DIVISION 5

Earth and Rock Pressures

Poussée des terres et des roches

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IN CONTRAST with the general upward trend in the number of books and articles the Society's members have to read, or rather would like to read, the number of contributions submitted to this division has decreased compared to the Paris Conference. Considering that this division treats problems of rock mechanics as well as those of earth pressure, this decrease seems strange. Does it mean that the questions of earth pressure are becoming too classical to be attractive? The General Reporter believes that, on the contrary, some questions of a general character still have not been answered. The lively discussions at the Paris Conference showed this very impressively. Another strange feature is the narrow range of subjects of the contributions and the fact that they do not follow the general questions discussed in Paris. Nevertheless, these questions emerge from between the lines and are obviously engaging the attention of the authors.

In order to renew the spirit of fruitful struggle and to provoke discussion the General Reporter has taken the liberty of criticizing the problems rather severely.

I. FUNDAMENTAL PROBLEMS OF EARTH PRESSURE ON WALLS

The Applicability of the Solutions of the Theory of Plasticity

When dealing with stability problems (active and passive earth pressure in this division) we strive to bring the methods of solution into proximity with the solutions of the classical theory of plasticity. A question arises regarding the way in which stability in soil and rock mechanics differs from the principles of the classical theory of plasticity. Two factors are of importance here.

The first of these is the volume changes of the soil or rock during shear deformation. Although the problems of dilatancy and contraction are considered fundamental to many solutions in soil mechanics, they are not considered in the province of this division. Dr. Bjerrum's (1961) far-seeing remarks during the panel discussion at the Paris Conference did not receive much attention. Nevertheless, dilatancy is of great importance in the domain of the stability of slopes because it predetermines whether a thin slip surface or a large plastic zone will develop, and thus it also influences the mechanism of movement in the field of earth pressure. The function of dilatancy may be suppressed but not eliminated by the volume changes of earth during the inclination of the wall. The strange fact that the movement necessary to reduce the earth pressure at rest to the value of active pressure is greater in dense sand than in loose sand (this is reciprocal to the stress-strain relations in the shear tests) can probably be explained by dilatancy.

In this respect the article by Rowe (1963) should be mentioned. In spite of some critical remarks when it was discussed and in spite of the questionable assumption of the minimum energy criterion, it states in an excellent way the general need of the modern theory of plasticity to study the problems from the point of view of the minimum potential energy (maximum of entropy) conditions.

Another example of how dilatancy affects the results is shown by the measurements of *Arthur and Roscoe (5/1).* Shear strain of sand grains was studied on a model and it is emphasized how small the shearing strain necessary to attain the value of passive pressure was in comparison to that in shear tests.

A second difference in the behaviour of earth bodies when compared to the mechanism of failure in the classical theory of plasticity is that the deformation of the earth body need not be reached only by shear. The flexibility of earth bodies was studied by Leonards and Narain (1963); their findings show how much the earth body is capable of bending if tensile cracks are allowed. When analysing the Vaiont slide on a model, the General Reporter (1964) was surprised to find that above the curved portion of the slip surface the body deformed by bending instead of by developing Prandtl's wedge. A zone of extension formed in the lower section of the body and a zone of compression with secondary slip lines in the upper section. Moreover, a cavity appeared along the portion of the more pronounced curvature of the slip line, a phenomenon known to engineering geologists. It may be said that the mechanism approaches the folding of geologic bodies. The resistance of such a zone of transition is smaller than that of Prandtl's zone.

Hence the General Reporter would like to suggest the following questions for discussion. What are the differences between the shear failure of soil and/or rock bodies when compared to the common theory of plasticity? Are they worthy of consideration? What methods of the modern theory of plasticity are helpful when dealing with the aforesaid features?

Limit States Design

The second problem of a general character seems to be the eternal question of limit states design. In summarizing the results of the discussion at the Paris Conference it can probably be stated that there is general agreement that if a wall is rigid the imaginary limit state of stability* is to be supposed to develop and a solution is to be based on the statical and kinematical conditions of such a state. But the General Reporter suggests that time has come to establish a consensus as to the sequence of the steps in the calculation related to this limit state. Some people may object that such a problem is general in character and does not belong only to the scope of Division 5. But, as Kézdi (1963) remarked, the questions of earth pressure occupy a key position.

