

A MODEL STUDY OF AIR TRANSPORT FROM UNDERGROUND OPENINGS SITUATED BELOW GROUND WATER LEVEL

Etude experimentale pour la circulation de l'air depuis une excavation souterraine située sous le niveau phréatique

Experimentelle Untersuchung über die Luftbewegung aus unterirdischen Hohlräumen unter dem Grundwasserspiegel

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SUMMARY

A parallel plate model (Hele-Shaw analogue) was used to study two phase fluid flow problems in jointed rock. The principal aim of the study was to determine the influence of groundwater on the flow of air from large underground openings. The information obtained was used to interpret the results of some borehole pumping tests that were performed in the field, and to predict the air leakage rates that might occur from large openings excavated in the same location. Drawdown tests were also performed to determine the head of water remaining above a large unlined opening, when the latter was located several diameters beneath the original groundwater level. Flow through uniformly jointed rock and through individual joints has been considered.

RESUME

Un modèle à plaques parallèles (analogie à celui de Hele Shaws) a été utilisé pour étudier les problèmes d'écoulement de fluides diphasiques dans les roches fissurées. Le but principal de l'étude était de déterminer l'influence de l'eau souterraine sur l'écoulement d'air depuis une excavation souterraine importante. Les informations obtenues ont été utilisées pour interpréter les résultats d'essais de pompage dans des sondages et prévoir les débits d'air pouvant intervenir autour de grandes excavations dans la même situation. Des essais de rabattement ont de plus été réalisés pour déterminer la charge de l'eau demeurant au-dessous d'une grande cavité sans revêtement lorsque le niveau se trouve rabattu de plusieurs diamètres de la cavité, au-dessous du niveau statique original. Les écoulements à travers une roche uniformément perméable et aussi à travers des fractures individuelles ont été considérés.

ZUSAMMENFASSUNG

Ein Modell mit parallelen Platten (ähnlich wie das Hele Shaws'sche Modell) wurde verwendet, um die Strömungsvorgänge einer zweiphasigen Flüssigkeit in zerklüftetem Fels zu untersuchen. Der Hauptzweck der Untersuchung war, den Einfluss des Grundwassers über die Luftbewegung aus grossen unterirdischen Hohlräumen abzuschätzen. Die gewonnenen Ergebnissen wurden für die Auswertung von einigen in situ Pumpversuchen und auch für die Abschätzung der Durchflüßmengen von Luft aus Hohlräumen in der gleichen Lage angewendet. Absenkungsversuche wurden auch durchgeführt, um die Wasserspiegellhöhe über einem grossen unverkleideten Hohlraum bei einer Absenkung von mehreren Hohlraumdurchmesser des Wasserspiegels zu bestimmen. Strömungen durch einen gleichmässig zerklüfteten Fels sowie durch einzelne Klüfte wurden betrachtet.

INTRODUCTION

The containment of fluids in underground openings in rock is common practice in those Scandinavian countries which have the benefit of widely distributed igneous and metamorphic foundation rocks. It is not normally economically attractive to consider complete artificial lining of these openings, so leakage rates and velocities of migration are largely dependent on the inherent rock mass permeability. There is a class of problems where fluid migration need present only a limited problem, if the local ground water level can be maintained so that the hydrostatic pressure exceeds the fluid

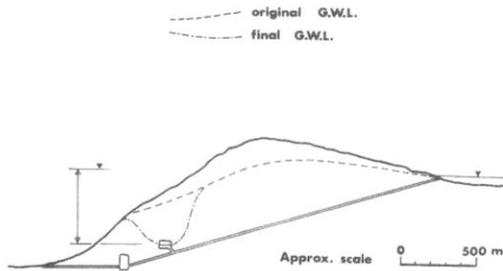
storage pressure. (Morfeldt (1).) However, a class of problems has recently come to notice in which it is difficult, if not impossible for ground water levels to be maintained. This is because the fluid may be at a much higher pressure than the local hydrostatic pressure, or alternatively, the opening and associated tunnels may be located at such shallow depth below the surface that localized drainage of the ground water occurs.

The model study to be described in this report was directed towards several complicated problems of

fluid flow through rock masses. In each case flow from (or towards) an underground opening was considered, and the latter was generally situated a short distance below ground water level. The principal aim of the study was to determine the influence of groundwater on the flow of air from large underground openings in jointed rock.

Some of the theoretical aspects of separate air or water flow from openings in jointed rock have been discussed previously. (Barton (2).) The present study takes the problem a stage further since it is concerned with air flow through water which is a much more complicated two phase problem.

Figure 1. Two engineering examples of changing groundwater conditions.



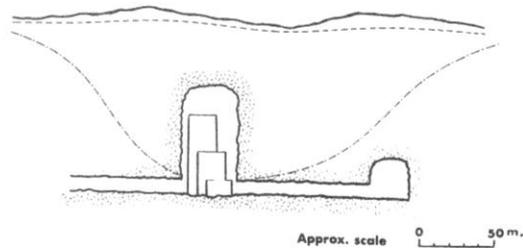
A) Compressed air surge chamber for hydro power scheme.

Figure 1 shows two engineering examples of the type of problems under consideration. Diagram A) illustrates an unlined compressed air surge chamber connected with a hydroelectric generating plant. Pressure surges in the water pressure tunnel caused by shut-down are designed to be absorbed by the body of compressed air trapped in the upper half of the chamber. The air pressure generated by the elevated top reservoir will in most cases exceed the local hydrostatic pressure, perhaps by as much as 10 atmospheres. Consequently water will be displaced from above the chamber and some form of air flow path will be developed up to the surface. Since a permanent installation of air compressors is needed to replenish the compressed air lost from the chamber, it is important to make some sort of estimate of requirements. This might have to include artificial sealing of the chamber. In either case the only field data available will be that from borehole pumping tests. Two such compressed air surge chambers have already been built in Norway (Driva and Jukla) and others are planned but so far these have not been put into operation. The present model tests of two-phase flow provide valuable information about the complicated problem of air through water flow.

A second engineering problem which is similar to the first is illustrated in diagram B) of figure 1. Several surface and underground sites are presently being investigated in Norway, to determine a suitable location for a nuclear power plant. Environment and safety considerations may require that an underground site is used. It is important to contain the radioactive gases that may fill these large openings in the event of a serious accident. The rock mass surrounding the openings must be sufficiently impermeable to limit the leakage and control the time for transport to the surface, so that any gas reaching the surface will present no hazard, either from the point of view of quantity

or activity. For economic reasons these openings will be built at shallow depth. It is therefore very likely that the ground water will be drained around such excavations since the dimensions are similar to the depth below surface.

Model tests were performed to investigate what depth the large opening needed to be below the undisturbed ground water level, for the drawn down level to still apply sufficient head to prevent gas leakage. Tests were also performed to estimate the rate of leakage of gas (air was used here) should the pressure exceed the head of ground water. In all cases where air leakage through water was measured,



B) Near surface opening for nuclear plant.

it was compared with the leakage rate of air when the model medium was quite dry. A comparison of these two experimental results enables one to make an estimate of air leakage from an opening in a saturated rock mass (air/water flow) based on the results of in situ pumping tests from boreholes, when injecting water into a saturated rock mass (water/water flow). Figure 2 illustrates the steps that are involved in this process.

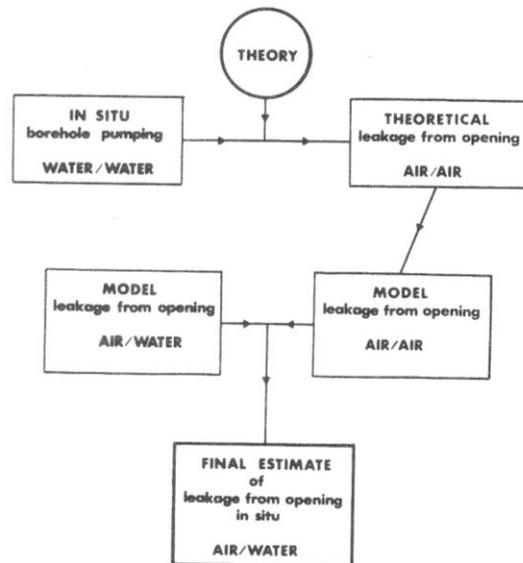


Figure 2. The steps involved in estimating the leakage of air from a large opening through a saturated rock mass, based on the results of conventional borehole water pumping tests.

I. DESCRIPTION OF MODEL FLOW ANALOGY.

The present study of air and water flow from near surface openings was based on the viscous flow analogue (or Hele-Shaw parallel plate model). This is a well known device for two-dimensional ground-water investigations. (See for instance Bear et al. (3).) It is based on the similarity between the differential equations which govern saturated flow in porous media (or isotropically jointed rock masses) and those describing the flow of a viscous fluid between two parallel plates with a capillary interspace. The model (or analogue since no physical permeable medium is involved) was first developed by Hele-Shaw (4) for studying potential flow patterns around variously shaped bodies. When the flow is laminar the velocity profile in the capillary space between the plates is parabolic. The equivalent permeability is obtained by analogy with the Darcy equation of flow through porous media and is equal to $b^2/12(\text{cm}^2)$, where (b) is the width of the interspace. This intrinsic unit of permeability is the same for water flow through the saturated interspace, and for air flow through the dry interspace.

There is thus a direct analogy between two dimensional flow in a porous medium and the flow between parallel plates. The velocity in the former case corresponds to the mean velocity in the latter. For this reason flow lines and equipotentials are geometrically similar between the analogue and the idealized field conditions. The analogue can also be used to reproduce experimentally the two-dimensional motion of the interface between two fluids in a porous medium. In the present model a fluid of low viscosity (air) is displacing a fluid of higher viscosity (water). When the velocity of the interface is sufficiently high instability develops in a direction perpendicular to the advancing interface. (Saffman and Taylor (5).) This problem is well known in the Petroleum Industry. Water drive methods for displacing oil into the producing well have to be carefully regulated to prevent unstable tongues or cones of water from penetrating and mixing with the oil, which has higher viscosity. The relative densities of the fluids does not matter provided the velocity is sufficiently large.

The relevance of an analogy for flow through porous media to flow through jointed rock masses must now be considered. It is obvious that any streamline traced by a dye injected into the parallel plate model or any interface between air and water moving through the model will only trace out a mean path compared to the same streamline or interface passing through a porous medium that is not quite homogeneous. The same will be true when the analogy is applied to flow through jointed rock masses, only more so. In fact for flow in the model to have any real relevance to flow through jointed rock masses the following conditions must be met :

1. Flow through both rock mass and model should be in the laminar range.
2. The rock mass should be jointed in a regular manner such that the directional permeabilities are of similar magnitude.
3. The dimensions of the openings from which flow occurs should be several times larger than the mean spacing of the rock joints.

In general, turbulent flow occurs only when abnormally large joints are found in the vicinity of

abnormally large gradients. Unfortunately this combination can often complicate the interpretation of borehole pumping tests, due to the large gradients associated with radial injection from a small diameter hole (see Baker (6) and Maini (7).) However in the case of flow from large openings such problems will seldom exist.

