

EVALUATION OF THE DEFORMABILITY OF ROCK MASSES FOR DAM FOUNDATIONS

**Analysis of deformability investigation
results of heterogeneous bedrock**

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EVALUATION OF THE DEFORMABILITY OF ROCK MASSES FOR DAM FOUNDATIONS

Analysis of deformability investigation
results of heterogeneous bedrock

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Synopsis :

Generally, in-situ deformability tests are performed to evaluate the deformability of the rock foundation for large-scale structures. The representative volume of bedrock used to evaluate the deformability depends on its degree of heterogeneity and discontinuity. If the tested volume is smaller, then the test results are widely scattered. We should therefore conduct a large number of tests to improve the reliability of the evaluation. On the other hand, when the tested volume is too much larger, the reliability of the evaluation is high, but it is not recommended from the economical viewpoint.

In this memorandum, we first review several types of deformability tests presently conducted in the world. Then we analyse the results of in-situ deformability tests performed for a heterogeneous conglomerate. Particularly as concerns the results of plate loading tests, we analysed the relation between the heterogeneity of bedrock and the deformability coefficient in the appendix.

Key Words :

Dam, Foundation, Rock mass, Deformability, In-situ test, Plate loading test, Heterogeneity, Scale Effect



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EVALUATION OF THE DEFORMABILITY OF DAM FOUNDATIONS IN ROCK MASSES

1. INTRODUCTION

When analysing the behaviour of dams, the dam - foundation as a whole and, for some problems, the whole dam - foundation - reservoir should be considered. For the design and monitoring of dam foundations, different levels of information about rock and rock mass properties are required according to the importance of each structure and its type, whether concrete and masonry, or earth and rockfill type. The main aspects to consider are the deformability, strength, watertightness and durability.

It is important to evaluate the parameters concerning the deformability of rock masses for dam foundations. However, due to the heterogeneous and discontinuous nature of rock masses, the evaluation of the parameters is largely influenced by the dimensions of the tested volumes.

The deformability characteristics of a rock mass are usually determined by means of in-situ tests, in which a load is applied in a specific manner, and the corresponding rock mass deformations are measured. To assess these characteristics, several types of tests are conducted such as in-situ and laboratory tests for the rock and discontinuities, which make it possible to obtain a satisfactory knowledge of the rock mass behaviour during and after the execution of the projects.

In some cases, the rock masses are only homogeneous on such a large scale that it is not economically advisable to perform tests involving such a large volume of rock mass in general. However, an insufficient test volume causes problems of scale effects¹⁾.

After Grossmann²⁾, the basic scale effect in the deformability evaluation can be easily explained by the theory of statistics as follows. Whenever a rock mass deforms, the global deformation in a certain direction is the sum of a large number of small aleatory individual deformations in the same direction, of the different constitutive elements of the rock mass. Thus, from the laws of statistics, the global deformation has a normal distribution, whose mean value is independent of the number of summed individual deformations. Therefore, if the tests performed are chosen randomly, the experimental results should always present the same mean deformability and a standard deviation proportional to the square root of a significant length of the tested volume.

From these considerations, the deformability results obtained from small-scale tests are much more variable than those from large-scale tests. Thus, a sufficient number of tests should be performed in order to compensate for the variability.

The tests for determining the deformability of rock foundations must be performed prior to design. During construction or the first filling of the reservoir, it is possible to obtain the response of the whole dam-foundation by monitoring and comparing the observed results with the predictions of numerical models. The

reliability of the models depends fundamentally on the parameters that govern the behaviour of the materials contained in the body of the dam and in the foundation. Thus, in this stage, it is important to use system identification techniques in order to re-evaluate the deformability parameters of the rock mass by taking into account the results obtained from monitoring and previous field tests^{3),4)}.

In this report, the common field tests usually performed for characterisation of the deformability of rock masses as dam foundations are described. These are the small-scale tests referred to in section 2 and the large-scale tests in section 3. Special mention is made of the problems that arise in their application and also the techniques used by LNEC, which have been involved in numerous rock mechanics studies concerning dam foundations and other geotechnical structures, both in Portugal and abroad.

These considerations were included in the interpretation of deformability tests for heterogeneous bedrock in section 4. Lastly, section 5 considers the scale effect problem, and describes a methodology considered more adequate for determining deformability and interpreting the deformability tests for heterogeneous bedrock.

This research was carried out at the Fill Dam Division, Dam Department of the Public Works Research Institute (PWRI), Tsukuba, Japan by a Principal Research Officer of the National Laboratory of Civil Engineering in Lisbon (LNEC).

2. BOREHOLE TESTS

2.1 Introduction

Deformability tests performed on a small area for dam foundations are usually executed in boreholes, because the tests are relatively economical to perform and in general the boreholes have already been drilled in order to survey the rock mass. Moreover, preparation of the borehole walls in the test stretch is not required.

The borehole deformability tests can be grouped into the following two main types^{2),5)}:

- i) the pressure load is applied by means of a flexible membrane completely adapted to the borehole walls with a rotationally symmetric pressure (referred to as dilatometers);
- ii) the pressure load is applied by means of rigid plates to two arcs of the borehole's circumference (referred to as borehole jacks).

In the first type of test, we have to consider the pressuremeters in which usually a global volumetric deformation is measured. These are used mainly for soil and weak rock behaviours and have accuracy limitations, because they measure volumes and not displacements, and so are not appropriate for all types of rock masses⁶⁾.

In Japan, development of the pressuremeter began in the latter half of the 50's, and the monocell type 'Public Works Research Institute K value Tester' was developed. This version was subsequently improved in the so-called LLT (Lateral Loader Tester) apparatus. Improvements were introduced in monocell and tricell methods.

Other equipment used for the first type of test are dilatometers with direct measuring devices in which several radial deformations can be obtained.

The second type of test corresponds to a more complex theoretical load situation, such as in the case of the Goodman jack. The greater robustness of this equipment in comparison with the dilatometer is why it is still in use⁵⁾.

According to LNEC's experience, the most suitable borehole tests for rock mass deformability characterisation are dilatometer tests. The following section describes this type of test and outlines the equipment developed by LNEC.

2.2 Dilatometer tests

The LNEC BHD (BoreHole Dilatometer) dilatometer consists of a steel body, enveloped by a rubber jacket, which transmits pressure to the borehole walls. It was designed to carry out deformability tests for rock masses in NX (76 mm diameter) boreholes, operating under normal conditions until 150 m deep (Fig. 1). The pressure is obtained by pumping water, which can reach 20 MPa. The deformation is measured along four diameters, 45 degrees apart, with the help of 4 pairs of sensors, which are connected to differential transducers^{2),7)}.

The deformability modulus E_d , also called dilatometer modulus, is evaluated by assuming a continuous, homogeneous, isotropic and linear elastic medium, according to the Theory of Elasticity for a borehole of infinite length with a uniform pressure acting on its wall:

$$E_d = (1 + \nu)\phi \frac{\Delta p}{\delta} \quad (2.1)$$

where ν is Poisson's ratio, ϕ the borehole diameter, Δp the pressure range and δ the diametrical deformation caused by Δp .

These tests are easy, quick and inexpensive, but the tested volumes are small, between 0.5 to 1 m³, and are thus not usually representative of an elementary volume of the rock foundation.

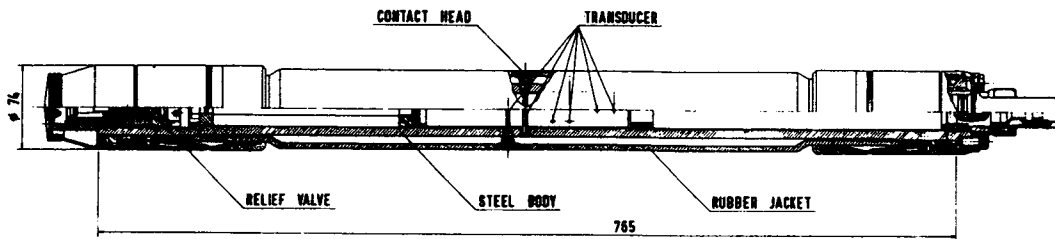


Figure 1. Borehole dilatometer BHD

2.3 Interpretation of dilatometer test results

Interpretation of the dilatometer tests is rather difficult, due to variations in the behaviour of the rock mass during tests. The formula given above, derived from the Theory of Elasticity, is only valid if the circumferential stresses in the borehole walls after applying the pressure remain compressive. In general, the initial stress in rock mass foundations is small, and the tensile circumferential stresses induced by the applied pressure are usually higher than the initial circumferential stress in the borehole walls due to the initial stresses. Thus, the deformability modulus obtained from this formula is not usually the true elastic modulus, but is the modulus of elasticity of an equivalent isotropic and homogeneous medium, with the same deformation as was measured during the test for the same load.

Several assumptions can be made for BHD tests, which can be classified as follows:

- i) the initial stresses of the rock mass are non-zero, and the tensile strength is non-zero;
 - ii) the initial stresses of the rock mass are non-zero, and the tensile strength is zero;
- or
- iii) the initial stresses of the rock mass are zero or nearly zero, and the tensile strength is non-zero.

Pinto⁷⁾ presented theoretical formulas which can be used to calculate the true modulus of elasticity of the rock mass, in cases where Eq. (2.1) is no longer valid. However, diagrams obtained by in-situ tests are not generally similar to the theoretical diagrams according to the formulas referred to above, because the effects are mixed with others, which disguise them completely. Thus, crack closure, heterogeneity, jointing, creep, etc., may alter completely this type of diagram. Only a continuous recording from the beginning of the tests will show every detail of the test curves. Sometimes, the difficulty of fitting the rubber jacket to the borehole walls may make it

difficult to obtain readings under low pressure.

For each load level, the dilatometer test provides results in four radial directions in a plane normal to the axis of the borehole. These different results may be influenced by the anisotropy of the rock mass mainly due to some aleatory discontinuity around the borehole. Thus, information concerning the anisotropy of the rock mass may be obtained from the dilatometer tests by taking the mean of the results for the same direction, from all the tests performed in a homogeneous zone of the rock mass²⁾.

According to our experience and the theoretical developments referred to previously, the deformability modulus obtained from Eq. (2.1) is usually smaller than the value obtained in large-scale field tests^{8),9)}. LNEC used an adequate methodology for determining the deformability, and this methodology will be presented in the final section.

3. LARGE SCALE DETERMINATIONS

3.1 Overview

Large-scale deformability tests can be divided into the following types:

- i) surface tests in which the load is applied on the natural ground or on a surface in an adit;
 - ii) flat jack tests in which the load is applied on the walls of one or more slots specially opened;
 - iii) seismic tests (refraction) between drilled holes or galleries;
- or
- iv) in-situ biaxial or triaxial tests.

Seismic tests can determine the dynamic moduli, by measuring the velocities of the P and S waves. The dynamic values are different from the static ones due to differences in duration and applied stress levels. In general, the dynamic values are high for the deformability moduli due to the smaller influence of joint properties compared with the corresponding static ones. Depending on the distance between boreholes or galleries, dynamic tests involve a large volume and can be correlated with large-scale static tests.

Biaxial or triaxial tests are rarely used due to the high costs involved.

Groups i) and ii) are mostly used for characterisation of the deformability properties of rock mass dam foundations, especially in the case of designing of dams with deformation of the foundation bed taken into consideration.

3.2 Plate loading tests

In these tests, the load is applied on an existing surface, but the response of the undisturbed rock mass to the applied loads is mixed with a superficial zone of the rock mass disturbed by the superficial stress release. Sometimes to avoid this, only the deformation occurring at a certain distance from the load surface is measured, but in this case, the measured deformations are smaller, and therefore the results are not very accurate.

According to the Japanese manual on the design of dams, the artificially loosened part of the tested surface must always be removed, which is a disadvantage of these tests.

In-situ tests for obtaining the deformability of a rock foundation should be based on a jack test¹⁰⁾. The rock surface is loaded by using a hydraulic jack through a loading plate, and the displacement of the rock foundation is measured.

If the load is applied to a rigid-disk type plate to obtain a uniform displacement behaviour, the deformability modulus is given by

$$E_r = \frac{(1 - \nu^2)(F_2 - F_1)}{2a(W_2 - W_1)} \quad (3.1)$$

If the load is applied by a flat-jack to obtain a uniform distribution load, the modulus is given by

$$E_r = \frac{2(1-\nu^2)(r_2 - r_1)(p_2 - p_1)}{W_2 - W_1} \quad (3.2)$$

In Eqs. (3.1) and (3.2), E_r is the deformability modulus, a the radius of the rigid disk, ν Poisson's ratio, F_1 and F_2 the load value of two points in a load vs displacement curve, p_1 and p_2 the pressure value of two points in a pressure vs displacement curve, and W_1 and W_2 the displacement values corresponding to the pairs F_1 and p_1 , and F_2 and p_2 , respectively.

In Japan, three coefficients are usually determined:

- i) coefficient of deformation D ;
- ii) tangential coefficient of elasticity E_t ;
- and
- iii) secant coefficient of elasticity E_s .

These are obtained with experimental values for a steep straight line using a bearing method¹⁰⁾.

LNEC has also extensive experience in using these types of tests, but since the 70's, large flat jacks tests have been mainly carried out. Figure 2 shows a double-plate bearing test with two hydraulic jacks applied to the walls of a gallery following a scheme normally used for dam foundations, with a loading area of about 1 m².¹¹⁾

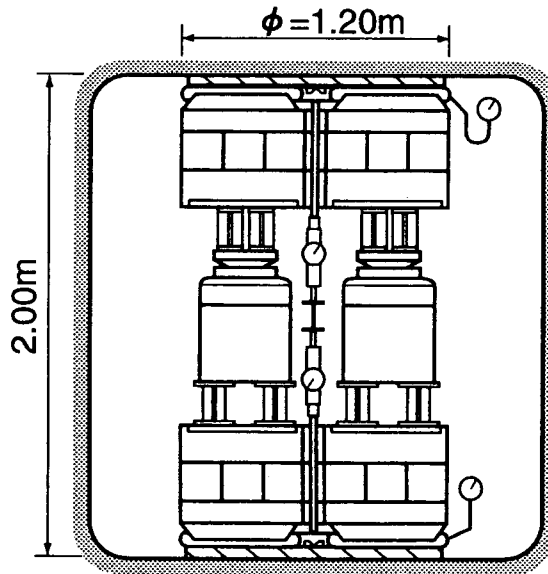


Figure 2. Plate loading test arrangement used at LNEC

3.3 Flat jack tests

By applying the load on the walls of a slot, the deformability of large volumes of rock masses in the undisturbed conditions can be assessed. LNEC considers that this is the most appropriate large-scale test for evaluating rock mass deformability.

