Third Rankine Lecture

RECENT WORK IN ROCK MECHANICS

by

ARMAND MAYER

INTRODUCTION

When your President told me, more than a year ago, that your Institution had chosen me to give the next Rankine Lecture I felt very honoured and embarrassed—honoured because I am well aware of the degree of distinction conferred by this choice, and embarrassed because, in the words of La Bruyere,* “Everything has been said, and we are here too late, for over three thousand years there have been people who can *think*.” This also holds true for soil mechanics.

I was helped by Professor Skempton, who suggested I should devote this Lecture to “rock mechanics”. This also is not virgin ground; in fact, I remember having had a Paper (Mayer, 1953)¹ published in *Géotechnique* in 1953, after the Zurich meeting, in which I summarized what was known at that time about rock mechanics and its applications to civil engineering. Since then the subject has developed and at present, after disasters which are still vivid in all memories, it has become even fashionable.

Let me recall that in 1957 a Symposium on Rock Mechanics was held in Paris, in which some of you present participated. In 1958 and 1961 two meetings on ground pressures took place in Leipzig and in Paris organized by mining engineers, where different Papers were also of interest to us.

Then, in 1961, at the Paris meeting on “soil mechanics”, Professor Skempton in his inaugural address declared that “rock mechanics was an integral part of soil mechanics and should form a special section at our next meeting”.

At the same time, the International Commission for Large Dams also decided to include in their agenda for the 1964 meeting a question on rock mechanics wide enough to allow for all developments.

In 1962 a group of Austrian, German, and Italian engineers formed at Salzburg an International Society for Rock Mechanics, centred around the research organization of Dr L. Muller. Meanwhile in France after the Symposium of 1957 the National Association for Technical Research formed an *ad hoc* commission—which included specialists in different fields, geologists, miners, civil engineers, oil engineers, and others interested in rock mechanics—for the purpose of fostering research in rock mechanics, and asked me to take the chair.

As a result, a laboratory devoted to the Mechanics of solids, aimed directly at fundamental research, was started at the École Polytechnique. This laboratory is attached to the Chair of Professor Mandel, and is under his and Mr Habib's direction. It celebrated its first anniversary in October 1962. I will refer later to some of the important work already carried out there.

In other countries there are also laboratories and scientists working in the same field, although they are perhaps more concerned with applied rock mechanics. I cannot cite them all, but I would like to mention the laboratory of Rock Mechanics Ltd, which needs no formal

* Tout est dit et l'on vient trop tard, depuis plus de trois mille ans qu'il est des hommes et qui pensent.

¹ See references on p. 118.

introduction; then there is at Bergamo (Italy) the laboratory of the Ismes, directed by Professor Oberti; there is the special section of the Building Research Laboratory in Lisbon, directed by MM. Rocha and Laginha Serafim; the studies of both these laboratories are concentrated on models.

Important work is also going on in Sweden and Norway where rock is to be found practically everywhere, as well as in Japan where seismic engineering has been developed in recent years.

Finally, in the United States of America there are several laboratories working on rock mechanics, of which I shall mention only the Bureau of Reclamation, and the Bureau of Mines. There are also a number of engineers among whom I want to cite a very prominent scientist, who has recently devoted several Papers to rock mechanics, and whom I should like to refer to at the beginning of this Paper—Professor Karl Terzaghi.

So it is safe to say that this subject happily proposed by Professor Skempton, while no novelty, is nevertheless of vivid contemporary interest.

Perhaps the best way I can project a picture of what it is today, will be by first glancing back to its state 10 years ago, and then following developments through, up to the present time, while throwing most light on the researches and applications which have been done in France.

ROCK MECHANICS IN 1953

Rock mechanics as used by the civil engineer 10 years ago was already split into two phases: first, the measurement of the mechanical characteristics of rocks and, second, the application of the knowledge gained to different practical problems.

The measures sought were:

- (i) the elastic modulus of the rock, and
- (ii) the stresses in the rock in situ.

The elastic modulus was generally measured in the field by the jacking method. This, to my knowledge, was used for the first time, around 1935, by the Irrigation Department in Algeria. A jack was placed across a gallery, its foot against one wall and its head thrusting against the opposite wall, where the rock surface had been carefully dressed flat and true. The thrust of the jack was increased and the rock deformation measured and expressed as a function of the applied stress.

The elastic modulus was calculated from the relationship:

$$\delta = \frac{\alpha P}{\gamma} \frac{1 - \nu^2}{E}$$

where E is the elastic modulus of the rock, δ is the measured deformation, P is the applied thrust, ν is the Poisson coefficient of the rock, and α is a coefficient characteristic of the stiffness of the bearing effective under the foot of the jack. This coefficient is usually about 0.5.

When the results of these measurements were checked in the laboratory it was found that—in the case of homogeneous, isotropic rock, or concrete—the calculated values agreed well with those obtained by direct measurement, and roughly with those calculated from measured sound velocities in the same kind of rock, using equations of the type:

$$V = \sqrt{\frac{E}{\gamma} \frac{1 - \nu}{(1 + \nu)(1 - 2\nu)}}$$

where the letters have the same meaning as before, V being the velocity of the sound waves and γ the density of the rock.

In the course of the past 10 years the jack method has become quite a standard procedure and some examples of the results provided by it will be described later.

Another method, tried at that time, particularly by the Irrigation Department of Morocco, used a certain length of gallery which was put under hydraulic pressure. The length required was driven into the rock and provided with a water-tight steel bulkhead to seal it off. The gallery was then filled with water and put under pressure. The variations of the diameter with the change in pressure were measured; hence the elastic modulus was computed.

This method has the great advantage that it involves large areas and thus gives good average values, whereas by the jack method the thrust is applied only to a small and possibly not representative area. On the other hand the gallery method is far more painstaking and more expensive; therefore it has been little used since the time when M. Bernard, Ingénieur des Ponts et Chaussées, and our laboratory applied it for predicting deformations in a pressure tunnel in Morocco. Those measurements confirmed that gallery values for elastic moduli agreed well enough with the average of a sufficient number of values obtained by the jack method so that this last procedure from there on was used practically alone.

Besides the elastic modulus of the rock, the second element sought was the value of the stress acting on the rock. Two methods were used for determining it.

The first assumed that the elastic modulus of the rock was known. The method was to cut a circular groove into the wall of the gallery. This relieved the stresses acting on the core. The increase in diameter of the core was measured by means of strain gauges. Knowing the elastic modulus of the medium the value of the stress could be computed.

The stress can also be measured directly by what has been called the flat-jack method which we suggested in 1951. To measure the stresses in the wall of an underground cavity, the procedure is to stretch several vibrating strings between anchor points cemented into the wall. The tension on each acoustic string is recorded. A groove is then cut into the rock between two of these gauges and a flat jack of the Freyssinet type is placed in it.