The requirement of principally laminar flow through the rock mass (implying linear pressure-flow relationships) should of course be maintained also in the model. This was readily checked by injecting potassium permanganate dye streams. In certain cases of high pressure and wide capillary spacing these streamlines became wavy and disturbed. Tests under these conditions could therefore be avoided.

The second requirement of regular jointing and isotropic directional permeabilities can be judged from the study by Wilson and Witherspoon (8), who developed a finite element approach for evaluating flow through jointed rock, and illustrated its application on a jointed dam foundation problem. Orthogonal, two dimensional joint systems having variable directional permeabilities were compared with the equivalent porous media. Their study showed that the use of an equivalent porous medium to replace a joint system could be quite accurate in certain situations, even when anisotropy was involved.

Anisotropic permeability cannot be modelled with a parallel plate apparatus and it is obviously essential to interpret the results accordingly. (Changing the capillary space locally gives the model inhomogeneous permeability characteristics, not anisotropy). This limitation will be greatly reduced if the relevant joint sets are tested in the field, for relation with the model permeability. The principal direction of flow in isotropic air/water experiments is vertical. Therefore the permeability of any two near-vertical joint sets will ultimately account for air leakage and time of transport to the surface. When the parallel plates are placed in a vertical position they represent a vertical cross section of the flow domain. In the present experiments the model borehole and opening were aligned with axes perpendicular to this vertical cross-section.

The third requirement; that the opening which is modelled should be of much larger dimensions than the mean spacing of the rock joints is not easy to satisfy. The flow of water or air from a borehole into a saturated rock mass will normally be controlled by flow in a limited number of joints.

The statistical method of interpreting the results of borehole pumping tests that was developed by Snow (9) has been utilized in interpreting some of the results of pumping tests that were recently performed by the NGL. The basic assumption is that the water-conducting joints, which may represent a small percentage of all the joints present, form a cubic intersecting network, as illustrated in Figure 3.

Table 1 shows some typical results obtained from an analysis of pumping tests that were performed in a 45° inclined borehole. The tests were performed in Norway as part of a preliminary field assessment for a large underground opening in rock, to house a nuclear power plant. (DiBiagio and Myrvoll (10) and DiBiagio (11).) The rock mass consisted chiefly of banded gneiss.

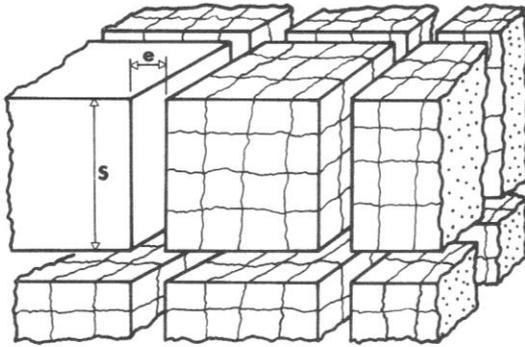


Figure 3. Idealized cubic arrangement of equivalent water-conducting joints.

Depth zone Metres vertically below surface	Equivalent joint and mass parameters		
	(S) spacing (metres)	(e) opening (cm)	(n) porosity
1.0- 18.7	1.51	0.0091	0.00018
18.4- 41.0	2.56	0.0068	0.000079
40.6-120.0	3.13	0.0060	0.000057

Table I. The geometry of water conductors predicted by Snow's method (9).

The equivalent spacing of water-conducting joints as tabulated above was approximately 4 to 15 times the spacing between all the joints observed in the drill core, excluding mechanical fractures, and excluding the occasional close swarms of fissures. (Fredriksen (12).) The implication that only a small percentage of joints in crystalline rocks can conduct water is supported by the fact that in the three depth zones, between 5 and 20% of the 5 metre stages showed zero flow, even though the relevant 5 metre lengths of cores were jointed in 15 to 30 places.

It appears that at greater depths such as those associated with compressed air surge chambers, leakage may be conducted by even fewer joints, even when the chamber is in excess of 50 metres in length. In such cases as this the assumption of a porous medium/isotropic rock mass flow analogy must break down, and the parallel plate flow experiments can only be interpreted in their relation to flow in individual joints intersecting the opening. In other words instead of utilizing the analogy relating flow between parallel plates and flow through a uniformly jointed rock mass, the experiments must be interpreted as distorted scale models. (It is not physically possible to construct a model of an individual rock joint of large area, without distorting the joint width. Geometrical similarity cannot be obtained).

In most of the model experiments to be described the distorted scale is not important since two model phenomena (for example air/air flow and air/water flow) are being compared with each other. We are not trying to relate either one of them numerically to the field problem.

II. BASIC TEST EQUIPMENT.

The parallel plate apparatus consisted essentially of two, 3 cm thick perspex plates measuring 100 cm by 120 cm, rigidly clamped parallel to one another

to form a narrow, vertical capillary space. A space width of approximately 0.50 mm was used for the preliminary single phase experiments, and this was increased to approximately 1.30 mm for the two phase (air/water) experiments, to reduce the surface tension effects. The perspex plates were stiffened and supported by a grid of reinforcing channels. These are shown in figures 4 and 5. The outside edges of the plates were clamped by a total of eighteen 'C' clamps, thereby pressing the two plates against an array of brass spacer pieces, which were distributed within the capillary space beneath each channel intersection. The spacers were machined from a 3 mm diameter brass rod. It is believed that the presence of these pieces in the capillary space had negligible effect on the overall flow characteristics. They reduced the total flow area by only 0.07%.

The four central adjustment screws shown in Figure 5 are typical of each of the twelve channel intersections. In general these screws were tightened so that the perspex plates were forced against the spacer pieces. When this did not produce symmetrical dye streamlines during pump-in tests from the model borehole the adjustment screws could be loosened or further tightened, to compensate for the inevitable local irregularities in the perspex plates.

The model borehole shown in Figure 5 preceded the much larger opening which was bored in the same position for later flow experiments. The same valve chamber was used to seal both openings. The centre of these openings was 37.5 cm below the open top surface of the model, 60 cm from both vertical boundaries, and 62.5 cm above the lower boundary. Therefore, excluding the top surface which represented an equipotential ground surface, the boundaries were 600 and 40 radii distant from the model borehole and opening respectively.

A) Model boundary conditions.

A water tight rubber seal was fixed round the base and two vertical edges of the capillary space to contain the water in water flow experiments. As such the rubber seal represented three boundary flow lines. This rather inflexible boundary condition was improved by having the four edges of each perspex plate bevelled, so that when the plates were placed together a triangular channel was formed between the plates and the rubber seal. Figure 6 illustrates the two possibilities created by this channel. A thin-walled rubber tube was placed within the three edge channels, with one end closed and the other end connected to either a compressed air line, or a vacuum. The edge channels were thereby either closed representing boundary flow lines, or open representing boundary equipotentials.

B) Model flow patterns.

The effect of these two boundary conditions on the flow lines is shown in Figures 7 and 8. Flow of water from the 2 mm diameter model borehole was traced by 8 injections of potassium permanganate dye. Each injection hole of 1 mm diameter was located 6 cm from the borehole centre and at 45° intervals. They represented negligible disturbance to the overall flow. The upper five flow lines obtained with "closed" model boundaries shown in Figure 7 appear to closely resemble the corresponding theoretical paths shown in Figure 9. However

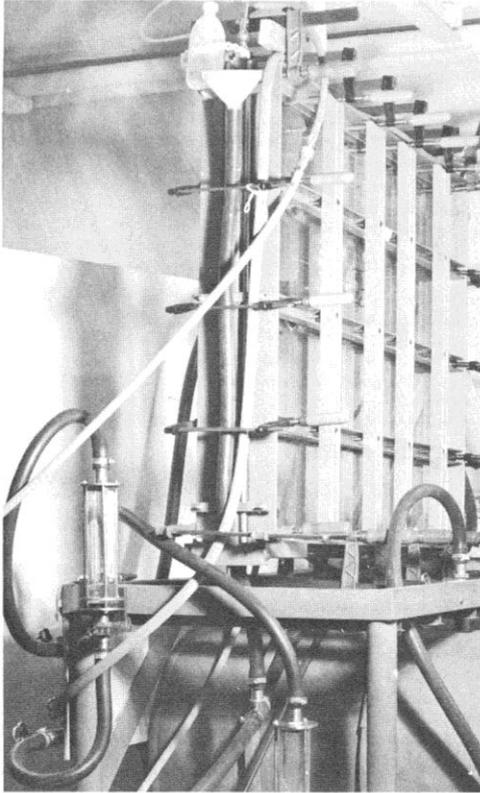


Figure 4. A general view of the reinforced parallel plate model and accessories.

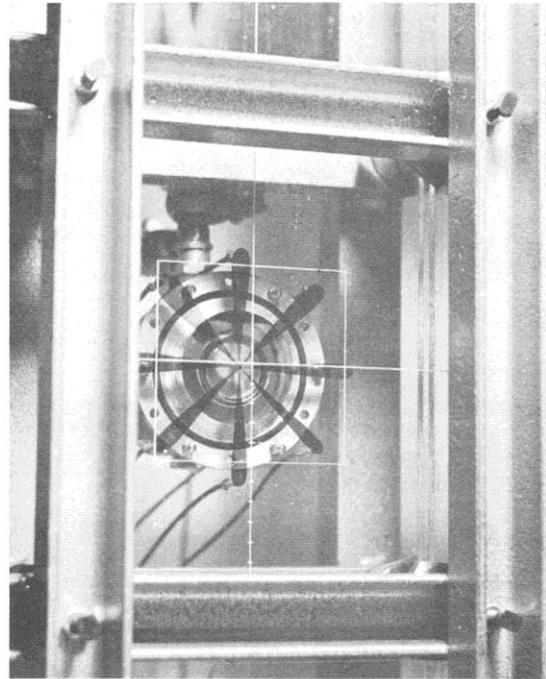


Figure 5. Radial return flow of water when the model borehole is opened to atmospheric pressure.

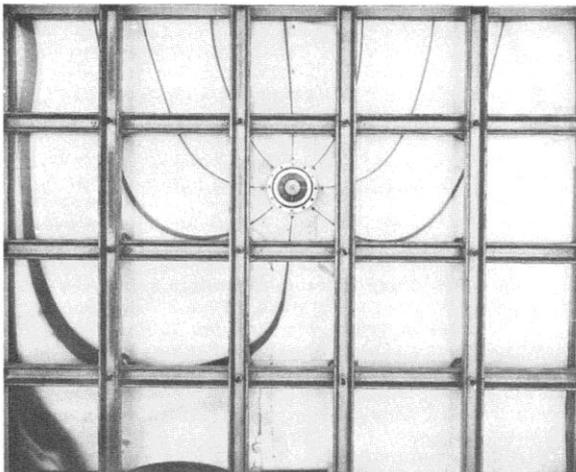


Figure 7. Flow lines obtained with "closed" model boundaries such that the two sides and base were boundary flow lines.