An example of this type of test is the LFJ (Large Flat Jack) test developed by LNEC and widely used by

this Portuguese institution in numerous studies, and it is also applied in weak rocks with values of deformability between 120 MPa and 2000 MPa¹²⁾.

The advantage of this particular test in comparison with other slot tests is that the slots are opened by a diamond-disk saw of 1 m diameter. The slot surfaces are generally smooth, and the jacks can apply the pressure directly onto the rock surface. The flat jack contains four deformaters, with electric strain gauges, which measure the variation of the slot opening. These jacks can be placed in slots opened side by side and tested either simultaneously or one by one, thus allowing the test to be conducted on a volume big enough to be representative of the rock mass. Figure 3 shows an arrangement of three flat jacks placed side by side.

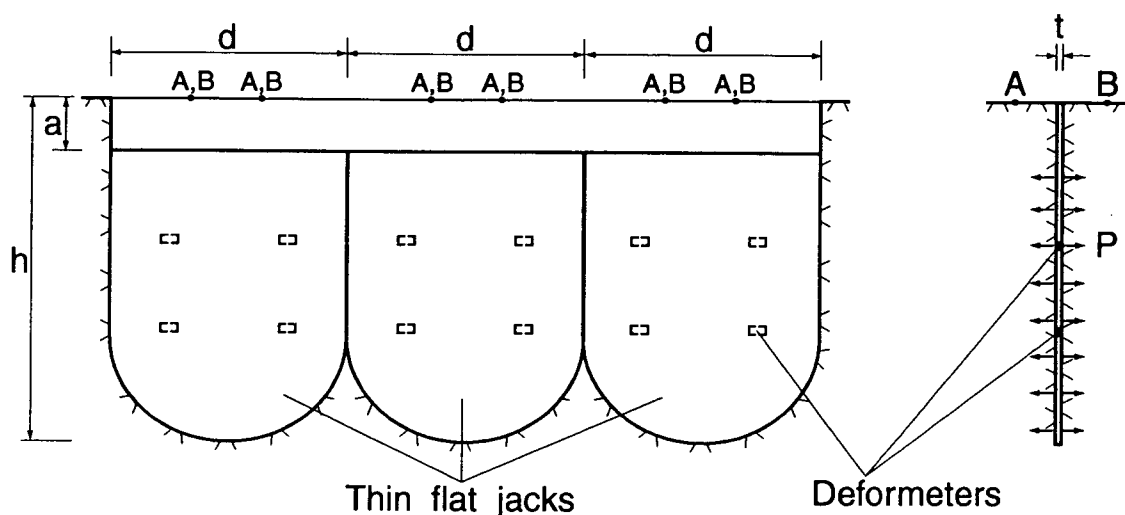


Figure 3. Large flat jack arrangement

The tests are usually interpreted using the Theory of Elasticity for the half-space, with a distributed normal load:

$$E_{LFI} = (1 + \nu^2) K \frac{\Delta p}{\delta} \quad (3.3)$$

where

E_{LFI} - deformability modulus

ν - Poisson's ratio

K - constant to be determined

Δp - pressure change

δ - deformation caused by Δp

The constant K is calculated by numerical methods and is a function of the number of LFJs tested, the chosen deformer and the depth of a crack that often develops in the plane of the slots during the tests, due to the tensions occurring there¹²⁾.

Recently, Pinto⁷⁾ proposed a new LFJ arrangement, in which each slot contains two independent jacks of

about half of the size of the former LFJ jacks, one on each side of the central borehole (168 mm diameter) (Figure 4). The deformeters are placed in three independent measuring columns, installed in the central borehole and in two NX size (76 mm diameter) boreholes. The advantages of the new arrangement are:

- i) the total testing time is reduced;
 - ii) the cost of the tests is reduced;
 - iii) the tests provide better information;
 - iv) the number of deformeters is multiplied by three;
- and
- v) long term creep tests can be performed more easily.

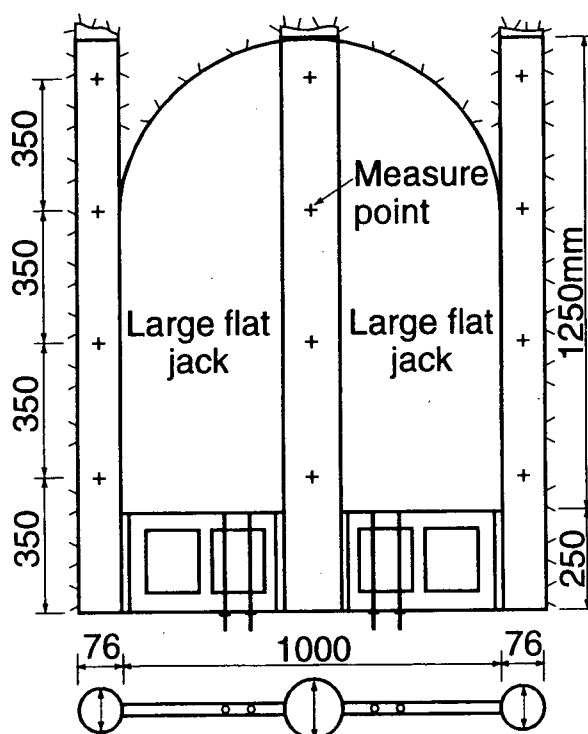


Figure 4. New Large flat jack setup

Another flat jack technique developed by LNEC is the SFJ (Small Flat Jack) method, using small-scale jacks, which is mainly used for stress measurements (Figure 5), but it can also determine the deformability of the rock mass using a formula obtained by a 3D finite element model assuming the rock mass as an isotropic or an anisotropic medium¹³⁾. For an isotropic medium, the modulus of deformability is obtained by

$$E_{SFJ} = K \frac{\Delta p}{\delta} \quad (3.4)$$

where E_{SFJ} is the deformability modulus, K a constant obtained through the finite element model, Δp the cancellation pressure and δ the measured displacements between pairs of points.

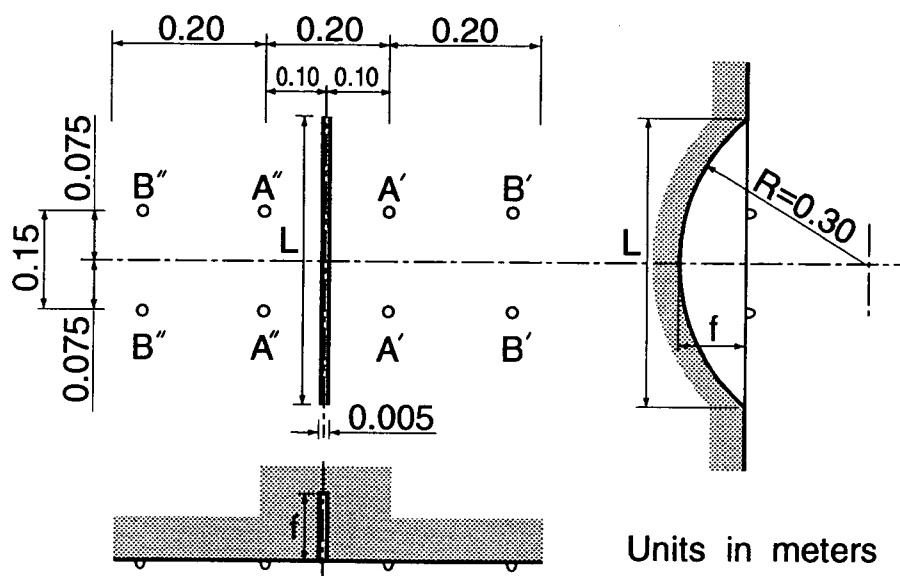


Figure 5. Small flat jack test. Geometry of a slot

4. DEFORMABILITY TESTS FOR HETEROGENEOUS ROCK FOUNDATION

4.1 Introduction

A study was performed on the deformability of a heterogeneous rock foundation.

An adit of about 80 m deep was dug, with the designation of TA-3 (Figure 6). A small forked adit TA-3', whose axis is normal to the adit TA-3, was also dug. In these adits several in-situ deformability tests were performed: large deformability tests, with two kinds of plate loading tests, and small-scale tests with pressuremeter tests. Some laboratory tests were also carried out, including uniaxial compression tests to determine the deformability modulus and uniaxial compressive strength, and measurement of the velocities of waves and specific gravity.

The rock mass consists of lapilli tuff, where only pressuremeter tests were carried out, and conglomerate. The plate loading tests were performed only in this last type of rock formation.

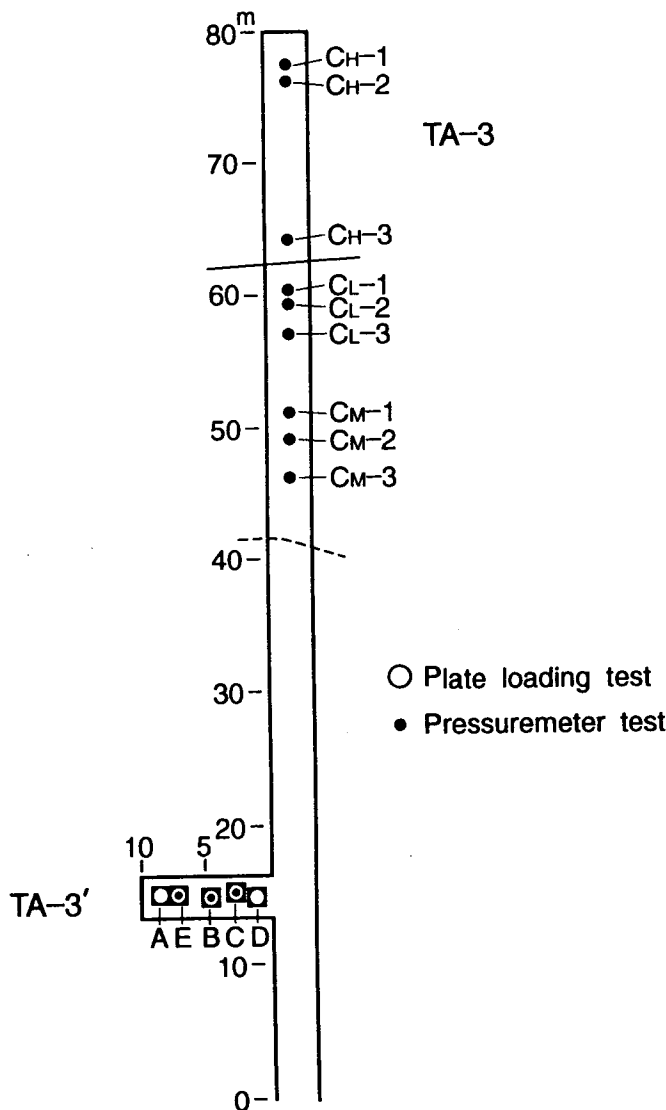


Figure 6. Adits TA-3 and TA-3'. Localisation of the in situ tests

4.2 Plate loading tests

The plate load tests were performed in the adit TA-3', 10 plate loading tests being carried out with rigid plates of 30 and 60 cm diameter. Tests using the two diameters were carried out in 5 places, designated A, B, C, D and E (Figure 7).

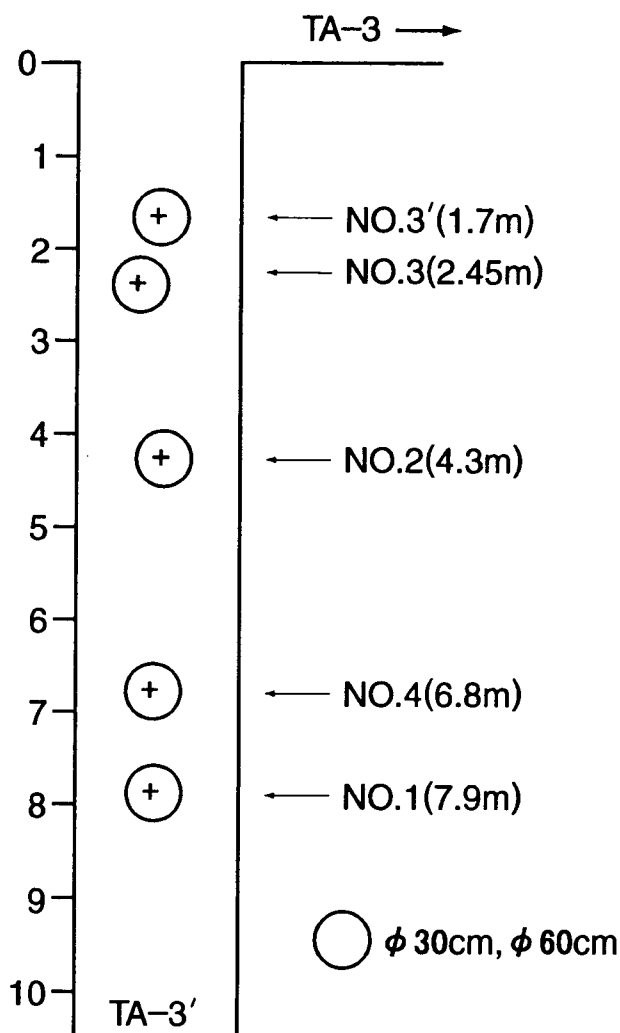


Figure 7. Adit TA-3'. Localisation of the plate loading tests

The loading pattern used was similar to that described in the Japanese manual¹⁰, using 4 incremental steps of 0.8, 1.6, 3.2 and 4.8 MPa, with 4 repeated cycles for the last increment of load, the last cycle being an endurance loading carried out for about 6 hours (Figure 8). In the rigid plate, 4 measurement points were placed in the circumferential border. The coefficient of deformation D , tangential coefficient of elasticity E_t and secant coefficient of elasticity E_s were calculated according to the manual and Eq. (3.1) referred to in section 3.2. Figure 9 shows an example of a load vs displacement curve followed during the tests, and the standards used for calculating the deformability coefficients.

