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Original papers and discussions of current papers should be submitted to the Manager of Technical Publications, ASCE. Authors must indicate the technical division, technical committee, sub-committee, and task committee (if any) to which the paper should be referred. The final date on which a discussion should reach the Society is given as a footnote with each paper. Those who are planning to submit material will expedite the review and publication procedures by complying with the following basic requirements:

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2. A summary of approximately 50 words must accompany the paper, and a set of conclusions must end it.

3. The manuscript (an original ribbon copy and two duplicate copies) should be double-spaced on one side of 8½-inch by 11-inch paper. Three copies of all illustrations, tables, etc., must be included.

4. The author's full name, Society membership grade, and footnot reference stating present employment must appear on the first page of the paper. (Authors need not be Society members).

5. Mathematics are recomposed from the copy that is submitted. Because of this, it is necessary that letters be drawn carefully, and that special symbols be properly identified. The letter symbols used should be defined where they first appear, in the illustrations or in the text, and arranged alphabetically in an Appendix.

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7. Illustrations must be drawn in black ink on one side of 8½-inch by 11-inch paper. Because illustrations will be reproduced with a width of between 3-inches and 4½-inches, the lettering must be large enough to be legible at this width. Photographs should be submitted as glossy prints. Explanations and descriptions must be made within the text for each illustration.

8. The desirable average length of a paper is about 10,000 word-equivalents and the absolute maximum is 15,000 word-equivalents. As an approximation, each full manuscript page of text, table, or illustration is the equivalent of 300 words.

9. Technical papers must be written in the third person.

10. A list of key words and an informative abstract should be provided for information retrieval purposes. (Information on preparation available on request.)

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by Leslie G. Bromwell .......................................... 114


the complete data in Figs. 15, 16, and 17 for more specific information on comparative performance.

2. Comparative performance of the hammers tested in this investigation under a fairly wide range of driving conditions indicate that there is considerable opportunity to select hammers designed to meet anticipated variations in field conditions. Slower heavy hitting hammers perform better for heavy driving while faster, lighter hitting hammers give greater speed of penetration under light to average driving conditions.

3. ENTHRU, or energy transmitted to the piles varied from approximately 25% to 60% of the manufacturer's rated energy, depending on the operating conditions and control, soil conditions, type of pile, cushion blocks used to protect the pile, and hammer and other field conditions.

4. The large differential between rated energy and ENTHRU and the variability in this relation is largely caused by energy losses at impact rather than a deficiency in the energy producing mechanism of the hammers. There was little evidence in this investigation on which to question the manufacturer's rating in this respect and much evidence to indicate the need for improvement in the methods and devices for transmitting energy to the piles. Detailed data on transmitted energy are provided in Table I, to which the reader is referred for more specific information.

ACKNOWLEDGMENTS

In addition to the cooperating agencies both public and private previously listed, there were a number of individuals in these organizations who carried responsibility for equipment design, conducting the tests, and assembling the data for the final project report. These important individual contributions have been acknowledged in the project report, and their number is such that it is hardly practicable to do so herein.

However, it is most appropriate to name one whose ability as an engineer, whose integrity as a research worker, and whose devotion to duty contributed much to this study, his last project. Reference is made to the late Leo V. Garrity, F. ASCE, Project Manager. His counsel and guidance have been sorely missed by his associates. This paper is being presented by the writer as a representative of these many contributors.

INTERSTITIAL PRESSURES ON ROCK FOUNDATIONS OF DAMS

By J. Laginha Serafim, F. ASCE, and Alejandro del Campo

INTRODUCTION

The percolation of reservoir water through the foundations of concrete dams, even when the rock mass is of good quality and of a minimum permeability, is always a decisive factor in the safety and performance of these structures. This water produces pore pressures whose estimation is a basic problem in the design of dams and in the study of the stability of the adjacent slopes. It can produce piping, sometimes at a slow rate but at other times rapidly, of the filler materials in the joints or faults of the rock mass, with consequent differential settlements in the foundations. It may produce deformation and opening of the joints, which, in turn, generally decreases the cohesion and angle of internal friction of the mass. Finally, in certain cases, it can accelerate the process of dissolution or alteration of the rock and its constituent minerals. It must be said that, when the filtration of water and its effects are properly studied, certain peculiar aspects of the behavior of dams can be explained and when the necessary measures and treatments are conducted in due time, their safety in relation to interstitial water pressures can be completely assured.

