SECTION X

GROUNDWATER PROBLEMS

GENERAL REPORT

A.E. BRETTING (Denmark)

X a. GENERAL GROUNDWATER INVESTIGATIONS. (9 papers)

H. CAMBEFORT in his paper: "Les Puits filtrants et la formule de Dupuit" reports experiments with ground water lowering to study the steadiness of the flow and especially to find the critical output of the well.

The piezometric heights are measured electrically and could be found accurately with an error of abt. 2 mm. The lowerings are plotted on logarithmic paper, whereby an approximate value of the effective range of the well is found.

The conclusion is that the formula of Dupuit will in general be well suited for the calculations.

R. GLOSSOP and V.H. COLLINGRIDGE in their paper: "Notes on Ground Water Lowering by Means of Filter Wells" and

H.J.B. HARDING in his paper: "Some recorded observations of Ground Water Levels related to adjacent Tidal Movements" report observations. The formula of Sichardt for the critical slope is confirmed.

H. CAMBEFORT in his paper: "L'écoulement des liquides à travers les millieux pulvérulents" reports on an interesting series of permeability-tests executed with 5 different soils. He finds as usual that Darcy's law has an upper limit, when turbulence occurs, but also a lower limit, which is practically not of importance.

The last fact seems however to be of considerable theoretical interest. It is found that the coefficient of permeability is decreasing considerably for very small velocities. No explanation is given, but similar results have earlier been found by Zunker.

The reporter propose the following explanation: It is supposed that the grains are covered with a thin sheet of hygroscopic water, which is bound very hard to the grains near the surface but more loosely in the outer part of the sheet. By very small velocities the free volume of voids is considerably restricted by the molecules bound to the grain, but the outmost of these are loosened by greater velocity and thus the free path of the stream is increased.

Zunker found that when the grains were cleaned with chemicals and the absorbed hygroscopic water relieved the permeability was constant until zero velocity.

Cambefort has further tried to express the complete hydraulic similarity by using the usual effective diameter of the grain ($<$ 10% limit). This seems not to be quite satisfying. A better accordance is found by adjusting the distribution of grain size according to the law of error (Gibrat's adjustment).

SHIN TE YANG in his paper: "On the Permeability of Homogeneous Anisotropic Soils" gives a theoretical discussion of the coefficient of permeability as a function of the direction of the flow. A representation similar to Mohr circle results.

JEAN FERRANDON in his paper: "Sur les lois de l'écoulement de filtration" gives a theorêtical treatment of the question of permeability based on tensors and matrices and valid for heterogenous and anisotropic material and for triaxial flow.

ALFRED BASCH in his paper: "Geometric Rules Governing Subsoil Water Flow" gives a purely theoretical study of the geometrical properties of the flow net, which seems to be useful by the elaboration of such nets.

M. SITZ in his paper: "On the Flow of Groundwater" discusses the capillary phenomenon and groundwater movement. Most of the conclusions seem rather selfevident, and it should hardly nowadays be necessary to discuss the difference between equipotential and equipressural lines.

M. SITZ in his second paper: "On the Flow of Groundwater" seems to mean that no tension exists in the capillary rise; he will hardly find many to participate in this opinion.

X b. SEEPAGE PROBLEMS OF DAMS AND LEVEES. (4 papers)

ZDENEK BAZANT jr. in his paper: "Critical Head for the expansion of Sand on the Downstream Side of Weirs" reports a series of model tests concerning piping. Diagrams are given for the relation between head and the proportion between depth of and width of foundation. Further the relation between the heads, which gave the beginning of expansion and complete piping,are studied. These ratios varied between 0,4 and 1,0.

R.A. BARRON in his paper: "The Effect of a slightly pervious top Blanket on the Performance of Relief Wells" reports model tests showing the effect of such a blanket in combination with levees. Both slightly pervious and completely unpervious top blankets are treated.

HARRY CEDERGREN in his paper: "Use of Flow Net in Earth Dam and Levee Design" gives a general review of the flow net method and states that this method also can be used to,get approximate solutions to certain non-steady seepage problems.

In this connection the reporter wishes to draw the attention to a newly published work by Erling Reinius: "On the Stability of the Upstream Slope of Earth Dams", Stockholm 1948.

C.I. MANSUR and W.R. PERRET in their paper: "Efficacy of Partial Cutoffs for Controlling Underseepage beneath Dams and Levees constructed on Pervious Foundations" report on model tests.

Theoretical solutions were only possible in a few cases. Graphical flow nets were used in other cases. Sand model tests were made for homogenous foundation. Electrical models for cases with two different strata.

The conclusions are that partial cutoffs with penetrations less than 98% had relatively little effect. Penetrations less than 25% gave practically no effect.

Good correlation was obtained between results from different methods.

In practice the cutoffs must not only penetrate the pervious strata, but it must itself be tight.

Synopsis Section X.

Several papers report on practical observations of lowering of groundwater. Darcy's law is confirmed, and the critical capacity of the well is studied. Model tests of permeability are reported, and it is found that Darcy's law has a lower limit as well as an upper one. An attempt to express complete hydraulic similarity by adjusting the distribution of grain-size to mathematical laws is made.

