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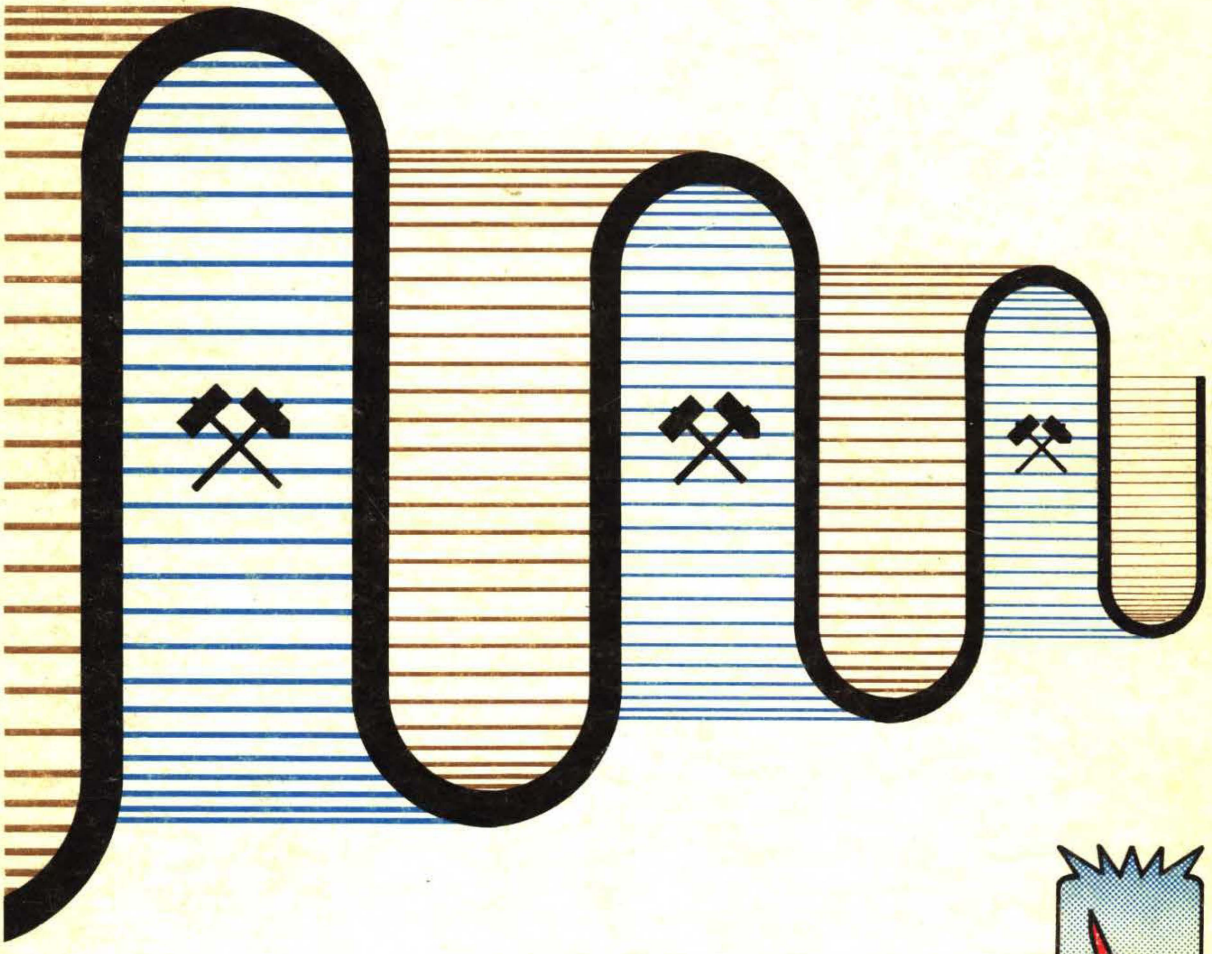
WATER IN MINING AND UNDERGROUND WORKS.

L'EAU DANS LES MINES ET TRAVAUX SOUTERRAINS.



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V. II

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NEW CONCEPT TO DESCRIBE FLOW THROUGH POROUS MEDIA

Bobok, E.

ABSTRACT : The present paper introduces a new treatment of the ground water flow. The velocity distribution in the pores is always changing stochastically, but the mean values show regularity. The velocity field of the flow through porous media is taken into consideration as a mean velocity field, ant the fluctuation about the mean, superimposed to it. In this base the equation of motion can be derived.

RESUME : La présente conférence introduit un nouveau traitement du débit de l'eau souterraine. La distribution de la vitesse dans les pores est toujours changée stochastiquement, mais les valeurs principales montrent la régularité. Le champ de vitesse du courant à travers le milieu poreux est pris en considération comme une moyenne de champs de vitesse, et la fluctuation autour de cette valeur qui lui est superimposée. A partir de cette base, l'équation du mouvement sera démontrée.

RESUMEN : Esta comunicación presenta un nuevo tratamiento del flujo del agua subterránea. La distribución de velocidad en los poros cambia siempre estocásticamente, pero los valores medios muestran regularidad. El campo de velocidades del flujo, a través de medios porosos, se considera como una media de los campos de velocidad, a la que se superponen las fluctuaciones sobre dicha media. Con estas premisas se deduce la ecuación del movimiento.

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To describe the movement of ground water a generally accepted method is known. Assuming laminar flow of fluid through porous media, general flow equations of ground water can be derived from Darcy's law and the equation of continuity. In some cases the equations for steady flow of a single fluid can be simplified into the Laplace equation, for which we can obtain solution by potential-theory methods. This is the way which is mostly followed.

This study has been motivated by three factors. First from a practical point of view, the new mine-opening works in Hungary are done in water dangerous areas. The classical preventive methods seem here insufficient. The new so-called instantaneous prevention technique against water intrusion in mines requires a better knowledge of the nature of ground water movement.

Secondly, during laboratory experiments the relationship between the flow rate and head loss was obtained non-linear, though the motion had been unquestionably laminar. Thus a non-linear resistance law occurs as soon as the inertia terms are taken into account, without the fluid being in true turbulent motion. Non-linearity can be found at very low Reynolds number /such $\approx 0{,}1$/.

Thirdly, to obtain a comprehensive view of flow through porous media, the balance equations of the transport theorem present themselves as the most advanced means. In spite of the partial success with the explanation of Darcy's law, we introduce a really dynamic description of seepage flow upon a general ground.

Our basic assumptions are the following. The detailed fine structure of the rock and the fluid in its pores is replaced by a continuous model of matter having appropriate continuum properties so defined as to ensure that as the macroscopic scale

the behaviour of the model duplicates the behaviour of the real material. We assume that the pores would be ignored individually if the pore sizes are rather small compared with the dimensions of the problem. The two-component /rock and fluid/ system forms a complex continuum. The essential mathemathical simplification of the continuum model is that the average properties in the infinitesimal volume dV surrounding point P are assigned in the limit to the point P itself. Thus one arrives at an equivalent but fictious continuous material, locally homogeneous and unstructured in which the real material is represented by continuously distributed field functions of its phisical properties: the density, velocity, stress, etc.

As an illustration of the limiting process by which the local continuum properties are defined, imagine a small volume ΔV surrounding a point P, and let ΔV_p be the pore volume of material in ΔV. Now the ratio $\Delta V_p / \Delta V$ will except as noted below, asymptote toward some limit as ΔV is made smaller. When ΔV reaches some value dV, which may be thought of as the infinitesimal volume of the complex continuum, significant random fluctuations begin to appear because the number of pores in ΔV is too small, and the notion of the ratio $\Delta V_p / \Delta V$ then loses practical utility. Thus the porosity at the point P is defined as

$$\Pi = \lim_{\Delta V \to dV} \frac{\Delta V_p}{\Delta V} = \frac{dV_p}{dV} \qquad /1/$$

in which the limiting process is carried only to dV with the understanding, however, that Π is associated with the point P itself. Thus we introduced a continuous scalar field of porosity $\Pi/\vec{r}, t/$.

Assuming that $\Pi\,/\vec{r},t/$ is a continuous field and the pores are filled perfectly with fluid, we get for mass of fluid per unit volume

$$\beta = \Pi\,\rho \quad , \qquad /2/$$

where ρ is the density of the fluid.

Let the local velocity of the fluid be designated by the vector $\vec{v}$. This is the velocity with respect to fixed coordinates. It is not the velocity of the fluid particles in the pores, but rather the average velocity with which the particles of fluid in a small region are moving. If the infinitesimal volume dV is rather small compared with the investigated domain of the flow, $\vec{v}/\vec{r},t/$ can be considered as a continuous vector field.

The principle of conservation of mass will be expressed as

$$\frac{\partial}{\partial t}\,/\Pi\,\rho\,/ + \operatorname{div}\,/\Pi\,\rho\,\vec{v}/ = j \qquad /3/$$

for the unit volume of the complex continuum.

This statement means that the rate of increase of fluid mass within the unit volume of complex continuum, and the rate at which fluid mass crosses into it, over its surface, are equal with the rate at which fluid mass is produced /or disappeared/ within the unit volume by diffusion, adsorption or chemical reaction.

Without mass production the right side of the equation is vanished:

$$\frac{\partial}{\partial t}\,/\Pi\,\rho\,/ + \operatorname{div}\,/\Pi\,\varrho\,\vec{v}/ = 0. \qquad /4/$$

For perfectly rigid rocks the porosity isn't depend on time, it varies only with position, thus

$$\Pi \frac{\partial \varrho}{\partial t} + \operatorname{div} /\varrho\, \Pi\, \vec{v}/ = 0. \qquad /5/$$

The product $\Pi \vec{v}$ is called as the local seepage velocity:

$$\vec{q} = \Pi \vec{v}. \qquad /6/$$

In an incompressible flow the density ϱ varies neither time, nor position, continuity equation becomes very simple form:

$$\operatorname{div} \vec{q} = 0. \qquad /7/$$

The flow in each pore-channel can be described by the equation of continuity and by the Navier-Stokes equation. Nevertheless this is not an easy way to solve any problems. Geometry of pore-system is of a wide variety of structures. If it were possible to prescribe the boundary conditions for only one pore-channel, the solution would be difficult and the results would have no any prectical utility. The full description of the flow in the pores requires an enormous number of equations, and the solution of these equations depends strongly on minute details of the initial and boundary data, while these details are never available in practice.

If the real in situ motion in the pores is replaced by a space--averaged flow, we get suitable means to solve practical problems. For mathematical treatment it is convenient to split up the velocity of the pore flow $\vec{u}$ into a mean $\vec{q}$, and a fluctuation $\vec{w}$ superimposing on it.

$$\vec{u} = \vec{q} + \vec{w}, \qquad /8/$$

where q is the local seepage velocity. Let's assume that the fluid is incompressible, and the velocity field u/r,t/ satisfies the Navier Stokes equations, and the equation of continuity:

$$\frac{\partial \vec{u}}{\partial t} + /\vec{u} \bullet \vec{u}/\nabla = \vec{g} - \frac{1}{\varrho} \nabla p + \nu \nabla^2 \vec{u} \qquad /9/$$

$$\nabla \vec{u} = 0 \qquad /10/$$

Substituting Eq.8. into Eq.9. and following the usual rules for averaging we obtain

$$\frac{\partial \vec{q}}{\partial t} + /\vec{q} \bullet \vec{q}/\nabla + \overline{/\vec{w} \bullet \vec{w}/\nabla} = \vec{g} - \frac{1}{\varrho} \nabla p + \nu \nabla^2 \vec{q} \qquad /11/$$

From this equation the following important facts are obtained.

The rate of increase of momentum is composed of three terms. The first term is the rate of increase of momentum within the unit volume, the second is the rate of momentum crosses into it over its surface with velocity $\vec{q}$, the third is the rate of momentum crosses over the surface of the unit volume by the fluctuations $\vec{w}$. The latter term is an irreversible transport of momentum, which leads to an intensive energy dissipation.

The righ side of the equation consists of the sum of external forces acting on the fluid within the unit volume.

It is convenient to write the equation of motion in such a way that on the left side of it the rate of momentum will be stood, which refers to the seepage velocity field $\vec{q}$.

$$\frac{\partial \vec{q}}{\partial t} + /\vec{q} \bullet \vec{q}/\nabla = \vec{g} - \frac{1}{\rho} \nabla p + \nu \nabla^2 \vec{q} - \overline{/w \bullet w/ \nabla} \qquad /12/$$

In this aspect the latter term can be considered as an added external force, expressing the interaction between the porous medium and the fluid.

This force acts on the fluid against the flow, and it can be expressed by the divergence of a virtual stress tensor:

$$\overline{/-\vec{w} \bullet \vec{w}/\nabla} = \frac{1}{\rho} \text{Div } /-\rho \overline{\vec{w} \bullet \vec{w}/} = \frac{1}{\rho} \text{Div } \underline{\underline{S}} \qquad /13/$$

As it is shown, the virtual stress tensor is simmetric:

$$\underline{\underline{S}} = -\rho \begin{bmatrix} \overline{w_x w_x} & \overline{w_x w_y} & \overline{w_x w_z} \\ \overline{w_y w_x} & \overline{w_y w_y} & \overline{w_y w_z} \\ \overline{w_z w_x} & \overline{w_z w_y} & \overline{w_z w_z} \end{bmatrix} \qquad /14/$$

therefore six unknown scalar function occur in the equation of motion.

In order to be able to make calculations of seepage flow it is necessary to find any expression for the additional virtual stresses, which relates them to the averaged quantities. Firstly we should assume the existence such a relationship, secondly we must look for this relationship in any reasonable form like the constitutive equation, moreover it should be in accordance with the Onsager's law. If this assumed expression substituting to the equation of motion leads to results corresponding with experimental data, it can be accepted as a confirmed relation.

As an illustration of the foregoing, an experimentally proved relationship, the Darcy's law will be derived.

As it is known the flow rate through porous media is proportional to the head loss and inversely proportional to the length of the flow path. The experimental verification of Darcy's law can be performed with water flowing at a rate Q through a cylindrical tank of radius R, packed with sand and having pierometer taps a distance L apart, as shown in Fig. 1.

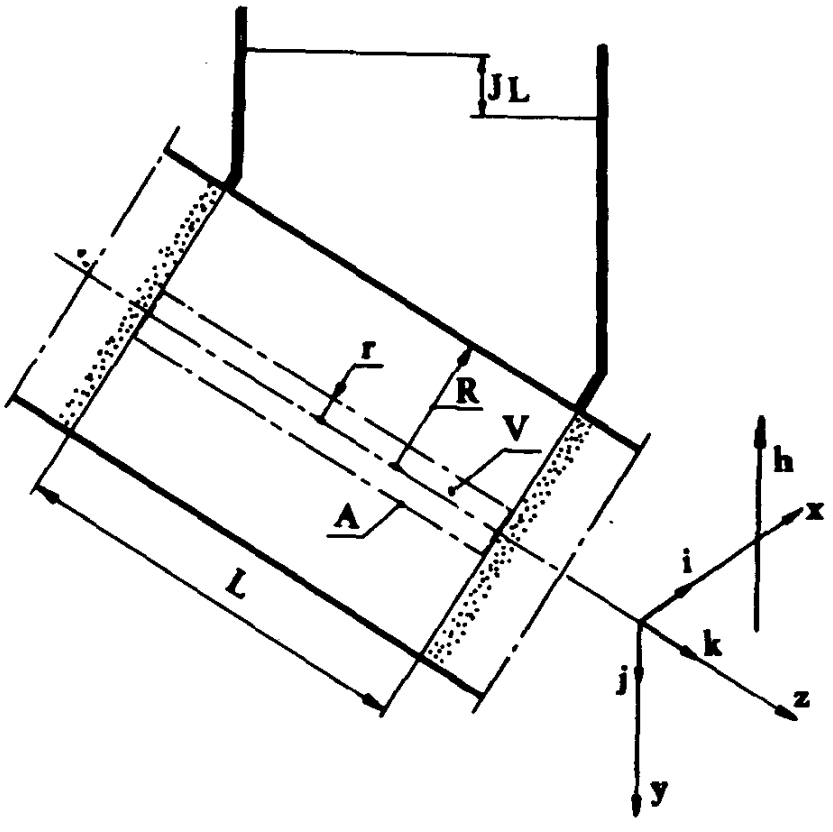

Figure 1.

Let's consider a coaxial cylindrical control volume of radius r, to which we write the equation of motion in the integral form:

$$\int_V \frac{\partial / \rho \vec{q}/}{\partial t} dV + \int_{/A/} \rho \vec{q} \, /\vec{q} d\vec{A}/ = \int_V \rho \vec{g} dV - \int_{/A/} p d\vec{A} +$$

$$+ \int_{/A/} 2\mu \, /\vec{q} \cdot \nabla + \nabla \cdot \vec{q}/ \, d\vec{A} - \int_{/A/} \rho \, \overline{/\vec{w} \cdot \vec{w}/} \, d\vec{A} \qquad /15/$$

This equation can be simplified, if the following assumptions are taken to consideration:

/i/ The velocity field has only axial component

$$q = q_z; \qquad q_x = q_y = 0$$

/ii/ The equation div $\vec{q} = 0$, will be

$$\frac{\partial q_z}{\partial z} = 0$$

/iii/ The flow is steady:

$$\frac{d\vec{q}}{dt} = 0$$

Thus the left side of the equation of motion will be vanished. Assuming the existence of a force potential, we can write:

$$0 = -\int_V \varrho \nabla \left(U + \frac{p}{\varrho}\right) dV + \int_{/A/} 2\mu \left/\vec{q} \bullet \nabla + \nabla \bullet \vec{q}\right/ d\vec{A} -$$

$$- \int_{/A/} \varrho \overline{\left/\vec{w} \bullet \vec{w}\right/} \, d\vec{A} \qquad /16/$$

In gravitational field

$$U = gh,$$

thus we get:

$$-\vec{k} \int_V \varrho g \frac{\partial}{\partial z} \left/h + \frac{p}{\gamma}\right/ dV = \varrho \, g \, J \, r^2 \pi \, L \, \vec{k} \, ,$$

in which J the hydraulic gradient:

$$J = -\frac{\partial}{\partial z} \left/h + \frac{p}{\gamma}\right/ = \frac{h_1 - h_2}{L} + \frac{p_1 - p_2}{\gamma L} \qquad /17/$$

We assume, that Stokes's principles can be generalized to the seepage velocity field. The linear nature of Darcy's law affirms this.

At least we assume that the tensor field $\overline{\vec{w} \bullet \vec{w}}$ is axially symmetric, and don't vary in axial direction.

Thus the integrands are constant on the surface of the control volume and we get the following expression

$$g\,J\,r^2\,\pi\,L\,\vec{k} + \mu\,\frac{dq}{dr}\,2r\,\pi\,L\,\vec{k} - \varrho\,\overline{w_r\,w_z}\,2\,r\,\pi\,L\,\vec{k} = 0 \qquad /18/$$

For the unit volume it is obtained

$$\frac{g\,J\,r}{2} + \pi\,\frac{dq}{dr} - \varrho\,\overline{w_r\,w_z} = 0 \qquad /19/$$

Now we must look for some reasonable assumption in accordance with Onsager's law for the only element of the virtual stress tensor:

$$-\varrho\,\overline{w_r\,w_z} = \Lambda\,\frac{dq}{dr}\,, \qquad /20/$$

where Λ is the conduction coefficient of the momentum, which characterizes the interaction between the rock and fluid. Generally it is not a constant quantity, but we feel strong temptation to consider Λ to an additional viscosity. This simplification is suitable to derive Darcy's law, but the measurements of the local seepage velocity lead to that result that Λ is not constant along the radius, it depends on the radius, and the inhomogenity of the velocity field:

$$\Lambda = \Lambda\,/r\,\frac{dq}{dr}/.$$

The simplest possibility is Λ = const, for which we must solve the following differential equation:

$$\frac{g\,J\,r}{2} + /\,\mu + \Lambda\,/\,\frac{dq}{dr} = 0 \qquad /21/$$

After integration we get:

$$q = - \frac{g\,J}{4\,/\mu+\Lambda\,/}\,r^2 + C \qquad /22/$$

The determination of the constant is not too easy, since the seepage velocity is not a real, but a virtual velocity. Thus we can't prescribe, that the tangential component at the wall will be vanished. If we choose to boundary condition

at $r = R$, $q = 0$, the flow rate is smaller than the measured value. This shows that the local seepage velocity at the $r = R$ will not be zero, the seeping fluid "slips" on the wall. Thus the constant must be determined from the flow rate:

$$Q = 2\pi \int_0^R q\,r\,d\,r \qquad /23/$$

After substitution we obtain

$$Q = - \frac{\pi g\,J\,R^4}{8\,/\mu+\Lambda\,/} + C\,R^2\pi \qquad /24/$$

Measuring the flow rates and the corresponding hydraulic gradient values, the Λ coefficient and the constant C /the local seepage velocity on the wall/ can be determined.

Since for a given experimental apparatus only Q and J varies, the relationship between the flow rate and the hydraulic gradient is

$$Q = K\,/J - J_o/ \qquad /25/$$

where J_o is an additional constant, which is a threshold hydraulic gradient at which the seepage motion will be started.

Thus we derived the Darcy's law on the base of continuum mechanics. This can be a possible proof of the introduced description of the seepage flow. The basic equations derived above will be applied subsequently to obtain analytic or numerical solutions to particular seepage flow problems.

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UTILISATION DE MODELES MATHEMATIQUES POUR LA PREVISION DES VENUES D'EAU EN TRAVAUX MINIERS

A. BONNIER (1) et A. KORGANOFF (2)

ABSTRACT : Estimating the inflow of groundwater in mine works is especially made difficult by the variability of the geometrical characteristics of these works and by the need of dewatering the water-bearing horizons. Mathematical models which simulate groundwater flow generally help to determine to a satisfactorily accurate degree the rates of inflow of the water to be drained and their variations. A comparison of the probable rates of inflow that models of this type provide and the real pumpage volumes is presented first in the case of a quary and second, in the case of a mine shaft.

RESUME : L'estimation des venues d'eau en travaux miniers est rendue difficile, en particulier, par la variabilité des caractéristiques géométriques de ces travaux, par la nécessité de dénoyer les horizons aquifères. L'utilisation de modèles mathématiques de simulation des écoulements souterrains permet, généralement, de prévoir avec une bonne précision les débits à exhaurer et leurs variations. Une comparaison entre les débits probables à exhaurer fournis par de tels modèles et les exhaures réelles est présentée d'une part dans le cas d'une carrière et d'autre part dans le cas d'une descenderie.

RESUMEN : El cálculo del aflujo de agua en las labores mineras es difícil, debido a la diversidad de características geométricas de estas obras, y a la necesidad de drenar los tramos acuíferos. El empleo de modelos matemáticos de simulación de los flujos subterráneos permite, generalmente, prever con una buena precisión los caudales a drenar y sus variaciones. Se presenta una comparación entre los caudales probables a drenar según el modelo y los drenajes reales, para el caso de una cantera y para el de una galería inclinada.

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Tout ouvrage minier, galeries, descenderies ou carrières, se comporte comme un drain et modifie en conséquence les écoulements souterrains. De ce fait, préalablement à la mise en exploitation de tout nouveau gisement, et en particulier d'un gisement sédimentaire, il est souhaitable de réaliser une étude hydrogéologique détaillée que l'on se place du point de vue de l'exploitant ou de celui de la puissance publique.

L'exploitant est principalement intéressé par la détermination des venues d'eau à exhaurer en fonction de la variation des caractéristiques dimensionnelles des ouvrages. Cette estimation lui permet notamment de définir les moyens d'exhaure, de décider s'il est ou non nécessaire de rabattre la nappe préalablement aux travaux, de calculer les coûts d'exhaure... Elle lui permet également de vérifier si les volumes exhaurés seront suffisants pour couvrir, à tout instant, les besoins en eau industrielle, tels que ceux nécessités par le fonctionnement de la station de traitement des minerais.

L'exploitant et la puissance publique sont tous deux intéressés, à des titres divers, par la prévision des modifications qu'entraineront les pompages miniers sur les nappes, les forages ou captages déjà existants. Enfin la puissance publique a l'obligation de s'assurer que les exploitations minières ne conduiront pas à un gaspillage des ressources en eaux souterraines que celui-ci soit quantitatif ou qualitatif.

1 - PROBLEMES HYDROGEOLOGIQUES PARTICULIERS AUX TRAVAUX MINIERS

Les travaux miniers présentent un certain nombre de particularités qui rendent l'estimation des venues d'eau difficilement abordable par le biais des formules traditionnelles de l'hydraulique souterraine. En effet, les paramètres géométriques des ouvrages varient rapidement en fonction du temps :

- les carrières s'approfondissent et/ou s'agrandissent,
- les galeries se creusent à des niveaux différents et ont des extensions et des directions variables d'un niveau à l'autre,
- le foudroyage peut mettre en communication des horizons aquifères initialement distincts.

Ces variations dimensionnelles s'accompagnent fréquemment d'une modification de la nature des écoulements :

. Une nappe initialement en charge peut, par suite des pompages, se transformer au moins localement en une nappe à surface libre. En ce cas les caractéristiques hydrodynamiques locales seront modifiées : la porosité gravifique de la formation aquifère se substituera à son coefficient d'emmagasinement, la transmissivité de cette même formation ne pourra plus être considérée comme constante puisqu'elle subira une variation continue, fonction de l'épaisseur de la tranche restant saturée.

. En un deuxième temps il est d'ailleurs possible que l'exploitation minière nécessite un pompage au mur de la formation aquifère. En fonction du modelé topographique de ce mur, il pourra donc y avoir localement dénoyage des terrains aquifères. Ce dénoyage pourra, en outre, n'être que temporaire compte tenu du programme de l'exploitation minière (par exemple abandon de l'exploitation dans un quartier, arrêts périodiques...).

La prise en compte de cet ensemble de particularités ne peut être envisagée que si l'on utilise des moyens de calcul puissants, c'est à dire qu'après mise au point de modèles mathématiques de simulation des écoulements souterrains.

2 - CONCEPTION D'UN MODELE MATHEMATIQUE ADAPTE AUX TRAVAUX MINIERS

Nous examinerons successivement les équations générales des écoulements souterrains, le modèle numérique et les dispositions adoptées pour tenir compte des particularités propres aux problèmes miniers.

2.1. Equations générales des écoulements souterrains

On sait tout l'intérêt qu'il peut y avoir de considérer le milieu bien sûr hétérogène, mais aussi anisotrope. Si, dans le domaine étudié, on est amené - pour des raisons pratiques - à assimiler presque partout le milieu à un milieu isotrope, il n'en reste pas moins que dans certaines zones caractéristiques, notamment présence de failles, singularités, la représentativité du modèle implique que l'on tienne compte de l'anisotropie.

Nous désignerons par :

$$K(x,y) = K^T = \begin{bmatrix} K_{11} & K_{12} \\ K_{12} & K_{22} \end{bmatrix} \qquad (2.1.1)$$

le tenseur symétrique des perméabilités dans un système de coordonnées rectangulaires planes Oxy. On sait que dans ces conditions les axes principaux Ox'y' sont orthogonaux, et que suivant ces axes les perméabilités K_1', K_2' sont les valeurs propres de la matrice K. Nous désignerons par K' la matrice diagonale de ces valeurs propres. Si l'on désigne par ailleurs par V la matrice orthonormée

$$V = [V_{.1}\ V_{.2}]$$

des vecteurs propres donnant les axes Ox'y' dans le système d'axes Oxy, K, K', V sont liés par les relations matricielles

$$K = VK'V^T \quad , \qquad (2.1.2)$$

$$VV^T = V^TV = I \quad .$$

Cela permet d'exprimer facilement K' et V en fonction de K, ou bien K connaissant K' et V. Rappelons que si α est l'angle de rotation des axes Ox'y' par rapport aux axes Oxy, on a :

$$V_{11} = V_{22} = \cos\alpha \quad ,$$

$$V_{21} = -V_{12} = \sin\alpha \quad , \qquad (2.1.3)$$

$$\operatorname{tg} 2\alpha = \frac{2K_{12}}{K_{22} - K_{11}} \quad .$$

En général α est connu.

Nous noterons, pour la suite, D et d les opérateurs

$$D = \left[\frac{\partial}{\partial x} \quad \frac{\partial}{\partial y}\right] \quad , \quad d = De = \begin{bmatrix} \frac{\partial}{\partial x} \\ \frac{\partial}{\partial y} \end{bmatrix} \quad , \qquad (2.1.4)$$

où e est le vecteur unité ($e^T = [1\ 1]$), et de même D' et d' en ce qui concerne les variables x' et y'. On a alors :

$$D' = V^T D \quad , \quad d' = V^T d \quad . \qquad (2.1.5)$$

Ceci dit, l'approximation de Dupuit des écoulements souterrains rapportée en chaque point aux axes principaux s'écrit :

$$m \frac{\partial h}{\partial t} = \frac{\partial}{\partial x'}, (K'_1 (h-z) \frac{\partial h}{\partial x'},) + \frac{\partial}{\partial y'}, (K'_2 (h-z) \frac{\partial h}{\partial y'},) + \varepsilon \quad , \qquad (2.1.6)$$

où ε intègre la drainance au mur et au toit ainsi que les débits par unité de surface d'apport ou de prélèvement (injection ou pompage, infiltration ou évapotranspiration), et où t est le temps, h le niveau piézométrique de la nappe, z la cote du mur, m la porosité utile si la nappe est à surface libre ou le coefficient d'emmagasinement S si la nappe est en charge, et enfin (h-z)K' la transmissivité horizontale T' si la nappe est en charge.

Sous la forme matricielle, l'équation (2.1.6) s'écrit :

$$m \frac{\partial h}{\partial t} = e^T D' ((h-z)K'd'h) + \varepsilon \quad ,$$

et compte tenu de (2.1.5) et (2.1.2) :

$$m \frac{\partial h}{\partial t} = e^T V^T D ((h-z) V^T K d h) + \varepsilon \quad , \qquad (2.1.7)$$

ou si la nappe est en charge

$$S \frac{\partial h}{\partial t} = e^T V^T D (V^T T dh) + \varepsilon \quad . \qquad (2.1.7')$$

Une approximation consiste à supposer que dans un voisinage du point (x,y), les directions principales de l'anisotropie sont invariantes, ce qui revient à dire que α et V ne sont pas fonction de x,y. Cela est d'autant plus valable que la direction de la singularité ou de la faille est constante sur un voisinage discret (voisinage correspondant à la discrétisation adoptée lors de la numérisation des équations (2.1.7) et (2.1.7'), dont nous parlerons plus loin).

Dans ces conditions (2.1.7) et (2.1.7') deviennent :

$$S \frac{\partial h}{\partial t} = e^T (V^T)^2 D((h-z) K dh) + \varepsilon \quad , \qquad (2.1.8)$$

$$S \frac{\partial h}{\partial t} = e^T (V^T)^2 D(T dh) + \varepsilon \quad . \qquad (2.1.8')$$

Posons :

$$W = (V^T)^2 \quad ,$$

$$W_{11} = W_{22} = \cos 2\alpha \quad , \qquad (2.1.9)$$

$$W_{21} = -W_{12} = \sin 2\alpha \quad .$$

L'équation (2.1.8'), par exemple, prend la forme simple :

$$S \frac{\partial h}{\partial t} = (W_{11} - W_{12}) \frac{\partial}{\partial x} (T_{11} \frac{\partial h}{\partial x} + T_{12} \frac{\partial h}{\partial y}) + (W_{11} + W_{12}) \frac{\partial}{\partial y} (T_{12} \frac{\partial h}{\partial x} + T_{22} \frac{\partial h}{\partial y}) + \varepsilon \quad . \qquad (2.1.10)$$

C'est seulement lorsque $\alpha = 0$ (d'où $T_{12} = 0$), que l'on peut écrire :

$$S \frac{\partial h}{\partial t} = \frac{\partial}{\partial x} (T_{11} \frac{\partial h}{\partial x}) + \frac{\partial}{\partial y} (T_{22} \frac{\partial h}{\partial y}) + \varepsilon \quad . \qquad (2.1.11)$$

On a un résultat analogue au signe près avec $\alpha = \frac{\pi}{2}$.

Cela revient à considérer que localement les singularités ont pour direction l'un des deux axes Ox, Oy.

Enfin, rappelons qu'il faut rajouter les conditions aux limites du domaine, qui peuvent être de Dirichlet (conditions de niveau), de Neumann (conditions de penteà et plus généralement de débit que l'on peut rassembler dans l'équation.

$$\zeta h + \mu \frac{\partial h}{\partial x} + \nu \frac{\partial h}{\partial y} - \mu' T \frac{\partial h}{\partial n} = g(t) \quad , \qquad (2.1.12)$$

où n représente la direction de la normale à la frontière, intérieure au domaine au point considéré, et où ζ, μ, ν, μ', et g sont variables avec le temps, et évidemment avec x,y.

La solution est fonction de l'état de la nappe au temps initial.

2.2. Modèle numérique

Si l'on veut prendre en compte l'anisotropie, et plus précisément les dérivées de T suivant deux directions orthogonales d'espace, on est tout naturellement conduit à un maillage régulier (en direction) de discrétisation. Cela milite en faveur d'une méthode aux différences finies (ce qui n'exclut pas des méthodes variationnelles, telles que celles que l'on utilise dans les méthodes aux éléments finis).

Nous avons opté pour un maillage rectangulaire d'espacements irréguliers suivant les 2 axes, et pour le schéma de numérisation implicite de type Crank-Nicholson que l'on sait être inconditionnellement stable dans des conditions assez générales et donnant une erreur d'approximation en :

$$O\left[(\Delta x)^2\right] + O\left[(\Delta y)^2\right] + O\left[(\Delta t)^2\right] \quad .$$

Le système d'équations algébriques qui s'en déduit est linéaire en cas de nappes en charge, et non linéaire en cas de nappes à surface libre. Nous avons dans ce dernier cas linéarisé le système en prenant pour valeurs les coefficients celles qui correspondent aux valeurs de h au temps précédent. L'approximation est parfaitement valable si la variation de h est relativement faible pendant le pas de temps Δt séparant 2 calculs de la nappe.

Le système à matrice des coefficients de structure bande, se résout par une méthode de sur-relaxation classique, l'accélération de la convergence étant obtenue par calcul du facteur de relaxation optimum.

Le modèle pour fonctionner suppose connu un grand nombre de paramètres, dont on n'est pas toujours en possession. L'extrapolation est quelquefois hasardeuse dans des zônes de faible densité de mesures. Par ailleurs, et ceci doit être souligné, le peu de valeurs connues, par exemple de transmissivité, sont des valeurs locales, alors que le modèle réclame des valeurs moyennes sur une maille. Par conséquent même en des points où l'on a des valeurs mesurées, il peut y avoir nécessité de corriger celles-ci d'un facteur tenant compte de l'environnement immédiat. Enfin, toutes les hypothèses faites quant aux solutions de l'équation aux dérivées partielles, et quant à sa discrétisation sont des hypothèses de continuité et de variations "douces" des paramètres (dérivées à valeurs bornées et de faibles variations). Et il ne peut en être autrement alors que la nappe elle-même est une "bonne" surface. Pour toutes ces raisons, il y a lieu de lisser les valeurs des paramètres, en l'occurence principalement de T qui intervient par ses dérivées au contraire des autres paramètres. On voit de la sorte que localement, il peut exister des différences quelquefois non négligeables entre les valeurs de calcul et les valeurs réelles des paramètres. En ce qui concerne le modèle, ce sont ces valeurs de calcul qui ont pour ainsi dire force de loi, dès lors qu'une distribution de T redonnera l'état de la nappe tant en régime permanent qu'en régime transitoire observés. Quelles que soient ensuite les hypothèses que l'on pourrait se fixer quant à l'exploitation de la nappe, les niveaux calculés correspondront, à la précision près, aux niveaux que l'on observerait si la nappe était réellement en exploitation.

2.3. Dispositions particulières propres aux problèmes miniers

Il s'agit essentiellement des 4 dispositions suivantes :

a) prise en compte de la transition entre les zônes en charge et les zônes à surface libre ;

b) prise en compte du dénoyage possible de certaines zônes, et de leur renoyage éventuel ultérieur ;

c) prise en compte de domaines connexes représentant des surfaces à trous dont les contours ($\tau + 1$ contours si l'on a τ trous) peuvent être variables dans le temps ;

d) calcul des débits traversant un contour fermé ou non quelconque.

En ce qui concerne la transition entre zônes en charge et zônes à surface libre, le passage se fait suivant la courbe de la surface du toit. On suppose qu'au voisinage de cette courbe il y a de faibles variations (même s'il n'y a pas rigoureusement continuité des dérivées) des paramètres T à K(h-z), et des h. Seul subsiste un saut brusque de S à m.

Le dénoiement et le renoiement méritent un peu plus d'attention. Le dénoiement a lieu dès que h < Z. Dès qu'un point est alors dénoyé on crée au voisinage du point (voisinage discret dans le modèle numérique), ou des points dénoyés, un domaine (représentant donc un trou) avec conditions de débit nul. Quant au renoiement nous admettrons qu'il se fait par montée de liquide, autrement dit avec une vitesse verticale, sous l'influence de la drainance au mur, des injections ou infiltrations, et des niveaux d'eau aux limites du contour du trou. Si l'on désigne par C ce contour par h_σ le niveau en un point d'abscisse curviligne σ du contour, par δ_σ la distance du point (x,y) considérée au point σ, et par $\bar{h}$ la moyenne pondérée,

$$\bar{h} = \frac{1}{\int_C \frac{d\sigma}{\delta_\sigma}} \int_C \frac{h\sigma}{\delta_\sigma} \, d\sigma \quad , \qquad (2.3.1)$$

on prendra en première approximation l'équation de renoiement :

$$h\ (t + \Delta t) = \bar{h}\ (t) + \frac{1}{m} \int_t^{t+\Delta t} \varepsilon(t)\ dt \quad , \qquad (2.3.2)$$

si

$$h(t + \Delta t) > Z \quad .$$

Reste les dispositions c) et d) qui ne présentent pas de difficultés particulières, et qui s'implémentent facilement sur le modèle.

3 - EXEMPLES D'UTILISATION DU MODELE MATHEMATIQUE

3.1. Estimation des venues d'eau en carrière

Sous environ 30 à 40 mètres de terrains stériles imperméables, la couche exploitée est constituée d'un grès assez fortement cimenté contenant une nappe initialement en charge.

L'exploitation de la couche minéralisée devant se faire par carrière, le problème hydrogéologique consistait à :

. évaluer les venues d'eau à exhaurer en fonction de l'extension et de la profondeur de la carrière,

. comparer ces venues aux besoins industriels et étudier si nécessaire, l'implantation de nouveaux forages pour assurer en toute période la couverture de ces besoins. Inversement, si les venues étaient supérieures au besoins, il convenait de rechercher le meilleur dispositif de réinjection.

Cette étude a été réalisée selon la méthodologie suivante :

a) Mise au point d'un modèle méthématique général de la nappe prenant en compte les diverses conditions aux limites, les pompages préexistants. Sur ce modèle couvrant plusieurs centaines de km2, la carrière, malgré la taille relativement restreinte des mailles (mailles carrées de 800 m de coté), ne pouvait être représentée que par un noeud auquel on imposait une condition de niveau variable avec le temps. Ce premier modèle général a permis de déterminer avec une approximation suffisante les niveaux piézométriques et leurs variations en tous points du champ et en particulier aux limites d'un modèle de détail centré sur la carrière.

Une estimation provisoire des débits arrivant en carrière, était fournie par ce modèle général, ce qui permettait de proposer une première adéquation ressources besoins et d'imposer dans les limites du modèle focalisé, les pompages complémentaires nécessaires.

b) Ce modèle focalisé à mailles variables, dont les plus petites avaient 100 m de coté, permettait de suivre l'extension dans le temps des travaux miniers. C'est sur ce modèle focalisé qu'à été étudiée la variation des débits à exhaurer La figure ci-jointe (fig. 1) fournit d'une part les débits estimés d'après le modèle mathématique et d'autre part les débits réellement exhaurés.

Les débits estimés d'une part varient par sauts correspondant à l'imposition de nouvelles conditions qu'il s'agisse de l'extension ou de la profondeur atteintes et d'autre part décroissent régulièrement à l'intérieur d'une période à conditions constantes, suivant en cela les lois de l'hydraulique souterraine.

Les débits réellement pompés, bien que relativement variables d'un mois à l'autre ce qui peut s'expliquer par les aléas du planning d'exploitation minière ont en moyenne une courbe de variations continuement croissante. Ceci exprime le fait que les travaux miniers ont eux-mêmes une extension continuement croissante alors que la modélisation oblige, par suite de contraintes liées au temps de calcul et donc au coût, de simuler ces variations de façon discontinue.

Malgré les erreurs inhérentes à cette modélisation des caractéristiques dimensionnelles des ouvrages miniers, la confrontation entre exhaures prévisionnelles et réelles s'avère très satisfaisante et cela bien que le modèle ait dû prendre en compte des conditions d'écoulement variables dans le temps (nappe en charge, nappe à surface libre, zônes hors d'eau), et l'influence de pompages extérieurs à la carrière eux-mêmes variables dans le temps.

c) Enfin, pour obtenir une image piézométrique de la nappe sur l'ensemble du champ, le modèle général a été à nouveau utilisé. En ce cas, les conditions imposées au droit de la carrière ont été des conditions de débit variables dans le temps.

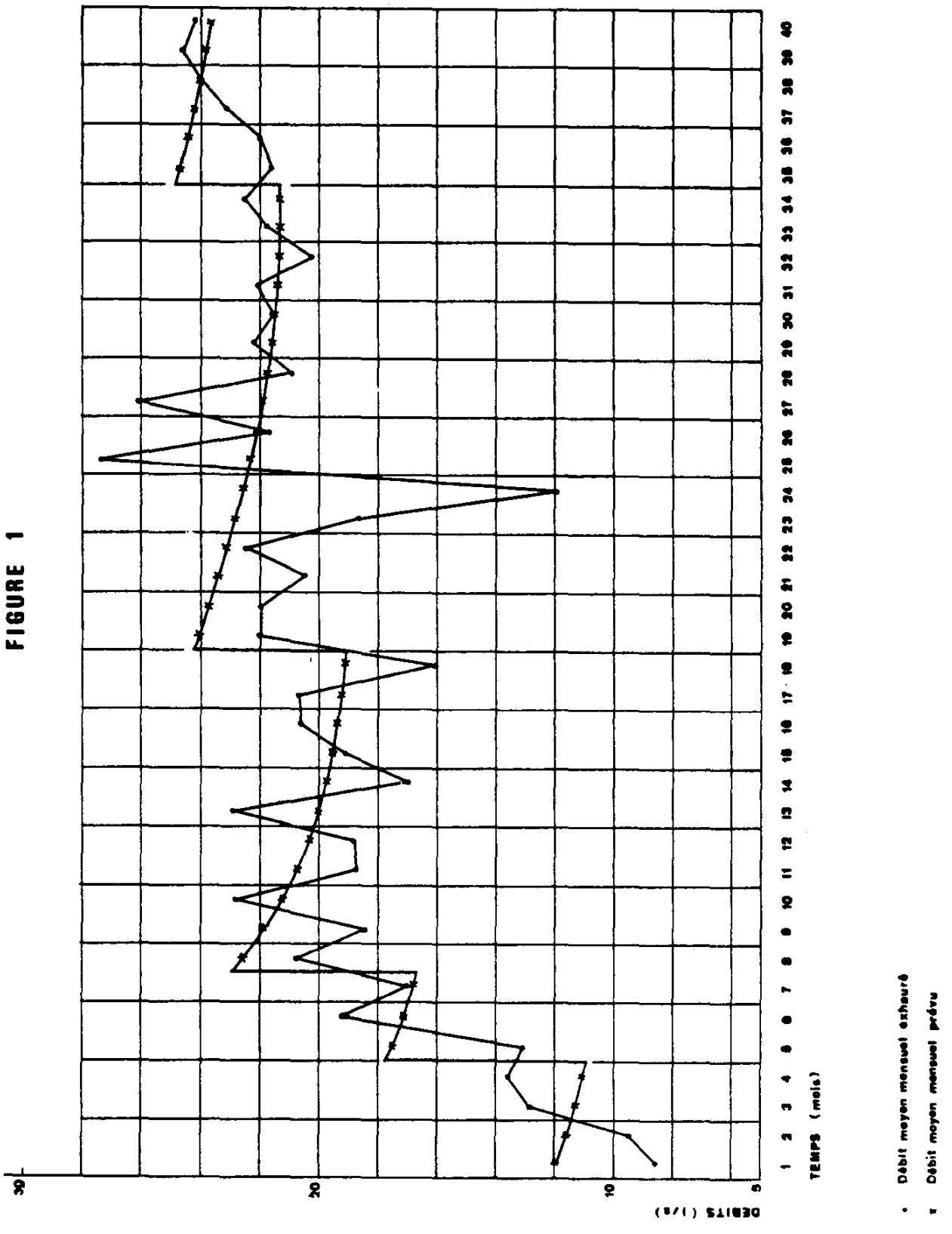

DEBITS (l/s)
30
20
10
0
TEMPS (mois)
1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 26 27 28 29 30 31 32 33 34 35 36 37 38 39 40
• Débit moyen mensuel écoulé
x Débit moyen mensuel prévu

FIGURE 1

Les pompages extérieurs à la carrière ont eux-mêmes été réestimés de façon à assurer à tout instant un équilibre entre les ressources et les besoins.

3.2. Estimation des venues d'eau en descenderie

Le problème posé était celui de la détermination des venues d'eau devant se produire dans une descenderie lors de la traversée d'un horizon grèseux de 50 mètres de puissance situé sous environ 140 mètres de terrains très peu perméables (argiles et arkhoses principalement).

La nappe contenue dans les grès était à l'origine une nappe en charge, dont le niveau statique était situé à environ 20 m sous le terrain naturel.

La méthodologie suivie a été identique à celle décrite dans le paragraphe précédent, c'est à dire utilisation d'un modèle général prenant en compte l'ensemble des conditions aux limites et existantes dans le périmètre modélisé puis d'un modèle focalisé centré sur les travaux miniers.

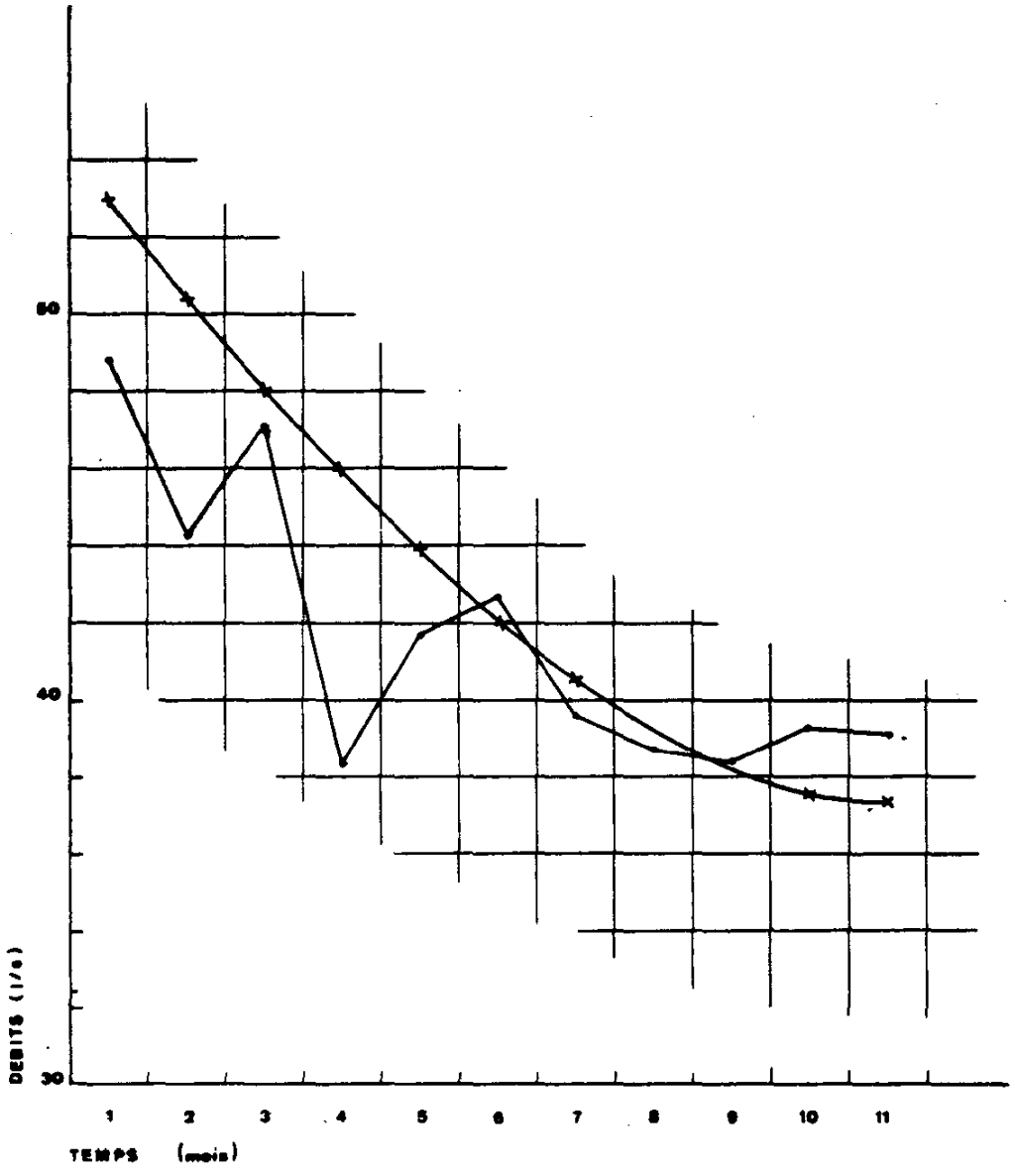
Compte tenu des reconnaissances préliminaires effectuées par forages et essais de pompage, il avait été décidé d'entreprendre le rabattement de la nappe environ 7 mois avant l'arrivée de la descenderie dans la couche aquifère au moyen de 2 forages équipés de pompes dont les crépines d'aspiration étaient situées à proximité du mur de l'aquifère. Ces forages étaient situés sur le tracé de la descenderie.

