one for both hard and soft rocks.

1.3 Outline of the draft standard
This standard comprises seven sections: 1. General rules to 7. Reporting.

1.3.1 General
This draft standard is “as a principle” applied to saturated rocks and rock like geomaterials. The aforementioned questionnaire showed that UU triaxial tests are frequently conducted on partially saturated hard rock specimens. This is why the phrase “as a principle” is used. The notes state, “This standard may be applied to partially saturated rocks”. While the unconsolidated-undrained (UU) triaxial compression test method on soils (JGS 0521-2000) requires more than three specimens, use of more than four specimens is recommended in this standard. This is because of the fact that rocks are generally less homogeneous compared with soils.

1.3.2 Test Apparatus
The UU triaxial compression test is frequently carried out to determine design values and input data for stability analyses, as well as to obtain appropriate strengths of rock masses and information necessary for rock mass classification. Reflecting this, there are many cases where the triaxial tests focus on obtaining deformation characteristics along with strength characteristics. Taking these trends into account, “measurement of circumferential displacements” and “measurement of longitudinal displacements on the sides of specimens” are referred to in the notes. Accuracy required in the measurement of longitudinal displacements is ±0.01% of the specimen height.

1.3.3 Preparation of specimens and measurement
The standard diameter of a specimen is set at 5.0cm and the height is double the diameter. This is based on the results of the aforementioned questionnaire, in which most organizations responded with these dimensions. Although the notes with respect to specimen dimensions state that “it is desirable to make the diameter of specimen more than five times the maximum rock grain”, other dimensions may be more appropriate for some rocks, such as conglomerate and rocks consisting of coarse grains. This is further explained in the commentary section.

1.3.4 Setting of specimens
“3. Preparation of specimens and measurement” states “specimens should be such that the both ends of specimens are flat and unevenness is less than 0.02mm”. However, some rocks, such as poorly consolidated coarse sandstones never meet this requirement. Capping is therefore referred to in the notes.

1.3.5 Test Method
The aforementioned questionnaire revealed that most organizations are using either stroke-controlled or strain-controlled test method. This is why this draft standard requires a compression process where the specimen is continuously compressed while keeping the longitudinal strain rate constant. However, this standard may be applied to both load-controlled and stress-controlled test methods, since there are organizations using these methods. In addition, taking into consideration the necessity of the evaluation of Poisson's ratio, the notes refer to the measurement devices for circumferential displacement.

1.3.6 Evaluation of Test Results
Equations for calculating longitudinal strains and principal stress differences are similar to those defined by the UU triaxial
test method (JGS 0521-2000). In addition to these equations, an equation for calculating the Poisson's ratio is given in the notes.

1.3.7 Reporting
Considering the fact that tests are often conducted to obtain deformation characteristics of rocks along with strength characteristics, the notes refer to the evaluation of deformation characteristics. “Secant slope of the principal stress difference vs. longitudinal strain curve at 50% of compressive strength” ($E_{s50}$) and “Tangential slope of the principal stress difference vs. longitudinal strain curve at 50% of compressive strength” ($E_{t50}$) are chosen as deformation moduli. Calculation method for these moduli is explained in the commentary section.

References

2. JAPANESE GEOTECHNICAL STANDARD “METHOD FOR UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST ON ROCKS”

Designation: JGS 2531-2000
Method for Unconsolidated-Undrained Triaxial Compression Test on Rocks

1. GENERAL

1.1 Purpose of Test
The purpose of this test method is intended to obtain strength and deformation properties of rocks, when subjected to unconsolidated and undrained triaxial compression.

1.2 Range of Application
This standard is mainly applicable to saturated rocks and rock-like geomaterials.

1.3 Definition of Terms
UU means shearing rock specimens under undrained condition, without consolidating them (isotropically) prior to shear.
Axial stress is applied in the longitudinal direction, while lateral stress is applied in the radial direction of the specimen. They are defined at mid-height of the specimen. The difference between the two stresses is termed principal stress difference. Isotropic stress state is defined as a stress state in which axial stress is identical to lateral stress. Cell pressure is the pressure applied in a triaxial cell. Lateral stress is equal to cell pressure. Unconsolidated-undrained compressive strength of rock is the maximum principal stress difference applicable to the specimen without consolidation, and without letting pore water move in and out of the specimen. Failure strain is the axial strain when the maximum principal stress difference is reached.

[Notes]
1.a. In case the method employed is partially different from this standard, details shall be clearly stated in the report.
1.b. Refer to the following specifications and standards for the issues not defined in this standard:
JGS 2521: Method for Unconfined Compression Test on Rocks
JGS 0542: Method for Undrained Cyclic Triaxial Test to Determine Deformation Properties of Geomaterials

1.1a. Tests shall be performed at different isotropic stresses in the required stress range, on as many specimens obtained from the same material as required, usually not fewer than four, to evaluate strength characteristics.
1.1b. This standard may be applicable to unsaturated rocks.
1.3 This test may be abbreviated as “UU Triaxial Test on Rocks.”

2. APPARATUS

2.1 Triaxial Compression Apparatus
A triaxial compression apparatus consists of a triaxial pressure chamber, cell pressure supply system, loading
system, together with load and displacement measuring devices. It shall satisfy the following conditions:

(1) Triaxial apparatus should be able to sustain the maximum axial compression load of the specimen and the maximum cell pressure.

(2) The apparatus should be able to apply pressure continuously to a specimen till the end of the test, with an accuracy of \( \pm 4\text{kN/m}^2 \) for pressures less than 200kN/m\(^2\), and of \( \pm 2\% \) for pressures greater than 200kN/m\(^2\).

(3) Axial displacement or axial load should be continuously applied at a constant rate of feed.

(4) Cell pressure should be measured with an accuracy of \( \pm 2\text{kN/m}^2 \) for pressures less than 200kN/m\(^2\), and of \( \pm 1\% \) for pressures greater than 200kN/m\(^2\).

(5) Axial compression load should be measured with an accuracy of \( \pm 1\% \) of the maximum axial compression load of the specimen.

(6) Axial displacement should be measured with an accuracy of \( \pm 0.01\% \) of the specimen height.

(7) Loading cap and pedestal should be impermeable.

2.2 Miscellaneous Accessories

(1) Specimen covering material

(2) Membrane stretcher

(3) O-rings or rubber bands

(4) Specimen size measurement tools should be able to measure the specimen height and diameter with an accuracy of or better than 0.1mm.

(5) Balance should be able to weigh the specimen to 0.01g.

(6) Sample trimming devices

[Notes]

2.1a. A typical triaxial testing apparatus is illustrated in Figure 1. Schematic views of two different triaxial pressure chambers are shown in Figures 2 and 3: loading piston and loading cap are rigidly connected (Figure 2); and are separated (Figure 3). In both types of the triaxial cells, the specimen can be mounted between cap and pedestal with the same diameter as that of the specimen. They are covered with a rubber membrane, and sealed with O-rings.

b. Circumferential displacement, if required, should be measured with an accuracy of 0.01% of the specimen circumference.

(1) Capacity of the cell pressure should be selected according to the purpose of the test. In case the maximum axial force is expected to become large, a loading frame with high rigidity should be used, so that the frame deformation will not affect the axial displacement measurement.

(5) When a load cell is mounted outside the triaxial cell, the measured axial force should be corrected for frictional force between loading piston and bushing. In case a load cell is installed inside the triaxial cell, the effect of cell pressure on the load cell reading should be calibrated, and correction should be made for measured axial force.

(6) When the main objective of the test is to obtain deformation properties of rocks, the measurement of axial displacement should be made on the lateral surface of the specimen by a proper technique.

(7) When there is a drainage hole or a porous disc is set in the cap and/or in the pedestal, the drainage hole should be closed off, using a flat rigid disc with the same diameter as that of the specimen.

2.2

(1)a. Rubber membrane, heat shrinkage tubing, or silicone rubber may be used as specimen covering material.

b. Rubber membrane should be longer than the membrane stretcher. The inner diameter of the membrane, when not stretched, should be in the order of 95% of the specimen diameter. The thickness of the membrane should be about 0.25 to 1.0mm.

(2) Membrane stretcher should be cylindrical, and its length and inner diameter should be 5 to 10% larger than the height and the diameter of the specimen. The stretcher should have a structure so that the rubber membrane can be stretched snugly against the inner side of the stretcher when vacuum is applied. If the cap is rigidly connected to the loading piston, it is desirable to use a membrane stretcher that can be split longitudinally into two pieces. In this case, the two pieces of the stretcher should be airtight when assembled.

(3) The inner diameter of O-rings should be about 80% of the diameter of the end cap or the pedestal where the O-rings are put around, so that they have sufficient confining force to prevent pore water from leaking.

(4) Specimen diameter should be measured by sliding calipers or by Pi tape.

(6) Miter box, sample trimmer, straight edge, surface grinder, diamond slab saw, and recording devices are used for sample trimming.
3. SAMPLE PREPARATION AND SIZE MEASUREMENTS

3.1 Sample Shape and Sample Size
(1) Test specimens shall be right circular cylinders.
(2) Standard specimen diameter shall be set as 5 cm.
(3) Standard specimen height shall be set as twice the diameter.

3.2 Sample Preparation
(1) Without finishing lateral surface of the specimen
   In case the borehole core diameter is the same as the diameter of the test specimen, the borehole core shall be cut to a desired length by a cutting machine. A surface grinder if required shall finish the specimen ends.
(2) Trimming method

3.3 Specimen Size Measurement
(1) Specimen diameter shall be measured in the vicinity of both ends and at mid-height to less than 0.1 mm, and the average value of the 3 measurements shall be assigned as initial diameter \( D_0 \) (cm).
(2) Specimen height shall be measured at more than 3 locations to less than 0.1 mm, and the average value of the...
3 measurements shall be assigned as initial height \( H_0 \) (cm).

(3) Mass of the specimen, \( m_0 \) (g) shall be measured to less than 0.01g.