Referring to the results of the Paris discussion, especially to the deductions of Brinch Hansen, but also to the conception of standards in those countries where the principle of limit state design was taken as a basis, two schemes of the calculation are possible.

The first scheme has two alternatives. *(a)* To multiply the active loads by the factor of overloading, by the factor of ignorance, and by the factor of safety (in the strict sense); to divide the passive loads by the factor of possible diminution; to calculate the guaranteed values of the means of the strength parameters (taking into consideration also the possible long-term reduction of strength); to carry through the proper statical calculation. *(b)* To multiply the active loads by the factor of possible overloading; to divide the passive loads by the factor of possible diminution; to calculate the guaranteed values of the means of strength parameters (taking into consideration also the possible long-term reduction of strength); to divide the strength parameters (c and $\tan\phi$) by the factor of ignorance and by the factor of safety (in the strict sense); to carry through the proper statical calculation. Both ways differ only slightly in approach. Some engineers prefer the first one because it gives the calculated line of rupture nearer to that which will actually develop if the structure fails. In both cases the results of calculation provide the required degree of safety. The advantage of the scheme is that it can be universal for all divisions. Its disadvantage is that it contradicts established methods in "passive" problems both for foundation engineering and for the computation of passive earth pressure.

The second scheme also has two alternatives. *(a)* In "active" problems (active earth pressure, stability of slopes) the arrangement of the first scheme. *(b)* In "passive" problems (bearing capacity, passive earth pressure) another arrangement: to multiply the active loads by the factor of overloading; to divide the passive loads by the factor of possible diminution; to calculate the guaranteed values of the means of strength parameters (taking into consideration also the possible long-time reduction of strength); to carry through the proper statical calculation; to divide the results by the factor of ignorance and by the factor of safety (in the strict sense). The need for two methods in the second scheme may be criticized. But it has the advantage that at least in passive cases the factor of safety and the factor of ignorance influence the result proportionally. The economy of introducing a more elaborate and trustworthy solution can be directly examined. The second scheme also respects the methods already introduced in "passive" cases.

*The General Reporter follows the nomenclature introduced in the U.S.S.R.: The critical states of the behaviour of the structure are called limit states. The limit state of stability, also called the first limit state, coincides with the state of ultimate load (collapse load) used by English-speaking engineers. Two other limit states are in general use: the limit state of deformation, at which the loss of operation of the structure occurs, and the limit state of fissures, which coincides with the state at ultimate load (used by English-speaking engineers).

If the mechanism of rupture is known, the design according to the limit state of stability is clear in principle. But, since the mechanism of failure in the theory of plasticity is predetermined by the way the structure arrived at its present state, the limit state of stability often requires the whole history of deformation to be analysed. This fact was emphasized by Rowe (1961) in the discussion at the Paris Conference ("I have not got two boxes, an elastic and a plastic box, and I do not have to put a problem in one or the other").

One may object that it is far-fetched to require such a uniformity in the calculation, but everyone who has gone through a Babel of discussions in actual cases of importance for lives and economy, will probably confirm the advantage of such uniformity.

To return to the results of the discussion at Paris concerning the limit states design it may be briefly stated that in the domain of flexible sheet pile walls the yielding of supports (anchor yield and yield in the embedment) was found to be of importance for the distribution of active pressure. An excellent review and an analysis of the different measurements and methods have since been given by Tschebotarioff (1962). The force (or moment)–deformation relations for both types of supports are discussed in the contributions to this Conference as well as in many other articles published since the Paris meeting.

Verdeyen et Nuyens (5/13) report the results of modelling the mechanism of rupture of the soil around the anchor plates and of measuring the resistance. It is of interest to follow how the mechanism of rupture varies with the length of the anchors. If the anchors are short the rupture zone in the soil in front of the anchor plates becomes connected with the zone of rupture behind the wall. The depth at which the anchor plate is embedded is of greater importance than the increase in the plate width.