Le graphique ci-joint (fig. 2) présente d'une part les débits réellement exhaurés par le système forages plus galerie et d'autre part les débits estimés d'après les résultats du modèle mathématique.

Le fait d'avoir placé les crépines des pompes au voisinage du mur de l'aquifère a permis de simplifier les calculs en modèle car il devenait possible sans erreur trop importante de considérer la descenderie comme un drain traversant immédiatement la totalité de l'aquifère, (c'est à dire ayant un délai de réalisation quasi nul).

Lorsqu'une telle approximation n'est pas licite, il est toujours possible de procéder, comme dans l'exemple précédent, en imposant des conditions de niveaux variables par paliers.

FIGURE 2


50
40
30
DEBITS (l/s)
1
2
3
4
5
6
7
8
9
10
11
TEMPS (mois)
• Débit moyen mensuel ecoulé
× Débit moyen mensuel prévu

METRO DE LYON.
CONSTRUCTION SOUS LE NIVEAU DE LA NAPPE PHREATIQUE

Bouyat, C. ; Picod, G. ; Fages, R.

ABSTRACT : The construction of the first line of LYON's Underground Railway which structures enter in the aquifer of Rhône's alluvions needed a complete study of the hydrologic relations between the rivers and the ground water table. Expounding the relations ground water table-river level, the mathematical models used have permitted to estimate the reverbations on the water table of different river rises for determined points of the concerned field and also, the influence of the underground works on the flood of the ground water and the consequences on the water table after stopping important pumping in the ground.
These different calculations have permitted to define the water levels to take for each phase of the realization and the future exploitation of the works.

RESUME : La construction de la première ligne du Métro de LYON, dont les ouvrages pénètrent dans l'aquifère des alluvions du Rhône, a nécessité une étude approfondie des relations hydrologiques entre les fleuves et la nappe phréatique. En explicitant les relations nappe-fleuve, les modèles mathématiques mis au point ont permis d'estimer les répercussions sur la nappe de diverses crues fluviales en de points déterminés du domaine étudié, ainsi que l'influence de la présence des ouvrages du Métro sur l'écoulement de la nappe phréatique et les conséquences de l'arrêt de pompages importants au voisinage du métro.
Ces diverses simulations ont permis de définir les niveaux d'eau à prendre en compte à tous les stades de la réalisation et de l'exploitation future des ouvrages.

RESUMEN : La construcción de la primera linea del Metro de Lyon, bajo el acuífero de los aluviones del Ródano, ha requerido un estudio muy completo de las relaciones hidrológicas entre los ríos y el acuífero freático. Los modelos matemáticos puestos a punto, al hacer explícitas las relaciones acuífero-río, han permitido estimar las repercusiones de diversas crecidas fluviales sobre el acuífero, en puntos determinados del dominio estudiado, así como la influencia de las obras del Metro en el drenaje del acuífero freático, y las consecuencias de las paradas de bombeos caudalosos en la proximidad del Metro. Esas diversas simulaciones han permitido definir los niveles de agua a tener en cuenta, en todas las fases de la realización y de la futura explotación de las obras.

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I - INTRODUCTION

La plus grande partie de l'agglomération lyonnaise est construite au confluent de deux cours d'eau importants, le Rhône et la Saône, sur les alluvions sablo-graveleuses déposées par les rivières (figure n° 1). Ces alluvions sont le siège d'une nappe phréatique puissante dont le niveau piézométrique moyen se situe généralement entre 4 et 6 m sous le terrain naturel.

La construction du Métro de LYON, nécessairement souterrain dans cette agglomération urbaine dense, a dû s'accommoder de la présence de cet aquifère particulièrement perméable, de l'ordre de 10^{-2} à 10^{-4} m/s, en plaçant les ouvrages le plus superficiellement possible. Néanmoins, plus de quatre vingt pour cent des travaux ont dû se dérouler "les pieds dans l'eau", ce qui a conduit la S.E.M.A.LY. à lancer une importante campagne de relevés piézométriques et une étude détaillée du système aquifère pour préciser les niveaux piézométriques statistiques à prendre en compte dans les calculs et sur les chantiers.

PRÉSENTATION SOMMAIRE DES TRAVAUX

Après avoir déplacé les canalisations et divers ouvrages de Services Publics hors de l'emprise des travaux, les terrassements à ciel ouvert ont été faits à l'abri de palplanches métalliques ou de parois moulées.

La perméabilité des alluvions, 10^{-2} m/s à 10^{-4} m/s, ne permettait pas la mise hors d'eau des fouilles par simple pompage, la S.E.M.A.LY. a eu recours à trois techniques différentes pour s'affranchir des venues d'eau par le fond de fouille :

1 - mise en place d'un béton immergé faisant bouchon en fin d'excavation (figure n° 2)
2 - mise en place d'un radier injecté dans le tréfond de la fouille avant terrassement (figure n° 3)
3 - ancrage des parois moulées dans le substratum molassique (figure n° 4)

Outre les aspects techniques de choix entre les diverses solutions de soutènements en fonction de la largeur de la chaussée, le choix entre béton immergé, radier injecté ou ancrage à la molasse ressortait de considérations économiques dont la synthèse apparaît sur les figures 5 et 6 pour les conditions de prix des Travaux du Métro de LYON.

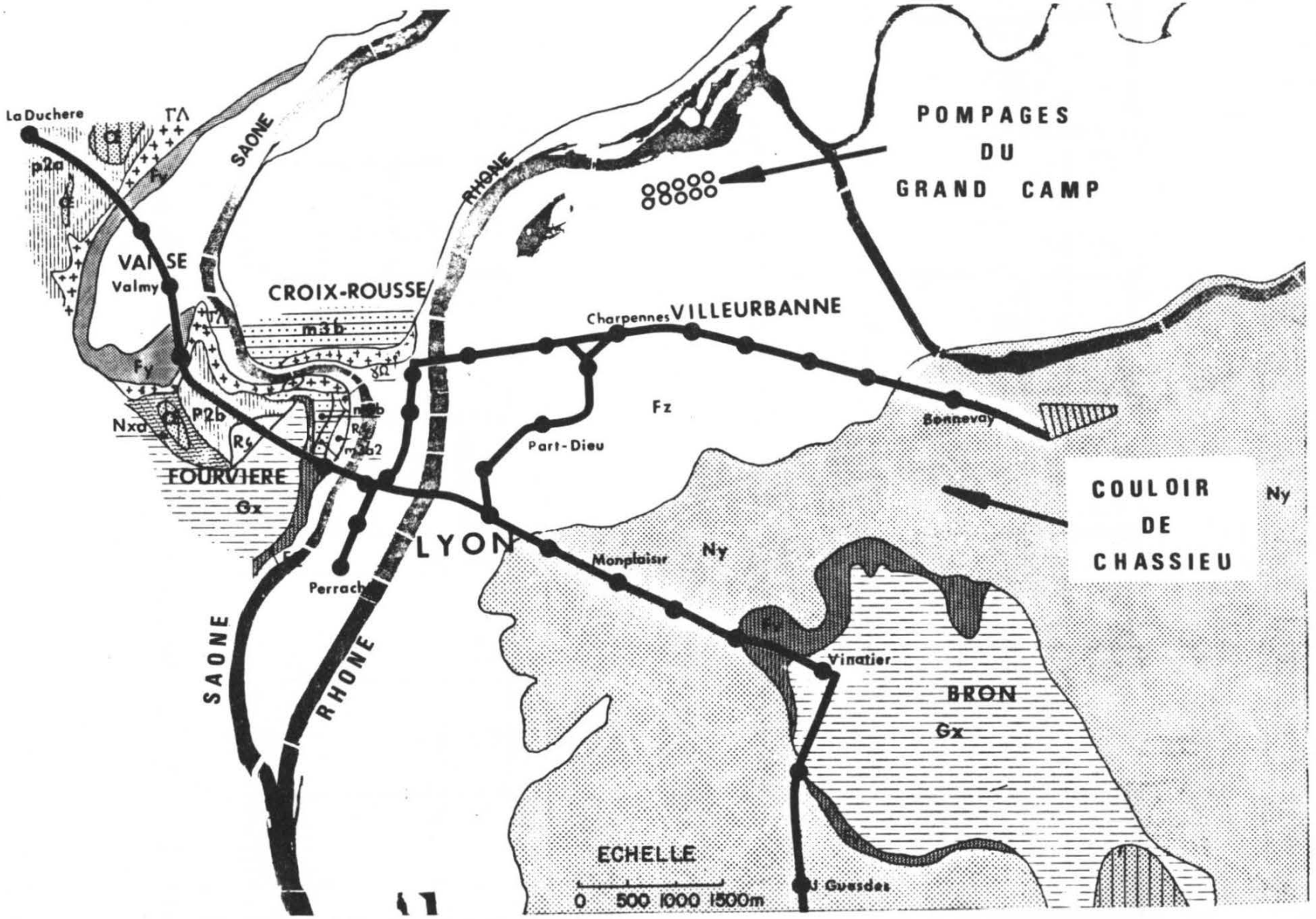


figure n° 1 - Situation hydrologique de l'Agglomération Lyonnaise

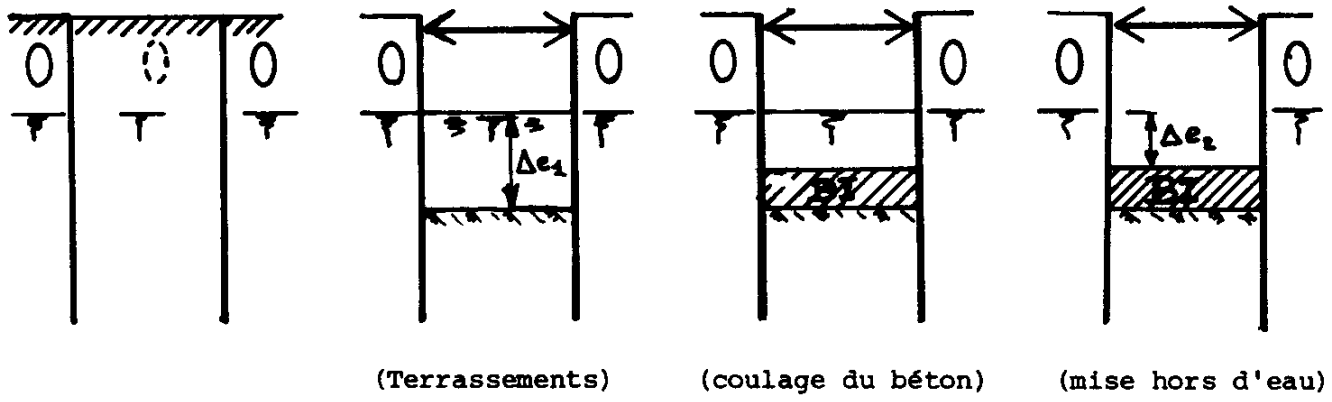


figure n° 2 - Mise hors d'eau de la fouille par bouchon de béton coulé sous l'eau .

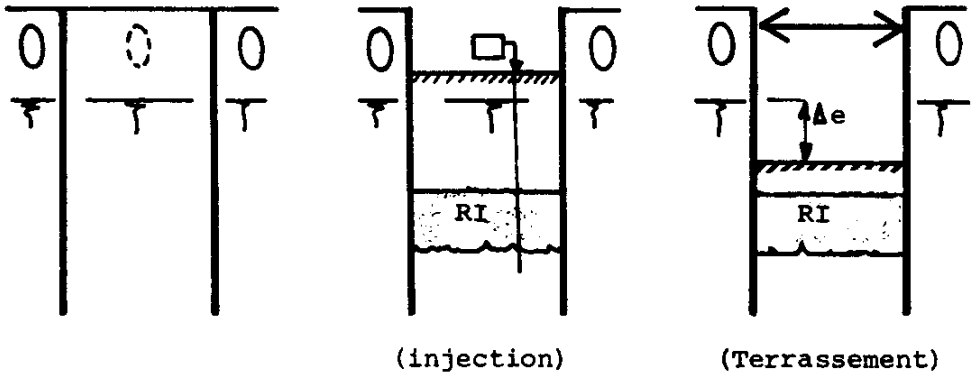


figure n° 3 - Mise hors d'eau de la fouille par bouchon de terrain injecté.

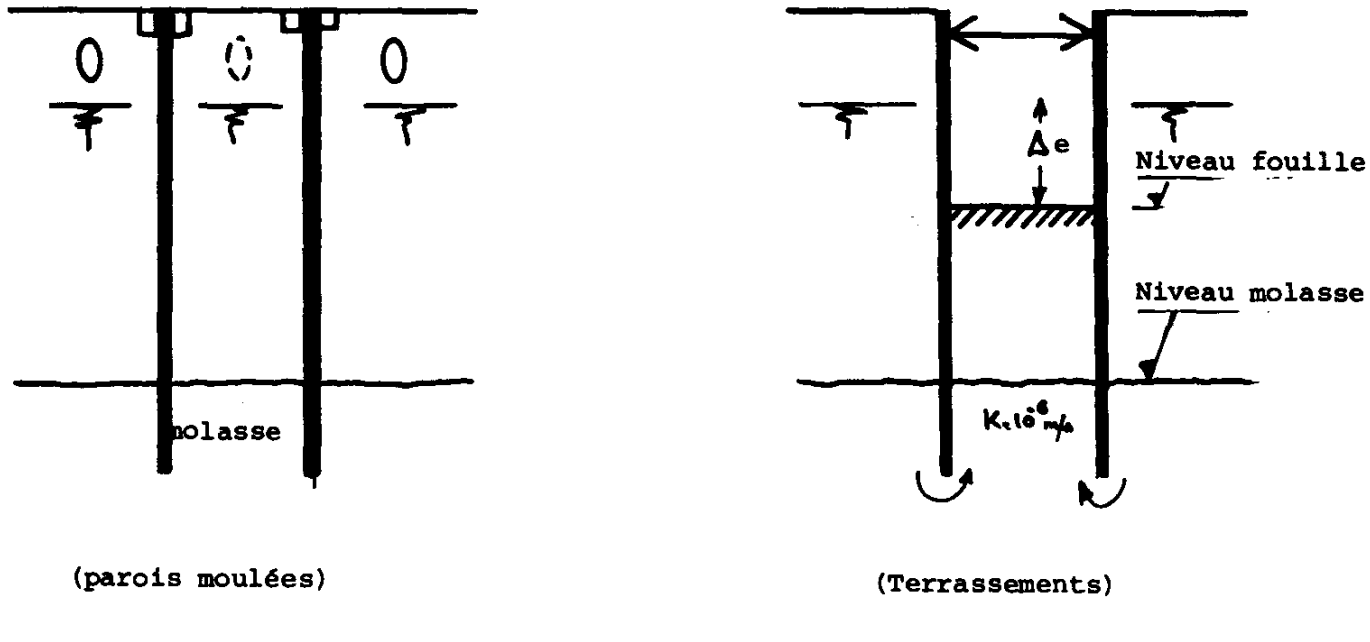


figure n° 4 - Mise hors d'eau de la fouille par rabattement à l'abri de parois moulées ancrées dans la molasse

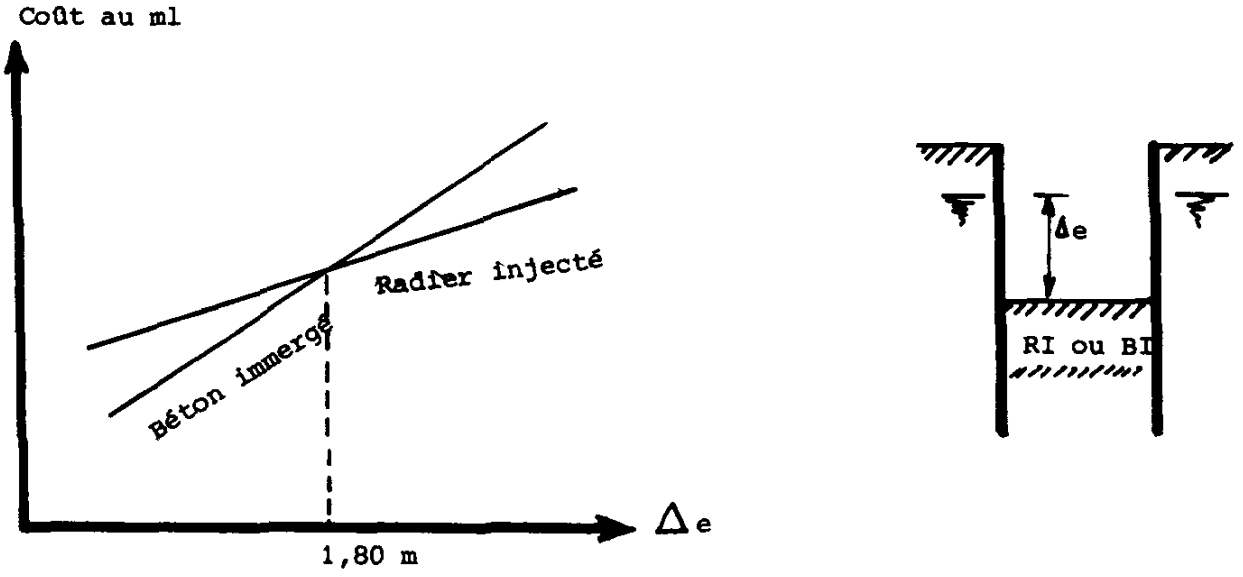


figure n° 5 - Choix entre béton immergé et radier injecté pour un soutènement en palplanches, en fonction de la charge d'eau Δe. (Métro de LYON - prix 1972)

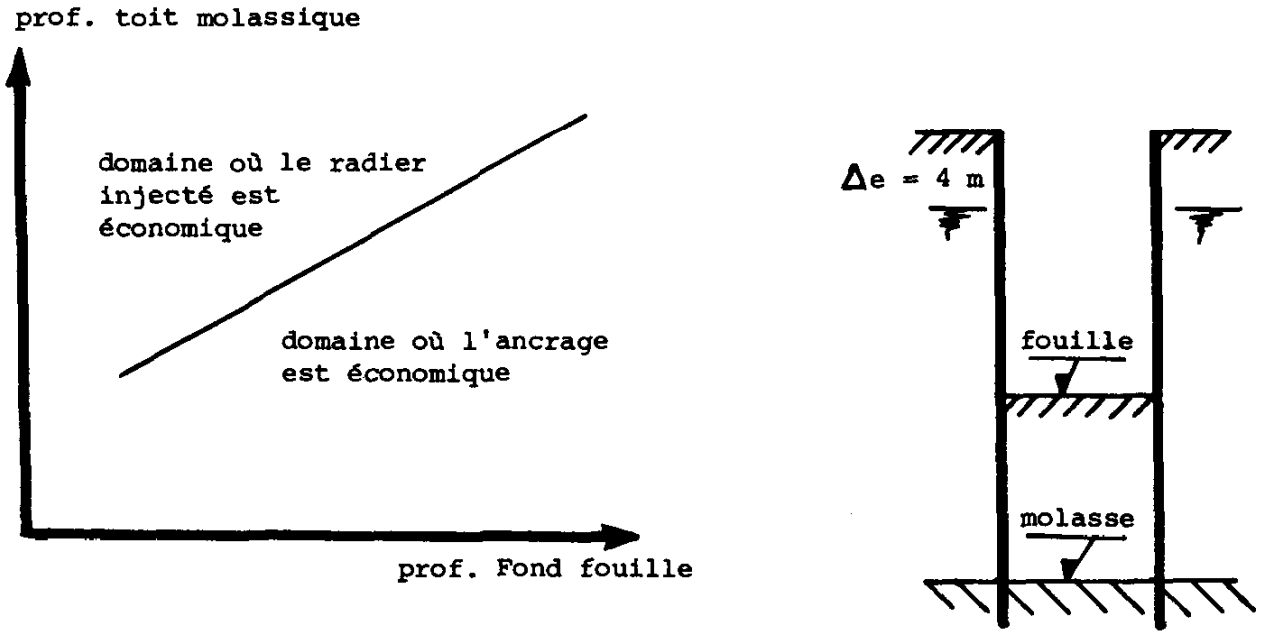


figure n° 6 - Choix entre l'ancrage des parois à la molasse et le radier injecté entre les parois moulées en fonction de la profondeur du fond de fouille et la profondeur du toit molassique. (Métro de LYON - prix 1972)

II - BUT DE L'ETUDE

L'importance considérable des sujétions que comptent les travaux en dessous du niveau piézométrique a conduit la S.E.M.A.LY. à entreprendre, dès la conception des ouvrages à construire, les études de l'évolution de la nappe phréatique de l'agglomération lyonnaise.

S'agissant de travaux très importants dans un site extrêmement perméable, il était tout d'abord nécessaire de s'entendre sur un vocabulaire adéquat et de clarifier les responsabilités de chacun dans le choix des niveaux à prendre en considération pour le dimensionnement des ouvrages et durant leur exécution.

Outre cet aspect de clarification, le but de l'étude a donc été de définir les niveaux à prendre en considération pour dimensionner les ouvrages vis à vis de leur régime d'exploitation et des sollicitations exceptionnelles, d'une part, et d'apporter, d'autre part, le maximum d'informations aux entrepreneurs pour qu'ils choisissent leurs risques de chantier avec la meilleure connaissance des niveaux statistiques.

En remarquant que la simple notion de "plus hautes eaux connues" n'a pas de sens statistique sur la nappe phréatique car il n'existe pas suffisamment de hauteurs de nappe observées, en particulier pour les niveaux "exceptionnels", l'étude faite aboutit à des valeurs CONVENTIONNELLES, obtenues par simulation.

Nous verrons plus loin pourquoi elles sont "conventionnelles".

Un point particulier important s'est greffé à cette étude par l'arrêt des pompages d'eau potable de l'Usine du Grand Camp de LYON. Bien que situés à proximité du Rhône, ces pompages de l'ordre de 8000 m^3/h provoquaient (ils sont aujourd'hui arrêtés) un rabattement de la nappe sensible sur le trajet du Métro de LYON et le réhaussement devait être prévu pour être pris en compte dans la stabilité des ouvrages et dans leur dimensionnement.

Les Niveaux à Considérer

Lorsqu'ils répondent à des préoccupations touchant les conditions d'exploitation des ouvrages à construire, les niveaux de nappe à définir sont de la responsabilité du Maître de l'Ouvrage : on parlera de l'inondation d'un parking au maximum une fois tous les dix ans par exemple ; on parlera du niveau de stabilité ultime de l'ouvrage aux poussées d'Archimède ; etc...

Que ces niveaux soient déterminés par un Bureau d'Etude extérieur ou par le Maître de l'Ouvrage lui-même, ces niveaux sont de sa responsabilité face au Maître d'Oeuvre et à l'Entrepreneur.

Ainsi pour les vérifications de stabilité et de résistance ultérieure de ses ouvrages, la S.E.M.A.LY. s'est orientéz vers la notion de TRES HAUTES EAUX CONVENTIONNELLES (THEC) en chaque point du réseau à construire, répercussion sur la nappe des évolutions du fleuve à ses "plus hautes eaux connues".

Ces THEC ne donnant pas automatiquement les sollicitations les plus défavorables pour des ouvrages remblayés où la poussée de l'eau a tendance à réduire les moments fléchissants en piédroits, cette remarque conduit à définir un niveau d'étiage de la nappe ou TRES BASSES EAUX CONVENTIONNELLES (TBEC).

Entre ces deux niveaux extrêmes, à caractère exceptionnel (faible probabilité d'apparition), il y a lieu de distinguer en général le niveau des "EAUX MOYENNES" (EM), le niveau des "HAUTES EAUX" (HE) et le niveau des "EAUX QUASI PERMANENTES" (EQP).

Ces niveaux sont liés à la vie normale de l'ouvrage en cours d'exploitation. Ce sera encore le Maître d'Ouvrage qui les définira.

Par exemple, pour les EQP, la S.E.M.A.LY. a retenu le niveau des eaux classées (nappe) dépassées trois cents jours par an. D'autres notions peuvent être retenues. Les HE pourront être définies comme le niveau de nappe dépassé au plus une fois tous les dix ans ; ou encore comme le niveau dépassé de moins d'une certaine hauteur d'eau pendant moins de x jours consécutifs ; etc.

Pour les calculs de ses ouvrages, la S.E.M.A.LY. a prescrit de considérer :

- les THEC et les TBEC comme des charges climatiques extrêmes
- les niveaux HE et EQP comme des charges d'exploitation variables autour des EM et subissant donc les majorations habituelles de charges variables.

Les ouvrages, subissant la poussée variable, généralement hydrostatique, de la nappe, il n'est pas inutile de rappeler que les pondérations pour charges variables s'effectuent, bien entendu, sur les variations de hauteurs d'eau (entre les HE et les EM par exemple) et non directement sur les poussées, erreur fréquente dans les calculs de dimensionnement.

Lorsqu'ils répondent à des préoccupations touchant au déroulement des chantiers, la responsabilité du choix des niveaux d'eau correspondants tient à la forme des marchés.

Dans le cadre de marchés sur appel d'offres, la responsabilité de ces niveaux est du ressort du Maître d'Oeuvre : ce sont des données pour l'Entrepreneur.

Dans le cadre de marchés sur concours, la concurrence entre les Entrepreneurs peut également porter sur le choix de ces niveaux eu égard aux risques que chacun admettra.

Les conditions d'exécution d'un chantier en site aquifère, résultent toujours d'un compromis d'ordre économique entre une évolution probable de la nappe, les délais et le coût des moyens mis en oeuvre : l'entrepreneur choisit, par exemple, entre

des soutènements provisoires pouvant s'opposer à toutes crues, et le coût moins élevé de soutènements plus légers avec prise en compte, ou non, d'éventuels arrêts d'équipes par immersion de la fouille. Il appartient au Maître d'Oeuvre d'en apprécier le réalisme et d'approuver éventuellement ces dispositions, par délégation du Maître d'Ouvrage.

Il est évident que ces dispositions de concurrence doivent apparaître dans les libellés des prix de rémunération (le forfait est à ce titre une disposition de concurrence).

Pour des travaux de radiers injectés par exemple, dont les dimensions doivent être décidées plusieurs mois à l'avance, la S.E.M.A.LY. a prescrit dans ses Cahiers des Charges la stabilité du fond de fouille jusqu'aux "Eaux à 300 jours", c'est-à-dire le niveau des eaux classées sur la nappe, dépassé, statistiquement, pendant 65 jours, non obligatoirement consécutifs, par an. En cas de dépassement, la fouille est immergée pour conserver sa stabilité et le délai de construction est prolongé d'autant de jours d'immersion et de nettoyage.

III - LES OUTILS DE SIMULATION

Afin de tenir compte de la présence d'une nappe phréatique sur le trajet de la Ligne A du Métro, la S.E.M.A.LY. a développé ses propres méthodes de calculs pour répondre aux questions suivantes :

1 - influence des fluctuations quotidiennes des deux fleuves sur la nappe phréatique, particulièrement sur le trajet de la Ligne A.

2 - définition des hauteurs d'eau à prendre en compte lors du calcul des ouvrages et lors des travaux.

3 - répercussion sur les hauteurs définies ci-dessus lors de l'arrêt (maintenant effectif) de la station de pompage du Grand Camp.

Une quatrième question s'est posée avec la mise en oeuvre de parois moulées ancrées à la molasse et barrant de ce fait, l'écoulement normal de la nappe : fallait-il s'attendre à un réhaussement effectif de la nappe à l'amont de ces écrans ?

Les questions ont été traitées différemment, suivant qu'elles fassent intervenir les fluctuations journalières de l'un ou des deux fleuves (questions 1 et 2), ou qu'elles correspondent à un effet pouvant être considéré constant sur toute la gamme des hauteurs de l'aquifère (questions 3 et 4).

1 - Simulation En Régime Transitoire

La méthode utilisée à consisté à simuler, jour après jour depuis 1913, l'évolution de la nappe phréatique en différents points caractéristiques du tracé du Métro. Suivant que l'on se trouvait sur la Presqu'île ou sur la Rive Gauche du Rhône, l'approche du problème a été un peu différente :

a) ETUDE ENTRE DEUX FLEUVES

La formulation fondamentale du modèle repose sur l'équation de la chaleur

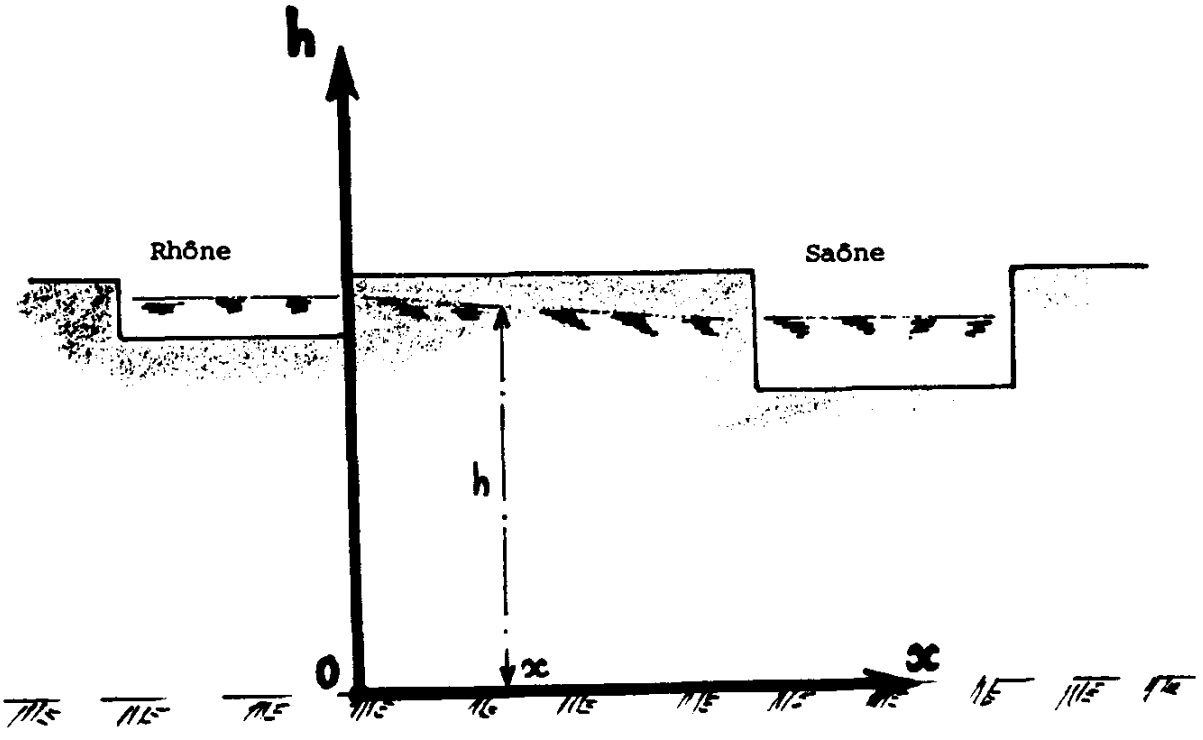
$$\frac{\partial^2 h}{\partial x^2} = \frac{1}{a^2} \frac{\partial h}{\partial t}$$

a^2 = diffusivité du terrain
x = distance du point de simulation au Rhône
h = puissance de la nappe.

dont on connaît la solution littérale pour les conditions aux limites de Cauchy :

$$h(x,t) = H_0(x,t) + H_1(x,t) + H_2(x,t)$$

H_0, H_1 et H_2 sont respectivement liés à la position initiale de la nappe, à l'influence du Rhône et à l'influence de la Saône.



Trois profils en travers ont été choisis en fonction de l'intérêt qu'ils présentaient pour les travaux et de la présence de piézomètres permettant un calage du modèle mathématique.

Les hypothèses simplificatrices suivantes ont été adoptées :

- les deux fleuves sont rectilignes, parallèles et horizontaux
- les profils choisis sont perpendiculaires aux fleuves
- le substratum est étanche et horizontal
- les caractéristiques hydrogéologiques (T/S) sont isotropes et homogènes.

b) NAPPE DE LA RIVE GAUCHE DU RHONE : UN SEUL FLEUVE

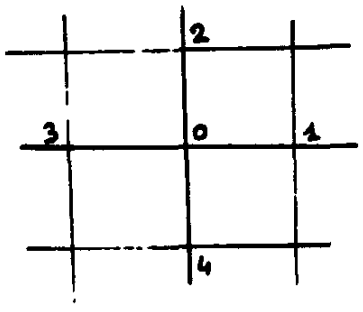
Le substratum relativement étanche et la surface libre de la nappe ayant des pentes voisines de l'horizontale, la loi de DARCY donne l'équation d'évolution théorique du niveau de la nappe :

$$\text{div } HK \overrightarrow{\text{grad } H} = S \cdot \frac{\partial h}{\partial t}$$

où
- K = perméabilité en un point
- S = coef. d'emmagasinement
- H = puissance de la nappe
- h = niveau de la surface libre

hypothèses : la perméabilité K est constante sur une verticale
les variations de h sont petites devant H.

La théorie des équivalences d'ABSI (°) permet la discrétisation dans l'espace de cette équation aux dérivées partielles sur un réseau à mailles carrées.



$$T_i = K_i \; H_i$$

$$2 \cdot \sum_{i=1}^{4} \frac{T_i}{T_0 + T_i} \cdot (h_i - h_0) = \frac{S}{T_0} \cdot \frac{\partial h}{\partial t} + \frac{q_0}{T_0}$$

q_0 = débit d'un puits éventuel situé au point O.

(°) E. ABSI et C. DHELLO : Théorie des Equivalences et Infiltrations de l'eau dans le sol. Annales I.T.B.T.P. - 1973 - (FRANCE) -

Les conditions aux limites sont mixtes :

- de type DIRICHLET sur les fleuves : le niveau h est connu par son évolution dans le temps.
- de type NEUMANN sur les frontières étanches (ici Sud et Sud-Est) $\frac{\partial h}{\partial n} = 0$

  (dérivée partielle suivant une direction normale à la frontière).

Plutôt que d'imposer, à chaque pas de un jour, la condition limite à la frontière-fleuve, opération longue et coûteuse, on a exploité la linéarité des équations en calculant, en chaque point de simulation, les fonctions d'influence des variations du fleuve sur la nappe. Ensuite, le programme de calcul travaille en différentiel, à partir des valeurs prises par le niveau journalier du Rhône entre 1913 et 1972.

2 - Simulation En Régime Permanent

Pour estimer les répercussions sur le niveau de la nappe phréatique de certains phénomènes, l'hypothèse simplificatrice de l'indépendance de ces répercussions vis à vis de l'évolution transitoire de la nappe a été adoptée au regard de la précision globale attendue.

C'est le cas des simulations des répercussions sur la nappe de l'arrêt des Pompages du Grand Camp et de l'effet de barrage partiel de l'aquifère résultant de la construction des ouvrages métro, en particulier lors de la mise en oeuvre de parois moulées ancrées dans le substratum étanche.

C'est une méthode de simulation parfois suffisante et économique. Sa mise en équation est simple : annulation de la dérivée partielle $\partial h / \partial t$ dans l'équation générale régissant le régime transitoire.

IV - LES DONNEES NECESSAIRES AUX SIMULATIONS

S'agissant de simulation des effets que produisent sur la nappe, en un point donné du système aquifère, les fluctuations des fleuves et autres apports ou départs d'eau sur la frontière du domaine étudié, il est nécessaire d'avoir la meilleure connaissance possible du système aquifère, quant à ses caractéristiques hydrologiques, et bien entendu d'être en possession d'un nombre aussi grand que possible de données sur les évolutions passées des fleuves et autres variations hydrauliques sur le domaine étudié.

a) LES DONNEES GEOTECHNIQUES

Outre les dispositions géométriques du site et la puissance de l'aquifère surmontant le substratum étanche, les données géotechniques nécessaires aux simulations portent essentiellement sur la perméabilité k du terrain et son coefficient d'emmagasinement S, pour chaque maille du quadrillage du domaine étudié.

Seuls des essais de pompage in situ peuvent donner un ordre de grandeur valable de ces caractéristiques hydrotechniques.

Bien que les performances des modèles actuels ne permettent pas de prendre en considération une variation de ces données le long de la verticale, il est utile de connaître les valeurs de k et de S à différentes profondeurs pour en obtenir une moyenne représentative de la zone concernée.

b) LES DONNEES HYDROLOGIQUES

S'il s'agit d'un système aquifère simple, alimenté par un seul fleuve sur une partie de la frontière, le domaine étant semi infini, la représentation de l'ensemble est relativement simple et sûre.

Quand il s'agit d'un domaine comportant des alimentations d'eau souterraines par des "couloirs" de déversement de bassins versants latéraux, la difficulté est grande pour apprécier leur grandeur ; de même des "soutirages" naturels dus aux écoulements de la nappe au travers des frontières. Quant aux pompages industriels éventuels, bien que leur déclaration auprès des Agences de Bassin soit obligatoire, leurs débits sont souvent fort mal connus (et généralement sous estimés pour des raisons fiscales).

La grande difficulté pour obtenir les données nécessaires au montage des simulations conduit à procéder au "calage" par approximations successives du résultat en fonction de relevés éffectifs des relations fleuve-nappe pour différentes hypothèses sur les grandeurs mal connues.

Ces relevés limnimétriques simultanés fleuve-nappe doivent être effectués régulièrement durant plusieurs mois pour être significatifs, d'autant plus longtemps que la perméabilité du terrain sera faible.

Une fois les paramètres de la simulation convenablement "calés" pour traduire au mieux les évolutions effectives de la nappe en fonction de celles du fleuve et éventuellement des apports ou soutirages déterminés, la simulation des répercussions d'évolutions conventionnelles du fleuve peut être engagée afin de prévoir les niveaux de nappe auxquels le chantier devra faire face notamment.

Par contre, pour le Bureau d'Etude et le dimensionnement des ouvrages, il est nécessaire de connaître les évolutions passées du fleuve pour faire l'analyse statistique de leurs répercussions sur la nappe. Dans ce cas, il faudra en général "actua-

liser" les données limnimétriques pour tenir compte des aménagements successifs du lit du fleuve, depuis les premiers relevés que l'on possède.

L'organigramme présenté page 713 rappelle le cheminement le plus général de ce travail de prévision statistique.

1 - Actualisation des Données Hydrologiques

Le calage des simulations étant établi pour des conditions hydrologiques données, l'étude de l'évolution piézométrique du niveau de la nappe phréatique doit être faite avec des données "cohérentes" avec celles de la période de calage adoptée, et les résultats obtenus seront exprimés en "valeurs" correspondantes à cette époque.

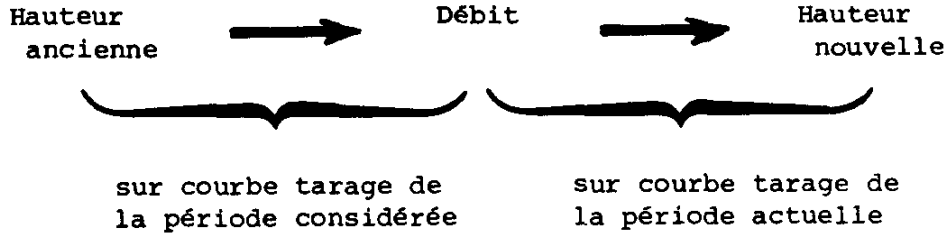
Il y a donc lieu d'actualiser les données hydrologiques utilisées dans la simulation, tant le niveau des fleuves alimentant la nappe que l'évolution des pompages industriels, que toute alimentation ou soutirage dans la nappe.

Certaines de ces actualisations résulteront du choix pur et simple de diverses hypothèses à titre de prévision des changements qui pourraient intervenir au cours de la période sur laquelle porte la simulation. D'autres résulteront d'une démarche plus scientifique, surtout pour l'établissement de résultats statistiques : c'est, en particulier, le cas de l'actualisation des données limnimétriques sur les fleuves.

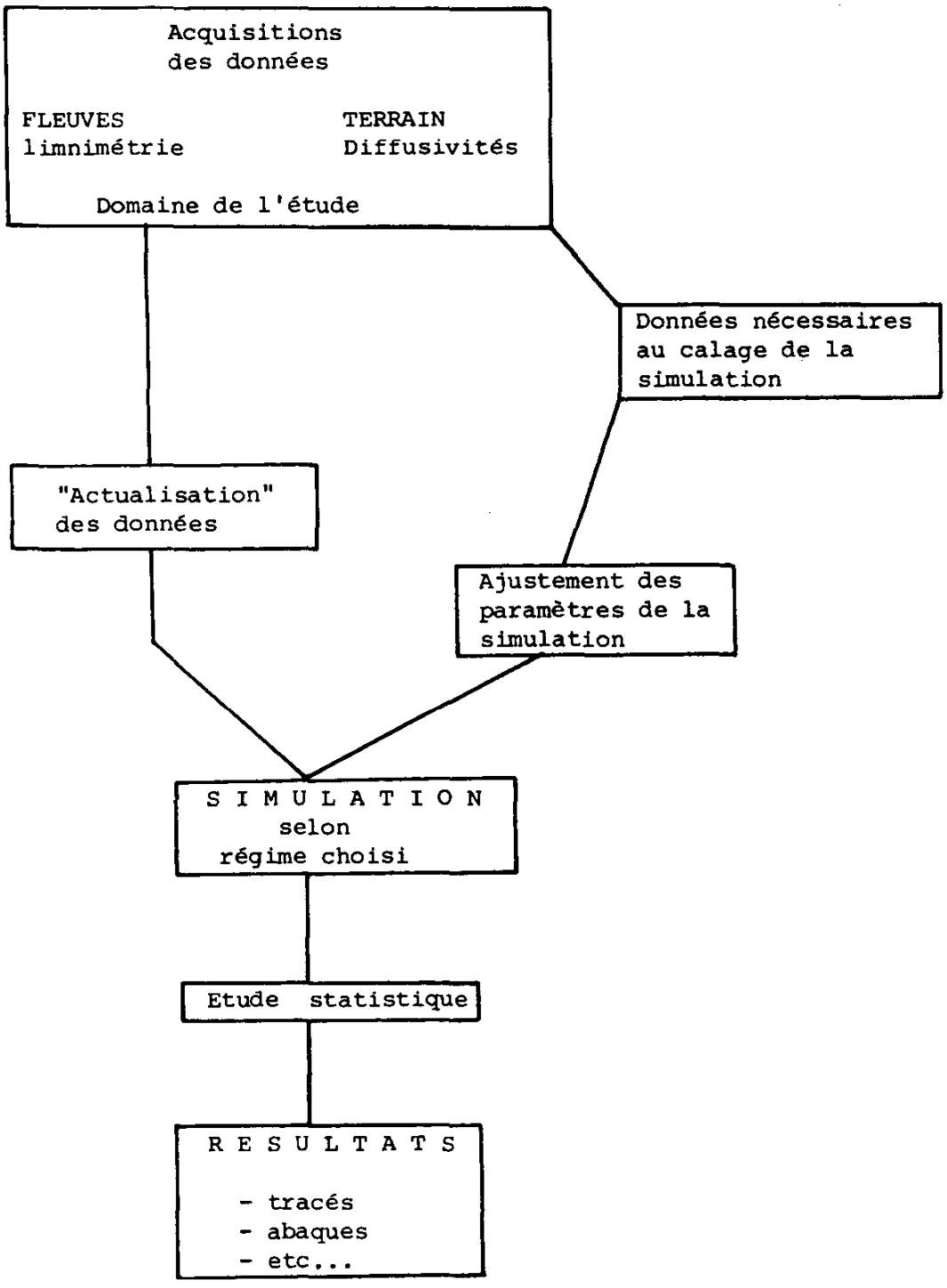
Actualisation des Données sur le Rhône

Pour la simulation des fluctuations passées de la nappe phréatique alimentée par le Rhône seul, l'actualisation des données a été particulièrement simple.

Le Service de la Navigation de LYON procédant à des relevés limnimétriques quotidiens depuis le début du XXème siècle, ainsi que les courbes de tarage Hauteurs-Débits correspondants aux aménagements successifs du lit du Rhône depuis 1913, l'actualisation des données limnimétriques s'est faite par la simple correspondance :



ORGANIGRAMME GENERAL DES SIMULATIONS



Outre l'hypothèse faite sur la véracité des côtes indiquées sur les relevés, cette solution est parfaitement fondée si l'on admet que les régimes hydrauliques des bassins versants du Rhône amont n'ont pas sensiblement changé depuis 1913.

En d'autres termes, cela signifie que les aménagements successifs qui ont régularisé le cours du Rhône n'en ont pas modifié le débit pluriannuel moyen. Ce qui paraît soutenable pour n'importe quel fleuve pour l'échelle humaine de temps.

ACTUALISATION DES DONNÉES SUR LA SAÔNE

Nous disposions là aussi de relevés limnimétriques journaliers du cours de la Saône depuis 1913 pour un point du domaine étudié. Toutefois nous ne disposions d'aucune courbe de tarage correspondante.

L'actualisation n'a donc pas pu se faire de façon aussi simple et aussi rigoureuse que pour le Rhône.

Après avoir tracé l'évolution journalière brute des côtes limnimétriques en notre possession, nous avons pu mettre en évidence (visuellement) des périodes apparemment homogènes, quant au régime d'étiage notamment, correspondant vraisemblablement à des conditions d'écoulement constantes (relation hauteurs-débits inconnue mais constante au cours de ces périodes).

Une étude statistique comparative basée sur la loi de GNEDENKO qui exprime l'écart absolu entre une fonction de répartition théorique et son approche empirique issue d'un échantillon de taille réduite pour une probabilité d'erreur définie, nous a permis d'établir les corrélations

Hauteurs anciennes ⟶ Hauteurs actuelles

en faisant les mêmes hypothèses que pour le Rhône.

2 - VALIDITÉ DES DONNÉES ACTUALISÉES - ASPECTS CONVENTIONNELS -

Une fois ce travail d'actualisation terminé, on peut se demander si les hauteurs trouvées sont véritablement les données à prendre définitivement en considération, ou si une deuxième intervention doit être opérée sur leurs valeurs.

Mise à part l'incertitude qui règne sur les résultats du calcul de leur actualisation, on est tenté de retoucher les valeurs ainsi trouvées sous prétexte d'aménagements du fleuve, en particulier pour les crues.