(4) If necessary a representative piece shall be selected from the trimmed-off portions, and water content shall be measured. Measured water content shall be assigned as initial water content \( w_0 \) (%).

(5) Initial conditions of the specimen shall be observed and recorded.

[Notes]

3.1

(2) For coarse-grained or brecciated rocks, or conglomerate, it is desirable that the specimen diameter be greater than 5 times the largest compositional grain.

3.2

a. It is desirable that the test specimen of weak rocks be made and tested immediately after sampling. In case there is duration between sampling and testing, care should be taken so as not to damage samples, to change water content, or to change sample condition in situ.

b. When preparing test specimens, portions of material, which are obviously disturbed during sampling, shall be excluded.

c. Specimen shall be prepared within a reasonable duration of time so as not to change water content of material. Care shall be taken so as not to disturb material.

d. When preparing specimens, material orientation shall be recorded for clarity.

e. In principle, material longer than the finished specimen height shall be used.

f. Specimen ends shall be finished flat to an accuracy of 0.02mm, and square to the specimen axis. The squareness shall not deviate from its perpendicularity by more than 0.001 radians or 0.05mm for 50mm diameter specimen.

g. Lateral side of the specimen shall be smooth and free of irregularities. It shall be straight over the full length of the specimen, with a deviation less than 0.3mm.

(1) If material is affected by wetting, it shall be wrapped with plastic film and trimmed by a cutting machine using a small amount of water or oil.

(2) If samples are sensitive to wetting, trimming method should be applied to borehole cores and sample blocks larger than the specimen diameter.

(3) In case samples are insensitive to wetting, recoring method should be applied to borehole cores and sample blocks larger than the specimen diameter.

3.3

(3) If necessary initial wet density, \( \rho_0 \) (g/cm\(^3\)), initial dry density, \( \rho_{d0} \) (g/cm\(^3\)), initial void ratio, \( e_0 \), initial effective porosity, \( n_0 \) (%), and degree of saturation, \( S_0 \) (%), may be calculated using the following equations:

\[
\rho_0 = \frac{m_0}{V_0}, \quad \rho_{d0} = \frac{m_s}{V_0},
\]

\[
e_0 = \frac{V_0\rho_s}{m_s} - 1,
\]

\[
n_0 = \frac{m_{0w} - m_s}{m_{0w} - m_w} \times 100,
\]

\[
S_0 = \frac{m_0 - m_w}{V_0\rho_s - m_s} \times \frac{\rho_s - \rho_w}{\rho_w} \times 100,
\]

Where

\( V_0 \) (cm\(^3\)) : initial specimen volume

\( V_0 = \frac{\pi}{4} D_0^2 H_0 \)

\( \rho_s \) (g/cm\(^3\)) : grain density of rock composition

\( \rho_w \) (g/cm\(^3\)) : density of water

\( m_s \) (g) : dry mass of specimen

\( m_{0w} \) (g) : mass after submerging specimen into water longer than 72 hours, and

\( m_w \) (g): apparent mass in water after submerging specimen into water longer than 72 hours

(3) When water content is determined on the tested specimen subsequent to the test, water content measurement on a cut-off piece may be omitted.

(4) Lithology of the trimmed specimen shall be observed, together with properties of bedding planes, laminae, and fissures, and the degrees of weathering and alteration. Offset angles of the bedding planes, laminae, and fissures from the specimen axis shall also be measured.

4. SPECIMEN SET UP

(1) Specimen shall be placed on the pedestal, onto which a loading cap is placed. The side of the specimen shall then be sealed with a covering material. The ends of the covering material shall be tightened with O-rings against the pedestal and the loading cap.

(2) After triaxial pressure chamber is assembled, confining fluid shall be introduced into the chamber.

[Notes]

4.a. After placing a loading cap on top of the specimen,
excessive seating load shall not be applied in the axial
direction of the specimen, until cell pressure is applied to
the specimen. Especially when the compressive strength
of the specimen is expected small, the seating pressure shall
not exceed 10kN/m².

(1)a. If the condition specified in Note 3.2f cannot be met, the
specimen ends shall be capped with material like plaster.
Capping thickness shall be in the order of 1 to 3mm. The
capping material should be stronger and more rigid than the
rock specimen.
b. Loading cap and pedestal without drainage holes shall be
used, otherwise flat rigid disks with the same diameter as
that of the specimen are placed beneath the loading cap and
on the pedestal.
c. In case rubber membrane is used, sample set up
procedure may be different, depending on the membrane
stretcher used.

(i) With two-pieced membrane stretcher
① Specimen is placed on the pedestal, such that the
specimen is concentric to the pedestal. Capping
material shall be applied, if required.
② Rubber membrane and O-rings are placed on the
two-piece membrane stretcher, and the membrane is
stretched snugly against the inner surface of the stretcher,
applying vacuum. In this condition the stretcher is put
around the specimen and the pedestal, so that they are
covered with the membrane.
③ Capping the top end of the specimen, if necessary, the
loading cap is lowered and touched the specimen. The
loading piston is then fixed to the upper board of the
pressure chamber. In case the loading cap is not rigidly
connected with the piston, it is placed concentrically on
the top end of the specimen to be capped, if required.
④ Upper and lower ends of the rubber membrane are set
free from the stretcher, and the membrane is tightened
with O-rings against the loading cap and the pedestal.
⑤ The membrane stretcher is then split into two pieces
and removed.

(ii) With cylindrical stretcher
① Test specimen is placed on an appropriate stand.
② O-rings are put around the loading cap and the pedestal.
This is not necessary when elastic bands are used to seal
the specimen.
③ Rubber membrane is placed on the membrane stretcher,
and the membrane is stretched snugly against the inner
surface of the stretcher, applying vacuum. In this
condition the stretcher is put around the specimen and
the pedestal, so that they are covered with the membrane.
④ Upper and lower ends of the membrane are set free
from the stretcher, and the lateral surface of the specimen
is covered with the membrane. The extra portions of
the membrane are folded back.
⑤ Capping the specimen ends if required, the test
specimen is placed on the pedestal, such that the
specimen axis is aligned with the pedestal axis. The
membrane is then straightened to cover the pedestal.
⑥ Capping the top end of the specimen, if necessary, the
loading cap is lowered and touched the specimen. The
loading piston is then fixed to the upper plate of the
pressure chamber. In case the loading cap is not rigidly
connected with the piston, it is placed concentrically on
the top end of the specimen to be capped, if required.
⑦ The membrane is straightened to cover the loading cap.
⑧ Membrane is tightened against the loading cap and the
pedestal with O-rings.

(iii) In case a membrane stretcher is not used
① Test specimen is placed on an appropriate stand.
② O-rings are put around the loading cap and the pedestal.
This is not necessary when elastic bands are used to seal
the specimen.
③ The specimen is covered with the rubber membrane.
The extra portions of the membrane are folded back.
④ Capping the specimen ends if required, the test
specimen is placed on the pedestal, such that the
specimen axis is aligned with the pedestal axis. The
membrane is then straightened to cover the pedestal.
⑤ Capping the top end of the specimen, if necessary, the
loading cap is lowered and touched the specimen. The
loading piston is then fixed to the upper plate of the
pressure chamber. In case the loading cap is not rigidly
connected with the piston, it is placed concentrically on
the top end of the specimen to be capped, if required.
⑥ The membrane is straightened to cover the loading cap.
⑦ Membrane is tightened against the loading cap and the
pedestal with O-rings.
d. When a heat shrinkage tubing is used, the following
sample set up procedure may be used:
① Test specimen is placed on the pedestal, onto which a
loading cap is placed. The axes of the loading cap, test
specimen, and the pedestal are then aligned.
Heat shrinkage tubing is put around to cover the loading cap, test specimen, and the pedestal.

With an appropriate heat source, the tubing is shrunk till it sticks to the cap, test specimen and the pedestal.

Membrane is tightened against the loading cap and the pedestal with O-rings.

Water or oil is generally used as confining fluid. For some cases, e.g., at low cell pressures, gas may be used as a substitute.

Triaxial cell shall be assembled together, following procedure most appropriate to the structure of the testing machine used.

5. METHOD OF TESTING

5.1 Isotropic Stress Application

(1) Load cell and displacement transducer are mounted.

(2) Cell pressure is increased, and the specimen is pressurized isotropically until the cell pressure reaches a desired initial isotropic stress.

5.2 Axial Compression Stage

(1) Load cell and displacement transducer are adjusted and initialized.

(2) Keeping the cell pressure constant, the specimen is continuously compressed in the axial direction, at a nominal axial strain rate ranging between 0.01 and 0.1% per minute. If it is difficult to keep the axial strain rate constant, however, the specimen may be compressed at an axial stress rate equivalent to the above-mentioned strain rate.

(3) Axial force, \( P \) (kN), and axial displacement, \( \Delta H \) (cm), or axial strain, \( \varepsilon_a \) (%), are monitored during compression.

(4) For stroke control tests, the specimen shall be compressed even after the maximum load cell reading is reached. Compression may however be terminated in principle when a residual stress state (no further principal stress difference) is reached.

(5) The specimen tested shall be taken out of the triaxial cell, and conditions of deformation and failure shall be observed and recorded.

[Notes]

5.1

(2) a. When the cap is rigidly connected to the loading piston, the axial force is applied to the specimen through the loading piston, simultaneously with cell pressure application to obtain an isotropic stress state for the specimen. Since the relationship between axial force and cell pressure may vary, depending upon the diameter and self-weight of the loading piston, it is necessary to know their relationship in advance.

b. Circumferential displacement measurement device may be mounted if required.

c. Nominal axial stress rate shall be in a range between 3 and 60MN/m² per minute.

5.2

a. Oven-dried mass of the specimen, \( m_0 \) (g), may be measured if required. This procedure may be omitted, provided the water content has been determined using a piece of trimmed-off portions of the specimen prior to the test.

(1) If required, lateral displacement transducer is calibrated and initialized.

(3) a. In case the axial compression force and axial displacement are not continuously recorded, an adequate sampling interval should be adopted, so as to draw a smooth curve of the principal stress difference vs. axial strain relationship.

b. Axial strain is monitored by strain gauges mounted on the lateral surface of the specimen.