Theoretical studies of permeability of nonisotropic and for heterogeneous material are made.

Geometrical properties of flow are studied.

Discussions of capillary action and general principles of groundwater movement seem to indicate that much uncertainly exists even today.

Model tests on piping below dams and seepage through the foundation of levees are reported.

General review of flow-net method is given and it is stated that this method can be used even for non-steady flow.

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SUB-SECTION X a

GENERAL GROUNDWATER INVESTIGATIONS

X a 11

DISCUSSION

R. MALCOR (France)

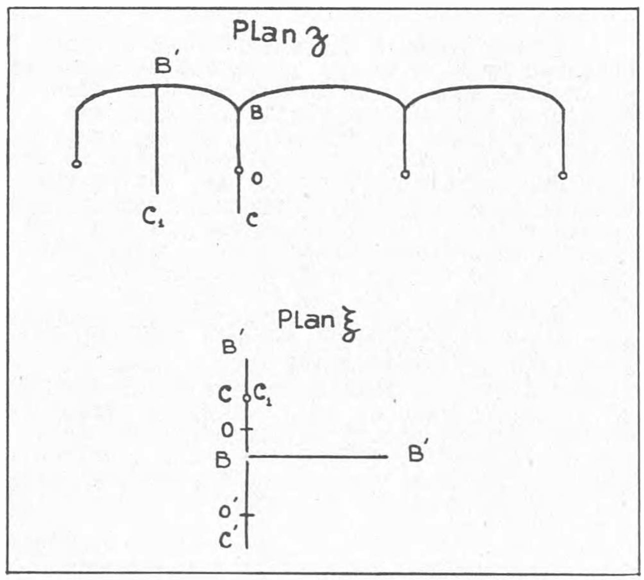
The problem I studied is the problem of the flow of water through porous mediums with a free surface. This is one of the last problems that could be solved with a sheet of paper and a pencil as Mr. Terzaghi said yesterday. The solution I gave is based on the consideration of the hodograph plan V or better the inverse of the hodograph plan ξ $\xi = \frac{1}{V}$

In using this way of studying, the free surface is transformed in a straight line, for instance, for this flow here towards a ditch, (see fig. I and Ia in the communication) the free surface becomes the straight line B B' and the vertical side of the ditch a semi circle A B whereas the horizontal ground A A' becomes a straight line. I have completely developped the solution of this problem, and I have also prepared the solution of the problem, where the side of the ditch is a slope or an overarching slope. I also developped the solution of the problem, where the impervious line is vertical instead of being horizontal (problem of the infinite depth of sheet piling) which leads me to the solution of a flow towards a very narrow ditch in an indefinite pervious medium.

These special problems are studied in my paper which is not until now published but will, I hope, be published in the 6th volume of the proceedings. I was since able to make also a study of the problem of drainage which looks rather complicated. That is the problem of the flow with a free surface towards an indefinite row of drains equally distanced in an indefinite pervious medium.

The figure is roughly like this and the solution is more simple than the first problem (problem of the report). In that case the function ω is (see notation in the report)

$$\omega = c\,L\left(\frac{\xi_1^2 + \frac{1}{K^2}}{\xi_1^2 + \frac{1}{V^2}}\right)$$

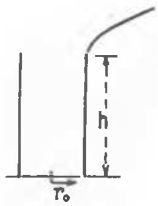


$$\frac{1}{V} = BC = \frac{1}{K} + \frac{1}{U} \qquad U = \frac{1}{OC}$$

Mr. Chairman, if I had two minutes more I would like to state that here in this point B (see figure) there is a mathematical singularity, the figure 1 shows a vertical tangent but the experience shows roughly an angle of 40 degrees instead of a vertical tangent.

In fact there is a vertical tangent but the stream line goes very quickly far from this tangent. In the communication (Ih 1) of Mr. D.N. Allen who studied roughly the same problem by means of the method of relaxation fig. 4 shows an angle too with the vertical side. (Point E of fig. 4 corresponds to our point B)

In case I the vertical tangent cannot be shown experimentally whereas with the ditch side consisting of a slope de VOS was able to demonstrate the tangency experimentally. Now in the cylindrical problem which I have not been able to study completely the tangent should be also vertical but it is interesting to note that in the cylindrical problem you can have very different shapes:



It is certain that when the radius r_o of the well is great compared to its height you have the same angle as on the two dimensional plan, so 40 degrees, but when you have a very narrow well it is probable that the vertical tangent can be demonstrated and this explains probably some remarks that are made by Mr. Cambefort in his paper where he says, that he had noticed sometimes an angle of forty-five degrees. I have to make a last statement. I showed my paper to Mr. Rückli and he told me that these considerations of hodograph have been studied as early as 1942 by a German author 1). I did not know anything about this book and I do not know whether the same problems have been studied by this author and by me.

REFERENCE

1) Breitenöder: Grundwasserströmung mit freie Oberfläche, Springer Berlin.

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X a 12

DISCUSSION

E.C. CHILDS (England)

A very general discussion such as that prepared by M. Sitz (Xa 1) cannot be expected to provide details as to how one sets about analysing and solving particular drainage problems. There is, indeed, a strong implication in that paper that equipotentials in such problems are concentric circles, whilst the example shown is from a hydraulic model. Such models suffer from the disadvantage that one gets an experimental result with negligible analysis and consequently with but little clarification of the basis conditions.