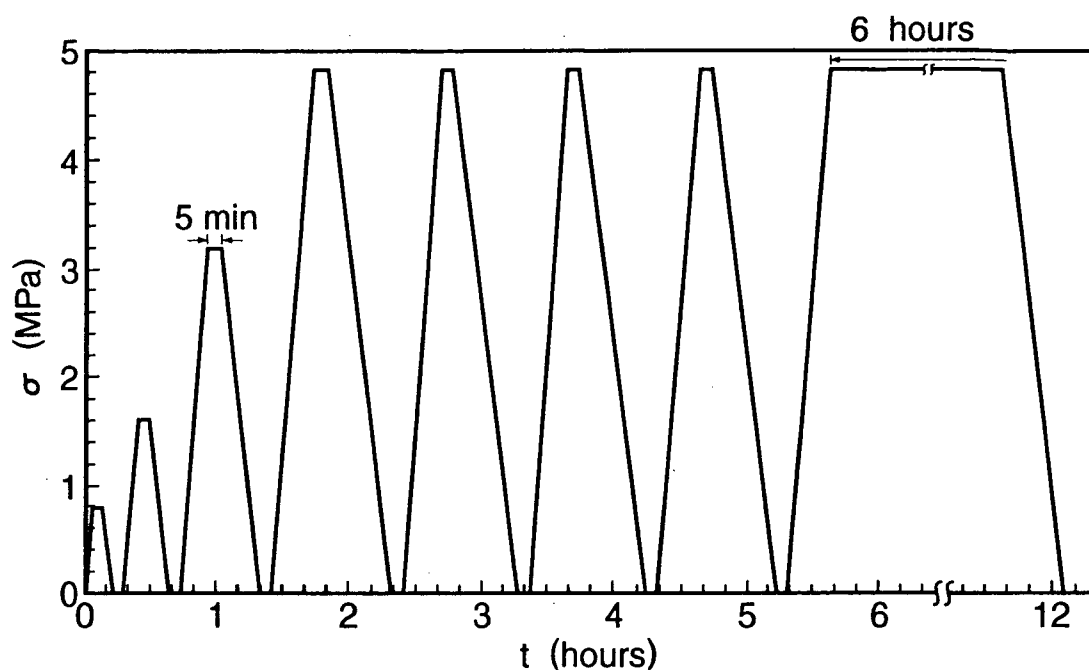


Figure 8. Loading pattern for the plate loading tests

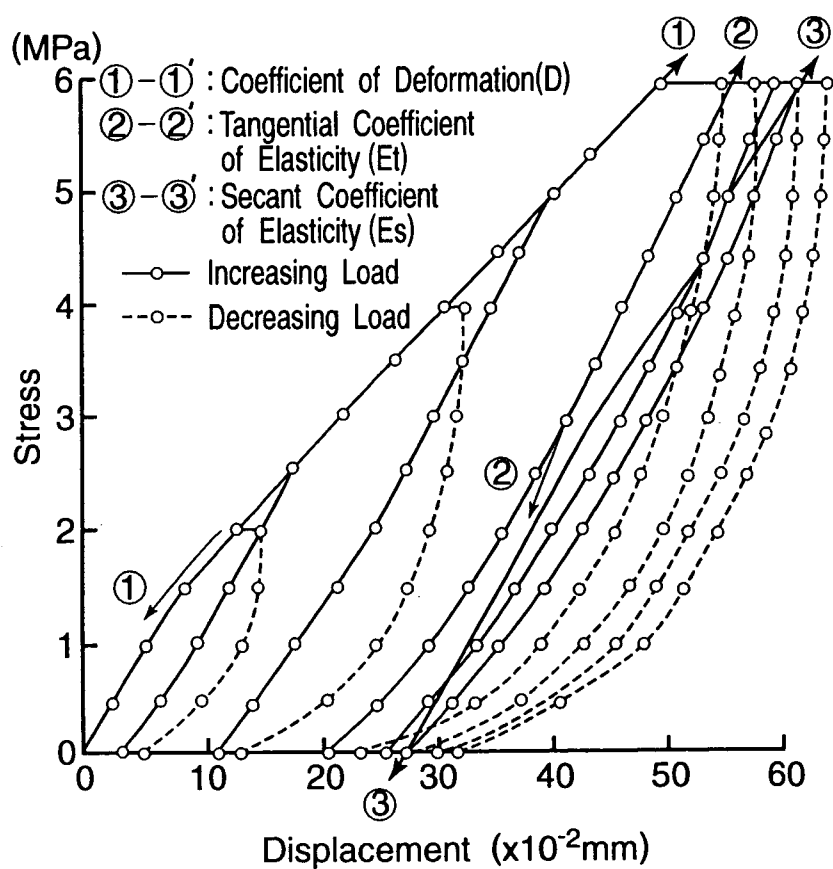


Figure 9. Example of a load vs displacement curves

Figure 10 shows the rock formations for the different plate loading tests, which makes it possible to visualise the heterogeneity of the rock mass in the scale of the tests.

The plate loading test results are summarised in Table 1, the deformability values being expressed in MPa. In order to calculate the coefficient of deformation D , the range of pressure considered was from 2 MPa (test B), or 2.2 MPa (tests C, D and E), or 2.4 MPa (test A), to 4.8 MPa (for all tests). E_t was calculated for a lower pressure between 1.0 and 2.8 MPa and for a higher pressure between 4.0 and 4.8 MPa. E_s was always determined for the range of pressure 0-4.8 MPa.

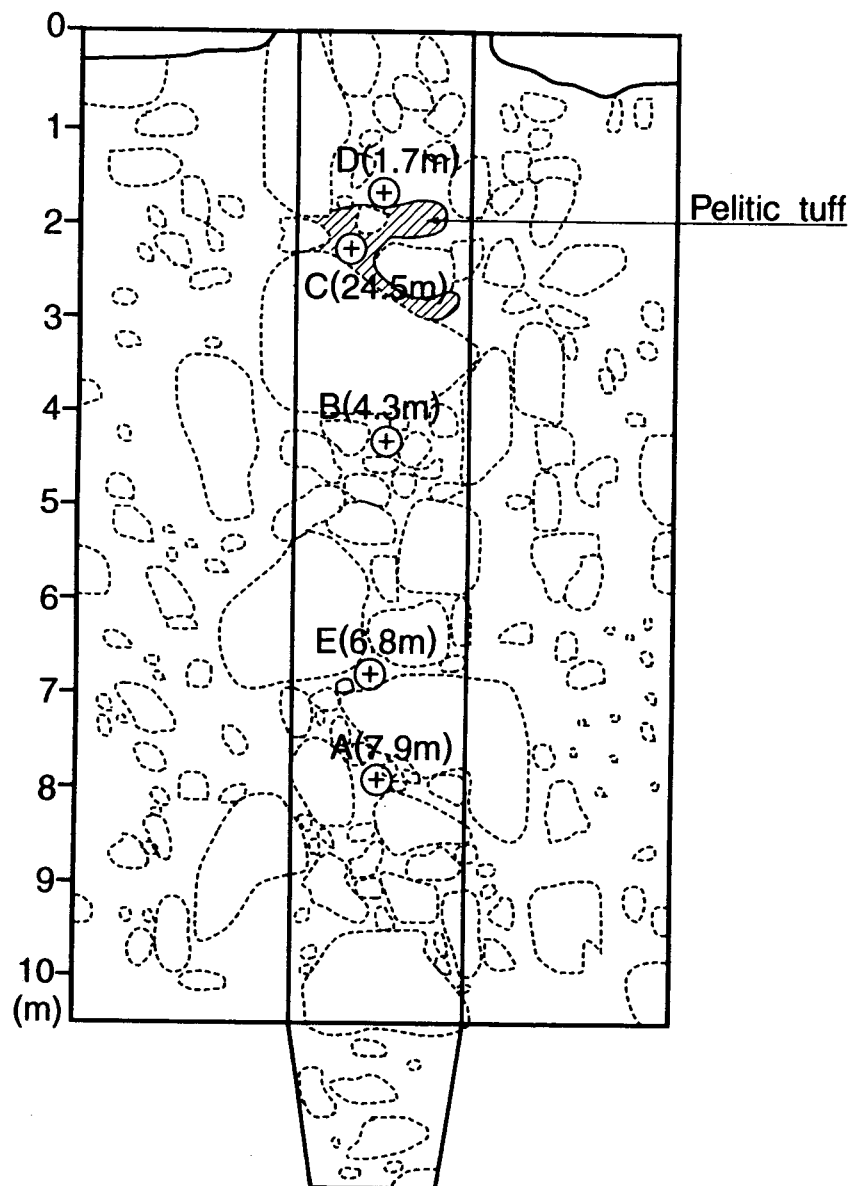


Figure 10. Rock formation heterogeneity involved by the plate loading tests

Table 1. Plate loading test results

Test	30cm plate			60cm plate		
	D (MPa)	E_t (MPa)	E_s (MPa)	D (MPa)	E_t (MPa)	E_s (MPa)
A	578	1014	818	886	1307	906
B	413	898	639	736	1159	789
C	47	439	410	487	953	641
D	634	1119	852	914	1413	931
E	736	1225	1019	1243	1667	1511

4.3 Pressuremeter tests

Several pressuremeter tests were carried out for a 66 mm diameter borehole, 6 tests corresponding to the lapilli tuff, the classes C_M and C_H on adit TA-3, and 6 more in a conglomerate rock mass, 3 being in the same location of plate loading tests B, C and E (adit TA-3'), and the other 3 tests performed in class C_L , between classes C_H and C_M , on the adit TA-3 (Figure 6).

Some general results of these tests are presented in Table 2. Regarding the tests included in the class C_L , in conglomerate, the deformability coefficients were; for D : mean 247 MPa, maximum 690 MPa and minimum 30 MPa; and for E_t : mean 730 MPa, maximum 1240 MPa and minimum 210MPa. For classes C_M and C_H , in lapilli tuff, the mean values for the deformability coefficients D and E_t were considerably higher; for C_M : 1717 MPa and 4520 MPa; for C_H : 2450 MPa and 8950 MPa; and for C_M+C_H : 2083 MPa and 6735 MPa.

Table 2. Pressuremeter test results

Test	Class	D (MPa)	E_t (MPa)
C _L -1	C _L	30	210
C _L -2	C _L	350	890
C _L -3	C _L	60	300
C _M -1	C _M	1260	3890
C _M -2	C _M	2160	6900
C _M -3	C _M	1730	2770
C _H -1	C _H	1870	13000
C _H -2	C _H	2890	5200
C _H -3	C _H	2590	8650
B	C _L	220	1240
C	C _L	130	540
E	C _L	690	1200

4.4 Laboratory tests

Various laboratory tests were performed. To evaluate deformability, uniaxial tests were carried out, and the modulus of elasticity E and also the uniaxial compressive strength σ_c were obtained. Table 3 shows the results of those tests.

For the elasticity moduli, a mean value of 199 MPa was obtained, with a maximum of 304 MPa and a minimum of 98 MPa. For the uniaxial compressive strength a small mean value was obtained of 1.4 MPa.

The deformability moduli obtained were generally lower than the values obtained with the in-situ plate loading tests.

Table 3. Laboratory uniaxial test results

Test	depth (m)	q (MPa)	E (MPa)
B	0.45–0.55	1.3	159
B	0.65–0.85	1.3	304
C	1.40–1.55	2.0	294
C	1.70–1.80	1.6	98
C	1.80–1.95	0.9	139

4.5 Overall analysis of results

All the deformability tests available in the same rock mass zone (in conglomerate, class C_L) are summarised in Table 4.

Table 4. Deformability moduli

Test	Deformability coefficients	maximum (MPa)	mean (MPa)	minimum (MPa)
Plate test (60cm)	D	1243	853	487
	E_r	1667	1300	953
Plate test (30cm)	D	736	482	47
	E_r	1225	939	439
Pressuremeter	D	690	247	30
	E_r	1240	730	210
Laboratory uniaxial test	E	304	199	98

Table 4 immediately suggests that both coefficients D and E_r decrease with reduction in test volume in the field tests performed. This is probably due to the fact that the tests were performed in classes where lower deformability coefficients were expected, and they were conducted this way for safety reasons (Figure 10). When

the test volume increases, as happened in plate loading tests with the 60 cm diameter rigid plate, the influence of the rigid blocks is more important. In fact, in the test scale used for the small plate loading tests, the rock mass cannot be considered as homogeneous (Figure 10).

The comparative analysis shows that the in-situ tests performed with a small diameter plate cannot represent the behavior of the rock mass if a conceptual model with a continuous approach is used. Thus, the mean deformability value for each coefficient obtained with the large-scale plate loading test is certainly closer to the real mean deformability value of the rock mass in the tested zone, but is probably still a lower value.