(4) For load control tests, compression shall be terminated when the axial displacement begins to increase abruptly.

(5) After compression test, the conditions of deformation and failure shall be observed and sketched from the direction where the state of the specimen can be seen most clearly. When a distinct shear plane is formed, the observation is made from an angle at which the shear plane becomes steepest. The shear plane shall be sketched such that its inclination can be directly read in the drawing. Observations are made to evaluate whether the specimen tested is homogeneous, and what condition impurities are in. The observation results are then recorded.

6. CALCULATIONS

6.1 State of Specimen prior to Test

Initial cross-sectional area of the specimen, \( A_0 \) (cm²), is calculated from the following equation:

\[
A_0 = \frac{\pi d_0^2}{4}
\]
6.2 Axial Compression Stage

(1) Axial strain of the specimen, \( \varepsilon_a (\%) \), is calculated using the following equation:

\[
\varepsilon_a = \frac{\Delta H}{H_0} \times 100
\]

Where \( \Delta H (\text{cm}) \) : axial displacement of the specimen.

(2) Principal stress difference, \( \sigma_a - \sigma_r (\text{MN/m}^2) \), at an axial strain of \( \varepsilon_a (\%) \) is calculated, as follows:

\[
\sigma_a - \sigma_r = \frac{P}{A_0} \left( 1 - \frac{\varepsilon_a}{100} \right) \times 10
\]

Where

\( P (\text{kN}) \) : axial compression force exerted to the specimen at an axial strain of \( \varepsilon_a (\%) \). \( P = 0 \) in the isotropic stress state.

\( \sigma_a (\text{MN/m}^2) \) : stress acting in the direction of specimen axis, and

\( \sigma_r (\text{MN/m}^2) \) : stress acting in the radial direction of the specimen.

(3) The relationship between principal stress difference and axial strain is plotted in a graphical form, where the vertical axis is the principal stress difference, and the horizontal axis is the axial strain.

(4) Maximum value of the principal stress difference is directly read on the figure described in (3) above, and is assigned as the compressive strength of the material tested.

[Note]

6.2 In case circumferential displacement is measured, lateral strain, \( \varepsilon_l \), and Poisson’s ratio, \( \nu \), are calculated using the following equations:

\[
\varepsilon_l = \frac{\Delta L}{\pi D_0} \times 100, \quad \nu = -\frac{\Delta \varepsilon_l}{\Delta \varepsilon_a}
\]

Where \( \Delta L (\text{cm}) \) : circumferential displacement of the specimen.

7. REPORTING OF RESULTS

The following items shall be reported regarding the test results:

7.1 Items with respect to Rock Samples

(1) Sample location: name of the place and depth
(2) Rock type
(3) Sampling method

7.2 Items with respect to Test Specimen

(1) Method of sample preparation

(2) Initial height, diameter and wet density of the specimen
(3) Initial water content of the specimen, if measured
(4) Observation results on the specimens

7.3 Items with respect to Test Results

(1) Cell pressure, and strain rate or stress rate
(2) Compressive strength and failure strain of the specimen
(3) Principal stress difference vs. axial strain curve
(4) Observation results of the specimen tested

7.4 Other Special Notes

[Notes]

7.1
(2) e.g., sandstone, granite, tuff and so on.

7.2
(2) If required, initial dry mass, initial dry density, initial void ratio, initial effective porosity, and initial degree of saturation of the specimen shall be reported.

(4) Lithology of the specimen should be reported along with geological features of the specimen, such as offset angles of bedding planes, laminae, and fissures from the longitudinal axis of the specimen.

7.3
(3) When circumferential displacement is monitored, principal stress difference vs. lateral strain relationship shall be reported, if required, together with lateral strain vs. axial strain relationship and Poisson’s ratio.

(5) If necessary, angle of internal friction, \( \phi_0 \), and the intercept at the vertical axis, \( c_0 \), obtained from the Mohr’s failure envelope shall be reported. If the envelope is nonlinear, the stress range at which \( \phi_0 \) and \( c_0 \) are determined shall be clearly stated.

7.4 a. If required, residual strength of the material tested shall be reported along with the angle of internal friction at this state, \( \phi_0 \), and the intercept at the vertical axis, \( c_0 \), determined from the Mohr’s circles for the residual stress state.

(2) If required, the secant modulus, \( E_{s50} \), and the tangent modulus, \( E_{t50} \), for each applied isotropic stress shall be determined from the initial portion of the principal stress difference vs. axial strain curve at 50% of the maximum strength of the material. The relationship between deformation moduli and isotropic stress shall then be
reported.

3 COMMENTARIES ON THE STANDARD

3.1 GENERAL

3.1.1 Test Purpose and Application Range of the Standard

To understand the mechanical behavior of rock mass, it is very important to grasp first the mechanical behavior of the rock that constitutes the rock mass. For this purpose, uniaxial compression tests are often carried out, since the testing machine and measurement devices required for the tests are relatively simple and easy to use. Many published uniaxial compression test results are available, including cross-relationships with the results of other rock tests. It is however difficult to describe fully the rock behaviors with the uniaxial compression test results only, when stress conditions are complicated as in the vicinities of huge bridge foundations, dam foundations, or underground structures. The strength and deformation characteristics of the rock obtained from the triaxial tests become especially important for the design of these structures. In evaluating the strength and deformation characteristics of the rock with triaxial compression tests, it is necessary to take the drainage condition during the tests into consideration, with respect to the designed structure and loading types, and the time periods of the construction. For example, when a long-term stability problem is considered, the tests should be conducted in consolidated drained condition and the creep problem should also be accounted for. Validity of applying the effective stress concept to hard rocks should be examined, depending upon the strain rates used in the tests, since hard rocks generally have high strength and very low porosity. Therefore it is not common to measure pore water pressure in triaxial compression tests and in axial compression stages in practice. Under the assumption that the pore water pressure has little influence on the mechanical behavior of the rock, triaxial tests are usually conducted on naturally dried samples without drainage. For this reason, the standard described here defines triaxial tests performed with the drainage valve closed as undrained and unconsolidated (UU) conditions. Furthermore, example test results shown are not entirely for saturated samples. The standard aims mainly for saturated samples, yet may be applicable to partially saturated samples, too.

In this section, soft rocks are distinguished from hard rocks, according to the uniaxial compressive strength $q_u$. Soft rocks have $q_u$ less than 10 to 20 MN/m$^2$, while hard rocks have $q_u$ more than 50 to 100 MN/m$^2$. Note that these values are valid for isotropic materials. Anisotropic materials with laminations and layered structures will be discussed later. For soft rocks to which the concept of effective stress is applicable based on Terzaghi's theory, the internal frictional angle of the material expressed in terms of total stress can be zero under UU condition. Figure 1.1 shows the results of triaxial compression (UU) tests on Shimajiri mudstone. Though there is some scatter, increase in strength cannot be seen over a cell pressure range between 0 and 2 MN/m$^2$. This means that the sample is fully saturated, and that the increase in cell pressure is compensated by the increase in pore pressure, keeping the effective stress constant.

On the other hand, Figure 1.2 shows the results obtained from the triaxial compression tests on a hard rock. The figure indicates that the strength is obviously dependent on the cell pressure. As will be discussed in Section 3.1.3, it is known that there is a range of stress where effective stress principle applies depending upon the strain rate, even for hard rocks with porosities less than 1%. This implies that the equation to describe effective stress corresponds to Equation 1 in Section 3.1.3, where the compressibility of water and soil skeleton is taken into consideration. Furthermore, a rapid advance in

![Figure 1.1 Mohr's circles for UU tests (Sinjyo)](image1)

![Figure 1.2 Failure envelop of rock (Brace)](image2)
numerical analyses in recent years has made it possible to describe in detail the characteristics of rock mass with different models, including one that considers the effect of discontinuities. In conducting these numerical analyses, it is very important to understand the mechanical behavior of the rock itself.

From the above viewpoint, experimental researches for rocks will be reviewed first. The mechanical behavior of the rock, different from soil, will then be discussed in detail.

3.1.2 Advancement in the Research into Mechanical Behavior of Rock

From the mid 1960's to the early 1980's, to understand the earthquake mechanism and the inner structure of the Earth, many researches on hard rocks had been conducted into the effects of temperature, isotropic stress (intermediate stress), strain rate, sample size and anisotropy. Published papers dealt with wide variety of subjects, such as failure criteria, the development of the fracture within the sample, and the change in elastic wave velocities. These papers have been reviewed carefully in the book called “Experimental Rock Mechanics” by Paterson\(^3\). According to the book the first paper was published by Karman\(^2\), also very famous for Karman's eddy and the column's buckling theory. He presented the experimental results on Carrara marble performed under an accurate cell pressure control. With uniaxial compression strength of 140\(\text{MN/m}^2\) in a high-pressure triaxial cell with a maximum cell pressure of 326\(\text{MN/m}^2\), he showed that the strength is dependent on the mean stress and that the marble possesses both brittle and ductile failure characteristics. The crystal structure of the marble after failure was also discussed, based on the consideration of the friction law, and the observation results under microscope.

Failure phenomenon of brittle material like rocks attracted many researchers, Griffith\(^4\) to begin with, whose interest were mainly theoretical developments. Here focus is placed on the experimental work where microcrack developments were observed in some manners. Generally speaking, in the first stage of a triaxial compression test on brittle material like rocks, pores begin to close due to the increase in principal stress difference, and the stress-strain relationships become convex downward. In the second stage, elastic behavior prevails, and the stress-strain relationship becomes fairly linear, though hysteresis still exists. In the third stage, a steady crack

\[\begin{align*}
\text{Region AB} & \quad \rightarrow \quad \text{Region BC} \\
\text{Region CD} & \quad \rightarrow
\end{align*}\]

Figure 1.3 Stress-strain relationship and areal density of microcracks in quartzite (Hallauer et al.\(^15\))
development may be observed and the axial stain stays linear but the lateral strain becomes nonlinear. As a result, the volumetric strain becomes nonlinear. In the fourth stage, fractures occur in particle structures within the zones of developing unstable cracks. Detailed descriptions are summarized by Paterson.