Measurements of the water pressures inside rock masses have been presented in many publications although the laws of flow have not yet been investigated in great detail. Comparisons always have been based on the assumption that water flows in a steady state through a homogeneous porous media. This corresponds to the assumption that the external conditions that could affect the flow, such as the reservoir level, the load on the foundations, etc., are kept constant and also that the internal conditions in the rock that might affect flow do not change in time, from point to point, or with direction.

Note.—Discussion open until February 1, 1966. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Soil Mechanics and Foundations Division, Proceedings of the American Society of Civil Engineers, Vol. 91, No. SM5, September, 1965.


2. Prof., Escuela Tecnica Superior de Ingenieros de Caminos, Canales y Puertos, Madrid, Spain.
In view of the fact that rock masses are a complex media, with properties not too well known, the use of more elaborate assumptions probably was not considered justified. With the need for construction of even larger dams, sometimes in sites where the rock is not hard or sound and has numerous joints and faults, a more thorough study of the interaction of water and foundations is necessary.

A general review of the hydraulic problems of the percolation of water in rock foundations is presented herein, considering in particular the jointed character of the rock masses, and an attempt is made to clarify these problems.

Notation.—The symbols adopted for use in this paper are defined where they first appear and are arranged alphabetically in the Appendix.

PERMEABILITY OF ROCKS AND ROCK MASSES

Laboratory determinations of the permeability of rock materials have shown, in general, low coefficients of permeability, particularly for unaltered rocks. Table 1 indicates values found in the literature.

<table>
<thead>
<tr>
<th>Type of Rock</th>
<th>Coefficient of permeability K, in centimeters per second</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Granite</td>
<td>0.05 to 0.2 \times 10^{-9}</td>
<td></td>
</tr>
<tr>
<td>Slate</td>
<td>0.07 to 0.16 \times 10^{-9}</td>
<td></td>
</tr>
<tr>
<td>Breccia</td>
<td>0.46 \times 10^{-9}</td>
<td></td>
</tr>
<tr>
<td>Calcite</td>
<td>0.7 to 9.3 \times 10^{-9}</td>
<td>3</td>
</tr>
<tr>
<td>Limestone</td>
<td>0.7 to 1200 \times 10^{-9}</td>
<td>4</td>
</tr>
<tr>
<td>Dolomite</td>
<td>4.6 to 12 \times 10^{-9}</td>
<td></td>
</tr>
<tr>
<td>Sandstone</td>
<td>160 to 12,000 \times 10^{-9}</td>
<td></td>
</tr>
<tr>
<td>Hard Mudstone</td>
<td>600 to 2,000 \times 10^{-9}</td>
<td></td>
</tr>
<tr>
<td>Black Schists (fissured)</td>
<td>100,000 to 30,000 \times 10^{-9}</td>
<td>4</td>
</tr>
<tr>
<td>Fiss grained Sandstone</td>
<td>200 \times 10^{-9}</td>
<td></td>
</tr>
<tr>
<td>Oolitic rock</td>
<td>1,300 \times 10^{-9}</td>
<td></td>
</tr>
</tbody>
</table>

Quite often it is taken as a rule that grouting is not required when filtration along a borehole 5 m in length, under a pressure of 10 kg per sq cm applied for 10 min, is less than 1 kg per sq cm per min per meter of length. The value of the coefficient of permeability corresponding to this filtration was estimated to be approximately 10,000 \times 10^{-9} cm per sec.

In general, rock masses present many zones of a considerably higher permeability than the one indicated by this figure. K. Terzaghi indicated values of the coefficient of permeability determined by in situ tests of 10,000,000 \times 10^{-9} and 100,000 \times 10^{-9} cm per sec for sandstone and muckstone formations, respectively. Such values, and others that have been determined, show that the permeability of the rock mass is normally considerably greater than that of the individual specimens. This indicates that the permeability of the mass must be attributed primarily to the filtration through the joints and other discontinuities. In fact, a formation of completely impervious materials having only horizontal joints 0.1 mm wide at regular intervals of 1 m would filtrate, horizontally, as much water as a homogeneous porous body of equal dimensions having a coefficient of permeability of 80,000 \times 10^{-9} cm per sec, i.e., 0.8 \times 10^{-4} cm per sec.