It is proposed to illustrate the general course of solution by reference to a highly idealized Ghyben-Herzberg lens system, as shown in fig. 1. This shows the cross section of a long permeable, vertical-sided "dyke" on an impermeable bed AB submerged in the sea, (of density ρ'), to heights C and D on the respective sides, the height difference being δ. With rainfall of intensity a, fresh water density being ρ, we have a water table EF, surfaces of seepage EC and FD, surfaces CG and DH at which fresh water seeps out against the pressure of sea water, a boundary GH between the fresh and salt ground water zones, (which boundary is a streamline for the steady state), and a zone AGHB in which sea water seeps from one side to the other.

We set up potential functions appropriate to each flow zone, namely

$$\phi = p + g\rho h$$
$$\phi' = p + g\rho' h$$

It is convenient to measure the height h relative to D and the pressure p relative to atmospheric pressure. The boundary conditions are then as set out in Table I, pressure being continuous across any boundary, and the general nature of the solution is at once apparent. The solution of our problem must be that solution of Laplace's equation which satisfies these

Table 1

Surface	p	ϕ or ϕ'	flux (transverse)
EF	0	$g\rho h$	a
EC, FD	0	$g\rho h$	unknown, emerges as part of the solution
DH	$-g\rho' h$	$gh(\rho - \rho')$	unknown
CG	$-g\rho'(h - \delta)$	$gh(\rho - \rho') + g\rho'\delta$	unknown
HB	$-g\rho' h$	0	unknown
GA	$-g\rho'(h - \delta)$	$g\rho'\delta$	unknown
GH	continuous across surface	$\phi - \phi' = gh(\rho - \rho')$	zero (for steady state only)
AB	unknown	unknown	zero (streamline)