It is therefore natural to expect these changes in rock structure be observed directly in experiments. Hallbauer et al. carried out a series of triaxial compression tests on a fine-grained quartzite. They loaded the specimens to different stress levels, and took the specimens out of the triaxial cell. They then measured the difference in areal density of the microcracks in vertical section at each stress level prior and subsequent to failure. Not to mention, a loading frame with high stiffness and technology to measure the microstructure of a rock were required. In the experiments, they observed over 2,000 microcracks in a 75mm long, 25mm diameter specimen of a quartzite, and calculated the crack density for a unit area of 4mm², as shown in Figure 1.3. It is reported that the length of the cracks observed was between 0.1 to 1.0mm, and that most microcracks occurred within the quartz particles. Detailed investigation revealed that the mean length and the mean width of the cracks were 300µm and 3mm, respectively. Most microcracks were oriented at angles less than 10 degrees to the longitudinal axis of the specimen.

It may also be seen in the figure that a localized deformation occurs even at low stress levels, and that zones of connected discontinuities are formed in the central portion of the specimen right before the peak strength is reached. This phenomenon is universal though it may be dependent on the mean stress. These are stress-induced cracks and are different from liquid-solid interfaces often found in quartz particles, and

Figure 1.4 (a) Stress-strain relationship of Westerly Granite at cell pressure of 100MN/m² (Brace et al.6) (b) AE signals associated with failure of rock (Scholz), (c) Generation process of faulting estimated from AE observation (Lockner et al.31)
consisting of microcracks with a diameter of about 10 mm. These liquid-particle interfaces are inherent cracks. Kudo et al.30) observed these cracks in various Japanese granites. They investigated in detail the relationships with mechanical anisotropy caused by the fabrics and structures of granite. Peng & Johnson37) and Olsson and Peng38) studied the geometrical characteristics of crack formations in different rocks, though the basic idea was the same as Hallbauer et al.15). Lockner et al.31) observed the crack development using acoustic emission technique. In their experiments, to prevent cracks from developing too quickly, axial load was carefully applied with a hydraulic servo control system, to produce the test results shown in Figure 1.4. Vinegar et al.48) observed the inside of a specimen with a computer tomography (CT) technique. It was found in their research that even though the density change of a rock is regarded isotropic under hydrostatic pressure, dilatancy (volume increase) due to the development of microcracks might occur in the central part of the specimen, while density increases (volume decreases) at the ends of the specimen under uniaxial pressure. The CT machine has an accuracy of detecting the 0.1% change in 2 mm² of CT figures. Wang et al.49) developed a laser speckle method to observe a fracture process zone occurring at the tip of a fracture. This is an effective method in observing the development of failure area in a specimen continuously without physically touching as load increases. Furthermore, Sugawara et al.43) continue their observations of different geomaterials from soft clay to hard rock with X-ray CT machine.

3.1.3 Concept of Effective Stress for Rocks - Verification by Experiments and Effective Stress Representation

Biot3) first proposed a concept of effective stress and established the theory for rocks as a porous medium. His research was introduced by Jaeger & Cook20) and Paterson16). Skempton42) evaluated the compressibility of soil skeleton and soil particles, in which he assumed the Terzaghi’s effective stress concept is valid for soils. Brace & Martin5) studied the validity of effective stress concept for Westerly Granite with a porosity less than 1%. In the experiment, the principal stress difference under triaxial compression was measured with strain rate ranging 10⁻⁷ to 10⁻³ (s⁻¹), as shown in Figure 1.5. In Figure 1.5(a), white circles and white triangles respectively represent the results for dried and wet specimens at a cell pressure of 156 MN/m². Black circles and black rectangles represent the results for wet specimen with an effective cell pressure of 156 MN/m² by apparently controlling pore water pressure. To investigate the chemical effect of the media used for applying cell pressure, two kinds of liquid, water and acetone, were used.

Figure 1.5(b) gives a conceptual description of Figure 1.5(a). It may be seen in the figure that the strength of the specimen gradually increases with the increase in strain rate in area BC, suggesting the strain rate dependent nature of the material. The results for dried or wet condition, or even for Cu condition follow the same linear trend. It is, therefore, reasonable to say that as long as the strain rate does not exceed the point C, the concept of effective stress is valid even for the rock with porosity less than 1%. Brace & Martin5) called the strain rate at point C a critical strain rate. For a strain rate above point C, even if the pore water pressure is controlled, the pore water pressure cannot dissipate fully within the specimen, as the strain rate increases, as shown by the black circles and rectangles in Figure 1.5(a). In this case, the strength given by the principal stress difference increases. They called this phenomenon as ‘dilatancy hardening’. It may be understood from the above discussion, that there is a certain range within

![Figure 1.5 (a) Influence of strain rate on failure strength of Westerly Granite at effective confining pressure of 156 MN/m²](image)

![Figure 1.5 (b) Schematic diagram of dilative hardening (Brace & Martin5)](image)

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which the concept of effective stress with strain rate dependency is valid for rocks with porosity less than 1%.

According to Budianski & O’Connell\(^3\), there are three possible states of the water existing in pores like cracks: 1) fully drained; 2) undrained connection; and 3) undrained isolation. For each state, a self-consistent solution exists. The solution suggests that elastic modulus of a material, macroscopically equivalent to the material with cracks, is largely dependent on the density of the crack. For example, the macroscopic Poisson’s ratio decreases with the increase in crack density under drained condition, and increases under undrained condition. Biot\(^3\) proposed the following equation taking water pressure within a crack into account:

\[
\sigma_{ij} - \alpha \delta_{ij} = 2G(e_{ij} + \frac{v}{1-2v} e_{kk} \delta_{ij})
\]

(1)

Where, \(\sigma_{ij}\) is a stress tensor, \(\alpha\) is a coefficient, \(p\) water pressure, and \(\delta_{ij}\) Kronecker’s delta. \(G\) and \(v\) are respectively macroscopic shear modulus and macroscopic Poisson’s ratio for materials with cracks. It was Griffith\(^4\) who studied logically the fracture theory for rocks with this characteristics.

Jaeger & Cook\(^5\) expanded the theory to the case where water pressure exists. Denoting tensile strength by \(T_0\), they expressed failure criteria, as follows:

1) for \(\sigma_1 + 3\sigma_3 \geq 4p\)

\[
(\sigma_1 - \sigma_3)^2 - 8\sigma_3(\sigma_1 + \sigma_3 - 2p) = 0
\]

(2)

2) for \(\sigma_1 + 3\sigma_3 < 4p\)

\[
\sigma_3 = -T_0 + p
\]

(3)

Though these equations do not necessarily satisfy all the strength conditions, failure criteria may be expressed without including water pressure, \(p\), when an effective stress representation, \(\sigma_{eq} = \sigma_{ij} - \alpha p \delta_{ij}\) is applied to the equations. It is very important in interpreting the experimental results. Note that a parabolic failure criterion shown in Figure 1.2 is also applicable to the condition 1).

In summary, the failure strength of rocks may well be given in terms of effective stress even for hard rocks, when the loading rate is very slow or the time for pore water pressure to dissipate is long enough. Note that expressions for \(\alpha\) have been studied by many researchers.

3.1.4 Anisotropy

Donath\(^6\) studied anisotropic behavior of rocks. Figure 1.6 gives an example of his test results. When the rock mass is obviously anisotropic, there are cases the strength anisotropy should be examined relative to the principal loading direction.

3.2 TEST APPARATUS

3.2.1 Conventional Triaxial Compression Test Machine

3.2.1.1 Maximum Cell Pressure and Maximum Axial Compressive Force (Study Committee for Uniaxial and Triaxial Compression Test Methods on Rocks\(^7\))

The maximum cell pressure varies depending upon whether the rock is soft or hard. For soft rocks it is commonly less than 10MN/m\(^2\), while it is often larger than 10MN/m\(^2\) for hard rocks. The medium used for applying cell pressure is usually water for soft rocks, and oil for hard rocks. In general loading capacity of the apparatus is in the order of 20 to 500kN for soft rocks and 200 to 5,000kN for hard rocks. Loading method is also different: Most soft rock apparatus use a stationary upper loading ram with a motor- or hydraulically-driven lower loading ram. The upper loading ram is fixed as for soft rocks, but the lower loading ram is driven hydraulically in most hard rock apparatus.

The rigidity of the loading frame should be considered according to the strength and the stiffness of the rocks tested. For most soft rocks, a frame with moderate stiffness is used. For hard rocks, however, frame stiffness should be high for correct load control throughout the entire loading process, and for an accurate monitoring of post-peak behaviors. It is
desirable to use a loading frame with high stiffness in order to measure the stress-strain relationship accurately in the vicinity of the maximum load, and in the post-peak stress state. For example, stiffness of at least 2MN/cm is required for soft rocks, and 10MN/cm for hard rocks.

3.2.2 Precision of Cell Pressure Control and Measurement

Precision of cell pressure control is expressed differently, depending upon whether or not the cell pressure is larger than 200kN/m². For pressures less than 200kN/m², the precision is evaluated with an absolute pressure value; while it is evaluated with a relative value to the cell pressure when pressure is larger than 200kN/m². Precision is evaluated with the absolute error in the small pressure range so that the precision is always better than ±4kN/m². When the cell pressure is supplied by air pressure, the cell pressure control valve that satisfies the above mentioned capacity and precision should be adopted. When the cell pressure is supplied by load through a piston cylinder (e.g., NGI type), it is necessary to verify if the precision specified here is satisfied by calibrating the relationship between the supplied pressure and the load.

Precision of pressure measurement is the same as that of the cell pressure control: For pressures less than 200kN/m², the precision is evaluated with an absolute pressure; whereas it is evaluated with a relative value to loading pressure when pressure is larger than 200kN/m². Precision is given in the absolute value in the small pressure range so that the precision is always better than ±2kN/m².