The results of the in-situ tests were compared with each other and reasonable correlations were found. Figure 11 compares deformability coefficient D ; The Figure shows the pairs between plate loading tests with a 30 cm rigid plate vs pressuremeter tests (D_{30}/D_d), plate loading tests with a 60 cm rigid plate vs pressuremeter tests (D_{60}/D_d) and plate loading tests with a 60 cm plate vs plate loading tests with a 30 cm plate (D_{60}/D_{30}). The gradients of straight lines in Figure 11 were obtained by using the method of least squares as follows:

$$\begin{aligned} D_{30} &= 1.12D_d \\ D_{60} &= 2.00D_d \\ D_{60} &= 1.61D_{30} \end{aligned} \quad (4.1)$$

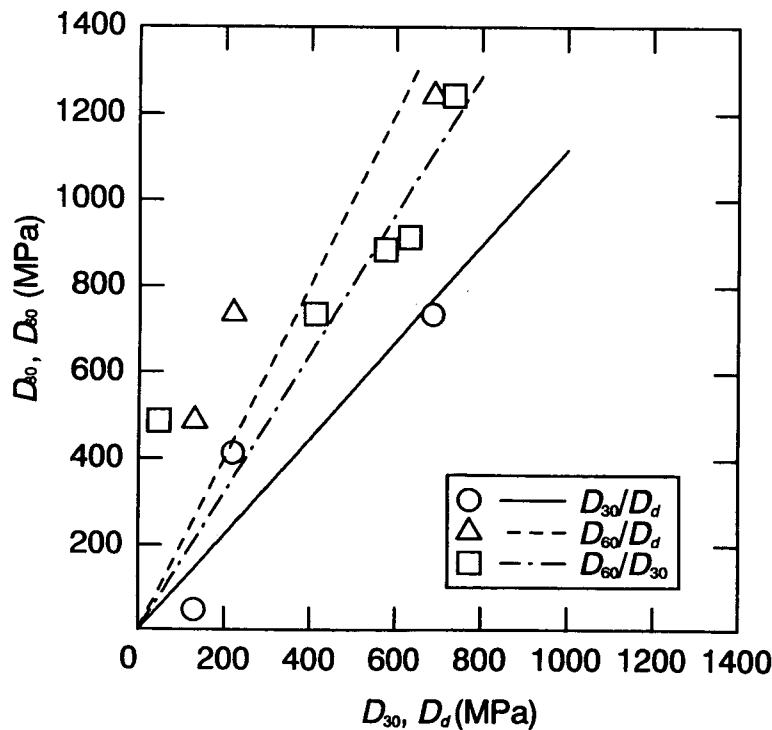


Figure 11. Comparisons between in situ deformability coefficients

Figure 12 compares tangential coefficients E_t and the similar correlations were found. The correlations for this coefficient were as follows:

$$\begin{aligned} E_{t30} &= 0.86E_{td} \\ E_{t60} &= 1.21E_{td} \\ E_{t60} &= 1.34E_{t30} \end{aligned} \quad (4.2)$$

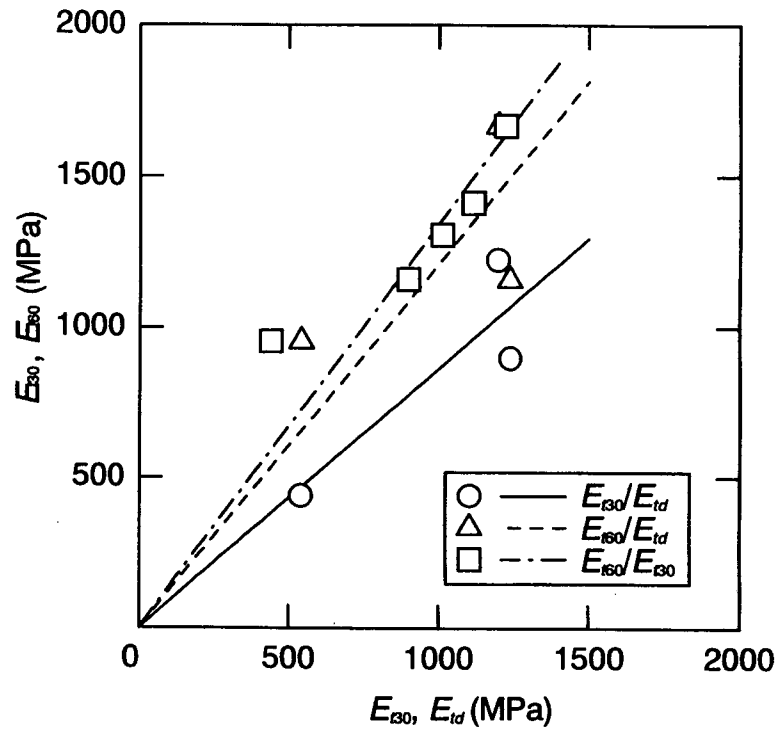


Figure 12. Comparisons between in situ tangential coefficients of elasticity E_t ,

5. FINAL REMARKS

5.1 Scale effects

Rock masses can be generally described as heterogeneous and discontinuous media, and their mechanical properties depend on both the rock material and the discontinuity sets. However, for extreme cases of very weak rock masses, as was studied for the heterogeneous rock foundation, the influence of the discontinuities on the mechanical properties of the rock mass, deformability and strength, is not so relevant.

Thus, due to the heterogeneous and discontinuous nature of rock masses, the evaluation of their mechanical characteristics is influenced by the dimension of the tested volumes. The problems caused by insufficient test volume have been analysed within the scope of the ISRM Commission on Scale Effects in Rock Mechanics.

Different techniques used to determine the deformability parameters of rock masses were analysed. Following LNEC's experience¹⁾, it is possible to have an idea of the different test volumes involved in each type of test and of the most suitable tests to be used according to the Representative Elementary Volume (REV concept) of the rock mass, which is the elementary volume to be considered when the continuous approach is followed. Table 5 shows some typical numbers concerning the test volumes involved.

Table 5. Volumes involved in deformability tests

Type of test	Tested volume (m ³)	Observations
Deformability of rock samples	$\sim 20 \times 10^{-3}$	
LNEC SFJ (Small Flat Jack)	$\sim 50 \times 10^{-3}$	depends on the depth of the jack
LNEC STT (Stress Tensor Tube)	$\sim 10^{-2}$	
Dilatometer test	0.5 – 1	
Plate Loading test	0.2 – 5	depends on the plate diameter
LNEC LFJ (Large Flat Jack)	10 – 30	depends on the number of slots

The basic scale effect problem in the deformability determination was referred to in the Introduction. If the locations of test sites are chosen randomly, then the results of tests performed properly should always give the same mean deformability, with the small tests giving much more variable results than the large-scale tests. Thus, many small-scale tests should be performed in order to compensate for their variability. However, in a large number of cases and due to safety considerations, the tests are located in classes where lower deformability is expected, so the mean values obtained by small-scale tests are lower in such cases compared with large-scale tests.

When dealing with the rock mass of a dam foundation, problems concerning safety are also related to the strength problems. The main difference of the strength problems in relation to the deformability problems, is that the behaviour of the rock mass depends on the properties of the weakest element, while the deformability depends on the contributions of all the elements. Statistically the bigger the test size, the weaker the rock mass can be.

5.2 Methodology for determination of deformability

Due to the high costs involved, the number of large-scale in-situ tests is kept to a minimum, and even the number of small-scale in-situ tests and laboratory tests is sometimes reduced to be below reasonable levels. Thus, the results for the deformability properties of the rock mass may be affected by inadequate sampling. The statistical analysis of the deformability results could thus be performed with an insufficient number of data, and selection of the location of the tests is often far from a random process.

An adequate methodology must be used to assess the deformability of rock masses as dam foundations in order to accurate sound conclusions about the deformability of the rock mass. The following methodology is proposed.

i) Zoning of the rock mass

The first stage is to zone the foundation rock mass, by taking into account the available geological information, the type of rock and its degree of weathering and the discontinuity network. Each zone can be considered homogeneous and its deformability parameters will be evaluated mainly by in-situ tests.

ii) Small-scale deformability tests

In each zone, several boreholes are made in order to prospect the rock mass and to make it possible subsequently to perform some borehole deformability tests without a significant increase in the costs. A sufficient number of borehole tests, whether dilatometer or pressuremeter tests, must be carried out: in general more than ten in each zone. The distribution of these tests may follow two distinct ways: the tests may be performed either in random locations using a significant number of tests in order to obtain the mean deformability value; or they may be performed in places where the deformability modulus of the rock mass is expected to be lower. Thus, when interpreting the results, the criteria used for choosing the location of the tests must be considered.

iii) Large-scale deformability tests

Large-scale tests are performed according to the mean values obtained in each zone. There are three

distinct situations, as shown in Table 6. For situation III, which represents high deformability rock masses, it is necessary to perform large-scale tests with high precision, involving a test volume of the order of the REV volume. In the same tested volume, where these tests were carried out, borehole tests should be performed, and the results obtained by both methods should be directly correlated. With the overall correlation obtained for each zone of the rock mass considered as a homogeneous medium, the other small-scale borehole test results may then be corrected.

Table 6. Large scale tests for deformability evaluation

Situation	E (MPa)	Large scale test
I	$E \geq 10,000$	advisable
II	$5,000 \leq E < 10,000$	necessary
III	$100 \leq E < 5,000$	necessary with high precision

5.3 Interpretation of results of deformability tests for heterogeneous rock foundation

In the interpretation of the results of deformability tests for the heterogeneous rock foundation, an important scale effect was found to exist concerning the deformability of the rock mass. The results of large-scale plate loading tests with a rigid plate of 60 cm diameter were more accurate than those of the small-scale plate loading tests, and were closer to the real deformability of the rock mass. However, based on the geotechnical information available, it is not possible to say that the scale of these tests is beyond the REV of the rock mass.

Considering the large-scale tests to be representative of the deformability behaviour of the rock mass, or considering other tests performed on a larger scale, it is possible to apply the methodology proposed in the previous section 5.2.

From the correlation derived between the deformability coefficients of the large-scale plate loading tests and the pressuremeter tests ($D_{60} = 2.00D_d$ and $E_{t60} = 1.21E_{td}$), it is then possible to correct the values obtained with the pressuremeter tests performed in class C_L . Then, the mean value obtained for both deformability coefficients will be lower, $D = 494$ MPa and $E_t = 883$ MPa, than the mean values obtained by the large plate tests. For the other classes tested, C_M and C_H in lapilli tuff, it is not possible to apply such correlations because it is another formation. Thus, in order to achieve a better characterisation of the rock mass, large-scale tests should be performed in the other formations and borehole tests should also be carried out in the same place in order to establish the correlation between these two types of test.

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APPENDIX

PLATE LOADING TEST IN CONSIDERATION OF HETEROGENEITY OF CONGLOMERATE

1. INTRODUCTION

Soft rock accounts for approximately one-third of all land in Japan, and in recent years circumstances have required many large-scale structures such as dams, bridges and tunnels to be constructed on/in this bedrock safely and economically. To evaluate the mechanical properties of such foundation rock, in-situ tests, such as the plate loading test, pressuremeter test, and rock shear test, which are larger in scale than laboratory tests, have been conducted. These tests were originally developed for hard rock foundations, but in practice, soft rock foundations are also tested and evaluated in the same manner. However, because soft rock has properties which are intermediate between soil and hard rock, it is difficult to evaluate the mechanical properties of a soft rock foundation with accuracy by using the testing method developed for hard rock foundations without modification. To accurately evaluate the mechanical properties of soft rock, it is necessary to study the properties of soft rock, perform an in-situ test in a manner appropriate to the properties, and analyse the test results.

When large-scale structures are constructed on a soft rock foundation, deformability among other mechanical properties of the foundation is considered to be the most important parameter, and the plate loading test is performed in order to evaluate the deformability and determine design values. In this test, a load is applied on a rigid circular plate with a diameter of 30 cm placed normally on the bedrock to measure the relationship between the load and the plate displacement and thus evaluate the deformability of the bedrock. The size of this plate is set to 30 cm due to such limitations as the capacity of the loading jack, the width of the exploratory adit and the ease of conducting of the test. However, when the tested bedrock consists of gravel and matrix as in the case of volcanoclastic rock, it has broad gradation. In addition, if there is a relatively large difference between the gravel and the matrix in hardness, a representative volume used to accurately evaluate the deformability of bedrock of this kind, the so-called REV (Representative Elementary Volume) is larger than that of sedimentary rock with smaller grain size such as sandstone and mudstone. Therefore, there may be cases where the scale of test using a loading plate with a diameter of 30 cm is not always large enough to evaluate the deformability of the bedrock. This is the problem of heterogeneity.

To evaluate the deformability of the bedrock having such properties, ideally, a test should be performed on a scale reflecting the REV of the bedrock. However, the scale of test will be too large if it is determined according to the REV, depending on the degree of heterogeneity of the bedrock, and so in practice many tests should be conducted on a scale smaller than the ideal one. Thus, in practice, when conducting the plate loading test for heterogeneous bedrock consisting of gravel and matrix, ease of performance and economic efficiency of the test should be considered as well as the heterogeneity of the bedrock. Before the test, the feasible test scale, that is

the diameter of the circular plate, is determined based on a geological judgement at a site which has been selected as having properties that are geologically representative of the bedrock.

Although the results of the plate loading test conducted in this manner need to be evaluated in terms of the heterogeneity of the bedrock, the effect of the heterogeneity of bedrock on mechanical properties has not been clarified yet. In order to study the relation between the heterogeneity of bedrock and the results of the plate loading test, we conducted a plate loading test using a circular plate with a diameter of 30 cm (conventional type) and another circular plate having a diameter of 60 cm at the same spot on conglomerate, which is considered to have heterogeneous strength and gradation on the normal test scale.

2. GEOLOGICAL FEATURES OF THE TEST SITE

Figure 1 shows a sketch of the bedrock observed in the exploratory adit where the tests were conducted. This is a forked adit branched off at a spot 15 m down from the entrance of the main adit excavated on the right bank of the river, and was excavated newly for these tests in the conglomerate layer of the middle-upper Miocene of Neogene. This conglomerate is pale dun - greenish dun, and is a heterogeneous bedrock consisting of gravel and matrix. The overall bedrock can be classified as C_L class. The bottom of the exploratory adit is relatively dry, and the ground water level is 40–80 cm below the bottom.