En effet, nombre d'ingénieurs estiment que les aménagements successifs des fleuves réduisent les fluctuations de leur niveau et "laminent" les crues. A notre avis, à moins de disposer de données techniques irréfutables, c'est une erreur d'accréditer cette idée ; surtout pour les crues les plus importantes. Certes, le réservoir d'un barrage peut, en se remplissant, atténuer les effets d'une crue dans la plaine en aval du barrage. Mais à de rares exceptions près, les consignes d'exploitation des barrages hydroélectriques visent à maintenir la hauteur de chute la plus forte, donc, à conserver un remplissage aussi élevé que possible pour prévenir une éventuelle sècheresse plutôt que de réserver, à tout instant, la possibilité d'"éponger" une crue. Il est en effet constaté, que des crues importantes, du moins sur le Rhône et sur la Saône, ont eu lieu de façon aléatoire à tous les mois de l'année. Cette réserve est donc généralement faible, et il suffirait qu'une deuxième crue suive rapidement la première pour que le barrage n'ait, de ce point de vue, aucune efficacité.

Pour autant qu'il n'y ait pas de risque d'inondation par débordement, il faut noter que si la présence d'un barrage peut abaisser, à son aval, le niveau de la nappe phréatique par abaissement du niveau du fleuve, la répercussion sur la nappe (c'est ce qui nous intéresse) des crues qui auront justement été "laminées" par le barrage, peut être supérieure à ce qu'elle aurait été en l'absence du barrage. En effet, par l'inertie des mouvements d'eau dans l'aquifère, une crue très haute mais de très courte durée peut n'avoir aucune répercussion sur le niveau de la nappe à une certaine distance du fleuve ; par contre, un réhaussement de moindre amplitude du fil d'eau pendant une longue durée peut y produire un réhaussement significatif de la nappe, ce qu'illustre la figure n° 7.

Ce point, nous amène à faire remarquer que la répercussion d'une crue décennale du fleuve sur le niveau de la nappe phréatique n'a absolument rien à voir avec le niveau que la nappe atteint ou dépasse une fois tous les dix ans.

Pour déterminer le niveau piézométrique DECENNAL DE LA NAPPE, il faut obligatoirement retracer l'évolution journalière de la nappe, pour en faire l'analyse statistique spécifique.

En ce qui concerne la traduction sur la nappe du concept des "plus hautes eaux connues" (PHEC sur le fleuve) il faut noter que l'on dispose rarement de données suffisantes sur le fleuve et sur la nappe sur une durée suffisante avant la crue. Nous sommes alors amenés à adopter une évolution arbitraire du fleuve avant le début

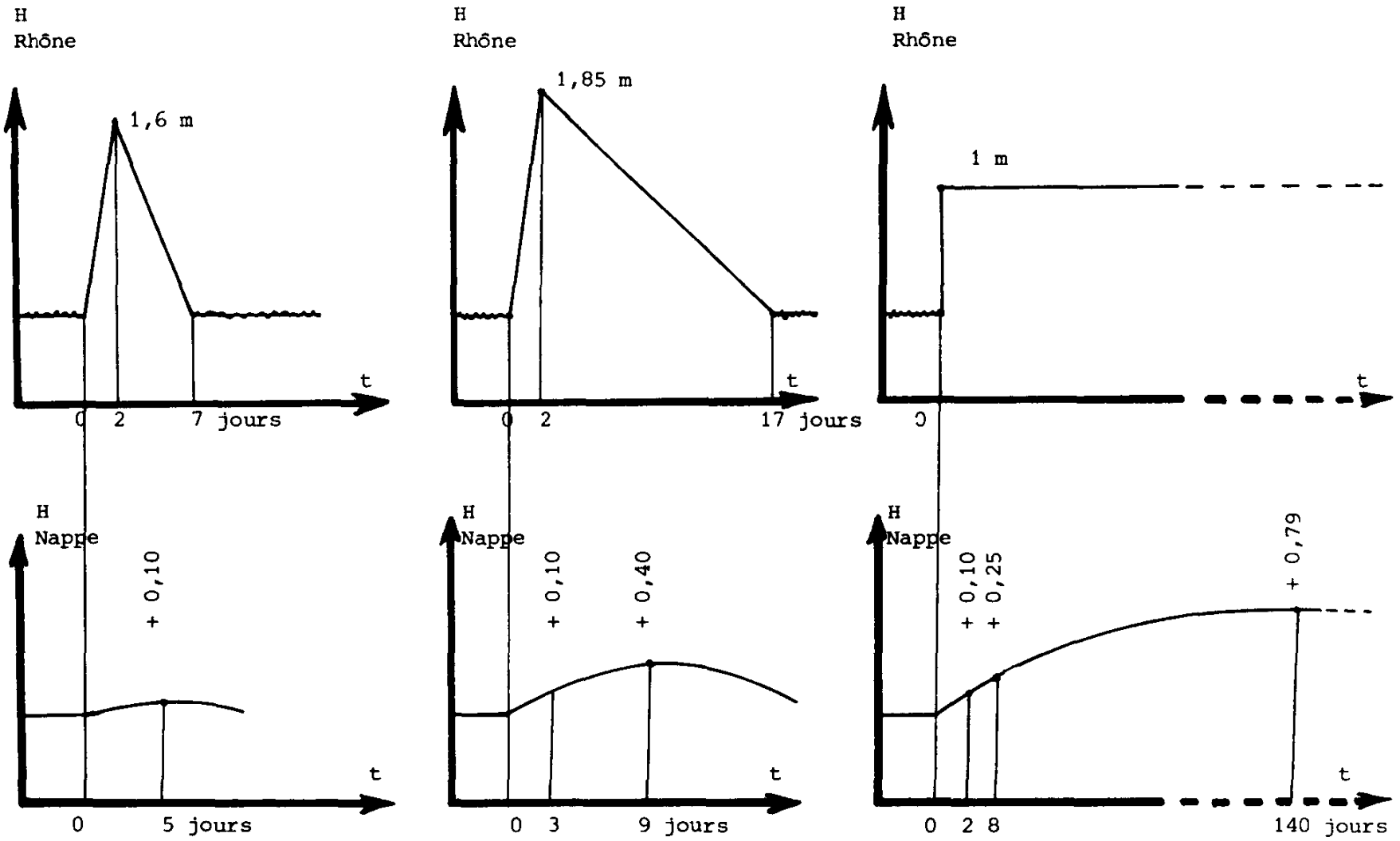


figure n° 7 - Comparaison des répercussions sur la nappe de trois évolutions types du Rhône, en un point situé à 0,8 km de distance du fleuve.

le la crue pour initialiser la simulation et déterminer un profil de nappe initial. Les résultats sont alors conventionnels (mais significatifs) et puisqu'ils ne correspondent pas forcément aux plus hautes eaux de la nappe qui auraient pu être constatées si des observations avaient été faites à l'époque (il s'agit bien du niveau piézométrique lié à l'infiltration et non au niveau d'eau provenant du débordement du fleuve), nous les désignons par : TRES HAUTES EAUX CONVENTIONNELLES.

V - QUELQUES POINTS PARTICULIERS

a) ARRET DES POMPAGES DU GRAND CAMP

Jusqu'au début de l'année 1977, la Compagnie des Eaux de la Ville de LYON exploitait la station de pompage du Grand Camp pour l'alimentation d'une grande partie de l'agglomération.

Bien que l'eau soit pompée à proximité immédiate du Rhône, quelques dizaines de mètres, le terrain graveleux sert essentiellement de filtre, le pompage qui s'élève à 180 000 m^3/jour en moyenne l'hiver, et peut atteindre 220 000 m^3/jour l'été, a une nette répercussion sur le niveau de la nappe phréatique, sans pouvoir dire quelle part de ces débits est prélevée dans la nappe proprement dite.

L'étude de l'arrêt des pompages devait donc permettre :

- de déterminer la zone d'influence des pompages
- de connaître les variations de l'aquifère consécutives à un arrêt de ces pompages.

La principale difficulté résidait dans le calage du modèle : en effet, il n'existe pas, dans la zone concernée, de relevés piézométriques comportant un véritable étiage (au moins un mois). De ce fait, les piézomètres relevés n'étaient pas suffisants.

Par ailleurs, certains paramètres ne pouvaient être estimés que de façon très approximative. En particulier, en ce qui concerne les apports du couloir glaciaire de CHASSIEU qui se déversent dans le domaine étudié, et les apports d'eau par le substratum dit "étanche".

Ne pouvant obtenir un calage correct de la simulation sur des piézomètres relevés - il était pratiquement impossible de faire varier les débits de pompage pour en étudier les répercussions - on a tourné la difficulté en opérant une injection d'eau correspondant aux débits prélevés et on en a étudié les répercussions, le réhaussement de la nappe donnant alors un ordre de grandeur de la dépression provenant des pompages.

Il faut dire que cette méthode de simulation s'est révélée particulièrement efficace puisque les observations faites lors de l'arrêt effectif des pompages ont démon-

tré la validité des prévisions faites par la S.E.M.A.LY.

Pour un réhaussement estimé à 50 cm pour le quartier de la Part-Dieu à LYON (au lieu de 1,50 m annoncé par une étude précédente) augmenté des 20 cm correspondant à l'effet des parois moulées barrant l'écoulement de la nappe dans ce secteur, le réhaussement effectif n'a pas dépassé 80 cm par rapport à des étiages comparables avant et après l'arrêt des pompages.

b) INFLUENCE DE LA PRESENCE DES OUVRAGES SOUTERRAINS

Les conséquences de la présence d'une ou de plusieurs lignes de Métro dans la nappe phréatique de la plaine de l'Est lyonnais sont peu importantes, particulièrement en ce qui concerne la première ligne (réhaussements de l'ordre de 60 cm au maximum).

Des relevés piézométriques récents, effectués de part et d'autre de l'ouvrage n'ont fait apparaître aucune perturbation notable (résultats égaux à plus ou moins 10 cm).

Traitée de façon théorique, la propagation d'une onde de crue ne paraît pas gênée par la présence des ouvrages.

VI - CONCLUSION

Les outils qui ont été mis au point par la S.E.M.A.LY. se sont révélés particulièrement sûrs dans leurs résultats, mais il convient d'attirer l'attention sur la taille des modèles de simulation de nappe et sur les erreurs qui peuvent en résulter. Etudiant le réhaussement prévisible de la nappe du fait de l'arrêt des pompages du Grand Camp à LYON, une première simulation montrait que l'isopièze + 1,5 m passait aux environs de la Part-Dieu. Déplaçant la frontière du modèle de calcul du cours Lafayette jusqu'au Sud de Gerland (3 km plus loin) le réhaussement n'est plus estimé qu'à + 0,5 m en ce même endroit, ce qu'ont confirmé les observations ultérieures. C'est dire l'importance des moyens de calcul à mettre en oeuvre pour ne pas commettre de graves erreurs.

Par ailleurs, du fait même de l'inertie de la nappe, les simulations faites en régime permanent, donnent des estimations souvent grossières (conditions aux limites absolument fixes par rapport aux simulations faites en régime transitoire (simulation pour un point géographique donné en prenant en compte la propagation de l'onde de crue à travers l'aquifère séparant le point étudié de la "source"). Les résultats peuvent être très différents et les plus-values pour l'étude la plus poussée, quand elle est possible (régime transitoire), seront souvent justifiées par les économies de construction (plusieurs millions de francs dans la construction du Métro de LYON).

En ce qui concerne l'expression des résultats, si les formes statistiques conviennent parfaitement aux préoccupations du Bureau d'Etude, les besoins des gens de chantier sont tout autres et l'expression des résultats selon la forme de la figure n° 7 leurs sont plus utiles. Ils leur permettent d'apprécier directement les effets auxquels ils doivent s'attendre d'une crue du Rhône pour tel ou tel chantier (géographique) : pour diverses évolutions types du Rhône, un tableau leur fourni le nombre de jour dont ils disposent après le début d'une crue pour voir la nappe monter de 10, 25 ou 50 cm, le nombre de jour pendant lequel la nappe continuera à monter après l'amorce de la décrue et quel sera le maximum atteint.

Avec de tels renseignements dont l'ajustement peut être fait rapidement dès qu'une crue ne ressemble pas à celles considérées dans les cas types, les gens de chantier peuvent être parfaitement maîtres des décisions à prendre et les délais dont ils disposent pour agir pour la meilleure conduite de leurs chantiers.

Vue d'ensemble de l'Agglomération Lyonnaise

DRENAJE DE UNA MINA HASTA NIVEL CONSTANTE

Fernández-Rubio, R.* ; Yagüe Ballester, A.**

ABSTRACT : During mining working, it is ordinarily required to drain in order to obtain a constant piezometric level. Flow is therefore variable. Thus, conventional methods of pumping test interpretation are not applicable to the observed water table lowering. On the basis of the constant head depression, we present a computation and interpretation method, which takes under consideration the observations in different piezometers. In order to facilitate the data treatment, we present a FORTRAN-computer program.

RESUME : Dans les travaux miniers, il faut réaliser le drainage, à débit variable, imposé par la nécessité d'obtenir une dépression du niveau piézomètrique qui permette l'exploitation. On ne peut appliquer, aux rabattements piézomètriques obtenues dans ces conditions, les méthodes habituelles d'application. Pour cette raison, nous avons développé une méthode de calcul et d'interprétation, basée sur le pompage à rabattement constant, avec des observations réalisées sur divers piézomètres de contrôle. On présente également un programme d'ordinateur, codifié en FORTRAN, qui facilite le traitement des données.

RESUMEN : En las labores mineras el drenaje se tiene que realizar, normalmente, a caudal variable, impuesto por la necesidad de conseguir una depresión del nivel piezométrico que permita el laboreo. A los descensos piezométricos obtenidos en estas condiciones no se les puede aplicar los métodos usuales de interpretación. Por ello hemos desarrollado un método de cálculo e interpretación, basado en el bombeo a descenso constante, con observaciones realizadas en varios piezómetros de control. Se presenta, también un programa de ordenador, codificado en FORTRAN, que facilita el tratamiento de los datos.

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PLANTEAMIENTO

El caso más habitual de captaciones de aguas subterráneas, es el que se efectúa a caudal constante, compatible con los recursos del acuífero; pero cuando se trata de realizar el drenaje de una mina, o de una excavación cualquiera, bajo nivel potenciométrico, lo normal es tener que establecer un régimen de bombeo de depresión constante, consistente en extraer todo el agua que afluye hasta un cierto nivel, impuesto por las labores subterráneas, o por la conveniencia de drenar el subsuelo.

Este régimen implica una reducción progresiva de caudal, a lo largo del tiempo, debido a la afección de las reservas, y al mantenimiento de dicha depresión constante, y trae consigo la no aplicabilidad de las fórmulas habituales de la Hidrogeología, desarrolladas para bombeos a caudal constante, en régimen permanente o transitorio.

JACOB Y LOHMAN (1952) desarrollaron unas fórmulas aplicables a este caso, para el cálculo de la transmisividad (T), y el coeficiente de almacenamiento (S), a partir de las observaciones realizadas en el punto de bombeo.

Ahora bien, puesto que con frecuencia se cuenta con observaciones de control potenciométrico en otros puntos, y además en la realidad es normal el que se produzcan interrupciones en el bombeo, o variaciones no controladas de caudal, hemos desarrollado un método de cálculo aplicable a este caso más general.

DEPRESION CONSTANTE SIN FILTRACION VERTICAL (OBSERVACIONES EN EL PUNTO DE BOMBEO

JACOB Y LOHMAN (1952), basados en las analogías entre las condiciones hidrológicas de un acuífero, y las térmicas de un sistema termal equivalente, han llegado a la resolución de la ecuación de la continuidad :

$$\frac{1}{r}\frac{\partial h}{\partial r} + \frac{\partial^2 h}{\partial r^2} = \frac{S}{T}\frac{\partial h}{\partial t}$$

para un bombeo en el cual el descenso permanece constante, y en el caso de un acuífero ideal homogéneo, isotrópico, de extensión areal infinita, y en el que T y S son constantes en todo tiempo y lugar.

Así, para

s_w = depresión constante en el pozo bombeado
r_w = radio efectivo del pozo bombeado

puede escribirse :

$$Q = 2\pi T s_w G(\alpha)$$

donde

$$\alpha = \frac{T\ t}{S\ r_w^2}$$

$$G(\alpha) = \frac{4\alpha}{\pi} \int_0^{\infty} x\ e^{-\alpha x^2} \left[\frac{\pi}{2} + tg^{-1} \left(\frac{Y_o(x)}{J_o(x)} \right) \right] dx \qquad (1$$

Los términos $J_o(x)$ e $Y_o(x)$ son funciones de Bessel, de orden cero, de primera y segunda clase, respectivamente.

La ecuación (1 no es tratable por integración, pero puede resolverse numéricamente, al reemplazar la integral por una sumación.

Así han sido tabulados los valores de $G(\alpha)$ (denominada función de pozo para depresión constante), correspondientes a valores de α desde 10^{-4} hasta 10^{-6} (JACOB Y LOHMAN, 1952).

Para determinar, gráficamente, los parámetros del acuífero (T y S), se dibuja, en papel semilogarítmico, la curva tipo de valores de $G(\alpha)$ versus α, y en papel transparente, de igual módulo semilogarítmico, los valores experimentales de Q versus t. La superposición conforme de ambas curvas, por traslación paralela de los ejes, y la elección arbitraria de un punto de acuerdo, nos da las parejas de valores :

$$\begin{matrix} G(\alpha) & Q \\ \alpha & t \end{matrix}$$

con los cuales se obtiene :

$$T = \frac{Q}{2\pi\ s_w\ G(\alpha)} \qquad (2$$

$$S = \frac{T\ t}{r_w^2 \cdot \alpha}$$

METODO SIMPLIFICADO

Como una simplificación a este método, JACOB Y LOHMAN (1952), han demostrado que para valores elevados de t, la función $G(\alpha)$, puede ser reemplazada muy exactamente por 2 / W(u), en la que W(u) es la función de pozo de Wenzel, para la resolución gráfica por el método de Theis. Igualmente han demostrado que, para valores de u suficientemente pequeños, W(u) puede ser reemplazada muy aproximadamente por

$$2.30\ \log_{10} \frac{2.25\ T\ t}{r_w^2\ S}$$

Al sustituir en (2, queda :

$$T = \frac{2.30\ Q}{4\pi s_w} \log_{10} \frac{2.25\ T\ t}{r_w^2\ S} \qquad (3$$

la cual puede escribirse :

$$\frac{Q}{s_w} = \frac{2.30}{4\pi T} \log_{10} \frac{t}{r_w^2} + \frac{2.30}{4\pi T} \log_{10} \frac{2.25\ T}{S}$$

Así pues, si se representan, en escala aritmética, los valores de Q/s_w, en función de los valores de t/r_w^2, en escala logarítmica, estos puntos deben de alinearse sobre una recta, cuya pendiente permite calcular T :

$$\text{pendiente} = \frac{\Delta\,(s_w/Q)}{\Delta \log_{10}(t/r_w^2)} = \frac{2.30}{4\pi T}$$

si se toma la abcisa $\Delta_{10}\ (s_w/Q)$, correspondiente a un ciclo logarítmico :

$$\Delta \log_{10}(t/r_w^2) = 1$$

y queda

$$T = \frac{2.30}{4\pi\,\Delta_{10}(s_w/Q)} \qquad (4$$

El coeficiente de almacenamiento puede calcularse al sustituir, en la ecuación (3, las coordenadas de cualquier punto de la recta, y el valor de T obtenido. En especial, si se toma, gráficamente, la intersección de la recta, con el eje $s_w/Q = 0$:

$$S = 2.25\ T\ (t/r_w^2)_o \qquad (5$$

Según los autores mencionados el método es adecuado para el cálculo de T, pero debe de tomarse con precaución para calcular S, debido a la dificultad de conocer el radio efectivo del pozo bombeado.

BOMBEO EN REGIMEN DE CAUDAL VARIABLE (OBSERVACIONES EN VARIOS PIEZOMETROS)

Como hemos expuesto anteriormente, este es un caso más general, en el desagüe de una mina o excavación, y a él le hemos aplicado el siguiente proceso de cálculo.

Transcurridos tiempos suficientes para que se cumpla la simplificación de Jacob, correspondiente a cada escalón de bombeo, y en cada piezómetro x, podemos escribir :

$$\Sigma x_n = \frac{0.183}{T} \Sigma \left(Q_n \log \frac{2.25\ T\ t_n}{S\ rx^2}\right) \quad (6$$

donde :

x_n = Depresión en el piezómetro x, transcurrido un tiempo t_n
r_x = Distancia desde el piezómetro hasta el punto de bombeo
Q_n = Incremento en el caudal de bombeo desde el tiempo t_n

La ecuación (6 podemos escribirla :

$$\Sigma x_n = \frac{0.183}{T} \Sigma \left(Q_n \log_{10} \frac{2.25\ T}{S}\right) + \frac{0.183}{T} \Sigma \left(Q_n \log_{10} \frac{t_n}{r_x^2}\right)$$

de donde :

$$\frac{\Sigma x_n}{\Sigma Q_n} = \frac{0.183}{T} \log_{10} \frac{2.25\ T}{S} + \frac{0.183}{T} \frac{\Sigma \left(Q_n \log_{10} \frac{t_n}{r_x^2}\right)}{\Sigma Q_n} \quad (7$$

la representación de $\frac{\Sigma x_n}{\Sigma Q_n}$ en función de $\frac{\Sigma \left(Q_n \log_{10} \frac{t_n}{r_x^2}\right)}{\Sigma Q_n}$ (8

en escala aritmética, para las observaciones realizadas en todos los piezómetros y en cualquier tiempo (diagrama global), debe dar una recta de pendiente angular 0.183/T, que permite calcular T.

Si tomamos la ordenada en el origen, en dicha recta :

$$\left(\frac{\Sigma x_n}{\Sigma Q_n}\right)_o = \frac{0.183}{T} \log_{10} \frac{2.25\ T}{S}$$

de donde :

$$S = \frac{2.25\ T}{10^{\frac{T}{0.183}\left(\frac{\Sigma x_n}{\Sigma Q_n}\right)_o}}$$

Aparentemente un problema que presenta esta metodología estriba en la complejidad de cálculo, pero en la realidad este cálculo es perfectamente abordable con un programa de ordenador, a partir de las observaciones de caudales y niveles potenciométricos, realizadas a lo largo del tiempo, tenidas en cuenta las distancias de estos piezómetros al punto de bombeo.

En nuestro caso se ha puesto a punto un programa, para el cálculo y dibujo de estas expresiones, que sigue el diseño estructurado (IBM, 1976) de la *figura 1* y cuyo listado se halla en la tabla 1 . Se comienza con la lectura de los datos

relativos a la descripción inicial del sistema de observación y de los descensos, con sus respectivos caudales y fechas en que se han tomado medidas. A continuación los datos se pasan a magnitudes absolutas y se calculan, en cada piezómetro y para cada dato considerado, la T y S según la ecuación 7. Por último se dibuja, según la subrutina DIBUJA, las parejas de valores indicadas en 8.

Caso de ser varios los puntos de bombeo, sería necesario aplicar el método con la suma, en cada punto de los descensos provocados individualmente por cada bombeo.

Un interés adicional de esta representación estriba en la posibilidad de controlar anomalías, que se evidenciarían en dicha representación lineal, y que podrían responder a la existencia de barreras positivas o negativas, en el acuífero, o a la alimentación inducida por el bombeo, consecuencia de un régimen multicapa, o al error de alguna medición o al más funcionamiento (colmatación) de algún piezómetro.

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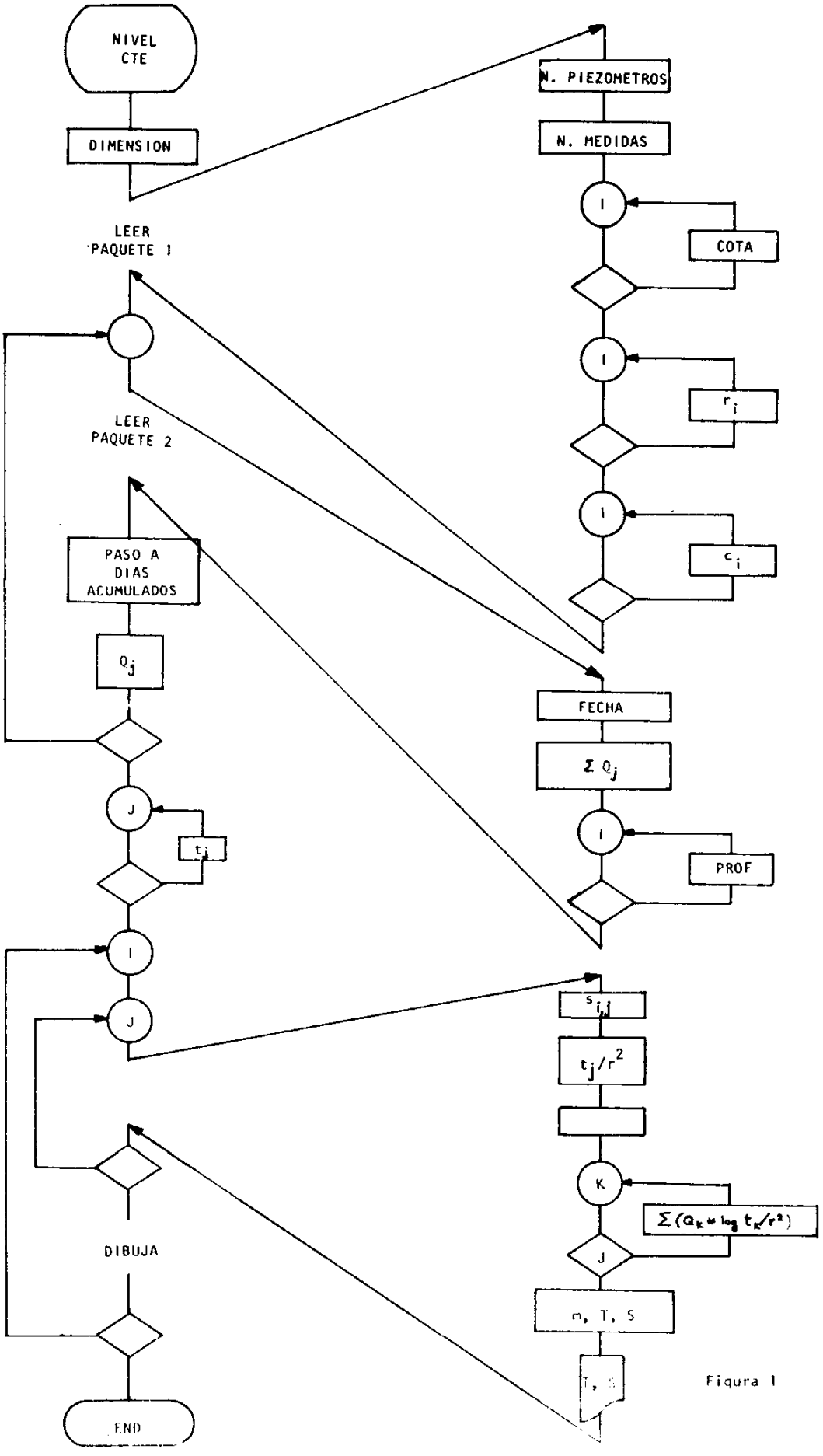
NIVEL CTE
DIMENSION
LEER PAQUETE 1
LEER PAQUETE 2
PASO A DIAS ACUMULADOS
Q_j
J
t_j
I
J
DIBUJA
END
N. PIEZOMETROS
N. MEDIDAS
I
COTA
I
r_i
I
c_i
FECHA
ΣQ_j
I
PROF
$s_{i,j}$
t_j/r^2
K
$\Sigma(Q_k * \log t_k/r^2)$
J
m, T, S
T, S

Figura 1

```
C      DRENAJE HASTA NIVEL CONSTANTE
C      I.. .....PIEZOMETRO NUMERO
C      J.. .....NUMERO DE PARTE
C      IANO.....ANO EN QUE SE MIDE
C      MES .....MES EN QUE SE MIDE
C      IDIA.....DIA EN QUE SE MIDE
C      CAUDAL(J)......LECTURA DEL CONTADOR EN EL PARTE J
C      PROF(I,J)......PROFUNDIDAD PIEZOMETRICA (PARTE J, PIEZOMETRO I)
C      C(I)..........PROFUNDIDAD PIEZOMETRICA INICIAL (PIEZOMETRO I)
C      XNIVEL(I,J)....DESCENSO PIEZOMETRICO (PARTE J, PIEZOMETRO I)
C      COTA(I).......COTA DE LA BOCA DEL PIEZOMETRO
C      NN. .....NUMERO TOTAL DE PIEZOMETROS
C      NPT .....NUMERO TOTAL DE PARTES CONSIDERADOS
C      XLONG(I)......DISTANCIA DEL PIEZOMETRO AL POZO DE BOMBEO
       DIMENSION COTA(10),C(10),PROF(10,100),XNIVEL(10,100),CAUDAL(100),
      -XLONG(10),TIEMPO(100),TP(100),DIFCAU(100),RR1(100),RR2(100),
      -ALINE(100),AMAXI(10),KSTEP(100)
       READ(5,1)NN,NPT,(COTA(I),I=2,5),(XLONG(I),I=2,5),(C(I),I=2,5)
1      FORMAT(2I3,R,4F8.2,R,4F8.2,R,4F8.2)
       J=1
2      READ(5,3,END=4)IANO,MES,IDIA,CAUDAL(J),
      -I,PROF(I,J),I,PROF(I,J),I,PROF(I,J),I,PROF(I,J)
3      FORMAT(2X,3I2,3X,F6.0,12X,4(I2,F4.0))
C      PASO DE FECHAS A DIAS ACUMULADOS
       TDIAS=(IANO-76)*365
C      FECHA INICIAL DE BOMBEO
       IF(TDIAS.LE.0)TDIAS=0
       IF(IANO.EQ.76)TDIAS=122
       IF(MES.EQ.1 )NDIAS=0
       IF(MES.EQ.2 )NDIAS=31
       IF(MES.EQ.3 )NDIAS=59
       IF(MES.EQ.4 )NDIAS=90
       IF(MES.EQ.5 )NDIAS=120
       IF(MES.EQ.6 )NDIAS=151
       IF(MES.EQ.7 )NDIAS=181
       IF(MES.EQ.8 )NDIAS=212
       IF(MES.EQ.9 )NDIAS=243
       IF(MES.EQ.10)NDIAS=273
       IF(MES.EQ.11)NDIAS=304
       IF(MES.EQ.12)NDIAS=334
       TDIAS=TDIAS+NDIAS+IDIA
       TP(J)=TDIAS
       DIFCAU(J)=(CAUDAL(J)-CAUDAL(J-1))/(TP(J-1)-TP(J))
       J=J+1
       GO TO 2
4      DO 10 J=1,NPT
10     TIEMPO(J)=TP(NPT)-TP(J-1)
       DO 60 I=2,5
       WRITE(6,20)
20     FORMAT('1',10X,'TRANSMISIVIDAD',6X,'ALMACENAMIENTO',/,10X,34('-'))
       XNIVEL(I,0)=0.
       HO=COTA(I)-C(I)
       DO 40 J=1,NPT
       XNIVEL(I,J)=HO-COTA(I)+(PROF(I,J)/100.)
       RR1(J)=TIEMPO(J)/XLONG(I)**2
       E=0
       DO 30 K=1,J
30     E=E+DIFCAU(K)*ALOG10(RR1(K))
       RR2(J)=E
       XM=(RR2(J)-RR2(J-1))/(RR1(J)-RR1(J-1))
       TX=0.183/XM
       A=(XNIVEL(J)-RR2(J))*XM/(DIFCAU(J)*XM)
       B=ALOG10(ABS(2.25*TX))-A
       SX=10**B
40     WRITE(6,50) TX,SX
50     FORMAT(10X,G12.4,8X,G12.4)
       NG=1
60     CALL DIBUJA(NG,NN,RR1,KSTEP,RR2,ALINE,AMAXI)
       STOP
       END
```

```
      SUBROUTINE DIBUJA(NN,NPUNTO,XMATR,KSTEP,R,ALINE,AMAXI)
      DIMENSION R(100),XMATR(100),KSTEP(100),ALINE(100),AMAXI(10)
      DO 50 L=1,NN
      NP=NPUNTO
      PEQUE=XMATR(1)
      XMAX=XMATR(1)
      DO 2 I=2,NP
      IF(PEQUE-XMATR(I))1,1,:
      PEQUE=XMATR(I)
    1 IF(XMAX-XMATR(I)).2,2 .
      XMAX=XMATR(I)
    2 CONTINUE
    4 MM=100
      DO 5 I=1,NP
      STEP=(100*XMATR(I)/XMAX)-(100*PEQUE/XMAX)
    5 KSTEP(I)=STEP+0.5 :
    8 WRITE(6,9)
    9 FORMAT(1H1)
      AMINI=R(1):
      DO 10 I=1,NP
      IF(R(I).GT.AMAXI(L)) AMAXI(L)=R(I)
   10 IF(R(I).LT.AMINI) AMINI=R(I)
      X=AMAXI(L)
      TX=X-AMINI
      AL=TX/48. :
      NCONT=0
      NLINE=0 :
   12 DO 15 I=0,MM
      ALINE(I)=' '
      IF(MOD(NLINE,5).EQ.0) ALINE(I)='-'
   15 IF(MOD(I,10).EQ.0): ALINE(I)=':'
      DO 30 I=1,NP
      IF((R(I).LT.X).OR.(R(I).GE.(X+AL))) GO TO 30
      J=KSTEP(I)
      ALINE(J)='*'
      NCONT=NCONT+1
   30 CONTINUE
      WRITE(6,35)X,(ALINE(I),I=0,MM)
   35 FORMAT(2X,F10.2,3X,'^',101A1)
      X=X-AL
      NLINE=NLINE+1
      IF(NCONT.GE.NP) GO TO 45
      GO TO 12
   45 WRITE(6,46)
   46 FORMAT(16X,102('>'))
      WRITE(6,48)PEQUE,XMAX
   48 FORMAT(12X,F6.1,96X,F6.1)
   50 CONTINUE
      RETURN
      END :
```

UN MODELO ESTOCASTICO PARA EL ESTUDIO DE LA DENSIDAD DE DRENAJE DE LA CUENCA SUPERIOR DEL RIO DARRO (GRANADA)

Gutierrez Jaimez, R.* ; Moreno Bas, E.*

ABSTRACT : The drainage pattern is controlated by the structure and it is a decisive factor in the infiltration rate. For this reason, the analysis of this pattern has a special interest in the underground works areas, for aiding the study of the preferential flow lines. In this paper we prove the accuracy of a lognormal model for describing the random variables "length of the link" and "areas of drainage" which we have observed. Starting with this bivariate model, we get a theoretical model for describing the random variable "density of drainage". After to study the adequate statistical test we see that we can accept the model to a high level of significance.

RESUME : Le réseau de drainage, facteur décisif dans l'infiltration est contrôlé par la structure, pour cela une analyse détaillée de celle-ci est d'un intérêt spécial dans les zones où des travaux souterrains vont être réalisés, pour aider l'étude des lignes d'écoulement préférentiel. Dans cette communication, nous démontrons l'exactitude d'un modèle lognormal pour décrire les variables fortuites "longueur du lit" et "surface de drainage" que nous avons observées. A partir de ce modèle de deux variables, nous obtenons un modèle théorique pour décrire la variable random "densité de drainage". Après avoir étudié l'analyse statistique adéquate, nous voyons que nous pouvons accepter le modèle à un haut niveau de signification.

RESUMEN : La red de drenaje, factor decisivo en la infiltración, está controlada por la estructura, por lo que un análisis detallado de la misma es de especial interés en las áreas donde se van a realizar trabajos subterráneos, para ayudar al estudio de las líneas de flujo preferencial. A partir de los datos medidos en la cuenca del río Darro (Granada), se ajusta un modelo estocástico bidimensional de tipo log-normal a las variables "longitud de cauce" y "área de drenaje", se constituye toda la inferencia estadística posible para dicho modelo, y se deduce una ley de distribución para la densidad de drenaje. Se prueba, además, el alto grado de ajuste de dicha distribución, y por tanto la aplicabilidad del modelo estudiado para la descripción de la red de sistemas de tercer orden analizada, y para la predicción estadística de variables aleatorias hidrogeológicas implicadas en dicho Modelo.

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1. INTRODUCCION

La finalidad del trabajo que presentamos es describir bajo el punto de vista probabilístico la variable, δ, densidad de drenaje que obtenemos como cociente de las variables "longitud de cauces", L, y "áreas de drenaje", A, de una cierta zona de la provincia de Granada. La información muestral de que disponemos consta de 83 datos de la variable (L,A), de la zona antes dicha, en base a los cuales se estima y contrasta un modelo logarítmico-normal.

A partir de este, una vez aceptado como válido, se deduce una ley teórica ya estimada con la que se describe las probabilidades con que δ está en intervalos (δ_0,δ_1) arbitrarios y pertenecientes al cambio de variación de δ. Dicha ley se contrasta con la información que se deduce de δ a partir de la muestra, concluyéndose que, en este caso, el modelo logarítmico-normal es aceptable a un alto grado de confianza.

Por otra parte es interesante resaltar que en la bibliografía en que se utilizan métodos matemáticos en relación con el problema de drenaje, hay dos tendencias bién definidas. Una que analiza estadísticamente los datos |por ejemplo en la línea descrita en DOORN KAMP & KING, 1971.cap.I| sin suponer un modelo probabilístico básico, con lo cual el conocimiento de las leyes teóricas que siguen las variables hidrogeológicas queda sin resolver. Y otra línea más reciente |de la cual el modelo más desarrollado es el de DAVID-FIX, 1961 y su perfeccionamiento por el modelo de SMART, 1976| que trata de analizar el problema citado, a partir del establecimiento de un modelo teórico de carácter aleatorio, el cual relacione las variables hidrogeológicas en juego, y del que pueda extraerse información probabilística sobre el fenómeno físico en cuestión.

En este trabajo, y entre otros fines más concretos que luego se detallan, lo que hemos pretendido en un método mixto probabilístico-estadístico, con el que creemos que se obtiene el máximo de información posible de los datos observados en la realidad, al tener en consideración las dos tendencias antes citadas.

2. LA DISTRIBUCION LOGARITMICO-NORMAL BIVARIANTE COMO DISTRIBUCION DE "L" Y DE "A".

Supuesto que se adopta un punto de vista probabilístico en el estudio de las variables L, "longitud de los cauces de la zona en estudio" y A, "áreas de drenaje hacia esos cauces de esa zona", el conocimiento más completo a que se puede aspirar es, desde luego, conocer las distribuciones univariantes y, en su caso, la bivariante que gobiernan dichas variables. El primer paso será pués de

cidir si dichas variables son independientes, y en este caso proponer las distribuciones univariantes que las gobiernan, o si dichas variables están ligadas entre sí y por tanto la descripción probabilística debe hacerse con una distribución bidimensional. La primera hipótesis es plenamente rechazada si se considera la información muestral dada en las Tablas I y II y en el estudio sobre el coeficiente de correlación que se hace en el apartado 3, estudio apoyado a su vez en las consideraciones que hacemos en el párrafo 2.1.. Supuesto aceptado estos cálculos, el problema que inmediatamente se plantea es la elección de la distribución bidimensional teórica de la variable (L,A). Por estudios precedentes, (véase DAVID-FIX) y por razones que se desprenden, de manera evidente, de la información muestral, la familia de distribución que se adopta como posibles leyes de (L,A) son del tipo exponencial, y de entre estas, del tipo logarítmico-normal por dos bloques de razones fundamentalmente. El primer grupo de razones que justifica la elección de este tipo de leyes como primera aproximación probabilística, las damos en el párrafo 2.1, y, bajo nuestro punto de vista, son suficiente para emprender la construcción de los test de hipótesis estadísticos correspondientes. El segundo grupo de razones son menos rigurosas bajo el punto de vista de la Estadística Matemática pero complementan las de 2.1 con algunas consideraciones de tipo intuitivo.

2.1. En general, dadas dos variables (X,Y) no independientes sabemos que las transformaciones $(f_1(.),f_2(.))$ tales que hagan que las variables $(f_1(L),f_2(A))$ se distribuyan según una normal bivariante, cumple que el valor absoluto del coeficiente de correlación ρ tiene el mayor valor posible, para toda elección de funciones $(g_1(.)g_2(.))$ (véase Jhonson-Kotz, 1972). Si se considera las transformaciones (log L, log A), puede verse que el valor del coeficiente de correlación muestral $\underline{r} = 0,8716$ que es el mayor de los obtenidos con otros tipos de transformaciones que se han considerado (por ejemplo, $(\sqrt{L}, \sqrt{A})$, (L^2, A^2), etc.).

Por tanto, si no hemos encontrado otras transformaciones que hagan que el coeficiente de correlación sea mayor que este es porque la distribución normal bivariante constituye una buena aproximación a la distribución de (log L, log A).

En consecuencia, tomando transformadas inversas se concluye que (L,A) siguen una distribución aproximada logarítmico-normal (véase Aitchinson-Brown, 1966).

La inferencia sobre dicho coeficiente $\underline{r}$ se realiza en el apartado 3, con la adopción de normalidad para (log L, log A).

2.2. Argumentos para considerar esa aproximación, menos rigurosos - que los dados pero que corroboran lo expuesto son los siguientes: las marginales teóricas de la lognormal bivariante son lognormal - univariantes, (véase Jhonson-Kotz, 1972), y como puede verse en las figuras 1 y 2 y más tarde demostramos por métodos analíticos, la información muestral asegura este hecho. Ciertamente el recíproco no es cierto, es decir pudieran L y A distribuirse según lognormales - univariantes y no ser (L,A) variables distribuidas según una lognormal bivariante.

Por otro lado, considerar esta aproximación proporciona no pocas ventajas de tipo técnico como por ejemplo simplicidad en el tratamiento de los estimadores de los parámetros que intervienen, propiedades óptimas de estos, tabulaciones, etc..

Finalmente diremos que, en una primera aproximación considerar las distribuciones lognormales como modelos del fenómeno que nos -- ocupa puede ser adecuado, aunque evidentemente hablamos solo a nivel de hipótesis de trabajo que será contrastada con la información muestral que disponemos.

3. INFERENCIA SOBRE EL COEFICIENTE DE CORRELACION LINEAL.

Como antes se dijo en el apartado 2.1 el coeficiente de correlación lineal resulta valer 0,8716, calculado sobre los datos logarítmicos. Aquí se completa la información sobre dicho coeficiente me--- diante la inferencia que es posible efectuar sobre él, en la hipótesis de normalidad. Fundamentalmente son dos los resultados teóricos básicos al respecto.

a). El comportamiento asintótico de la distribución del citado coeficiente en el muestreo, cuando tomara el valor cero, es decir cuando la independencia entre las dos variables fuese total. En este caso

$$|r| \; / \; \frac{1-r^2}{n-2} \xrightarrow{d} t_{n-2;2\alpha}$$

Para la deducción de este resultado ver por ejemplo Anderson, -- 1958, pág.61.

b). El comportamiento asintótico de la citada distribución en el caso de un valor teórico no nulo ρ cualquiera. En este caso, se - sabe que: el coeficiente r se distribuye con una densidad dada por:

$$n=N-1;\ \left|2^{n-2}(1-\rho^2)^{n/2}.(1-r^2)^{\frac{n-3}{2}}/(n-2)\ \pi\right|.\sum_{\alpha=0}^{\infty}\frac{(2\rho r)^{\alpha}}{\alpha}\ \Gamma^2\left(\frac{n+\alpha}{2}\right)$$

en una Normal bidimensional con coeficiente teórico ρ. (véase Anderson, 1958, pág. 69).
Utilizando la transformación de Fisher se aproxima dicho resultado por este otro asintótico:

Llamando $Z = \frac{1}{2}\ln\frac{1+r}{1-r}$ ("transformación de Fisher") y

$z = \frac{1}{2}\ln\frac{1+\rho}{1-\rho}$ se verifica que $N-1(Z-z) \xrightarrow{d} N(0,1)$.

En función de este resultado se obtienen los contrastes (de diferencias de coeficientes de correlación, significativas, etc.) y los intervalos de confianza de un coeficiente de correlación muestral r.

En el caso presente, teniendo en cuenta el valor calculado del coeficiente y el tamaño muestral puede llegarse a estos resultados concretos, basados en los resultados de a) y b) antes citados.

(1) El intervalo de confianza al 95% correspondiente al valor r = 0,8716 calculado, con N = 83, resulta ser este:

Prob. $|tg\ h\ (Z - 0,22336) \leq \rho \leq tg\ h\ (Z + 0,22336)|$ =

= Prob. $|tg\ h\ (1,11385) \leq \rho \leq tg\ h\ (1,56057)|$ =

= Prob. $|0,80157 \leq \rho \leq 0,9155| = 0,95$

(2) Por otro lado es descartable la independencia total, puesto que:

$$\frac{|r|}{\frac{1-r^2}{n-2}} \longrightarrow \frac{0,8716}{0,0544} = 16,022$$ que es claramente significativo -

frente a $t_{81g.l.;2\alpha} = 2,719.\ (\alpha = 0,05)$

(3) Utilizando además el contraste frente a $\rho = 1$ es descartable, - igualmente al 95%, dicho valor teórico para la población, a par-

tir del dato muestral $r = 0{,}8716$ ya que el valor $|Z-Z|\sqrt{n-3}$ es significativo frente al valor de la distribución teórica.

Podemos decir en resúmen, que dada la muestra en estudio, el coeficiente de correlación lineal de la población correspondiente toma un valor entre 0,80157 y 0,9155, con una confianza del 95%, siendo descartables tanto la independencia total, como el caso $\rho = 1$ de dependencia funcional máxima (coincidencia de las dos rectas de regresión).

4. ESTIMACION Y CONTRASTE DE LA LEY LOGARITMICO-NORMAL A LA "LONGITUD DE LAS CUENCAS (L)" Y "AREAS DE DRENAJE (A)".

Por el análisis realizado, en el apartado 3, del coeficiente de correlación de los datos empíricos que aparecen en las Tablas I y II, y las razones dadas en el apartado 2, vamos a considerar la hipótesis siguiente: la longitud de las cuencas, L, es una variable aleatoria de distribución logarítmico normal que representaremos por $\Lambda(L|\mu_1,\sigma_1^2)$; el área de drenaje, A, es otra variable aleatoria, no independiente de L, de distribución logarítmico-normal que representaremos por $\Lambda(A|\mu_2,\sigma_2^2)$; y la variable aleatoria bidimensional (L,A) supondremos se distribuye según una logarítmico-normal bidimensional que representaremos por $\Lambda(\bar{\mu},\Sigma)$, en donde $\bar{\mu}$ representa el vector bidimensional y Σ la matriz de 2 x 2, asociadas a la ley.

Las expresiones de estas leyes son las siguientes. Para la longitud L.

$$d\Lambda(L|\mu_1,\sigma_1^2) \begin{cases} \frac{1}{L\sigma_1\sqrt{2\pi}} \cdot \exp\{-\frac{1}{2\sigma_1^2}(\log L - \mu_1)^2\}dL, & \text{si } L > 0 \\ 0, & \text{si } L \leq 0 \end{cases}$$

y análogamente para A, sin más que sustituir los parámetros (μ_1,σ_1^2) por (μ_2,σ_2^2). La bidimensional viene dada por la expresión

$$d\Lambda(\bar{\mu},\Sigma) = \begin{cases} \frac{1}{L.A.\ |\Sigma|.2\pi} \cdot \exp\{-\frac{1}{2}|((L-\mu_1)(A-\mu_2)).\Sigma^{-1}.((L-\mu_1)(A-\mu_2))|\}.dL.dA, & \text{si } L > 0,\ A > 0 \\ 0, & \text{en otro caso} \end{cases}$$

Evidentemente, la aceptación o el rechazo de estas hipótesis tendremos que hacerlos en base a tests estadísticos apropiados al problema. La estimación y el contraste de estas leyes, con su consiguiente aceptación o rechazo, se lleva a cabo en las subsecciones 4.1, 4.2 y 4.3 .