During the process of cutting this groove the stresses on the rock are relieved and the tension on the acoustic string becomes less. When the flat jack has been placed, hydraulic pressure is supplied until the initial state of stress in the rock—as shown by the tension on the acoustic cord—has been restored to its initial value. The hydraulic pressure in the flat jack is now equal in value to the stress initially present in the rock; that is to say to the component of stress in the direction vertical to the surface of the jack.

As the flat jack is welded, it cannot take a pressure greater than about 200 kg/sq. cm, which corresponds to a depth of rock less than 1,000 m. But within this limit the method has been used with great success by M. Tincelin in the iron mines in eastern France. Here the galleries are at a depth of only 200 m and the stresses in the rock are of the order of 50 kg/sq. cm.

The flat-jack method has recently been taken up and developed by various specialists in order to measure stresses at great depths in boreholes. I will return to these developments later.

Finally, during these past 10 years model tests, taking into account the deformation of the rock, have been widely developed in order to compute the stresses in the structures.

This method has been brought to a high degree of refinement by Professor Oberti at the Ismes at Bergamo, and by M. Rocha and M. Laginha at Lisbon.

This was the state of rock mechanics in 1953 as it was known to me at the time. For the sake of completeness I would mention here the studies made at the Bureau of Reclamation, by Dr Muller of Salzburg and by Professor Hast of Stockholm, of which I had not yet heard. However, these studies did not to my knowledge involve methods other than those that have just been described.

1953-1963: RECENT DEVELOPMENTS

As has been said before, during this period many different organizations have devoted large parts of their efforts to the study of rock mechanics. In describing some of their activities I will concern myself mainly with the studies made in France and with which I have myself had contact either directly or indirectly, first in the laboratory and subsequently in the field.

A. Laboratory studies

Before considering the mechanical properties of rocks and how these properties are modified by applied stresses, it is my object to mention recent studies on the alteration of rock; that is to say, on the changes brought about in rock by the action of water, by the variation in the water-vapour content of the atmosphere ambient to the rock, and by the changes in temperature.

(a) *The alterability of a rock.*—This property was studied, in liaison with Électricité de France, first by M. Farran and then by M. Thenot, in the mineralogical laboratory of the Faculty of Sciences at Toulouse.

The alterability of a rock is a function of its mineralogical nature and of its degree of fissuration. Both these factors were studied intensively.

In regard to fissuration, a distinction was made between different classes of cracks, according to their size, as follows:

A *macrofracture* has a width more than 0.1 mm. Sometimes it is empty, but often it is filled with alteration products.

A *microfracture* has a width less than 0.1 mm, but still of the order of the 0.01 mm. These microfractures are usually difficult to see by the naked eye. They represent schistose joints or narrow fractures but they can extend to cover major surface areas. They usually follow well-defined directions in space.

In addition to these two classes of cracks there is also the *isotropic matrix fissuration*, where the width is less than one μ and the length of the fissure of the order of the length of a crystal. This kind of microfissuration can be seen in a petrographic section cut from a piece of rock that has been suitably impregnated with coloured resins.

This kind of fissuration can be quantitatively evaluated by the fissure area per unit volume of rock, and in spite of the very small width of the fissure it strongly influences the rapidity of the alteration process; that is to say the rate at which the minerals of the rock are attacked by water.

Recent work has shown that water can travel through a soil or a rock of low porosity either in the vapour phase or, in certain cases, in the adsorbed phase (Société Hydrotechnique de France, *Compte-rendu des VIIèmes Journées de l'Hydraulique*). A capillary of only a few molecules in diameter is enough to permit the flow. The walls of the fissure acquire a monomolecular skin of water and through this skin the mobile water molecules travel along in the adsorbed phase.

M. Farran and M. Thenot have shown that the alterability of a rock can be expressed as the product of two factors. The first they call the specific alterability of the rock. The second is its specific surface.

The specific alterability is an experimental quantity determined as follows: A cylindrical sample of rock is prepared, 3 cm high by 4 cm in diameter. This is mounted so that water can be applied to one end under a pressure of 250 kg/sq. cm. The water will travel along through the interior of the cylinder and issue at the other end. The barrel surface of the cylinder must be carefully sealed off here to prevent any superficial flow. The rate of flow is recorded and the rate of change with time determined. The effluent water is examined by photometric analysis.

If the rate of flow remains constant, and the removal of elements (particularly calcium

ions) is small, the rock can be called of low alterability. For example, in the case of granites, which in practice are of very low alterability, calcium is removed at a rate lower than 5 mg per litre of filtrate and per hour of leaching.

But if the rate of flow is found to diminish during the run, this can be presumed to be due to blocking of the capillaries by reaction products, or to the swelling of clay minerals, both signs that the rock is alterable. To express the matter quantitatively it may be said that the rate of decrease of permeability with time is a measure of specific alterability.

The second factor is the specific surface of the rock, which is a function of its micro-fissuration. Determination takes the form of a measurement of the permeability of the rock to air. This air-permeability is a characteristic of the rock which has received great attention from the specialists in the exploration of oilfields. They have developed a variety of instruments for measuring it, all based on the simple principle that air is sucked, under a given pressure differential, through a sample of rock of standard dimensions. The volume of air passed through in unit time is a measure of the air permeability of the rock. An apparatus of this kind is in use at Toulouse (Fig. 1).

The following Table shows the results of some permeability tests on granites from different sources. The tests were made for Électricité de France by the Laboratoire de Minéralogie de Toulouse. The column on the right briefly describes the aspect and behaviour of the granite tested.

As all the samples in these tests were granites, of one sort or another, it may be assumed that the specific alterability is the same in all cases. Thus the variable that determines alterability is in each case the permeability to air of the rock.

It will be seen that there is a very good correlation between permeability to air and the condition of the rock. For a case like this, where a group of rocks all of the same nature are compared with each other, the air permeability is therefore quite a good criterion for the alterability of the rock.

Fig. 1. Air permeability measuring device

Table 1

Sample	Permeability to air in cm/sec.	Remarks
A	1.8×10^{-12}	Very good condition.
B	$2.5 \text{ to } 3.2 \times 10^{-12}$	Very good condition.
C	1×10^{-11}	Rock in very good condition, no alteration.
D	$1-3 \times 10^{-11}$	Rock is compact, stained with rust along large fractures but the main part of the rock is not coloured. No deep alteration. Some surface attacks.
E	$3 \text{ to } 4 \times 10^{-11}$	The quartz crystals are bared and the rock is coloured in the mass.
F	$1 \times 10^{-11} \text{ to } 3 \times 10^{-10}$	The rock is friable and looks as though it will soon break down into sand.
G	$1.2 \text{ to } 3 \times 10^{-10}$	The permeability is very high, but the granite is little altered. There are vacuoles from the disappearance of feldspars.
H	$1.4 \text{ to } 3.2 \times 10^{-10}$	The rock crumbles easily. The increased permeability corresponds to a more advanced disintegration.
I	$1.4 \text{ to } 5 \times 10^{-9}$	The rock is obviously in a state of rapid change. Tinted with rust in the mass, crumbles easily.
K	$1.4 \text{ to } 5 \times 10^{-9}$	

These measurements provide a means for grading rocks in order of increasing alterability, but even where this property cannot be expressed with quantitative precision the observations can still reveal facts of the greatest practical importance. When the Carla pressure tunnel, for example, was being driven for the Électricité de France, the laboratory found that certain parts of the rock wall showed high alterability values. In order to obtain a practical check on the laboratory findings, some areas of this rock were left unlined and thus exposed to the action of the water, together with some less alterable areas for comparison. When the tunnel was subsequently emptied, several months later, it was indeed found, as predicted, that the alterable areas had suffered considerable attack, whereas the non-alterable areas were not affected. Thus the laboratory predictions were confirmed.