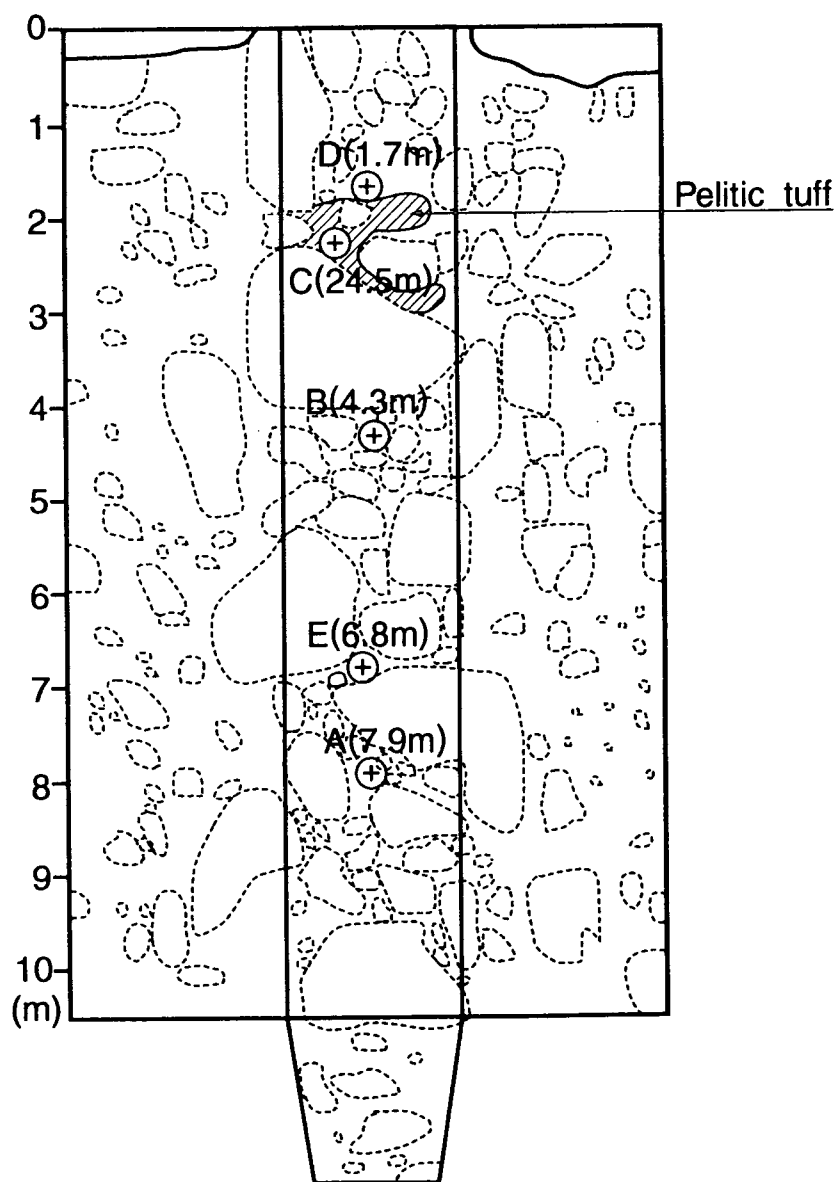


Figure 1. Geological features of the test site

The gravel is made up of lapilli tuff, coarse-grained tuff and mudstone, and much of it is solid. On the whole, it is near-circular or near-angular gravel. The main gravel size ranges from 5 to 50 cm, but huge gravel measuring 2 m in size is found in places. Individual pieces of gravel are distributed in such a manner that they lie close together as if they mesh with each other, and though they form an indistinct lateral line on the side wall of the exploratory adit, but show no such regularity at the bottom.

The matrix consists of sandy tuff and pelitic tuff, has a low degree of consolidation, and is soft. On the whole, a large portion of the matrix constitutes sandy-tuff matrix (sandy matrix); although some part of the matrix is slightly firm, it can be easily collapsed by a hammer. Pelitic-tuff matrix (pelitic matrix), which is contained in the matrix in a small-scale lenticular form, is so soft that it can be dented with finger pressure. In particular, this pelitic matrix is distributed somewhat widely around the bottom 2.0 to 2.5 meters from the entrance of the forked adit.

3. OUTLINE OF THE TEST

The test site consists of gravel of various sizes and small-scale matrix that lies in between. Assuming that the deformability of the entire bedrock is affected by the properties and distribution of the matrix because the strength of the matrix is far lower than that of the gravel, we avoided the areas where gravel is in tight formation when selecting test spots, and selected areas where the matrix and gravel are roughly equal proportion.

We first removed wastes from the exploratory adit, and roughly determined the test spots. Then, we dug down 30 to 50 cm to remove loosened bedrock at the bottom, adjusted the location of the test spot after observing the condition of the bedrock again, and finally determined the test spots as shown in Fig. 1.

In order to study the effect of the heterogeneity of conglomerate on the test results, we used plates of different diameters, 30 cm and 60 cm, and conducted a plate loading test at the same spot. The detailed sketches of the bedrock subjected to the tests are shown in Figs. 2(a), 2(b), 2(c), 2(d) and 2(e). At each spot, we first conducted a test using a plate with a diameter of 30 cm, and then performed a test using a plate with a diameter of 60 cm. The loading pattern is given in Fig. 3. As for loading, we first applied a preliminary load so that the plate fitted the bedrock well, and then applied a step-by-step load. The other test specifications were in accordance with "A Guideline for Testing Methods of Deformability and Shear Strength of In-situ Rock Masses –Commentary and Application to Design of Structures– (1983)" published by the Japan Society of Civil Engineers.¹⁾