4.1. *ESTIMACIONES Y ERRORES DE LAS ESTIMACIONES PARA LOS PARAMETROS DE LAS DISTRIBUCIONES DE L Y A.*

Los estimadores de los parámetros que aparecen en cada una de estas leyes que se van a considerar se obtienen por el método de la máxima-verosimilitud, y las estimaciones que daremos están apoyadas en los datos empíricos que aparecen en la Tabla I para la variable L, y en la Tabla II para la variable A. La expresión de los estimadores y sus varianzas es la siguiente: (véase KENDALL Y STUART, vol. 2, 1967)

$$\hat{\mu}_1 = \frac{1}{n} \sum_{j=1}^{n} \log L_j \quad , \quad \theta_1^2 = \frac{1}{n} \sum_{j=1}^{n} (\log L_j - \hat{\mu}_1)^2 \tag{1}$$

$$\mathrm{Var}(\hat{\mu}_1) = \frac{\hat{\sigma}_1^2}{n} \quad , \quad \mathrm{Var}(\hat{\sigma}_1^2) = \frac{2\sigma_1^4}{n} \tag{2}$$

$$\hat{\mu}_2 = \frac{1}{n} \sum_{j=1}^{n} \log A_j \quad , \quad \hat{\sigma}_2^2 = \frac{1}{n} \sum_{j=1}^{n} (\log A_j - \hat{\mu}_2)^2 \tag{3}$$

$$\mathrm{Var}(\hat{\mu}_2) = \frac{\hat{\sigma}_2^2}{n} \quad , \quad \mathrm{Var}(\hat{\sigma}_2^2) = \frac{2\hat{\sigma}_2^4}{n} \tag{4}$$

habiéndose determinado las expresiones (3) y (4) a partir de la información de Fisher en virtud de la eficiencia de los estimadores. Además, por ser las distribuciones de $(\hat{\mu}_j, \hat{\sigma}_j^2)$, $j = 1,2$, del tipo exponencial podemos asegurar la suficiencia de dichos estimadores (véase S. ZACKS, 1971). Si consideramos las expresiones (1) y (2), y los datos de la Tabla I, las estimaciones que resultan, junto con su varianza, son las siguientes:

$$\hat{\mu}_1 = 0{,}4190 \quad , \quad \hat{\sigma}_1^2 = 0{,}4945 \tag{5}$$

$$\mathrm{Var}(\hat{\mu}_1) = 0{,}0060 \qquad \mathrm{Var}(\hat{\sigma}_1^2) = 0{,}0059 \tag{6}$$

Así mismo, si tenemos en cuenta las expresiones (3) y (4) y los datos de la Tabla II, las estimaciones que se obtienen, y sus varianzas, vienen dadas por:

$$\hat{\mu}_2 = -1,8369 \quad , \quad \hat{\sigma}_2^2 = 1,0148 \tag{7}$$

$$\mathrm{Var}(\hat{\mu}_2) = 0,0122 \quad , \quad \mathrm{Var}(\hat{\sigma}_2^2) = 0,0248 \tag{8}$$

Si determinamos la distribución logarítmico-normal para las estimaciones dadas en (5), esto es, la distribución teórica de la variable "longitud de las cuencas", podemos calcular la probabilidad de que una cuenca tenga una longitud comprendida entre dos valores fijados. Para visualizar los resultados hemos listado las frecuencias teóricas junto con las obtenidas empíricamente de la variable log L, (el hecho de trabajar con log L es simplemente por comodidad en la integración de la ley), para la partición del eje real que aparece en la Tabla III.

Análogamente, considerando la estimación dada en (7), los valores teóricos obtenidos con el modelo logarítmico-normal para la variable "áreas de drenaje", aparecen en la Tabla IV listadas junto a los valores empíricos. (realmente se han listado las frecuencias para la variable log A, por la misma razón que en la variable L.).

4.2. *CONTRASTE DE LA HIPOTESIS DE QUE L Y A SON VARIABLES DISTRIBUIDAS SEGUN LOGARITMICO-NORMALES.*

El test estadístico que vamos a utilizar para medir la posible discrepancia entre los valores teóricos y empíricos que aparecen en la Tabla III para el caso de L y en la Tabla IV en el caso de A, es el conocido "test χ^2". Aunque los resultados que este test establece son asintóticos, en nuestro caso puede utilizarse sin problemas debido a que la cantidad de datos es "suficientemente grande" --- (n = 83), de acuerdo con los órdenes de aproximación del estadístico χ^2 que se considera, a la distribución límite.

En el caso de la variable L, el valor experimental del estadístico χ^2 es 3,6496, lo que significa que la hipótesis de que L sigue una distribución logarítmico-normal de parámetros los dados en (5) puede aceptarse a un nivel de confianza del 93%. En consecuencia, dicha hipótesis será aceptada.

Repitiendo los cálculos para la variable A, el valor experimental del estadístico χ^2 es 2,5439, lo que nos dice que la ley lognormal de parámetros los dados en (7) puede aceptarse como distribu---

ción de A con un nivel de confianza del 92%.

4.3. *ESTIMACION Y CONTRASTE DEL MODELO LOGNORMAL BIVARIANTE.*

Los estimadores que proponemos para los parámetros del modelo bivariante son también máximo-verosímiles, y sus expresiones vienen dadas por: (véase KENDALL y STUART, 1967,)

$$(\hat{\mu}_1, \hat{\mu}_2) = \left(\frac{1}{n} \sum_{j=1}^{n} \log L_j, \ \frac{1}{n} \sum_{j=1}^{n} \log A_j\right) \tag{9}$$

$$\hat{\Sigma} \equiv \begin{pmatrix} \hat{\sigma}_1^2 & \hat{\sigma}_{12} \\ \hat{\sigma}_{21} & \hat{\sigma}_2^2 \end{pmatrix} = \begin{pmatrix} \frac{1}{n} \sum_{j=1}^{n} (\log L_j - \hat{\mu}_1)^2 & \frac{1}{n} \sum_{j} (\log L_j - \hat{\mu}_1)(\log A_j - \hat{\mu}_2) \\ & \frac{1}{n} \sum_{j=1}^{n} (\log A_j - \hat{\mu}_2)^2 \end{pmatrix} \tag{10}$$

estimadores cuyas propiedades son bién conocidas y óptimas cuando el tamaño de la muestra es suficientemente grande. En nuestro caso, el tamaño muestral es n = 83, con lo que puede asegurarse que sus varianzas están próximas al menor valor posible, que son centrados y, por supuesto, suficientes para la distribución de dichos vectores es exponencial. Un análisis detallado de las propiedades dadas y de las distribuciones de estos estimadores pueden encontrarse en la obra de KENDALL y STUART, 1967, o en la de ZACKS, 1971. El estimador del coeficiente de correlación y la dispersión de los estimadores (9) y (10) son:

$$(\text{con } \hat{\sigma}_{21} = \hat{\sigma}_{12}) \qquad \hat{\rho} = \frac{\hat{\sigma}_{12}}{\hat{\sigma}_1\ \hat{\sigma}_2} \tag{11}$$

y la matriz de dispersión por:

$$V_1(\hat{\mu}_1,\hat{\mu}_2) = \begin{pmatrix} \frac{1}{\hat{\sigma}_1^2} & -\frac{\hat{\rho}}{\hat{\sigma}_1\ \hat{\sigma}_2} \\ -\frac{\hat{\rho}}{\hat{\sigma}_1\ \hat{\sigma}_2} & \frac{1}{\sigma_2^2} \end{pmatrix} \frac{n}{(1-\hat{\rho}^2)} \tag{12}$$

$$V_2(\hat{\sigma}_1,\hat{\sigma}_2,\hat{\rho}) = \begin{pmatrix} \frac{2-\hat{\rho}^2}{4\hat{\sigma}_1^2} & -\frac{\hat{\rho}^2}{4\hat{\sigma}_1^2\hat{\sigma}_2^2} & -\frac{\hat{\rho}}{2\hat{\sigma}_1^2} \\ -\frac{\hat{\rho}^2}{4\hat{\sigma}_1^2\hat{\sigma}_2^2} & \frac{2-\hat{\rho}^2}{4\ \hat{\sigma}_2^4} & -\frac{\hat{\rho}}{2\hat{\sigma}_2^2} \\ -\frac{\hat{\rho}}{2\hat{\sigma}_1^2} & -\frac{\hat{\rho}}{2\hat{\sigma}_2^2} & \frac{1+\hat{\rho}^2}{1-\hat{\rho}^2} \end{pmatrix} \frac{n}{1-\hat{\rho}^2} \tag{13}$$

con lo que las estimaciones que se obtienen considerando los datos empíricos, (aquellos cuyas marginales son las Tablas I y II), son los siguientes:

$$(\hat{\mu}_1,\ \hat{\mu}_2) = (0{,}4190 \quad -1{,}8369)\ , \quad \hat{\Sigma} = \begin{pmatrix} 0{,}4945 & 0{,}6174 \\ 0{,}6174 & 1{,}0148 \end{pmatrix} \tag{14}$$

con varianzas para $(\hat{\mu}_1,\hat{\mu}_2)$, las dadas por

$$V_1^{-1} = \begin{pmatrix} 0{,}0008134 & 0{,}0010156 \\ 0{,}0010156 & 0{,}0016693 \end{pmatrix} \tag{15}$$

y para $(\hat{\sigma}_1,\hat{\sigma}_2,\hat{\rho})$ las dadas por

$$V_2^{-1} = \begin{pmatrix} 0{,}00589 & 0{,}0091862 & 0{,}0012476 \\ 0{,}0091862 & 0{,}024814 & 0{,}0025604 \\ 0{,}0012476 & 0{,}0025604 & 0{,}000695 \end{pmatrix} \tag{16}$$

Si este modelo bivariante puede ser aceptado como bueno, las líneas de regresión de L sobre A y de A sobre L darían una expresión funcional de como varia la variable L cuando lo hace la A y recíprocamente. Además los coeficientes de la función quedarían automáticamente determinados por los valores dados en (14) y (11).

Desde luego, el admitir el modelo como válido para describir el fenómeno llevaría consigo construir un test estadístico apropiado al caso, no obstante, ya sabemos algo al respecto, pues las marginales de esta distribución serían en efecto logarítmico-normales que han sido aceptadas a un alto nivel de significación. Sin embargo, sabemos que esto no es suficiente, Es decir, las marginales pueden ser las adecuadas y sin embargo la conjunta no está determinada en general, unívocamente por estas. Un posible test sería el siguiente: ya que una de las finalidades básicas del trabajo es encontrar la distribución de probabilidad para la densidad de drenaje, vamos a determinar la distribución de dicha densidad de drenaje a partir de la lognormal bivariante aquí obtenida. Hecho esto, contrastaremos con la distribución empírica de dicha densidad. Nótese que esto es conceptualmente distinto a los contrastes realizados para L y para A, pues en estos casos las leyes teóricas fueron estimadas a partir de los datos empíricos.

La obtención de la ley de la densidad de drenaje a partir del modelo bivariante y el contraste con los datos empíricos de dicha densidad se dan en el apartado 6., y como veremos los resultado son positivos. Por esta razón, en el apartado siguiente vamos a dar líneas de regresión que se deducen con el modelo lognormal-bivariante.

5. LINEAS DE REGRESION DE L SOBRE A Y DE A SOBRE L.

5.1 Supuesto el modelo lognormal aceptado, esto es la variable aleatoria bidimensional (L, A) se distribuye según $\Lambda(\Pi,\Sigma)$, puede verse (véase por ejemplo Johnson y Kotz "Distributions in Statistics" Vol.4, John Wiley, 1972, pág.15) que la variable condicionada $(L|A)$ se distribuye según una lognormal de parámetros

$$\Lambda\left(\hat{\mu}_1 + \frac{\hat{\mu}\,\hat{\sigma}_1}{\hat{\sigma}_2}\,(\log A - \hat{\mu}_2),\ (1-\hat{\rho}^2)\,\hat{\sigma}_1^2\right)$$

y que $(A|L)$ sigue otra lognormal de parámetros

$$\Lambda\left(\hat{\mu}_2 + \frac{\hat{\rho}\,\hat{\sigma}_2}{\hat{\sigma}_1}\,(\log L - \hat{\mu}_1),\ (1-\hat{\rho}^2)\,\hat{\sigma}_2^2\right).$$

Por tanto, las líneas de regresión vendrán dadas por las esperanzas matemáticas de cada una de estas leyes, esto es:

Línea de regresión de (L|A)

$$E|L|A| = A^{\hat{\rho}\frac{\hat{\sigma}_1}{\hat{\sigma}_2}} . \exp\{\tfrac{1}{2}(1-\hat{\rho}^2)\hat{\sigma}_1^2 + \hat{\mu}_1 - \hat{\rho}\,\hat{\sigma}_1 \frac{\hat{\mu}_2}{\hat{\sigma}_2}\}$$

Línea de regresión de (A|L)

$$E|A|L| = L^{\hat{\rho}\frac{\hat{\sigma}_2}{\hat{\sigma}_1}} . \exp\{\tfrac{1}{2}(1-\hat{\rho}^2)\hat{\sigma}_2^2 + \hat{\mu}_2 - \hat{\rho}\,\hat{\sigma}_2 \frac{\hat{\mu}_1}{\hat{\sigma}_1}\}.$$

cuyas estimaciones son inmediatas considerando (14) y (11). Así -- pués los resultados numéricos son:

$$E|L|A| = 2,1340 \; . \; A^{0,4247} \qquad (17)$$

$$E|A|L| = 4,2022 \; . \; L^{1,7887} \qquad (18)$$

5.2 *REGRESIONES DE PRIMERA Y SEGUNDA CLASE.*

Todo problema de ajuste de distribuciones probabilísticas, como el presente, pretende enel fondo predecir con el mayor grado de precisión posible los valores de ciertas variables implicadas en el modelo, a partir del conocimiento de otros datos, de otras varia--- bles.

La primera aproximación al respecto, y en un modelo bidimensional como el que aquí se propone, es utilizar las líneas de regre--- sión correspondientes. A continuación, e independientemente de las predicciones dadas para el modelo log-normal bidimensional |ver 5.1| vamos a construir la inferencia sobre las rectas de regresión basadas en la variablebidimensional normal, asociada a la lognormal citada, ya que en este caso es posible utilizar resultados exactos para los contrastes, bandas de confianza y tolerancia etc. de las citadas rectas de regresión normales.

Los datos numéricos básicos son los siguientes:

$\bar{L} = 0,419$	$\bar{A} = -1,8369$
$S_L^2 = 0,4945$	$S_A^2 = 1,0148$
$S_L = 0,7032$	$S_A = 1,0073$

$$S_{LA} = 0,61738$$

$$r = 0,8716 \quad r^2 = 0,75968 \quad 1-r^2 \quad 0,24032$$

5.3 En primer lugar se construyen las rectas de regresión de primera clase, con datos no agrupados (tamaño muestral N=83), y las correspondientes hipérbolas que dan los límites de confianza -- así como los límites de tolerancia correspondientes. Estas hi-- pérbolas, así como las elipses que más tarde aparecen en la regresión de segunda clase, constituyen "bandas de confianza" --- realmente, midiéndonos en cada valor predicho de una variable a partir del valor dado de la otra, el error cometido en la pre-- dicción. En ambos casos vamos pués a construir las bandas dejan do variable el valor a que se condiciona la predicción, por lo que tiene sentido hablar de hipérbola y elipse pués sus ecuaciones son las que nos dan los límites de las bandas en donde está contenida la predicción más o menos el error correspondiente.

En efecto sucesivamente calculamos:

(a) *Recta de regresión de A|L.*

$$A = -1,8369 + 1,2485\ (L - 0,419)$$

$$= -2,36002 + 1,2485\ L$$

(a.1) *Significación frente a cero del coeficiente angular* b_{AL}.

$$\frac{b_{AL}-b_{AL}}{S_{b_{AL}}} = \frac{b_{AL}}{S_{b_{AL}}} = \frac{1,24849}{6,0886\text{x}10^{-3}} = 205,053$$

claramente significativo frente a $t_{2\alpha;n-2}$ g.l.

(a.2) *Límites de confianza para el valor* b_{yx} *teórico.*

$b_{AL} \pm t_{2\alpha;}S_{b_{AL}}$ con $v = n-2$ g.l. si $n-2 < 200$.

En este caso:

$1,24849 \pm 1,664,6,0886\text{x}10^{-3} =$

$= 1,24849 \pm 0,01013.$

(a.3) *Límites de confianza para el valor* a_{yx} *teórico.*

En este caso $a_{AL} = -2,36002$ con unos límites dados por

$$-2,36002 \pm t_{2\alpha;n-2} \cdot S_{a_{yx}} \quad \text{(ya que } n < 200\text{)}$$

donde $S^2_{a_{AL}} = S^2_{b_{AL}} \cdot \frac{\Sigma x^2}{n}$

(b) *Predicción de A dado L. Límites de confianza.*

$\bar{y} + b_{AL}(L-\bar{L}) \pm t_{2\alpha;n-2} \cdot S_{AL}$ que toma la forma:

$$-2,36002+1,2485\ L \pm t_{2\alpha;81} \cdot \ 4,367.10^{-5}(L-0,419)^2+2,996.10^{-5}$$

que es la hipérbola de confianza correspondiente.

(c) *Predicción de L cuando se da A, según la recta de regresión calculada.*

$$L\ A = \bar{L} + \frac{A - \bar{A}}{b_{AL}} = 0,419 + \frac{A - 1,81369}{1.24849}$$

(c.1) Límites de confianza de esta predicción.

Al ser $k^2 = t^2_{2\alpha} \cdot S^2_{b_{LA}} / b^2_{LA} = 7,7577.10^{-5} << 0,05$ se puede tomar $1 - k^2 \simeq 1$ con lo que las expresiones correspondientes a los límites indicados se simplifican quedando de esta forma:

$$0,419 + \frac{A - 1,81369}{1,24849} \pm 0,00705 \ . \ \ 1,0696 + (A - 1,81369)^2$$

5.4 *Regresión de segunda clase e Inferencia asociada.*

Como es sabido, en los problemas estadísticos con dos variables x e y y pueden darse, en la práctica, dos casos bién definidos de relación funcional lineal:

(a) Que una variable produzca un efecto en la otra (efecto casual)

(b) Que ambas variables sean "casuales" (regresión de segunda clase).

Se caracterizan porque en el primer caso existe una sola recta de regresión (A|L), que se estudió antes para nuestro problema concreto, la cual se emplea para cálculo en ambos sentidos -por eso antes en el punto (C de 5.3) la utilizamos de A a L en lugar de L a A-.

Y en el caso (b) anterior hay dos rectas de regresión distintas, por consiguiente la inferencia de A a L es distinta de la de L a A.

Aquí desarrollamos los cálculos e inferencia de segunda clase para perfeccionar los de primera antes estudiados, ya que en el fenómeno hidrogeológico en cuestión suponemos que esta segunda posibilidad puede ser más realista en general (la fluctuación estadística casual de ambas variables L y A).

En efecto se encuentran sucesivamente los siguientes resultados, para nuestro problema:

(a) Rectas de regresión.

Toman la forma:

$$A|L = -1,8369 + 1,24849\ (L - 0,419)$$

$$L|A = 0,419 + 0,60837\ (A + 1,8369)$$

(b) Los límites de confianza son de la forma

$$\bar{A} + b_{AL}(L-\bar{L}) \pm S_{b_{AL}} \quad 2F.S_L/n-(n-2)(L-\bar{L})^2$$

$$\bar{L} + b_{LA}(A-\bar{A}) \pm S_{b_{LA}} \quad 2F.S_A/n-(n-2)(A-\bar{A})^2$$

que nos proporciona respectivamente las elipses de confianza de las regresiones A|L y L|A respectivamente:

$$A|L = -1,8369+1,24849(L-0,419)\pm 0,078.\ \ ^2F_{2;81}$$

$$.40,549/83-81(L-0,419)^2$$

$$L|A = 0,419+0,60837(A+1,8369)\pm 0,038 \quad 2\,F_{2;81}$$

$$83,2136/83-81(A+1,8369)^2$$

(c) En forma análoga obtenemos los límites de tolerancia que vienen dados por sendas elipses de tolerancia, que resultan ser estos:

$$A|L = -1,8369+1,24849(L-0,419)\pm 0,078 \quad 2,85.F_{2;81}\cdot$$

$$\cdot \quad 40.549/83-81(L-0,419)^2.$$

$$L|A = 0,419+0,60837(A+1,8369)\pm 0,038 \quad 2\,F_{2;81}\cdot$$

$$\cdot \quad 83,2136/83-81(A+1,8369)^2$$

Como es sabido es interesante representar gráficamente tanto estas elipses así obtenidas como las hiperbolas de la regresión de primera clase vista en (5.3 (b)). Para ello son de interés las pendientes de los ejes principales y los semiejes de las elipses, que son los siguientes respectivamente:

$$b_o \; - \frac{1}{b_o} = \frac{S_A - S_L}{2S_{AL}} \pm \sqrt{1+\left(\frac{S_A - S_L}{2S_{AL}}\right)^2} = 0,42137 \pm 1,08515$$

$$l_1 = \frac{F_{2;81}}{83.81} \quad 123,7626 \pm \quad 15137,181 - 3245,3089 =$$

$$= 0,0121 \quad F_{2;81} \cdot 15,28511 = 0,41177$$

$$l_2 = 0,0121 \quad F_{2;81} \cdot 3,7270 = 0,09940$$

$$l_1; l_2 = k \quad S_L+S_A \pm \sqrt{(S_L + S_A)^2 - 4(S_L S_A - S_{LA}^2}$$

con $k = \frac{F_{2;n-2}}{n(n-2)}$ (para elipse de confianza) y $k = \frac{F_{2;n-2}\cdot(n+1)}{n(n-2)}$ (para la elipse de tolerancia).

Por ejemplo en el caso de la elipse de confianza resultan estos valores calculados para los datos hidrogeológicos básicos de este trabajo.

Y análogamente se calculan los semiejes de la elipse de tolerancia.

Para completar los cálculos y las representaciones gráficas antes aludidas -y de las que aquí prescindimos- son de interés los datos sobre las tangentes a las elipses anteriores y sobre las ordenadas en el origen, fácilmente calculables conociendo los datos ya empleados en la determinación de las elipses básicas. Con todos estos datos, se determina fácilmente el "rectángulo" de confianza y de tolerancia mediante las tangentes horizontales y verticales a las citadas cónicas. En estos rectángulos nos movemos siempre, a la hora de utilizar las rectas de regresión de segunda clase como base de predicción de L dado A o bién de A dado L, en nuestro problema concreto de tipo hidrogeológico, y al 95%.

6. DISTRIBUCION DE LA DENSIDAD DE DRENAJE $\delta = \frac{L}{A}$

Supuesto que la variable bidimensional (L, A) se distribuye según el modelo logarítmico-normal bivariante dado en el apartado 4, la densidad de drenaje δ, definida como el cociente L/A, tiene la distribución resultante de determinar la conjunta (L/A,Z) y determinar a partir de aquí la marginal mediante la integración en Z. Ahora bién, si L, A) sigue una logarítmico-normal, esto equivale a que (log L, Log A) se distribuya según una normal bivariante de parámetros $(\bar{\mu},\Sigma)$. Con esto la transformación, de Jacobiano la unidad, dada por

$$\begin{pmatrix} \log \delta \\ Z \end{pmatrix} = \begin{pmatrix} 1 & -1 \\ 0 & 1 \end{pmatrix} \begin{pmatrix} \log L \\ \log A \end{pmatrix} ,$$

transforma dicha normal bivariante en la normal $N(C\bar{\mu} , C\Sigma C')$ siendo C la matriz de la transformación y habiendo utilizado la"prima" para indicar la operación de transponer. Operando resulta que (log δ , Z)' se distribuye según una normal de parámetros.

$$\bar{\mu}^* = \begin{pmatrix} \hat{\mu}_1 - \hat{\mu}_2 \\ \hat{\mu}_2 \end{pmatrix} \quad y \quad \Sigma^* = \begin{pmatrix} \hat{\sigma}_1^2 - 2\hat{\sigma}_{12} + \hat{\sigma}_2^2 & \hat{\sigma}_{12} - \hat{\sigma}_2^2 \\ \hat{\sigma}_{12} - \hat{\sigma}_2^2 & \hat{\sigma}_2^2 \end{pmatrix}$$

con lo que inmediatamente se deduce que δ se distribuye según una lognormal de parámetros

$$\Lambda(\hat{\mu}_1 - \hat{\mu}_2,\ \hat{\sigma}_1^2 - 2\hat{\sigma}_{12} + \hat{\sigma}_2^2)$$

siendo los parámetros que aquí aparecen los que se determinan a partir de los dados en el apartado 4, expresión (14).

Nótese que el resultado será aceptable si la hipótesis de partida es válida, es decir si (L, A) es lognormal bivariante. Bajo esta hipótesis la lognormal obtenida es la $\Lambda(2{,}256\ ,\ 0{,}2744)$.

Para comprobar si esto es cierto, consideremos los datos empíricos de δ y determinemos sus análogos teóricos con la ley obtenida.

Los resultados que se obtienen aparecen en la Tabla VI bajo el epígrafe "frecuencias teóricas". Estas han sido listadas junto a los datos empíricos que aparecen en esa misma Tabla bajo el epígrafe "frecuencias empíricas", (como en los casos anteriores listamos log δ en vez de δ por comodidad en los cálculos).

Como en los casos anteriores el grado de discrepancia entre las distribuciones teórica teórica y empírica lo medimos con un test χ^2. La χ^2 experimental que se obtiene vale 2,5566 lo que equivale a decir que la ley obtenida es aceptable a un nivel de significación superior al 95%. Por este motivo no solo aceptamos dicha ley para la distribución de δ, sino que la logarítmico-normal bivariante puede también tomarse como válida lo que a su vez lleva consigo la aceptación de las líneas de regresión dadas en el apartado 5.

AGRADECIMIENTOS.- Agradecemos las orientaciones que en el campo de la Hidrogeología nos ha proporcionado el Profesor Fernández-Rubio. Damos las gracias al Profesor J.L. Díaz que nos facilitó los datos básicos de la cuenca analizada. Por último, nuestro agradecimiento a la Dra. Vila Miranda por el procesamiento de los datos utilizados en este trabajo, y a Dª Soledad Cantos Hoces por su cuidadosa mecanografía.

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TABLA I

DATOS EMPIRICOS DE LAS LONGITUDES DE LOS CAUCES DE LA RED IMPLANTADA EN LOS SITEMAS DE TERCER ORDEN (L_j)

(Cuenca Superior del Río Darro)

1,06	0,43	2,75	1,67	2,67	0,95	1,03	1,70	1,46	1,35
3,88	0,58	7,22	1,67	6,68	3,49	2,48	1,48	2,58	0,80
4,47	4,39	2,13	2,12	2,17	2,38	4,13	1,60	1,10	0,95
1,13	0,62	1,41	0,93	1,60	1,24	1,40	1,73	0,86	0,88
1,36	2,59	0,66	2,62	1,17	0,59	0,47	1,24	0,66	0,67
0,50	0,92	1,20	3,35	1,52	0,55	1,57	0,58	1,28	2,85
0,95	2,81	2,03	2,65	0,70	1,02	4,51	4,72	5,54	1,28
0,17	5,91	2,16	1,40	1,82	0,98	1,34	2,37	1,78	3,27
1,80	1,25	0,65							

Fuente: D. José Luis Díaz Hernandez, del Grupo de Trabajo de Hidrogeología de la Universidad de Granada (Dominio Bético S. Str)

TABLA II

DATOS EMPIRICOS DE LAS SUPERFICIES DE DRENAJE (A_j) DE LOS - CAUCES DE LA TABLA I.

0,221	0,020	0,212	0,140	0,176	0,062	0,069	0,156	0,154
0,181	0,304	0,052	1,189	0,156	1,988	1,296	0,558	0,397
0,462	0,078	1,247	0,833	0,384	0,351	0,239	0,341	0,477
0,243	0,085	0,059	0,063	0,035	0,129	0,071	0,177	0,078
0,145	0,239	0,110	0,137	0,206	0,838	0,060	0,418	0,060
0,030	0,035	0,089	0,078	0,060	0,035	0,054	0,134	0,363
0,100	0,054	0,134	0,020	0,081	0,265	0,101	0,336	0,528
0,347	0,062	0,053	0,355	0,610	1,137	0,109	0,079	0,531
0,104	0,074	0,134	0,127	0,156	0,154	0,127	0,414	0,226
0,254	0,034							

Fuente: D. José Luis Díaz Hernandez, del Grupo de Trabajo de Hidrogeología de la Universidad de Granada (Dominio Bético S. Str)

TABLA III

FRECUENCIAS EMPIRICAS Y TEORICAS DE LA VARIABLE LOG L

INTERVALOS		FR. EMPIR.	FR. TEOR.
$-\infty$	-0,987	1	1,89
-0,987	-0,706	2	2,66
-0,706	-0,425	7	5,00
-0,425	-0,143	6	8,03
-0,143	0,138	12	11,02
0,138	0,419	16	12,90
0,419	0,700	10	12,90
0,700	0,982	12	11,02
0,982	1,26	7	8,03
1,26	1,544	5	5,00
1,544	1,825	3	2,65
1,825	∞	2	1,89

TABLA IV

FRECUENCIAS EMPIRICAS Y TEORICAS DE LA VARIABLE LOG A

INTERVALOS		FR. EMPIR.	FR. TEOR.
-∞	-3,852	2	1,89
-3,852	-3,046	7	7,65
-3,046	-2,643	8	8,03
-2,643	-2,240	11	11,02
-2,240	-1,837	16	12,90
-1,837	-1,434	10	12,90
-1,434	-1,031	11	11,02
-1,031	-0,628	9	8,03
-0,628	0,178	6	7,65
0,178	∞	3	1,89

TABLA V

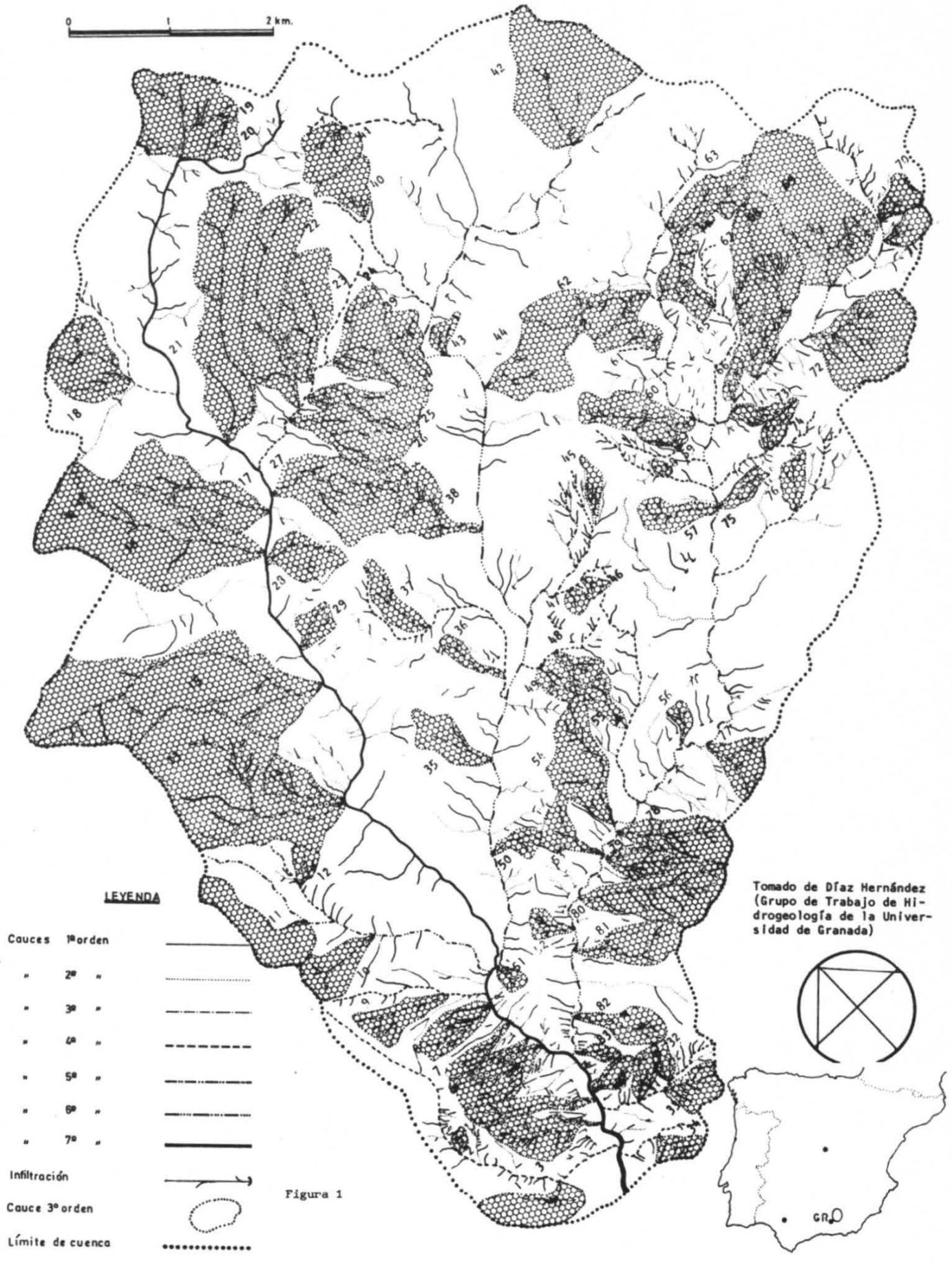
VALORES EMPIRICOS DE LA DENSIDAD DE DRENAJE $\delta_j = \frac{L_j}{A_j}$; j=1,-,83

4,796	21,500	12,972	11,929	15,170	15,323	14,928	10,897
9,481	7,459	12,763	11,154	6,072	10,705	3,360	2,693
4,444	3,728	5,584	10,256	3,585	5,270	5,547	6,040
9,079	6,979	8,658	6,584	12,791	16,102	17,937	17,714
10,930	13,099	9,040	15,897	9,655	7,238	7,818	6,423
6,602	3,091	11,000	6,268	19,500	19,667	13,429	13,933
8,462	11,167	14,286	17,037	8,955	9,229	15,200	10,185
12,463	29,000	15,802	10,755	9,406	8,363	3,845	7,637
11,290	19,245	12,986	7,738	4,872	11,743	2,152	11,130
20,769	18,919	13,582	7,717	8,590	15,390	14,016	7,899
7,965	4,921	19,118					

TABLA VI

FRECUENCIAS EMPIRICAS Y TEORICAS DE LA VARIABLE LOG δ

INTERVALOS		FR. EMPIR.	FR. TEOR.
$-\infty$	1,208	3	1,89
1,208	1,627	8	7,66
1,627	1,837	6	8,03
1,837	2,046	9	11,02
2,046	2,256	13	12,90
2,256	2,465	13	12,90
2,465	2,675	12	11,02
2,675	2,885	10	8,03
2,885	3,304	8	7,65
3,304	∞	1	1,89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" 5º "
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" 7º "
Infiltración
Cauce 3º orden
Límite de cuenca
Tomado de Díaz Hernández (Grupo de Trabajo de Hidrogeología de la Universidad de Granada)
GR

Figura 1

RELIABILITY AND THE FACTOR OF SAFETY DUE TO PIPING

Harr, M. E. and Sipher, D. J.

ABSTRACT : A hybrid finite element program was used to evaluate the effect of the randomness and the uncertainty of the coefficient of permeability on the exit gradient. A log normal distribution was used to assign permeabilities to the elements of the finite element grid. It was found that the distribution of exit gradients could be approximated by a normal distribution having a mean value approaching that of a perfectly homogeneous soil. An upper bound of the coefficient of variation of the true exit gradient was obtained from a 99 % confidence interval of the chi-square distribution. This coefficient of variation was used with the theoretical solution of confined sheet pile flow to determine the probability of piping (i. e., F. S.) piping < 1.0), which corresponds to an exit gradient greater than 1.0.

RESUME : Une étude, combinant les techniques d'éléments fins et de probabilité, a été effectuée pour évaluer l'effet de l'incertitude et de la variabilité associé au coefficient de perméabilité, sur le gradient hydraulique de sortie. Une distribution logarithmique normale a été utilisée pour assigner des perméabilités aux éléments du modèle. On a trouvé que la distribution des gradients de sortie peut se rapprocher d'une distribution normale, avec une moyenne similaire à celle d'un sol parfaitement homogène. Une limite supérieure du coefficient de variation du gradient de sortie a été obtenue pour une limite de confiance de 99 %, d'une distribution chi-carrée. Ce coefficient de variation a été utilisé ainsi que la solution théorique au problème de l'écoulement confiné autour du pilotage pour déterminer la probabilité que le phénomène de siphonnement se produise.

RESUMEN : Combinando las técnicas de elementos finos y de probabilidad, se ha efectuado un estudio para evaluar el efecto de la incertidumbre y variabilidad asociada con el coeficiente de permeabilidad, sobre el gradiente hidráulico de salida. Se ha utilizado una distribución logarítmica normal, para asignar permeabilidades a los elementos de la malla del modelo. Se encontró que la distribución de gradientes de salida, se puede aproximar con una distribución normal, con una media similar a la de un suelo perfectamente homogeneo. Se obtuvo un límite superior del coeficiente de variación del gradiente de salida, para un límite de confianza de 99 %, de una distribución chi-cuadrado. Este coeficiente de variación fue utilizado junto con la solución teórica al problema de flujo confinado alrededor de un tablaestacado, para determinar la probabilidad de que el fenómeno de sifonamiento ocurra.

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INTRODUCTION

The classical approach for the transmission of boundary energy through soil bodies has been to consider soil as a homogeneous, continuous medium. Such assumptions have permitted the discovery of deterministic solutions to many soil mechanics problems. When dealing with seepage problems the soil body was considered to be a continuous medium of many interconnected openings which serve as the fluid carrier. It has long been recognized that even for uniform spheres and ideal packings these interconnected openings are not regular but consist of cavernous cells interconnected by narrower channels. As natural soils contain particles which deviate considerably from a spherical shape and from uniform size, natural pore channels defy rational description.

In spite of the obvious uncertainties involved, important classes of seepage problems have been solved using a macroscopic deterministic approach. Fundamental to this approach is the assumption that the soil is homogeneous and that it has the same permeability at each point. These assumptions lend themselves readily to theoretical solutions.

What if the permeability is not the same at each point? Are the theoretical solutions still valid? Are the curves that define flownets still smooth and regular? This paper will investigate such questions with respect to one important class of seepage problems, i.e. the exit gradient for a system of confined flow exhibiting a downstream cut off.

PIPING

Piping is a consequence of the internal erosion of a soil mass beneath a water retaining structure. Piping is initiated when, at some point along the downstream soil surface, the weight of a submerged soil particle is acted upon by an unbalanced seepage force. In effect an energy transfer takes place. The total energy dissipated is that associated with the difference in water elevations from one side of a structure to the other. This energy transfer is called the head loss, and the head loss per unit length is the hydraulic gradient. The seepage force (Harr, 1962) can then be expressed as $i\gamma_w$ where i is the gradient. Equating this to the submerged unit weight of the soil results in a measure of the critical hydraulic gradient required for piping to begin. This gradient, i_{cr}, is approximately equal to unity. In general,

$$i_{cr} = (S_s-1)/(1+e) \qquad (1)$$

The factor of safety with respect to piping is the ratio of the critical gradient to the actual gradient along the discharge surfaces.

THEORY

The theoretical solution for the exit gradient for a vertical cut off, such as a sheet pile (see Figure 1) is (Harr, 1962)

$$I_e = \frac{h\pi}{4KTm}(1-m^2t^2)^{\frac{1}{2}} /m't \qquad (2)$$

where I_e = exit gradient
T = thickness of permeable layer
h = water head
K = complete elliptic integral of the first kind of modulus, m
m = $\sin\pi s/2T$
and m' = $\cos\pi s/2T$
where s = the depth of embedment of the sheetpile
t = $(\cot\pi s/2T)(\tan^2\pi x/2T + \tanh^2\pi s/2T)^{\frac{1}{2}}$
where x = horizontal distance from the sheetpile

For the considered section the maximum gradient occurs next to the sheetpile (x = 0) and is given as

$$I_e max = h\pi/4KTm \qquad (3)$$

It is standard procedure to employ (Khosla et al, 1954) a factor of safety ranging from 4 to 7. The lower value is for gravels and the upper value for fine sands.

COMPUTER MODELING

A finite element computer program was selected to model real soils, which can be highly variable. This analytical tool allows one to accomodate for variations in soil properties from point to point. The elements of a finite element grid can be thought of as representing small units of homogeneous soil that when grouped together represent a non-homogeneous mass.

As the resulting exit gradients were obtained for finite elements rather than at specific points along the discharge surface, a standard procedure for plotting the results had to be established. To do this, the vertical exit gradient for each element was considered to act along the vertical edge of the element (see Figure 2).

For s/T values in the range of interest, numerical values given by the finite element solution were found to be within approximately 10% of those given by the exact theoretical solution Equation (3) (see Figure 2), for homogeneous soils.

Initially a region of dimensions T by 3T was selected as shown in Figure 3a.

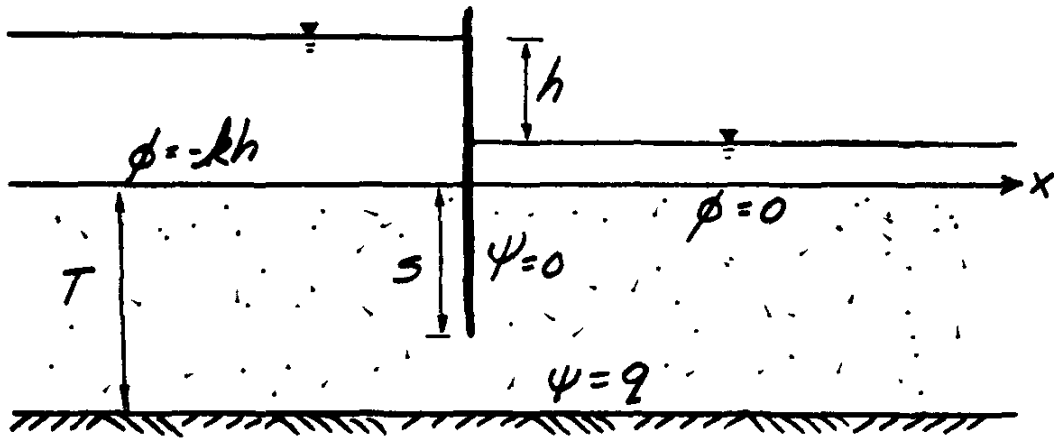
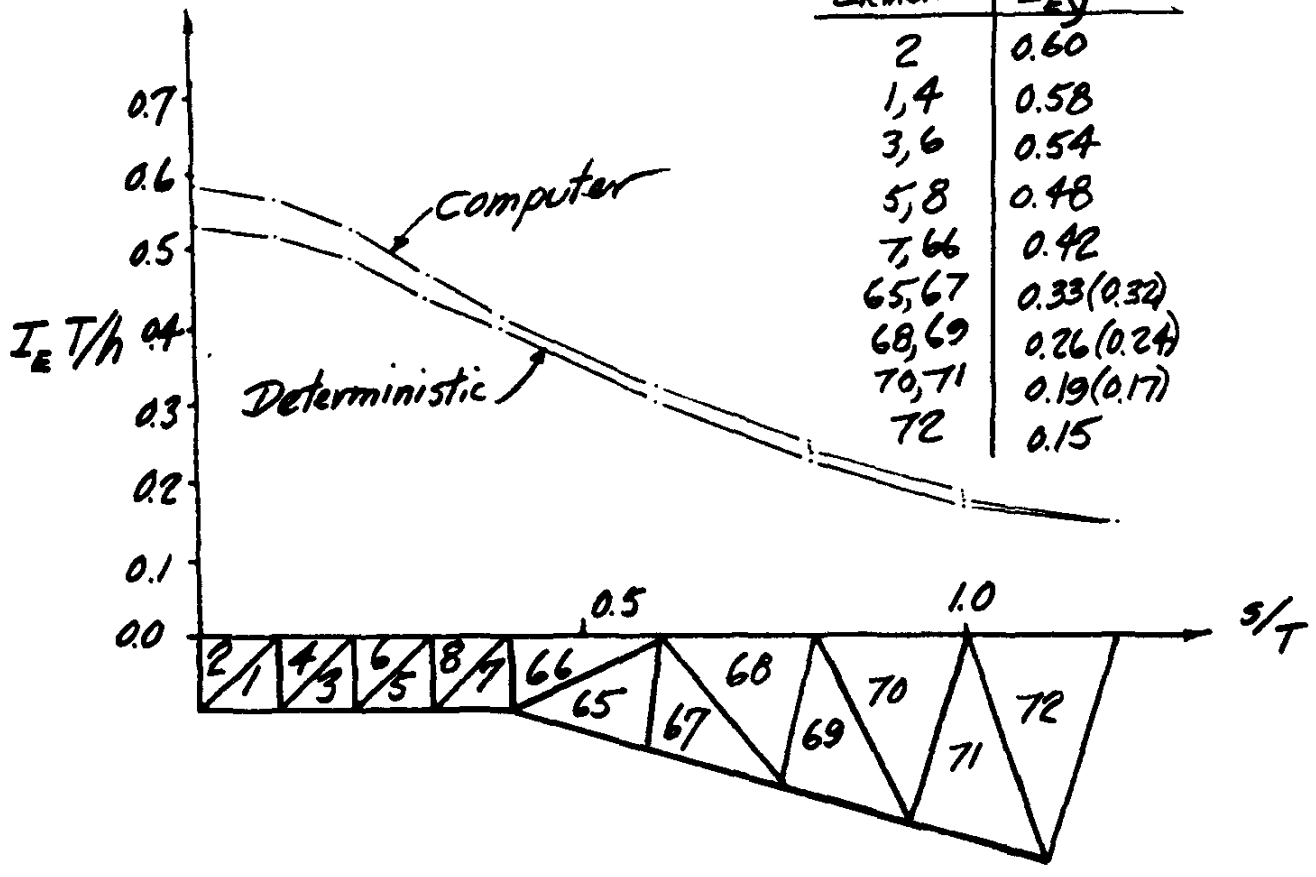


Figure 1: Geometric and Boundary Conditions

Figure 2: Computer vs. Deterministic Results

Element	I_{Ey}
2	0.60
1,4	0.58
3,6	0.54
5,8	0.48
7,66	0.42
65,67	0.33(0.32)
68,69	0.26(0.24)
70,71	0.19(0.17)
72	0.15



with the noted boundary conditions. This grid proved to be too large for efficient computer use and was reduced to the size shown in Figure 3b. The boundary conditions used along the right boundary of the new grid were taken to be the nodal values of that surface for the homogeneous conditions of the larger grid. A sensitivity analysis was performed which indicated that large changes in these boundary values had but very little influence on the exit gradients near the sheetpile.

To assign random permeabilities to the elements of the grid, a mean value of the coefficient of permeability was multiplied by a random number obtained from a lognormal distribution and assigned to an element. A lognormal distribution allows permeabilities to range from zero to infinity, thus representing soil units from a boulder to a complete void. The resulting element coefficients of permeability displayed three orders of magnitude.

RESULTS

The first computer run produced the results shown in Figure 4. This illustrates the degree of deviations that can be expected from exact theoretical solutions for even a slightly non-homogeneous soil mass.

100 runs were made for a ratio of s/T = 0.55. The coefficient of permeability was taken to be log-normal with lnk = N(0,1). The resulting exit gradients and their characteristics for each element on the discharge surface are summarized in Table 1. The element numbers are keyed to Figure 2. The mean values of the exit gradients for each element are also plotted with their corresponding deterministic values in Figure 5. The finite element solution is seen to conform quite well to the closed-form solution.

The statistical data in Table 1, the coefficients of skewness and kurtosis, β_1 and β_2, indicate that the distribution of exit gradients may be approximated by a normal distribution $\beta_1 = 0$, $\beta_2 = 3.0$ without undue error.

The coefficients of variation of the exit gradients are seen, Table 1, to vary from 34% to 44% for the elements sampled. A chi-squared analysis for a one-tailed 99% confidence limit gave an upper limit value of 45%.

PROBABILITY OF PIPING

With the distribution of the exit gradient taken to follow a normal distribution, the probability can easily be had of it exceeding its critical value; that is, the probability of piping. This is shown schematically on Figure 6 as the shaded area.

Figure 7 shows that the probability of piping decreases rapidly with increasing distance from the piling. This figure also indicates the contribution of the thickness of the layer to the security against piping. As noted, the region of insecurity is confined to a lateral distance from the piling of 0.2T, where T is the thickness of the permeable layer.