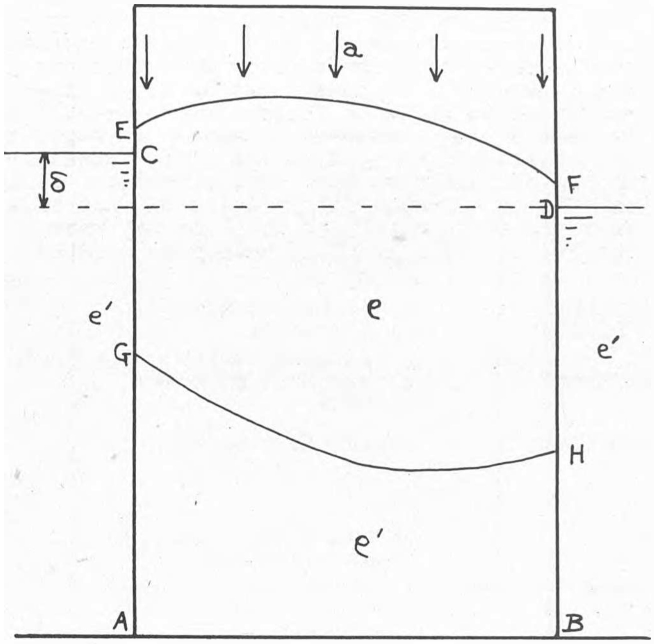

FIG. 1

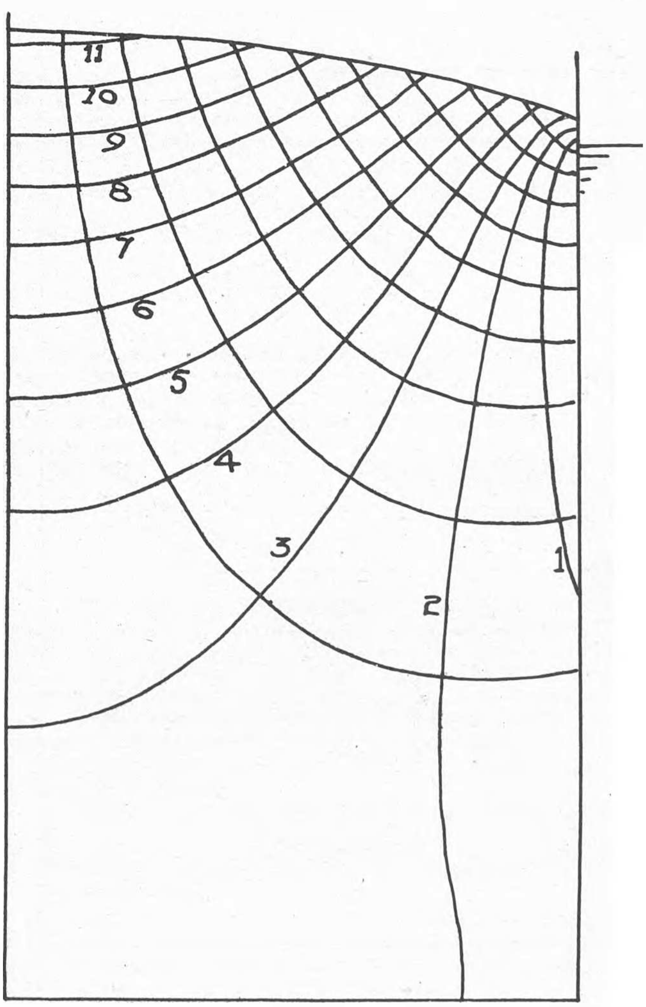

FIG. 2

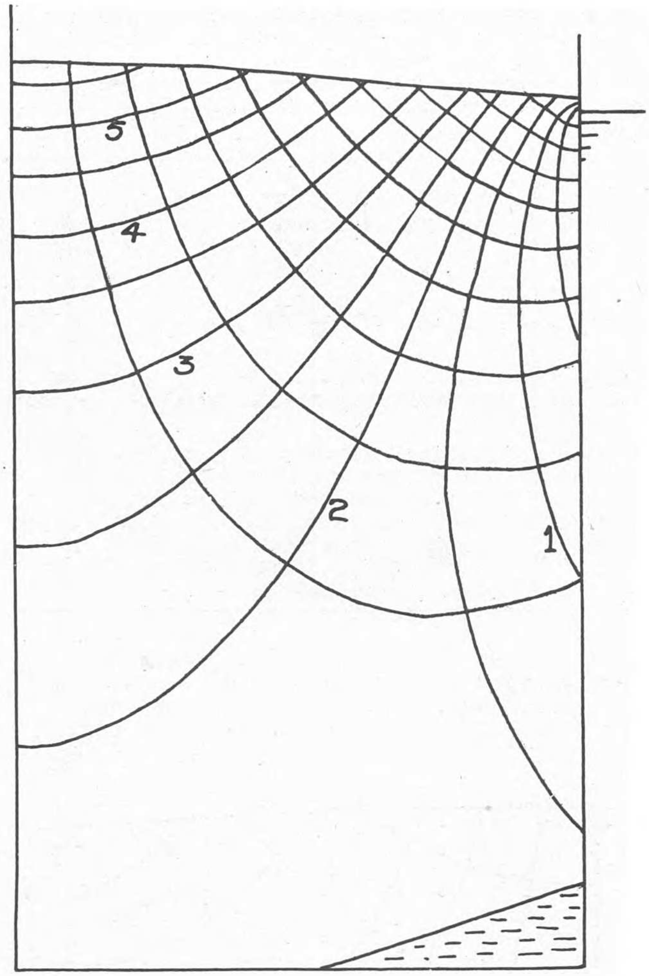

FIG. 3

boundary conditions. Only rarely can one proceed 7), 10) without recourse to some method of trial and error, as, for example, by the method of relaxations 9) or of graphical flow-net construction 1).

We proceed here by the method of electric analogues, which has been described elsewhere in connection with drainage problems, 2), 3), 4), 5), 6) including the solution for the non-steady state 6) as a sequence of steady states. We present here only the simple symmetrical case for $\delta = 0$; a more complete discussion will appear elsewhere. For this case the electric boundary conditions analogous to Table 1 are given in Table 2. and the final solutions in figs. 2 to 6, in which only one half of the symmetrical solution is shown in each case. The scale may be interpreted at will, the equipotentials being arbitrarily evaluated although consistent between all the diagrams. The rainfall rates decrease from fig. 2 to fig. 6 in the proportion 1.00 : 0.52 : 0.22 : 0.09 : 0.02. As an example of interpretation, if the "dyke" is 200 m wide, submerged to a height of 150 m in the sea, and if $\rho'/\rho = 1.02$ whilst the permeability is assumed to be 1.0×10^{-7} or about 100 millidarcys, corresponding to a fine sand, then the water table at the mid-point varies from 20 m above sea level for a rainfall rate of about 60 cms per month (fig. 2) to 1 m above sea level for a rainfall rate of about 1 cm per month (fig. 6); in fig. 2 the sea water is excluded form the

Table 2

Boundary section	potential	current input	remarks
EF	αh	uniformly distributed	α fixed by selecting one point on EF arbitrarily EF found by trial and error
EC, FD	αh	uncontrolled	Potential imposed by electrode
DH	βh	uncontrolled	$\beta/\alpha = (\rho - \rho')/\rho$ potential imposed
CG	βh	uncontrolled	potential imposed
GH	βh	zero	GH found by trial and error