It is evident, then, that all of the factors, such as the state of stress, temperature variations, etc., that control the opening of the joints would have a pronounced effect on the capacity of a rock formation to conduct water.

JOINTS IN A ROCK MASS

In addition to faults, folds, and contact planes that greatly affect behavior, rock masses are divided by systems of more or less parallel joints. Quite often three or four separate systems are observed in a rock mass, which tend to divide it into an array of blocks. The openings and spacing of the joints vary from system to system and even in a particular system they are not constant. These joints probably were caused by internal forces or temperature variations of the crust and by erosion of the rock’s surface.

For three systems of joints the respective frequencies can be denoted by \( F_1, F_2, F_3 \), the average spacings \( d = 1/F \) by \( d_1, d_2, d_3 \), and the average joint openings by \( e_1, e_2, e_3 \).

The joints are sometimes interrupted, and in most cases they decrease in number or increase in spacing with depth. In the case of metamorphic and sedimentary formations, the planes of stratification or schistosity constitute one of the systems of joints. The mechanical and hydraulic behavior of the rock mass depends, to a great extent, on whether or not the joints are filled and also on the roughness of the joint surface. Representation of the joints and faults on a hemispherical diagram is extremely useful in the interpretation of phenomena involved in rock masses. Such diagrams are currently obtained during the geological prospecting of the sites. In addition, the statistical distribution of joint openings observed in various galleries,

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6 Legeon, M., "Barrages et Geologie," Rouge, Lausanne, Switzerland, 1933.
trenches, and large diameter bore holes at the construction site are valuable data.

DETERMINATION OF THE OPENING OF JOINTS

A measurement of the width of opening of the various joints and of the frequency of their occurrence would give an indication of the value of the permeability of the rock mass. Although direct measurements could be devised, it seems that a more reasonable approach would be to estimate such information through indirect means as most probably the gaining of access to the joints will disturb the rock mass and, therefore, their openings.

\[ q = \frac{\pi e^2}{12 \mu} \left( \frac{p_1 - p_2}{\ln \frac{d}{r_0}} \right) \]

in which \( r_0 \) = the radius of the bore holes. From Eq. 1 the value of \( e \) can be calculated, all other variables being known. When conducting such a test the

Field permeability tests in individual, pairs, or even groups of borings, in which the permeability could be studied by joint, are feasible and have been recommended in certain cases to evaluate the size of important joints. Referring to Fig. 1, the quantity of water, \( q \), that would filter from A to B along a joint \( aa' \), of uniform opening, \( e \), is a function of that opening and in the case that no filler materials exist, then \( e \)

\[ q = \frac{p_1 - p_2}{12 \mu} \ln \frac{d}{r_0} \]

Referring to Fig. 1, the quantity of water that flows out of the tube in B must be equal to that supplied to the zone being tested in A.

---

Another indirect estimation of "effective" width of joints, based on stress-deformation diagrams from in-situ deformability tests, should be possible, particularly if the joint openings are relatively large. Previous work has shown\textsuperscript{10,11} that the curves from such tests in jointed material indicate a marked settlement of the rock mass during the first loading cycle which is caused by the closing of the joints. As can be seen in Fig. 2(a), during the first application of load to a given stress, the rock settled, and after the second cycle the stress-displacement diagrams become repetitive. The permanent settlements are also a function of time. It has been observed, however, that the final values are approximately obtained within the first few minutes of the cycle. Considering that the stresses become negligible at a distance of twice the dimension, $D$, of the loaded area, then $l_1$ in Fig. 2 is of the same order of magnitude as the total opening of the joints normal to the load and within a distance of 2 $D$. Although this value is a rough estimate of the openings, there is little doubt as to its validity because the settlements observed in such tests in rock masses are not observed\textsuperscript{12} if the mass has previously been grouted to a distance of 3 to 4 $D$.

Study of the results of many in-situ tests and their companion laboratory tests have led to the deduction\textsuperscript{10,11} that the increase in rigidity of the rock observed with load (after the third loading cycle, Fig. 2), and the pronounced
curvature of the unloading part of the cycle are the results of the closing on loading and subsequent delayed opening on unloading of the micro-joints present in the rock material. The sum of the displacements resulting from the closing of the micro-joints is, then, approximately equal to the value, $l_2$.