Fig. 2. Porosity tests. Samples of schist. Between diameters δ and $\delta+d$, there are $N\delta\delta$ pores per sq. cm

dropped it into a bucket of water. The piece of rock had hardly touched the bottom of the bucket before it literally exploded into dust. The explanation was that in its natural state, in the body of the rock, this schist was saturated with water. The piece lying on the ground and thus exposed to the air must have dried out. When this piece of dried-out rock was immersed in water, the water penetrated into the capillaries, enclosing spaces containing dry air in which a high vapour pressure quickly developed due to the very small meniscus radii, causing in effect multiple explosions in the schist with almost instant disintegration.

The remedy was simply to prevent the newly cut surface from drying out, by applying rapidly a surface gunnite coat to hold the moisture in.

I have on occasion come across rocks with somewhat similar characteristics but not so pronounced, particularly in eastern France where there is a schist called "cardboard schist" because when a core of this material is exposed to the air it flakes off into thin, parallel disks that look like cardboard. The engineers studying a site for an excavation, to take the foundation structure of an ironworks, were not pleased to find this kind of schist there. But it was necessary merely to cover the newly bared ground with a coating of bitumen to seal in the moisture and thus avoid all difficulties. With this precaution the schist is quite capable of supporting very heavy loads. What is necessary, here again, is to prevent a desiccation of the ground, followed by re-humidification.

(b) *Deformation and rupture.*—On the initiative of the Institut Français du Pétrole a

Similar practical tests were made at Bort, also with affirmative results, and since this second confirmation the Électricité de France have made it their practice to have alterability tests made on the rock surface of a tunnel before deciding whether or not to provide a protective lining. In this way alterability measurements can allow considerable economic savings to be made.

There are other types of rock where certain kinds of behaviour can be explained only in terms of the pore dimensions and the resulting permeability. This is particularly true of certain types of schist (Fig. 2).

A case of this sort occurred during the construction of the Fourn el Geiss dam in Algeria. It was intended to cut an almost vertical wall in the schist and then apply a concrete lining. The terrain seemed well adapted to the procedure till somebody picked up a piece of schist which was lying on the ground and had therefore been exposed to the air for a long time, and

special study of these properties of rocks was made by the laboratory of the École Polytechnique, which acquired equipment for studying the intrinsic curves of natural rocks, and also the effect of rock flow.*

For the study of the intrinsic curve the laboratory has available a triaxial apparatus with a capacity for horizontal pressures up to 1,000 kg/sq. cm and vertical pressures up to 7,000 kg/sq. cm. Measurements currently made include crushing of rock with lateral constraints of zero, 250, 500, and 1,000 kg/sq. cm.

The vertical pressures causing rupture at the different lateral constraints describe the intrinsic curve of the rock. It is interesting to observe that all the rocks examined in these studies behave in the same way with changing lateral stresses. With zero lateral constraint, that is, when only vertical pressure is applied and precautions are taken to prevent surface friction on the head and base of the sample, all the rock samples ruptured along vertical planes parallel to each other. If some lateral stress is applied, the sample ruptures by inclined planes of failure, as would a plastic clay. If the lateral constraint is increased sufficiently, the sample of rock first squashes out by plastic deformation into the shape of a barrel, and then ruptures by a series of planes of failure parallel to each other and symmetrical to the axis of the sample (see Figs 3 and 4).

In the presence of very high lateral constraint and consequently high friction values, the sample of rock under high vertical pressure does not rupture at all but merely becomes deformed by plastic flow, without there appearing in the rock any fissure visible to the naked eye. The samples after this test all have the same aspect, whether the rock is

a plastic clay, a piece of sylvinitite, or gypsum, limestone, or granite. In each case a Mohr diagram can be drawn, showing an intrinsic curve delimiting the zones of plastic deformation and of rupture. This is not a straight line system but rather a parabola with its apex on the Ox axis at a point whose abscissa represents the tensile strength of the rock.

On the basis of these studies it is possible to predict the conditions under which a given rock will flow or rupture. Thus when a tunnel is driven through rock it is often possible to predict how and where cracks will develop. Knowing this, steps can be taken to prevent rock falls, by rapidly intervening before the ultimate state of stress has had time to develop in the exposed rock, and by inserting tie-bolts across the probable planes of rupture.

This method was applied on a large scale during the driving of the Mont Blanc tunnel (Renaud, 1962). Here the rock, which was a granite, had already acquired a stratified structure from the tectonic movements that had formed the mountain chain. It was found that there was already a preferred direction for the planes of rupture. But the mountain load applied to the walls of the tunnel as the drilling proceeded, developed vertical stresses, and caused cracking in nearly vertical planes at the springing of the vault. Bolts were inserted as soon as a length of gallery was opened, and although this may not have entirely prevented the development of fissures, it certainly helped to counterbalance dangerous effects. In this way several tens of thousands of tie-bolts were inserted to act against shearing of the rock and so to prevent major rock falls (see Figs 5, 6, and 7).

* Similar studies had been made at the Engineering Laboratory of the Bureau of Reclamation at Denver and published under "physical properties of some typical foundation rocks" in 1953 (see Fig. 4).

Fig. 3. Rupture of samples under varying lateral constraint

(c) *Rock flow*.—These tests relate to a load rapidly applied to a sample of rock. A different problem is presented by deformations taking place in rock that is permanently stressed. A study of these effects is made difficult by the long periods of time during which the samples of rock have to be kept under load.

In the laboratory of the École Polytechnique there are in use twenty-four apparatuses for applying stress to rock samples. Among the many experiments begun some will continue through periods of time of up to several years.

At present it may be said that the deformations, shown after a given period by different rocks under stress, differ considerably with the nature of the rock. With certain rocks like unsaturated gneiss, up to 80% of their compressive strength, there is no noticeable delayed deformability. With others a maximum deformation is reached after a certain time. A third possibility which, until now, had not been detected, is a flow with a progressive hardening of the material. This happens for instance in the case of chalk, of potash, and of sedimentary iron ore as mined in the east of France.

The time-deformation log, after a short straight variation for low stresses, gets curved; the deformation decreases until a new ratio is reached for which the variation again is linear. This period corresponds the hardening of the material. It is followed by an increase of the deformations until the rupture occurs. In other words the material first behaves like an elastic body, then, progressively, gets more and more resistant under the load and remains so for some time. It finally gets softer, deforms and breaks down (see Fig. 8).