3.2.2.3 Load Cell

Estimating the compressive strength of the specimen, maximum axial compression load subjected to the load cell is calculated using the specimen diameter. Among several load cells with different capacities being prepared, a load cell to be selected shall have loading capacity larger than the estimated maximum axial compression load and required measurement accuracy. Tolerance of load cell to the maximum compressive force is specified as big as that for the cell pressure.

The load cell can be installed within or outside the pressure chamber. For those used inside the chamber, the load cell value can be directly read. However, when the cell pressure is very large, it is necessary to verify if the load cell value is stable and not drifting. In addition, some load cells on the market are self-calibrating types, and they are structured in such a way that the effect of the change in cell pressure on the reading can be automatically cancelled out.

In case a load cell is used outside the pressure chamber, measurement error will increase when a small stress is measured or the piston is subjected to a bending moment. It is necessary to make some corrections for the frictional force between piston and bushing of the pressure chamber. Mounting a load cell outside and another inside the pressure chamber, and applying an axial force on the loading piston with a desired cell pressure, this frictional force can be evaluated as the difference in readings between the two load cells.

Load cells mounted inside the pressure chamber are becoming popular in recent years, especially when measuring extremely small strains. Generally, the accuracy of load measurement is not so good as that of deformation measurement. Therefore the future challenge will be how to improve accuracy of extremely small stress measurements.

3.2.1.4 Deformation Transducer

For axial displacement (axial strain) measurements, displacement transducers are set in place, in such a way that a relative movement between the specimen bottom and the loading piston can be monitored. Differential transformer type displacement transducers, non-contact type proximeters, electronic dial gauges, and magnet-scales are generally used. In case main objectives of the test are to know deformation characteristics of the specimen at very small strain levels, and/or with high precision, axial displacement (axial strain) is directly measured on the side of the specimen, using strain gauges or LDT (Local Deformation transducers). Figure 2.1
shows an example of the axial displacement (axial strain) measurement with LDT (Goto et al.12).

Circumferential displacement (circumferential strain), or lateral displacement (lateral strain) is usually measured using strain gauges, ring type transducers, circumferential transducers and so on.

3.2.1.5 Cap or Impervious Circular Disc
When a testing apparatus uses a loading cap and a pedestal with a drainage hole or a porous disc, an impervious circular disc whose diameter is the same as the specimen diameter is usually used. In case a small deformation of a rock is measured, the stiffness of the cap or pedestal should be higher than the rock stiffness. This is why special steel for high-speed machining tools (SKH), and nickel-chrome-molybdenum steel, (SNCM) or the like is used as material of the loading cap and the pedestal in testing crystalline rocks such as granite. SKH is heat treated at 1,300 °C, and can be used in tests at temperatures up to several hundreds centigrade. It consists of chrome, molybdenum, tungsten, vanadium, cobalt and so on. For test at ambient room temperature, SNCM is used. There are other materials used, such as nickel-chrome (SNC), and chrome-molybdenum steels (SCM).

3.2.2 Other Accessories
3.2.2.1 Rubber Membrane or Heat Shrinkage Tube
Rubber membrane should be about 40mm longer than the specimen. Its thickness should be carefully chosen to avoid rupture and air/water leakage. It is usually 0.5mm thick. In UU tests, undisturbed specimens are used, and soil particles or shells larger than sand particles may rupture the membrane. Much attention should therefore be paid to its thickness. Rubber membrane is usually made of natural rubber, silicon rubber, neoprene rubber and so on. Since heat shrinkage tube, usually made of fluorine resin, is hard, it is only suitable for specimens with small deformation, such as hard rocks.

Usual setup method of the rubber membrane is to put a thin tubular membrane around a specimen. Occasionally liquid silicon rubber is directly painted onto a specimen with a brush and the test is performed after the painted rubber dries up. In special cases, such as long-term tests, tests with high temperature and high pressure, care must be taken for the possibilities of deterioration in rubber membrane (e.g., Koizumi et al.27) and medium liquid seepage into specimen by osmosis (e.g., Takahashi et al.40).

3.2.2.2 Membrane Stretcher
The membrane stretcher is usually selected according to how the loading cap is connected to the loading piston. When the cap is rigidly fixed to the piston, a two-piece stretcher is used. Otherwise both two-piece and hollow cylinder stretchers may be used. In either case, the stretcher should be manufactured smoothly so as not to scar the rubber membrane. The diameter and height of the stretcher should be approximately 5mm larger, and about 10mm higher than the specimen dimensions.

3.2.2.3 O-Ring or Elastic Band
In case O-rings are used to fasten rubber membrane, especially when a thick membrane is fastened, the membrane should be wrinkle free and free of sand particles so as not to cause water leakage. Metal hose clips or metal wire may also be used in place of O-rings. In such cases, there is possibility of water leakage at knotted parts on the outer circumference of the membrane, when the cell pressure is high and the membrane is thick. To avoid leakage, it is recommended to use more than one clip or more than one loop of wire.

3.3 SPECIMEN PREPARATION AND MEASUREMENT
3.3.1 Specimen Size and Shape
Fundamental shape of the specimen used in a conventional triaxial compression test is in general cylindrical. This is common to all suggested test methods and standards, regardless of which institution may be (e.g., ISRM18) or ASTM). The standard height and diameter of a specimen are respectively 10.0 and 5.0cm. Depending upon the purposes of tests and the status of the specimen, the diameter may vary in a range between 3.5 and 50.0cm and the height between 8.0 and 100.0cm.

For soft- and hard-gravely rocks, conglomerate, relative gravel size to the specimen size may become a problem. When the gravel size is much smaller than the specimen size, it has no effect on the strength. The ratio of the specimen diameter (D) to the gravel (d_max) varies in general from 5 to 20. It is reported by Kawasaki et al.27) that when this ratio is between 1 and 10, there is no size effect on the uniaxial compressive strength. In the ISRM standard18) the ratio is specified to be larger than 10. In the Japanese Geotechnical Society standard (JGS 0530-2000: Preparation of specimens of
coarse granular material for triaxial tests), the ratio \(D/d_{\text{max}} > 5\) is permitted. In this standard, it is therefore stated desirable to use a specimen with the ratio of \(D/d_{\text{max}}\) larger than 5.

For cylindrical specimens used in conventional triaxial compression tests, it is known that there is shape effect as shown in Figure 3.1 (Mori33)). This is due to a confining effect from the frictional force between loading plate and specimen end. To reduce this effect, the ratio of height to diameter should have a value larger than 2.0 to 2.5. This value is usually set at 2.0 (Study Committee for Uniaxial and Triaxial compression Test Method for Rocks19)). In the ISRM standard18), the ratio is assigned to be 2.0 to 3.0.

3.3.2 Preparation of Specimen

Trimming is a method of preparing soft rock specimens. In preparation with knife, attention should be paid not to disturb the specimens. Applying bending moment should be avoided. If the change in water content is expected to affect the specimen condition, it should be trimmed as quickly as possible. In addition, sample disturbance at the ends and side surface of the specimen should be minimized. In recent years, machines to finish specimen sides are developed for frozen samples and soft rock samples. With these, sample preparation time can be shortened, and human error due the difference in workers skills can be reduced (e.g., Koizumi et al.28)).

In hard rock, there are two methods of preparing specimens: one is re-coring specimens from block samples in the laboratory; and the other is to use directly drilled cores. The latter is very popular. The end surfaces of hard rock specimens are usually ground with surface grinder.

For both soft and hard rocks, it is very important to keep the physical properties (e.g., water content) and the mechanical properties (e.g., compressive strength) of rock specimens unchanged during the sample preparation.

3.3.3 Specimen Measurement

As for the initial condition of the specimen, recording discontinuities in the specimen can be helpful to understand and interpret the test results. Therefore it is important to observe and record inclination angles of bedding planes, laminae, and cracks from the specimen axis, and geological information related to rock characteristics, such as grain size.

3.4 SPECIMEN SETUP

3.4.1 Notes on the Use of Heat Shrinkage Tube

Heat shrinkage tube is used mainly for hard rocks, since it is stiff and its effect on the specimen deformation can be significant. Heat shrinkage tube is specially processed such that it shrinks back to the original size upon heating. It is made of fluorine resin, silicon rubber and so on. When heated, the inner diameter of the tube shrinks to a size a little smaller than the specimen diameter, and the tube sticks to and wraps around the entire specimen. Heat shrinkage tube is usually fixed to the loading cap and pedestal of the loading frame with O-rings.

It is reported in a research paper by Osada et al.35) on the confining effect of the heat shrinkage tube, that a cell pressure becomes about 50kN/m² in the initial stage when a tube is applied, and that it increases to about 200kN/m² in the failure stage by axial compression. When the specimen side is uneven, additional care must be taken, since shrinkage tube may tear at the corner of a concavity.

3.4.2 Notes on Direct Application of Silicon Rubber

When coating a rock specimen directly with silicon rubber, it is difficult to apply a thin layer, and the layer tends to become thick. Silicon rubber is therefore used mainly for hard rocks, such that the effect of silicon rubber on specimen deformation can be negligible if the layer is thick and stiff.

Silicon rubber is divided into two groups: one-component type and two-component type. One-component type reacts with moisture in the air and solidifies, when pushed out of a sealed container such as tube and cartridge. On the other hand, two-component type consists of a rubber base and a curing catalyst. When the catalyst is added and mixed with the rubber base at a normal temperature, it solidifies and becomes an elastic body. An attention should be paid, since silicon rubber sometimes attacks copper.
3.4.3 Cell Pressure Medium

Air, water or oil is used as cell pressure medium. An advantage, when air is used, is that waterproof measure is unnecessary for measurement devices, such as strain gauges. Acrylic pressure cells are safe because tensile loads are designed not to exceed one-tenth of the tensile strength. It is however very dangerous and the pressure cell may explode at high pressures if it is defective. It is therefore desirable to use air pressure not exceeding 1 MN/m².