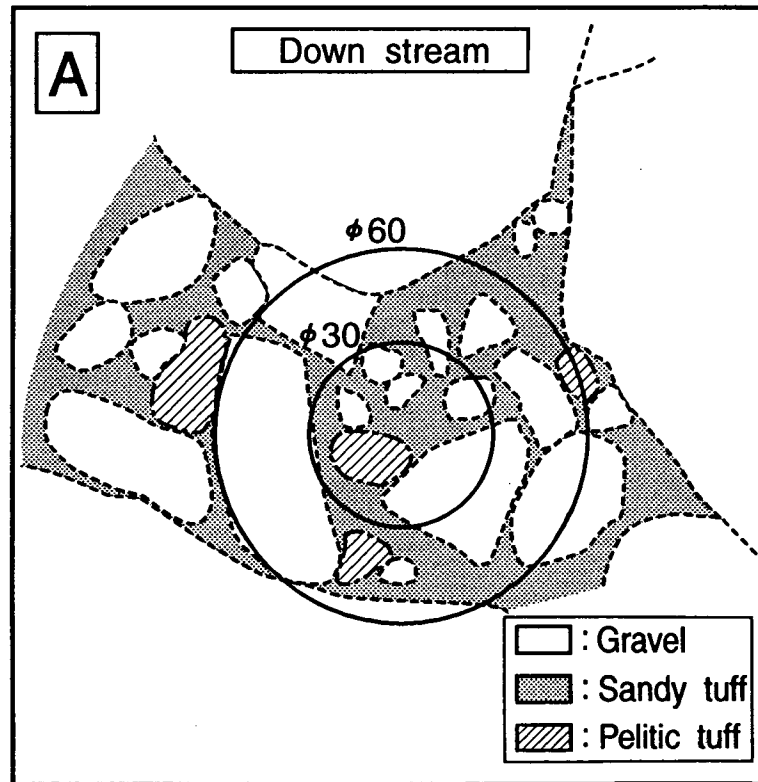


Figure 2(a). Detailed sketch of the test surface A

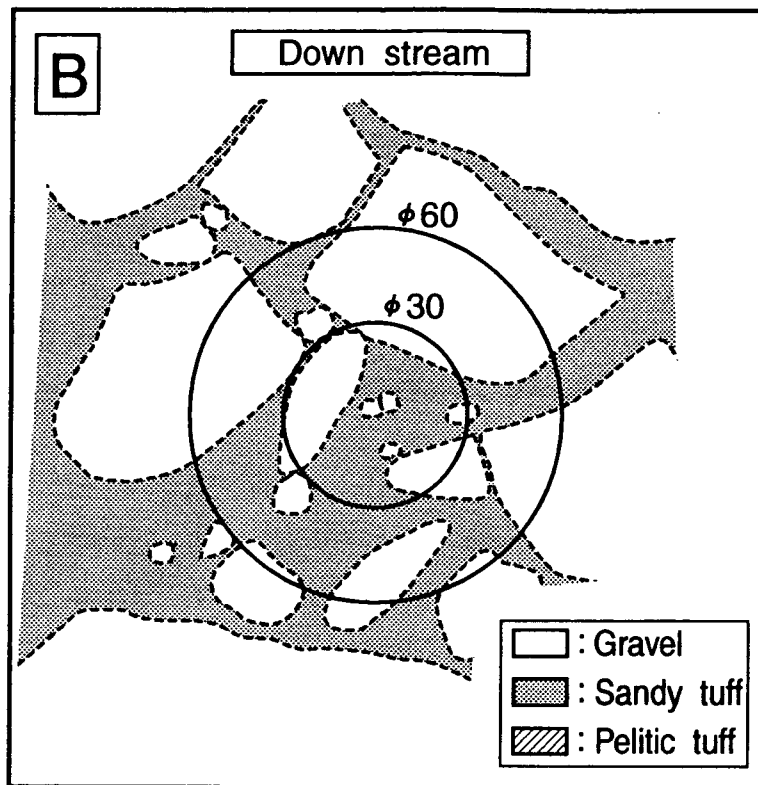


Figure 2(b). Detailed sketch of the test surface B

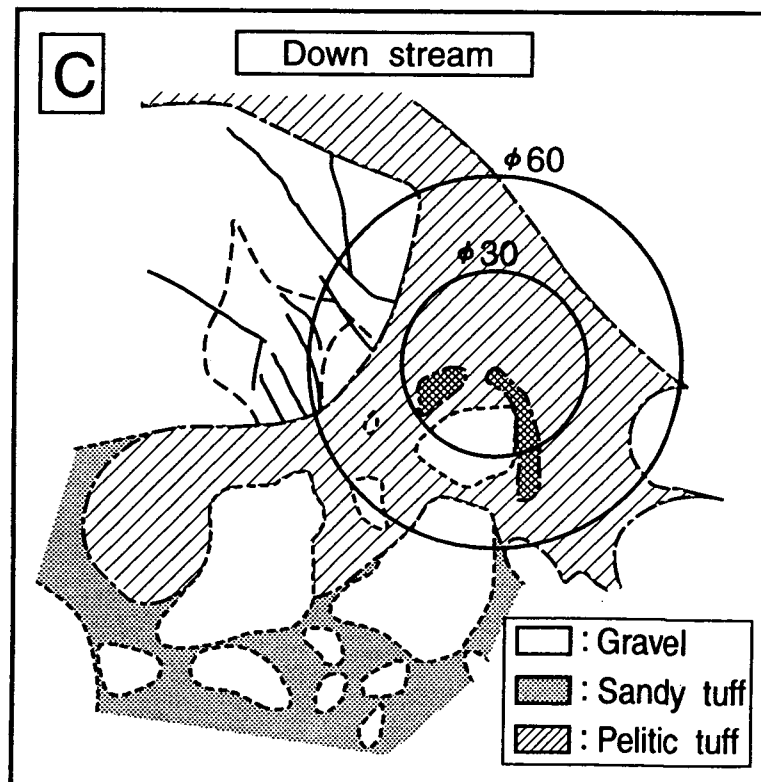


Figure 2(c). Detailed sketch of the test surface C

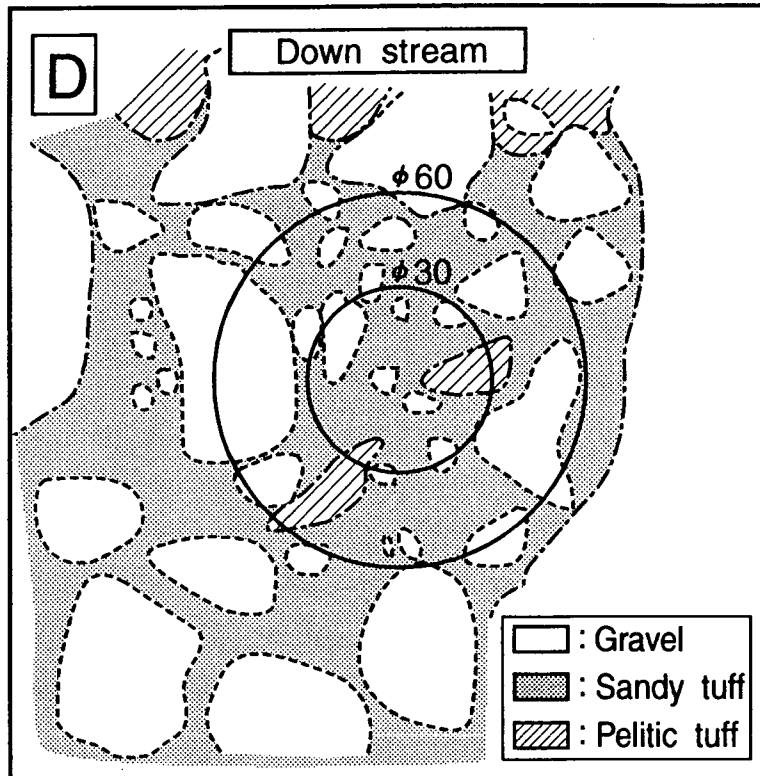


Figure 2(d). Detailed sketch of the test surface D

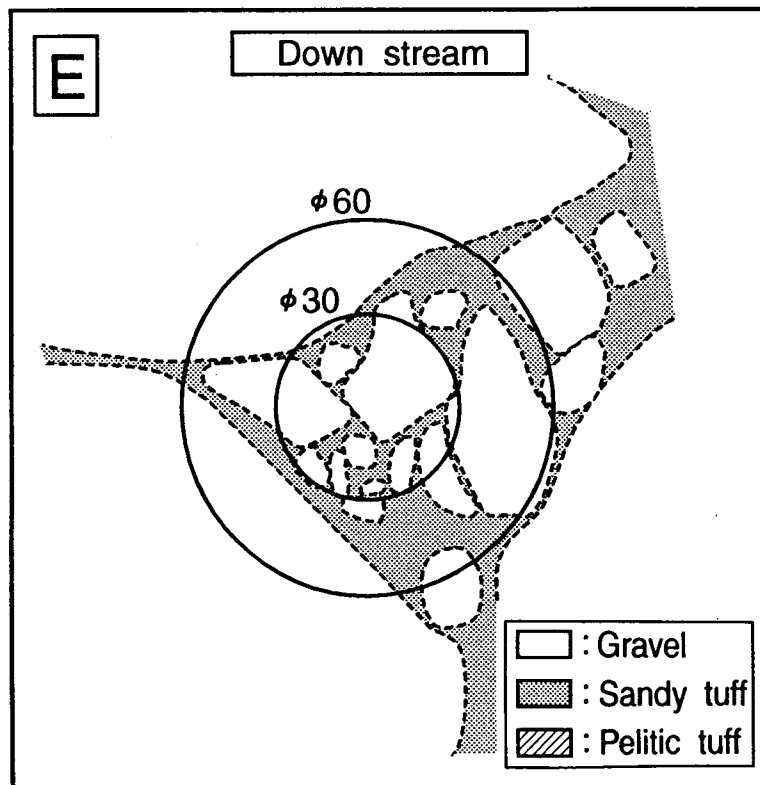


Figure 2(e). Detailed sketch of the test surface E

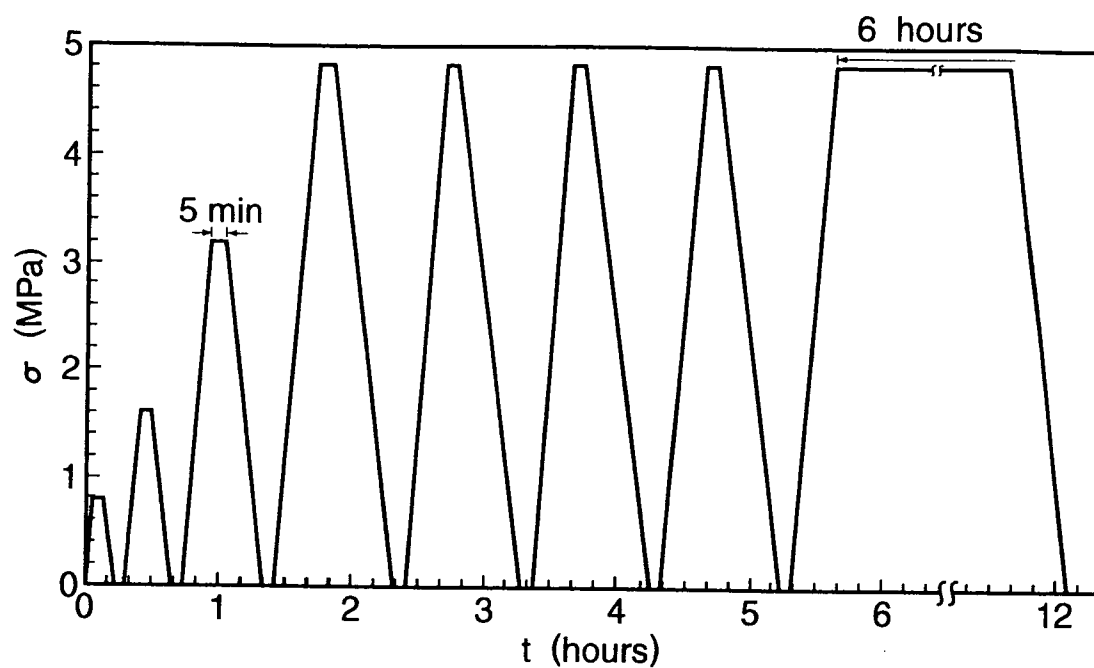


Figure 3. Loading pattern

After completion of the test, we conducted an open-cut survey to observe the condition of the bedrock directly below the test spot, and made sketches of the bedrock at several depths. The matrix and gravel area ratio of the test surface at each test spot is shown in Table 1.

Table 1. The matrix and gravel area ratio of the test surface

Test spot	Diameter of the plate (cm)	Area ratio (%)	
		Matrix	Gravel
A	30	58	42
	60	38	62
B	30	55	45
	60	43	57
C	30	87	13
	60	58	42
D	30	82	18
	60	64	36
E	30	28	72
	60	28	72

4. TEST RESULTS AND CONSIDERATIONS

4.1 Relation between loading stress and displacement

Figures 4(a), 4(b), 4(c), 4(d) and 4(e) show the relation between the loading stress and the displacement of the plate measured at each test spot. The displacement of the plate represents the mean value of the displacement measured at the four points on the plate. The horizontal axis indicates the dimensionless displacement as a percentage, which is obtained by dividing the displacement by the diameter of the plate. In this test, we repeated loading and unloading several times up to the maximum stress after the loading shown in the figure, and kept on loading under the maximum stress for several hours to study the creep characteristics of the bedrock. Discussion of this is left for a subsequent report.

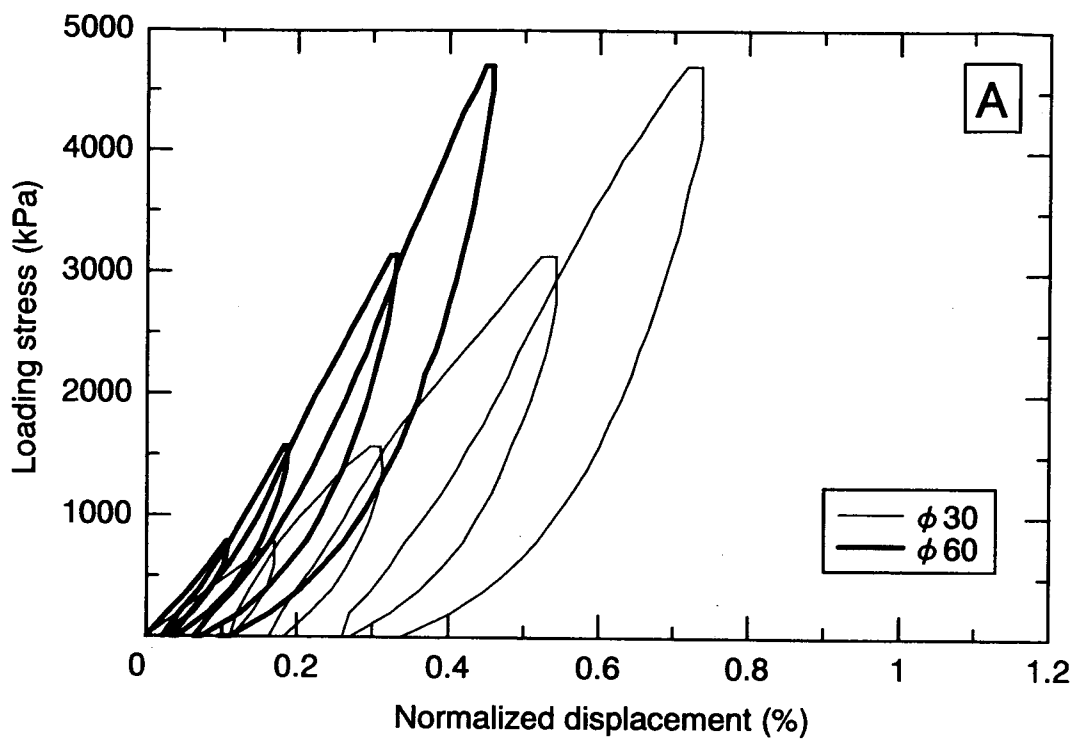


Figure 4(a). Relation between the loading stress and the displacement of the plate at test spot A

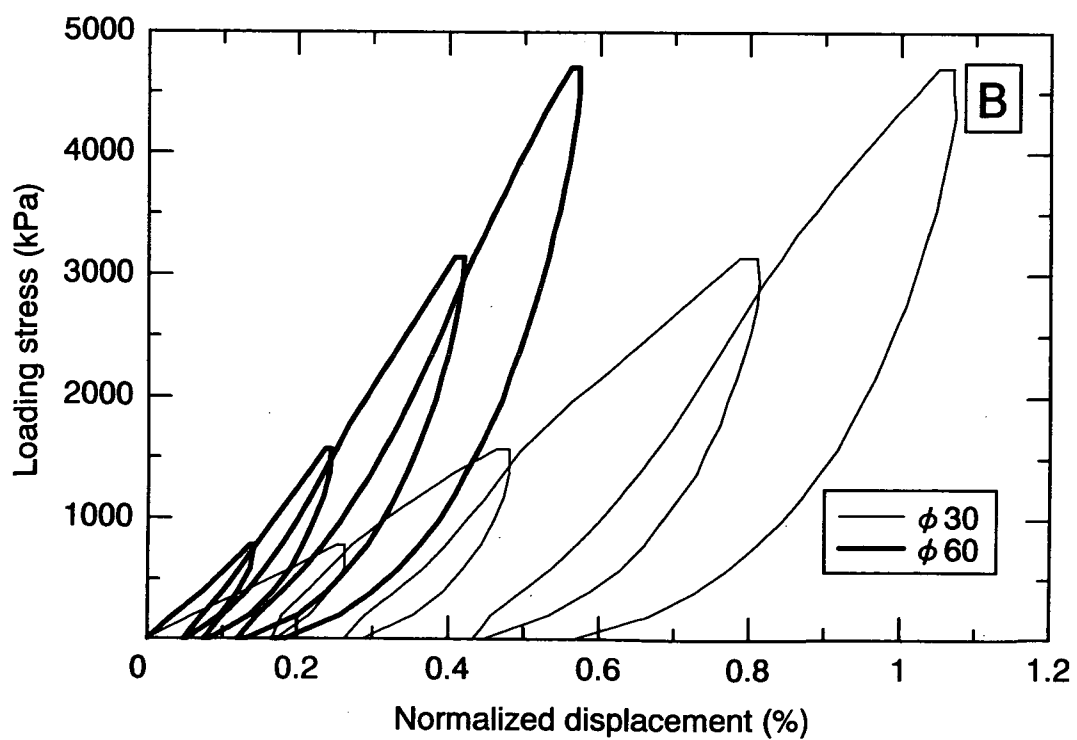


Figure 4(b). Relation between the loading stress and the displacement of the plate at test spot B

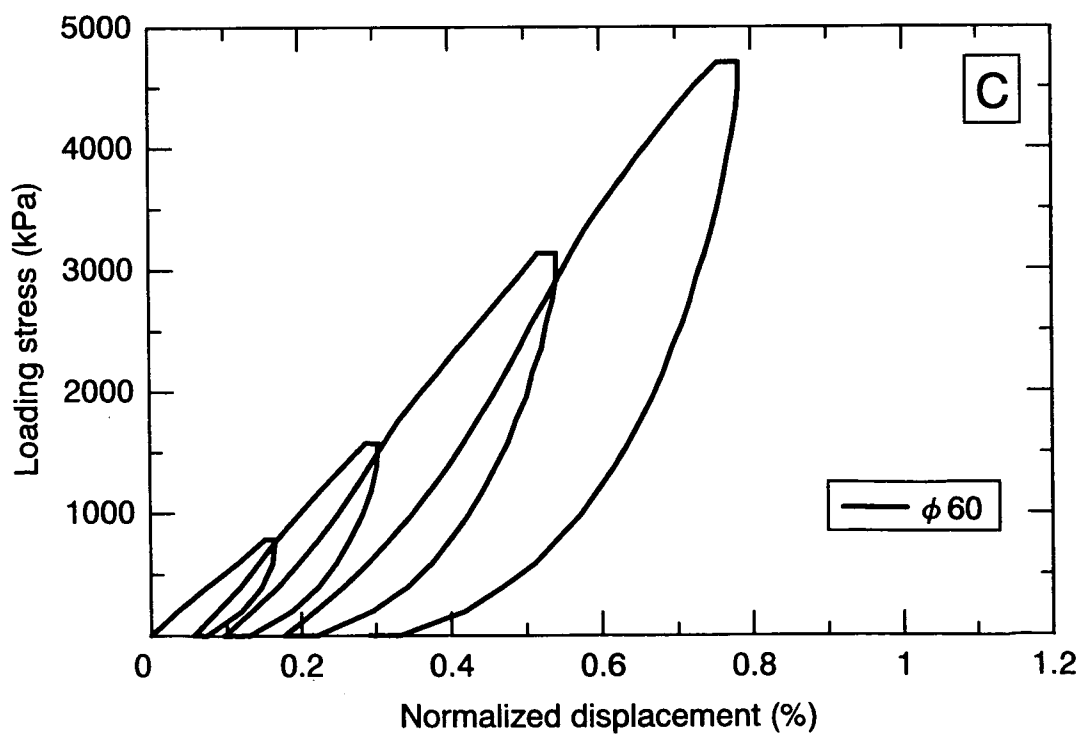
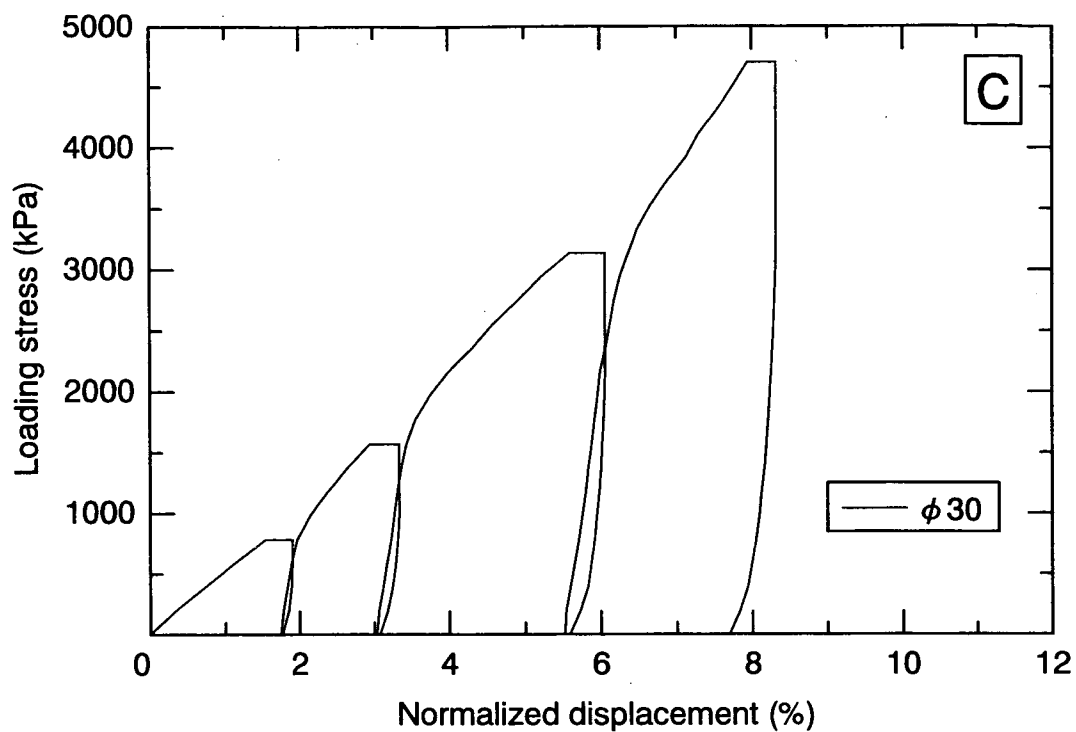