\textbf{FIG. 3.—ASSUMED BEHAVIOR OF A ROCK MASS}


\textbf{CONCRETE DAMS}

\textbf{FIG. 4.—PRESSURES AT THE BASES OF DAMS WITHOUT DRAINAGE}\textsuperscript{13}

In a first approximation, rock masses can be treated as a linear compactable media with a preliminary consolidation, $E_t$. The stress-strain diagram for such a material could theoretically be shown as is shown in Fig. 3. Therefore, in most cases, after the foundation has been consolidated by grouting, it will be the micro-fissures, having a total opening approximately
equal to $l_2$ in a distance, 2 D, which will determine the quantity of water that filtrates through the rock mass.

MEASUREMENT OF PORE PRESSURES IN DAM FOUNDATIONS

Fig. 4 and 5 indicate values of pore pressures measured in the foundations of dams. They were plotted from diagrams in the final report of the ASCE Committee on Uplift. In Fig. 5, the measurements made by TVA on Fontana Dam are also shown. Both figures indicate pore pressures measured at a


FIG. 5.—PRESSURES AT BASES OF DAMS WITH DRAINAGE

Concluded that the values of the pressures in the foundations of the three other dams depicted in Fig. 4 do not correspond to a steady state flow. The same conclusion would be reached by comparing the results shown in Fig. 5 with the theoretical distribution of pressures (Fig. 6) in the base of a dam having

a continuous line of drainage (a gallery) at the rock interface. Recently examined data indicates that the pore pressure measured downstream of a system of foundation drains is always considerably lower than the theoretical calculations using steady state equations and a constant coefficient of permeability. This can be explained either by considering that the grouting of the foundations was effective and reduced the coefficient of permeability upstream of the drainage network, or by stating that the pore pressures downstream of the drains are not exactly determined by the maximum height of the reservoir but by a lower height as a consequence of the variation throughout the year. It can be said that, in some dams, the measurements of the quantity of water filtrating from the drains shows a variation corresponding to the level of the reservoir, but with a definite time lag. This time lag gives an average indication of the over-all delay in response to the foundation of the dams to external conditions that affect the filtration of water. In addition, diagrams of pore pressure measured in many Bureau of Reclamation dams do not indicate an appreciable variation in those pressures downstream of the line of the drains during the year but do show such a fluctuation upstream of that line. In the case of Cabril Dam, such a variation of pressure with the reservoir water level was noticeable and a time lag was observed. Similar observations for Norris Dam are shown in Fig. 8. In addition, interesting measurements showing the dependence of the water pressure in the rock on the water level of the reservoir have been reported for Roche-au-Moine, Knincky, and other dams. It should be noted that


FIG. 8.—EVOLUTION OF PORE PRESSURES AT THE BASE OF NORRIS DAM

In Knincky dam, the interstitial pressures during the filling of the reservoir were lower than during the period in which it was emptied. The data indicates that the percolation in rock foundations of dams occur under a nonsteady state condition. The fact that the time lags are normally not long (apparently from a few days to one month) shows that the opening of the joints is comparatively wide corresponding to considerably larger coefficients of permeability than that of concrete or ordinary rock material.
Studies of time-dependent flow in fissured materials like rock masses are scarce. Recently, Russian authors\textsuperscript{23} studied the problem considering fissured and permeable rock having two liquid pressures, one in the fissures, the other in the pores. The pressure jumps within the system, i.e., the transfer of the liquid between the blocks and the fissures, will account for the nonsteady state once the compressibility of the liquid was considered. In this formulation of the problem, the importance of the presence of air was not considered and the effect of the variation of opening of the fissures was discarded. Such factors will be considered in the present study but the rock blocks will not be assumed porous.

CIRCULATION OF WATER IN A FISSURE

In fissures with small openings, the circulation of water in a steady state of flow is laminar. Considering such a fissure with a constant opening, \( e \), and taking the \( x \)-axis at mid-height and running through the opening, the \( y \)-axis vertical and perpendicular to the \( x \)-axis and normal to the fissure, the total “driving” force, \( \delta p / \delta x \) \( e \), is caused by the difference, \( dp \), in pressures at the ends of the fissure. The tangential stress, \( \tau \), at any point will be such that

\[
\frac{\delta \tau}{\delta y} = \text{const} = \frac{\delta p}{\delta x} \tag{2}
\]