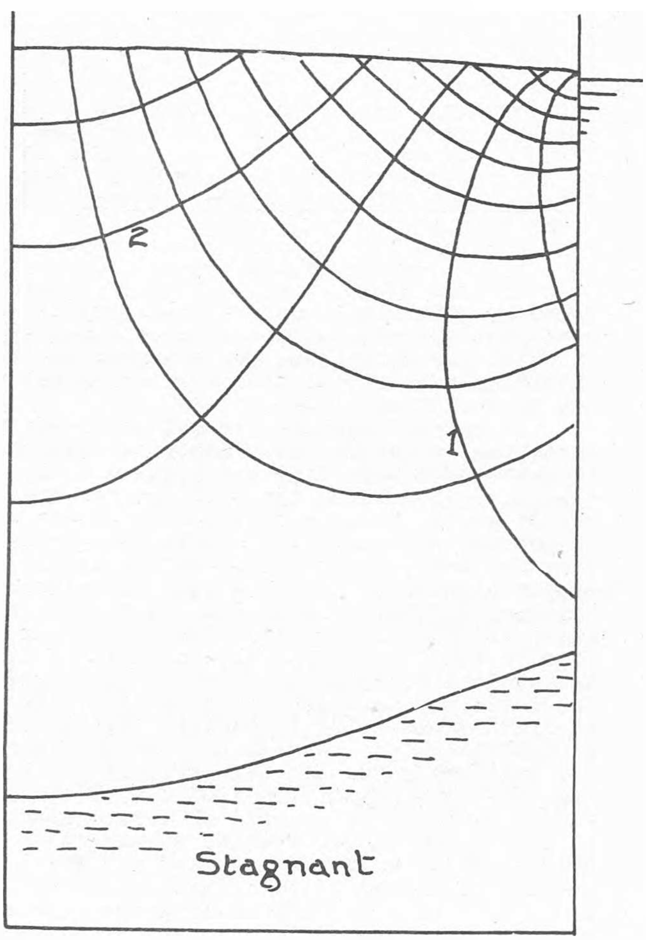

FIG. 4

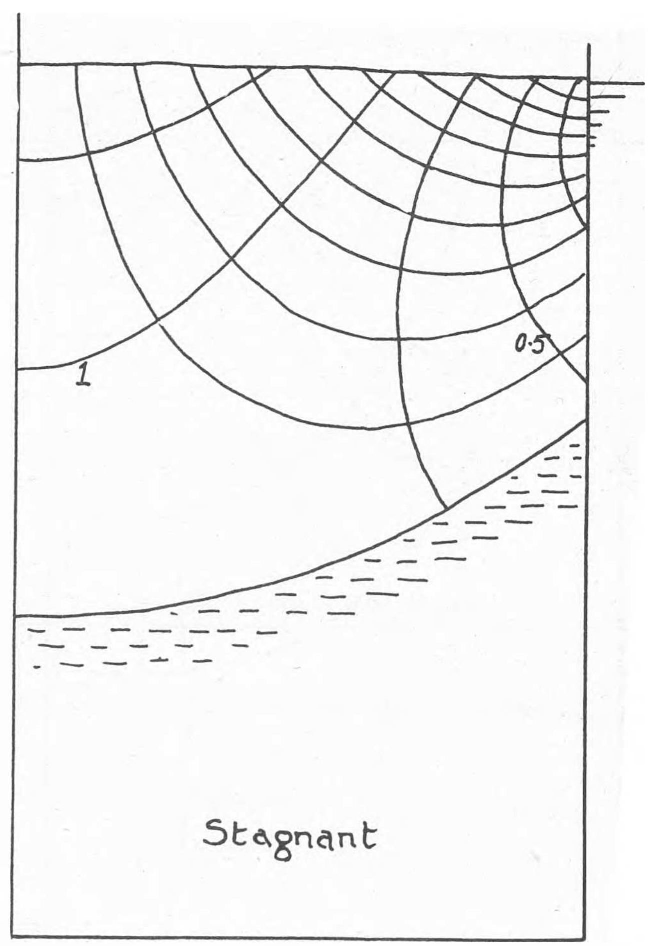

FIG. 5

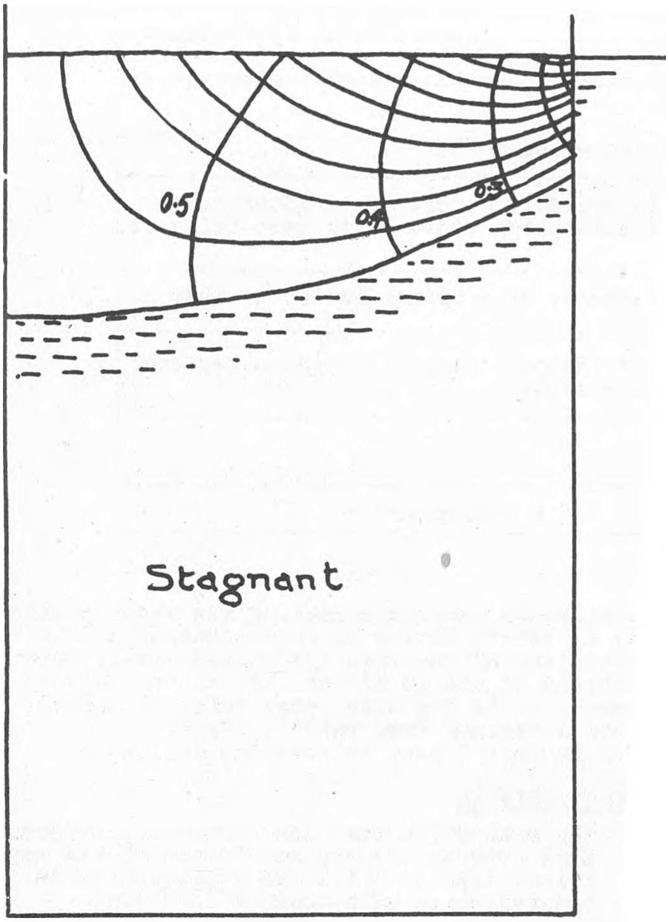

FIG. 6

dyke entirely whilest in fig. 6 the salt water has risen to 45 m below sea level at the mid point.