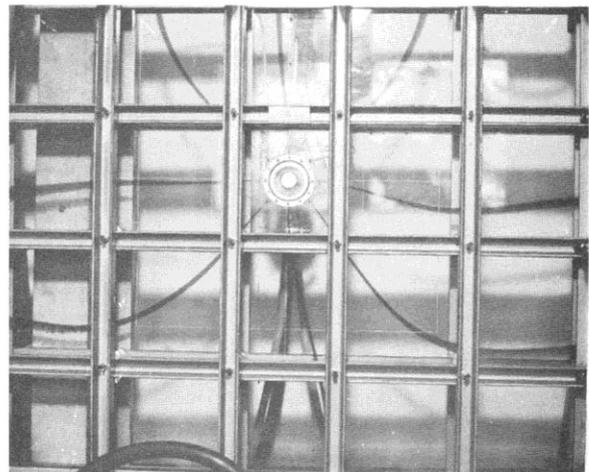
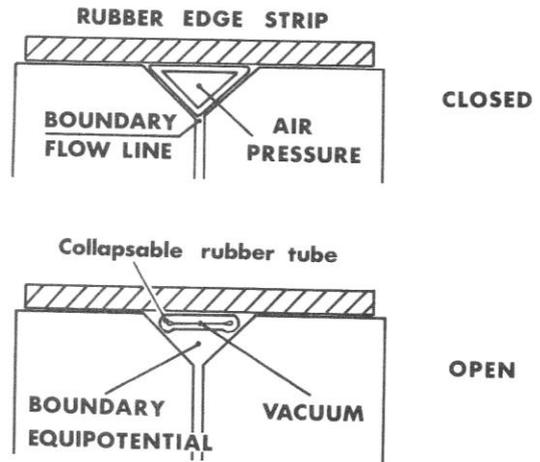


Figure 8. Flow lines obtained with "open" model boundaries such that the two sides and base were boundary equipotentials.

a certain degree of concentration of the flow lines was produced by the "closed" boundary flow lines, and as will be seen shortly, this caused elevated pressure distributions and shortened transport times along the shortest vertical path to the surface. These subsequent tests showed that the "open" model boundaries (Figure 8) produced vertical flow characteristics which agreed closely with the theoretical predictions, and this boundary condition was therefore adopted for all subsequent tests.

Figure 6. Two boundary conditions that are obtained by pressurizing or collapsing a rubber tube placed within the side channels.



III. EQUATIONS FOR FLUID FLOW FROM HORIZONTAL CIRCULAR OPENING TO GROUND SURFACE.

The simplest example of flow from a circular opening drilled in a permeable isotropic medium is that of perfectly radial flow, where the medium is theoretically infinite, and the flow is characterised by circular concentric equipotentials. These conditions, which are most frequently assumed in oil well technology are not applicable to the present study. Here we are concerned with flow from a long horizontal circular opening towards an equipotential ground surface which is assumed to be linear and saturated with the same fluid up to the

surface. The standard method for developing empirical solutions to these problems is to imagine that the equipotential ground surface is a linear boundary midway between a "source" and an equal and opposite "sink". This concept is illustrated by Figure 9. The equations describing the two dimensional flow under these conditions are developed in several texts, for instance by Muskat (13). The equations that are relevant specifically to flow in individual joints as distinct from porous media were summarized in a previous report (Barton (2).).

A) Pressure distribution for water flow.

The theoretical pressure distribution for an incompressible fluid flowing from a circular opening along the shortest vertical path to the surface is as follows :

$$P_h = \frac{P_r}{\log_e (2D/r)} \cdot \log_e \left(\frac{2D-h}{h} \right) \quad (1)$$

where :

- P_h = excess pressure at height (h) vertically above the centre of the opening.
- P_r = excess pressure at centre of opening of radius (r).
- D = depth of centre of opening below surface.

The logarithmic pressure decay which is represented by this equation is very marked for flow from a small diameter borehole, but less severe from an opening of large diameter situated the same distance beneath the ground surface. It is assumed that flow is laminar throughout, otherwise a much more severe drop in pressure occurs close to the opening as shown by Baker (6). Equation 1 is relevant to flow in a porous medium, or to flow in a uniform vertical joint intersected by the opening, provided that the above condition is satisfied.

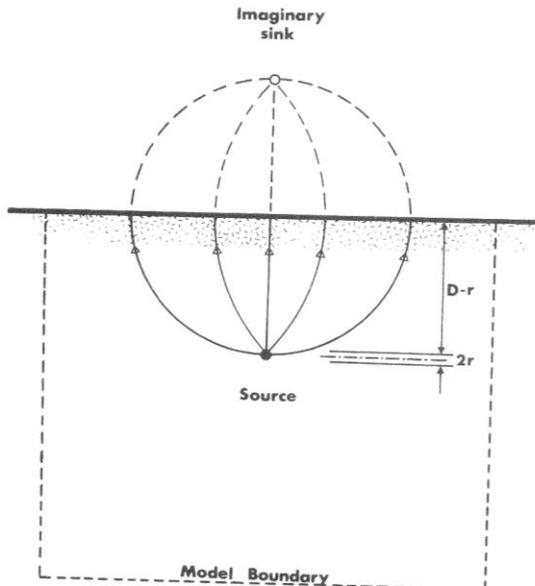


Figure 9. Flow towards a horizontal ground surface which represents a linear boundary midway between a "source" and an equal and opposite "sink".

B) Pressure distribution for compressible gas flow.

When the flowing fluid is a gas, compressibility effects are a source of complication since (unlike for the flow of liquids at engineering pressures) the density varies along the flow path. The pressure distribution for steady state gas flow is obtained by replacing the symbols for excess pressure (P_h, P_r etc.) appearing in the expressions for pressure distribution of liquids, by the same symbols to the power $(1+m)$. (see Muskat (13).) The symbol (m) represents the thermodynamic character of the gas expansion. For isothermal expansion this is simply done by raising to the power 2. It should be noted that the symbols P_h, P_r appearing in equation (1) were pressures in excess of atmospheric pressure. For the case of gases it is necessary to work in terms of absolute pressures. Hence with P_h and P_r as absolute pressures, equation 1 becomes for gas flow :

$$P_h^2 - 1^2 = \frac{(P_r^2 - 1^2)}{\log_e \left(\frac{2D}{r}\right)} \cdot \log_e \left(\frac{2D-h}{h}\right) \quad (2)$$

It is instructive at this stage to examine the difference in pressure distribution between steady state liquid and gas flow. The following numerical examples will be evaluated using equations 1 and 2

1. Horizontal borehole of radius (r) = 2.3 cm
2. Horizontal opening of radius (r) = 6.0 metres

Depths of centres below ground surface (D) = 40 metres

Excess pressures in borehole and opening (P_r) = 2 atmospheres

Figure 10 gives the pressure distributions for water (in excess of local hydrostatic pressure) and air, when injected from the borehole (curves W_b and A_b) and from the large opening (curves W_o and A_o). The following points can be observed :

1. The pressure gradients are much steeper close to the borehole than close to the large opening. A pressure decay of 50% occurs within approximately $1\frac{1}{2}$ metres of the borehole, yet more than 17 metres from the centre of the large opening. This difference could be even more accentuated if there were significant pressure losses due to inertia effects at the joint entries. A combination of high pressure and wide joint openings would cause a rapid acceleration of flow round the small radius joint entries, and if turbulence developed significant pressure losses would occur, in excess of the logarithmic decay shown in Figure 10. (see Baker (6) and Maini (7).)
2. At low injection pressures such as those that have to be used in the model tests, compressibility effects will be minimal. The air pressure distribution curves will lie closer to the water pressure distribution curves.

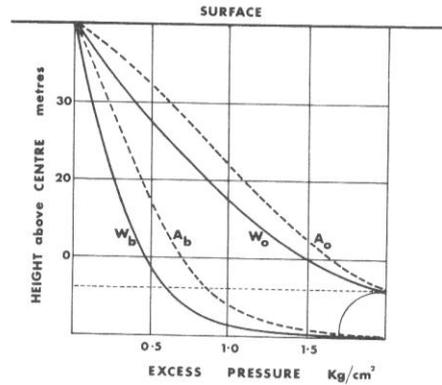


Figure 10. Theoretical distribution of excess pressure for air (A) and water (W) flowing from a small borehole (b) and a large opening (o)

C) Volumetric water flow rate.

The volumetric water leakage rate from an underground opening of length L (cm) is as follows :

$$Q = \frac{2\pi(K_m)Lg(P_r)}{\mu \log_e (2D/r)} \quad (3)$$

- where
- Q = volumetric flow rate (cm^3/sec)
 - K_m = equivalent mass permeability (cm^2)
 - g = acceleration due to gravity ($981 \text{ cm}/\text{sec}^2$)
 - P_r = excess pressure in opening (gm/cm^2)
 - μ = dynamic (or absolute) viscosity of water ($\text{gm}/\text{cm} \cdot \text{sec}$)
 - D, r = see figure 9

The equivalent mass permeability (K_m) is the porous media equivalent of a regularly m jointed rock mass. When considering flow in a single vertical joint, (K_m) is replaced by (K_j) the equivalent joint conductivity, and the length (L) is replaced by (e), the mean joint opening (cm). We can therefore modify the above equation to describe flow in the parallel plate model, whose equivalent permeability is $b^2/12$ (cm^2), where (b) is the width of the capillary space. Equation 3 can therefore be rewritten as :

$$Q = \frac{\pi b^3 g (P_r)}{6\mu \log_e (2D/r)} \quad (4)$$

In the present experiments this formula was used to check the manometer readings of (P_r) for a measured flow rate (Q) and capillary space (b).

D) Volumetric flow rate for compressible gas.

Equations 3 and 4 which were for incompressible flow of liquid, are also applicable to flow of compressible gases through the dry medium. It is only necessary to alter the value of viscosity to that of the gas, and to interpret the calculated flow rate as that occurring at the algebraic mean absolute pressure of the flow domain. (Muskat (13).) Figure 11 shows that the ratio of viscosities between air and water is approximately 1/73 at a common fluid temperature of 10°C.

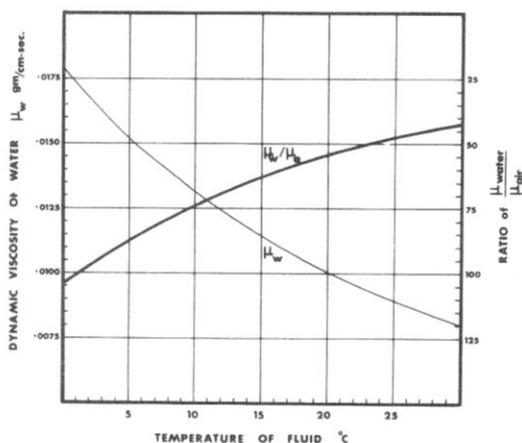


Figure 11. The dynamic viscosity of water, and the ratio of viscosities for water and air as a function of fluid temperature.