Because the flow is laminar, if \( \mu = \) the viscosity of the water and \( v_x = \) the velocity of the flow, then

\[
\tau = \mu \frac{\delta v_x}{\delta y} \tag{3}
\]

which yields, by substitution of Eq. 3 in Eq. 2,

\[
\frac{\delta p}{\delta x} = \mu \frac{\delta^2 v_x}{\delta y^2} \tag{4}
\]

Considering that for \( y = \pm e/2, v_x = 0 \), it is found that

\[
v_x = \frac{1}{2\mu} \left( y_e^2 - \frac{e^2}{4} \right) \frac{\delta p}{\delta x} \tag{5}
\]

Eq. 5 indicates that the velocity of the flow has a parabolic distribution across the opening of the fissures. The average velocity, \( V \), is found from

\[
V = \frac{1}{12 \mu} \frac{\delta p}{\delta x} \left[ y_e \right] - \frac{1}{2}
\]

Eq. 6 shows that a rock mass with a system of parallel fissures of width, \( e \),


separated by a distance, \( d \), is equivalent, with respect to the filtration of water in the direction of the fissures, to a porous body having a coefficient of permeability equal to

\[
K = \frac{1}{12 \mu} \frac{e^3}{d} \tag{7}
\]

The movement of water through the fissures becomes turbulent only when the Reynolds number, \( R = Ve \gamma / \mu \), reaches approximately 1,200, corresponding to joints with a width of opening approximately 0.2 cm, when subject to a unit hydraulic gradient. In turbulent flow the friction coefficient, \( f \), depends on the roughness of the walls of the fissure and the hydraulic gradient and is proportional to the square of the velocity. Thus,

\[
\frac{\delta p}{\delta x} = f \frac{\gamma v^2}{2e} \tag{8}
\]

in which \( \gamma = \) the specific weight of the water and \( g = \) the acceleration of gravity. Assuming a coefficient of friction of 0.06, and a joint opening of 10 mm, and solving for a unit hydraulic gradient, the velocity will be \( V = 2.55 \) m per sec and the flow rate will be 0.026 cu m per sec for each linear meter of opening. As such joints can be filled with cement or other grouting materials, it is improbable that such flow conditions will exist in the foundation of a dam.

In the case of a joint that decreases in opening from \( e \) to \( e/2 \) in a length \( L \), along which the pressure dropped by \( p = \gamma H_o \), the total flow, \( q \), will remain constant (Fig. 9). Thus,
downstream direction plays in the build-up of high interstitial pressures. Such a decrease in width can be the result of an increase in compressive stresses in a direction normal to the fissures downstream of the surface of infiltration.

It should be noted that in formations of compact and hard rocks with few joints but having major faults, the percolation of the water will occur primarily along such faults. The condition of filtration must be investigated in such cases in view of the preceding theory.

CONDITIONS OF PERCOLATION OF WATER IN A JOINTED ROCK MASS

Consider a parallelepiped of rock mass, dV, as shown in Fig. 10 having faces parallel to three assumed systems of joints in the mass and having a sufficient number of these joints within the volume so as to be considered representative of the mass from the point of view of water filtration. The difference between the quantity of liquid in weight that, in a unit of time, flows in and out of the volume, is

\[ \frac{\delta q}{\delta t} = - \left[ \frac{\partial (\gamma \nu_x)}{\partial x} + \frac{\partial (\sigma \nu_y)}{\partial y} + \frac{\partial (\gamma \nu_z)}{\partial z} \right] dV \]  

If it is assumed that the variations in \( \gamma \) with respect to \( x, y, \) and \( z \) can be neglected, this simplifies to

\[ \frac{\delta q}{\delta t} = - \gamma \left[ \frac{\partial \nu_x}{\partial x} + \frac{\partial \nu_y}{\partial y} + \frac{\partial \nu_z}{\partial z} \right] dV \]  

Considering the components of the velocity expressed in terms of the average width, \( e_x, e_y, \) and \( e_z, \) and the average distances, \( dx, dy, \) and \( dz, \) in the joints, (plane and without any tiling material), then

\[ v_x = - \frac{1}{12 \mu} \frac{\partial p}{\partial x} \left( \frac{e_y^3}{d_y} + \frac{e_z^3}{d_z} \right) \]  

\[ v_y = - \frac{1}{12 \mu} \frac{\partial p}{\partial y} \left( \frac{e_x^3}{d_x} + \frac{e_z^3}{d_z} \right) \]  

\[ v_z = - \frac{1}{12 \mu} \frac{\partial (p + \gamma z)}{\partial z} \left( \frac{e_x^3}{d_x} + \frac{e_y^3}{d_y} \right) \]  