When water is used as cell pressure medium, it should be deaired in advance. In addition, prevention measures against corrosion on metal parts, and waterproof treatments for strain gauges are necessary.

Mineral oil is usually used as cell pressure medium. The advantage to use mineral oil is that strain gauges can be used 'as are' because the oil is electrically non-polar. Oil is divided into two groups: pressure bearing oil and heat bearing oil; the latter is used for tests at high temperatures.

Attention should be paid to heat bearing oil like silicon oil, since it permeates through rubber membrane and/or heat shrinkage tube of silicon rubber. In cyclic loading tests, a measure should be taken to cool down the pressure medium oil, because heat is generated due to oil viscosity.

3.5 TESTING METHOD

3.5.1 Application of Cell Pressure

When the specimen is set up in the triaxial cell, the longitudinal axis of the specimen should be concentric to the central axes of the pedestal and the load cell. After the specimen is placed, the cell pressure medium is introduced into the cell. When water or oil is used as cell pressure medium, air is first purged from the triaxial cell, and the cell is filled completely with water or oil. The loading platens are then carefully placed against the specimen, and the cell pressure is increased to a predetermined level using hydraulic pump or other pressurizing devices. The cell pressure should be increased at a constant rate as much as possible, and the deformation of the specimen during application of cell pressure should be continuously monitored. Once the cap is firmly fixed against the piston, the axial load and the cell pressure is increased simultaneously until the predetermined level of the cell pressure is reached. Since the applied axial load is dependent on the diameter and weight of the piston, it is necessary to determine beforehand the axial load required to reach the selected cell pressure. After the hydrostatic pressure reaches the predetermined value, the axial load is increased while keeping the cell pressure constant until the corresponding peak strength is observed in the axial stress-axial strain curve. Furthermore, in case the output of displacement transducers is not initially steady because of the temperature or cell pressure fluctuation, the axial load should not be increased until after the outputs become steady.

The cell pressure should be measured with pressure-indicating devices (pressure gauges or pressure transducers) during the test. The cell pressure level is chosen taking into consideration the design and construction depths of the ground for which the test results will be used. The capacity of the triaxial cell must also be sufficient for the test. For example, a maximum capacity of several hundreds of MN/m² may be necessary for hard rocks, while several MN/m² may be enough when testing soft rocks. Since the maximum depth that man can reach directly at this moment is about 3 km, the maximum capacity required is approximately 100 MN/m².

The cell pressure may change during the test as the rock specimen deforms and the piston comes into the cell. Therefore, a servo-controlled device or a dummy piston should be used to keep the pressure constant, or at least minimize the pressure change during the test. When oil is used as the cell pressure medium, or when the cell pressure is adjusted mechanically, a dummy piston with the same diameter as that of the loading piston should be used so that it can move backward in accordance with the forward movement of the loading piston. Using the dummy piston, one can minimize the cell pressure change in a triaxial cell. When using a mechanical means, it is however difficult to allow for the volumetric change of the rock specimen. According to the method suggested by the International Society for Rock Mechanics (ISRM), the change in cell pressure during the test should be less than 1%.

3.5.2 Axial Loading

3.5.2.1 Deformation of a Rock Specimen

There are two methods used to measure the deformation of a rock specimen. One method measures the deformation directly using electrical resistance strain gauges, LDTs, or a circumferential deformation meter mounted on the specimen. The other method uses an LVDT or a dial gauge located outside the specimen. Whichever is used, it is essential to adjust carefully the initial point (zero point), the span and the
sensitivity, immediately before axial loading is commenced.

When an electrical resistance strain gauge is used, the quality of bonding greatly influences the measurement result. The gauges must therefore be attached to the surface of the specimen very carefully. Furthermore, if strain gauges are used for soft rocks, gauge sensitivity may be decreased by the rigidity of the gauge base and the resistance element. It is therefore recommended to select a strain gauge of low rigidity when testing soft rocks. In addition, for some selected cell pressure levels, the sensitivity of the strain gauge may be pressure dependent, and should be examined beforehand.

When measuring rock deformation using a device installed outside the triaxial cell, the axial strain of the rock specimen may be overestimated due to the influence of a so-called "bedding error" caused by a loose layer or incorrect end finishing of the specimen. Therefore, adjustments may be necessary, and the results may have to be corrected, depending on the testing device used.

Figure 5.1 shows principal stress difference-strain curves measured by strain gauges. The rock specimen was Tage tuff, and tests were performed at a cell pressure of 2MN/m². The specimen was 30mm in diameter and 75mm in height, and was jacketed with silicon rubber. The axial loading was controlled with the displacement transducer installed outside the triaxial cell. The displacement of the specimen was measured directly by strain gauges attached longitudinally and circumferentially on the surface of the specimen. The two circumferential strain gauges were located orthogonally with respect to one another. The volumetric strain was calculated as the sum of the axial strain and the twice the circumferential strain readings. The figure represents the relation between principal stress difference and axial, circumferential and volumetric strains.

As shown in the figure, the volumetric strain changed from compression to dilation at a principal stress difference of about 18MN/m², and the rock specimen significantly dilated after the peak principal stress difference was reached. In this test, the load and strain data were acquired at a sampling rate of one point per second, the rate high enough to read a continuous change in the strain variation with stress. It was observed after the test that a single shear plane had formed at an angle of about 35 degrees to the specimen axis. Fortunately, since the strain gauges happened to be located just off the shear plane, the strains were measured successfully from the beginning to the end of the test. Had the shear plane formed underneath the strain gauges, they would have been disconnected, and the measurements would have become impossible from that moment. This possibility is a disadvantage of using strain

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Figure 5.2 Results of triaxial compression tests on Inada-granite with circumferential measurements (Yamaguchi et al.)
Figure 5.2 shows the results of triaxial compression tests for Inada granite under various cell pressures. The specimen was 50mm in diameter and 100mm in height. In the test, the specimen was jacketed with a heat shrinkage tube, and an axial deformation transducer and chain-type circumferential deformation transducer were set around the tube. The specimen was loaded at a constant rate of circumferential deformation of 0.02mm/min. In the test, the so-called Class II behavior was successfully obtained under a constant circumferential deformation rate after peak stress was reached. It is generally impossible to obtain Class II behavior under a constant axial-strain rate. As shown in this figure, volumetric strain changed from compression to dilation as principal stress difference increased, and there was a significant volume increase in the rock after the peak principal stress difference. Compared to measurements with strain gauges, measurements with the displacement transducers are rarely influenced by a localized crack even on rock specimens of Class II type.

Since the complete stress-strain relationship may not be obtained when displacements or strains are measured locally, the method for measuring displacement must be chosen taking into consideration the rock type and its failure mode.

3.5.2.2 Loading Rate

Loading rate may range from a very slow rate of $10^{-14} \text{s}^{-1}$, corresponding to the deformation of the earth's crust, to a very high rate, as in rock blasting with explosives. It is however not practical to cover such a wide range of loading rate with a single testing device. Thus, a loading rate of 0.01 to 0.1 %/min. is usually used, and the ISRM\(^{16}\) recommends a loading rate at which the specimen reaches peak strength in 5 to 15min. or from 3 to 60MN/m$^2$/min.

The strength of a rock is influenced by the rate of loading: the slower the loading rate, the lower the strength. However, the effect of loading rate on the strength is small compared to other factors such as cell pressure. The servo-controlled testing machines that are now available enable us to control loading rate accurately. Furthermore, deformation behavior after peak strength has now been investigated in detail with a testing machine with high stiffness.

3.5.2.3 Fracture Strength and Residual Strength

The difference between failure strength and residual strength is defined as the stress drop. The index represents the intensity of the fracture. The smaller the index becomes, the more ductile the rock deformation behavior. When the deformation of the rock specimen becomes ductile, the stress drop is significantly reduced, and it is sometimes hard to pinpoint the peak strength. Once the deformation behavior changes to a strain hardening type, it may become impossible to determine the peak strength. In that case the strength is alternatively defined as the stress at a deformation of 2, 3 or 5%. Figure 5.3 shows how to determine strength-deformation behavior using stress-strain curves.

In general, the amount of the strain up to failure is called “ductility". Handin\(^{16}\) has classified rock-deformation behavior using ductility as follows:

- Less than 1% very brittle
- 1 to 5% brittle
- 2 to 8% moderately brittle (transitional)
- 5 to 10% moderately ductile

![Figure 5.3](image)
More than 10% ductile

The mechanism of the transition from brittle to ductile is not yet well understood. Research into this mechanism will be very important in the future.

3.5.2.4 Loading Control (Strain-Rate Control and Stress-Rate Control)

There are two ways to control the loading rate: by the strain rate control or by the stress rate control. Generally, strain-rate control is used for soft rocks, while both strain-rate and stress-rate controls are used with similar frequency for hard rocks.

Although the ISRM\(^{18}\) suggested that testing should be performed principally under a constant stress rate, a rock specimen tested using stress-rate control fractures suddenly when the stress reaches a maximum value. Thus, the testing machine is unable to measure the rapid deformation of the specimen beyond peak strength. Therefore, testing under a constant stress rate may not be appropriate for the purpose of obtaining a complete stress-strain curve, representing the deformation behavior from the beginning to some point after the peak strength.

On the other hand, the stress rate is not held constant throughout the test when the strain-rate control method is used. The strain-rate control method enables us to observe post-peak deformation behavior. For strain-rate control method, either the axial strain rate or a circumferential strain rate can be controlled. Deformation behavior after the peak strength is divided into two Classes. They are schematically represented in Figure 5.4. Class I deformation behavior is typically strain-softening behavior, while the gradient of stress-strain curve is positive in the case of Class II behavior. In general, using an axial strain-rate control method, it is possible to obtain a complete stress-strain relationship for rocks displaying Class I behavior. However, Class II deformation behavior cannot be obtained using axial strain-rate control, because unstable failures may occur rapidly after the peak strength is attained. To obtain a complete stress-strain relationship of Class II-type rocks, a circumferential strain-rate control method is necessary.