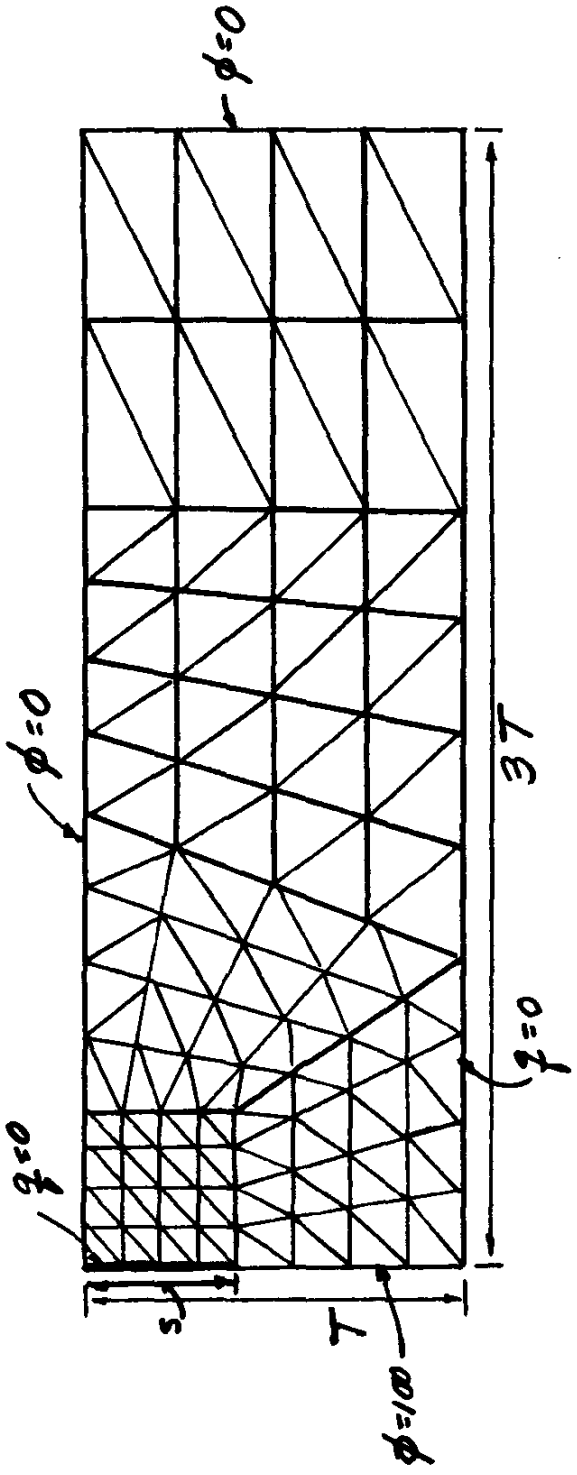
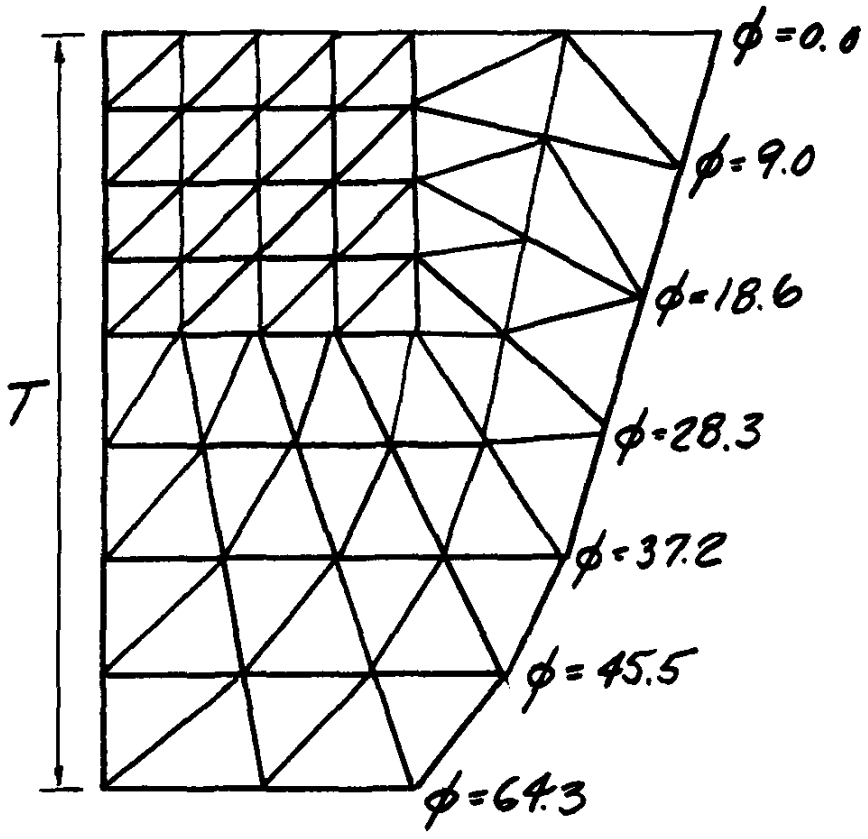


Figure 3a : Finite Element Grid

Figure 36: Finite Element Grid



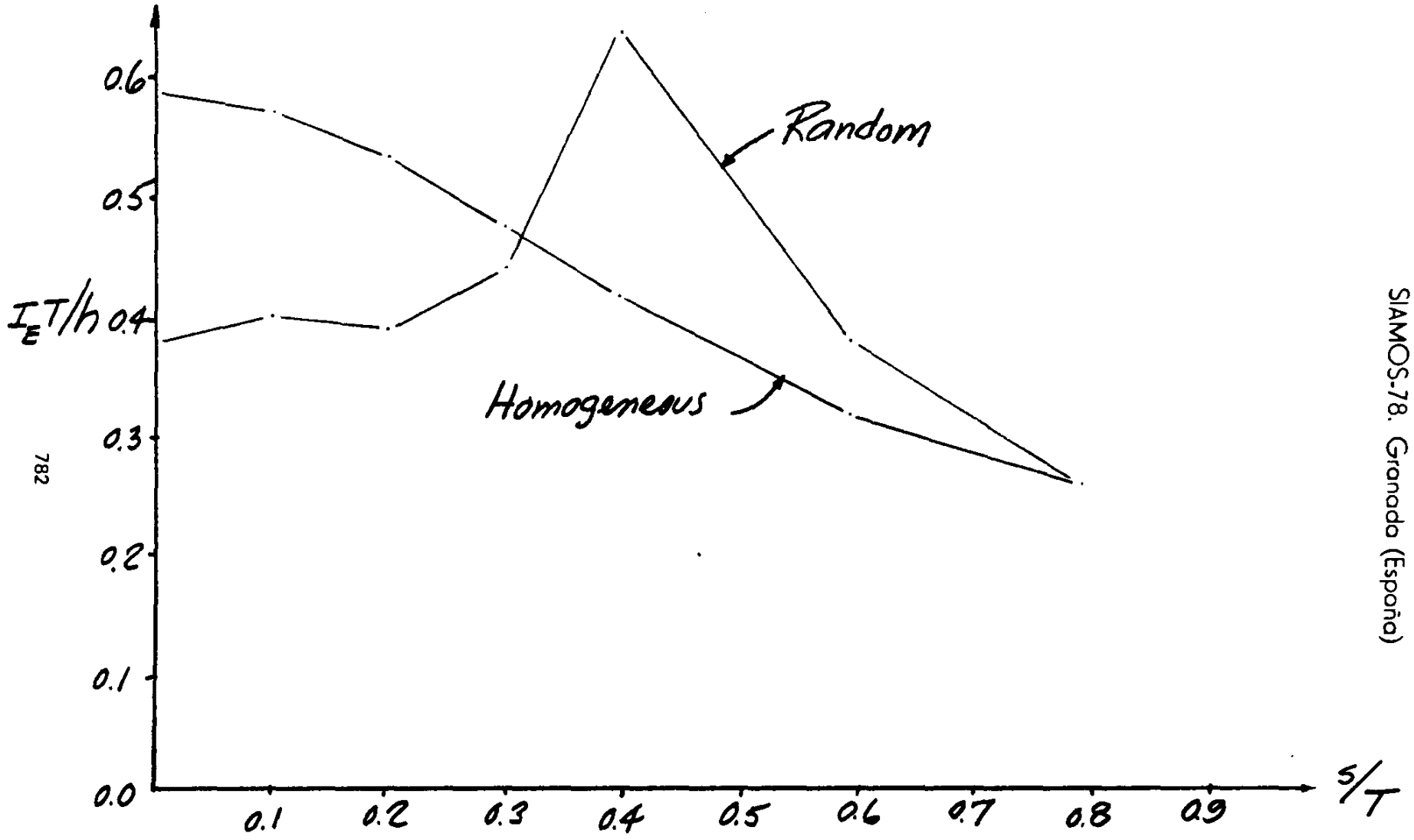


Figure 4: Single Random Run

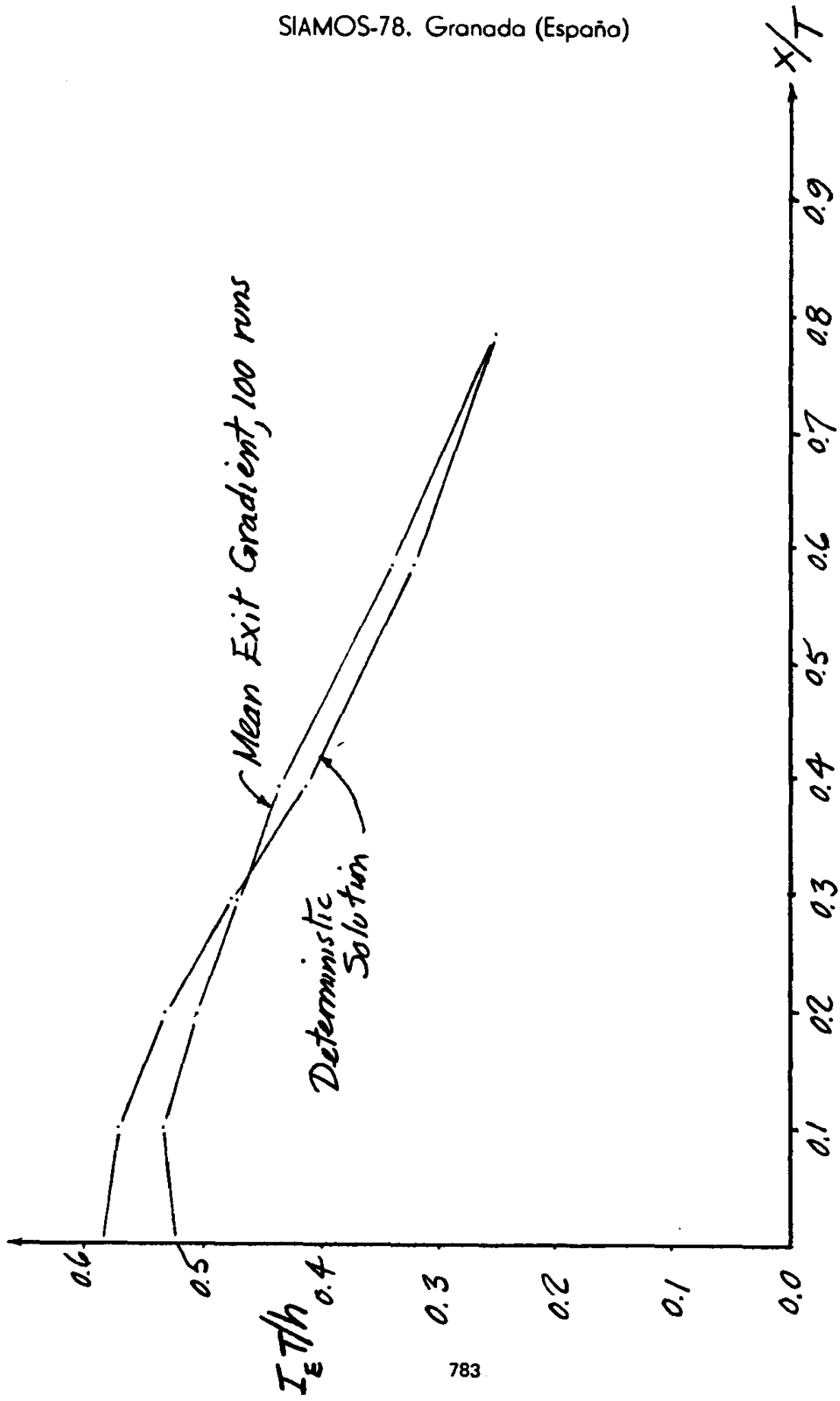


Figure 5: Exit Gradient (100 runs)

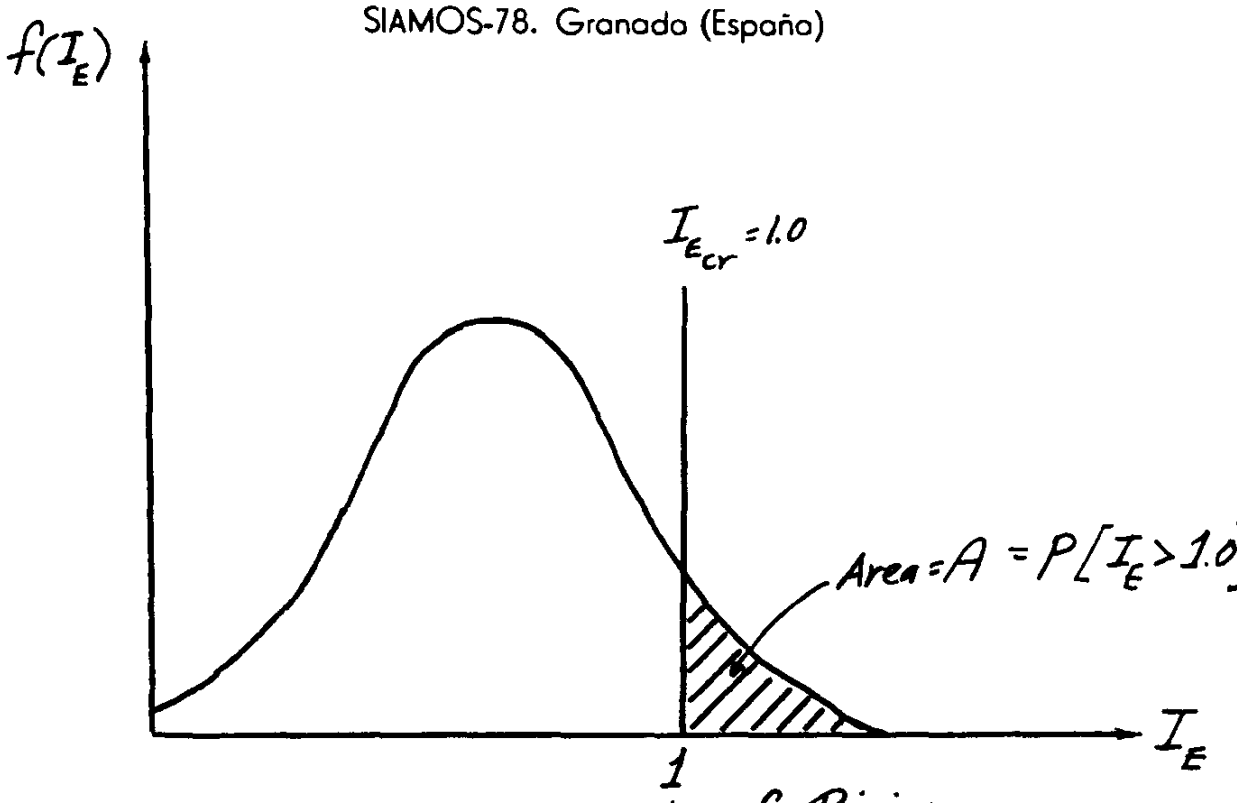


Figure 6: Probability of Piping

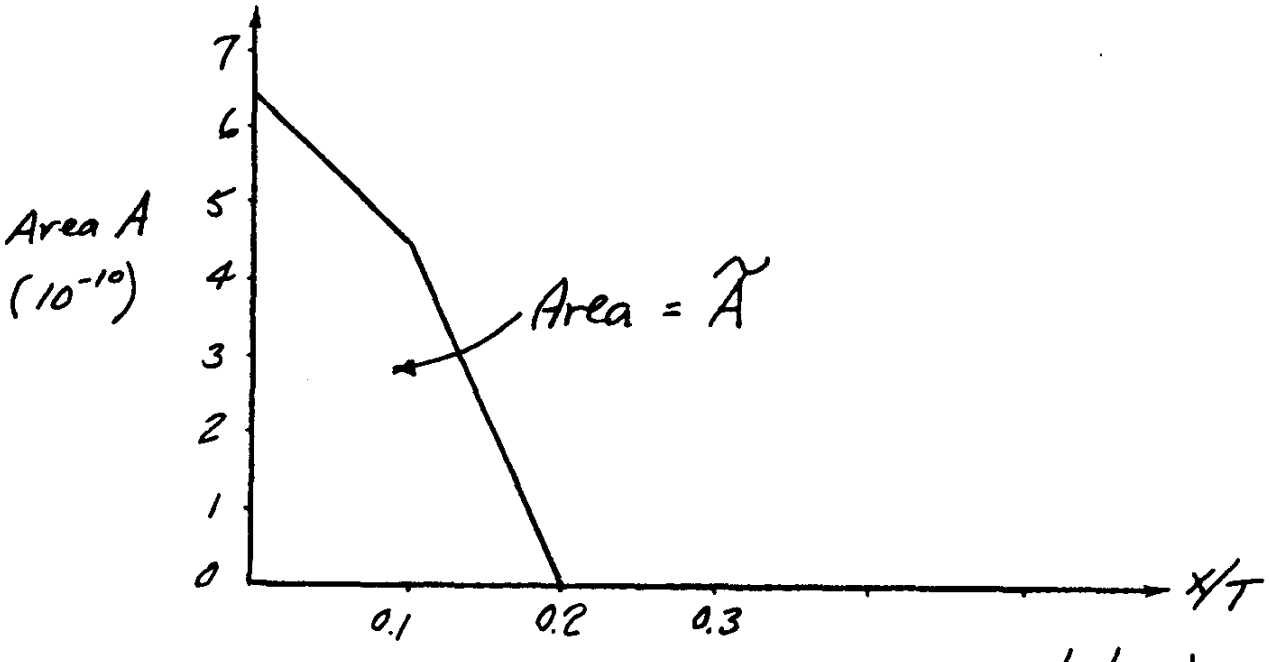


Figure 7: Probability of Piping vs Lateral Distance from Piling

Having a measure of the probability of piping at all points on the downstream discharge surface, the next matter at hand is to estimate the overall probability of the piling system, of length L.

Taking $\bar{d}$ to be the mean diameter of a soil particle in centimeters (such as the classical d_{10} size) the number of particles in one square meter of the discharge surface will be

$$N = (100/\bar{d})^2 \tag{4}$$

Defining $\bar{A}$ as the average probability of failure due to piping over the lateral distance of x = 0.2T, as shown to be the region of interest in Figure 7, we have that the probability of piping for the length of sheetpiling is

$$P_p = \bar{A}(0.2T)NL$$

Noting that $0.2\bar{A} = \tilde{A}$ or the area so designated in Figure 7, we have for the overall probability of failure

$$P_p = \tilde{A}TL(100/\bar{d})^2 \tag{5}$$

where T and L are in meters and $\bar{d}$ is in centimeters.

EXAMPLE

Given: s/T = 0.55, h/T = 0.50, T = 10m, L = 100m, medium sand with $\bar{d}$ = 1.0mm.

Find: probability of piping.

Solution: Obtain the deterministic solution for the exit gradient from Equation (2) for some values of x/T. These are given in the second column of Table 2. The corresponding values for the standard deviation, S_{I_E}, with a coefficient of variation of 45% are given in column 3.

The standard normal variates for the critical gradient $I_E = 1$, $z = (1 - I_E)/S_{I_E}$, are listed in column 4 of the Table.

The corresponding normal variates are tabulated in column 5. These can be had from the expression (Harr, 1977)

$$P[I_E \geq 1] = \frac{1}{z}(2\pi)^{-\frac{1}{2}} \exp[-z^2/2] \tag{6}$$

These are translated into $\bar{A}$ values in column 6 and finally into $\tilde{A}$ in column 7. Hence, for the given conditions, we have from Equation (5)

$$P_p = \tilde{A}TL(100/\bar{d})^2 = 6.63\text{x}10^{-11}(10\text{m})(100\text{m})(1\text{x}10^6)/\text{m}^2 = 6.63\text{x}10^{-2} \text{ or } 6.6\%$$

Thus, the reliability of the system is 93.4%. It is noted that the factor of safety for this case is 1/0.27 = 3.7.

In Table 3 are given some factors of safety (F.S.), their corresponding probabilities of piping (P_p) and their reliability ($1 - P_p$).

Khosla, Bose and Taylor (1954) recommend that the following factors of safety be applied as critical values of exit gradients: gravel, 4 to 5; coarse sand, 5 to 6; fine sand, 6 to 7.

SUMMARY AND CONCLUSIONS

a) A hybrid finite-element, probabilistic model was proposed to determine the probability of failure due to piping in a non-homogeneous layer exhibiting a single sheetpile cut-off.

b) It was found that the distribution of the exit gradient at a point on the discharge surface can be represented as a normal distribution with a mean value equal to the deterministic exit gradient and with a coefficient of variation of 45%.

c) The critical zone for piping is along the discharge surface within a distance of 0.2T from the piling, where T is the thickness of the homogeneous layer.

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Table 1

Element Number (see Figure 2)	2	1/4	3/6	5/8	7/66	65	67
Mean exit gradient $\bar{x}$	0.533	0.544	0.516	0.480	0.443	0.339	0.332
Standard deviation s	0.232	0.226	0.205	0.163	0.162	0.123	0.097
Coefficient of variation V_c	44%	42%	40%	34%	37%	36%	29%
Coefficient of kurtosis β_2	2.54	3.18	3.75	2.88	2.69	2.54	2.63
Coefficient of skewness β_1	0.45	0.62	0.85	0.06	0.19	0.44	0.45
Deterministic homogeneous value, I_E	0.595	0.580	0.538	0.483	0.422	0.326	0.321

Table 2

1 x/T	2 I_E	3 S_{I_E}	4 z	5 $P[I_E>1]$	6 $\bar{A}_i$	7 $\tilde{A}_i$
0.0	0.27	0.12	6.08	$6.18(10^{-10})$	$4.85(10^{-10})$	$4.85(10^{-11})$
0.1	0.26	0.12	6.17	$3.51(10^{-10})$	$1.78(10^{-10})$	$1.78(10^{-11})$
0.2	0.25	0.11	6.82	$0.05(10^{-10})$	$0.02(10^{-10})$	-
0.3	0.22	0.10	7.80	-		
						$\tilde{A} = 6.63(10^{-11})$

Table 3

Soil	$\bar{d}$	s/T	h/T	I_E	F.S.	T	P_n (% for 100m)	Reliability
Gravel	25mm	0.55	0.50	0.27	3.73	10m	<0.1	>99.9
Coarse Sand	5mm	0.55	0.50	0.27	3.73	10m	0.3	99.7
Fine Sand	1mm	0.55	0.50	0.27	3.73	10m	7.8	92.2
Fine Sand	1mm	0.55	0.50	0.27	3.73	5m	3.9	96.1
Gravel	25mm	0.55	0.55	0.30	2.95	5m	0.5	99.5
Coarse Sand	5mm	0.55	0.55	0.30	2.95	5m	3.2	96.8
Fine Sand	1mm	0.55	0.55	0.30	2.95	5m	≈100	0
Gravel	25mm	0.70	0.65	0.26	3.91	10m	<0.1	>99.9
Gravel	25mm	0.70	0.80	0.31	3.18	10m	17.4	82.6
Fine Sand	1mm	0.85	0.90	0.25	3.94	10m	0.4	99.6

SYSTEMS APPROACH FOR MINE WATER CONTROL

Kesserü, Zs.*; Bogárdi, I.*; Szidarovszky, F.**

ABSTRACT : Design and operation of mine water control warrants the use of systems analysis. Objectives of the system are defined (economic, life protection, environmental). A system model of economic mine water control is presented. Elements of the systems model are formulated : input, state variable, output, state transition function, output function. Due to stochastic natural environment, the systems model of mine water control has also stochastic elements. The model refers to Hungarian mining conditions but principles are common for any kind of mining under water hazard.

RESUME : La conception et la mise au point des modes de protection contre les eaux souterraines rendent possible d'utiliser des méthodes d'analyse de système. Les objectifs de l'emploi de ce système sont définis (rentabilité, protection de la vie et environnement). On présente un modèle de système économique pour la lutte contre les venues d'eau. Les éléments de ce dernier sont : entrées, variables d'état de transition et sorties. Par suite de l'environnement naturel de caractère stochastique, le modèle de système comporte également des éléments stochastiques. Mis au point pour les conditions minières hongroises, il est basé cependant sur des principes généraux valables dans n'importe quelle mine luttant contre les eaux.

RESUMEN : La concepción y puesta a punto de los métodos de protección contra la irrupción de aguas subterráneas, hacen posible el empleo de la técnica de análisis de sistemas. Se definen los objetivos del empleo de estos sistemas (economía, protección de la vida y medio ambiente). Se presenta un modelo económico para la lucha contra las irrupciones de agua en la mina. Los elementos de este sistema son : entradas, variables de estado, función de transición y salidas. Debido a la naturaleza estocástica del medio ambiente, el modelo tiene igualmente elementos estocásticos. El modelo, puesto a punto para minas húngaras, está basado en principios generales válidos en cualquier trabajo minero con problemas de agua.

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1. INTRODUCTION

This paper analyses typical objectives /life-protection, economic, environmental/ of mine water control systems, reviews some Hungarian works on systems approach for mine water control and introduces a new mathematical model.

Dynamic programing /DP/ model can be used for an economic optimal design of mine water control system. Under Hungarian conditions the control of mine water is one of the key elements of the mining production and especially in the case of planned mines in the Transdanubian Mountains, near Budapest, it is of primary importance to develop mine water control systems appropriate from both economic and engineering aspects [KAPOLYI, 1977]. The fact is that due to the natural conditions a considerable part, 10-20 % of total production costs is related to water control in that region. Thus, the economics in mine water system may affect the economic feasibility of the overall mining activity. The task is aggravated by the fact that water hazard - similarly to other natural hazards - can be known in advance only with uncertainty; thus stochastic methods are required. In addition to the task of finding economic mine water control systems, environmental criteria should be also observed which are related in that region mostly the water budget of the regional karstic aquifer [SZILÁGYI et al., 1978].

A number of engineering methods for mine water control has been elaborated [SCHMIEDER et al., 1975] and recently a new control method, the so called "instantan" method has been developed which seems to be more effective than the previous ones [KAPOLYI, 1975].

This paper describes first the objectives of mine water control and gives examples on the application of systems analysis in that respect /Section 2/. Next, the typical problem of mine water control, pertinent to the Budapest region /Section 3/ is outlined. In this vein, the mine water hazard is characterized, and principles of its calculation are shown. Possible alternatives of control methods are summarized and water budget constraints due to environmental protection are discussed.

The core of this paper aims at the elaboration of the DP model /Section 4/. First, main assumptions are fixed, variables are defined and the objective function is formulated. State variable, the yield of mine water inrushes can be regarded as a random variable, thus a stochastic DP problem should be solved. Advantages of the DP formulation are evident in view of the computational saving as compared to optimum seeking with total enumeration.

In Section 5 numerical data and functions for the application of the model are provided. Numerical results on the optimal mine water control system will be given at the conference.

Section 6 summarizes conclusions reached and recommendations for further research with the ultimate goal that the model be applicable under similar mining conditions and model principles be useful to solve other kind of sequential development problems of mining /e.g. ventilation, gas protection, haulage/.

2. MINE WATER CONTROL

Mine water control constitutes a subsystem of the mining system and is connected, on the one hand, to other subsystems /haulage, ventilation, etc./ through common elements /e.g. cuts/,and,on the other hand, to the natural environment, through drainage wells, sealing holes, etc. Engineering definition of mine water control systems can be found in SCHMIEDER et al. [1975] . Mine water control induces changes in the interaction between rocks and water around mining spaces [KESSERÜ, 1978]. Mine water phenomena include water movements, as inrushes and movements of gas and rock, caused by mine water. Among these movements those induced by the mining activity are regarded as water hazard and those induced by control elements mean the effect of the control system.

2.1. Objectives of mine water control

Objectives of a mine water control system are related to life protection, economics and the environment.

The effectiveness of life protection can be measured by the risk of fatal accidents caused by mine water phenomena.

The economic objective calls for such a control which results in minimum water related costs and mining losses regarding the total life-time of the mine. In many cases the economic control means a constrained optimum, since e.g. an annual production rate is to be maintained.

The environmental objective refers to the smallest disturbance of the natural environment. The effectiveness of reaching this objective can rarely be expressed in monetary units but in natural units of water quality, quantity, land subsidence, etc.

2.2. Systems analysis of mine water control

Systems analysis seeks to answer the following main questions.

a./ What is the required or optimal degree of reaching different objectives?
b./ What is the system satisfying the above objectives?

Concerning life protection, investigations have been made to estimate the required magnitude of the risk of fatal accidents mentioned [SCHMIEDER et al., 1975]. As a result of these investigations Hungarian mining safety regulations prescribe system elements satisfying life protection objective. Among the three ob-

jectives life protection requirements are thus generally fixed, no trade-off with other objectives can be analysed, and as constraints can be included in a systems model.

An example of the economic analysis of mine water control system can be found in KESSERÜ and PRUZSINA [1970], who estimated an economic capacity for the conveyance system and the pumping station.

It has been demonstrated that the economic and environmental objectives are often in conflicts with each other [BOGÁRDI et al., 1978]. In this paper the environmental objective is considered as a constraint, namely the upper limit of maximum drained water. Since numerical value of this constraint can be changed the model shown in Section 4 is regarded as one phase of a multiobjective analysis.

Clearly, a mine water control system has a number of interrelated natural and manmade elements. Though the reliability of such a system is not easy to estimate, investigations have been made to determine system reliability, that is,1 - probability of failure, since this is a key figure of design and operation [SCHMIEDER et al., 1975].

In the next Section typical mine water control in the Budapest region is briefly described.

3. MINE WATER CONTROL IN THE BUDAPEST REGION

3.1. Water hazard

Coal and bauxite deposits are located under the karstic water level in a fractured rock aquifer. Water hazard may stem from either the upper layers or the bed rock material. Water inrushes would yield more or less sediment into the underground spaces. The amount of sediment can be estimated from inrush data /flowrate, volume/ and rock properties. Total yield of water inrushes into a given space depends on rock properties, piezometric pressure, and the magnitude of the space, which, in turn, is known from the mining plan.

Total yield of inrushes during a time period /stage/ can be taken as the sum of inrush events along the space corresponding to the stage considered. Subsequent inrush events are practically independent. Both the magnitude and number of inrush events can often be regarded with good approximation as random variables [SCHMIEDER et al., 1975]. As a result, stochastic methods are available to characterize water hazard [SCHMIEDER, 1976], and in this way probability density function /pdf/ of the total yield of inrush events per stage can be estimated. The method of estimation is based on geological analogy and/or the measured rock properties.

Direct statistical samples of inrushes for the planned mine cannot evidently be available. However, as mining operation starts and proceeds, more and more samples on water inrush events could be observed. Thus, pdf of inrush events and total yield can already be estimated with greater confidence for the following stages.

3.2. Control system

The mine water control system /Fig. 1/ consists of two main parts

a./ protection of production activity, e.g. faces by various methods, and

b./ water /and sediment/ conveyance and pumping equipments.

The economic analysis may cover both parts of the control system or either of them. This paper considers the first part, thus decision alternatives considered include fixed conveyance and pumping design /cuts, shafts, pumping station/. The common practice is accepted, that the conveyance and main pumping equipments are designed with due allowance to a high reliability prescribed in safety regulations and to other requirements /ventilation, haulage, etc./.

Control methods of mine water can be divided into two main groups [SCHMIEDER et al., 1975]:

a./ Preventive /active/ protection aims at the elimination, dedrease or postponement of water inrush occurrences by
- sealing with some artificial protective layer or with the use of natural protective layers;
- dewatering with shafts and wells out of the mining space to decrease piezometric head.

b./ Subsequent /passive/ protection covers
- the collection, conveyance, and pumping of water from mining spaces;
- the abandoning of, and the rescue from flooded spaces;
- the drainage of the flooded spaces.

In most of the cases the above methods are applied in a proper combination. A recently recommended combination is the instantan method by which water inrushes to be caused by the mining are brought about in advance in borings or cuts [KAPOLYI, 1975].

Thus, the decision model on mine water control design should be capable of selecting among the various alternatives including the combinations thereof.

In the case of the planned mines investigated we seek a proper combination of
- the instantan method,

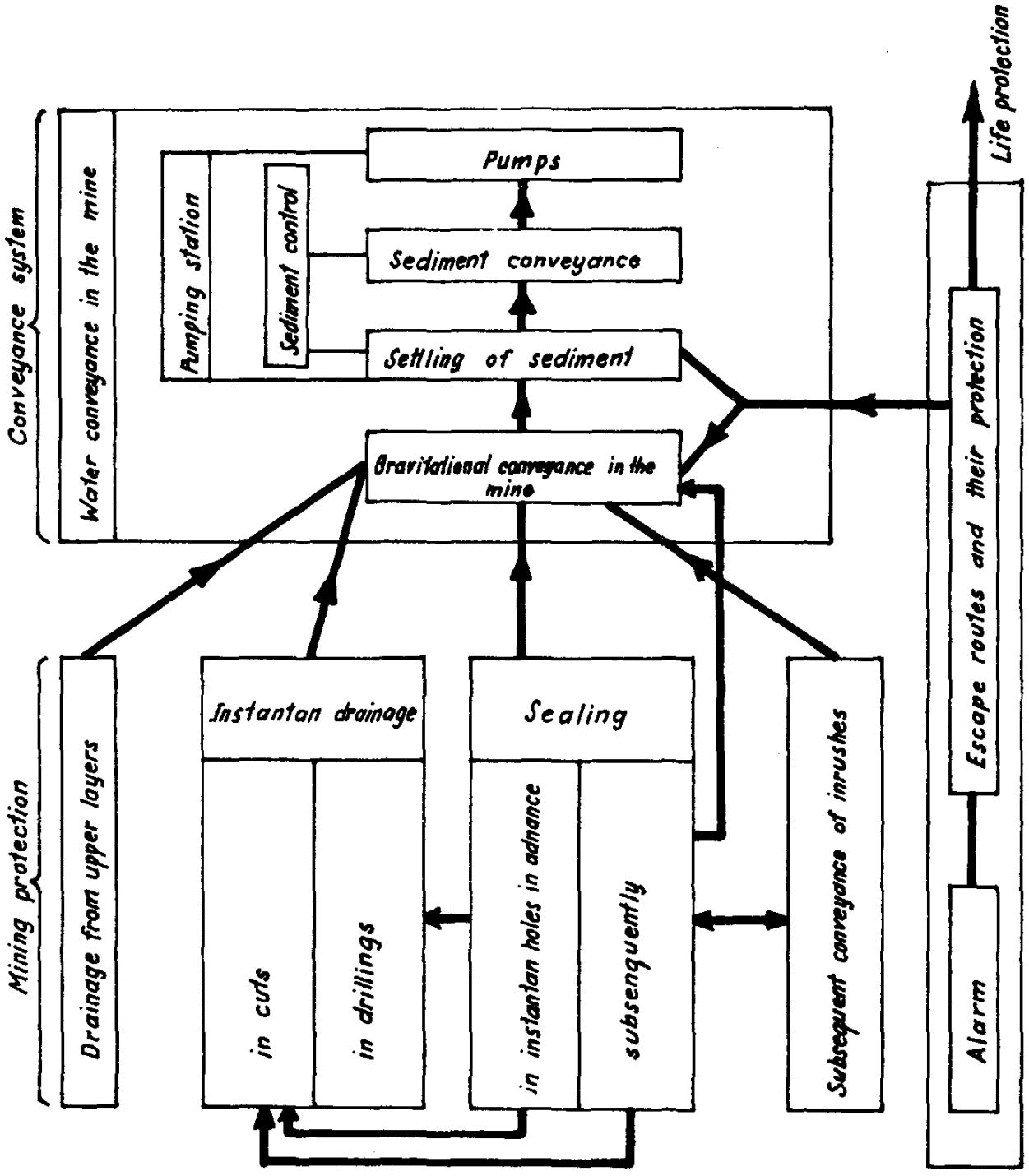
Life protection
Escape routes and their protection
Alarm
Conveyance system
Water conveyance in the mine
Pumping station
Sediment control
Pumps
Sediment conveyance
Settling of sediment
Gravitational conveyance in the mine
Mining protection
Drainage from upper layers
Instantan drainage
in cuts
in drillings
Sealing
in instantan holes in advance
subsequently
Subsequent conveyance of inrushes

Fig. 1

- dewatering,
- sealing /in advance and after inrushes/, and
- subsequent protection.

By the help of such a combination a reduction can be reached in:
a./ the total yield of inrushes /by sealing/,
b./ the ratio of yields in the production spaces to the total yield /in the holes of instantan and other preventive drainage methods/.

The realization and operation of mine water control measures requires considerable costs. If subsequent protection is applied more or less damages should be also reckoned with. These are related to less production, adverse working conditions, smaller efficiency of machine fleet and other loss factors [KESSERÜ, 1977]. Construction and operation costs as well as the losses occur in different stages during the lifetime of the mine. As a consequence, present value /discounted/ costs and losses should be calculated and compared.

The environmental objective refers in the region to the least disturbance of the regional karstic water budget. Namely it might be possible that such a high amount of mine water should be drained /economic optimum/, that cannot be tolerated due to environmental hazards /drying-out of waterworks, thermal baths, etc./.

4. DYNAMIC PROGRAMMING MODEL

DP model is developed in four steps. First the main assumptions are given /4.1./, and model variables and functions are defined /4.2./ as state variable, input, output, state transition function and output function. Next the goal function of the model is formulated /4.3./ and the way how DP algorithm can be used for the optimalization is shown /4.4/.

4.1. Assumptions

Decision model is based on the following three assumptions:

I. Mineral resources, desired annual production and, thus, the life-time T of the planned mine is given. Total life-time can be divided into characteristic stages $t = 1, 2, \ldots, i, \ldots, T$. Stages may be years; however, the periods between stages may be as desired and not necessarily constant.

II. On the basis of a mining sheduling plan, for each stage the area of underground spaces $F(t)$ is denoted which consists of the operating and abandoned faces in various blocks.

III. The pdf of the total yield of water inrushes per stage, $f\ [\Delta \underset{\sim}{Q}(t)]$ can be estimated.

Symbol ~ refers to a random variable.

4.2. Definition of the variables and functions
Input $\tilde{Q}(t)$ for stage t is the total yield of water inrushes until this stage; a random variable.

State variable is the total yield of inrushes, $\Delta\tilde{Q}(t)$ during stage t.

Output $\tilde{Q}(t+1)$ for stage t is the total yield of inrushes after stage t, this output equals the input for stage $(t+1)$. Output is thus determined by the input, state variable and decision set defined next. For each stage t we can decide on the method of water control, to be used over the area $F(t)$. Possible set of decisions can be characterized by vector $\underline{d}(t)$; the components thereof consist of the specifications of possible water control methods outlined in the previous section, e.g. the total length of instantan borings, the number of sealings /groutings/, total amount of sealing material, all for stage t.

State transition function can be given thus as

$$\tilde{Q}(t+1) = g\left[\tilde{Q}(t),\ \Delta\tilde{Q}(t),\ \underline{d}(t)\right] \qquad (1)$$

where function g can be of any type, not necessary linear, given sometimes in a table form with discrete numerical values.

If density function $f[\Delta\tilde{Q}(t)]$ is given for each stage, starting from $t = 0$ we can calculate pdf of output $\tilde{Q}(1)$ for each possible decision in $t = 1$, then pdf of $\tilde{Q}(2)$ etc. It might happen that simulation is necessary to calculate these pdf in the case of complicated distributions. Simple formulas can be used in every case, however, to calculate the expected value and variance of output $\tilde{Q}(t+1)$ [BENJAMIN and CORNELL, 1970].

Output function regards total water-related costs incurring during state t in the function of input, state variable and decision vector:

$$\tilde{\underline{r}}(t) = D_t R_t\left[\tilde{Q}(t),\ \Delta\tilde{Q}(t),\ \underline{d}(t)\right] \qquad (2)$$

where D_t is the discount factor in order to calculate present values /mostly for $t = 0$/ of costs incurring in different stages; R_t is the output function which covers water-related costs of investment, operation and maintainance during stage t.

In practice, function R_t is generally nonlinear, estimated in discrete points and given graphically or in table. It can be seen that as a result of the random state variable, output function $\tilde{\underline{r}}(t)$ is also random.

4.3. Goal function
Such a mine water control system is sought that water-related costs be minimum throughout the total life-time. Thus the goal function is

$$\underset{\sim}{Z} = \sum_{t=1}^{T} \underset{\sim}{r}(t) \qquad (3)$$

This goal function is to be minimized by finding a sequence of decisions, $\underline{d}_1^*$, $\underline{d}_2^*$, ... $\underline{d}_T^*$ that

$$\underset{\sim}{Z}^* = \min \sum_{t=1}^{T} \underset{\sim}{r}(t) \qquad (4)$$

$$\underline{d}_1^*, \underline{d}_2^*, \ldots, \underline{d}_T^*$$

Since members of the goal function are not independent, it is not possible to minimize from stage to stage. This can be seen directly from state transition function (1), since $\underline{d}(t)$ realized in stage t affects inputs for all later stages, thus all output functions $\underset{\sim}{r}$. In reality, this may mean that it not advisable to drain high yields in the initial stages of mining, that is, some sealing is required in order to eliminate excessively high inputs $\underset{\sim}{Q}(t)$ in later stages.

It is a further problem that for each random value $\underset{\sim}{Z}$ of the goal function an optimum sequence of decisions could be found. In such case the expected value of the goal function is common to be minimized:

$$Z^* = \min E \left[\sum_{t=1}^{T} \underset{\sim}{r}(t) \right] \qquad (5)$$

Output $\underset{\sim}{Q}(T)$ of the last stage T corresponds to the highest amount of karstic water drained. For environmental reasons a maximum amount Q can be specified. This upper limit means that a high probability, α should be prescribed that output $\underset{\sim}{Q}(T)$ will be less than the environmental limit:

$$P\left[Q(T) < Q\right] \geq \alpha$$

4.4. DP algorithm
Model of mine water control /Fig. 2/ constitutes a model with serial elements /stages/. Since output in t equals input in (t+1), optimization of the goal function (5) can be performed by dynamic programming [BELLMAN, 1962]. It is a final value problem since an upper limit for $\underset{\sim}{Q}(T)$ is prescribed. As a consequence forward dynamic programming can be used, with the recursive equation

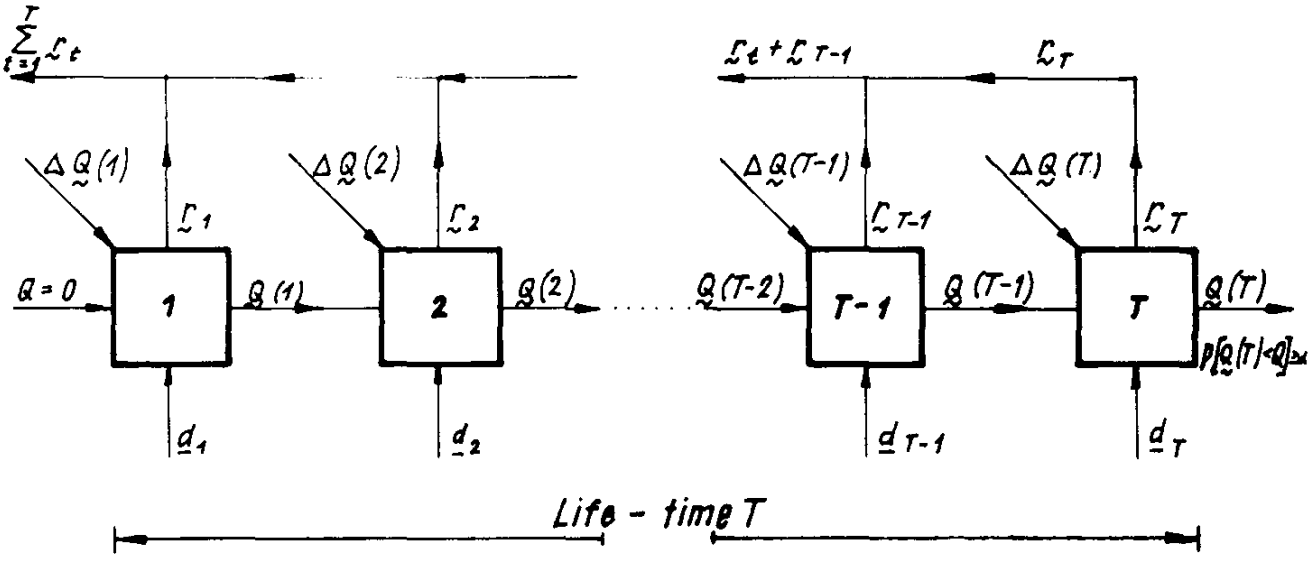

Fig. 2.

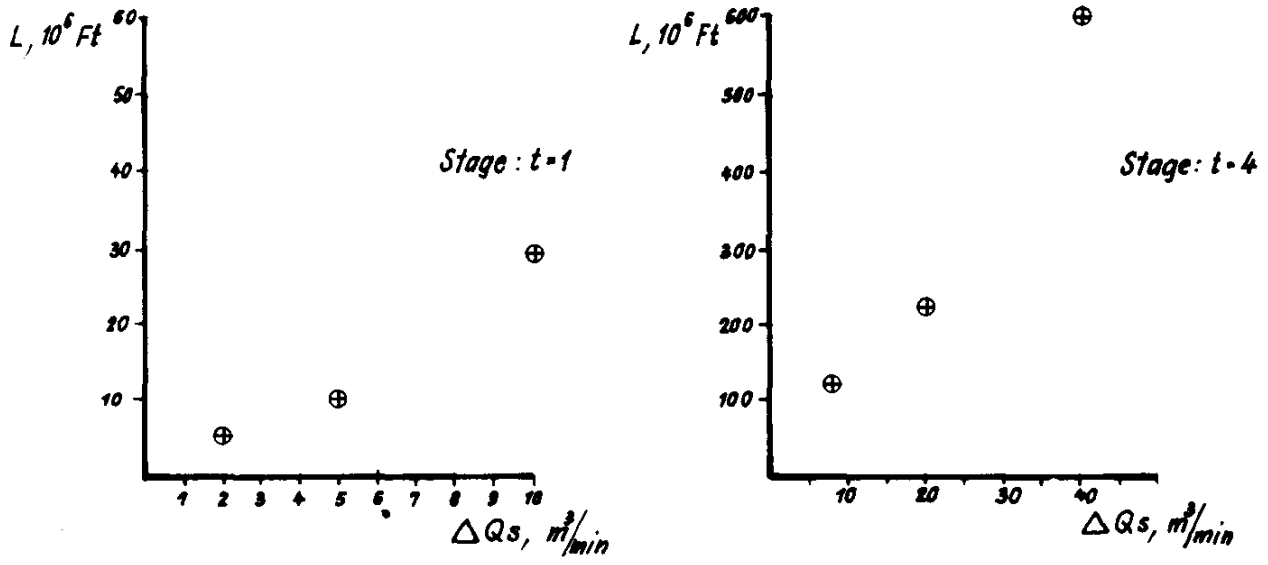

Fig. 3.

$$b_{i+1}\left[Q(i+1)\right] = \min_{\underset{\sim}{d}_{i+1}} \left\{ E\left[\underset{\sim}{r}_{i+1}\left(\underset{\sim}{d}_{i+1},\ \underset{\sim}{Q}(i+1)\right)\right] + b_i\left[\underset{\sim}{d}_{i+1},\ \underset{\sim}{Q}(i+1)\right] \right. \quad (6)$$

where b is the minimum of the expected value sum of output functions $\underset{\sim}{r}$ until stages 1, 2, ..., i:

$$b(i) = \min E\left[\sum_{t=1}^{i} \underset{\sim}{r}(t)\right] \quad (7)$$

Thus, recursively, from stage to the next stage the final stage T can be reached and b(T) will be expressed in the function of Q(T). Then the environmental limit Q, that is, the final value can be substituted in the recursive equation for b(T), and b(T-1), b(T-2) ..., b(1) can be calculated backward by the help of state transition functions, and the optimal sequence of decisions, $d^*(T)$, $d^*(T-1)$ ..., $d^*(1)$ can be found.