When the flow rate of air is to be estimated for a given flow temperature, both the curves given in Figure 11 will need to be used. The required value of viscosity for air is obtained by dividing the value of μ_w by the ratio μ_w/μ_a .

The interpretation of the calculated air flow rate as that occurring at the algebraic mean absolute pressure of the flow domain is common both to purely radial flow, and to flow towards a linear equipotential. The product of the flow rate and the mean absolute pressure (in atmospheres) produces the result in conventional "normal" pressure flow units, i. e. normal cm^3/sec .

For convenience the theoretical example of radial flow from a length of borehole drilled in an isotropic, permeable medium will be considered first. The method for calculating the transport time for flow from the borehole walls out to some radius (R) in the medium, is to divide the volume of conducting pores by the volumetric flow rate. (See for instance Jacob (14).)

$$t = \frac{\pi R^2 n}{Q/L} \quad (5)$$

where t = transport time (sec.)
 R = radius of flow cylinder (cm)
 n = porosity due to fluid conducting pores

Q = volumetric flow rate (cm^3/sec) per length, L of flow cylinder (cm)

When considering flow along individual rock joints or flow in the capillary space of a parallel plate model, the porosity (n) is equal to 1.0 since the whole medium is water filled, and the length (L) becomes (b) or (e) the width of the capillary space or joint opening respectively.

Flow from an opening towards a linear equipotential is a little more complicated than the above radial flow example. An expression for transport time along the shortest vertical path to the surface through an isotropic permeable medium is given by Jacob (14). His development has to be modified for the case of flow in a single vertical joint intersected by a borehole, and this case also corresponds to the transport time for flow in the parallel plate model.

$$t = \frac{D^2 \mu \log_e \left(\frac{2D}{r} \right) \left[\frac{2}{3} - \frac{r^2}{D^2} + \frac{r^3}{3D^3} \right]}{2g (K_j) (P_r)} \quad (6)$$

For small values of opening radius (r) relative to depth below surface (D) this time approaches:

$$t = \frac{D^2 \mu \log_e \left(\frac{2D}{r} \right)}{3g (K_j) (P_r)} \quad (7)$$

The equivalent joint conductivity (K_j) can once again be expressed as $b^2/12$ (cm^2) for the case of flow in the parallel plate model.

IV. PRELIMINARY TESTS WITH MODEL BOREHOLE, FOR COMPARISON WITH THEORY.

A) Pressure distribution from model borehole to surface.

An important check on the relative merits of the two boundary conditions was provided by measurements of the pressure gradients, taken at several points between the model borehole and the surface. The static pressure was measured in vertical plastic standpipes which were connected to the water filled capillary space by 1 mm diameter holes, carefully drilled through, and perpendicular to, the rear perspex plate.

Figure 12 shows the pressure decay for four different excess pressures that were applied to the model borehole. The four full curves were obtained with "open" model boundaries, such that the base and two sides were boundary equipotentials (see Figure 8). Three pressure decay curves are shown having the same excess pressure of 15 cm of water in the model borehole. It can be seen that the theoretical curve (T) predicted by equation 3 lies acceptably close to the experimental pressures measured with "open" boundaries (O), but well below those measured when the model has "closed" boundaries (C). The concentrated flow pattern illustrated in Figure 7 was responsible for elevated pressures at all measuring points between the borehole and the surface.

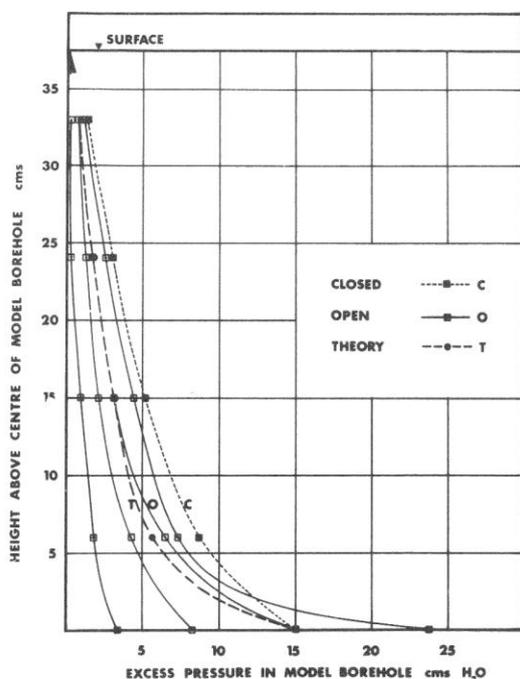


Figure 12. Logarithmic pressure decay from model borehole to surface, and comparison of two boundary conditions with theory.

B) Transport time from model borehole to surface.

Many of the earlier tests performed in this parallel plate model were to investigate the possibility of studying the transport time for transient air flow, for comparison with the steady state solution. The leakage of radioactive gases resulting from an accident in a nuclear plant actually involves a sudden increase of pressure in the underground opening, and it was anticipated that the transport time for this transient case would be shorter and therefore more critical than that of steady state flow, as a result of compressibility effects.

In the circumstances it proved impossible to investigate this aspect, due to the very great difficulties of adequately colouring or otherwise tracing the air, and the associated difficulty of detecting and measuring extremely short transport times. By comparison it proved extremely easy to measure the transport time of transient flow tests using dyed water.

Transport time was measured from the instant that the borehole valve was opened up to the time when the coloured flow "bloom" reached the top surface of the model, a distance of 37.5 cm. Manual timing with a stop watch was checked against a cine film of each test. In the case of water (which is incompressible) the transport time for transient flow is the same as that for steady state flow, since there are no compressibility effects. Table II shows a comparison of the transport times for "open" and

"closed" boundaries, and the theoretically predicted transport times for two-dimensional flow from an opening in a semi-infinite isotropic medium beneath a horizontal ground surface. The theoretical transport times were calculated using equation 6.

Excess pressure in borehole (cm of water)	Transport time in seconds		
	Boundary conditions		
	(O) open	(C) closed	(T) theory
34.1	7.6	6.9	7.51
29.6	8.7	6.9	8.65
23.8	10.0	8.6	10.76
15.1	17.0	12.0	16.94
11.9	24.2	16.0	21.50
8.2	31.2	19.2	31.08

Table II. Comparison of theoretical transport times for flow in semi infinite medium with those for "open" and "closed" model boundaries.

It is reasonable to conclude from the comparison of pressure distributions and transport times that, as far as vertical flow is concerned, "open" model boundaries produce quite an acceptable approximation to the results that would be expected for flow beneath an unlimited horizontal ground surface. In any case the two artificial model boundaries are not without practical significance. Vertical shear zones of high permeability (corresponding to "open" model boundaries) or vertical clay filled fault surfaces (corresponding to "closed" model boundaries) all too frequently intersect a theoreticians assumptions of in situ conditions.

V. ILLUSTRATION OF MODEL SCALES, BY COMPARING WITH FIELD TEST RESULTS.

As indicated earlier there are two ways of interpreting the results of experiments in parallel plate models. It is most usual to employ the analogy between flow in an isotropic porous medium and flow in the capillary space between parallel plates. The isotropic porous medium is here extended to include, in approximate terms, a uniform jointed rock mass. This extended analogy is valid provided flow is in the laminar range, and provided the mean joint spacing is small compared to the dimension of the openings from which flow is occurring. This extended analogy will seldom be valid for the case of flow from a small diameter borehole, and may not even be valid in the case of flow from a large underground opening if the joint spacing is exceptionally large. In such cases it is necessary to interpret the parallel plate experiment as a distorted scale model, where the capillary space represents a single equivalent water conducting joint, representing those that intersect the particular rock mass.

A) Field pumping tests.

A comprehensive set of borehole pumping tests were recently performed in Norway as part of a field assessment for a sub-surface nuclear power plant (DiBiagio and Myrvoll (10) and DiBiagio (11).) Water pump-in tests were performed from 2 and 5 metre sections of one particular inclined (45°) borehole of 4.8 cm diameter drilled in a gneissic rock mass which was more or less saturated to the

surface. Following these conventional permeability tests, a series of air pump-in tests were performed from several packer positions down the same borehole. Small quantities of radioactive xenon gas tracer were injected into the air flow pipe after steady state conditions had been reached at the given input pressure.

Transport times were measured as the interval between injecting the tracer into the borehole, and the first indication of tracer in the observation holes near the ground surface. When the packered section of the borehole was approximately 30 metres vertically below ground surface, transport times of between 12 and 28 minutes were obtained for borehole pressures varying from 1.6 to 0.3 atmospheres respectively.

The water pump-in tests were conducted by Entreprenørservice A/S using leather cup packers spaced at 5 metres. The whole 180 metres of borehole was tested in this manner. At a later date NGI performed a series of tests with a 2 metre spacing of Geonor expanding packers. These tests were run between 20.0 and 62.5 metres and can be compared with the corresponding 5 metre stages run between 21.5 and 62.5 metres.

The table of results given below was obtained using Snow's method (9). The idealized rock mass is assumed to be regularly jointed and flow is assumed to occur in a cubic network of equivalent "parallel plate" joints as illustrated in Figure 3. Estimates of equivalent joint opening (e), equivalent joint spacing (s) and rock mass porosity (n) are obtained by the method.

2 metre spacing			5 metre spacing		
e (cm)	s (metres)	n	e (cm)	s (metres)	n
0.0063	2.50	7.6×10^{-5}	0.0061	2.13	8.6×10^{-5}

Table III. Comparison of 2 metre and 5 metre packer spacings.

The slightly increased water conducting potential illustrated by the 5 metre tests may be a reflection of the higher testing pressures used (10 kg/cm^2), compared to 2 kg/cm^2 in the 2 metre stages. There was some evidence of slight joint opening at pressures between 5 and 10 kg/cm^2 , even in the competent gneissic rock mass tested here. However, the results are extremely close in practical terms.

Following this systematic water testing between 20.0 and 62.5 metres, four special test zones were selected, and each was individually water tested before commencing air injection experiments. The test zone between 42.6 and 45.6 metres indicated an equivalent mass permeability of 0.67 Lugeons at pressures below 2.5 kg/cm^2 . This value is consistent with the overall results given in Table III, since if we assume there is only one water conducting joint in this 3 metre long test zone, it can be shown that the equivalent joint opening is 0.0068 cm. We therefore have a test zone which can be considered representative of the idealized conducting joints between 20.0 and 62.5 metres. Air injection tracer tests conducted in this zone will now be compared with the model tests, to illustrate the use of model scales and

methods of full scale prediction.

B) Model predictions.

The geometrical scale of the model was determined by the depth below surface in the field, which in this example was 27 metres. Since the depth of the model borehole was 37.5 cm the geometric scale was as follows :

$$S_g = \frac{\text{field dimension}}{\text{model dimension}} = \frac{2700}{37.5} = 72$$

therefore

$$S_h = \frac{\text{field pressure (head)}}{\text{model pressure (head)}} = 72$$

A typical model test selected from Table II was run at an excess pressure (head) of 15.1 cm of water, and gave a transport time for water flow of 17.0 seconds, when the capillary spacing was 0.05 cm. This model pressure head represents 10.9 metres (or 1.09 kg/cm^2) in the field. The transport time for water is converted to that for steady state air flow by the difference in viscosities between air and water, provided the intrinsic permeability $K (\text{cm}^2)$ remains the same for air and water flow. At 10°C this viscosity difference is approximately 73. Therefore the transport time for air flow at the same model pressure would be $17.0/73 = 0.233 \text{ sec}$.