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X a 13

ADDITIONAL REMARKS (BY LETTER) ON PAPER Xa 10

M. SITZ (Palestine)

This article was prepared by me in extraordinary unnormal conditions, while staying in Jerusalem in war conditions. Although believing until now that the work is worth of consideration, there occurred certain discrepancies. Unfortunately, the work has already been printed in volume V of the Proceedings.

Seeing no other way, I beg to make the following remarks:

Cancel page 289 and read them as follows:

2. Maximal Stresses Sustained by the Capillary Film. Fig. 2.

When a capillary fringe reaches a limit height of 10 metres, its surface (W.L.I. or capillary film) 6) will sustain a maximum difference of pressure : +Pa - O = Pa x)

The capillary film in the case acts towards the capillary fringe similar to a perfect vacuum above a free surface of water, but with the following difference: in the case of the capillary film an additional pressure Pa of the zone above it exists and this creates stresses in it. Let us compare two cases of situation of surface of saturation: 1) at W.L.I. 2) surface of saturation being lowered from W.L.I. 10 metres below it down to W.L. II. xa)

The distribution of pressure in both cases will be, as shown in fig. 2.

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x) Pa - pressure of one atmosphere

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xa) It is assumed here, and further in this Report, that the soil under discussion is so heavy that it can bear a limit capillary rise of 10 metres.

	case I	case II
above W.L.I	constant pressure of one atmoshpere	
at W.L.I.	Pressure is equal to one atmosphere	A jump in pressure occurs falling abruptly from one atmosphere above it to zero below it.
Below W.L.I.	Hydrostatic distribution of pressure, this being similar to that of	
	an open water container	a closed water container, in which a perfect vacuum is provided
at W.L. II	Pressure is equal to	
	2 atmospheres	1 atmosphere

We may derive distribution of pressure in case II from that of case 1 after subtraction of $\Delta A + \Delta B = 1$ Pa below W.L.I, fig. 2. In case II the capillary film (at W.L.I) sustains a difference in pressure of one atmosphere.
......
difference in pressure of two atmosphere after further subtraction of ΔA.

3. Natural capillary Rise and Rise of Underground Water by Means of a Perfect Vacuum.

Compare two cases of pumping water by means of a perfect vacuum: 1) from a free surface and 2) from an underground water-bearing layer, In the first case water will rise 10 metres above the water surface. In the second case the W.L. will remain unchanged; this takes place due to the drawing down of the W.L. to a limit of 10 metres. When underground from these - 5 . Here, one atmosphere is being accounted for sustaining the resistance of flow through capillary interstices, height which is to be divided by two, thus causing the water yielded by a perfect vacuum to be discharged at the elevation of the existing static level. This is instead of the 10 metres lift above surface level, as is the case, when water is pumped from a regular free water surface.
Furthermore I want to make the following

POST SCRIPTUM

a) The assumption that the difference of pressure between the top and bottom of the capillary film at W.L.I. is variable, is in contradiction with that of Jukowsky.
b) The existence of a permanent cone was overlooked by Jukowsky.
c) In my first Report 6) it was assumed that the surface of saturation is being regulated to W.L.II.
d) Compare fig. 5 with ordinary moisture-density curves. Note, as well the close relationship between fig. 4 from this Report with fig. 10 from the first Report 6).

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X a 14

DISCUSSION

J. MANDEL (France)

I wish to underline the interesting features of the experimental and theoretical research which correlate the laws of filtration in porous media to the laws of flow in cylindrical tubes. The idea is certainly not a new one, but very interesting progress may have been made in this direction and I should like to draw the attention to Mr. Cambefort's and Mr. Ferrandon's statements.

When an incompressible fluid flows through a cylindrical tube with a diameter D, the loss in pressure i per unit of length $\left(i = \frac{d\varphi}{dx}\right)$ (φ indicating the pressure) depends on the variables D, ρ (specific gravity), V (mean speed along a straight section) ν (kinematic viscosity), e (mean depth of pits in the tube). The essential homogeneity of the formulae allows this dependency to be expressed in the form of the relation:

$$\lambda = f(R, \mu)$$

between the 3-non-dimensional values:

$\lambda = \frac{g D i}{V^2}$ coefficient of loss in pressure;

$R = \frac{VD}{\nu}$ Reynolds' number;

$\mu = \frac{e}{D}$ coefficient of roughness.

If the curves $\mu = C^{te}$ are plotted on a graph the abscissae of which are Log R and the ordinates Log λ, we obtain the following figure according to Nikuradse's experiments. Each curve is composed of 3 parts: (a) for the low values of R a straight line Log λ + Log R=C^{te}

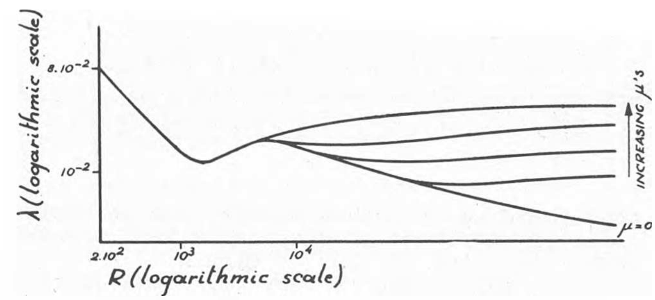

FIG. 1

independent of μ, corresponding to Poiseuille's laminar flow $\left(i = R\frac{\nu}{D^2}\right)$ (b) for the high values of R a straight line $\lambda = C^{te}$

$\left(i = k^1\frac{\nu^2}{D}\right)$ corresponding to a flow in which turbulence is established, (c) a transitional zone corresponding to the establishment of turbulence.