Due to the random state variable stochastic DP must be applied [METLER et al., 1975], which does not mean the change of DP principles but multiplies computation requirements. In that respect, the type of pdf for the state variable is of great importance. In the case of discrete probabilities or some basic distributions /e.g. normal/ difficulties are not expected, but often simulation is the only available tool to optimize numerically by the DP algorithm.

In the next Section data referring to a planned mine in Hungary are given.

5. EXAMPLE DATA FOR THE MODEL

In this Section a real-life mine water control problem of a planned Hungarian mine is regarded. The necessary data for using the model are given in order to facilitate model application in other cases.

5.1. Stages and mine water inrushes per stages.

Table 1 contains characteristic time periods, that is, stages for a mine with a total life-time T = 30 years. Also, statistics /mean and standard deviation, s.d./ of total steady flow of inrushes per stages, $\Delta\underset{\sim}{Q}(t)$ are given.

Table 1

Period t	Duration years	$\Delta\underset{\sim}{Q}(t)$, Total flow m³/min mean	s.d.
1	3	10	7
2	7	15	7
3	8	20	7
4	7	40	10
5	5	5	5

The pdf of $\Delta\underset{\sim}{Q}(t)$ can be taken as lognormal.

3.2. Decision set

$$\underline{d}(t) = [d_c(t),\ d_d(t),\ d_s(t)]$$

$d_c(t)$: total length of cuts for "instantan" drainage during period t, in thousands of meter;

$d_c(t)$: total length of drillings for "instantan" drainage during period t, in thousands of meter;

$d_s(t)$: total amount of sealing during period t, in thousands of m^3.

From the theoretically infinite number of decision alternatives, a discrete set of possible alternatives has been selected for that example. Table 2 shows that in each stage 8 possible decisions are assumed. In real life the number of alternatives may be much higher but model principles will not change.

Table 2

Discrete set of decision alternatives

Period, t		decision alternatives							
		1	2	3	4	5	6	7	8
1	d_c	0	3	3	3	3	3	3	3
	d_d	0	0	6	10	6	6	10	10
	d_s	0	0	0	0	4	7	4	7
2	d_c	0	36	36	36	36	36	36	36
	d_d	0	0	72	110	72	72	110	110
	d_s	0	0	0	0	6	15	6	20
3	d_c	0	40	40	40	40	40	40	40
	d_d	0	0	80	140	80	80	140	140
	d_s	0	0	0	0	8	16	8	20
4	d_c	0	40	40	40	40	40	40	40
	d_d	0	0	80	140	80	80	140	140
	d_s	0	0	0	0	14	40	14	50
5	d_c	0	6	6	6	6	6	6	6
	d_d	0	0	12	20	12	12	20	20
	d_s	0	0	0	0	2	6	2	8

5.3. State transition function

$$Q(t+1) = Q(t) + \Delta Q(t) + \Delta Q\left[d_s(t)\right] \qquad (8)$$

where $\Delta Q\left[d_s(t)\right]$ is the sealed flow of inrushes in t. The total flow $\Delta Q(t)$ to be pumped up to the surface, can be expressed

$$\Delta Q(t) = \Delta Q\left[d_c(t),\ d_d(t)\right] + \Delta Q_s(t) \qquad (9)$$

where $\Delta Q\left[d_c(t),\ d_d(t)\right]$ is the flow of inrushes drained in cuts (d_c) and drillings (d_d) of instantan control;

$\Delta Q_s(t)$ is the portion of total flow of inrushes appearing in mining operation spaces and causing economic losses.

For the sequential calculation of state transition function (9) it is necessary to fix $Q(o) = 0$ and $P\left[Q(T) \leq Q = 120\ m^3/min\right] \geq \alpha = 0{,}95$. In addition, values of $\Delta Q(d_s)$, $\Delta Q(d_c, d_d)$ and ΔQ_s are given for every stage and decision alternatives in Table 3 after KAPOLYI [1976].

Table 3

Period, t stage		Decision alternatives 1	2	3	4	5	6	7	8
1	$\Delta Q(d_s)$	0	0	0	0	6	8	8	10
	$\Delta Q(d_c,d_d)$	0	5	8	10	2	0	2	0
	ΔQ_s	10	5	2	0	2	2	0	0
2	$\Delta Q(d_s)$	0	0	0	0	7	12	10	15
	$\Delta Q(d_c,d_d)$	0	8	12	15	5	0	5	0
	ΔQ_s	15	7	3	0	3	3	0	0
3	$\Delta Q(d_s)$	0	0	0	0	12	16	15	20
	$\Delta Q(d_c,d_d)$	0	10	16	20	4	0	5	0
	Q_s	20	10	4	0	4	4	0	0
4	$\Delta Q(d_s)$	0	0	0	0	22	32	30	40
	$\Delta Q(d_c,d_d)$	0	20	32	40	10	0	10	0
	ΔQ_s	40	20	8	0	8	8	0	0
5	$\Delta Q(d_s)$	0	0	0	0	2	3	4	5
	$\Delta Q(d_c,d_d)$	0	3	4	5	1	0	1	0
	ΔQ_s	5	2	1	0	2	2	0	0

Data in Table 3 represent mean values. Coefficient of variation in $\Delta Q\ (d_c, d_d)$ is the same as in $\Delta Q(t)$. However, the effect of sealing, $\Delta Q\ (d_s)$ also exhibits uncertainties with a coefficient of variation: 20 %.

5.4. Total costs per stages
According to equation (2) total costs $r(t)$ per stages can be written

$$r(t) = D_t \left\{ \text{pumping} \left[Q(t)\right] + \text{losses} \left[\Delta Q_s\ (t)\right] + \text{cut cost} \left[d_c(t)\right] + \text{drilling cost} \quad d_d\ t \quad + \text{sealing cost} \left[d_s\ (t)\right] \right\}$$

Costs will be given in 10^6 Forints /1 Ft equals about 5 US cents/.

a./ Pumping costs = $a(t)\ .\ Q\ (t)$

If $Q(t)$ is in m^3/min:

	t = 1	2	3	4	5
a (t)	1,0	2,3	2,7	2,3	1,7

b./ Cut costs = $10\ d_c(t)$

c./ Drilling costs = $0,9\ d_d(t)$

d./ Loss function due to mine water can be estimated from experiences in existing mines in some discrete points at most. Loss functions are different in different stages. The difference is caused by changing mining conditions and technology as well as mineral production rate. Naturally, lower losses occur at the initial and last stages, t = 1 and 5 than in time of full operation, t = 3 and 4 . As an example Fig. 3 shows estimated loss function values for stage 1 and 4.

Given the above numerical data and functions for the DP model, the solution is straightforward on the basis of the recursive equation (6). Because of limited space numerical results and sensitivity analysis will be discussed in a separate paper.

6. CONCLUSIONS AND RECOMMENDATIONS
Main conclusions reached so far and recommendation for further research can be summarized as follows:

a./ Systems approach means an effective tool to design and operate mine water control systems.

b./ For the satisfaction of various objectives of mine water control a complex system is warranted.

c./ DP is an efficient modelling tool for the sequential economic

design of mine water control systems, with due allowance to environmental constraints.

d./ Under certain hydraulic and/or geologic conditions, subsequent inrushes per stage, $\Delta Q(t)$ are not independent; in fact the flow of earlier inrushes may decrease as more and mor new inrushes occur [SCHMIEDER et al., 1975]. Present DP model assumes no hydraulic interrelationship, thus needs further development in the case this assumption does not hold.

e./ In the case of several mines in a region, a joint system of mine water control seems necessary since there are regional objectives of economics and the environment DP model similar to that introduced in this paper is capable of the design of such joint system [CASTI, 1977].

f./ Though several models of mine water control are available, there is a need to develop a general systems model which is connected both to the mining system and the environment.

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MATHEMATICAL SIMULATION OF HYDRAULIC SCREEN TO DRAIN MINE AND CONSTRUCTION WORK

Loupanec, M.

ABSTRACT : The introduction of the paper specifies aspects of the hydraulic screen optimization as a drainage system for mine or construction work. The project of the screen respecting all the optimization aspects mentioned is solved by means of mathematical simulation. Stochastic models starting from methods of mathematical statistics and probability give data for the final deterministic model described by partial differential equations for physically defined relation water-rock. The conclusion of the paper contains requirements for a hardware and software of the computerized system.

RESUME : Au commencement de l'article, on spécifie les aspects du problème pour optimiser une barrière hydraulique destinée à drainer des constructions ou des ouvrages miniers. Le plan d'une barrière qui prend en considération tous les aspects en question est résolu à l'aide d'un modèle mathématique. Le modèle stochastique, dérivé par les méthodes des statistiques et du calcul de probabilité, fournit les informations nécessaires pour créer le modèle final déterministe des équations aux dérivées partielles qui décrivent le système physique eau-roche. Pour terminer, on présente les exigences de hardware et software d'un ordinateur convenable.

RESUMEN : Al comienzo del artículo se analizan los aspectos de optimización de una pantalla hidráulica, como sistema de drenaje de minas o construcciones en el subsuelo. Tenidos en cuenta todos esos aspectos, se ha resuelto un proyecto de pantalla, con ayuda de un modelo matemático. El modelo estocástico, confeccionado a partir de métodos estadísticos y probabilísticos, suministra los datos necesarios para establecer el modelo determinístico final, basado en las ecuaciones de derivadas parciales que definen las relaciones físicas roca-agua. Finalmente se señalan las exigencias del hardware y software del sistema de computación.

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INTRODUCTION

The drainage system is a very important part of a mine or construction being founded under the level of underground water. It is possible to say that in many cases the possibility of effective drainage is a limiting factor in opening a mine or realization of the construction. In the majority of cases the drainage system directly influences the way of opening or situation and technology of the construction.

The project of a concrete drainage system must start from given geological, hydrogeological and hydrological conditions, which must be respected within the whole complex together with the requirements of mining or constructional activity. This determines a wide scale of methods of isolation of mine or construction work from underground water beginning from tamponing of cracks, through injecting the failures, clay membranes, concrete walls, drainage galleries and shafts, hydraulic screens and barriers, etc., up to transferring a river bed. It is desirable to obtain optimum solution of the project and operation of a drainage system concerning, as far as possible, all the mentioned criteria. In many cases an optimum effect of drainage is obtained by the combination of several basic methods.

The paper indicates the optimization of a very frequently applied basic method of mine or construction work drainage.

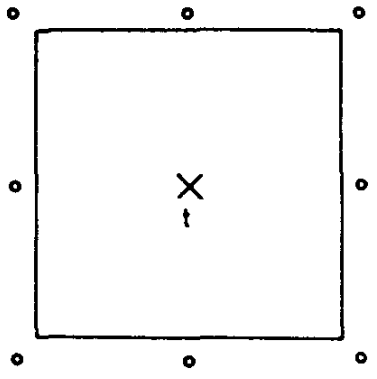
1. APPLICATION OF HYDRAULIC SCREEN FOR DRAINAGE

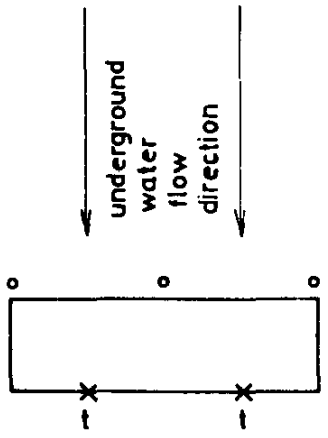
Hydraulic screen can be used for drainage practically under all the hydrogeological conditions: in the case of free and strained level of underground water, porous and joint permeability (in the case that the system of joints is well interconnected).

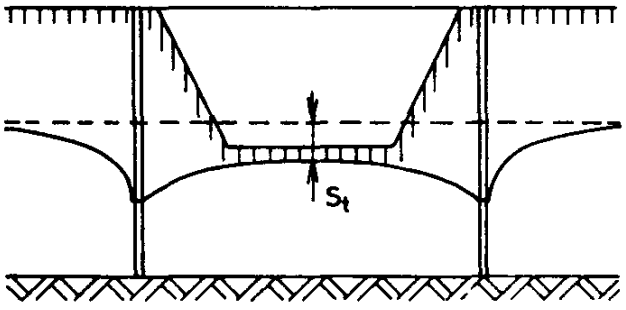
Generally, the hydraulic screen presents a closed system of wells (Fig. 1) situated at the nearest possible distance from the edge or from the centre of the plan area of the drained space (further only the drained area). For the first examination of the reality of the hydraulic screen project it is possible to use the well known method of the "large diameter well" to calculate steady flow with filtering parameters being usually at the disposal from the preceding hydrogeological investigation. The equation

$$Q_c = \frac{2\pi ks \cdot m}{\ln \frac{R}{r}} \qquad (1)$$

holds true for the mean of weighed coefficient of filtration (k),







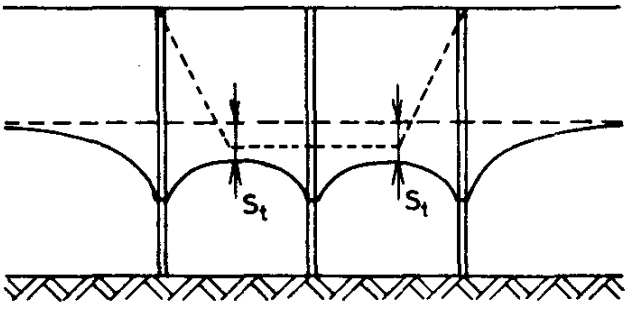


Fig. 1

Fig. 2

required reduction within the drained area (s), for the thickness of water-bearing stratum (m), level (H) of the free level is m = 2H-s, further for the "large diameter of the well", generally for the mean or weighed distance of the drained area center of gravity from its boundary (r) and finally for the range of the depression due to the "large diameter well" (R) drainage effect. The range of the depression is determined for orientation, e. g. from empiric relations after Kusakin

$$R = 3000\ s\ \sqrt{K} \qquad (2)$$

or after Sichardt

$$R = 575\ s\ \sqrt{H \cdot k} \qquad (3)$$

Quite often it is possible to meet the non-closed hydraulic screen (fig. 2), which is suitable from the point of view of considerable savings when establishing the screen with insignificantly increased operation costs. It can be used first of all to drain areas with considerably predominating dimension in one direction. The non-closed hydraulic screen, situated along the longer side of the area being drained, usually ensures the required reduction also in the direction of the shorter dimension. In the contrary case, it is sometimes possible to close the non-closed screen at its edges in non-permeable lines (Q = 0). The non-closed screen can be used even in the case that the non-permeable line is situated at the longer side of the area being drained. In the case of small inflow of underground water into the area being drained, it is more convenient to prefer the non-closedscreen rather than the closed one even at the very cost of adequate extension of the non-closed screen behind the borders of the area being drained. It is evident that in all the mentioned cases the non-closed screen is situated at the side of predominating inflow of underground water. If this inflow is directed against the mining direction of the mine, the screen can be repeatedly situated in a sufficient forefield before the mining front. When mining reaches the screen, the screen is liquidated. Therefore the wells in such screens must not be equipped with steel but with such a material which can be easily extracted (ceramic casing, concrete centering, etc.).

In well permeable water-bearing strata, where large capacities are pumped with relatively flat depressions, the necessary reduction would call for non-effectively large number of wells in a hydraulic screen. This apprehension is real especially with closed hydraulic screens used to drain large areas. To obtain the required reduction even in the center part of this area, the screen is completed with a system of shallow wells, drilled and later pumped from the drainage level of the mine or from the bottom of the construction work. It is also possible to use the system of horizontal wells, eventually drainage seam or pits of small profile. The drainage seam or pit is advantageous as a supplementary drainage system even for the non-closed hydraulic screen.

A combination of a hydraulic screen with a supplementary method of drainage usually decreases the range of the depression due to the drainage of the mine or construction work. This range is especially large with small dynamic reserves of the area. If there are significant holding areas within a possible range of depression, or even a potential source of underground water pollution, it is convenient to combine hydraulic screens with hydraulic barriers. They are created by infusion wells into which water pumped from hydraulic screen is driven in. Thus the depression due to drainage of the mine or construction work is decreased. This protection can be used in many cases at the very cost of slightly increased capacities in the hydraulic screen wells.

Similarly it is possible to combine a hydraulic screen with all the other methods of passive and active isolation of a mine or construction work from underground water.

In the conclusion of the chapter on the suitability of application of the hydraulic screen to drain mine and construction work, it is necessary to carry out an economical consideration of the screen project. A hydraulic screen, as compared with other methods of isolation of mine and construction work from underground water, under similar conditions, presents lower investment costs, but higher operation costs caused by continuous pumping of hydraulic screen. In the majority of cases the operation costs can be compensated so that water being pumped from a hydraulic screen, usually not yet polluted due to exploitation, etc., is used to improve to river flow, to supply inhabitants and industry with water, etc.

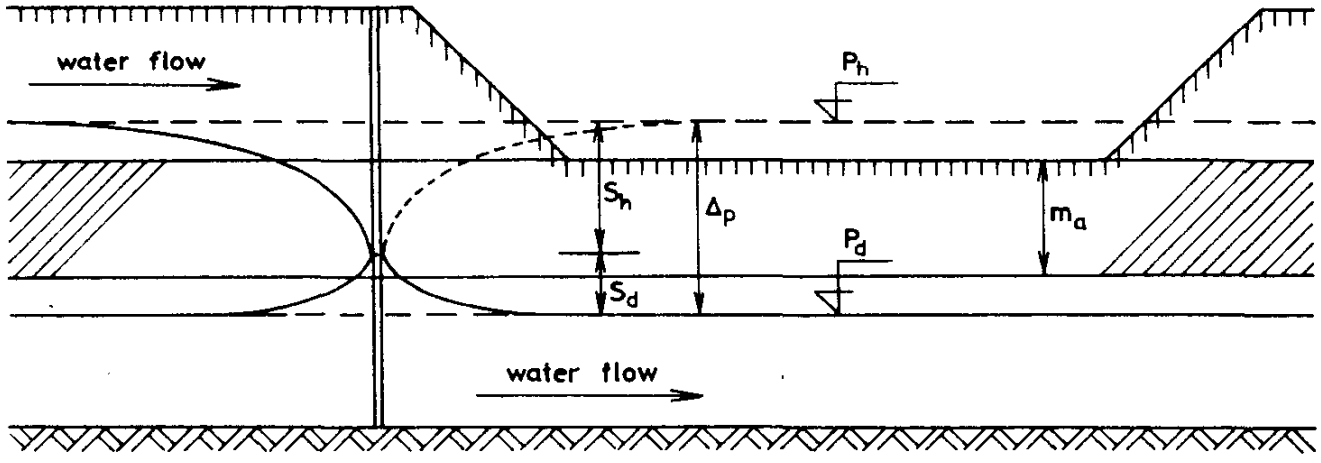


Fig. 3

Under some hydrogeological conditions there is an endeavour to use infiltration hydraulic screen (Fig. 3) for water being polluted by mining activities or else, which cannot be used for the above mentioned purposes. The principle of the infiltration hydraulic screen is in that the water from the strata disturbed by mining or construction activities is transferred due to natural draining ability of the wells into isolated lower strata, not disturbed by mining or construction activities. The condition is that the transmissibility

$$T = k \cdot m \qquad (4)$$

given by the coefficient of filtration (k) and the flow profile (m) is larger in lower strata where water is transferred to than in the upper strata being drained. Another condition is that the difference of piezometric niveau

$$\Delta p = p_h - p_d$$

of the upper (p_h) and lower (p_d) series of strata is larger than the sum of the reductions due to drainage of the upper strata (s_h) and the increase due to the resistance during infiltration of water from hydraulic screen wells into the lower strata (s_d). The following holds true

$$s_h + s_d \leq \Delta p.$$

Among others it is necessary to meet the condition that the weight of impermeable series of strata isolating the lower permeable strata from the bottom of the mine or construction ditch is larger than the upward hydrostatic pressure in the lower water-bearing strata; otherwise a fracture of isolating impermeable strata (Artesian roof) into the mine or construction work could take place. In the case of weighed specific weight (f_a) of the Artesian roof rocks of given thickness (m_a) there holds true that

$$p_d < m_a \quad f_a.$$

In the case that polluted or otherwise unusable water cannot be drained from the space of the mine or construction work by means of an infiltration hydraulic screen or similar drainage system with spontaneous drain (pit, tunnel, etc.) it is necessary to carry out a careful calculation, whether the costs for pumped hydraulic screen, i. e. investment costs and in the case of long-time drainage also operation costs, are proportional to the resulting effect of mining or constructional activity.

2. MAIN ASPECTS OF OPTIMIZATION OF THE PROJECT AND OPERATION OF HYDRAULIC SCREEN

Some aspects of a conceptional character concerning rational project and operation of a hydraulic screen have been mentioned in the foregoing chapter. All of them are directed towards one object: a necessary reduction of underground water level in the space of the mine or construction work at minimum costs with minimum affection of existing underground water regime in wider surroundings.

If this is projected onto the detail solution of a hydraulic screen, it is possible to lay out the following optimization aspects:

1. Optimum situation of the screen in predominating underground water flow direction, with regard to hydrogeological situation, respecting the boundary conditions in horizontal and vertical directions to enable to obtain maximum drainage of the space of the mine or construction work.

2. Determination of the optimum natural underground water level for which the hydraulic screen is to be dimensioned. The task is to propose maximum natural level with regard to the importance of the mine or construction work and scheduled time of drainage. Simultaneously with the course of the maximum level during individual hydrological years and during important periods of the year it is necessary to follow also minimum natural underground water level, to be able to determine the amplitude course.

3. Optimum situation of individual wells of the screen as close as possible to the centre of the drained area, in the places with the most favourable filtering parameters so that in the most unfavourable places (t) of the drained area the required reduction is obtained by superposition of the reductions due to optimum number of wells (p)

$$s_t = \sum_{i=1}^{p} s_{t,i}$$

4. A proposal of the optimum number of wells of convenient diameters to obtain balanced pumping of the screen. In no well of the screen it is permitted to exceed safety reduction which still does not permit origination of negative hydraulic effects on the skin of the well (skin-effects, obtaining critical intake rate when suffosion begins to appear) even in the case of maximum pumping of the screen to ensure required reduction with maximum proposed natural underground water level. This is closely connected with the proposal of suitable manipulation order for the operation of hydraulic screen.

5. Drainage of the mine or construction work presents mostly a large interference with existing regime of underground water in a wide surrounding. There is a question how to hold the consequences of this interference within optimum limits: to obtain a certain balance between positive and negative consequences. The starting point in the evaluation of the consequences of drainage is spreading of the depression in the space and time.

What must be taken into consideration is ageing of the screen wells, spreading of the depression to the places of expressively different hydrogeological medium as compared to that in which the mine or construction work is proposed (a change of filtering parameters, boundary conditions). One part of this prognosis is also a protection of underground water from pollution. It is necessary to consider all the water sources, potential sources of pollution, their mutual position with the possibility of displacement with regard to the direction; and to the rate of flow of underground water.

3. MATHEMATICAL SIMULATION OF THE OPTIMUM PROPOSAL OF THE PROJECT AND OPERATION OF THE HYDRAULIC SCREEN

Respecting the aspects of optimization stated in the foregoing chapter presents a complex solution of considerably wide problem which mostly exceeds the limits of the mine and construction work. It is evident that optimization cannot be solved empirically. One of the ways how to solve this problem is a mathematical simulation using the digital computers. It is very quick, mobile, when solving necessary variants of the model and considerably less expensive when we compare it to other comparable methods.

In the mathematical simulation of the hydraulic screen it is possible to speak of a stochastic simulation which is followed by deterministic simulation.

3.1 STOCHASTIC SIMULATION

Stochastic simulation solves the existing regime of underground water not affected by drainage of the mine or construction work good knowledge of which is necessary to propose the screen (Loupaneo, 1969). The initial point is the time series of hydrological observations, being most of all purposeful, limited by the time interval usually of several years. An analysis of these empirical series presents a static model. From the point of view of the theory of random events, the series of hydrological observations presents a random process taking place under unchanged physical conditions within the arbitrary time. A generation of hydrological series and prognoses of hydrological characteristics within the arbitrary time presents a dynamic model. Stochastic simulation makes use of the mathematical statistics and a theory of probability. Its simulation contains always a random element (Balek, 1976; Ibbitt, 1974).

Stochastic simulation takes place in several phases on selected probability levels (e. g. $\alpha = 0.01$, i. e. 99% probability safety), on which the zero hypothesis is accepted or rejected.

3.1.1 PREPARATORY PHASE
As the statistic and probability methods call for the assumption of normality, all the hydrological series must be subjected to the coincidence test, whether they have normal or at least approximately normal distribution. Among parametric coincidence tests we can mention e. g. χ^2-test, among the non-parametric ones e. g. Kolmogorov-Smirnov test for one selection.

Extreme values of observation in empiric series are often loaded with enormous random error. An elimination of such an observation is carried out by the test of an extreme deviation. Among the parametric tests we can name e. g. Grubbs's test, among the non-parametric ones e. g. the Dixon's test.

A systematical error which usually affects all the values of observations in a hydrological series is more difficult to be identified. It is possible to use the less known method test of randomity, the test for the number of iterations (i). With the number of observations n>40 the division of the groups of observations of the same selected type (iterations) approaches the normal one with the mean μ_i and decisive deviation σ_i . The criterion of the test is the standard value

$$u_i = \frac{i - \mu_i}{\sigma_i} \qquad (5)$$

An unsatisfactory test of randomity with generated series indicates an unsuitable mechanism of generation of the time series.

Criteria of all the mentioned tests are compared at the selected level of importance with their critical values, tabulated in every statistics textbook (e. g. Nosek, 1972; Raisenauer, 1970).

3.1.2 DETERMINATION OF STATISTIC PARAMETERS OF THE SERIES
This phase is used to ascertain, except for the parameters currently used in parametric tests (extreme amplitude, mean, decisive deviation, median) also chronological trends of observations, empiric frequency and distribution line or even the line of repetition and also the determination of quantities, variation range, coefficient of variation (C_v) and a coefficient of assymetry (C_s).

Some statistical characteristics are used when preparing the balance equations, first of all when specifying their terms in the arbitrary convenient time period. The left side of this equation separates the underground drain determining dynamic reserves of the area where the mine or construction work is situated. This dotation must be taken into calculation when draining (Dub, 1969).

3.1.3 SERIES TRANSFORMATION

Hydrological series are often discrete (rain, irregular water consumption). For the analysis with continuous series it is recommended to carry out a convenient transformation of discrete series. The transformation of the series can suitably emphasize its dynamics. When simulating infiltration of the rain water to the underground water level, it is necessary to cumulate the rain water (a transformation of the original series by gliding totals) within a certain period to enable the process of infiltration. The series are frequently smoothed due to transformation (e. g. gliding totals) or some observations are emphasized (e. g. weighed mean).

Transformation mechanisms may be very complicated. If several series take part in the transformation process (first of all at the input), we speak about superposition. Both procedures are often taken in the process of series generation (Grushevskij, 1973).

3.1.4 CORRELATION AND REGRESSION ANALYSIS OF THE SERIES

While correlation determines the degree of dependence of variables (e. g. by correlation coefficient R), regression describes this dependence by the function of an approximation.

A practical application distinguishes twice and more multiple correlation and regression analysis.

The T-test with twice-multiple analysis is satisfactory to evaluate the coefficient of correlation (R_{yx}) between one dependently (y) and one independently (x) variable.

$$t = \frac{R_{yx}}{\sqrt{1 - R^2_{yx}}} \cdot \sqrt{\nu} \qquad (6)$$

where $\nu = n-2$ is the number of degrees of freedom for n observations. A regression relation between both variables can be ge-

nerally written in the term of the sum of arbitrary known functions $\varphi_j(x)$ and unknown parameters b_j.

$$y = \sum_{j=0}^{p} b_j \varphi_j(x) \qquad (7)$$

The coefficient of correlation ($R_{yx_{1,2....p}}$) between dependently variable (y) and independently variables ($x_1, x_2, \ldots x_p$) is evaluated by means of the F-test.

$$F = \frac{R^2_{yx_{1,2...p}}}{1-R^2_{yx_{1,2...p}}} \cdot \frac{\nu_2}{\nu_1} \qquad (8)$$

where $\nu_1 = p$ and $\nu_2 = n-p-1$ are the degrees of freedom for n observations. A regression relation between these variables can be generally written in the term of the sum of arbitrary known functions of independently variables $\varphi_j(x_1, x_2 \ldots x_p)$ and unknown parameters b_j.

$$y = \sum_{j=0}^{p} b_j \varphi_j(x_1, x_2 \ldots x_p) \qquad (9)$$

If the variable on the left side of the equation (9) is substituted by the group of variables $y_1, y_2 \ldots y_l$, the multivariation analysis also called the canonical analysis is obtained. For the evaluation of the coefficient of the canonic correlation $R_{y_{1,2...l}\, x_{1,2...p}}$, F-test of the following form is used:

$$F = \frac{R^2_{y_{1,2...l}\, x_{1,2...p}}}{1-R^2_{y_{1,2...l}\, x_{1,2...p}}} \cdot \frac{\nu_2}{\nu_1} \qquad (10)$$

where $\nu_1 = p$ and $\nu_2 = n-p-l$ are the degrees of freedom for n observations under the condition that $l \leq p$. The general canonic regression relation can be written in the form similar to equation (9)

$$\sum_{k=0}^{l} c_k \cdot \varphi_k (y_1, y_2 \cdots y_1) = \sum_{j=0}^{p} b_j \cdot \varphi_j (x_1, x_2 \cdots x_p) \quad (11)$$

The results of the dispersion analysis for the evaluation of the coefficient of correlation are evaluated at a selected level of importance and for given degrees of freedom with critical values of Student's division (T-test) or Fischer-Snedecor division (F-test).

The correlation and regression analyses are widely applied during simulation of the drainage system of a mine or construction work. Individual types mentioned above of the analyses are used according to the number of variables (time series of observed hydrological events), which are at the disposal.

The correlation analysis is used to ascertain the genesis of underground water in the given area, its origin, e. g. rain water infiltration or surface water or underground in-flow, the position and direction of which is defined. Similarly the correlation is used to specify the border conditions in a horizontal direction: determination of impermeable line (Q=0), constant level (H=const.) and an unlimited water-bearing stratum. The border conditions in a vertical direction can be determined by the correlation of levels of underground water in observed wells made in various water-bearing strata.

The regression analysis can follow the correlation analysis in those cases where the correlations indicate a dependence between hydrological effects at selected level of importance. The regression is used to ascertain the proportion of individual factors (climatic, hydrologic, hydrogeologic) to the genesis of underground water in the given area. Quantification of underground water infiltration with regard to the border conditions in horizontal and vertical directions is carried out similarly. On the ground of proved important dependence of short-time observed level of underground water on long-time observed affecting factors the time series of variation of underground water level is generated, eventually a prognosis of long-time characteristics of this variation is carried out. A natural underground water level in the place of pumping is areally interpolated by the regression by means of not-affected wells in wider surroundings of the mine or construction work.

In the correlation analysis it is possible to use with an advantage the method of the so called serial correlations which is essentially the principle of autocorrelation applied to more variables.

The time lag between mutually affecting hydrological effects (e. g. a retardation which shows itself in the proportion of rain water on the dotation of underground water) is determined from the phase shift (k) for an infiltration cycle

$$y_i = f\,(x_{i+k})$$

where the correlation coefficient is the largest. In this way it is easy to determine the rate of natural infiltration of underground water in horizontal and vertical directions (Chow, 1964; Draper, 1966; Malevanyj, 1972; Matalas, 1964; Rektorys et al., 1963).

3.1.5 APPROXIMATION OF EMPIRIC DISTRIBUTION LINE

Chapter 2 emphasizes the importance of maximum natural level of underground water for the optimum proposal of the project and operation of the hydraulic screen. If long-time observed significantly affecting factors of underground water are not at the disposal for the prognosis of this level for the whole time of operation of the screen, it is not possible to use the prognosis by means of regression as stated in par. 3.1.4. It is, however, possible to use an approximation of empiric distribution line of short-time observation of underground water by suitable division (e. g. log-normal, Gumbel's, binomic, triparametric Γ). The long-time characteristics of underground water level is extrapolated by means of the mass curve with suitable division. The period of short-time pumping tests to ascertain the filtering parameters is classified also on the ground of this curve. Until now the best satisfaction in the case of underground water is given the mass curve approximation.

$$y = \int_{x_o}^{+\infty} \varphi(x)\,dx \qquad (12)$$

Pearson's classification of the 3rd type

$$\varphi(x) = \frac{(x-x_o)^{\frac{a}{d}}\, e^{-\frac{x-x_o}{d}}}{\left(\frac{a}{d}+1\right)\Gamma\left(\frac{a}{d}+1\right)} \qquad (13)$$

for optimum parameters a, d (Aleksejev, 1960; Hátle, 1972).

With regard to the extent of the paper it is not possible to specify some other methods used in stochastic simulation, such as harmonic analysis, trend analysis, discrimination and vice versa cluster analysis, factor analysis and some methods of the theory of games. They are, anyway, used in hydrology up to now very seldom. Their wider application is restricted first of all by relatively short empiric series of observations and insufficient number of observed variables (Satran, 1973; Vilkas, 1973).

3.2 DETERMINISTIC SIMULATION

A deterministic model in hydrology starts from physically defined relations water - rock described by partial differential equations. These, in general term, include anisotrophy, space assymetry of flow, infiltration from the surface or from the neighbouring water-bearing strata and other influences including incomplete wells, dip and non-linear course of the strata and bedrock. The solution of these differential equations (by the analytic-conform method, the method of nets or the method of final elements) is laborious, but with regard to more and more effective computers, it is feasible. Another situation is in collection of input data (parameters) for these generally formulated differential equations. The deterministic model takes the data, except for the stochastic model, first of all from hydrologic investigation and pumping tests. Current means of investigation, technology of drilling and pumping, are not at the level which would be required by a generally formulated mathematical apparatus (Hálek, 1973).

These reasons represent the usual limits of the deterministic model, which is simplified to be adequate to the quality of input parameters. These are usually made gradually more precise by verification of the mathematical model so that it gives the best corresponding to the given natural conditions. Such a method of simulation corresponds well also to the optimization of the hydraulic screen used to drain the mine or construction work.

For illustration let us state an example of such a simplified simulation on the ground of linearized differential Businesq equation of axially symmetrical non-stabilized flow of underground water towards a complete well

$$a\left(\frac{\partial^2 U}{\partial r^2} + \frac{1}{r}\,\frac{\partial U}{\partial r}\right) = \frac{\partial U}{\partial t} \qquad (14)$$

for the function of potential (U), coefficient of pressure conductivity (a), coefficient of filtration (k) and the distance of given point from the centre of the well (r). The condition of the

described flow is isotropic, horizontal water-bearing stratum. The solution of the equation (14) by e. g. analytic-conform method leads, under certain assumptions, to the equation simulating an independently pumped well in an unlimited water-bearing stratum with constant consumption (Q) in the time (t).

$$U = \frac{Q}{4\pi k} \quad W(x) \tag{15}$$

for

$$x = \frac{r^2}{4at} \tag{16}$$

where W(x) is the integral-exponential function $-E_i(-x)$.

Using the principle of superposition of partial effects from the individual wells, it is possible to simulate

a) cooperation of individual wells in the screen
b) respecting the line border conditions by means of the method of a conform transformation (reflection of wells)
c) a volume of the consumption of underground water from the individual wells of the screen in the given time at prescribed reductions in the wells and that even in the case that pumping does not start in the whole screen simultaneously, but in the arbitrary wells in the arbitrary time.
d) a reduction of the underground water level in an arbitrary point of the mine or construction work and its surroundings with an arbitrary consumption (constant in time) from the individual wells in the screen and that even in the case of interrupted pumping in an arbitrary well.

These possibilities of deterministic simulation enable, when respecting the proposed natural underground water level and on the other hand a safety reduction, to maintain fully the aspects of the optimization of the project and operation of hydraulic screen (Bánský, 1976; Kazda, 1976; Loupanec, 1978).

3.3 PROPOSAL OF NATURAL UNDERGROUND WATER LEVEL FOR DIMENSIONING THE SCREEN

Par. 3.1 determines, by means of the stochastic simulation, the maximum and minimum natural underground water level and thus also the amplitude of its variation. Its long-time prognosis is known, as well as the course during individual hydrological years and also characteristic periods (wet, dry).

If the amplitude of the underground water level variation is small, the screen is dimensed for long-time maximum level which can be proportionally decreased if it is possible to flood a part of the mine or construction work.

On the contrary, if the amplitude of underground water level variations is high, heavy-duty pumps being dimensed for maximum natural level would pump only small quantities during the time when the level is low and therefore they would work non-effectively. The optimum solution of the screen operation in this situation calls for the consideration of three variants:

1. If the conditions allow, first of all the number of the screen wells and their mutual position, the safety reduction and also the range of the depression, the screen is dimensed for natural level, reduced as compared to the maximum level by the same amount which it is possible to reduce the level under the bottom of the mine or construction work by. Using this variant, the amount pumped from the screen is approximately constant even during the period of low natural level, when the level is reduced under the level required by mining or constructional activity. An eventual increase of natural level above the proposed one is then safely retained in the retention space under the mine or construction work. This variant of the optimization of the screen operation is advantageous in the case of frequent, intensive variation of natural level of underground water. It is possible to use a stochastic prognosis of this variation and to control pumping in the screen in advance so that the retention space under the mine or construction work is maintained in the required size only during the necessary time.

2. If the dislocation of wells allows, it is possible to interrupt pumping in some wells during the period of low natural level. The remaining wells may be pumped at constant capacity. This variant of the optimization of the screen operation assumes lower permeability of the drained horizon, in which depressions with higher curvature are realized.

3. If the conditions do not allow to apply the first or the second variant, it is necessary to install except for heavy-duty pumps for pumping during the periods with high natural levels, also the small-capacity pumps for low levels. With the low suction heights of the used pumps and considerable reductions, it will be in most cases an installation of the pumps in the wells which must be proposed as wide-profile wells.

3.4 SAFETY REDUCTION OF THE LEVEL IN PUMPED SCREEN WELLS

During no variant of the optimum operation stated in par. 3.3 the reduction in the screen wells may reach critical reduction during which negative hydraulic effects begin to appear on the skin of the well. A real intake rate is evaluated first of all

$$v_s = \frac{Q}{2\pi r \cdot h \cdot p_f \cdot a} \qquad (17)$$

for capacity (Q), well radius (r), effective length of a filter (h), relative number (p_f) and safety coefficient (a) which depends first of all on the age of the well. The relative number describes the flow profile at a correct selection of perforation and strew-round, this number will be identical with the effective porosity of the water-bearing stratum (μ). In this case the skin effect will be the smallest. The real intake rate must not exceed the critical rate during which suffosion takes place.

4. A HARDWARE AND SOFTWARE FOR MATHEMATICAL SIMULATION OF THE HYDRAULIC SCREEN

The mathematical simulation of the optimum project and operation of the hydraulic screen calls not only for a powerfull computer, but also for an effectively arranged programme system.

The hardware input must at least digitize the analog registered data on hydrological observations and pumping tests. More advantageous is direct storing of these automatically scanned informations in the input medium of the computer, either decentralized or remote transfer into registering centre. In the case of the 3rd generation computer we can assume a satisfactory operation speed, a capacity of the internal memory of approx. 500 KB, a mobile external memory, a strong operation system allowing the multiprogramming or even the time sharing. The real time data processing can be used only in the case of peak solution of drainage optimization, when pumping in the hydraulic screen is controlled automatically by the computer through servomotors of the pump valves. In any case it is necessary to provide the computer with a graphical output, the best one would be a plotter or a digigraph in off-line regime.

A part of the software should be a data base of the time series, which must enable storage of practically arbitrary number of series, their precise identification and a direct access during the manipulation, actualization and effective checking of the series. The system of programmes for the actual mathematical simulation must solve all the tasks described in the chapter No 3. It will take still a certain time untill this system will operate automatically with minimum intervention of an operator. Today we find sufficient when the individual phases of the model are processed by independent subsystems (main programme + sub-routines, common for the whole system). A unit-built system is advantageous during the actualization, expansion or mutual interconnection of the subsystems. It is desirable that the programme systems enable, except for the tabular output, also a graphical output - drawing schemes, maps or block schemes (Loupanec, 1978).

CONCLUSION

In the conclusion of the paper on mathematical simulation of the optimum project and operation of the hydraulic screen for drainage of mine or construction work it is necessary to point out the necessity of the optimization. The optimization can save considerably high investment costs and first of all the long-time operation costs. The problem of no less importance is also a preventive solution of underground water protection with regard to the mine or construction work. The mathematical simulation using the computers presents an effective means of this optimization. On the other hand it is necessary to realize that the process of optimization starts from the natural conditions, the intricacy and variability of which is sometimes very difficult to be schematized mathematically at the required level. It will be therefore necessary to correct, in some cases, the mathematical model empirically. The endeavour is, with the development of hydrogeological investigation, the technology of drilling and pumping and with future development of measuring techniques, computer hardware and software, together with gradual precisioning of the mathematical simulation, to reduce those empiric interventions and on the contrary to increase the number of drainage systems controlled by the computer in the real time.

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FREE SURFACE FLOW IN POROUS MEDIA BY FINITE ELEMENT METHODS

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ABSTRACT : The steady state flow through porous rigid media is analysed. Governing equations and boundary conditions are set up and discussed. Variational methods and finite element approach are described and the solving system of equations for quadrilateral networks of triangular linear condensed elements is obtained. Non-homogeneity and anisotropy are discussed. Further discussion on the finite element techniques and on the convergence of the numerical process. Some sugestions for improvement and quick determination of the exit point as a preliminary step to obtain the free surface are made, in order to reduce the total number of iterations. After the presentation of some examples, a rigorous formulation of the problem for heterogeneous media is put forward and since it cannot yet be practicized, the fundamental points for an approximate solution with a small number of iterations is reviewed.

RESUME : On analyse l'écoulement à travers des milieux poreux rigides. On établit et discute les équations qui gouvernent le phénomène et les conditions sur les frontières. On présente la méthode variationnelle avec approximation par des éléments finis et on obtient le système d'équations qui résoud le problème pour des réseaux quadrangulaires d'éléments triangulaires linéaires, condensés. On discute la non-homogénéité et l'anisotropie, les techniques des éléments finis et la convergence du processus numérique. Etant donnée l'importance de la position du point de sortie pour la fixation de la surface libre de l'écoulement, une suggestion est faite pour la détermination de ce point. Après la présentation de quelques exemples, on donne une rigoureuse formulation du problème pour les milieux hétérogènes et lorsqu'il n'est pas possible de la pratiquer, on avance les points à retenir pour une solution d'approche, avec un petit nombre d'itérations.

RESUMEN : Se analiza el flujo permanente, en medios porosos rígidos. Se establecen y discuten las ecuaciones básicas y las condiciones en los límites. Se describen los métodos variacionales y las aproximaciones por elementos finitos ; se obtiene la solución a los sistemas de ecuaciones para mallas cuadradas de elementos triangulares lineales condensados. Se discuten los problemas de la no-homogeneidad y anisotropía, de las técnicas de elementos finitosyde la conver-

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gencia del proceso numérico. Se hacen algunas sugerencias para mejorar la determinación rápida de la posición del punto de salida, como un paso para obtener la superficie piezométrica, y poder reducir el número total de iteraciones. Por último, tras mostrar algunos ejemplos, se presenta una formulación rigurosa para los medios heterogeneos, y, aunque no ha podido todavía ser practicada, se adelantan los puntos fundamentales para una solución aproximada con un número reducido de iteraciones.

1 - INTRODUCTION

The water percolation through porous media is governed by the well knwon Darcy's law. This experimental law states the proportionality between the specific flux vector $\underline{q}$ and the hydraulic gradiente $\underline{i}$.

$$\underline{q} = K\,\underline{i} = -K\,\mathrm{grad}\,\phi \tag{1}$$

The specific flux vector $\underline{q}$ is the flow rate per unit total surface through which flow takes place. Its components in cartesian x,y,z coordinates, are q_x, q_y, q_z respectively. The hydraulic gradient $\underline{i}$, is the piezometric head loss or energy loss due to friction per unit of length of percolation. Its components are $i_x = -\partial\phi/\partial x$, $i_y = -\partial\phi/\partial y$, $i_z = -\partial\phi/\partial z$.

When the medium is homogeneous and isotropic, in relation to percolation, K is a scalar constant, the permeability, and we may write (1) as:

$$q_x = K\,i_x = -K\frac{\partial\phi}{\partial x}\;;\quad q_y = K\,i_y = -K\frac{\partial\phi}{\partial y}\;;\quad q_z = K\,i_z = -K\frac{\partial\phi}{\partial z} \tag{1'}$$

For the flow in the direction of the unit vector $\hat{s}$ we have

$$q_s = \underline{q}\cdot\hat{s} = -K\,\mathrm{grad}\,\phi\cdot\hat{s} = -K\frac{\partial\phi}{\partial s} \tag{2}$$

For homogeneous but, anysotropic media instead of a scalar K there exists a permeability matrix $[K]$ such that

$$\underline{q} = [K]\cdot\underline{i} = -[K]\cdot\mathrm{grad}\,\phi, \quad (3') \qquad K = \begin{bmatrix} K_{xx} & K_{xy} & K_{xz} \\ K_{yx} & K_{yy} & K_{yz} \\ K_{zx} & K_{zy} & K_{zz} \end{bmatrix} \tag{3}$$

K is symmetric.

As a consequence of anysotropy the vectors $\underline{q}$ and $\underline{i}$ are noncolinear except in certain directions of space x_1, y_1, z_1, the principal directions or eigen vectors of the matrix $[K]$. This means that the directions of streamlines do not coincide with those of the normals to equipotentials (ϕ=constant).

Given the K_{ij} of matrix K the principal directions of permeability can be obtained by well know methods (e.g. Bear, 1972). If $\underline{1}, \underline{2}, \underline{3}$ are the principal directions of permeability and K_1, K_2, K_3 the eigen vectors of $[K]$ or principal permeabilities, the components of the specific flux vector q_x, q_y, q_z satisfy the equations

$$q_1 = K_1 i_1 \,, \quad q_2 = K_2 i_2 \,, \quad q_3 = K_3 i_3 \tag{5}$$

If the medium is non-homogeneous the permeability K would be a function of the space coordenates x,y,z. Hence instead of stating the relationship (1) we could state the following

$$\underline{q} = -\,\mathrm{grad}(K\phi) \tag{9}$$

where $K\phi = \phi$ would be the velocity potential.

However is an erroneous form of stating Darcy's law (Bear, 1972), since in that case

$$\underline{q} = -(\mathrm{grad}\,K)\,\phi - \phi\,\mathrm{grad}\,K \tag{9'}$$

Therefore for ϕ =constant we could have flow due solely to variation of permeability which is impossible.

2 - GOVERNING EQUATIONS FOR STEADY FLOW

Mass conservation for a control volume imposes to the steady flow of a incompressible fluid through a rigid porous medium, the following relationship:

$$\mathrm{div}\,\underline{q} + \bar{Q} = 0 \,, \quad \frac{\partial q_x}{\partial x} + \frac{\partial q_y}{\partial y} + \frac{\partial q_z}{\partial z} + \bar{Q} = 0 \tag{10}$$

where $\bar{Q}$ is the externally applied flux, e.i., the volume of fluid externally added per unit of time and per unit volume of global flow space.

Since $\underline{q} = [K].\,\mathrm{grad}\,\phi$

We have

$$\mathrm{div}\left\{[K].\,\mathrm{grad}\,\phi\right\} + \bar{Q} = 0, \quad \frac{\partial}{\partial x_i}\left(K_{ij}\frac{\partial \phi}{\partial x_j}\right) + \bar{Q} = 0 \,, \quad i,j = 1,2,3 \tag{11}$$

If x,y,z are the principal directions of permeability and $x_1 = x$, $x_2 = y$ e $x_3 = z$, (11) becomes

$$\frac{\partial}{\partial x}\left(K_x \frac{\partial \phi}{\partial x}\right) + \frac{\partial}{\partial y}\left(K_y \frac{\partial \phi}{\partial y}\right) + \frac{\partial}{\partial z}\left(K_z \frac{\partial \phi}{\partial z}\right) + \bar{Q} = 0 \tag{12}$$

For the solution of steady state or permanente flow problems we have to add to (11) or (12) boundary conditions.