If

\[ K_x = \frac{\gamma_o}{12 \mu} \left( \frac{e_y^3}{d_y} + \frac{e_z^3}{d_z} \right) \]  

\[ K_y = \frac{\gamma_o}{12 \mu} \left( \frac{e_x^3}{d_x} + \frac{e_z^3}{d_z} \right) \]  

\[ K_z = \frac{\gamma_o}{12 \mu} \left( \frac{e_x^3}{d_x} + \frac{e_y^3}{d_y} \right) \]
then Eq. 16 may be written as

\[ \frac{\partial q}{\partial t} = K_x \frac{\partial^2 p}{\partial x^2} + K_y \frac{\partial^2 p}{\partial y^2} + K_z \frac{\partial^2 p}{\partial z^2} + \frac{\partial K_x}{\partial x} \frac{\partial p}{\partial x} + \frac{\partial K_y}{\partial y} \frac{\partial p}{\partial y} + \frac{\partial K_z}{\partial z} \frac{\partial p}{\partial z} + \gamma_o \frac{\partial \varepsilon}{\partial z} \]  

\[ \text{(19a)} \]

The above values of \( K \) can be reduced by a certain extent to be determined to account for the effect of the filling materials of the joints and the roughness of their surface.

If it is considered that the weight of water contained in the volume, \( dV \), is proportional to the percentage of the volume of voids, \( \varepsilon \), less the percentage of the volume of air, \( a \), then

\[ q = \gamma (1 - a) \varepsilon dV \]  

\[ \text{(20)} \]

from which the quantity of water retained within the rock in a unit time is

\[ \frac{\partial q}{\partial t} = (1 - a) \varepsilon dV \frac{\partial \varepsilon}{\partial t} + \gamma \varepsilon dV \frac{\partial (1 - a)}{\partial t} + \gamma (1 - a) \frac{\partial \varepsilon dV}{\partial t} \]  

\[ \text{(21)} \]

The first part of the second term of Eq. 21 refers to the effective volumetric compressibility, \( \beta \), of the liquid. If the liquid has a specific weight, \( \gamma \), at a pressure, \( p \), and a specific weight of \( \gamma_o \) at atmospheric pressure, \( p_o \), then

\[ \gamma = \gamma_o \beta p + \gamma_o \]  

\[ \text{(22)} \]

from which

\[ \frac{\partial \gamma}{\partial t} = \gamma_o \beta \frac{\partial p}{\partial t} \]  

\[ \text{(23)} \]

In addition, it can be seen that the second part of the second term of Eq. 21 refers to the variation of the volume of air (bubbles). At constant temperature, the increase in pressure will produce a decrease in volume (Boyle) of the air and its solution in the water (Henry):

\[ a(p + p_o) = a_o p_o - 0.02 (1 - a)p \]  

\[ \text{(24)} \]

from which

\[ \frac{\partial (1 - a)}{\partial t} = \frac{a_o p_o + 0.02 p (1 - a)}{(p + p_o - 0.02 p)^2} \]  

\[ \text{(25)} \]

Finally, the third term of Eq. 21 corresponds to the decrease of the volume of voids, which is related, in this case, to a decrease in the width of the joints and fissures, which, in turn, can be considered proportional to the sum of the principal stresses, \( \varepsilon \). This corresponds to the consideration that the deviatoric component of the state of stress in the rock does not change the opening of the joints or fissures,

\[ \varepsilon = \frac{\beta \gamma}{\beta} = \varepsilon_x + \varepsilon_y + \varepsilon_z \]  

\[ \text{(26)} \]

namely,

\[ \frac{\partial (\varepsilon dV)}{\partial t} = dV \frac{\partial \varepsilon}{\partial t} \]  

\[ \text{(27)} \]