This observation made in the laboratory has already been checked in the field and used in mines to control the stability of different stopes. Even in cases where nothing shows that the compression on pillars increases, accurate measures indicate a slow settlement which progresses regularly. As long as the settlement varies with the time in a linear way, the risk

of a rupture is still remote. But as soon as the relation is no longer linear there is a danger of collapse and either the mine must be vacuated or the pillars should be strengthened. It is interesting to note that this behaviour has already been observed both in iron ore and in chalk mines and that these experiments have shown a possibility of improving the safety in a number of cases.

At the moment this question is only making a start but it seems certain that in a relatively short time important results will be achieved in this line which shall apply not only to mines but also to civil engineering works.

As a complement to these brief observations the following are different values of the maximum flow to instantaneous deformation for particular materials:

	per cent
granite	up to the present, less than 10
calcareous iron ore	10-20
coal	30-40
chalk	100
sylvinite (potash)	500

Fig. 8. Rock flow

Fig. 4. Limestone sample before and after triaxial compression (Bureau of Reclamation, 1953)

Fig. 5. Mont Blanc tunnel. The Jumbo in place—In foreground, the rock bolts

Fig. 6. Mont Blanc tunnel. General view of the inlet into the Mont Blanc tunnel

Fig. 7. Mont Blanc tunnel. Fissured granite and rock bolts

(a) Section showing the two parts of the deformation gauge

(b) Measuring device placed inside the conic outer shells

(c) Measuring device sliding between the outer conic shells

Fig. 9. Instrument devised by the C.E.B.T.P. (Paris) for the measuring of the modulus of elasticity of the rock in situ

These figures show how important it is to know the speed and the value of the rock flow for the computation of the stability of a structure set on rock, particularly in the case of hydraulic works.

(d) *Elastic moduli as found by static and by dynamic methods.*—A question that has interested research people for several years is the difference between the elastic modulus of a rock as measured by a static method, and the same quantity as obtained from the measured velocities of sound waves. In the laboratory, when a substance which is at least macro-homogeneous—for example, concrete—is tested by both methods, the values found agree quite well. In situ the results are often very different.

It has often occurred, as an example, that the static method has given moduli ranging between 4,000 and 40,000 kg/sq. cm, whereas the calculated modulus on the basis of sound velocity through the rock has ranged far above 100,000 kg/sq. cm. The explanation sometimes given is that the jack measurements are on small areas of a rock that have developed surface fissures, whereas the waves are transmitted through intact rock at a certain depth.

The problem deserved closer study, particularly in order to confirm whether the relationship understood up to the present to exist between the velocity of waves and the elastic modulus of the rock, was really valid under all circumstances, or whether it should be corrected for the effects of the applied stresses.

Measurements were therefore made in the laboratory on rock samples that had previously been subjected to high stresses. The samples were tested by both the static and the dynamic method, and the elastic modulus was calculated in each case. The stresses previously applied ranged all the way up to the beginning of plastic flow.

It was found that this previous stressing hardly affected the results of the static test, but that the velocity of sound through the rock was strongly influenced by the internal disorganization accompanying the incipient flow.

For example, tests on gypsum showed that a previous hydrostatic pressure of 500 to 1,000 kg/sq. cm caused the velocity of sound to fall from 3,600 m/sec down to 1,800 or even only 1,200 m/sec. On the other hand, if the stress/strain curve is drawn for different applied loads it is apparent that the elastic modulus calculated from the static tests remains unchanged.

From these considerations it emerges that the simple equation:

$$V = \sqrt{\frac{E}{\gamma} \frac{1-\nu}{(1+\nu)(1-2\nu)}}$$

is not adequate when applied to waves transmitted under load, but requires a further term to allow for this. This problem is the subject of research at present under way at the École Polytechnique.

Moreover, even these remarks do not entirely cover the problem of the divergencies found in the field. In the case of rock known to have suffered tectonic deformations, and which had previously been subjected to very high stresses, the elastic modulus calculated from sound velocity was much higher than the value calculated from the static test. As the result of stresses is to reduce the sound velocity, it may be supposed that if there had been no tectonic movements the two values would still be wider apart.

Such are the lines of study that this laboratory has adopted in the course of its first year of existence. But laboratory work is only part of the research and, even though the studies in the field have for a long time lacked sufficient basic support, the essential part of the research in rock mechanics must be the field work.

It is now intended to describe some of the improvements that have been made in the course of the past few years in field methods, in ways of measuring the properties of a rock in situ.

Finally, I will refer to the artificial modification of a natural terrain to improve it for the engineer.

B. *The measurement of rock characteristics in situ*

(a) *The modulus of deformation.*—During the past 10 years the methods already described for measuring rock characteristics in the field have been used where required by the civil engineers. But these methods have their drawbacks. The jack method for finding the elastic modulus relates to small wall areas in the gallery where the test is made. A great many measurements are required to give an average, and even this value tells nothing about the rock at depth. Velocity measurements do, of course, give average values related to certain wave paths; but the waves practically always travel in sound rock, avoiding the obstacles; that is the fissures. The values obtained are therefore generally too high. There is also the uncertainty caused by the effect on the velocity of stresses left over by ancient tectonic movements, a problem to which the laboratory has drawn attention.

It was therefore decided to examine the possibility of developing a device that would measure stresses and deformations at depth in a borehole. The Électricité de France have formed a working party whose task it is to bring together the various existing apparatus for making these measurements, to examine them, and to make comparative tests. A report on this work will be submitted to the next Congress for Large Dams in 1964.

Here it should be remembered that the requirement is merely for an apparatus capable of measuring the modulus of deformation of the rock in situ, not for making direct measurements of the ground stresses.

The following types of apparatus will now be discussed:

(1) *The C.E.B.T.P. apparatus*

This instrument has been developed by Dawance, Bonvallet, and Noel. It is based on the principle of the flat jack, but is so constructed that it can be introduced into a borehole (see Figs 9(a), (b), and (c)).

The apparatus comprises a solid steel cylinder divided by an axial cut into two halves. The two halves are partly hollowed out to receive a pair of rubber bags, which are filled with oil and so arranged that by increasing the oil pressure the bags tend to push the steel half-cylinders apart, the apparatus thus acting as a jack.

An induction extensometer sensitive to 0.01 mm is located between the two rubber bags and allows for an accurate measurement of relative movement between the two steel halves. The apparatus thus measures the stress applied and the deformations of the borehole. The figure obtained in the field is then compared to a scale drawn in the laboratory after tests on plaster, lean concrete, and hard concrete, which relates their modulus of deformation to the figures given by the instrument. The elastic modulus of the rock is thus obtained.

This jack must be set firmly in place in the drill hole. Initially this was done by casting plaster or an alloy of very low melting point around the instrument. The process was quite delicate since it is necessary to recover the apparatus after the measurement.