Figure 4(c). Relation between the loading stress and the displacement of the plate at test spot C

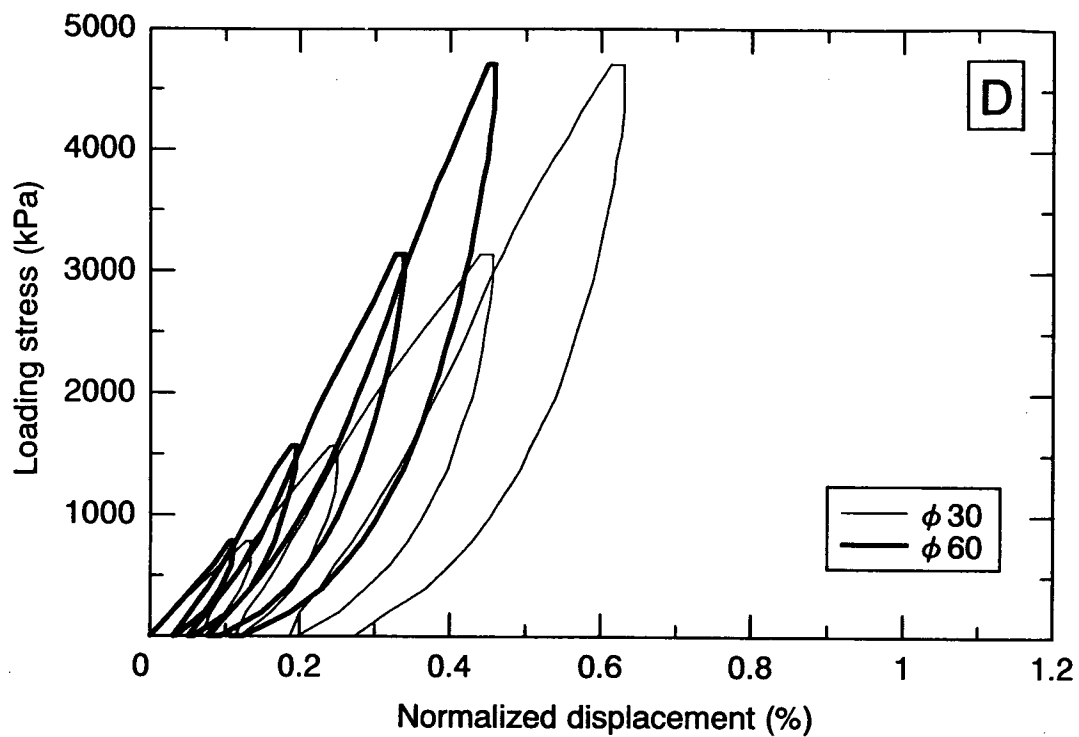


Figure 4(d). Relation between the loading stress and the displacement of the plate at test spot D

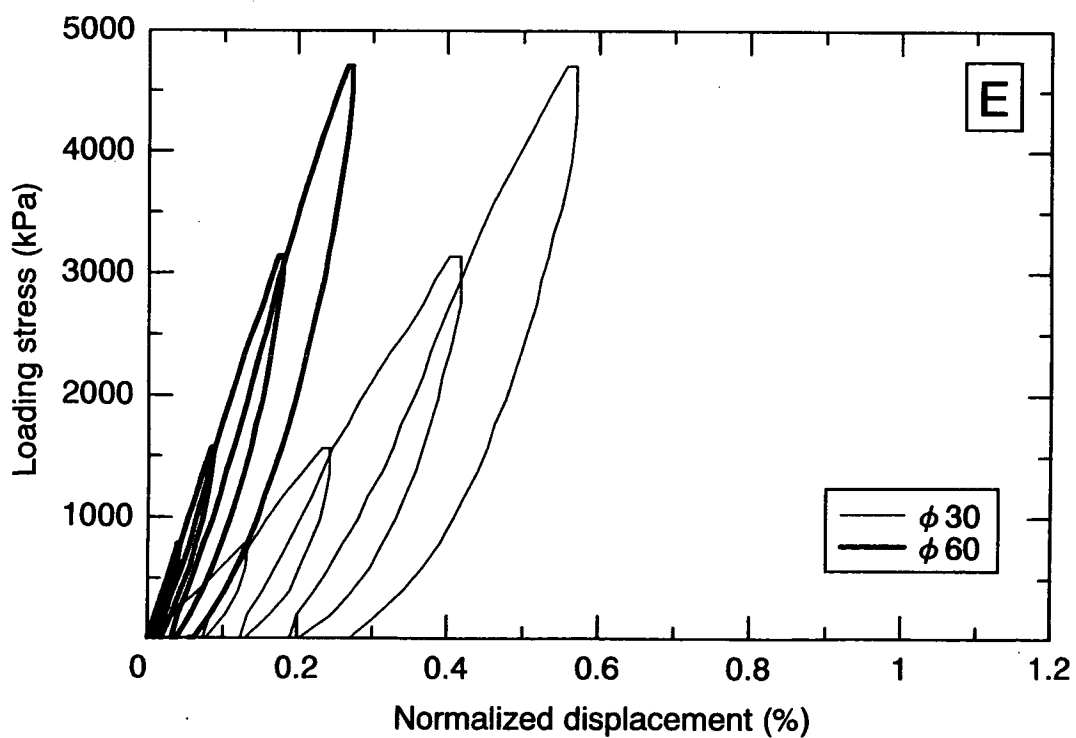


Figure 4(e). Relation between the loading stress and the displacement of the plate at test spot E

The figure shows that although there is a slight difference for each case, the slope of the tangent of the curve enveloping the virgin stress section increases as the displacement of the plate increases; in other words, the curve indicates that the bedrock gets harder as the loading stress increases. This phenomenon in a plate loading test is often observed in bedrock with many cracks, and is believed to occur when the cracks in the bedrock gradually close up with an increase in loading stress, and the bedrock near the test surface turns massive²⁾. Although we could not find cracks in the conglomerate subjected to this test when we observed the bedrock, we believe the phenomenon occurred when a separate surface, which was formed at the boundary between the gravel and the matrix due to disturbances such as the release of in-situ stress at the time of excavation of the exploratory adit or impact during the formation of the test surface, closed up.

In the test conducted at the spot C using a plate with a diameter of 30 cm, an extremely large displacement of the plate was measured as compared with the other test spots, but the test performed at the same spot using a plate with a diameter of 60 cm did not show such different behaviour from the results obtained from the other spots. The sketch of the test surface of the test spot C (Fig. 2(c)) shows that although part of the plate having a diameter of 30 cm is placed on the gravel, most of it lies on the soft pelitic matrix. On the other hand, when the plate with a diameter of 60 cm is similarly placed on the pelitic matrix, the gravel area ratio is larger than that of the test using the plate with a diameter of 30 cm (Table 1). Therefore, we can assume solely from the observation of the test surface that the difference of the results between the two tests is due to a difference in the gravel area ratio and the distribution pattern of the gravel contained in the test surface. However, the open-cut survey results at this test spot revealed that the pelitic matrix ranged up to 30 cm below the test surface, and, in addition, huge gravel on which the plate having a diameter of 60 cm was placed was supported by the sandy matrix distributed below the pelitic matrix. This means that in the test using the plate having a diameter of 30 cm, the loading stress acted on the pelitic matrix in the upper part, producing a significant amount of deformation on the part of the pelitic matrix, while in the test using the plate with a diameter of 60 cm, the loading stress acted on the sandy matrix in the lower part through the gravel. We must therefore assume that the large difference in the results between the two tests conducted at the spot C was due to a difference not only in the gravel area ratio and distribution pattern of the gravel contained in the test surface but also in the matrix on which the loading stress acted. Thus, when gravel is contained in the test surface, we should evaluate the test results, taking account of the properties of the matrix which supports the gravel.

We measured the amount of displacement of the plate at the time of the maximum stress of each loading cycle (total displacement), and the amount of displacement of the plate at the time of zero stress that immediately followed (permanent displacement), and obtained the ratio of permanent displacement to total displacement. The trend of this ratio with respect to the dimensionless displacement is shown in Fig. 5. The test conducted at a spot using the plate with a diameter of 30 cm showed a larger value than the test conducted at the same spot using the plate with a diameter of 60 cm. This means that the effect of irreversible displacement is greater for the test conducted using the plate with a diameter of 30 cm. For the bedrock subjected to this test, the irreversible

displacement factors mainly refer to the closure of separate surface between gravel and matrix and the consolidation of bedrock around the test surface, loosened due to the release of in-situ stress, rather than to the plastic softening of the bedrock. It has been pointed out for some time now that a plate having a larger diameter is less subject to what is known as disturbance such as the above-described, and this test also produced results reflecting this. In most of the tests in which a plate with a diameter of 60 cm was used, the plate was placed on the huge gravel, and the fact that relatively great loading stress acted on the deeper and less loosened part may be one of the causes of this trend. We assume this to be one of the characteristics observed in a plate loading test conducted on heterogeneous bedrock such as the conglomerate subjected to this test.

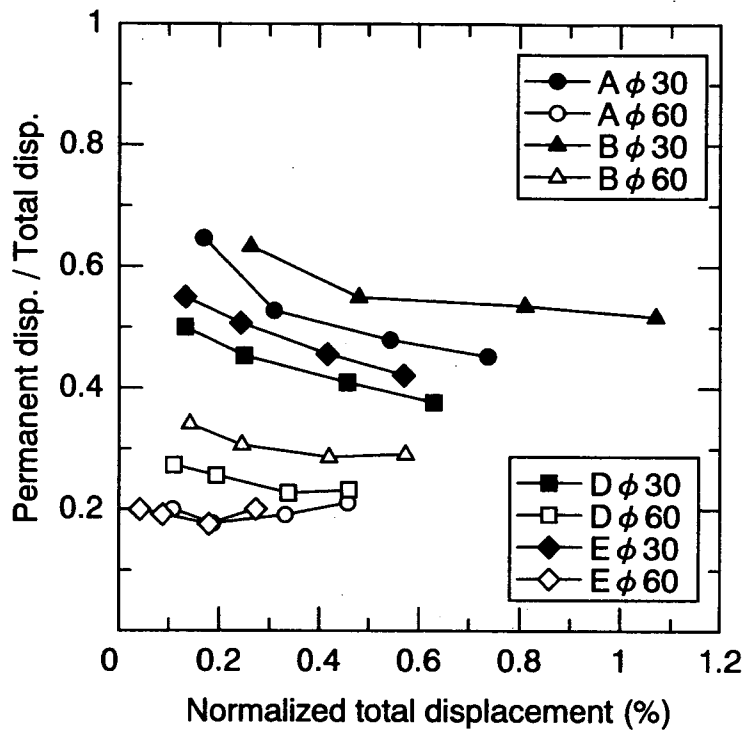


Figure 5. The ratio of permanent displacement to total displacement

4.2 Effect of gravel on deformability modulus

A comparison of the results of tests conducted at each spot using a plate with a diameter of 30 cm with those of tests performed with a plate having a diameter of 60 cm clearly indicates that the deformability of bedrock is larger than that of the latter. Table 2 shows the deformability modulus of bedrock in each test case. To calculate deformability modulus, the following conventional formula used to determine design values was used. It is based on the theory of elasticity and is applicable as long as the loading stress and displacement can be regarded to be approximately proportional.

$$D = \frac{a\pi(1-\nu^2)}{2} \cdot \frac{\Delta p}{\Delta \delta} \quad (4.1)$$

where D = deformability modulus, a = radius of the plate, ν = Poisson's ratio (set to 0.3), Δp = increment of loading stress used for calculation, $\Delta \delta$ = increment of displacement used for calculation.

Table 2. Deformability moduli

Test spot	Diameter of the plate (cm)	Deformability modulus D_{30}, D_{60} (MPa)	Stress region for calculation (kPa)	D_{60}/D_{30}
A	30	566	2352–4704	1.53
	60	868	2156–4704	
B	30	405	1960–4704	1.78
	60	721	1960–4704	
C	30	46	2156–4704	10.4
	60	477	1176–4704	
D	30	621	2156–4704	1.44
	60	896	2156–4704	
E	30	721	2156–4704	1.69
	60	1218	1372–4704	

Figure 6 shows the relation between deformability modulus and the gravel area ratio of the test surface. However, the result of the test conducted at spot C using a plate with a diameter of 30 cm, where the matrix is pelitic and a large deformation was generated, is excluded here. With linear regression, in the tests using both plates, one with a diameter of 30 cm, and the other with a diameter of 60 cm, those with a larger gravel area ratio tend to show a larger deformability modulus. However, the data are scattered so widely that it is difficult to determine whether they are correlated with each other. To review the effect of the gravel volume ratio in the bedrock on deformability modulus, we estimated the gravel volume ratio just below the test surface from the sketch of the bedrock made in the open-cut survey. We obtained the volume of gravel by integrating the area of the gravel observed just below the test surface down to the depth of the diameter of the plate in the direction of depth. The ratio of this volume to the volume of the column whose depth is equal to the diameter of the plate just below the test surface is given as the gravel volume ratio of the bedrock. Figure 7 shows the relation between the gravel volume ratio obtained in this manner and deformability modulus. However, as in the case of Fig. 6, the data are scattered so widely that there is no clear correlation.