3 - BOUNDARY CONDITIONS

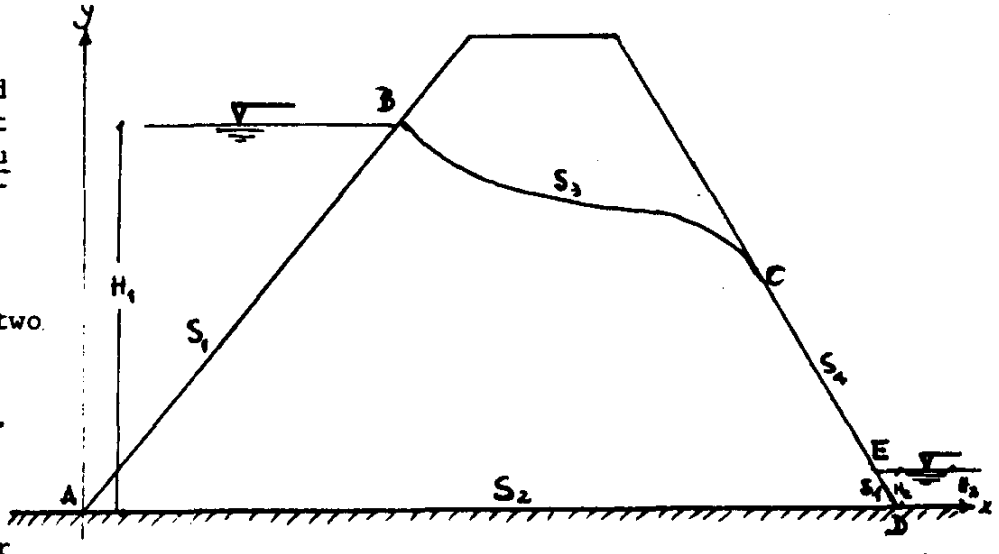
There are three kinds of boundaries: S_1 and S_4 (AB, ED and EC, Fig.1) where the potential is prescribed, ϕ =constant., S_2 (AB) where flux is prescribed and S_3 (BC) where the potential and the flux are prescribed.

A problem where there are conditions of potential type is called Dirichlet

boundary value problem.

Conditions of prescribed flux refer to a diferent type of problem: the Neumann boundary value problem.

The problems as that of Fig. 1 where there are two or three types of boundaries is called mixed boundary value problems.



On surface BC (S_3) both potential and flux are prescribed. On the other hand the boundary itself is "apriori" unknown. The potential on the surface BC, called phreatic surface, must equalize the elevation had, i.e.,

$$\phi(x,y) = y \tag{13}$$

and the normal component of the specific flux vector $\underline{q}$ must be null, i.e.:

$$q_n = \underline{q} \cdot \hat{n} = 0 \tag{14}$$

where $\hat{n}$ is the unit outward normal to S_3.

In isotropic media the condition (14) becomes

$$grad\,\phi \cdot \hat{n} = \frac{\partial \phi}{\partial n} = 0 \tag{15}$$

On impervious boundaries S_2 (AD), the prescribed conditions are also (15) for isotropic media and (14) for anisotropic soils.

The value of potential on S_1 (AB) is $\phi = const = H_1$ and on ED is $\phi = H_2$. On S_4 (seepage face CE), $\phi = y$.

Points such that C, E, D, B and A common to two kinds of boundaries are singular points. For correctness of the solution they must be treated in convenient way (Figs. 2 and 3).

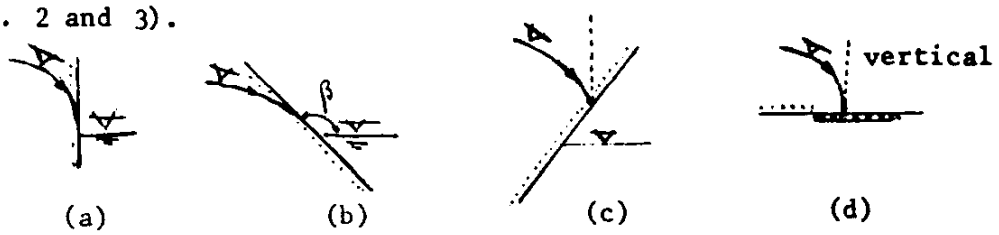


Fig. 2. Intersection of a phreatic surface with seepage face (isotropic media)

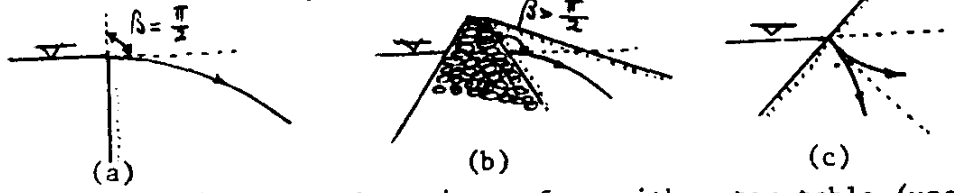


Fig. 3. Intersection of a phreatic surface with water table (upstream)

We notice that the phreatic surface is tangent to the vertical except when the merging angle $\beta > \frac{\pi}{2}$, in which case that surface is tangent to the seepage face (Fig. 2.b). In the case of the intersection with the up steam water table the phreatic surface is tangent to water table except in the case of $\beta < \frac{\pi}{2}$ (Fig. 3.c). For that case the phreatic line is tangent to the normal to the slope and may bend upwards (earth embankment) or downwards (ditch bank).

4 - ORDINARY METHODS OF SOLUTION

Analytical solutions on flow through porous saturated media without phreatic surface has been developed using the potential theory (Muskat, 1937, Polubarinova-Kochina, 1962). This type of approach is indicated for homogeneous aquifers and a wide range of shapes can be solved by maping techniques. However, for complicated geometry and boundary conditions as well as for non-homogeneous media and/or nonlinear flow, analytical methods are unsuitable.

Those difficulties have led to the development of numerical methods that permit the treatment of complex boundary conditions and a first approach to problems of heterogeneous anisotropic media.

Shaw and Southwell (1941) applied the relaxation method to the percolation through porous material and Finnemore and Perry (1968) adapted that technique to the use of computers. Other finite diference solutions have been obtained by Jeppson 1966, 1967, 1968 a, 1968 b, 1968 c, 1968 d, 1969 . Although finite diference techniques allow to deal with complex boundary conditions and phreatic surface problems as well as with anisotropy and heterogeneity the system of linear equations for solution in a computer cannot be easily set up. For these reasons Jeppson as limited his study to homogeneous media and have taken the cartesian coordinates as functions and potential ϕ and stream ψ as independente variables.

More recently finite elements approach has had wide spread application to field problems of all kinds due to easy fitting to boundary of complex geometry and to easy formulation and set up of the system of linear equations to which the problem is reduced.

Hundreds of papers,various treatises (see for ex. Zienkiewicz 1971, Desai 1972) have been writen on finite elements methods most of them relating its application to stress-strain problems in elastic media. Although an easy adaptation of elastic solutions can be done to fluid flow problems, a large number of papers have been writen also on finite elements applied to fluid mechanics. In particular free surface flow through porous saturated bodies (Taylor and Brown (1967); Finn (1967); Volker (1969); Zienkiewicz et al. (1966); Witherspoon et al.(1968); Neuman and Witherspoon (1970); France et al. (1971); Desai (1972 and 1976); Martins and Vargas(1976); Rui Correia (1977), etc.).

In what concerns phreatic surface problems a dificulty arises due to the fact that the numerical process must generate not only the values of the potential

at the network points and rate of flow at points of known potential on the boundaries, but also the coordinates of the phreatic surface, itself. For this reason a trial phreatic surface is fixed "a priori" and changed in each iteration in order to fit both boundary conditions at S_3:

$$\phi(x,y) = y \quad , \quad (13) \quad \text{and} \quad \frac{\partial \phi}{\partial n} = c \qquad (15)$$

Condition (15) is a noflow condition in the direction of the outward normal $\hat{n}$ i.e., the phreatic line is a stream line.

The changing of the trial phreatic surface can be acomplished either by moving the nodal points of the network (Taylor and Brown (1967); Neumann and Witherspoon (1970), R. Correia (1977)) along their columns or maintaining a fix network (Desai, 1976 a, 1976 b).

In the method of movable points there are two cases: the first (Taylor and Brown 1967; Finn (1967)) assumes noflow on the starting phreatic line (condition (15)) and calculate the differences ϕ-y. After each iteration, the upper node in each column is changed in order to meet the condition (13). A new step follows, again assuming noflow at the new phreatic surface F.S..

In the second case of movable nodes (Neuman and Witterspoon (1970); R. Correia (1977)) each iteration is done in two steps: in the first step the condition (13) is imposed at the assumed phreatic surface and, in the second step geometry is not changed but the noflow condition (15) is imposed at the assumed phreatic surface and on the seepage surface (S_4, Fig. 1) the rate of flow calculated in the previous step is imposed at the nodal points. Further, the diferences ϕ-y are evaluated at the nodal points of the phreatic surface and the upper nodes changed in order to meet condition (13). Next a new iteration begins.

For the case of fixed network, Desai (1976), an initial F.S. is assumed, which may include all the domain inside the physical boundaries. A first solution is obtained assuming known potentials at S_1 and S_4 and considering nodes of S_3 as interior nodes which is equivalent to assume S_3 as a stream line.

In a second step with the potentials knwon at nodes, points of ϕ=y are search for, along nodal lines such as AB. Usually, those will be found by interpolation between two consecutive nodes with potentials one less than y and the other greater than y. The curve joining such

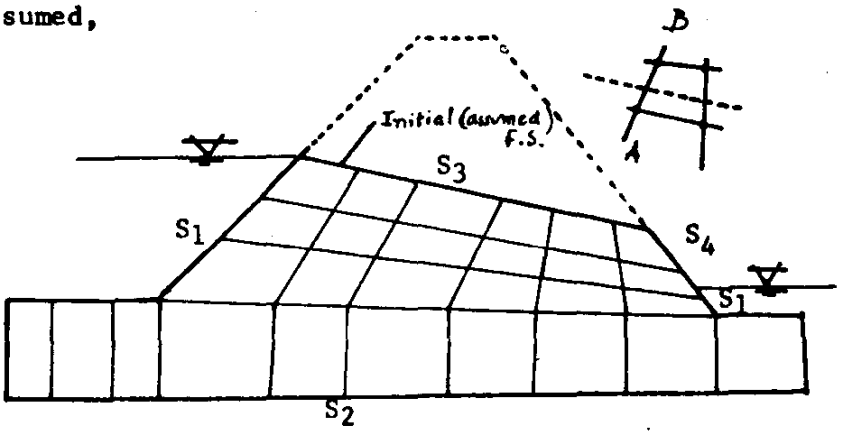


Fig. 3. Desai's fixed network

interpolated points gives the first approximation for the ES. Next, the elements through which the approximate ES. passes are identified and the specific flux q_n normal to ES is calculated by means of the gradient of the known potential ϕ and the permability tensor. From the specific flows the flux is calculated at the neighbour nodes: the residual flow. Injecting at a further step fluxes at the same nodes but with changed sign a new set of potentials is obtained and from them a new position of the ES. The process is continued until ϕ_{i+1} do not differ "signifficantly" from ϕ_i.

5 - DISCUSSION OF THE METHODS OF SOLUTION.

Although free surface flow is a much discussed subject [Colin W. Cryer (Sept. 1977) elaborated a bibliography on that subject with 3.300 references], only recently Cryer and Fetter (June, 1977) nave proved the existence and uniqueness of solutions for the free surface problem. The same authors also proved the convergence of the numerical solutions based on the finite element methods to the exact solution. However, Neumann and Witherspoon relate dificulties of convergence near the exit node (Fig. 1) common to phreatic surface and downstream seepage face. Although those authors atribute that dificulty to Taylor and Browns method the fact is that they themselfs use a correction factor α and an additional correction β to get convergence in their method. Even so they admit that sometimes convergence is not reached after 25 iterations and provisions are taken in the computer program to stop calculation. Our experience with the method also confirms that for anysotropic media convergence may not be reached when the node C, where phreatic surface meets seepage face, is badly guessed at the initial step. R. Correia also refers the number of 32 iterations to get convergence by Taylor's method. In what concerns Desai's method, the number of iterations is not published. However, Bromhead (1977) discussing Desai's paper (1976 a) call the atention to the singular entrance point B (Fig. 1). In that point, as we have seen (Fig. 3c) the phreatic surface is normal to the slope, but this condition as those refered in Fig. 2 for point C of merging between phreatic surface and seepage face are valid only for isotropic media. If the aquifer is anisotropic boundary conditions must be stated as follows:

$$q.\hat{n} = 0 \qquad (16)$$

on impervious boundaries or on the phreatic surface. In the anisotropic case condition (16) is different from (15) since it involves the permeability tensor, i.e.,

$$\underline{q}\cdot\hat{n} = K_{ij}\cdot\frac{\partial\phi}{\partial x_i}\cdot n_j \;, \qquad i,j = x,y,z \qquad (17)$$

On the other hand stream lines are no more normal to equipotential lines. In particular at the entrance point the phreatic surface is not normal to the upstream free. Since that angle is not constant but varies from node to node of the network, it is not always easy to fix "a priori" the angle between the phreatic surface and the upstream face at entrance point. Although condition (16) associated to condition (13) on neighbour points of the phreatic line will tend to fix the correct orientation of the phreatic surface at the entrance point, to ensure a perfect solution at that singular point as well as at the downstream exit point the best solution would be to map the actual flow region into another according to the ratio of the principal permeabilities $\lambda = K_1/K_2$ If the principal directions of permeability coincide with x and y axises the governing differential equation (12) becomes for homogeneous anisotropic media

$$K_x \frac{\partial^2 \phi}{\partial x^2} + K_y \frac{\partial^2 \phi}{\partial y^2} + K_z \frac{\partial^2 \phi}{\partial z^2} - \bar{Q} = 0 \qquad (18)$$

For two dimensional flow without acrestion $\bar{Q}$

$$\frac{\partial^2 \phi}{\partial \left(\frac{x}{\sqrt{\lambda}}\right)^2} + \frac{\partial^2 \phi}{\partial y^2} = 0 \qquad (19)$$

where $\sqrt{\lambda} = \sqrt{K_x/K_y}$. If we put $x' = x/\sqrt{\lambda}$ (21), i.e., if we divide the horizontal lengths by $\sqrt{\lambda}$, the anisotropic problem is reduced to an isotropic one. Also an heterogeneous ortotropic layered medium (Fig.16) can be transformed in a isotropic one.

In the case of anisotropy with the principal axies not parallel to the x and y the problem still can be reduced to an isotropic one (Harr, 1962).

The advantage of transforming the anisotropic medium in a isotropic one by simple changing the geometry before the numerical process is carried out, for free surface flow problems, is that the singular points where free surface flow meets upstream and downstream faces can be treated in a correct way at each iteration (Figs. 2 and 3). And such points are the sources of the instability of the numerical process (Neuman and Witherspoon).

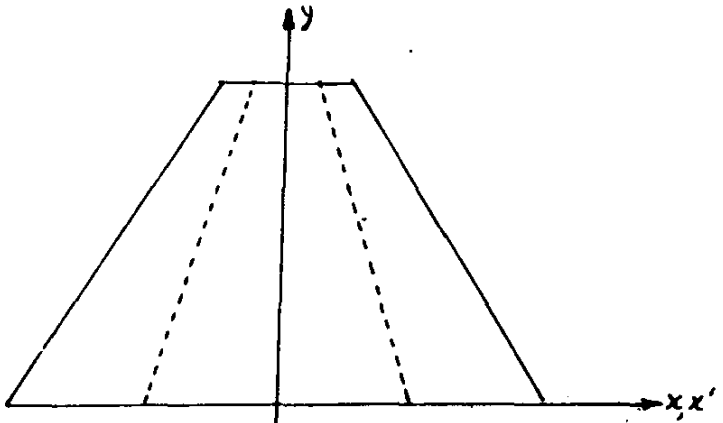


Fig.5 - Anisotropic medium contraction when $K_x > K_y$

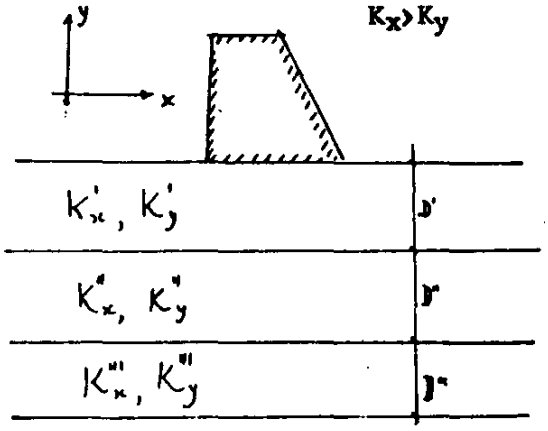


Fig.6 - Anisotropic heterogeneous layered medium

6 - FLOW THROUGH INHOMOGENEOUS MEDIA

For nonhomogeneous isotropic media Bear (1972) call the atention to the fact that $\phi = K(x,y,z)\cdot\phi$ is not a velocity potential, as we have already seen (no.1) it was true hence we would have

$$\underline{q} = -\,grad(K\phi) = -\left(grad\,K(x,y,z)\right)\cdot\phi - K(x,y,z)\,grad\,\phi$$

and for a constant hydraulic head ϕ we would have flow, due only to the variation of permeability, which is physically impossible. Therefore for continuously variable permeability we must write

$$\underline{q} = -K(x,y,z)\,grad\,\phi \qquad (3'')$$

However, the most common case is that in which there are two or more homogeneous media separated by surfaces where permeability is discontinuous. Such surfaces must be treated as internal boundaries satisfying certain conditions, e.g., the refraction law for the stream lines.

Let us consider a flow region divided in two subregions R' with permeability K' and R" with permeability K". We could solve the governing eq. (12) for ϕ, with $K = f(x,y,z)$ having a discontinuity over the curve C in the two dimensional case (Fig. 7). However, the best way is to divide the problem in two subproblems denoting potential in R' by ϕ' and in R" by ϕ''. We then search for a solution for ϕ' in R' and a solution for ϕ'' in R" satisfying their external boundary conditions on C' for ϕ' and C" for ϕ''. Also additional conditions must be stated on internal boundary C. Such conditions are:

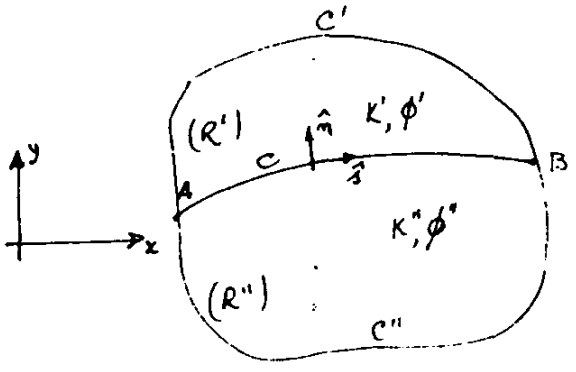


Fig. 7

$$\frac{\partial\phi'}{\partial s} = \frac{\partial\phi''}{\partial s} \quad \text{at all points of C} \qquad (20)$$

Also

$$q'_n = q''_n \;;\quad \underline{q}'\cdot\hat{n} = \underline{q}''\cdot\hat{n} \;;\quad (\underline{q}''-\underline{q}')\cdot\hat{n} = 0 \;;\quad K'_{ij}\frac{\partial\phi'}{\partial x_i}\cdot n_j = K''_{ij}\frac{\partial\phi''}{\partial x_i}\cdot n_j \qquad (21)$$

From condition (20) we should have on C : $\phi'(s) = \phi''(s) + b$ (22)

where b would be an arbitrary constant. However, since on the points A and B common to both regions we must have $\phi'(A) = \phi''(A)$ and $\phi'(B) = \phi''(B)$. Hence (20) is equivalent to have

$$\phi'' = \phi' \qquad (23)$$

In the finite element discretization using isoparametric triangular elements (Neuman and Witherspoon (1970)) if C does not passes through any element, condition (23) is fulfilled on all points of C, since equalization of ϕ' and ϕ'' on any nodes i and i+1 on boundary C implies equalization in every point of C between those nodes.

The same cannot be said from condition (21). This condition must substitute condition (11), valid for internal nodes. Therefore, the procedure of treating the inhomogeneous media as an homogeneous one with only different permeabilities from

element to element when crossing the boundary C between two media, is incorrect and one should expect effects even on the convergence process.

7 - VARIATIONAL METHOD

As seen the continuity of flow gives the following governing equation for steady state:

$$\frac{\partial}{\partial x}\left(K_x \frac{\partial \phi}{\partial x}\right) + \frac{\partial}{y}\left(K_y \frac{\partial \phi}{\partial y}\right) + \frac{\partial}{\partial y}\left(K_y \frac{\partial \phi}{\partial z}\right) + \bar{Q} \tag{12}$$

with boundary conditions

$$\phi = \phi_o \text{ on } (AB \text{ and } \bar{C}D \text{ in Fig.1}), \tag{24}$$

i.e., on the boundaries where potentials are fixed.

$$K_x \frac{\partial \phi}{\partial x} \cdot n_x + K_y \frac{\partial \phi}{\partial y} n_y + K_z \frac{\partial \phi}{\partial z} n_z + \bar{q} = 0 \tag{25}$$

on the boundaries where flux $\bar{q}$ per unit of surface is added or subtrated to the system.

For isotropic media ($K_x = K_y = K_z$) and on impervious parts of the boundaries (surface S_2= AD on Fig. 1), (25) becomes:

$$\frac{\partial \phi}{\partial n} = 0 . \tag{25'}$$

For the phreatic surface S_3 there are two conditions:

$$\phi = \bar{y} \quad (13) \quad , \quad q_n = 0 . \tag{14}$$

(13) states that on S_3 the potential must be equal to elevation and (14) is the equivalente of (25) and becomes equal to (25') for isotropic media.

According the well-known Euler's theorem of the calcul of variations (12) is equivalent to the minimization of the funtional

$$w\left[\phi(x,y,z)\right] = \iiint_R \frac{1}{2}\left\{\left[K_x \phi_x^2 + K_y \phi_y^2 + K_z \phi_z^2\right] - \bar{Q}\phi\right\} dx\,dy\,dz \tag{26}$$

where

$$\phi_x = \frac{\partial \phi}{\partial x}\,;\ \phi_y = \frac{\partial \phi}{\partial y}\,,\ \phi_z = \frac{\partial \phi}{\partial z} .$$

In fact Euler's theorem states that given the functional

$$w\left[\phi(x,y,z)\right] = \iiint_R f\left[x, y, z, \phi(x,y,z), \phi_x(x,y,z), \phi_y(x,y,z), \phi_z(x,y,z)\right] dx\,dy\,dz$$

the necessary and suficient condition to have a minimum over a bounded region R is that the unknown function f (x,y,z) satisfies the differential equation:

$$\frac{\partial f}{\partial \phi} - \frac{\partial f}{\partial \phi_x} - \frac{\partial f}{\partial \phi_y} - \frac{\partial f}{\partial \phi_z} = 0 . \tag{27}$$

Applying (27) to (26) we get (12).

If instead of the integral (26) the following is used

$$w(\phi) = \iiint_R \frac{1}{2} \left\{ \left[K_x \phi_x^2 + K_y \phi_y^2 + K_z \phi_z^2 \right] - \bar{Q}\phi \right\} dx\,dy\,dz + \iint_{S_2} \bar{q}\,\phi\, dS, \qquad (28)$$

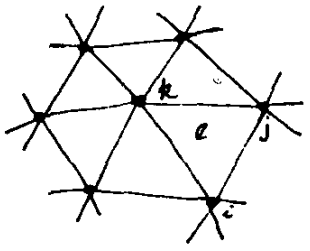
it can be shown (Zienckiewicz (1972)) that the minimization of (28) automatically includes the boundary condition (25).

8 - FINITE ELEMENTS APPROACH

In minimizing(28) by finite elements method the flow domain R is subdivided in a number of elements whose sides form a network.

Following the above author we substitute the unknown function $\phi(x,y,z)$ by a number of simple local functions N_i (x,y,z), N_j (x,y,z), N_k (x,y,z),..., defined on each element e,

$$\phi = \sum_e \left[N_i, N_j, N_k, \cdots \right] \left\{ \phi \right\}^e, \qquad (29)$$



such that when $x=x_i$, $y=y_i$, $z=z_i$; $N_i = 1$ and $N_j = N_k = 0$

When $x=x_j$, $y=y_j$, $z=z_j$; $N_j = 1$ and $N_i = N_k = 0$

etc.. (the same for the other nodes of element e)

With this definition it is evident that the integral (28) becomes a summ over the number of elements in which the flow region have been subdivided and since N_i, N_j, N_k are known functions of x,y,z, the functional w will be converted in a function of the parameters ϕ_i, the values of ϕ at the nodes of the network. Therefore the problem of minimizing a functional w becomes a problem of minimizing a function of n variables $\phi_1, \phi_2, \ldots, \phi_n$ so many as the number of nodes where the potential is unknown.

To get the minimum of $w(\phi_i)$ we must have

$$\frac{\partial w}{\partial \phi_i} = 0 = \sum_e \frac{\partial w^e}{\partial \phi_i} \qquad (30)$$

where the sumation is extended to all the elements of region R and to boundaries, in what concerns the surface integral of (28).

For each element e

$$\frac{\partial w^e}{\partial \phi_i} = \iiint_{R_e} \left\{ K_x \phi_x \cdot \frac{\partial \phi_x}{\partial \phi_i} + K_y \phi_y \cdot \frac{\partial \phi_y}{\partial \phi_i} + K_z \phi_z \frac{\partial \phi_z}{\partial \phi_i} - \bar{Q} \frac{\partial \phi}{\partial \phi_i} \right\} dx\,dy\,dz + \iint_{S_2} \bar{q} \frac{\partial \phi}{\partial \phi_i} dS \qquad (31)$$

But

$$\phi_x \equiv \frac{\partial \phi}{\partial x} = \left[\frac{\partial N_i}{\partial x}, \frac{\partial N_j}{\partial x}, \frac{\partial N_k}{\partial x}, \cdots \right] \cdot \begin{Bmatrix} \phi_i \\ \phi_j \\ \phi_k \\ \vdots \end{Bmatrix}^e, \qquad (32)$$

etc.

etc. and therefore $\frac{\partial \phi_x}{\partial \phi_i} = \frac{\partial N_i}{\partial x}$, etc. (33)

Substituting (32) and (33) into (31), we have

$$\frac{\partial W^e}{\partial \phi_i} = \iiint_{R_e} \left(\left\{ K_x \left[\frac{\partial N_i}{\partial x}, \frac{\partial N_j}{\partial x}, \frac{\partial N_k}{\partial x}, \dots \right] \frac{\partial N_i}{\partial x} + K_y \left[\frac{\partial N_i}{\partial y}, \frac{\partial N_j}{\partial y}, \frac{\partial N_k}{\partial y}, \dots \right] \frac{\partial N_i}{\partial y} + K_z \left[\frac{\partial N_i}{\partial z}, \frac{\partial N_j}{\partial z}, \frac{\partial N_k}{\partial z}, \dots \right] \frac{\partial N_i}{\partial z} \right\} \begin{Bmatrix} \phi_i \\ \phi_j \\ \phi_k \\ \vdots \end{Bmatrix}^e - \bar{Q} N_i \right) dx\,dy\,dz + \iint_{S_2^e} \bar{q} N_i\, dS = 0 \qquad (34)$$

Considering all the nodes of the element i,j,k,..., we may write in a compact form

$$[h]^e \cdot \{\phi\}^e + \{F\}^e = 0 \qquad (35)$$

where

$$h^e_{rs} = \iiint_{R_e} \left(K_x \frac{\partial N_r}{\partial x} \cdot \frac{\partial N_s}{\partial x} + K_y \frac{\partial N_r}{\partial y} \cdot \frac{\partial N_s}{\partial y} + K_z \frac{\partial N_r}{\partial z} \cdot \frac{\partial N_s}{\partial z} \right) dx\,dy\,dz \qquad (36)$$

is the "stifness matrix" and $r, s = 1, 2, \dots, N$ (N = nº of nodes)

$$\{F\}^e = - \iiint_{R_e} \bar{Q} N_r\, dx\,dy\,dz + \iint_{S_2^e} \bar{q} N_r\, dS \qquad (37)$$

is the "force vector".

Assembling the equations (35) for all the elements, we get the usual "equilibrium" equations:

$$[H] \cdot \{\phi\} + \{F\} = 0 \qquad (38)$$

where $H_{ii} = \sum_e h_{ii}$ (39) and $\bar{F}_i = \sum_e \bar{f}_i^e$ (39'), the summation beeing extended to all the elements that meet at node i, and $H_{ij} = \sum_e h^e_{ij}$ (39"), the summation being extended to the elements with the nodes i and j in common.

Before we go on, let us stress the analogy between (38) and the equilibrium equations for the elastic media. To the displacements in elastic body there corresponds the potentials in the flow medium. To the forces there corresponds rates of flow. Therefore $\bar{Q}$ is nothing but body forces and $\bar{q}$ is nothing but loads applied on the surface S_2. To impervious boundaries there corresponds surfaces of the elastic body without load and to equipotential boundaries there corresponds portions of the surface of the elastic body with imposed displacements. Certainly, the points on the boundary where displacements are imposed (null in particular) get reactions from the exterior. Therefore, points where potential is fixed have a rate of flow coming from or going to outside. If the body is in equilibrium as a whole, then the reactions must be in equilibrium with the surface loads, i.e., if there are no sources nor sinks, the rate of flow into the

system at some boundaries must balance the rate of flow on the others, if the aquifer does not expand nor contract.

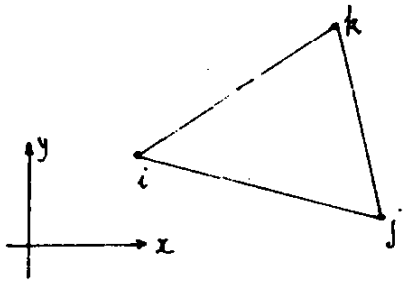
9 - SOLVING SYSTEM OF EQUATIONS FOR TRIANGULAR LINEAR ELEMENTS

If there is no flow in the z direction eqs.(36) and (37) become

$$h_{ij}^{e} = \iint_{R_e} \left(K_x \frac{\partial N_i}{\partial x} \frac{\partial N_j}{\partial x} + K_y \frac{\partial N_i}{\partial y} \cdot \frac{\partial N_j}{\partial y} \right) dx\, dy . \tag{40}$$

For the triangular linear element the shape function for node i is

$$N_i(x,y) = \left(a_i + b_i x + c_i y \right) / 2\Delta \tag{41}$$



where

$$a_i = x_j y_k - x_k y_j \;,\; b_i = y_j - y_k \;,\; c_i = x_k - x_j \tag{42}$$

$$2\Delta = \begin{vmatrix} 1 & x_i & y_i \\ 1 & x_j & y_j \\ 1 & x_k & y_k \end{vmatrix} = 2 \left[\begin{array}{c} \text{area of triangle} \\ ijk \end{array} \right] \tag{43}$$

As can be seen by substitution in (41), $N_i(x_i, y_i) = 1$

The other shape functions N_j and N_k would be obtained from (41) by circular permutation of subscripts.

Differentiating (41) and similarly for N_j and N_k and substituting in (41) one obtains the element "stiffness" matrix

$$\left[h\right]^{e} = \frac{K_x}{4\Delta} \begin{bmatrix} b_i b_i & b_i b_j & b_i b_k \\ & b_j b_j & b_j b_k \\ Sym & & b_k b_k \end{bmatrix} + \frac{K_y}{4\Delta} \begin{bmatrix} c_i c_i & c_i c_j & c_i c_k \\ & c_j c_j & c_j c_k \\ Sym & & c_k c_k \end{bmatrix} \tag{44}$$

Also performing the integration $\int_{R_e} \bar{Q} N_i \, dx\, dy = \iint \frac{\bar{Q}}{2\Delta} \left(a_i + b_i x + c_i y \right) dx\, dy$

the body force vector $$\left\{ f \right\}_{e}^{e} = - \frac{\bar{Q}\Delta}{3} \begin{Bmatrix} 1 \\ 1 \\ 1 \end{Bmatrix} \tag{45}$$

would be obtained; i.e. the rate of flow per unit volume $\bar{Q}$ assumed to be constant, can be divided in equal parts by the three nodes of the element.

Having scattered the h_{ij}^{e} of each element on the nodes of the network as refered in (39), (39') and (39''), we come to the system of linear equations

$$H_{rs} \phi_s - F_r = 0 \tag{46}$$

$$r, s = 1, 2, \ldots, N$$

where N is the number of nodes.

Some of the nodes, say M, lie on the boundaries S_1, S_3 and S_4 (Fig. 1) where the potential is stated. Therefore the number of unknowns can be reduced to N-M. After putting in right hand side of (46) the known values, we have

$$H_{rn}\phi_n = F_r - H_{rm}\phi_m \quad , \qquad m \neq n \tag{47}$$

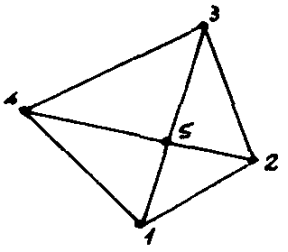
where $\underline{m}$ are the nodes where the potential is fixed.

After getting the N-M potentials from the linear system of equations (47), we can substitute them in the remaining set of M equations.

$$H_{mn}\phi_n = F_m \tag{48}$$

and obtain the rates of flow at nodes $\underline{m}$.

From a computer programming point of view it should be notice that the partial matrices H_{rn} of the coefficients and H_{rm} of the constant terms can easily be extracted from the global "stiffness" matrix H_{rs}; r,s = 1,2,...,N, if we codify the nodes where the potential is unknown, say nodes type 0, and the nodes where the potential is known, say nodes type 1.
It should also be refered that although the basic "stiffness" matrix is that of the triangular element, the "stiffness" matrix for a quadrilateral element is obtained by condensation at a further step in the program (Desai and Abel, 1972).



10 - NON-HOMOGENEITY AND ANISOTROPY

Zienkiewicz (1971) call the attention to the fact that the functional (28), to be minimized, has no derivatives of the permeability K_x, K_y and K_z. From that situation he infers that the permeabilities can change abruptly between elements or be allowed to vary within, since account of such a variation is taken in integrals evaluating the element matrices. However, one should notice that, even in homogeneous media there is no continuity of specific flux rate q_n normal to the common side of two contiguous elements, as there should be. This happens, for example, in the triangular linear elements, which have continuity of ϕ on the sides in common, but not continuity in the derivatives of ϕ and therefore of q_n. Of course as long as the number of elements tends to infinity the "chapeau" functions, by which we substitute the potential, tends to the actual $\phi(x,y)$ and therefore the derivatives of ϕ tend to be continuous and hence q_n becomes continuous at the limit . In the case of non-homogeneous media we already have seen (21) that the continuity of q_n must be explicitment formulated at the internal boundary C (Fig. 7) where permeability K_{ij} is discontinuous. The increasing number of elements will assure continuity of the derivatives of ϕ but not that of $q_n = \underline{q} \cdot \hat{n} = K_{ij}\frac{\partial \phi}{\partial x_i} n_j$.

Therefore for the case of abrupt change on an internal boundary C (Fig. a) (21) and (23) should be used instead of the "equilibrium" equations (38). Since for triangular linear elements $\frac{\partial \phi}{\partial x} = const = b_i \phi_i / 2\Delta$ and $\frac{\partial \phi}{\partial y} = const = c_i \phi_i / 2\Delta$ (49)

(21) would simply become

$$\begin{bmatrix} K'_{xx} & K'_{xy} \\ K'_{yx} & K'_{yy} \end{bmatrix} \begin{Bmatrix} \sum b'_i \phi'_i \\ \sum c'_i \phi'_i \end{Bmatrix} \frac{1}{2\Delta'} [\eta_x, \eta_y] =$$

$$= \begin{bmatrix} K''_{xx} & K''_{xy} \\ K''_{yx} & K''_{yy} \end{bmatrix} \begin{Bmatrix} b''_i \phi''_i \\ c''_i \phi''_i \end{Bmatrix} \frac{1}{2\Delta''} [\eta_x, \eta_y] \quad (50)$$

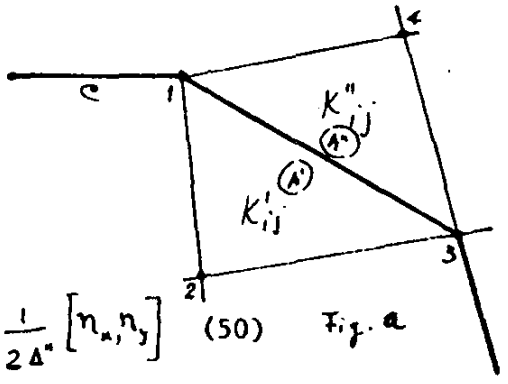


Fig. a

with $b'_i = y_j - y_k$, $c'_i = x_k - x_j$, etc.,

where i,j,k are circular permutations of 1,2,3 and 1,3,4 for triangles A' and A" respectively.

In the case of homogeneity but with anisotropy the element "stiffness" matrix must be calculated in local coordinates x_1, y_1 i.e. in the principal directions of the permeability. However, since potentials ϕ are scalar quantities, the assembling of the "stiffness" matrix for the whole network can be perfomed in the usual way.

Before we go further let us have a brief reference in relation to the best type of elements to be used. The linear triangular element used by Zienckiewicz (1971), Neuman and Witherspoon (1970) and others, has the advantage of simplicity and the values of ϕ are continuous on the side common to each pair of contiguous elements. As refered above (No.9) the triangular mesh can be transformed in a quadrilateral one, by condensation, but that does not increase accuracy. Recently other types of elements have been used such as the quadrilateral quadratic element (R. Oliveira (1977)). Although a higher degree accuracy for this type of element may be espected, continuity of ϕ and its derivatives on the common side of each pair of contiguous elements remains to be shown.

11 - FURTHER DISCUSSION OF FINITE ELEMENTS TECHNIQUE AND SOME SUGESTIONS FOR IMPROVEMENT

We have already seen (No.5) that methods based on deformation of the upper elements may do not converge near the exit point where phreatic surface meets the seepage face (point C in Fig. 1). Neumann and Witherspoon (1971) also refer outflow at the singular point common to the upstream face of the dam and the impervious base. Also the number of iterations necessary to get convergence is rather large. We think that this large number of iterations and also the instability of the process has something to do with the relationship between the imposed maximum error $|y - \phi| < \varepsilon$ and the mesh size. In fact with a large mesh size

we can expect that the difference $y-\phi$ changes sign from one iteration to the next maintaining in both an absolute value $|y-\phi|$ greater than the imposed maximum error ϵ.

Another point to refer is the error in the rate of flow. Neumann and Widerspoon (1971) and others impose no maximun error on the rate of flow at free surface. However, when the condition $|y-\phi|<\epsilon$ is reached the flow through the final phreatic surface will not be zero. Our experience have shown that in some cases the minimum $|y-\phi|$ in the process does not correspond to the minimum flow through the phreatic surface.

Since a good approximate initial position of the exit point C is essential for "convergence" in any method, we must state a way of determinating that position as a preliminary phase. Going into the physical process we can see that with a few initial steps we can obtain pratically the final position of the exit point C.

In fact if a trial free surface (assumed a polygonal or straight line) is fixed at a high level and we impose $\phi = y$ on AC_1, the system will respond with an average inflow through AC_1. If AC_n is fixed too low, and $\phi = y$ - the system will respond with an average outflow through AC_n. Therefore, the correct exit point C is that between those sucessive points C_i an C_{i+1} for which the average rate of low through AC_i and AC_{i+1} are of oposite signe. Therefore the exit point may be obtained interpolating between hese residual average rates of flow of oposites signs of the last steps. We get the position of C and although that of a provisional free surface. This is obtained by interpolation between AC_{n-1} and AC_n.

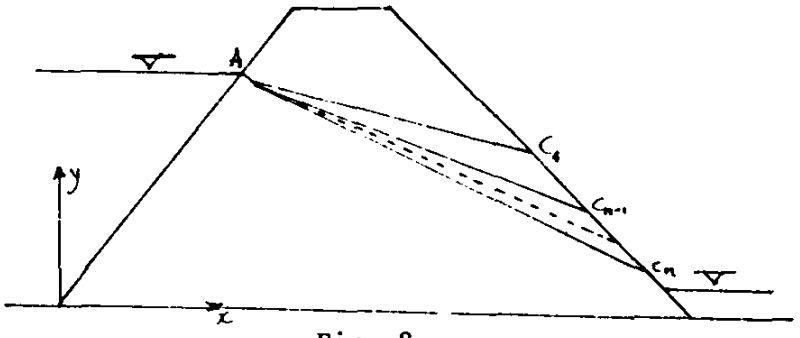


Fig. 8

After this preliminary steps the change in the shape of the free surface, assumed at start, can be done by Desai's fixed network, for example. If in a different way we take AC_n too low and assume no flow on it, the system will respond with potential ϕ_{C_n} larger than y_{C_n} since the flow region is restricted. Then we must move C uppwards until $\phi_{C_n} - y_{C_n} < 0$.

12. SOME EXPERIENCES USING SOME OF THE SUGESTIONS MADE

As a first example we refer an homogeneous isotropic dam (Fig. 9) with a vertical filter. We started with a trial exit point C_1 at the elevation of 84,00 m and went down to elevations of 74.00 m, 64.50 m and 54.00 m, imposing in each calculation $\phi = y$ on AC_i. For elevation of 64.50 we obtained the average inflow of $+.46\times10^{-6}m^3/s.m$ through AC_3 and for elevation of 54.00 we obtained an outflow of $-.32\times10^{-6}m^3/s.m$ through AC_4. Doing a linear interpolation between these positions of C in proportion of the flow rates for the intermediate points, we obtained a tentative free line which practically coincides with that obtained by a large number of iterations with a deformable mesh (Martins and Vargas, 1976).

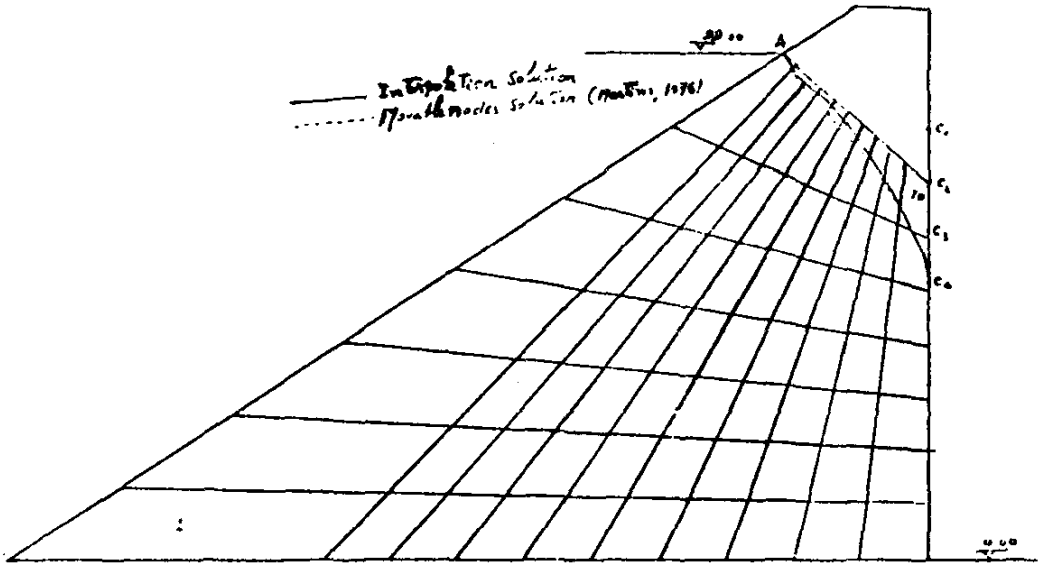


Fig. 9 (1st. Ex.) - Homogeneous dam with vertical filter. F.E. mesh.

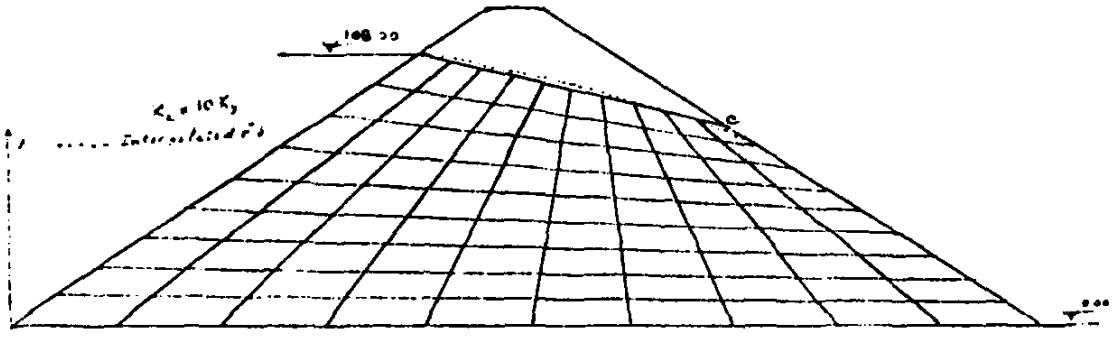


Fig. 10 (2nd. Ex.) - Anisotropic dam F.E.mesh. Notice on F.S. after determination of exit point C.

A second example refers to an anisotropic homogeneous dam (Fig.10). The ratio between the horizontal and vertical permeabilities is 10.We first got an approximate exit point at elevation 80.00. Further we impose the condition of noflow on the estimated free surface and did the interpolation sugested by Desai (1976). The result very much approaches a preceedings one with a large number of iterations (Martins and Vargas, 1976).

As a third example we present the seepage flow out of a ditch . Although the first interpolation according to Desai seems not to give a good free surface line (Fig. 11), nevertheless the flow rate out of the ditche pratically coincides with that given by Jeppson (1968 c).In fact we obtained the total outflow of $Q_t = 0{,}995 \times 2m^3/(s \times m)$ for $K = 10^{-6}$ m/s which gives $Q_t/KD = 6{,}6$ pratically coincident with that given by the same author (1968 a, p.280, T/D = 4 and H/D = 4).

As a forth example (Fig. 12) the problem of the seepage through an heterogeneous aquifer is solved by interpolation within a fixed mesh with no flow at the upper surface (Desai, 1976) and,alternatively, solved after the exit point is obtained by trial according the foregoing sugestion. No significative diferences have been found between the two techniques. In both cases the rate of flow is about 20% higher than that given by the Dupuit approximation (Bear, 1972, p. 373).

Finally, a fifth example (Fig.13) deals with an heterogeneous, anisotropic earth dam. The problem have been solved with a fixed network. The first interpolation according to Desai gives a solution somewhat far from Correia's one (1977). It should be notice that both methods does not includ discontinuity of permeability on the boundaries between the two media as an autonomous internal boundary (no. 6). Correia's program uses quadrilateral quadratic elements and ours uses a simple quadrilateral mesh obtained by condensation of triangular elements. His solution has a much smaller drop of potential through nucleous than ours.

13 - LAST DISCUSSION AND CONCLUSIONS

The convergence of the finite elements approach to the solution of the free surface flow problem means essentially that for properly posed problems when the maximum size of the mesh tends to zero the finite elements solution tends to the exact one, which is unique. It does not mean that for a given mesh with a fixed number of nodal points and a given form of discretization there is a polygonal free surface such that the potential ϕ exactly coincides with the elevation $\bar{y}$, the rate of flow being simultaneously null at the vertices.

Therefore, for a given mesh, we cannot impose at will a limit to the maximum error $\varepsilon = |\phi - \bar{y}|$, independent of the mesh size. However, we feel that for a given mesh and a given form of discretization, there is a polygonal free surface which minimizes the error ε. To get it we might begin to consider a generalized form of funtional $\omega = \int_v f[x, y, \phi(x,y), \phi_x, \phi_y, \bar{y}(x,y)]\, dv$ associated to the functional $\varepsilon = |\phi - \bar{y}|$, both to be simultaneously minimized within the physical domain where the flow can exist . The solution of such a problem would require basic theoretical knowledgements perhaps not yet available.