Considering that \( \gamma_o \) is approximately \( \gamma \), then Eqs. 19, 21, 23, 25, and 27 yield

\[ \frac{\partial q}{\partial t} = K_x \frac{\partial^2 p}{\partial x^2} + K_y \frac{\partial^2 p}{\partial y^2} + K_z \frac{\partial^2 p}{\partial z^2} + \frac{\partial K_x}{\partial x} \frac{\partial p}{\partial x} + \frac{\partial K_y}{\partial y} \frac{\partial p}{\partial y} + \frac{\partial K_z}{\partial z} \frac{\partial p}{\partial z} + \frac{\partial \gamma_o}{\partial z} \frac{\partial \varepsilon}{\partial z} \]  

\[ \text{(22)} \]

An estimation of the importance of the factors that cause the unsteady state of flow, i.e., the accumulation of water inside the joints, indicate that the air entrapped is of primary importance. Eq. 28 constitutes a general differential expression for the flow of water through a rock mass.

The influences of temperature variations were not studied. In certain cases, temperature variations can contribute markedly to the opening and closing of the joints and thereby affect both the coefficients of permeability and the weight of the water contained in the elemental volume, \( dV \). In fact, if it is considered that the water of the lower level of the reservoir is at a temperature that is \( 5^\circ \) lower than that of the rock mass and that the joints are \( 1 \) m apart, they will open a maximum of \( 0.05 \) mm.

An expression for isotropic and homogeneous porous materials similar to Eq. 28 can be found elsewhere.\(^{17}\) It has been shown\(^{17}\) that such an equation includes, as a particular case, the expression of the theory of consolidation of Terzaghi.

Eq. 28 indicates that if rapid variations of the external conditions or of the total stresses in the rock mass occur, pressures can be built up in the interior of the joints of the rock mass that can cause a decrease in over-all strength of the mass. Computations show that this is especially important in the case of thin joints filled with impervious clays. It can also be concluded that the state of stress in the rock mass is important when considering pore pressures. In fact, not only does it affect the permeability by decreasing the width of the joints but it also affects the volume available to filtering water when the
mass is compressed. Numerical estimates of such influences for simple cases can be obtained from the previous equations.

STEADY STATE OF FLOW

In the case where the permeability coefficients are constant in all directions, i.e., constant opening of the joints in all directions, and when the flow does not change from time to time, Eq. 28 vanishes and Eq. 19 becomes

$$K \frac{\partial^2 \phi}{\partial x^2} + K \frac{\partial^2 \phi}{\partial y^2} + K \frac{\partial^2 \phi}{\partial z^2} = 0 \quad (29)$$

Eq. 29 can be used when the coefficients $K$ are large and the variations of the external conditions with respect to time are so small as to make $\frac{\partial p}{\partial t}$ and $\frac{\partial \phi}{\partial t}$ negligible.

The coefficients of permeability determined from Eq. 18 depend on the average cubic distance between joints as well as the "effective" average opening, which, in turn, depend on the state of stress in the rock, on the interstitial pressures themselves, on temperatures, and on the possible filler materials and the characteristics of the formation. Observations of the deformations of rock under the weight of the dam suggest that important variations in the properties of a formation can occur and these variations can differ significantly over small distances.

When speaking of dam foundations, the problem can sometimes be reduced to a case of two-dimensional flow

$$K \frac{\partial^2 \phi}{\partial x^2} + K \frac{\partial^2 \phi}{\partial y^2} = 0 \quad (30)$$

which reduces to the Laplace equation by using the change of variables

$$Y = \sqrt{\frac{K_x}{K_y}} \phi \quad (31)$$

which, in turn, yields

$$\frac{\partial^2 \phi}{\partial x^2} + \frac{\partial^2 \phi}{\partial y^2} = 0 \quad (32)$$

Thus it can be seen that the solutions obtained by either analytical or numerical integration of Eq. 32 can be used also for anisotropic rock masses in which the openings of the joints in one direction are different than in another.