The method used at present is to anchor the jack in the hole by means of two conical half-shell pieces similar to those used by Professor Potts, that fit together to form a steel sleeve. In operation, the assembly forms a tight plug seal in the drill hole. After a measurement has been made, one of the conical elements is pulled out and the assembly can be withdrawn from the hole, or it can be moved to another position higher or lower in the hole. With this facility, the apparatus can be used to examine the whole length of a drill hole in quite a short time.

By this method it should be possible to make a rapid examination over the length of a borehole measuring the modulus of the rock at all depths and in different directions, and thus to obtain the data required for calculating the abutments of a dam (see Figs 10 and 11).

A further point of considerable interest has been brought out by these studies. Tests were made in the laboratory on blocks of plaster, and subsequently on blocks of concrete, to find out how the elastic modulus changed with increasing load. The block was placed between the plates of a press and stressed under controlled conditions. It was found that the stress/strain curve had a kink dividing it into two parts, each providing a distinct value of the elastic modulus. It has been found by experience that this kink corresponds to the pressure in the test block. It is therefore assumed that, in the field, the kink should occur as soon as the pressure provided by the jack becomes equal to the external pressure on the gauge. This is a method for determining this pressure. This has been checked in the field several times and allows the measurement of a pressure in the ground.

It may therefore be said that this apparatus allows complete testing over the whole length of a borehole, giving for each depth the elastic modulus of the rock and also the ground pressure at that depth.

It also has an interesting potential in that it has a very small diameter. The one now in use is of conical shape, the overall diameter being only 70–80 mm. This instrument can therefore be sent down an ordinary borehole, thus providing the most economical mode of use.

It may be felt that this description does not sufficiently refer to the formulas which have been devised in order to obtain the rock characteristics from the deformation under stress.

Fig. 10. Measurement of the modulus C.E.B.T.P. of elasticity in a borehole with the gauge. Test on the gneiss at La Bathie Roseland. Test 1.

Fig. 11. Measurement of the modulus of elasticity in a borehole with the C.E.B.T.P. gauge. Test on the gneiss at La Bathie Roseland. Test 2.

They have been reviewed but as they all are based on homogeneity, isotropy, no fissuration, they really did not apply to actual field work.

(2) *The Janod-Mermin apparatus*

This apparatus was developed almost 10 years ago by Janod and Mermin, engineers on the staff of the Électricité de France, and was put into operation in 1956 to help with the study of the deformation of the rock under stress in the terrain to be passed through by the lower parts of the Roseland pressure tunnel.

It comprises a steel tube 770 mm long, 164 mm diameter, with a wall thickness of 10 mm. The steel tube is enclosed in an outer sleeve of annealed aluminium, the two tubes being joined by ring seals at either end so as to leave a hollow space between the two.

In operation, this jacket space is filled with water, which is put under pressure. The outer surface of the aluminium sleeve presses against the rock wall.

For the measurement of lateral extension there are two pairs of distance feelers arranged to measure lateral extension in two directions at right angles to each other. Each feeler passes through the wall of the steel tube by means of a pressure-tight piston bush. The outer tip of the feeler rests in contact with the inner surface of the aluminium sleeve and the inner tip touches an adjustable wedge, which transmits movement to the sensor of the instrument proper.

This apparatus, once in place, allows determination of the stress/strain curve for the rock in two directions at right angles. The assembly is extremely robust and can give readings up to about 250 kg/sq. cm, which is the limit for the joints.

(3) *The apparatus made by Mr Kujundzic, of the Cerni Institute in Belgrade (l'Institut d'Hydraulique Jaroslav Cerni)*

This apparatus is derived from the Janod-Mermin, and Mr Kujundzic has used it a great deal in Yugoslavia. Instead of the aluminium sleeve the Kujundzic apparatus uses a rubber cylinder with a wall thickness of 10 mm, which ensures good pressure distribution on the rock wall. In this case lateral extension is measured by feelers passing through pressure-tight glands in the wall of the instrument, and acting on a Huggenberger cell.

In spite of its size and the diameter of the borehole required, the Kujundzic apparatus is apparently in frequent use in Yugoslavia.

So far as is known to the Author these are the only instruments that have been developed for measuring directly the elastic modulus of rock in situ. There are quite a number of instruments for measuring stresses in boreholes. Some do this directly by simultaneous measurement of deformation in two planes at right angles, assuming that the rock is homogeneous and isotropic, and applying the corresponding formulas.

Other instruments allow indirect determination of the stresses based on the knowledge of the elastic modulus of the rock as previously measured on a core sample in the laboratory.

There are in particular the instrument developed by Professor Potts of the University of Durham, Newcastle; the apparatus of Mr Leeman of the South African Council for Scientific and Industrial Research; that of Professor Hast of Sweden; the instrument of Mr Obert of the American Bureau of Mines; the instrument of Mr May of the Department of Mines of Canada; and that of Mr Averchin of the U.S.S.R.

This long list shows how great is the interest that has developed in the course of recent years in many countries in this problem.

These instruments have all been built for mining engineers for the measurements of depth stresses.

There is not much to say about the instruments used for the dynamic measurements. These are the same as those used in geophysical work and, except for the use of transistors,

no great changes in design have occurred during the past 10 years. Practically the same methods are used today in all countries.

The shear resistance of rock.—The advantage of the measurements of moduli, which have been described, is that they give values which can be used as such in the computation of structures. Other tests have been devised, particularly the measurement of the shear resistance of the rock and of the adherence at the rock-concrete contact surface. The trouble is that these tests are very costly, very difficult to carry out, and, even so, the resulting figures are nothing else but general information and not a basis for the calculation of the structures. As an example of this type of tests, the set-up adopted by Habib and Bernède for a series of tests carried out for the Cie Nationale du Bas-Rhône et du Languedoc will be described shortly.

In a gallery, a short tunnel is driven, leaving on both sides a protruding cube cut out of the rocky mass. These two protruding elements are embedded in a concrete block reaching across the tunnel and cut in two in the middle so that a flat jack can be inserted between both halves. The lower face of the block is laid on a slab coated with bitumen so as to reduce the friction. When the block has been poured the space between the block and the end of the tunnel is filled with concrete, strongly reinforced along the bottom face of the block. A slot parallel to this face and of the same length as the block is kept open while pouring the concrete so that another flat jack or better two of the same dimensions can be placed in this slot. Using these different jacks, compression and shear stresses can be applied to the rock. If the compression is kept constant, the shear stresses can be increased until rupture occurs. The shear resistance of the rock can thus be calculated. If the rock can be considered as homogeneous and isotropic and if several tests could be made with different values of the compressive strength it would be possible to draw the corresponding Mohr diagram. As the tests are costly and painstaking, one test only is generally made, so that the information obtained can only be used as a check of the values reached by other methods (see Fig. 12).

A similar set-up makes it possible to measure the adherence between the rock and the concrete. In the cases of which I have known, the plane or better the surface of rupture was entirely in the rock. This is not surprising, as generally these measurements are made only when the resistance of the rock is doubtful and because the test can only be made after high explosive has been used to drive the tunnel so that cracks have been produced near the surface.