Hueckel, et al. (5/5) studied the distribution of passive earth pressure on the surface of anchor plates. The results are an excellent supplement to the measurements of the resistance of the anchor plates presented to the Conferences in London (Hueckel, 1957), and Aachen (Hueckel, 1961).

The analytic study of the resistance of plane elements in general, as a basis for the calculation of the resistance of the anchor plates, is the object of an interesting contribution by *Biarez, et al. (5/2)*. As in the papers previously mentioned the importance of the depth of embedment of the plate is shown, the relation of depth to plate width being taken as a criterion for the varying mechanism of rupture.

Stability of Rigid Walls in Clay

This is the third question of fundamental importance. As early as the 1930's Terzaghi pointed out that a steady, although extraordinarily slow, inclination of the wall was to be expected. Since that time more has been learned about the long-term strength of clay and numerous contributions on this subject were submitted to previous conferences. From the point of view of practical design, a 30 per cent reduction, at most, from the standard test strength gives a safe value, although even at this value a logarithmic decrease in deformation with time was observed, that is, movement did not stop. Recently the investigations of earth pressure in clay by Šuklje and Vidmar (1961) and Vidmar (1963) have shown that even higher reductions than 35 per cent should be considered.

It may be noted that a need for relaxation shear tests appears to exist in order to analyse the results. It is of interest that such tests (Mencl, 1964) give only about 30 per cent (and sometimes less) of the standard shear strength, whereas long-term shear-strength tests give the aforementioned 70 per cent. Probably a very small movement of no importance to the civil engineer is really necessary to keep strength at some degree of mobilization.

It is obvious that a reliable method of computation of active pressure in cohesive soils would be of considerable aid to the requirements of practice. It is even more necessary because the use of the "strength" theory for long-term structures is questionable.

Observation of Existing Structures

Because of the complexity of the problems connected with flexible walls the significance of measurements on existing structures was emphasized at the Paris Conference. New and interesting results have been presented by Endo (1963); the amount and the distribution of pressure on sheet pile walls during and after the excavation of a 7-to-9-m deep cut are shown. With regard to the comparison of the results of measurements with those of existing rules, Endo's measurements approach the conclusions of Tschebotarioff (1962).

II. PROBLEMS OF SPECIAL INTEREST

Four problems are covered in the contributions.

Active pressure of sand subject to vibration. This problem, the subject of wide interest at conferences on earthquake engineering, is represented here by the contribution of *Ichihara (5/6)*. Measurements on an elaborate large-scale apparatus show that the displacement of the wall produces a decrease in the coefficient of earth pressure during vibrations with low freqencies, but that the point of application of the resultant is situated at nearly half the height of the wall and that the angle of friction on the wall is only 25 per cent of that found in static conditions.

The failure of a high crib wall. Of great interest for workers inexperienced in this type of wall (as is the General Reporter) is *Tschebotarioff's (5/12)* finding of how susceptible the structures are to damage by settlement. In the case given the wall was curved outwards (concave when observed from the backfill) which contributed to its failure. The decreased strength of the backfill because of the small intermediate principal stress probably also contributed. Rules of practical significance are suggested.

Soil pressure on buried tubes. Three contributions show the growing interest in this field. *Habib et Luong (5/3)* analyse the danger of collapse by buckling of thin flexible tubes and evaluate the results of both theoretical analysis and intuitive experiments by introducing a formula for critical pressure. The general picture of the interaction between tube and soil is described by *Luscher and Höeg (5/8)*, showing the three factors of mobilized resistance: the growth of lateral passive earth pressure during a small deformation, the beneficial intervention of the soil into the mechanism of buckling, and the arching of the soil. The influence of a varying system of embedment of the tube is studied by *Malishev (5/9)*. The author starts from his formulae published in the *Proceedings* of the Brussels Conference (Malishev, 1958) and presents a method of calculation for practical use.