It should be noted that the most important flow of water through the foundation of a dam will generally occur along wide joints and faults. When studying percolation under steady flow conditions, this can be taken into account by considering such singularities in the mass as boundary conditions under which the fluid cannot flow.

or as strips of a much more permeable media. This then reduces the problem to one of filtration of water between two materials of different coefficients of permeability which can be solved numerically. 17

The importance of drainage in reducing and controlling interstitial pressure in the foundation of a dam has been emphasized in many publications. To be effective, drains must cross the various joints and faults in the mass. It would seem preferable to locate them in zones that are decompressed when the dam is loaded, which condition generally occurs near the upstream face. However, drainage of the rock near the downstream face should not be neglected. In many instances it has been found, however, that drainage alone is often not sufficient to reduce pressures to an acceptable value. Grouting of the fissures together with the attendant filling of weak zones of the rock with concrete and, occasionally, pre stressing have been found to be necessary procedures to strengthen the foundations and consequently reduce interstitial pressures. Such treatments affect the flow greatly and must be taken into account when computing the interstitial pressures. Recent observations and studies tend to indicate that the effect of grouting through single curves of holes is suspect in the control of seepage. 15 The difficulty involved in avoiding flow of the grout to zones of higher void ratios that are often far removed from the theoretical surface of the "grout curtain" is obvious.

However, another method for reducing seepage, that seems of interest in the case of concrete dams with rock foundations, would be the use of an impervious blanket of compacted clay or concrete (gunnite, for instance) which would be placed upstream of the dam, particularly at the intersections of joints or faults with the surface of the ground. In cases of thin arch dams founded in poor rock, which does not lend itself readily to grouting, the buildup of high interstitial pressures in the rock of the slopes and piping of filler material from the joints caused by high hydraulic gradients can be a major problem. Prestressing of the rock mass with cables anchored at great depths in deep drainage galleries can be considered a necessary precaution in such cases.

STABILITY OF SLOPES NEAR THE DAM

The seriousness of recent reservoir slope failures has shown that studies of the interstitial pressures must be made for parts of the rock mass for better estimates of stability. Failures will be more frequent when the phreatic line changes suddenly as a result of heavy rains or important changes in the level of the reservoir, particularly if the permeability of the rock mass is small. The case of a rapid decrease in the interstitial pressure towards the ground surface is of great consequence and quite dangerous.

The problem of computing pressures in steep slopes must be regarded as three-dimensional and, therefore, more complicated than what is usually considered. Not only is three-dimensional flow quite complex but it must be expected that there will be important differences in the permeability between...
the upper part of the slopes and the valley floor. In such cases, it is also more difficult to make the ground impervious by grouting, etc. The control of seepage by means of horizontal galleries and inclined or vertical shafts and drains seems to be the only practical solution in most cases, particularly when mountains near the dam are high and the phreatic waters from these mountains create interstitial pressures even higher than those produced by the impounded waters at or near to the base of the dam.

Another fact that must be kept in mind is that when a reservoir is created, certain changes in the prevailing condition of the earth's crust must be expected. In addition to the fact that the bottom of the reservoir will deform because of the weight of the impounded water, the upper part of the valley will rise as a result of buoyancy caused by penetration of the reservoir water into the rock mass of the slopes. This will tend to decrease the effective stress in the mass and, possibly, will cause greater width of certain joints and faults.

CONCLUSIONS

Results obtained by observations of the interstitial pressures in the rock foundations of concrete dams indicate that such pressures cannot be computed with sufficient accuracy by merely considering a steady state of flow through homogeneous, isotropic, and porous bodies. The time lags depicted in Figs. 9 and 10 between the water levels in reservoirs and the pressures measured in the foundations indicates that it is highly desirable to consider the conditions that cause an unstable state of percolation. In addition, the fact that the filtration of water through such masses is primarily dependent on the availability of open joints demonstrates the importance of the opening and closing of these passages resulting from internal and externally applied stresses. Filtration through joints also leads to contemplation of the anisotropic character of the rock masses and the variations with respect to time and distance of their coefficients of permeability in all directions. Results of continuous observation of the interstitial water pressures in the foundations and of the quantities of water that filter through the drains in relation to the water level and temperature are necessary for a better understanding of the problems involved.

The equations established herein are a preliminary attempt to define in a quantitative manner these complex problems of percolation of water and interstitial pressures inside rock masses subjected to external stress. It is thought that such statistical studies of rock masses, taking into account the scattered character of the variables, e.g., joint openness, joint spacing, joint orientation, and joint continuity, will result in a better understanding of the basic problems involved in the filtration of water in the foundations of concrete dams and in rock slope stability.

Finally, although the results obtained by the present approximate analytical or numerical methods for obtaining pore pressures are suspect, there is no reason to discard the procedures when designing large dams until more research has been accomplished and the results made available. However, the specific problems associated with peculiar geological features of each site may be considered in detail when determining the interstitial pressures in foundations of dams.