3.6 CALCULATIONS OF TEST RESULTS

3.6.1 Area Correction

The equation of the principal stress difference in the standard (section 6.2(2)) is computed, assuming no volume change in the specimen during the test (volume change \(\Delta V = 0\), or volumetric strain \(\epsilon_v = 0\)). This assumption is valid only for saturated materials with stiffness equal or smaller than that of soft rocks. For materials with higher stiffness, volume change does occur even under undrained condition during axial compression. The assumption therefore does not hold (Kita et al.\(^{24}\)). In case volumetric strain \(\epsilon_v\) (%) is measured during axial compression, the principal stress difference \((\sigma_a - \sigma_r)\) (MN/m\(^2\)) at the axial strain \(\epsilon_a\) (%) is calculated from the following equation:

\[
(\sigma_a - \sigma_r) = 10P \left(1 - \frac{\epsilon_a}{100}\right) A_0 \left(1 - \frac{\epsilon_v}{100}\right)
\]

Where the volumetric strain is calculated using either one of the next two equations:

1) When lateral strain \(\epsilon_l\) is measured:

\(\epsilon_v = \epsilon_a + 2 \epsilon_l\)

2) When volume change \(\Delta V\) (cm\(^3\)) is directly measured

\(\epsilon_v = 100 \frac{\Delta V}{V_0}\)

3.6.2 Correction for the Origin

When the longitudinal displacement of a specimen is
measured externally, e.g., as a movement of the loading ram, the relationship between principal stress difference and axial strain becomes in general S-shaped, as shown in Figure 6.1. One of the reasons for this is bedding error. This is a measurement error due to incomplete contact between the specimen end finished flat and the loading cap (or the specimen end and the pedestal) caused by lack of parallelness between the two. The bedding error also occurs when the specimen ends are somewhat disturbed during sample preparation, and the thin layer of disturbed rock is compressed unevenly (Tatsuoka & Kohata46). As a consequence, the axial strain given by the external measurement includes an amount coming from the bedding error, and measured axial strain is therefore overestimated. To exclude bedding error, it is necessary to perform displacement measurement directly on the side of the specimen.

There are however cases where external displacement measurements may be more practical, e.g., when there is a structural problem in the testing system and internal measurement is not possible, or when only the compressive strength of the material is required. In such cases, the origin must be corrected, as follows. To obtain the strain at maximum principal stress difference, linear portion of the S-shaped principal stress difference vs. axial strain curve just past the inflection point is first extrapolated in the direction of the origin. The intersection with the axis of the axial strain is defined as a new origin. The subsequent axial strains are then corrected by this amount. This procedure is referred to as origin correction. As mentioned above, axial strain evaluated from the external displacement measurement includes an amount for the bedding error in triaxial compression tests on stiff materials such as soft and hard rocks. For this reason, a great care must be taken, since the axial strain at maximum principal stress difference may still be overestimated even after the origin correction.

Figure 6.2 shows an example of a drained triaxial compression test performed on a siltstone retrieved from the Kazusa Group at the tip of the Miura straight, Kanagawa, Japan (Tatsuoka et al.47)). Prior to compression, the samples were isotropically consolidated at a cell pressure corresponding to the in situ vertical stress. In the figure, the principal stress difference-axial strain relationship by the external displacement measurement is S-shaped, while that by the LDT (Local Deformation Transducer, Goto et al.12) measurement (internal) is not S-shaped. Similar results of triaxial compression tests on Akashi sandstone are shown in Figure 6.3 (Kohata et al.26)). In this figure, the principal stress difference-axial strain relationships are S-shaped, whether the displacement is measured internally, or externally. This S-shaped relationship is not due to bedding error, but is considered a reflection of the true rock property. It is therefore not necessary to correct origin for the measurement results on the side of the specimen.

3.7 REPORTING OF RESULTS

The strain rate is calculated from the change in axial strain per unit time obtained from the displacement measurements. It is commonly determined as an average rate in the elastic deformation stage, where the principal stress difference has a linear relationship with axial strain near the origin, just after the
axial loading started.

A Mohr's circle for a specimen can be determined from the values of major and minor principal stresses. A series of Mohr's circles are then collected for different values of minor principal stress, as shown in Figure 7.1. Drawing an envelope of the circles, cohesion and angle of shear resistance can then be obtained.

Method to obtain strength parameters, cohesion $c_u$ and the tangential angle $\phi_u$, from the relationship between maximum principal stress difference and mean stress is as follows. As shown in Figure 7.2, the maximum shear stress is plotted as Y-data, while the mean stress is plotted as X-data. Assuming the relationship between the two is linear, a straight line is fitted by the least square method, and the slope and the Y-intercept are obtained. The strength parameters are then calculated from these values.

3.7.1 Representation and Interpretation of Test Results (Typical Example Results)

Progressive process in brittle failure observed in a rock specimen under triaxial compression has been reported by Brace$^4$ and Bieniawski$^2$, and is summarized here from the microstructure aspects, i.e., with respect to microcrack behaviors (Figure 7.3).

First Stage: With initial application of the principal stress difference, inherent cracks and pores begin to close, producing nonlinear, concave upward stress strain curves.

Second Stage: Stress-strain relationships become linear.

Third Stage: Convex upward on principal stress difference vs. axial strain relationship up to maximum principal stress difference. In this region inherent cracks that closed in the first stage begin to slide. Stress induced cracks begin to form and open up, leading to inelastic volume increase (dilatancy). Volumetric strain shifts from compression to expansion, and the pore pressure changes from positive to negative values, associated with stable increase in crack propagation and crack coalescence.

Forth Stage: Post-peak stress region. Crack growth becomes unstable and the cracks become localized, leading to a macroscopic failure of the specimen.

3.7.1.1 Typical Examples of Soft Rock and Artificial Soft Rock

Figure 7.4 shows a typical principal stress difference vs. axial strain relationship for Tage tuff with uniaxial compression

![Figure 7.4 Stress-strain curve of Tage tuff](Study Committee on UC and TC test methods$^{19}$)

Figure 7.5 shows a typical principal stress difference vs. axial strain relationship for artificial soft rock with uniaxial compression

![Figure 7.5 Stress-strain curve of artificial soft rock](Study Committee on UC and TC test methods$^{19}$)
strength of about 10MN/m² for a mean stress of 0.49MN/m². The axial strain (horizontal axis) is measured on the specimen surface. The principal stress difference vs. axial strain curve is concave upward at low stress levels, and becomes linear at a stress level half the maximum principal stress difference. After stress-strain relationship shows the convex upward portion, the peak stress is reached. The specimen then experiences macroscopic fracture of brittle failure.

Figure 7.5 shows the principal stress difference vs. axial strain curve of an artificial soft rock, with an isotropic stress of 0.49MN/m². For sample preparation of the artificial soft rock refer to the relevant literature[19]. The axial strain is measured on the specimen surface in the same way as Tage tuff. The principal stress difference vs. axial strain exhibits almost linear relationship up to a stress level half the maximum principal stress difference. After that, the principal stress difference vs. axial strain curve becomes convex upward, and finally the maximum stress is reached. In the post-peak stress region, the specimen behaves in a plastic manner, and does not show strain softening.

3.7.1.2 Typical Example of Hard Rocks

Observations by Ibanez & Kronenberg[17] are introduced here as a typical example for shale. The shale comes from the Wilcox formation, Louisiana. The specimen is prepared to a cylinder of 12.4mm in diameter, and 25.4mm in length. Samples are cored and loaded at three orientations: parallel, perpendicular and at 45 degrees to bedding. Samples are sealed with either thin polyolefin shrinkable tube (0.03mm thick) or lead jacket (0.025mm thick). Axial loads are measured internally, with corresponding stresses determined to a precision of 0.3MN/m³. Figure 7.6 shows Mohr's envelopes with respect to the bedding plane and loading directions. The compressive strengths parallel, perpendicular, and at 45 degrees to bedding are strongly dependent on cell pressure. Mohr's envelopes are distinctly non-linear and friction coefficients decrease with increasing cell pressure. Figure 7.7 shows principal stress difference vs. axial strain curves for different orientations. Strengths of shale samples loaded parallel and perpendicular to bedding are greater than those for samples loaded at 45 degrees to bedding. Thus this shale samples show a small but measurable anisotropy. Figure 7.8 shows the principal stress difference vs. the orientation of bedding relationship for Martinsburg slate and shale. Obvious anisotropy is recognized in the shale.
Presented as typical examples for sandstones are the test results on Kimachi sandstone from Shimane prefecture, and those on Sanjome andesite from Fukushima prefecture (Sato et al.38)). Experiments were conducted by applying cell pressure and pore pressure on the cylindrical specimen, and axial principal stress difference was increased until the ultimate strength was attained. Tests were carried out with stroke (displacement) control in the axial direction. Strain rate was kept constant at 3x10^{-4} s^{-1}. Figures 7.9 and 7.10 are the principal stress difference vs. the axial strain curves for the andesite and the sandstone, respectively. The tangential Young's modulus decreased with the increase in principal stress difference. The larger the effective pressure, the greater the Young's modulus and maximum principal stress difference. Figures 7.11 and 7.12 are the principal stress difference vs. the volumetric strain curves for the sandstone and the andesite, respectively. Though distinct dilatancy is observed in any condition, the extent of the effect of cell pressure and pore pressure on dilatancy is not clearly shown. In all the specimens, the pore pressure is decreased with the increase in principal stress difference. This phenomenon as well as AE event count rate confirms the occurrence of stress-induced microcracks in the specimen. Occurrence of microcracks is one of the typical behaviors of rocks under the deviatoric stress, as reported in many references.

3.7.2 Effect of Strain Control Modes on Strength and Deformation Characteristics

A servo-controlled testing apparatus gives options to conduct tests under any arbitrary control conditions to some extent, such as axial strain control or circumferential strain control mode. Here, discussion by Yoshinaka et al.52) will be introduced, in which the differences caused by control modes have been investigated for Inada granite.