Effect of an underground explosion is analysed by *Vesić (5/15)*, using an approach more or less from the point of view of classical soil mechanics. In this way a clear and simple method of calculating the radius of the cavity caused by the explosion as well as other factors is given. Although the author remarks that the assumed simplifications of the mechanism and the statical rather than dynamical approach may produce deviations from the actual state, the results are in good agreement with observations from nuclear explosion tests.

III. TECHNIQUES OF MEASUREMENT

Many of the contributions describe the results of tests in which new and ingenious methods of measurement were used or examined. Arthur, *et al.* (1964) introduced a method of calculation of stresses in a soil mass based on measured strains, using an X-ray technique for the determination of the strains. Recently, in another contribution, *Arthur and Roscoe (5/1)* have examined the necessity of using the X-ray technique and have concluded that the displacement at the plane of contact of the soil with a glass wall does not differ from that inside the body, if certain precautions are observed.

Although an excellent analysis and a list of earth pressure cells were published by Hamilton (1960), still other new modifications are forthcoming (Arthur and Roscoe, 1961). The use of effective earth pressure cells made possible the findings, by *Verdeyen et Roisin (5/14)*, of values for the pressure transmitted to a flexible bulkhead by a linear force at the surface of the backfill. The results deserve attention because of a complete disagreement with the results of usual methods of calculation. The results were partly presented at the Paris Conference (Verdeyen and Roisin, 1961).

IV. ROCK PRESSURE ON WALLS AND TUNNELS

The questions of rock mechanics in so far as rock pressures are concerned, especially in the field of application of measurements and in the invention of new techniques of measurements, have been the subject of broad and lively interest. The seven international meetings in America sponsored jointly by the Colorado School of Mines, the University of Minnesota, the Pennsylvania State University, the University of Missouri, and the Society of Mining Engineers (AIME), as well as the International Conference on State of Stress in the Earth's Crust (Santa Monica, 1963) and the International Conference on Strata Control and Rock Mechanics (New York, 1964) have contributed greatly to this development. The activities of the National Committee of England as well as of the Internationales Büro für Gebirgsmechanik of the German Academy of Science, Berlin, and the Internationale Gesellschaft für Felsmechanik in Salzburg, should be mentioned in Europe, as well as the Eighth International Congress on Large Dams (Edinburgh, 1964). If several books and many other publications are considered as well, it is difficult to present a review of the activities in this field since the Paris Conference without approaching the task from a narrow point of view. In spite of controversial opinions the General Reporter intends to show that in many fields the way of thinking in the problems of soil mechanics is very helpful in the study of problems of rock pressure. In this way the tradition of the Paris Conference can be followed (Mencl, 1961).

Tunnelling in Soft Ground

There is no doubt that this problem is to be studied by using directly the methods of investigation and analysis of soil mechanics. Three contributions belong to this category. *Ward and Thomas (5/16)* discuss the results of measurements of earth pressure in tunnel linings in the London Clay. As in the preceding reports of these authors values of pressure amounting to 75 to 100 per cent of the hydrostatic overburden pressure were recorded. The great number of vibrating-wire strain gauges used to perfect the investigation deserves attention. The explanation of the elliptical deformation of the rings seems a little too elaborate. This phenomenon can be probably explained by the need to mobilize the passive earth pressure in the horizontal direction before the long-term hydrostatic conditions are re-established.

In the contribution of *Sutherland (5/11)* a statical analysis of a novel procedure is shown, namely driving shafts (with a diameter of 7 to 8 ft) by jacking through the sand mass (8 to 23 ft in thickness) upwards from the tunnels driven underneath the sea bed. The force necessary for the jacking operations is carefully investigated using the similarity with other problems in soil mechanics as well as the results of measurements on models. This force was verified by the findings at the site. As in many other cases of pushing blind pipes through dense sandy soils the force necessary was greater than that determined by using existing theoretical formulae. The comparison of three ways of investigation is of great interest, especially the fact that the results of model tests proved correct in a quantitative manner.

How much the ingenious methods of theoretical soil mechanics are helpful in predicting the behaviour of a tunnel in soft ground is shown in an excellent way by the contribution of *Bent Hansen and Nielsen (5/4)*. This contribution is of great interest both for method of investigation and for the development of new solutions in the theory of consolidation.