The time scale converting events in the model to equivalent events in the field is obtained from the following relationship, which is given by Aravin and Numerov (15) :

$$S_t = \frac{S_g \times S_n}{S_k} \tag{8}$$

where S_t = time scale, S_g = geometric scale
 S_n = porosity scale, S_k = permeability scale.

Firstly the equivalent porous medium analogy will be considered and secondly the distorted scale model.

(i) EQUIVALENT POROUS MEDIUM

$$S_k = \frac{0.67 \times 1.3 \times 10^{-10}}{1/12 (0.05)^2} = 4.18 \times 10^{-7}$$

$$S_n = \frac{0.000081}{1.0} = 8.1 \times 10^{-5}$$

(0.000081 is the mean statistical result (Snow (9).) from table III).

$$S_t = \frac{72 \times 8.1 \times 10^{-5}}{4.18 \times 10^{-7}} = 13960$$

(ii) DISTORTED SCALE MODEL OF SINGLE JOINT

$$S_k = \frac{1/12 \times (0.0062)^2}{1/12 \times (0.050)^2} = 1.54 \times 10^{-2}$$

(from mean of table III results)

$$S_n = \frac{1.0}{1.0} = 1.0$$

$$S_t = \frac{72 \times 1.0}{1.54 \times 10^{-2}} = 4670$$

TRANSPORT TIME PREDICTIONS :

(i) equivalent porous medium

$$t = 13960 \times 0.233 = 3250 \text{ seconds} = \underline{54.2 \text{ minutes}}$$

(ii) distorted scale model of single joint

$$t = 4670 \times 0.233 = 1090 \text{ seconds} = \underline{18.2 \text{ minutes}}$$

The steady state air tracer field tests performed in the zone from 42.6 to 45.6 metres gave a transport time of 14 minutes for an excess pressure of 1.0 kg/cm^2 , which is comparable with the 1.09 kg/cm^2 of the scaled model. The closeness of the distorted scale model prediction and the inaccuracy of the porous medium analogy illustrates a most important feature of model interpretation. It is obviously incorrect to apply porous medium porosity concepts to the calculation of transport time through rock masses, when the tracer is timed in a direction which coincides with a particular joint set, in this case near vertical. The wide joint spacing (approximately 2.5 metres for water conducting joints) and the relative smallness of the borehole diameter invalidate the use of a porous medium analogy.

It is interesting to note that the predicted transport times are in a ratio of 3 to 1. This is an exact reflection of the ratio of porosities of a rock mass with a cubic joint system, and a rock mass with only one joint set of the same spacing and opening.

It may be unwise to try to explain the small difference in transport times between that predicted by the distorted scale model (18.2 minutes) and the field result (14.0 minutes), since the times are much closer than theory and practice normally come in this complex area of geotechnical testing. However, the presence of water in the rock mass should be mentioned as a complication. Although the air injection tests that were performed in the field were steady state, so that some form of opened flow path would probably have developed, the surrounding water would undoubtedly have concentrated the "flow lines" (just as "closed" model boundaries do) and thereby probably have reduced the transport time. The theoretical prediction of 18.2 minutes for a dry rock mass might therefore be reduced. Most of the remainder of this paper will be concerned with air flow through a saturated rock mass, which is beyond the scope of present theory.

VI. AIR FLOW FROM A BOREHOLE BELOW GROUNDWATER LEVEL.

The rate of air leakage from a large underground opening such as a compressed air surge chamber is obviously dependent on the position of the groundwater level relative to the chamber elevation and air pressure. Instances of dry, fully drained rock masses must be uncommon and consequently the air leakage problem is one of water displacement and involves all the complications of two phase flow and an unstable interface, since air of low viscosity is displacing water of high viscosity. (Saffman and Taylor (5).)

At present the simplest design method is to assume that during operation the whole rock mass in the vicinity of the chamber and up to the ground surface will become fully drained through displacement of water. The air leakage through the "dry" rock mass can then be estimated from the results of borehole tests using water injected into the originally saturated rock mass, and by converting this water leakage (water/water flow) to air leakage (air/air flow) by allowing for the difference in viscosity. (The ratio of μ water/ μ air is approximately 73/1 at 10°C). This conservative approach is based on the experimental evidence that the intrinsic permeability K (cm^2) is a constant for a particular porous medium and essentially independent of the fluid used. (Muskat (13).) Obviously in practice there are complications in applying this to fluid flow through rock joints. For air flow the rock mass will never be fully drained, and for water flow never fully saturated. In fact the history of movements of the air/water interface may be important. The interface between air and water moves faster in the wider connected joints than in the finer joints. (De Wiest (16).) In fact the water will be more or less non conducting in the fine joints, although presumably still capable of transmitting pressure.

This reduction in effective flow cross section will be further reduced due to the way that fingers of air leave behind a certain thickness of water to either side, particularly if the flow velocity is low. This has been clearly demonstrated on a macroscopic scale by a series of parallel plate experiments of immiscible flow conducted by Saffman and Taylor (5). The effect might be more marked for rock joints having characteristically rough surfaces.

Some of these very complicated detailed effects have been investigated on a small scale by Danielsen (17) and in a continuing study at the Geological Institute, Trondheim. Artificial, interlocking rock joints were created by longitudinal splitting of rock core, and these joints were tested for air and water leakage in a specially constructed pressure cell. Attempts were made to compare air flow through dry joints with water flow through saturated joints and the important intermediate case of air flow through partially wet joints. The difficulty of assessing the degree of wetness (which changes during air flow) and relating it to the unknown field conditions makes application of the results difficult.

In the present study we are going to assume that joint conductivity (K_j , cm^2) is a constant for a rock mass, whether the permeating fluid is water or air. Attention will therefore be directed to the large scale geometrical effects of air displacing water in the vicinity of openings. It is felt that these large scale effects are of most importance to the steady state leakage of air, although the detailed study of flow through partially saturated joints will have relevance to the initial leakage rates when an air flow path is first being established.

A) Model test results.

Figure 13 shows three typical photographs of air transport through the water filled capillary space for a variety of excess air pressures. The exact value of excess pressure operating during each test was unknown as the air/water interfaces moved very rapidly and greatly disturbed the water surface

particularly at high pressures. Consequently the only way of estimating the excess pressure was to subtract the head of static water above the borehole (after completion of a test) from the total air pressure recorded by the manometer. Fluctuations of groundwater level in response to air injection were also noted during the field tests in Norway (DiBiagio and Myrvoll (10).) but on a much slower time scale.

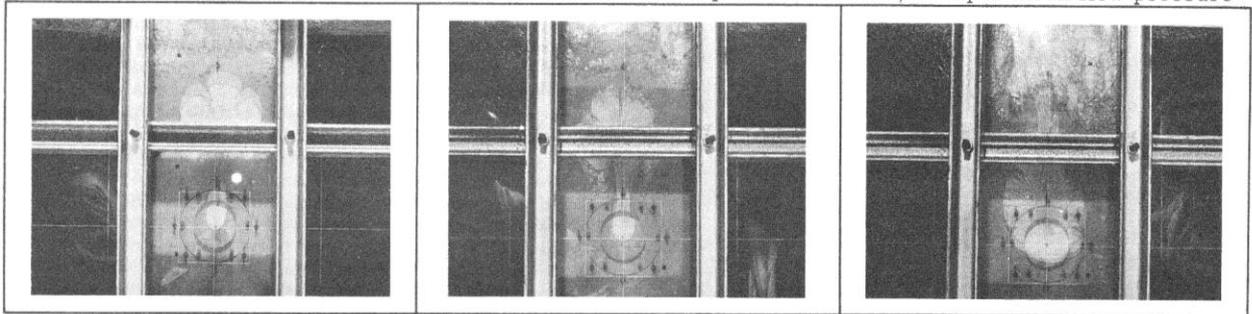


Figure 13. Characteristics of air transport from model borehole towards a water surface, under excess injection pressures of 0.6, 1.1 and 103 cm of water.

The transport of bubbles which was observed at low excess pressures is unlikely to be correctly scaled between the model and field due to surface tension differences and also greater compressibility of air at the much higher field pressures. The bubbling development is an oscillating one: air pressure builds up until the surface tension effects at the borehole entry are exceeded. The escape of a certain volume of air as a bubble causes a small reduction in pressure in the supply line, and the next bubble will not escape until the pressure again rises. These small pressure oscillations were recorded in the flow metres, where oscillations of a few percent were observed. Similar effects were recorded in the field tests at low injection pressures. (Ref. 10). At higher model pressures the air/water interface shapes were probably quite representative of the mean interface separating air and water in the field. The air flow rate is mostly controlled by the distance to which water is displaced away from the borehole, since it is close to the borehole that most of the pressure is dissipated due to the high velocity of flow. At the highest excess air pressure (103 cm of water) the water was displaced a distance of up to 18 diameters below the model borehole and 25 diameters to each side. At a pressure of 11 cms of water the corresponding distances were only 5 and 9 diameters respectively.

Following this series of air/water tests a comparative sequence of air/air tests were performed, using the same plate spacing and injecting air into the dry capillary space. The air flow temperature was measured during both classes of test and a careful check was made on the plate spacing. No temperature correction proved necessary, but a small correction to flow rate was made to allow for the slight increase in capillary spacing when the space was water filled for the air/water tests. The correction was based on the cubic relationship between capillary spacing and flow rate, which is illustrated by equation 4. Comparison of flow rates for the same air pressure revealed that the presence of water reduced the air flow by only about 15 to 20% at high pressure.

Figure 14 shows the flow rate/pressure relationships for the two classes of test. The curvature of the two plots is a reflection of the onset of turbulent flow. However, the additional pressure losses induced by turbulence will be almost the same at equal flow rates, and so the results can be compared. (Even in the laminar range a plot of flow rate versus pressure is curved for air flow, unless the flow rate is expressed in litres/min. per mean flow pressure

(Muskat (13).) in place of the conventional normal litres/min.)

Figure 15 shows the air/water flow expressed as a percentage of the air/air flow rate. At low pressures when the water is hardly displaced from the borehole the reduction in flow rate is obviously large. However, at high air pressures there is only a 14% reduction in flow rate due to the presence of water. The zone within 20 diameters of a borehole pumping test is obviously all important, no matter how the air is conducted outside this zone.

B) Comparison with field tests.

The air injection field tests, reported by DiBiagio (11) have been referred to earlier in connection with transport times. The zones chosen for air injection tests were individually water tested before conducting the air tests. The heavy lines marked (W) shown in Figure 16 are the results of the water tests that were performed in zones 53.0-56.0 and 38.4-40.4 m over the pressure range 0 to 2.5 kg/cm² (Most of the water test data (10) is for higher pressures, but it has been assumed here that a linear extrapolation to zero pressure is justified).