3.7.2.1 Stress-Strain-Time Relationships

Figures 7.13 and 7.14 show the relations between principal stress difference, strain and elapsed time. The difference between these figures is the strain component that is under control; constant axial strain rate for Figure 7.13 and constant circumferential strain rate for Figure 7.14. It can be seen in Figure 7.13 that the principal stress difference linearly increases up to a peak value, while the circumferential strain exhibits a rapid increase when approaching the peak principal stress difference and then failure occurs. On the other hand, in Figure 7.14, both the principal stress difference and axial strain increase almost linearly up to near the peak principal
3.7.2.2 Stress-Strain Relationships

The principal stress difference-strain relation under constant axial strain rate is shown in Figure 7.15 and that under constant circumferential strain rate, in Figure 7.16, together with the volumetric strain calculated based on the axial and circumferential strains. Below 50% of the failure strength, in spite of different strain rates, the curves for axial, circumferential and volumetric strains roughly fall on a same straight line, i.e., Young’s modulus and Poisson’s ratio can be regarded independent of strain rate in the range the tests covered.

3.7.2.3 Comparison of the Elastic Constants Measured under Different Control Modes

The comparison of Young’s modulus measured under different control modes is made as shown in Figure 7.17. The Young’s modulus was determined in a strain range of $2 \times 10^{-4}$ to $1 \times 10^{-3}$ and the correspondent stress level of 12 to 60 MN/m$^2$. It is apparent that the Young’s modulus is independent of the control modes.

Concerning Poisson’s ratio, in spite of considerable discrepancies, the effect of the control modes is not remarkable as shown in Figure 7.18. One of the reasons for the discrepancy may be the non-linearity of the radial strain.

3.7.2.4 Axial Strain-Radial Strain Relationship

Under constant axial strain rate, the axial strain-radial strain relation after dilatancy outset is dependent on the strain rate, as shown in Figure 7.19. The higher the strain rate, the smaller the radial strain under the same value of the axial strain.

Under constant radial strain rate, as shown in Figure 7.20, the axial strain-radial strain relation also shows strain rate dependence, but not in a systematic manner. Under 10 MN/m$^2$ cell pressure, stress-induced microcracks are predominantly oriented in the axial direction. Under the constant axial strain rate, as shown in Figure 7.13, the principal stress difference persistently increases up to the peak value. However, under the constant radial strain rate, as shown in Figure 7.14, the increment of the principal stress difference become smaller when approaching the peak value. This suggests that under the constant axial strain rate the energy supplied from external loading is enough for the microcrack propagation. Whereas under the constant radial strain rate, the situation is opposite.
Therefore due to the obstruction of heterogeneity the microcracks propagate and stop intermittently. Thus, the axial strain-radial strain relation behaves in a nonsystematic manner.

3.7.3 Definition of Deformation Modulus

Definition of the deformation modulus is illustrated in Figure 7.21. In this standard, secant modulus and tangential modulus are calculated respectively as $E_{x,50}$ and $E_{x,50}$, for the point on the principal stress difference vs. axial strain curve at 50% of the compressive strength, when required. When the test results with the external displacement measurements are used, deformation modulus is calculated using the relationship between principal stress difference and axial strain, which is corrected for the origin. On the other hand, deformation modulus is calculated directly from the relationship between principal stress difference and axial strain, when the displacement (or strain) is measured on the side of the specimen. Note that the former case contains measurement error, and the calculated deformation modulus is therefore underestimated (e.g., Study Committee on UC and TC test methods).

Should the flatness and evenness of the specimen ends be difficult to be ensured, it is desirable to apply capping material like plaster on the top and bottom specimen ends. It is reported by Study Committee on UC and TC test methods that $E_{x,50}$ obtained for capped material can be larger than that for not-capped material, in case displacement is measured externally. Test results obtained to date are mostly $E_{x,50}$ by the external displacement measurements. In this standard, it is therefore suggested to obtain $E_{x,50}$ if necessary.

Axial strain as in the principal stress difference vs. axial strain varies, depending upon how the origin is determined. Consequently, $E_{x,50}$ calculated as $(\sigma_x - \sigma_o)/\varepsilon_x$ is affected by the origin location. On the other hand, the tangential slope at a certain point on the principal stress difference vs. axial strain...
relationship is calculated as \( d(c* - o*)/d e^* \), and is not affected by the location of the origin. In this standard, \( E_{19} \) is therefore defined for this reason. Note that thus defined deformation modulus too is underestimated for the test data from the external displacement measurements. In addition, it is generally known (Tanaka et al.\(^{45}\)) that deformation modulus is dependent on strain and pressure levels. In calculating deformation modulus, it is desirable to state clearly what values of axial strain and principal stress difference are used.

3.8 A TESTING METHOD NOT INCLUDED IN THE STANDARD (MULTISTAGE TRIAXIAL COMPRESSION TEST)

Starting a triaxial compression test from a low cell pressure, and repeatedly increasing cell pressures in steps once the axial stress reaches the maximum value at the selected cell pressure, it is possible to obtain several Mohr's circles using a single rock specimen. This procedure is called multistage triaxial test, and was originally developed by Kovari & Tisa\(^{39}\).

In a conventional triaxial compression test, several peak-strength and residual-strength measurements are usually carried out for a single cell pressure. Therefore, the test must be conducted under different cell pressures using two or more specimens to investigate the triaxial compressive strength characteristic of the rock. Accordingly, it is recommended that more than four specimens be tested under selected cell pressures. In contrast, the multistage triaxial test uses only one specimen to obtain the triaxial compressive strengths under different cell pressures. It is therefore a very useful method for determining the triaxial strength of a rock, especially when very few rock samples can be retrieved from boreholes because of the poor rock conditions in situ.

The quality of a rock frequently varies in the field, and a rock may possess different strengths locally, even if it comes from the same rock formation. Practically, it is very difficult to obtain uniform rock samples consistently. It is especially hard to obtain a sample from the weaker part of a rock formation with a low RQD, but the strength data for the weak rock may be the most important for the design and construction of structures in that rock mass. The multistage triaxial compression test using only one specimen can be a helpful method for reducing the number of necessary drill holes, as well as for the accurate and safe design and construction of a rock structure.

The following problems must be considered in the application of a triaxial compression test:

1) Strength may be underestimated due to stress history.
2) The multistage triaxial test is an alternative to the conventional method.
3) The strength measurement obtained from multistage triaxial test using a single specimen may or may not be a valid representation of the strength of the rock.
4) A method for judging peak strength has not yet been established.
5) It is currently hard to control the testing machine at stresses near the peak strength.
6) Human error in machine operation may generate invalid data.

3.8.1 Stress Path in a Multistage Triaxial Test

As shown in Figure 8.1, there are two principal stress paths in a multistage triaxial compression test. In the first, the cell pressure increases in steps after axial stress reaches peak strength (Path 1), and in the second, the cell pressure is decreased to a hydrostatic state after the peak strength is reached and before the cell pressure is increased to the next selected pressure (Path 2). The peak strength for each cell pressure is defined as the strength at that cell pressure in both
procedures. In the multistage triaxial compression test, axial loading under maximum cell pressure is continued after peak strength is attained to obtain a complete stress-strain relationship. The cell pressure is subsequently reduced to a selected minimum pressure, and the axial stress is again increased while maintaining that cell pressure to obtain the residual strength. Figure 8.2 shows the result obtained on sandstone from a multistage triaxial compression test using Path 1. The maximum cell pressure was 20MN/m² in this test.

In the test shown in Figure 8.2, a Hoek-type triaxial cell was used. The Hoek cell was developed for the testing of hard rocks; it is schematically shown in Figure 8.3. The membrane (rubber sealing sleeve) used in the Hoek cell is made from a relatively hard rubber 1–2mm thick with upper and lower flanges. When installing the membrane into the triaxial cell, the membrane is first inserted into the cylinder while expanded, and it is then turned down at far end to make the flange. The membrane is the most critical element of this cell, and only the specimens are replaced for the succeeding tests. In triaxial compression tests, the Hoek cell is mounted on a loading frame.

3.8.2 Determination of Peak Strength
Several methods for judging whether or not axial stress has reached the peak strength have been suggested, but none has yet been accepted as a standard. One proposed method uses the acoustic emission (AE) count rate to define the peak strength, when the AE count rate exceeds a predetermined level. Other methods set the peak strength at the point when a stress drop of more than 0.05MN/m² is observed or when the ratio of increase in principal stress difference to that of lateral strain decreases to a predetermined value.

3.8.3 Examples of Multistage Triaxial Compression Test
In this section, examples for hard and medium-hard rocks are described.

Kovari & Tisa²⁹ tested sandstone and marble under various cell pressures. The results are shown in Figures 8.4 and 8.5. As shown in Figure 8.4, the peak strength is independent of stress history, and is a function of cell pressure only.
Furthermore, as shown in Figure 8.5, the peak strength is also independent of stress history. Therefore, it may be concluded that the multistage triaxial compression test should be applicable for obtaining the peak strength and residual strength of hard rocks such as sandstone and marble.

On the other hand, Chang & Jumper conducted multistage triaxial compression tests on moderately ductile oily-shale using three different stress paths. The stress paths used in the tests are shown in Figure 8.6. For the stress path shown in Figure 8.6(a), it was very hard to change the cell pressure, and consistent results were not obtained. However, the test results with the stress paths shown in Figures 8.6(b) and (c) are consistent with those obtained using the conventional triaxial tests. Therefore, although various types of stress paths can be used in a multistage triaxial test, some paths may not be suitable for given rock type for consistent results. Akai et al. applied the multistage triaxial compression test to a very porous sandy silt (porosity = 54.4%) with high water content, w = 48%. For this soft rock, it was very hard to stop the axial loading just before peak strength was attained before changing the cell pressure. The actual procedure employed was to reduce the axial stress to a hydrostatic state just before peak strength was reached and then to change the cell pressure. This way, they were able to obtain the consistent results.

References


21) JGS: Geotechnical engineering handbook, Ch. 5: Rock mechanics, Sec. 5.6: Physical property mechanics, JGS, pp.274-280, 1999 (in Japanese).