Shear Strength of Rocks

It is in the domain of shear strength that the philosophy of soil mechanics should influence ways of thinking in rock mechanics. The considerable function of dilatancy in the amount of shear strength if the movement occurs across strata is nearly unknown to investigators in rock mechanics (Mencl and Paseka, 1963), although in underground conditions it may contribute to the arching of rocks, to the growth of concentrated stress resulting in rockbursts, and such. The beneficial effect of bolting is very much due to dilatancy. The influence of the strength of blocks on the occurence of dilatancy should be considered without losing sight of Bird's example (1961) of small values of shear strength of gravel with weak grains. The change in behaviour of rocks at great depths may be explained as a loss of dilatancy by analogy with the behaviour of sand at very high pressures (Vesić and Barksdale, 1963).

Theory of Models

Because of the complexity of problems of rock mechanics the use of models is still increasing. The contribution of *Oberti and Fumagalli (5/10)* from the laboratory in Bergamo, Italy, illustrates this method of investigation in an excellent manner. Because of laws of scale the rock is represented in the models by a soil. But to find a soil really equivalent in mechanical behaviour to a rock a knowledge of the properties of both materials is necessary. Let the General Reporter be allowed to recall the successes in modelling of his colleagues at the Mining Institute of the Czechoslovak Academy of Sciences, attained by introducing the dilatancy as a principal factor.

Measurements In Situ

As mentioned previously, growing activity in the field of measurement of stress and deformation in rock and of its support at sites is helpful in the understanding of the mechanical behaviour of rock. The investigations by Potts (1964) and his colleagues in the field of mining as well as the measurements by Rabcewicz (1964) in tunnelling are successful examples.

Krsmanović and Buturović (5/7) present results of measurements of pressures of weak rock on a tunnel lining over a period of 650 days, using 10 to 12 flat jacks in each section. The results, as well as the deformation and mode of failure of the lining, indicated in principle that the pressure existed in the abutments at first, and that only in the course of time did the pressure in the crown increase. The General Reporter recalls that the same phenomenon was observed several times (Mencl, 1964 and Mencl, *et al.*, 1964), whenever "classical" methods with crown-bar timbering and unreinforced concrete linings were used in weak rock. A timely grouting of the gap behind the lining in the crown is a considerable aid.

V. PROPOSALS FOR DISCUSSION

The following subjects are suggested for discussion.

1. What are the differences between the shear failure of earth bodies and the bodies of the common theory of plasticity? Are they worth considering? What methods of the modern theory of plasticity are helpful when taking into consideration the phenomena of dilatancy and flexibility in soil and rock bodies?
2. Is it of practical use to seek uniformity of calculations if the limit state of stability is concerned, and if so, what scheme is to be recommended?
3. Relaxation phenomena in earth pressure problems and methods of calculation of earth pressure in cohesive soils.
4. Further development of the methods of design of flexible walls.
5. The similarity in the mechanical behaviour of soils and rocks and its influence on the methods of analysis of rock pressure.

SUJETS DE DISCUSSION

Il est recommandé que les sujets suivants soient considérés pour fin de discussion.

1. Quelles sont les différences entre la rupture de cisaillement des matières du sol et celles de la théorie courante de plasticité? Ces différences méritent-elles d'être considérées? Quelles méthodes de la théorie moderne de plasticité sont utiles lorsqu'on tient compte des phénomènes de dilatabilité et de flexibilité des matières du sol et du roc.
2. Est-il nécessaire de viser à l'uniformité des calculs lorsqu'il est question de l'état limitatif de flexibilité, et si tel est le cas, quelle méthode devrait être recommandée?
3. Les phénomènes de relâchement des problèmes de la poussée de couverture et les méthodes de calcul de la poussée de couverture des sols cohérents.
4. L'élargissement des méthodes de conception des parois flexibles.
5. La similitude du comportement mécanique des sols et des roches et son influence sur les méthodes d'analyse de la poussée des roches.

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