The lines marked (T) were obtained from the above by first multiplying the water flow rates by 80, which is the ratio of viscosities for water and air at a temperature of 8°C, representing field conditions. The air flow rate obtained is in litres/min. and is the theoretical flow rate at the mean flow pressure for air flowing through the same rock mass when dry (Muskat (13).) This flow rate was corrected to normal litres to give the lines marked (T) in Figure 16.

The dotted lines marked (A) are the results of two series of air injection tests that were performed at different times during the summer of 1971. It is interesting to see that these lines lie a little below the theoretical air/air lines. These field results therefore compare in broad terms with the model results. For some reason the air injection tests

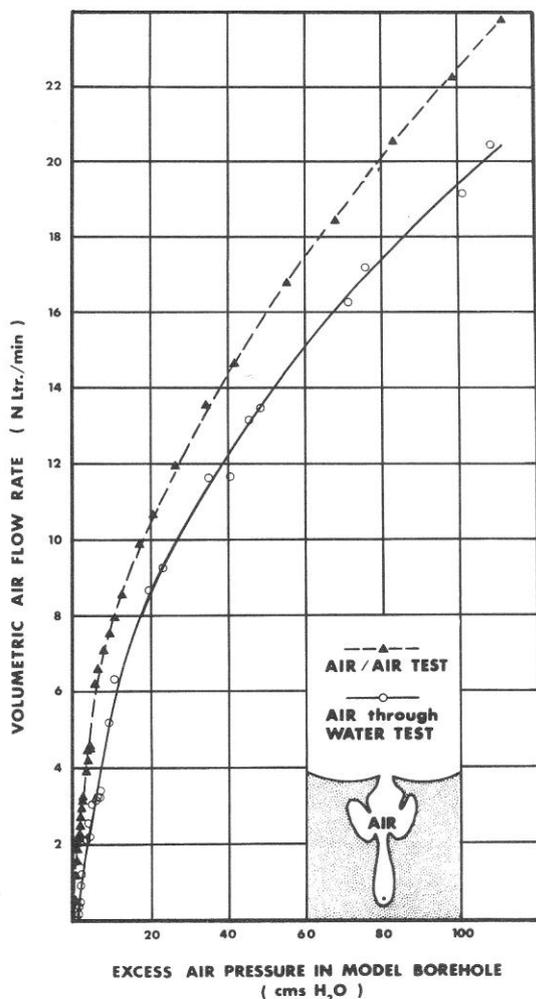


Figure 14. Flow rate-pressure relationships for air/air test and air through water test showing the onset of turbulent flow at pressures above 10 cm of water.

performed in the zone between 42.6 and 45.6 metres showed flow rates for air through water which were considerably higher than the theoretical air/air flow rates. Blowing out of joint in-fillings might be one of several possible reasons for this anomalous result.

The remainder of this paper will be concerned with air leakage from large underground openings, in which field there is unfortunately no data available for comparison with the model results. For these tests the model borehole was increased in diameter to 3.0 cm, and increased model scales were used to convert this to a realistic prototype dimension.

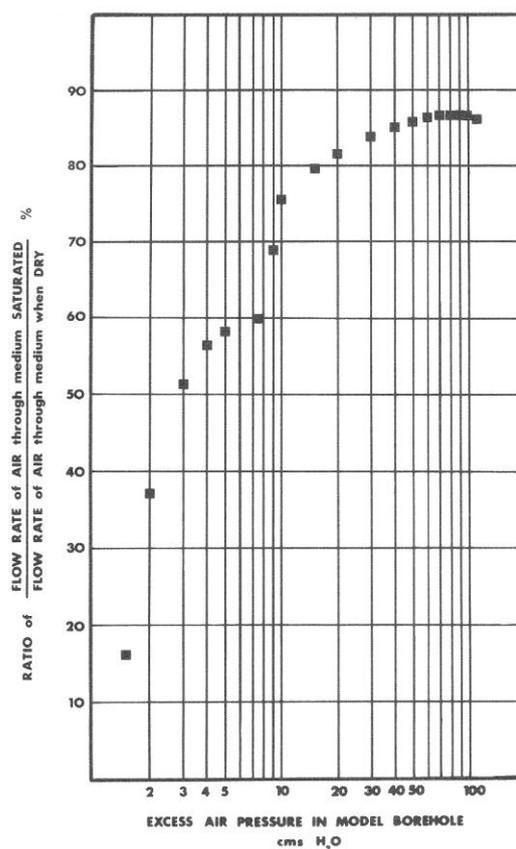


Figure 15. Percentage reduction of air flow rate due to the presence of water.

VII. DRAWDOWN OF GROUNDWATER LEVEL ABOVE NEAR SURFACE OPENINGS.

Large underground openings located near to the ground surface, such as those that may be used for housing nuclear power plants, require a relatively impermeable roof of rock to control both the leakage rate and transport time of any possible radioactive gases that might be released into the openings in case of accidents. Blow out pressures in the region of 2 atmospheres have been considered in this context.

The presence of a ground water surface above these openings obviously improves their safety enormously. A ground water level only 20 metres above the roof of the opening would be sufficient to contain the above gas pressure. Locating the roof of the opening at a depth of 40 metres below surface in a rock mass which was more or less saturated up to the surface would seem to remove the possibility of any leakage of radioactive gases.

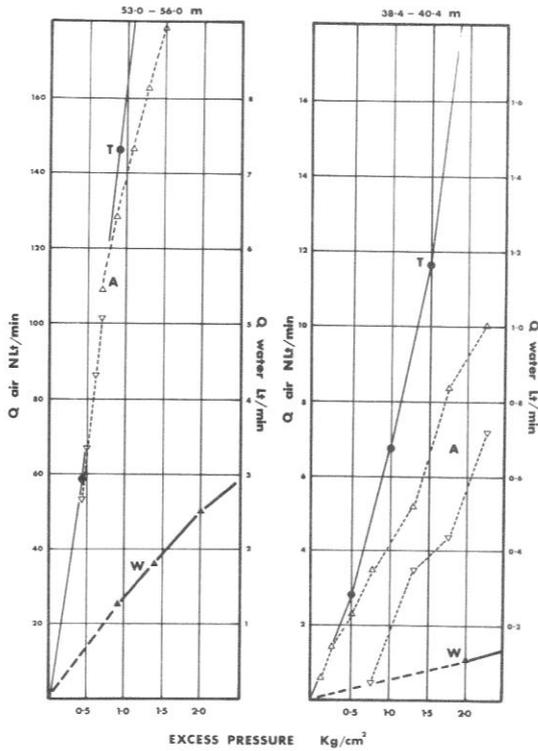


Figure 16. Comparison of theoretical (T) air/air flow rates, which are obtained from the water/water (W) tests, with the field measurements of air/water flow (A) which were reported by DiBiagio and Myrvoll (10).

However, unless the near surface openings are lined to prevent any groundwater leaving the rock mass there will be a significant draw down of the ground water above the openings. In a regularly jointed rock mass the problem is a purely geometric one. An unlined opening of a certain diameter located a certain distance below the water table will cause a given amount of draw down above its roof, no matter if the rock is extremely impermeable or permeable. A very impermeable rock mass will simply produce less inflow of water. The amount of draw down will remain constant provided there is constant replenishment of water at a distance from the opening. If there is little replenishment then obviously the ground water level in the whole area will be gradually lowered.

Figure 17 shows the 3.0 cm diameter model opening that was bored in place of the model borehole. When opened to atmospheric pressure to allow drainage, an undisturbed water surface located 1.66 diameters above the centre of the opening will just be drawn down to the roof of the opening. The "undisturbed" water level was measured at the extremities of the model (a distance of 20 diameters) during steady state draw down.

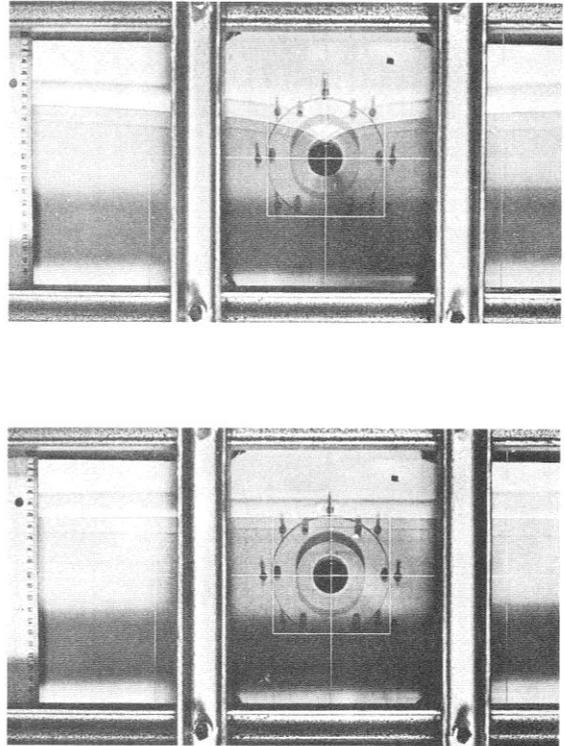


Figure 17. Level of undisturbed groundwater that is just drawn down to the roof of an unlined circular opening located in an isotropic permeable medium.

A series of draw down tests were performed using a special arrangement in the boundaries, which supplied water from each side and controlled the level of the "undisturbed" water surface. Two of these tests are shown in Figure 18. The results of the whole series are presented in dimensionless form in Figure 19.

As an example we will use the results to estimate what depth an idealized circular opening needs to be located below the undisturbed ground water level for the drawn-down level to just apply a 2 atmospheres water pressure at the roof of the opening. It is assumed that the rock mass (or single vertical major joint) has isotropic permeability, and that the opening is open to atmospheric pressure.

$$\begin{aligned} \text{Assume } d &= 30 \text{ metres} & h/d &= 1.167 \\ & & h &= 35 \text{ metres} \end{aligned}$$

Figure 19 shows $H/d = 2.06$, $H = 61.8$ metres

These results suggest that the roof of the opening will need to be 46.8 metres below the undisturbed ground water level, for a sufficient head of water to be left above the roof to prevent the gas leakage referred to earlier.

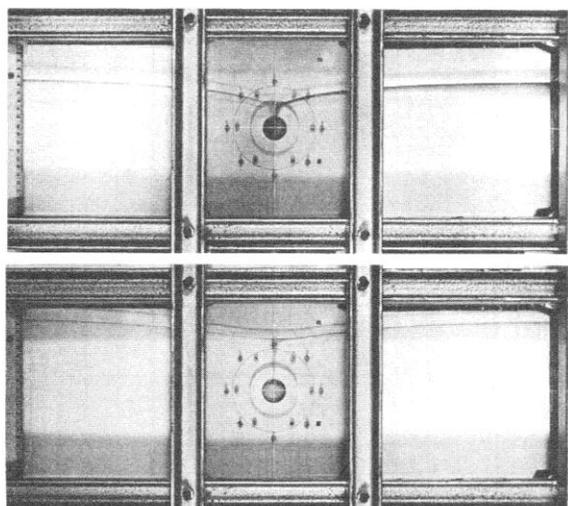


Figure 18. Two typical steady state draw down tests in which the undisturbed water level was too high to allow complete drawdown above the opening.