Mr Kujundzic from Belgrade and Dr Muller from Salzburg have made similar tests with more or less expensive arrangements. The trouble with these tests is that they are much more costly and painstaking than the ordinary jack tests, and, in spite of it, they give only particulars of one location and, even so, for a superficial area of rock. The information gained can therefore not be considered as representative of the average material.

C. *An example of a study on rock in situ*

Among the many studies that I might describe I will select one, now under way for a dam in the south of France, because these studies typically show what great differences there can be between elastic moduli for the same rock, calculated on the basis of different methods of measurement.

This dam is intended as a defence against floods. The suggested site is in a gneiss massive, and seemed at first to be quite favourable, even though lying between various conspicuous surface irregularities.

The rock is rather cracked, particularly on the left bank of the river. On the other hand, there are present in the river bed large blocks of rock which seem quite sound, little fissured, and not altered, and which suggest that the fissuration on the banks is fairly superficial.

The preliminary study of the project had suggested the insertion of an arch dam, and it was to confirm this idea that a reconnaissance programme was launched, which included driving four galleries into the rock, one on each bank upstream and one on each bank downstream of the intended position for the dam.

Since the instruments for measuring elastic moduli at depth in a drill hole which have been described before are now up to experimental prototypes, jacks were used, practically of the same type as those used 10 years ago in the preliminary rock studies for the bearings of the dam at Tignes.

Fig. 12. Shear resistance of rock test in situ

jack the stress/strain curve climbs with little steepness. If, however, the pressure is allowed to rise to a moderate value and is then taken off the jack, the return line drawn in this way is steeper and cuts the x-axis at a point determined by the residual deformations remaining at the support areas. If the pressure is again increased the stress/strain line rises as a nearly straight line the slope of which is independent of the pressure previously applied, but can differ for different support points (Fig. 13).

Thus two slopes were obtained. The one first recorded was called E' and that obtained in the repetition run E . The latter corresponds a quasi-elastic deformation but whether it can be regarded as the true elastic modulus of the rock is difficult to say.

When pressure is first applied the rock gives way comparatively easily, an effect corresponding to the closing of the microfissures in the affected zone of the rock. This zone is that for which Boussinesq, having calculated the stresses in depth on the assumption that rock is an elastic semi-solid, had found appreciable values.

Thus, even if the rock is not perfectly elastic, the physical phenomena are still the same; that is the stresses in depth resulting from a thrust applied to the surface are a function of the thrust applied and of the bearing area of the pad. The first load on the pad gradually closes the fissures as far as a certain depth. On the application of further loads the rock behaves as a quasi-elastic substance; for example, the initial curve for increasing pressures is not subsequently covered by the return curve for decreasing pressures, but the subsequent curves, there and back, all cover each other perfectly.

A consequence of this behaviour is that, for each position in the gallery, two values E' and E are obtained for the modulus of deformation—the first for the initial loading, the second for subsequent cycles.

In order to compare the results with the dynamic method the propagation velocity of sound waves was also measured. Inside the galleries it was at once apparent, merely by glancing at the exposed rock wall, that the cracks were not confined to the surface but, in fact, penetrated far into the terrain. The engineer in charge was therefore requested to drive these galleries, which were only about 12 m deep, to greater depths. This is at present being done and tests will be resumed shortly.

But the results of these first tests are of considerable interest in their own right. The jacks were loaded up to as much as 100 tons thrust, corresponding to an applied pressure of 80 kg/sq. cm on the rock wall. This is more than 50% higher than the pressure applied by an arch dam.

While the pressure in the jack was being increased, a running record was made of the elastic modulus; that is, of the slope of the tangent to the stress/strain curve. It was found that when thrust is first applied by the

If the latter is not the true elastic modulus of the rock, then at least it is a quasi-elastic modulus.

Table 2 shows the values of E' and E measured at five different places at the site in question. The Table also shows the ratio of the residual strain to the maximum measured strain, for each cycle. The final column shows, by extrapolation, the amount of flow to be expected after one year of application of the maximum stress. This figure gives an idea of the magnitude of the possible deformation after construction. In all the tests the amount of further settlement after the initial settling of the pad was very little. This could mean that the microfissuration was not extensive and was quickly closed by the first application of load to the pad of the jack. Further deformation would then be very little. Under these circumstances the value given in the Table for the elastic modulus would remain valid irrespective of the duration of the stressed condition.

It will be seen that the total elastic modulus is quite low, even along the bottom of the valley where the rock appears to be sound. These values are particularly important for the engineer designing a dam because this kind of structure always rests on decompressed rock. Actually in the case under investigation the quasi-elastic modulus E was always found to be of the same order of magnitude as the initial modulus E' .

After this first survey the engineers could not feel confident that an arch dam could be constructed in this position without expensive consolidation of the massive. The decision was taken to drive the exploratory galleries further in and to see if and where the rock might become more compact and better able to provide a good bearing.

While these static measurements were being made, the same laboratory was simultaneously making an extensive series of dynamic measurements. These dynamic tests are done much more rapidly than the static tests, and they were consequently in greater numbers. Examination of some forty separate measurements revealed the following grouping.

1. The measurements made near the surface on the fissured part of the massive showed propagation velocities close to the value 2,000 m/sec.
2. The measurements made on the apparently sound parts of the massive, the non-fissured part gave velocities grouped around 4,000 m/sec.

From these measurements it is clear that the results do give a general idea of the fissuration. But do they provide anything more quantitative than a general comparison?

As already mentioned the usual expression, which assumes that the medium is elastic and isotropic, takes the form:

$$V = \sqrt{\frac{E}{\gamma} \frac{1-\nu}{(1+\nu)(1-2\nu)}}$$

But if this expression is used to calculate the elastic modulus of the rock from the wave velocities measured at the positions of the three jacks 2, 3, and 5, and compare these calculated

3*+

Fig. 13. Jacking test in gneiss in a tunnel 7 m from the entrance at St Jean du Gard

Table 2
Results of mechanical tests in situ

Test No.	Gallery place of test	Nature and state of rock	Direction of stress	Elastic modulus: kg/sq. cm*			Rock flow after 1 year under stress of: 55 kg/sq. cm (microns)
				E' Total	E Rapid reversibility	Relative residual depth of impression: W_r/W_t %†	
1	<i>R.D. downstream</i> 7 m from the inlet (floor)	Altered gneiss Many clefts and fissures	Almost vertical dips towards South	9,600	24,000	59-50 43-40	50
2	<i>R.D. downstream</i> 7-50 m from inlet (wall on side of mountain)	Ditto.	Almost horizontal, dip towards South	28,000	42,000	40-29 17-16	15
3	<i>R.D. downstream</i> 9 m from inlet (wall on side of mountain)	Ditto.	Horizontal	18,500	30,000	50-30 16-14	—
4	<i>R.D. downstream</i> 11 m from inlet (wall away from mountain)	Mylonitized zone, vertical dip	Horizontal	6,000	17,000	60-45 31-29	40
5	In the bed of the Gardon	Apparently sound	Horizontal	50,000	96,000	50-27 14	—

* Deformation under the pad obtained by extrapolation.