In these experiments on porous media Mr. Cambefort did rediscover this form of curve and if the quantitative correspondence is not yet satisfactory (which is not at all astonishing, since tubes have variable dimensions of straight section and this divergence in dimensions should obviously be taken into account), the result is nevertheless very interesting.

Mr. Ferrandon proceeds from Poiseuille's Law. By adding the relative discharges in tubes of different directions, he manages to represent the rapidity of filtration in an anisotropic medium by the formula.

$$\overline{\nu} = \overline{-k}.\ \overline{\text{grad}}\ \varphi$$

k being a secondary symmetrical tensor, which he calls "permeability tensor". Consequently there is not one coefficient of permeability, but three coefficients of permeability corresponding to the 3 main tensor directions, which become equal if the medium is isotropic.
This new idea of permeability tensor seems to me to be of the utmost significance and illustrates the part played by statistical data applied to soils.

Mr. Ferrandon then proceeds to the calculation of hydrodynamic actions, taking as the sum the actions of the water on the walls of the tubes, between 2 level sections d s apart. He admits that the actions of the water on the walls of a tube between the 2 sections envisaged have a resultant force which is in the direction of the axis of the tube. On this particular point I do not agree with him. Let us ignore (as Mr. Ferrandon does) the influence of gravity, i.e. Archimedes thrust.

The hydraulic actions on the walls of the tube are therefore statically equivalent to the forces exerted on the two extreme sections of the tube. (equilibrium of the water contained

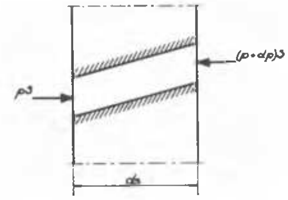

FIG. 2

in the tube between these two sections).

Now I have demonstrated strictly by a calculation which is reproduced in the annex that: "In the permanent laminary flow of a liquid within a tube having absolutely any cylindrical form, the sum of the restraining forces being exercised in the liquid at any plane section of the tube is always normal at that plane section".

Hence the actions of water on the walls of tubes are normal at the plane sections to which Mr. Ferrandon refers. The calculation and the result become more simple. We find that the hydrodynamic action is represented at an almost constant factor (whether the medium is isotropic or not) by the vector-grad φ, which is in agreement with my findings at the end of the first paragraph of my paper read at this Congress (Id 9).

Annex.
Permanent laminary flow of a liquid in a cylindrical tube. Oz is the direction of the generative end of the tube. O X, O Y represent the 2 rectangular axes in a straight section. The classical solution (Boussinesq) of this problem is (u, v, w, being the components of speed):

$$u = o \quad v = o \quad w = W\ (x,y)$$

W(x,y) being a function determined by the equation:

$$\frac{\delta^2 w}{\delta x^2} + \frac{\delta^2 w}{\delta y^2} = A \quad \text{(constant)}$$

and the condition $w = o$ on the contour (c) of the straight section. Consequently, for the constraining forces (according to Navier's formulae for viscous liquids) we get:

$$n_x = n_y = n_z = -p \qquad t_{xy} = o$$

$$t_{xz} = \mu\frac{\delta w}{\delta x} \qquad t_{yz} = \mu\frac{\delta w}{\delta y}$$

The geometrical sum of the constraining forces on a plane section of normal α, β, γ will be:

$$F_x = \gamma\iint t_{xz}\,dS \qquad -\alpha\iint p\,dS$$

$$F_y = \gamma\iint t_{yz}\,dS \qquad -\beta\iint p\,dS$$

$$F_z = \alpha\iint t_{xz}\,dS + \beta\iint t_{yz} - \gamma\iint p\,dS$$

$$F_z = \alpha\iint t_{xz}\,dS + \beta\iint t_{yz} = dS - \gamma\iint p\,dS$$

But the integrals emanating from the t_{xz}, t_{yz} are nought, for

$$t_{xz}\,dS = \frac{1}{\gamma}\iint t_{xz}\,dx\,dy = \frac{\mu}{\gamma}\iint\frac{\delta w}{\delta x}\,dx\,dy = \frac{\mu}{\gamma}\int_c w\,dy = 0$$

$$t_{yz}\, dS = \frac{1}{\delta}\iint t_{yz}\, dx\, dy = \frac{\mu}{\delta}\iint \frac{\delta w}{\delta y}\, dx\, dy = -\frac{\mu}{\delta}\int_c w\, dx = 0$$

Consequently, the geometrical sum is normal at the plane section referred to.