Obviously in practice there are additional complications due to reverse capillary forces, drawing water into the air filled opening due to the interface effect. As can be seen from the photographs the model opening remained full of water during the tests. There were thus no such additional capillary problems in the model.

There is no exact mathematical solution which can be used to replace experimental results such as those presented. Just as for conventional draw-down problems with vertical wells, solutions can only be obtained by ignoring the phreatic curvature and vertical flow components as in the Dupuit-Forcheimer well discharge formula. (Bear et al. Reference (3), Ch. 12).

The same problem has been considered by Wittke and Louis (18) who presented some equations for flow rate to a horizontal underground opening in rock. However, their formulation did not include the dimensions of the opening. This was accounted for in the radius (R) representing the distance from the opening of undisturbed ground water conditions. This was taken as 100 metres in one numerical example, which is probably a serious underestimate. In view of the fact that (R) does not enter the equation in logarithmic form, large errors are possible. Admittedly the experimental results presented in Figure 19 must be a few per cent inaccurate due to the limited distance of the model boundaries which should theoretically be infinitely far away. However, the results are based on opening dimensions and water levels and consequently no guesswork is involved.

VIII. AIR LEAKAGE FROM OPENINGS BELOW GROUND WATER LEVEL.

In the event that the ground water level is drawn down excessively, or that the opening is insufficiently deep compared to the water table, the

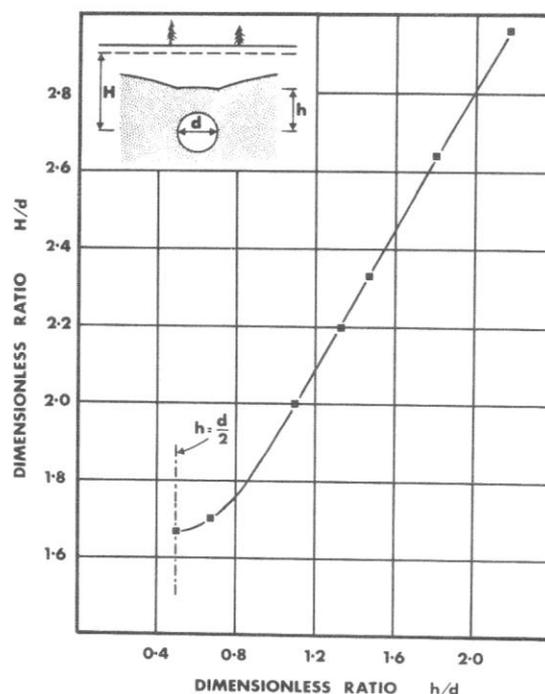


Figure 19. Dimensionless plot of model drawdown tests for estimating the head of water above the roof of a near surface opening.

radioactive gases will probably blow a clear path to the surface by displacing the water from the vicinity of the opening. Thus in the hypothetical case where precautions are not taken against gas leakage, considerable flows may reach the surface.

The series of tests to be described in this section were designed to compare these gas flow rates (air was used here) with those that would occur if the medium was quite dry. This method is the same as that used earlier in presenting the results of the model borehole tests. The information is utilized in field problems by following the flow chart shown in Figure 2.

Unfortunately it was not possible to combine draw down and subsequent air leakage in the same model test, as the model response time was so small. Consequently the air leakage tests were performed with an initially horizontal, undisturbed water surface. At the end of each test the height of the static water level above the centre of the opening was recorded. This head was subtracted from the total air pressure head to obtain an estimate of excess air pressure.

Figure 20 shows one typical result of these water displacement tests. The whole series of tests has been described in more detail elsewhere. (Barton (19).) It will be noticed from the photograph that close to the opening the water was blown up above the local displaced water level. However, this was only a localized effect. The general tendency was for the water to be piled up towards the boundaries, rather as if a severe draw down test was being conducted. This emphasises the problem of

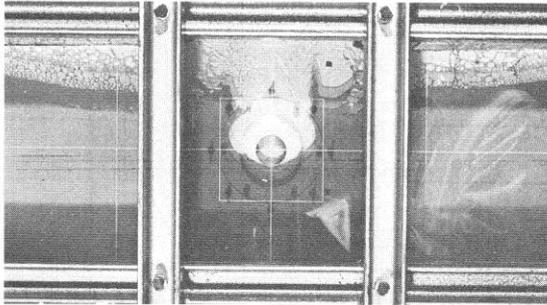


Figure 20. Water displacement resulting from an excess air pressure equal to 4.0 cm of water. Static water level 2.7 diameters (8.2 cm) above centre of opening.

estimating the relevant excess air pressure operating during any particular test.

Figure 21 shows the flow rate-pressure relationships for three different model ground water levels, together with the results of the air/air tests. The three series of tests can be converted to hypothetical field scales by selecting a realistic diameter for the full scale openings (say 30 metres) and using this same geometric scale (30m/3.0 cm = 1000/1) to convert the air pressures, which were expressed as equivalent heads of water. Thus taking the test shown in Figure 20 as an example, the model air pressure becomes 4.0 atmospheres at full scale. The centre of the opening is 2.7 diameters below the static mean water level, which can alternatively be expressed as a depth of 82 metres.

The reason for the leakage from the opening not being closer to the flow rate through the medium when dry is that the water is displaced below the opening little more than one diameter at the highest air pressures. By comparison the model borehole tests showed that water was displaced approximately 14 borehole diameters below the hole when at a comparable air pressure. As indicated earlier the permeability of the zone close

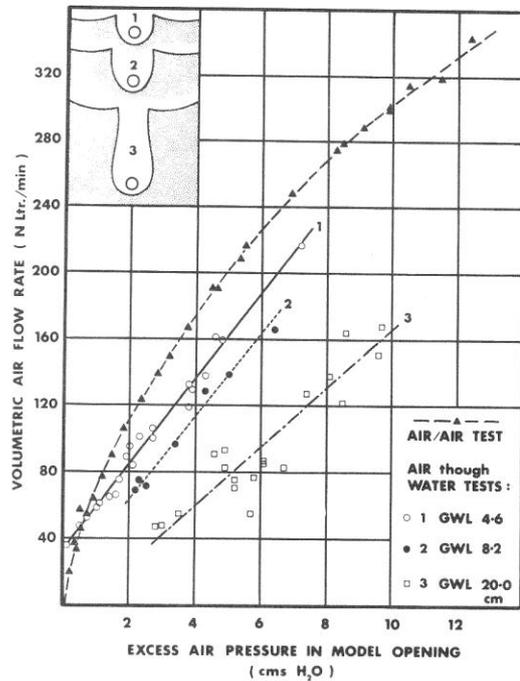


Figure 21. Flow rate-pressure relationships for water displacement tests using three different water levels.

to a borehole almost completely determines the overall flow characteristics. The effect is less marked for flow from a large opening due to the less severe pressure gradients.

Figure 22 shows the percentage reduction in flow for all three depths below the water surface. The results for the model opening should be compared with those given in Figure 15, for the model borehole tests. The table below shows that the predicted results of leakage from borehole tests are surprisingly similar to those of a shallow opening.

QUANTITY	MODEL		SCALES		FIELD PREDICTIONS	
	Borehole	Opening	Borehole	Opening	Borehole	Opening
Diameter	0.20 cm	3.0 cm	100	1000	20 cm	30 m
Depth below water level	30 cm	4.6 cm	100	1000	30 m	46 m
Excess air pressure	(i) 20 cm (ii) 60 cm	2.0 cm 6.0 cm	100	1000	2 kg/cm ² 6 kg/cm ²	2 kg/cm ² 6 kg/cm ²
Reduction of flow due to presence of water	(i) 18.5% (ii) 14%	23.5% 17.5%	1/1	1/1	18.5% 14%	23.5% 17.5%

Table IV. Comparison of predicted air leakage reduction for a borehole and large opening in the field.

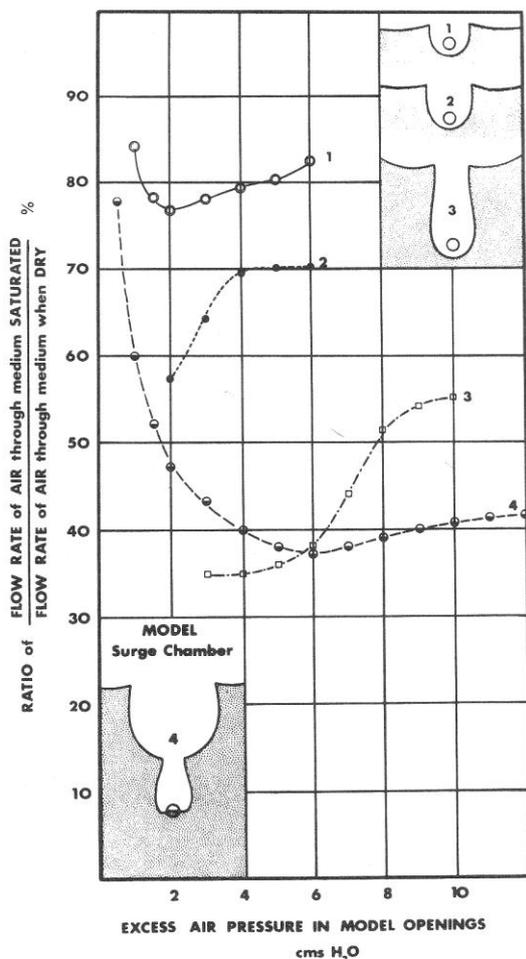


Figure 22. Comparison of percentage reductions in air flow rate due to the presence of water.

IX. AIR LEAKAGE FROM COMPRESSED AIR SURGE CHAMBERS.

This problem was discussed in the introduction and is illustrated in Figure 1 (A). Unlike in the previous problem where air displaced water and leaked from all parts of the opening, here we are concerned with air leakage only from the upper half of the opening. The compressed air is pressurized by the total head of water between the air/water interface and the top reservoir, therefore both water and air will in this case leak from the opening. Both the air and the water are at the same elevated pressure compared to the surrounding hydrostatic joint water pressure.

In practice there are additional complications since the surge chamber is normally located close to the main water pressure tunnel. The hydrostatic joint water pressure is therefore elevated locally, within the logarithmic pressure decay zone surrounding the pressure tunnel. This is obviously an important method which can be used to reduce the air leakage

rate. A long chamber of small diameter located parallel and close to the main pressure tunnel will reduce the pressure differential appreciably.