† W_r (residual) W_t (total) W_r/W_t for each cycle and in chronological order.

dynamic values with the static values previously found, no agreement is reached as following Table will show.

Table 3
Elastic modulus, in bars (bar=1.02 kg/sq. cm)

Position of jack number	Total modulus E'	Quasi-elastic modulus E	Dynamic modulus E
2	28,000	42,000	290,000
3	18,500	30,000	275,000
5	50,000	96,000	620,000

Agreement is not good even between the dynamic modulus and the final static modulus E after the first settling of the pad of the jack; in fact, even there, the dynamic modulus is 6 to 10 times higher. It is evident that these dynamic values do not give a reliable picture of the mechanical quality of the rock.

Discrepancies of this kind are systematically revealed by tests carried out in the field and it was considered worth while that Habib should undertake a study of this problem and try to discover the relationship between wave velocity, as measured at the site, and elastic modulus as understood by the civil engineer.

These differences have also been evident for some time to M. Bollo of the Société de Recherches Géophysiques, who has very kindly allowed me access to his records for the preparation of the present lecture. In his reports, M. Bollo always quotes propagation velocities as measured by micro-seismograph. When he finds it necessary to quote an elastic modulus he compares the dynamic values obtained to static measures made at the site without involving theoretical expressions. It should be mentioned here that all the studies that have been referred to, were made on fissured terrains which have in the past suffered tectonic movements. What the results would be in compact, non-fissured granite, such as is found in Sweden, is not known. But in regard to all the rocks tested in France in practically no case has there ever emerged a satisfactory agreement between the dynamic and the static values for elastic moduli.

D. The artificial improvement of rocks in situ. Treatment by grouting

It is not my intention to end this presentation with the confession of the shortcomings of our laboratory. I prefer to mention the continued advance that has been made recently in the improvement of the mechanical properties of rock notably by means of grouts.

Before describing what is being done today, let me first go back more than 10 years, to Morocco, where a study was being made on the lining of the gallery at Im Fout.

The results have been reported by Mr Bernard (1953) to the 1953 Conference at Zurich.

In order to investigate the possibility of lining a low-pressure tunnel with unreinforced concrete, grouting tests were made so as to compare the modulus of deformation before and after, as well as the final prestress obtained. This was done in a gallery with lining in place, by injecting behind the lining a charge of cement grout to a depth of 4 m into the rock, under a driving pressure of 30 kg/sq. cm, and using 175 kg of cement per square metre of gallery wall.

In order to obtain a picture of the deformations in the lining, eleven acoustic wires were installed, each 20 cm long. These instruments indicated high compressive stresses (89 kg/sq. cm on the average) corresponding to about 2/3 of the internal pressure required in a tube of the

same diameter and wall thickness to develop a uniform external pressure equal to the injection pressure. After 14 days the deformations had still a residual $\frac{2}{3}$ of their initial values. By this time the cement had set hard and the stresses applied to the lining by the rock had become finally stabilized.

The effect of these measures is shown in Table 4 which gives the elastic modulus of the rock before and after the injection, as measured by a jack in the gallery.

It will be seen that as early as 1953 grouting was successfully used to improve artificially the mechanical qualities of a rock in situ. A second effect achieved at the same time was a substantial prestressing of the lining, in the sense of a compressive stress, with evident mechanical advantage. But the most interesting thing about this early operation is that, already then, the improvements to the characteristics of a terrain have been measured and the effect of the work strictly controlled.

To what extent the abutments of a dam can be improved by this method had been successfully demonstrated since 1947-48 at the Castillon dam, where one of the abutments was consolidated by the injection of 9,000 metric tons of ternary grout with cement.

Finally, let me describe a recent example, the dam at Avène, an arch dam that was finally constructed after extensive grouting following a careful study of the possibilities of artificially consolidating the abutments.

Table 4

Direction of measurement relative to dip of rock	Elastic modulus of rock				
	Before injection Jack test			After injection	
	Initial elastic modulus: E'	Final elastic modulus: E	Test in gallery	Jack test: E	Test in gallery: E
Perpendicular	10,000	57,000	25,000	100,000	97,000
Parallel	12,500	43,500	Not measurable	200,000	236,000

Both static and dynamic tests were made on the site. The static tests were carried out in a tunnel 2.50 m in diameter and 22 m long through quartzite rocks at a level 10 m below the position for the abutment on the right bank, the tunnel being driven to near a fault well known in this part of the site.

In the floor of the gallery, two shafts were sunk with the intention of making measurements in the direction of the abutments and in a direction at right angles to this.

Two series of measurements were made, the first before and the second after the experimental consolidation of the rock. After making the first series of measurements, with the jack thrusting against the natural rock, the gallery was lined and the shafts filled with sand. The polished places on the rock wall that had been used for applying the pad of the jack were protected by covering them with several layers of paper.

The rock was then grouted from the interior of the gallery. These injections were done in two phases. In the first phase injection under a pressure of 5 kg/sq. cm was made at the plane of contact between the concrete and the rock. A second series of injections was then made at positions forming a halo at a distance of 2 m, the pressure this time being near 50 kg/sq. cm.

That the elastic modulus of the rock was sharply increased by this treatment is evident from Table 5.

From this Table it will be seen that at the far end of the gallery, near the fault, where the

Table 5

Elastic moduli							
Emplacement	Injection	Stress at right angles to dip			Stress in direction of dip		
		E' Initial elastic modulus	E Elastic modulus	Irreversible residue % of total per cycle	E' Initial deformation	E Elastic deformation	W Irreversible residue % of total per cycle
Bottom of gallery	Before	9,600	48,000	72-42-26	19,600	34,500	40-25-5
	After	54,000	93,000	42-27-12	51,000	120,000	44-34-14
Halfway along	Before	5,000	60,000	77-56-34	4,000	22,500	75-53-13
	After	65,000	170,000	83-34-11	72,000	190,000	45-13-3

rock before injection showed a very poor elastic modulus, the treatment brought a particularly great improvement (Fig. 14).

On the basis of these first tests the engineer in charge of the project requested the Société de Recherches Géophysiques to make a series of micro-seismic measurements, for correlation with the results of the static measurements. By this means it was possible to make a wider reconnaissance of the site and thus describe the borders between the cracked surface area and the zone of compact quartzites. By correlation with the static measurements it was then possible to estimate the elastic moduli for the various regions.

Moreover, this second study allowed an effective mapping of the areas that required treatment by grouting.

CONCLUSIONS

The actual possibilities of rock mechanics as seen through the eyes of a civil engineer may be summed up as follows.