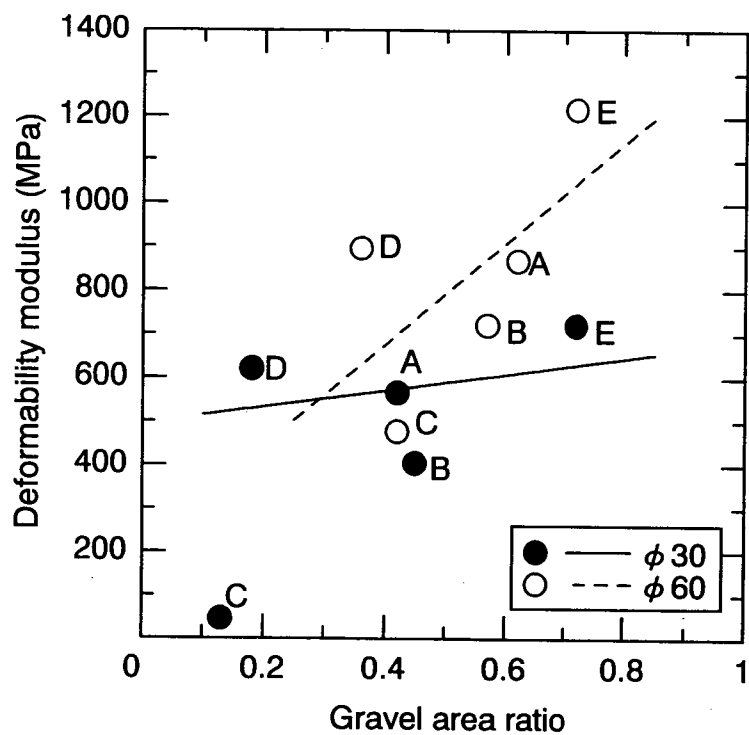


Figure 6. The relation between deformability modulus and the gravel area ratio of the test surface

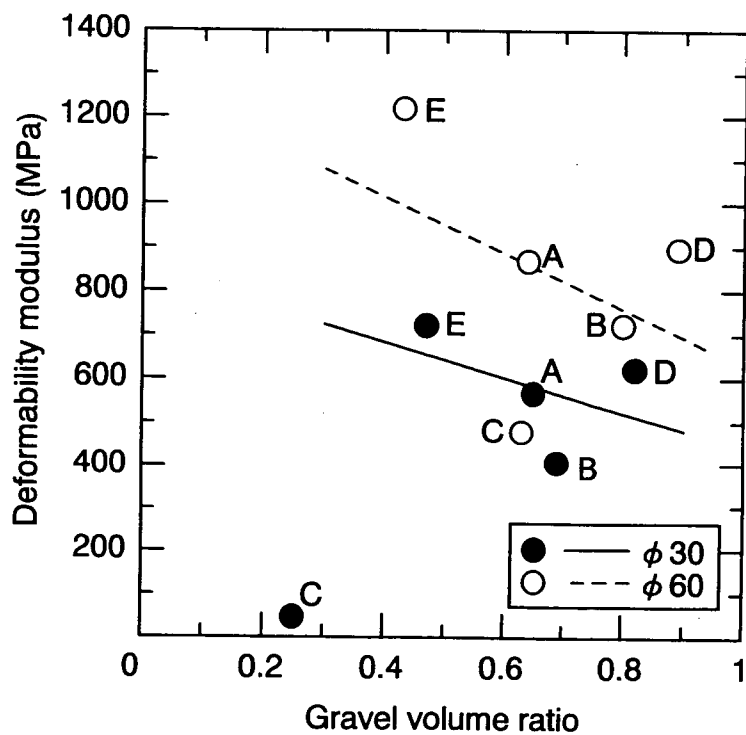


Figure 7. The relation between deformability modulus and the gravel volume ratio

Thus, in this test, we could not establish a clear correlation between the gravel area ratio in the test surface and deformability modulus or between the gravel volume ratio just below the test surface and deformability modulus. In a plate loading test conducted on bedrock containing huge gravel as large as 2 m in size such as the conglomerate subjected to this test, the loading stress spreads to a much wider area because part of the plate is placed on the huge gravel, and we assume that this is one of the effects that the gravel has on the test. In this test, part of the plate having a diameter of 60 cm was placed on such huge gravel in most cases, so the loading stress spread to a wider area, resulting in smaller displacement of the plate. A larger deformability modulus was thus obtained than with the test using a plate with a diameter of 30 cm. Therefore, in this test the gravel size and the location of gravel in the test surface had larger effects on the test results than the gravel area or volume. In particular, when part of the plate is placed on huge gravel which ranges from the exploratory adit to the natural ground in the back, it is expected that the gravel itself is unlikely to show any displacement with loading.

4.3 Magnitude of deformability modulus due to different diameters of plate and test spots

Figure 8 shows the relation between the deformability modulus D_{30} obtained from a plate loading test conducted at a spot using a plate with a diameter of 30 cm and the deformability modulus D_{60} obtained from a test performed at the same spot using a plate with a diameter of 60 cm. The figure shows that D_{60} is approximately 1.4 to 1.8 times as large as D_{30} , with the exception of data taken at spot C where the properties of the matrix are very different from those of other spots, and when regressed with a straight line that passes through the origin, a relatively clear correlation between D_{30} and D_{60} ($D_{60} = 1.6 \times D_{30}$) can be observed.

Naturally, the value of D_{60} is considered to have been affected by the consolidation due to the load history during the test using a plate with a diameter of 30 cm. Therefore, we will briefly study this effect through elasticity analysis by using the finite element method. When a load is applied to the surface of bedrock, a relatively large stress acts around the loading plate, and the stress decreases as one moves away from the loading plate. Therefore, a significant increase in the elasticity modulus of bedrock due to consolidation can be observed around the loading plate. From this, we first simulated a plate loading test on a uniform elastic foundation using a plate with a diameter of 30 cm to determine the elements where the vertical stress increased by more than 10 % of the loading stress. Then, by increasing the input elasticity modulus of those elements, we could simulate bedrock subjected to a load history. However, the rate of increase of this input elasticity modulus was determined based on the results of in-situ tests. In other words, in each test we conducted using a plate with a diameter of 30 cm, the tangent modulus of elasticity of bedrock measured at the time of loading in the final cycle after being subjected to the maximum load history was almost twice the deformability modulus, with the exception of the results for the test spot C. So in our simulation of a plate loading test using a plate with a diameter of 30 cm after the input elasticity modulus was increased around the loading plate, we calculated the elasticity modulus for the entire bedrock to be about twice that before the increase in the input elasticity modulus. Figure 9 shows the mesh used for the calculation. We used the model reflecting the effect of this load history and the uniform bedrock model not

reflecting the effect of the load history and simulated a plate loading test using a plate with a diameter of 60 cm. The elasticity modulus for the bedrock reflecting the effect of load history turned out to be about 1.2 times as large as the elasticity modulus for the bedrock not reflecting the effect of load history.

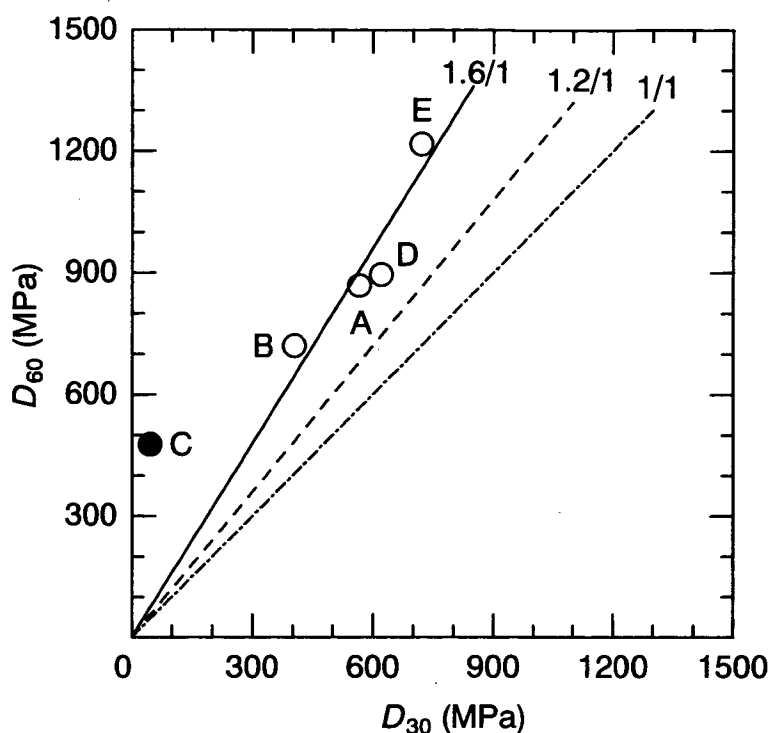


Figure 8. The relation between D_{30} and D_{60}

Since plates of different diameters were used in this test at each spot, the magnitude of D_{30} with respect to D_{60} in the in-situ test is unlikely to be influenced by the hardness of the matrix. Also, in the above study, we did no more than increase the deformability modulus by approximately 20 % for the load history in the test using a plate with a diameter of 30 cm. Therefore, clearly there is at least one other effect that is considered different from those effects, namely, the effect produced by changing the diameter of the plate, which changes the diameters or distribution of the gravel of the test surface, and increases the deformability modulus.

The magnitude of deformability modulus in a series of tests using a plate with a diameter of 30 cm showed the same tendency as for a plate with a diameter of 60 cm. In other words, the evaluation of the magnitude of deformability of bedrock at different spots does not depend on the plate diameter, and this is significant for practical purposes. Since the test results also indicated this tendency despite the fact that the gravel size and the distribution of gravel near the test surface were totally different in each test case, it is possible that the magnitude of deformability modulus that differs from one test spot to another does not depend much on the gravel size or the

distribution of gravel but reflects the hardness of the matrix. We cannot make a conclusion to this effect because in this test we did not survey the hardness of the matrix at each test spot. In the future, we plan to conduct a simple test on the matrix of each test spot such as a needle penetration test, and to perform laboratory tests with collected samples to evaluate the mechanical properties of the matrix. The results of those tests will then be fed back to the evaluation of the results of this test.

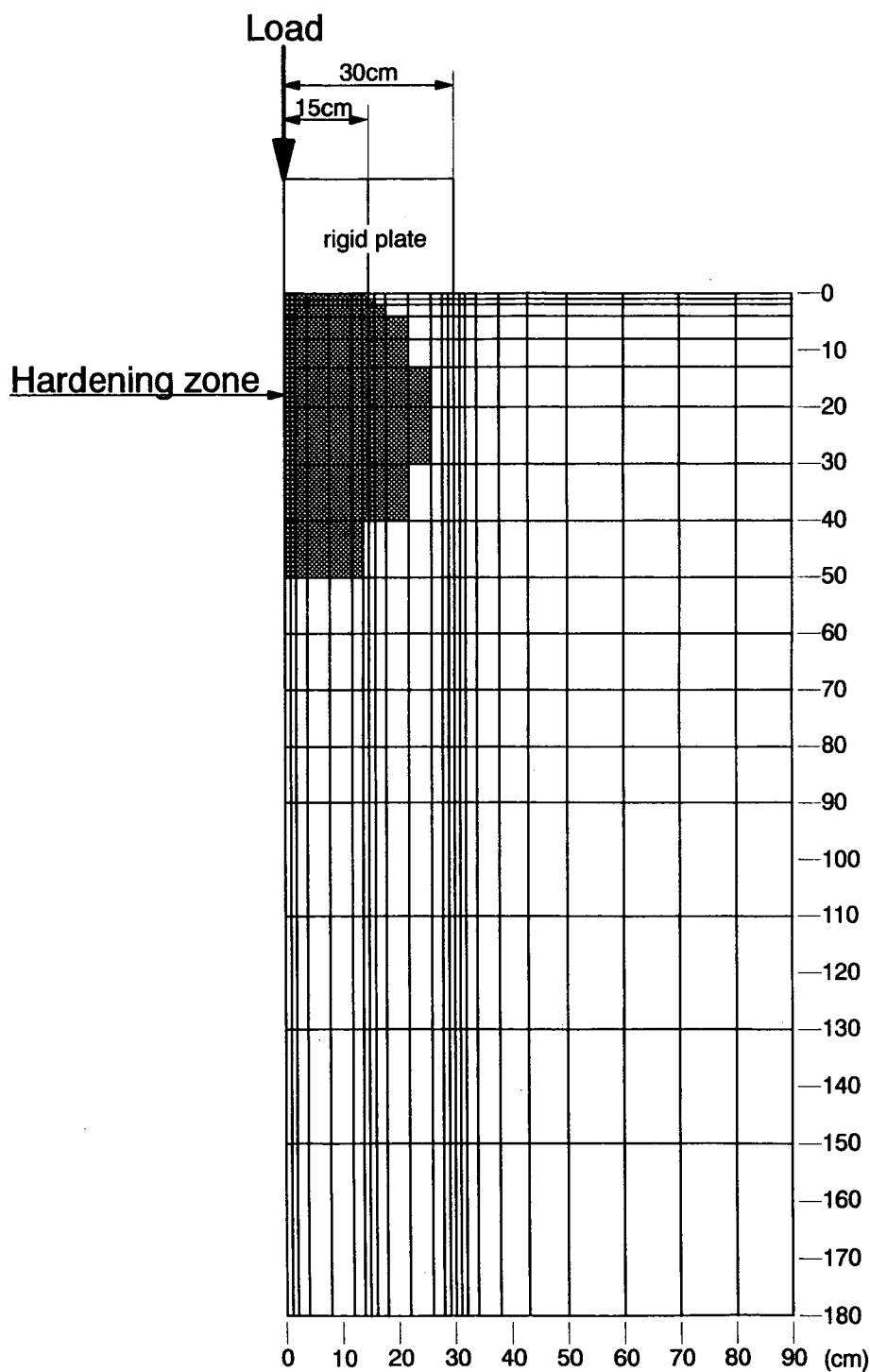


Figure 9. Mesh for the finite element method

5. FUTURE SUBJECTS

We analysed the results of the plate loading test and could qualitatively evaluate some of the effects of heterogeneity of conglomerate. However, various factors resulting from the heterogeneity of the conglomerate other than those we evaluated in this test are considered to exist, about which we can only make assumptions as yet. We need to collect a great deal of detailed data, and to clarify the relation between the heterogeneity of bedrock and the results of plate loading tests in the future.

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