However in the present model study such complications were ignored. The hydrostatic joint water pressure was modelled by filling the capillary space with water. This hydrostatic distribution was only distorted by the $\frac{1}{2}$ air/ $\frac{1}{2}$ water flow from the single model surge chamber. More complicated tests using two openings, one water filled to represent the pressure tunnel, and the other filled with half air and half water, could be performed at a later date.

After preliminary trials and modifications to the water supply pipes it proved quite easy to keep the air/water interface close to the centre of the chamber. Figure 23 shows the interface a little above the centre. A small increase in air pressure (and therefore flow rate) was needed to press the water level down and maintain it at the centre. The air supply to the upper half of the valve chamber was measured for pressure, temperature and flow rate when the interface was stabilized at the mid height of the chamber.

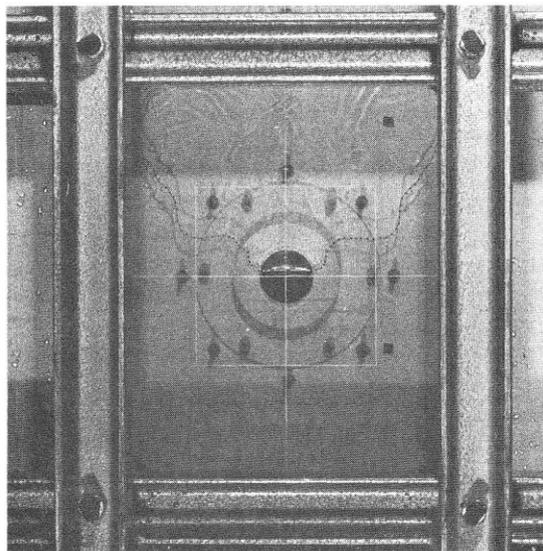


Figure 23. Model compressed air surge chamber showing the air/water interface slightly above the mid-height of the opening.

Figure 24 shows one of the tests in operation. The whole series of tests were run at a range of excess pressures of 0.6 to 11.6 cm of water. If the 3.0 cm diameter model chamber is converted to a full scale of 15 metres using a geometric scale of 1/500, this range of model pressures represents 0.3 to 5.8 atmospheres. The mean model depth below the water surface of 29.5 cm converts to 147.5 metres at full scale. The two compressed air chambers which have been constructed in Norway are both situated at greater depths below surface. However, pre-construction joint water pressure measurements at Jukla (DiBiagio (11).) indicated an effective head in the region of 150 metres, as modelled.

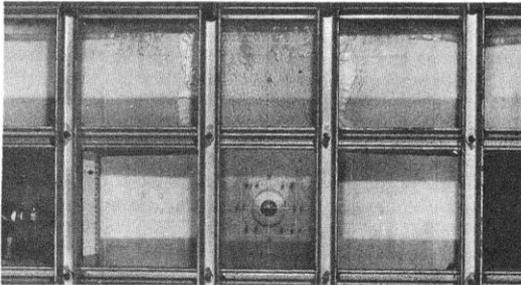


Figure 24. Model compressed air surge chamber operating at an interface pressure equal to 7.5 cm of water.

Figure 25 shows the results of all the surge chamber tests, with the air leakage rates again compared with the air/air tests. The ratio of the air flow rates between saturated and dry tests were given in the collective plot of Figure 22. Despite the fact that air can only displace the water around the upper half of the opening in the present tests (curve No. 4) the results are seen to be quite similar to those in which air flow occurred from the whole opening (curve No. 3). The reason for this is that displacement of water from below the opening (test No. 3) is partly wasted potential since the air has to flow upwards eventually.

Prediction of chamber leakage from the results of borehole field tests.

In a previous report (Barton (19).) two hypothetical but realistic examples were given to illustrate how the results of water leakage tests (from boreholes) could be utilized in obtaining an approximate estimate of air leakage rate from a compressed air surge chamber located in the same rock mass.

Firstly it was assumed that the dimensions of the hypothetical surge chamber (15 metres diameter and 50 metres long) were large enough for the rock mass to be treated in approximate terms as an isotropic permeable medium. A Lugeon value of 0.01 (equivalent to a mass permeability (K_m) of $1.3 \times 10^{-12} \text{ cm}^2$) was used to arrive at an estimate of air-leakage from the surge chamber. Secondly, it was assumed that instead of an isotropic permeable rock mass, there was just one major vertical joint which intersected the borehole perpendicularly, and later intersected the long axis of the surge chamber which was to be excavated along the borehole axis.

The air leakage rate predicted for the 50 metres length of surge chamber when the interface pressure was 10 kg/cm^2 above the local ground water pressure, was 114 normal litres per minute, when based on the water leakage of 0.01 Lugeon. For simplicity no account was taken of hemispherical end effects.

To make the second example comparable to the first it was assumed that the mean opening of the single major joint was equal to the sum of the openings of all the joints intersected in the 50 metres length of

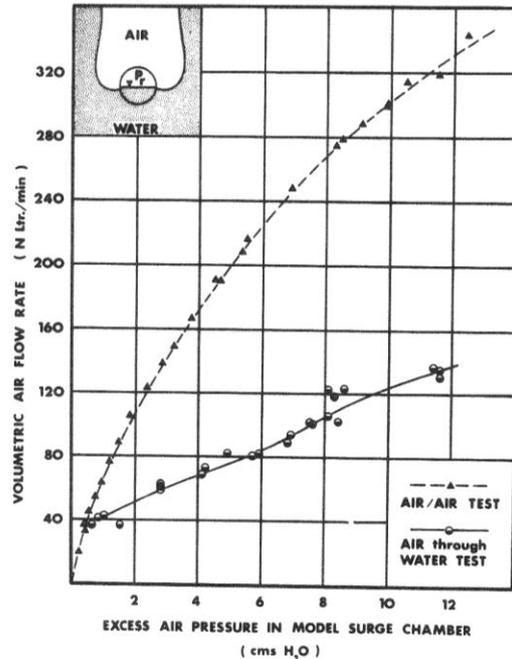


Figure 25. Flow rate-pressure relationships for a series of model surge chamber tests.

borehole in the previous example. The statistical method of analysing borehole test results developed by Snow (9) was used to estimate the spacing and opening of the equivalent cubic arrangement of water conducting joints which had the same overall permeability (0.01 Lugeon) as that of the tested rock mass.

It was assumed that 40% of the 3 metre long test stages showed zero flow at the test pressure of 10 kg/cm^2 . The average number of joint intersections per test length is 0.9, when the percentage of stages showing zero water leakage is 40%. The equivalent joint spacing is therefore equal to 3.34 metres. The porosity of the equivalent cubic rock structure representing the water conducting joints is equal to 0.0000124, and the equivalent joint opening (e) is equal to 0.00138 cm (13.8μ). Fifteen joints of 0.00138 cm opening per 50 metres length of borehole give a total opening of 0.0209 cm (209μ), which was taken as the mean opening of the single major joint.

The air leakage rate from the surge chamber was in this case estimated to be 13300 normal litres per minute. It is obvious that such a leakage rate would be intolerable compared to the leakage rate obtained in the case of the uniformly jointed example. Individual treatment of major joints would be required, which would fortunately be much cheaper than attempting to artificially line all the upper half of the chamber.

CONCLUDING REMARKS.

The detection of the major or minor joints which are likely to have important conducting properties will normally be facilitated by the inward seepages which can often be observed from several points round the exposed joints. The fact that seepage often occurs from only a few points instead of from right round the exposure is a complication, and suggests that in some cases where joints are very tight, conduction is likely to be occurring along meandering channels within the joint plane. This type of conduction was discussed by Sabarly et al. (20). It has actually been demonstrated by Maini (7), who showed dye traces being deflected during flow through transparent replicas of rough rock joints, which were moulded with clear epoxy resins. Stagnant or dead water areas seem to occur in "closed" joints, since stress transfer may occur at many points when the normal stresses are sufficiently high.

A further complication in the interpretation of flow through individual joints in rock is the additional resistance to viscous flow caused by the wall roughness. The theoretical smooth wall opening (e) assumed to represent the rock joint is approximately correct from the point of view of conducting properties but is physically an underestimate of the real opening. It should be much easier to visually detect fine fissures than indicated by the extremely small equivalent smooth wall openings.

The results of the joint flow experiments performed by Danielsen (17) were analysed so that true openings (relative to the closed position) could be compared with the theoretical smooth wall openings. His experiments with true openings of 0.02, 0.01 and 0.005 cm (200, 100 and 50 μ respectively) gave flow rates equivalent to smooth wall openings of only 100, 27 and 8 μ respectively. These fresh, unweathered tension fractures could be completely closed, unlike the weathered joint in porphyry which was studied by Sharp (21). This natural, slightly weathered joint did not have asperities which completely interlocked with each other. Consequently when "closed" it still conducted water. Sharp found that the equivalent smooth wall opening of the joint was as much as 0.039 cm (390 μ). When this "closed" value was added to his increments of opening (0.025, 0.050 and 0.125 cm) the combined opening gave close agreement with the theoretical smooth wall opening, obtained from the flow tests. The results plotted in Figure 26 show the ratio of actual opening and equivalent smooth wall opening for a range of smooth wall openings from 10 to 1000 μ . The three circular points were derived from Danielsen's experiments (17), and the three squares from Sharp's results (21).

This underestimate of true opening will be most marked in cases where the joints are very tight and rough walled. In more planar, weathered and more open joints the smooth wall opening may be a good estimate of both conductivity and true opening. In such cases the development of non linear laminar and turbulent flow at small gradients, as noted by Sharp (21), will be an additional complication in the interpretation of

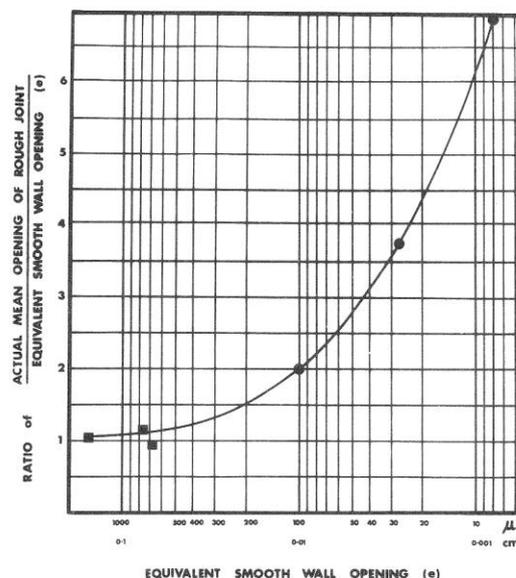


Figure 26. The ratio of actual joint openings and their equivalent smooth wall openings, both having equal conducting properties.

all the foregoing results of air leakage. However the total joint openings studied by Sharp (0.063, 0.089, 0.165 cm) are very large in comparison to most Norwegian experiences (excepting clay filled joints). It is for this very reason that operation of unlined compressed air surge chambers is considered feasible.

Consequently the current emphasis on the early onset of non linear flow (and the resulting breakdown of our linear Darcy assumptions) is unlikely to be relevant to the present problems, where joints are generally tight, and gradients are limited due to the size of the openings considered.

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