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X a 15

DISCUSSION

G. DROUHIN (Algeria)

With regard to the circulation of water in the soil it would, I think, be of use to mention an observation which is quite instructive, in that it stresses the fact that laboratory tests may lead to false conclusions when the very type of flow is not well known and the soil structure is too fragile for undisturbed samples to be drawn from it.

I refer to the research going on at present in Algeria on the "Hautes Plaines Oranaises" in the Chott-ech-Chergui region, whose aim is the exploration and exploitation of a very important aquiferous stratum. It is not within the scope of this subject to explain the very special circumstances which surround the existence of these resources. Suffice it to say, that we have here a continental clayey formation, in which water circulates from bottom to top as a result of pressure from the substratum, to evaporate in the layer above, the water level remaining a few decimeters below the surface of the ground and the capillary ascent of the water keeping the surface at Chott damp, even after the driest of summers.

The problem was to evaluate this evaporation, one of the indirect methods employed being to gauge the vertical speed of circulation. To this end a series of piezometers were constructed at increasing depths, to obtain a graph of pressure losses and on the other hand laboratory tests were carried out to measure the coefficient of permeability of samples which had been disturbed as little as possible.

The latter operation systematically yielded coefficients of permeability of the order of $n \times 10^{-7}$ cm/min, differing by a few units, and the comparison of these values with the pressure losses measured, of the order of 5 cm per m. yielded very low speeds of ascent. The values found were out of proportion with anything which could have been predicted, both the examination of the hydrological balance of the system under investigation and the microclimatological measurements, the probable order of magnitude of the coefficient of permeability being $n \times 10^{-3}$ cm/min.

The key to the mystery was found quite accidentally - in course of the excavation of a test well, when this well was already some fifty meters deep, it was discovered that the soil, seemingly impermeable in small particles, was peppered with small channels lined with a crystalline seam, the diameters of which varied from 1/2 to 5 mm. These small channels are mostly in perpendicular direction but they are anastomosized in every direction. Once attention had been drawn to this point, these small channels were found everywhere and at every depth, particularly if the very numerous cores drilled were carefully examined.

Methods are being devised to measure on a large scale the permeability of this peculiar structure the explanation of which may quite easily lie in the hydrogeological history of the locality. It has been well established, however, that permeability experiments on small samples cannot yield results. The statistical distribution of the small channels would have prohibited the interpretation of the measurements, even if the drawing and preparation of the samples to be tested had not caused occlusion of the superficial part of the test pieces.

This shows that investigations in situ are not invariably easy, but that it is essential to pursue them to great length in order to obtain a sound understanding of the phenomena, in the absence of which laboratory methods run the risk of being inadaptable and leading to false conclusions. This likewise gives food for thought, in that for an important problem it is essential to employ several methods of evaluation based on the most varying methods possible, in order to obtain cross-checks.

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SUB-SECTION X b

SEEPAGE PROBLEMS OF DAMS AND LEVEES

X b 5

WRITTEN DISCUSSION ON PAPER Xb 4 AND 11.4

D.P. KRYNINE (U.S.A.)

The papers by Messrs. Mansur and Perret (Xb 4) and by Messrs. Turnbull and Mansur (11.4), entitled "Efficacy of Partial Cut-offs for Controlling Underseepage beneath Dams and Levees Constructed on Pervious Foundations" (Xb 4) and "Model Studies of Drainage Wells for Dams and Levees on Pervious Foundations" (11.4) respectively, have very much in common. Both papers have been prepared by the members of the Waterways Experiment Station in Vicksburg, Miss., of the Corps of Engineers, U.S. Army, and refer to the model studies (including electrical models) of the seepage under Mississippi River levees. The results of these model studies are in satisfactory agreement with existing theoretical data along these lines as quoted in the references given in the papers.

It has been established in these model studies that cutoffs with a 100% penetration only (i.e. extending through the pervious foundation to reach the underlying impervious layer) reduced the underseepage or landside pressures. Cutoffs with smaller penetration were not so efficient; particularly cutoffs with penetration less than 25 per cent had practically no effect at all. Again, the longer the path of seepage flow the less effective were partial cutoffs. The impression gained by the writer from paper Xb4 is that partial cutoffs should not be used, especially cutoffs with small penetration.

In contrast, models of drainage wells on the landside of a levee were a rather desirable means of decreasing the hydraulic head and preventing erosion (boiling) at the landside, thoug in this case the amount of seepage water (total flow) was increased a certain amount. In the case of a stratified foundation, it is important that the wells penetrate into the principal water-carrying strata.

The tests revealed that openings in cutoffs or, in the case of a levee, excavations in the impervious upper layers on the riverside of that levee (such as borrow pits) decrease the efficiency of the cutoffs and the drainage wells, respectiviey. This conclusion could be anticipated _a priori_, however.

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x) Paper no. 11.4 has not been received by the Editorial Committee.

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X b 6

WRITTEN DISCUSSION ON PAPER Xb 2

R.G. HENNES (U.S.A.)

It is believed that this paper will be of interest to those who are concerned with levee and earth dam construction, and especially those who may not have ready access to the American references listed by the author. The practical value of the paper is enhanced by the author's charts, and is assured by the very considerable experience of the Waterways Experiment Station staff in this general field.

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