The static and dynamic methods for testing rock in situ, which were developed about 10 years ago, are still applied in the field, in spite of their uncertainties. Already today micro-seismic measurements of the propagation velocities of sound waves can be correlated with direct static measurements of the elastic modulus to provide an approximate picture of the distribution of mechanical properties in a terrain. But one can with confidence predict a time when it will be routine procedure in every reconnaissance borehole to make a systematic check of the main mechanical characteristics of the rock, on the spot, just as the permeability and the percentage of recuperation are already determined now, these tests sometimes following a direct scrutiny of the drill hole by television.

But perhaps the most rapid progress is taking place today in the laboratory, particularly in two directions, the first being the very important development of methods for measuring the alterability of a rock, the second the study of how the mechanical properties of a rock are changed by applied stresses.

Fig. 14. Consolidation of rock by means of cement grouting. Moduli of deformation before and after grouting at Avène

These directions of advance, along which movement has already begun, will in the course of the next 10 years certainly bring us new knowledge and effective new techniques in this field.

And now I would ask the British Society for Soil Mechanics once again to accept my thanks for the great honour they have bestowed on me in inviting me to discuss these matters. I also tender my most sincere thanks to all those who by their work through the years have made this discussion possible, particularly to M. Habib who has made such valuable contributions as head of the laboratory under Professor Mandel, to M. Bernède who was in charge of all the tests reported here, to M. Bollo and to the Société de Recherches Géophysiques who placed at my disposal some of their reports, and finally to M. Bauzil, Directeur Général de la Compagnie d'Aménagement du Bas-Rhône et du Languedoc, who granted me authority to quote them.

REFERENCES

- BERNARD, P., 1953. "Mesure des modules élastiques et application au calcul des galeries en charge" ("Measurement of moduli of elasticity and their application to the design of pressure tunnels"). *Proc. 3rd Int. Conf. Soil Mech., Zurich*, 2:145-156.
- MAYER, A., 1953. "Les propriétés mécaniques des roches" ("The mechanical properties of rocks"). *Géotechnique*, 3:8:329-338.
- RENAUD, J., 1962. "Le tunnel routier sous le Mont-Blanc" ("The tunnel under Mont Blanc"). *Travaux*, 338:835-848.
- SOCIÉTÉ HYDROTECHNIQUE DE FRANCE, MEETING, 1962. "Symposium on Bulles et Gouttes" ("Symposium on drops and bubbles"). General Report on question 6, by A. Mayer, p. 415, and Special Report by Professor G. Eyraud (Lyon), p. 427.

Mr Harding said that before calling on Dr Charles Jaeger to propose a vote of thanks to M. Mayer, he wished to make it clear that with so eminent a French engineer addressing the meeting the President of the Institution of Civil Engineers would certainly have taken the Chair had he been able to do so, but he had had to go to Africa for a long week-end. That was why an inferior substitute was in the Chair!

Mr Harding went on to say that Dr Jaeger was himself a distinguished lecturer to the Institution, and while not wishing to forestall anything that Dr Jaeger might have to say, wished to mention something that the Doctor might feel diffident about saying, and that was how appreciative they all were of the courtesy that M. Mayer had shown and the trouble he had taken in giving the Lecture in English, and in such very good English. It was a courtesy which people from Great Britain very seldom extended abroad.

Dr Charles Jaeger said that the British National Society of Soil Mechanics had invited Professor Mayer to give the third Rankine Lecture in London. He was the second foreign lecturer to do so. The Rankine Lecture was an important event in the engineering calendar and the most important yearly event for the Society. They thanked M. Mayer for having accepted the invitation to come to London to discuss problems of rock mechanics. To give a Rankine Lecture demanded a considerable effort and was a responsibility even for a man so gifted and so well informed as M. Mayer.

They had, however, even more important reasons to be grateful to him. In Paris in June 1957, M. Mayer had presided over a congress or symposium on rock mechanics. In his Presidential Address he had mentioned that his first connexions with rock mechanics dated back to the early stages of his long and busy career, when he had started work in the mines of the Saar. At that time he had been struck by the inadequacy of the mines organization and the haphazard methods used in digging out the coal and had suggested the creation of a special research branch with a view to the scientific study of mining problems. In this he had not been successful and nothing had been done. As was well known, M. Mayer had subsequently had a brilliant career in the field of soil mechanics, but he could never dismiss those rocks and mines from his thoughts, and that was why in 1957 he had organized the congress in Paris on the subject.

Many of those present would have had similar experiences. Some major research work on rock mechanics had been published as early as 1926. The period from 1921 to 1926 was one in which the first vital measurements had been made; the first tests on rock in situ had been made at that time in the Amsteg Tunnel and the basic ideas on which they worked today had been laid down. After that, however, little of great importance had been carried out for many years in the field of civil engineering as distinct from mining engineering.

Many members of their organization had been worried about the prolonged silence which had prevailed on technical problems concerning rock. A university friend had advised them not to touch rock mechanics since no mathematics would ever be able to cope with the strange lack of isotropy and the stranger behaviour of rock: "Don't touch the subject" had been his advice. M. Mayer was one of the few who had never given up working on the subject. His 1953 and 1957-58 publications were well known in Great Britain.

In his Rankine Lecture M. Mayer had gone to the bottom of some of the most important problems of rock mechanics. His remarks about what French specialists call "rock alterability" are of major importance.

It was worth mentioning that Dr Laginha had made parallel investigations on the changes occurring in rocks which he called "alteration of rock" and had proved that there were some relations between this "alteration of rock" and the compressive strength of the rock, the tensile strength of the rock, and the elastic modulus of the rock. These changes of the rock texture, which could be observed chemically or under the microscope and measured, might be one of the main physical properties of rock.

A phase of research was being entered when some vital links could be detected between petrographic and micro-structure of the rock and other physical properties of rock samples. These could then be connected with other important properties of rock in situ.

As M. Mayer had suggested in his Lecture, there was no great benefit to be reaped from expensive tests in situ so long as no endeavour was made to understand the facts about the micro-structure of the rock. There was still a long way to go, but they could look forward to the day when a series of standard tests in the laboratory and parallel tests in situ would make it possible to classify rock for civil engineering work.

In his remarks in favour of rock grouting, M. Mayer had given indirectly a most interesting comment on some of the ideas expressed 2 years ago by Professor Casagrande in the first Rankine Lecture. M. Mayer, somewhat in contradiction to Professor Casagrande's opinion, had emphasized the vital importance of rock grouting for consolidation of the rock.

Dr Jaeger went on to say that he had in his mind's eye a list of authors from many countries, such as the United States of America, Austria, Germany, Yugoslavia, Portugal, and Great Britain, also a list of positive contributions which he would like to compare with the most important contributions made by their French colleagues. This third Rankine Lecture would be remembered as the occasion when such a synthesis had been made possible thanks to the precise information contributed and the clear analysis of facts given by M. Mayer. That was the main reason for their deep indebtedness to him.

Dr Jaeger concluded by saying that it therefore gave him great pleasure to propose a very warm vote of thanks to M. Mayer for his most interesting and important Lecture.

The vote of thanks was carried with acclamation.