Meanwhile we may search the best position of the free surface according the following lines:

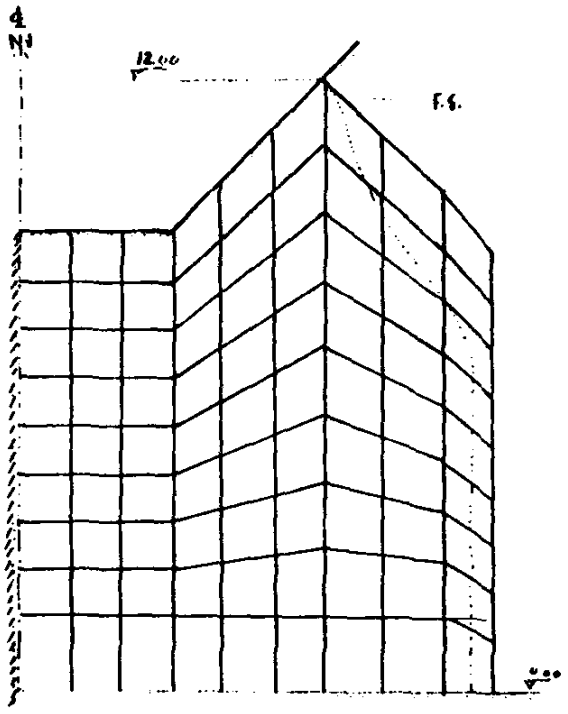


Fig. 11 (3rd. Ex.) - Flow out of a ditch. F.E.mesh Desai's first interpolation for F.S.

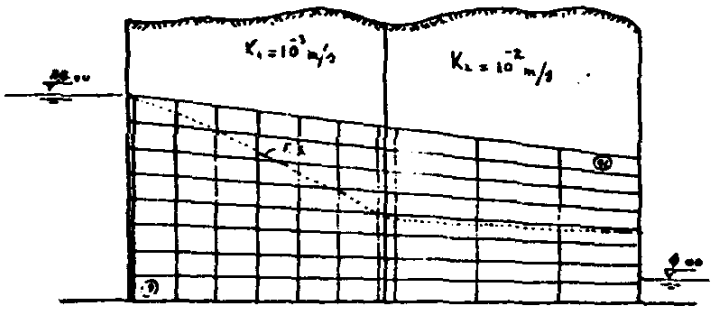


Fig. 12 (4th. Ex.) - Seepage through heterogeneous aquifer. F.E.mesh.

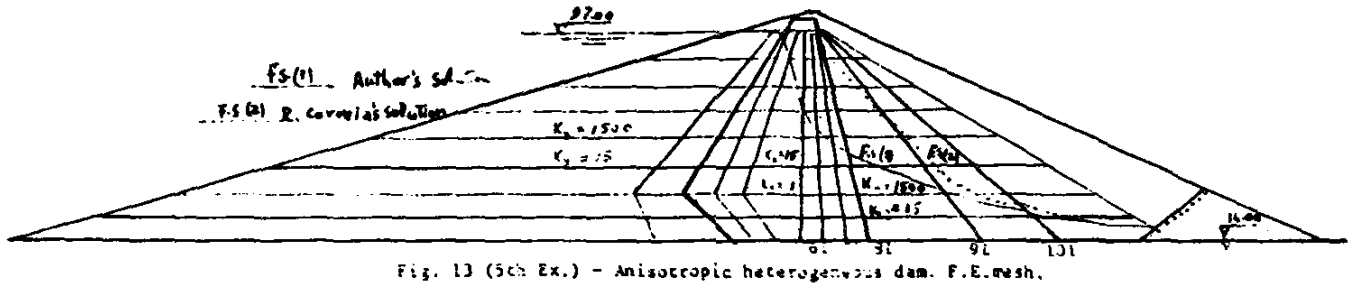
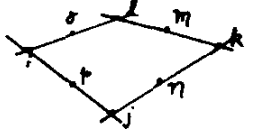


Fig. 13 (5th Ex.) - Anisotropic heterogeneous dam. F.E.mesh.

1. Begin with the determination of exit point since this point is critical in relation to the position of the free surface as stated before (no.11) at the provisional free surface assumed to be some polygonal (e.g. a straight line) between the entrance point and the current tentative exit point. Calculating the flow rate Q_i at each nodal point i at on the corresponding free surface, we average Q_i. When Qaverage becomes negative, i.e., the flow domain becomes so restricted that on the trial free surface there is outflow, as an average, we stop the process and obtain the approximate exit point by interpolation between the two last positions. We also get an interpolated free surface. Further, we recalculate - setting a noflow condition on the just obtained free surface. The actua shape of the free surface may now be obtained doing the kind of interpolation sugested by Desai (1976) for a fixed network.

ii. For a properly posed problem in heterogeneous media, we must consider internal boundaries where the permeability is discontinuous. This cannot be done correctly treating the nodes at those boundaries as internal normal nodal points, unless we used a kind of mixed finite element where at corners i,j,k,l the unknows would be potentials and at midsides m,n,o,p the unknowns would be normal specific rates of flow q_n.



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SEEPAGE CHARACTERISTICS THROUGH AN ABANDONED TAILINGS PILE

Morilla, Alberto G. and Fortier, David. H.

ABSTRACT : Twelve tailings pile have been installed in the Coeur d'Alene Mining District in northern Idaho since 1968. Several of these ponds have recently been filled and abandoned. The abandonment of mine tailings piles may have detrimental effects on the water resource system in the immediate area by leaching of precipitation through the pile material.
The Idaho Bureau of Mines and Geology and the University of Idaho, College of Mines in cooperation with the U. S. Bureau of Mines have conducted a study to determine the hydrogeological factors that control the movement of ground water through the abandoned Page tailings pile in this area in Northern Idaho. A data collection network was designed to collect data on ground water potential and ground water quality. Analysis of water level data showed the existence of a ground water mound under the east portion of the tailings pile. The flow system in the pile responds both to precipitation events and to periods of no recharge. A finite element steady-state mathematical model was constructed to analyze the ground water flow system in the Page pile. A sampling and testing program of the pile provided information on values of hydraulic conductivity. This data was incorporated as input to the mathematical model. Operation of the model showed that the location and fluctuations of the regional ground water table and the quantity of recharge to the tailings pile from precipitation were the primary controlling factors for the location and height of the ground water mound under the east side of the pile.
A sewage lagoon system was installed on the abandoned pile after the beginning of this study. A previous study reported on the water level rises resulting from the construction and filling of the lagoons. Output data from operation of the model, with the sewage lagoons system located on the pile, showed similar water level rises, and a maximum potential leakage of seven percent of inflows.

RESUME : Depuis 1968 douze bassins de décantation ont été installés au Nord d'Idaho. Plusieurs de ces bassins ont récemment été remplis et abandonnés ; leur abandon peut avoir des effets défavorables sur les ressources hydrauliques de la zone immédiate, en conséquence de la lixiviation des eaux de précipitation à travers les matériaux du bassin de décantation.

University of Idaho - U. S. A.

Le Bureau de Mines et Géologie d'Idaho et le Collège des Mines de l'Université d'Idaho, en coopération avec le Bureau U. S. de Mines ont mené une étude pour déterminer les facteurs hydrogéologiques qui contrôlent le mouvement de l'eau souterraine à travers le bassin de décantation abandonné de Page au Nord d'Idaho. Un système fut désigné pour recueillir des données sur l'eau souterraine en puissance et la qualité de celle-ci. L'analyse de données du niveau d'eau a montré l'existence d'un seuil d'eau souterraine dans la partie Est du bassin. L'écoulement dans le bassin de décantation est dynamique et montre tant les influences des précipitations que les périodes de non-recharge.
Un modèle mathématique en régime permanent d'éléments finis fut construit pour analyser l'écoulement de l'eau souterraine dans le bassin de Page. Un programme d'échantillonnage et d'essai du bassin a fourni des informations sur les valeurs de la conductivité hydraulique. Ces données furent incorporées comme entrées dans le modèle mathématique. L'opération du modèle montra que la situation et les fluctuations du niveau piézomètrique régional et la quantité de recharge de précipitation dans le bassin étaient les facteurs primaires de contrôle pour l'emplacement et la hauteur du seuil d'eau souterraine sous le flanc Est du bassin de décantation.
Après le commencement de cette étude, un système de lagune d'eaux résiduelles fut installé sur le bassin abandonné. Une étude préalable informa sur la remontée du niveau d'eau résultant de la construction et du remplissage des lagunes. Les données de sortie de l'opération du modèle avec le système de lagunes résiduelles situé sur le bassin montraient des remontées similaires du niveau d'eau. Les résultats du modèle indiquaient une infiltration potentielle maximum de 7 % des entrées.

RESUMEN : Desde 1968 se han ubicado doce balsas de decantación al Norte de Idaho. Recientemente algunas de estas balsas han sido colmatadas y abandonadas, lo que puede acarrear efectos defavorables en los recursos hidráulicos del área inmediata, como consecuencia de la lixiviación de los materiales apilados, por las aguas de precipitación.
La Oficina de Minas y Geología de Idaho, y el Colegio de Minas de la Universidad de Idaho, en cooperación con la Oficina de Minas de USA han realizado un estudio para determinar los factores hidrogeológicos que controlan el movimiento del agua subterránea a través de una de estas balsas de decantación. Se proyectó un sistema para recopilar datos sobre la piezometría del agua subterránea y su calidad. El análisis de los datos piezométricos mostró la existencia de un umbral de agua subterránea en la parte Este de la bolsa de decantación. El flujo a través de estos sedimentos puso de manifiesto tanto las influencias de las precipitaciones como de los periodos de no-recarga.
Para analizar este flujo del agua subterránea, se ha elaborado un modelo matemático de elementos finitos en régimen permanente. Un programa de muestreo y ensayos en los sedimentos ha aportado información sobre los valores de la conductividad hidráulica. Estos datos fueron incorporados como entradas en el modelo matemático, el cual ha puesto en evidencia que la situación y las fluctuaciones del nivel piezométrico regional y la cantidad de recarga en el apile por precipitación, son los factores condicionantes de la ubicación y altura del umbral piezométrico en la parte E de la balsa de decantación. Tras iniciar este estudio, se instaló un sistema de lagunas para aguas residuales en la balsa abandonada. Un análisis pre-

vio aportaba información relativa a la subida del nivel de agua, como consecuencia de la construcción y llenado de las lagunas. Los datos de salida del modelo, con el sistema de lagunas de aguas residuales localizadas en la balsa, mostraban subidas similares del nivel piezométrico, y una infiltración potencial máxima del 7 % de las entradas.

INTRODUCTION

Abandoned mine tailings piles are potential sources of long-term pollution of water resources. A number of tailings piles have been filled and abandoned in the Coeur d'Alene Mining District in Northern Idaho. This paper includes an analysis of the hydrologic factors that control movement of ground water through an abandoned tailings pile in the Coeur d'Alene Mining District in an effort to gain some understanding of the long term impacts of abandoned tailings piles on the water resources of the area.

PAGE TAILINGS PILE

The Page tailings pile, the oldest abandoned tailings pile in the district, was selected for investigation. The Page pile is located along the South Fork of the Coeur d'Alene River approximately one mile west of Smelterville, Idaho in the Coeur d'Alene Mining District (Figure 2). The pile has a surface area of about 70 acres with an average depth of 25 feet. It contains approximately 2.8 million cubic yards of tailings. The tailings pile was selected as a study area because it was small enough to be instrumented as a unit. The pile is located on a swamp area and is underlain by about 90 feet of alluvial material (Norbeck, 1974). Mine wastes were first discharged into the tailings pond at the study site in 1926 with the development of the flotation technique for metal recovery. The pond was operated continuously until 1969 when mining and milling ceased.

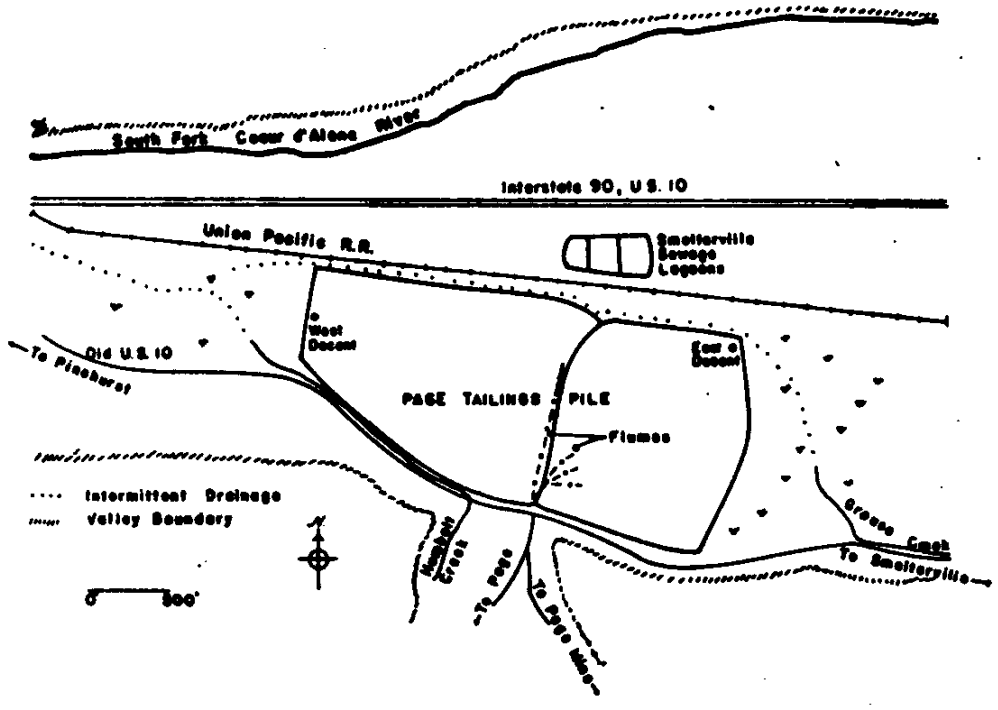


FIG. 2 LOCATION OF THE ABANDONED PAGE TAILINGS PILE

The Page tailings pile can be divided into two parts; the east pond was primarily formed by the tailings from 1926 to 1948. The center road that crossed the pile was originally the west embankment of the east pond. In 1948 the west side of the pile was started and materials were occasionally deposited in the east pile.

In 1956, sand backfill was initiated in the operation of the mine. The tailings slurry from 1956 to 1969 was mostly a slime mixture as compared to the sandy slime mixture deposited earlier. Most of the east part of the pile and the bottom of the west part are believed to be composed of coarser materials than the top of the west pile because most of it was deposited prior to the initiation of sandfill operations.

The deposition of tailings was accomplished by a network of flumes. A flume from the mine was located along the center dike road. Decants were located at the extreme west end of the west pile and the northeast corner of the east pile. Natural zoning of materials should exist within the pile because of the method of deposition. The size distribution of tailings in the tailings pile should be controlled by two major factors: (1) the location of discharge points from the flume system and (2) the use or non-use of the sandfill technique. Some consolidation and reduction of permeability may be present at depth in the east portion of the pile because of the greater period of deposition and the greater depth of the pile. A major change in land use occurred on the Page tailings pile in 1973. A sewage lagoon system, which is used to treat sewage from several towns in the area, was located on the abandoned tailings pile. The construction and filling of the lagoons has had a major impact on the flow system in the pile. Hitt (1974) described the impact of the construction and filling of the lagoons on the flow system in the pile. The ground water flow characteristics that existed prior to the installation of the lagoons will primarily be described in this paper.

Water quality data have been collected to estimate the environmental impact of the abandonment of the Page tailings pile. However, results from that part of the study are not included in this paper.

DATA COLLECTION NETWORK

A data collection network was designed to obtain data on the characteristics of ground water movement through the pile and the impact on the water resource system in the basin. Data collected included precipitation, atmospheric pressure, surface water discharge and quality and ground water potential and quality. A continuous precipitation recorder was installed at the site to verify the data from the precipitation station located at Kellogg. A recording microbarograph was installed near the pile to monitor changes in atmospheric pressure to help interpret the fluctuations of ground water potentials. Data were collected on the horizontal and vertical distribution of potential both within the tailings pile and in the surrounding alluvium. A total of sixty-eight piezometers were installed at various depths inside the pile with an additional fifty-two sites located in the alluvium surrounding the pile (Figure 3).

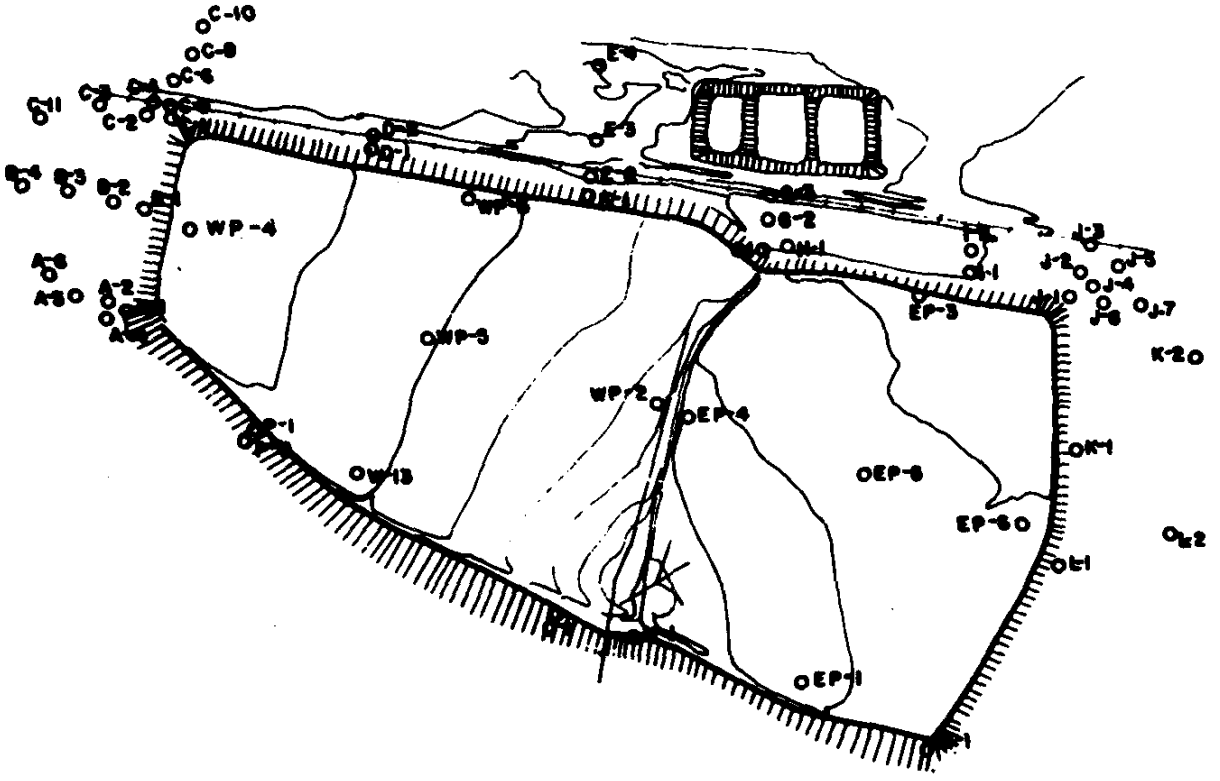


FIG. 3 MAP OF THE PAGE TAILINGS PILE AND VICINITY SHOWING THE LOCATION OF DATA COLLECTION SITES

The only source of ground water recharge to the tailings pile is direct precipitation. The monthly precipitation and typical hydrograph from the flow system are shown in Figure 4. A close correlation may be seen between the cumulative departures from the mean precipitation and the typical hydrograph. The flow system in the tailings is dynamic; it responds to recharge events and to periods of no recharge. The configuration of the water table in the Page pile and surrounding alluvium has been discussed in a previous paper (Ralston and Morilla, 1974) (See Figure 5). A ground water mound is present under the east portion of the tailings pile. The contours of water level elevation of the ground water system in the alluvium indicate that the pile does not have a major impact on the regional flow system. An east-west vertical section, presented in Figure 6, shows the pattern of water movement within the pile. The equipotential lines in the tailings material are connected with those in the regional flow system to show the direction of water movement. The existence of the ground water mound in the east side of the pile may be explained in several ways. First, much of the tailings in the east side were deposited prior to removal of the coarser fraction of the slurry for sand backfilling the mine.

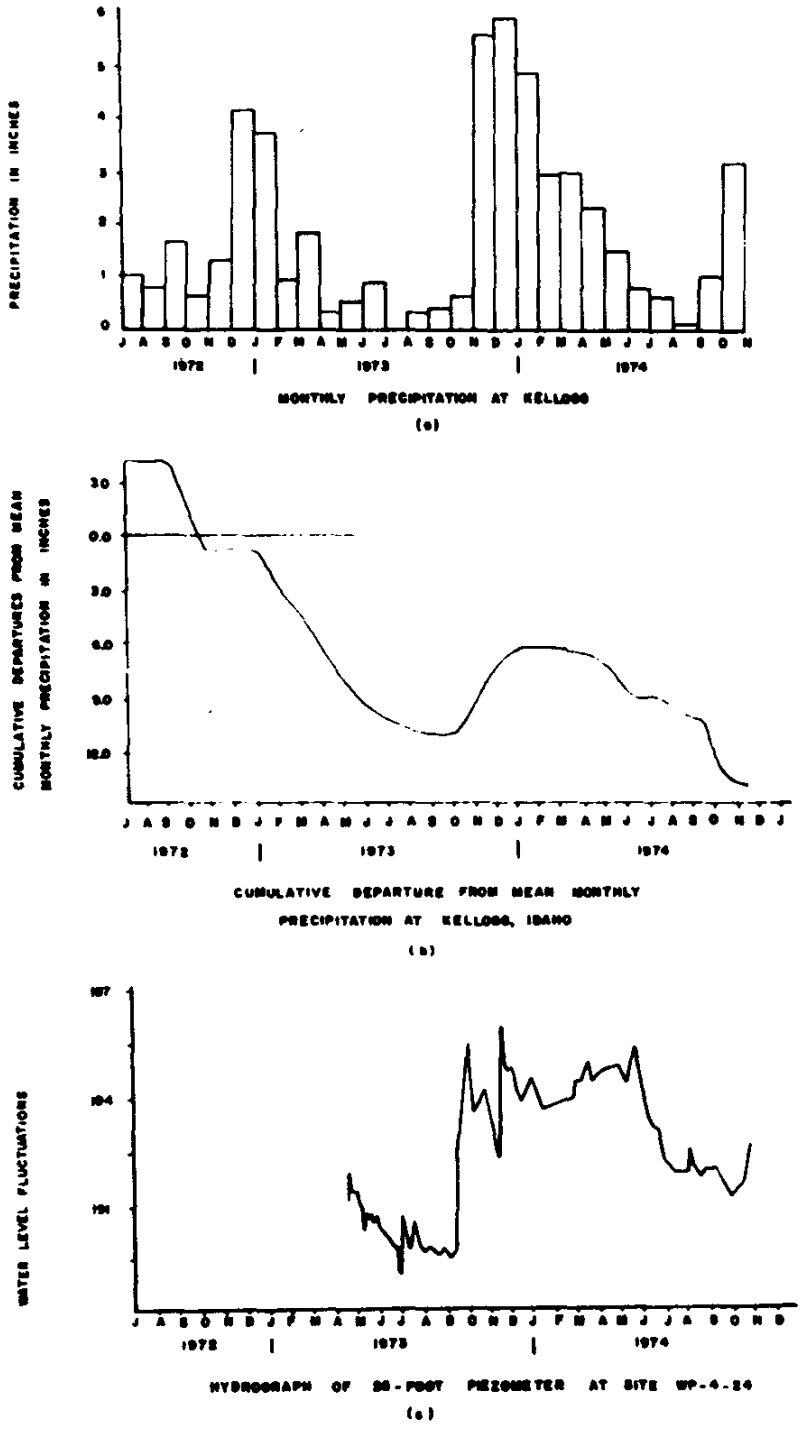
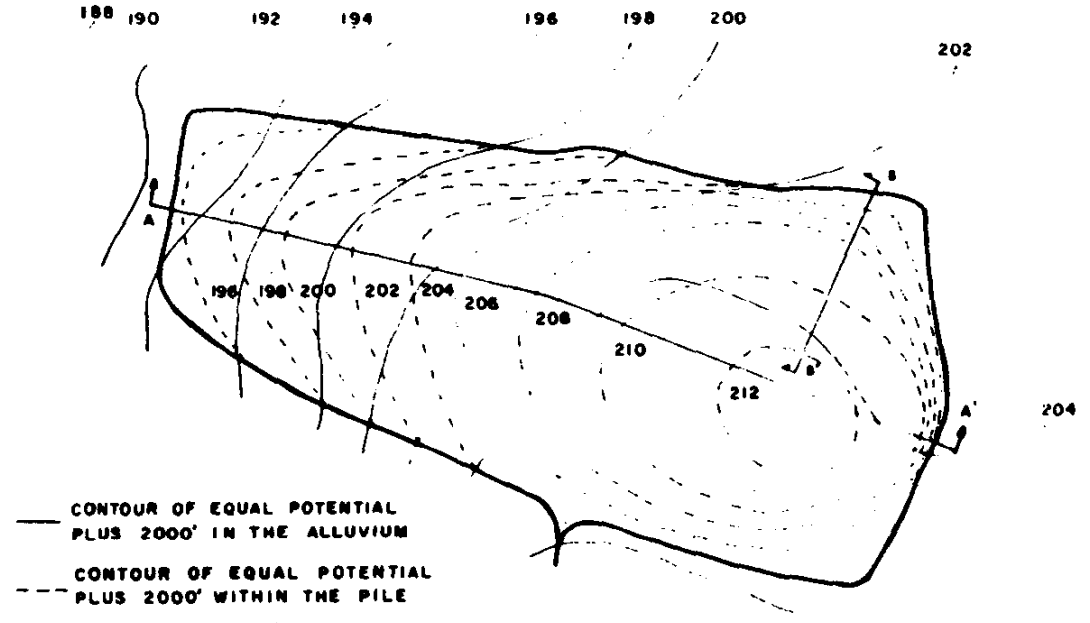


Figure 4 MONTHLY PRECIPITATION, CUMULATIVE DEPARTURES FROM MEAN PRECIPITATION AND RELATED WATER LEVEL FLUCTUATIONS IN PIEZOMETER-PAGE PILE

FIG. 5 CONTOURS OF THE WATER TABLE IN THE PAGE PILE AND SURROUNDING ALLUVIUM



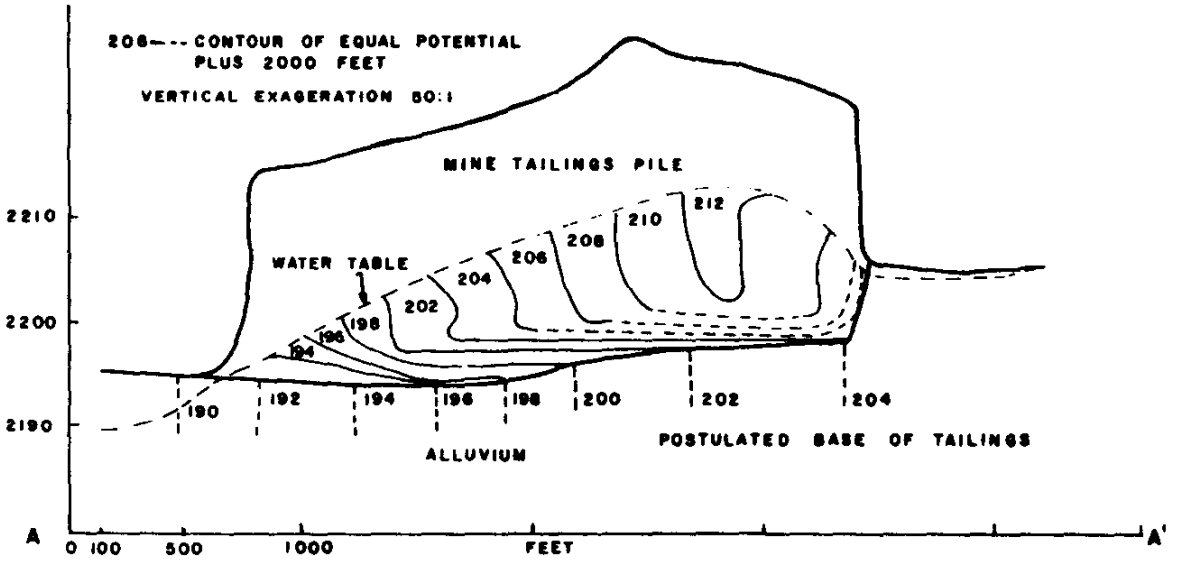


FIG. 6 POTENTIAL DISTRIBUTION WITHIN THE PAGE PILE AND ALLUVIUM ALONG EAST-WEST CROSS-SECTION A-A'

The hydraulic conductivity of the east pile would on the average, be greater than that of the west pile. At the same time, the east half of the pile has been in existence approximately twice as long as the west portion. It is possible that a compacted layer with a lower hydraulic conductivity may have formed at the bottom of the east pile as a result of increased loading of overburden. This phenomena has been reported in earlier investigations by researchers working on an operating tailings pond (Williams, Kealy and Mink, 1973). The downward movement of water would be restrained to some extent by the existence of the compacted layer and the water would be forced to mound under the east side of the pile. An alternative explanation for the mound is presented later in the paper.

HYDROLOGIC PARAMETERS

Determination of the distribution of hydraulic conductivity of the tailings materials required the extraction of undisturbed samples of tailings at various depths and locations in the pile. Six investigation holes were drilled with a hollow-stem power auger. A total of sixteen samples were obtained by inserting Shelby tubes as the holes were being drilled. Data on hydraulic conductivity obtained by the falling head permeameter method are presented in Figure 7. In several piezometers a slight decrease in hydraulic conductivity occurs with depth. Distinct differences in hydraulic conductivity with depth that would indicate the existence of a compacted layer were not observed. A more extensive drilling and sampling program is required to describe the variations of hydraulic conductivity inside the pile.

The above data suggest that the materials that compose the Page tailings pile may be considered as homogeneous. The hydraulic conductivity of these materials was estimated to be 2.9×10^{-6} cm/sec. This computed value of hydraulic conductivity along with a typical value of 30 cm/sec. for the hydraulic conductivity of the alluvial materials was incorporated as input to a steady-state finite element model of the flow system in the Page tailings pile.

FINITE ELEMENT MODEL

A model of the flow system in the tailings pile was constructed using an available finite element computer program. The finite element program was developed by R. L. Taylor at the University of California, Berkeley, and has been used in several studies by the U.S. Bureau of Mines (Kealy and Busch, 1971). The program can handle steady-state two dimensional confined and water table flow conditions and uses an iterative technique to locate the steady-state phreatic surface for the free surface points. A cross-section defining the geometry of the problem and the boundary conditions serves as the first step for the construction of the mathematical model. The cross-section is divided into discreet elements delineated by model points forming a mesh configuration. Each element is assigned its material type and characteristic hydraulic conductivity. The recharge to the pile is modeled

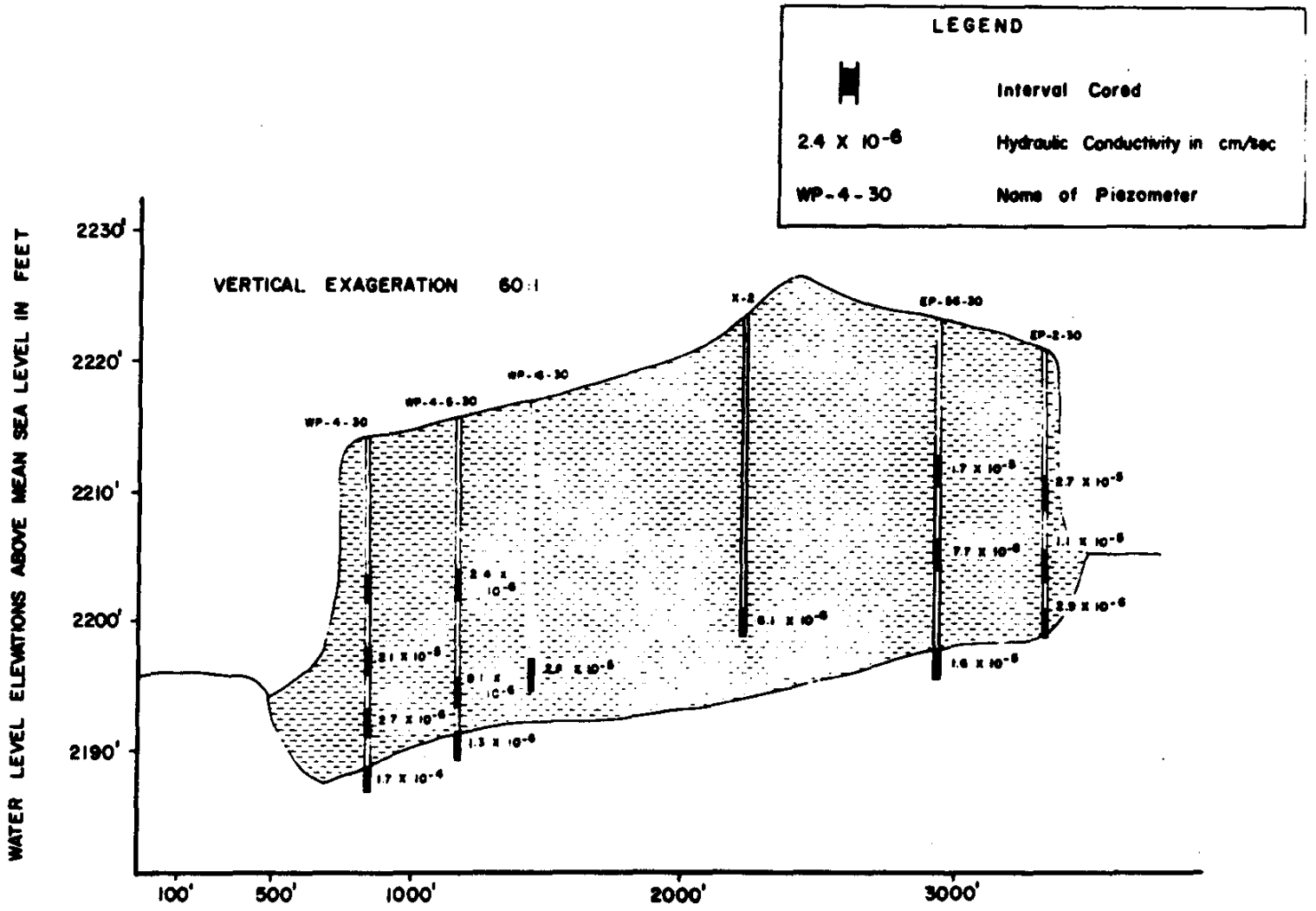


Figure 7. LOCATION OF CORE SAMPLES AND LAB HYDRAULIC CONDUCTIVITY VALUES FOR SAMPLES FROM THE STUDY AREA

as flow into the mathematical model. Output consists of nodal locations, potentials and element velocities and directions, Figure 8. The flow system in the pile is dynamic and the finite element program used depicts the system only as steady-state. The model thus has certain limitations. To gain some experience and knowledge a small model of the cross-section B-B' of Figure 5 was constructed. With the experience gained with the construction of the first model a more efficient model of the full cross-section of the pile was constructed. Several runs using different recharge rates show the dynamic character of the flow system, Figure 9. The extent and location of the ground water mound varies with the recharge input to the tailings. The ground water levels calculated using the model correlate closely with the measured water levels. The model is thus responding in a manner very similar to the physical system. The location of the regional ground water table was found to be an additional controlling factor on the existence and location of the ground water mound in the east side of the pile. Outputs from the finite element program for the same recharge rates and different pressures differentials at the bottom of the pile are illustrated in Figure 10. A distinct drop in water levels occurs for the same recharge rate when the regional water table is lowered.

The finite element model is instrumental in illustrating the hydrological interconnection between the flow systems inside and outside the tailings pile by showing the impact of the location of the regional water table on the water levels inside the pile. The mathematical model is also useful in verifying some of the conclusions obtained from an earlier U.S. Bureau of Mines sponsored study on the short term impacts of the location of a sewage lagoon system on the tailings pile, Hitt (1974). The changes in water level were monitored by field piezometers before, during and after lagoon filling. The subsequent impact is shown in Figure 11. Only a localized area beneath the sewage lagoon system responded to the filling of the lagoons. The sewage lagoon system was incorporated in the finite element model. The configuration of potentials before and after filling of the lagoons from model operations are shown in Figure 12. The model results are similar to measured water levels.

The computer program output allowed the calculation of the potential leakage from the sewage lagoon system into the tailings pile. Summation of the element flows normal to the bottom of the lagoons gives a potential discharge from the sewage lagoon system. This discharge is equal to seven percent of the total daily inflow to the system. This would be the maximum leakage that would occur from the system if the hydraulic conductivity of the materials under the lagoons were the same than that of the rest of the pile. The bottom of the lagoons were slightly compacted during the construction period. Solid accumulation on the lagoon floor over a period of time will create an organic mat. The combined effect of these two factors should significantly reduce the potential for discharge of sewage water into the pile.

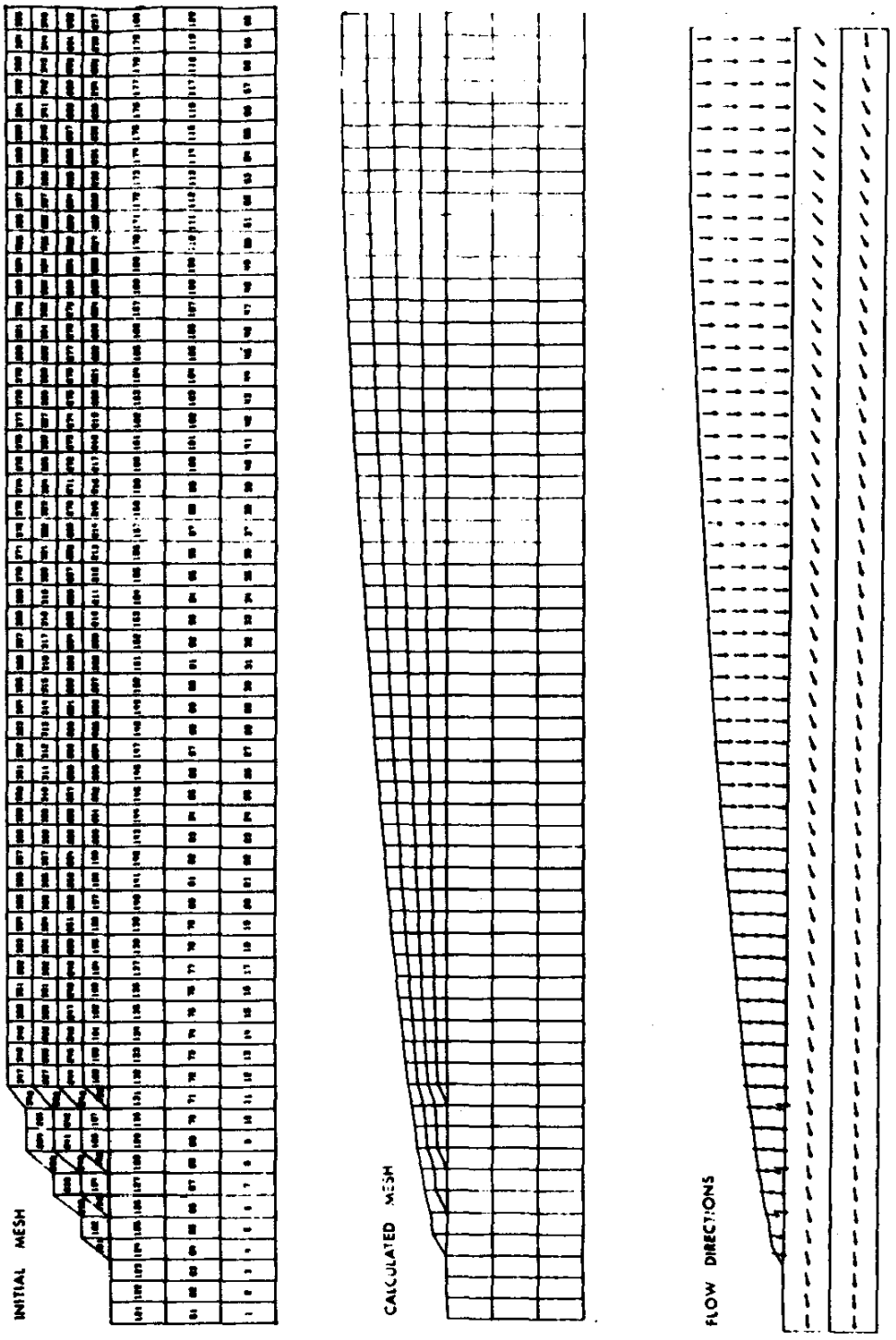


FIGURE 8. COMPUTER GENERATED PLOTS OF THE INITIAL MESH, CALCULATED MESH AND FLOW DIRECTION FROM THE FINITE ELEMENT MODEL OF THE PAGE TAILINGS PILE

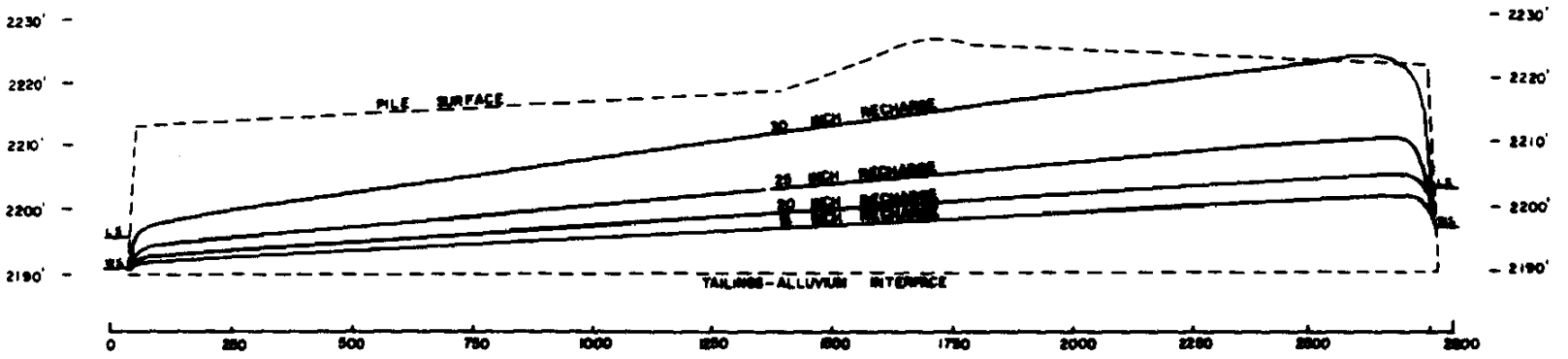


Figure 9 WATER TABLE CONFIGURATION RESULTING FROM DIFFERENT RECHARGE INPUTS TO TAILINGS PILE

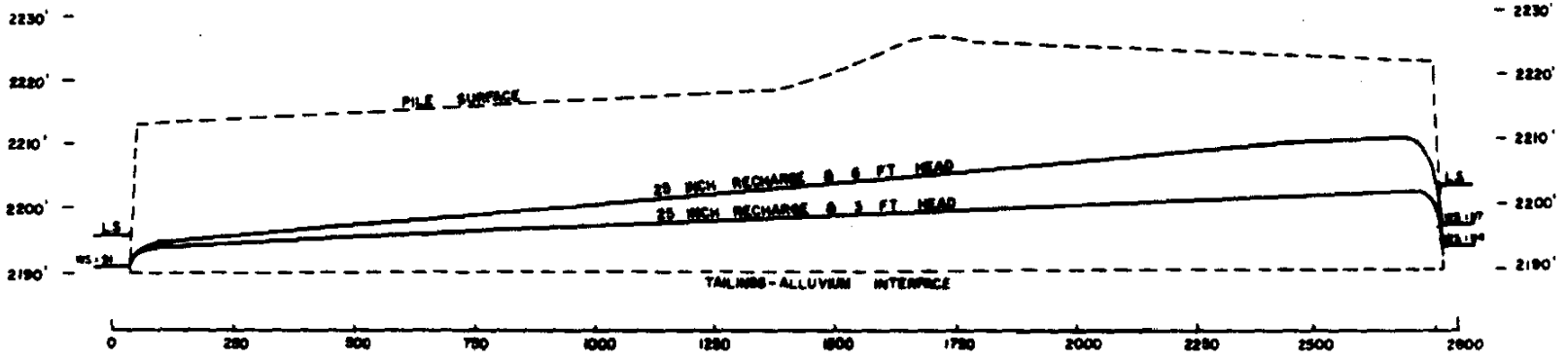


Figure 10 CONFIGURATIONS OF WATER TABLE INSIDE TAILINGS PILE FOR 25 INCH RECHARGE AND 6 AND 3 FEET OF HEAD PRESSURE DIFFERENTIALS ALONG BOTTOM OF PILE

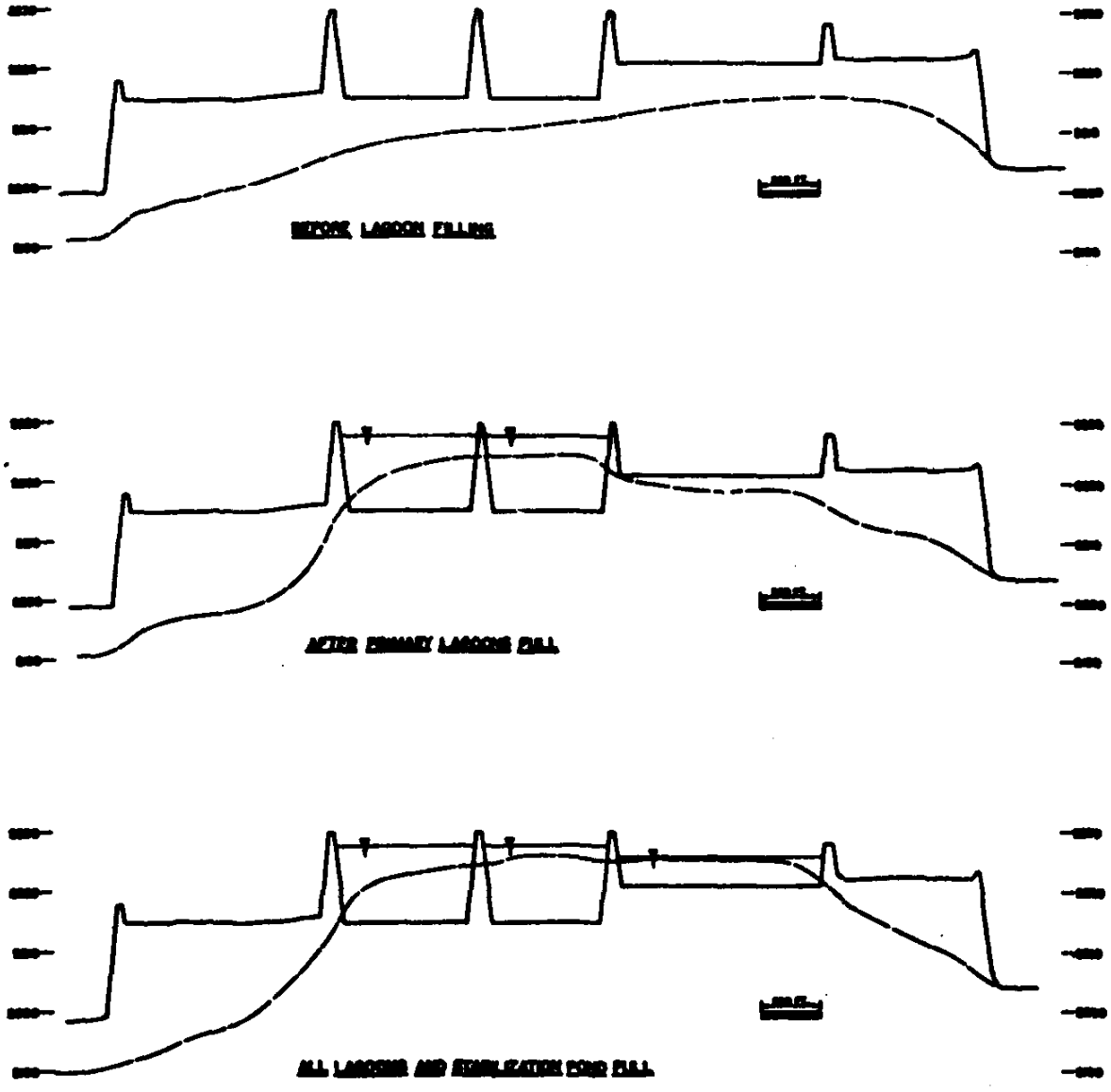
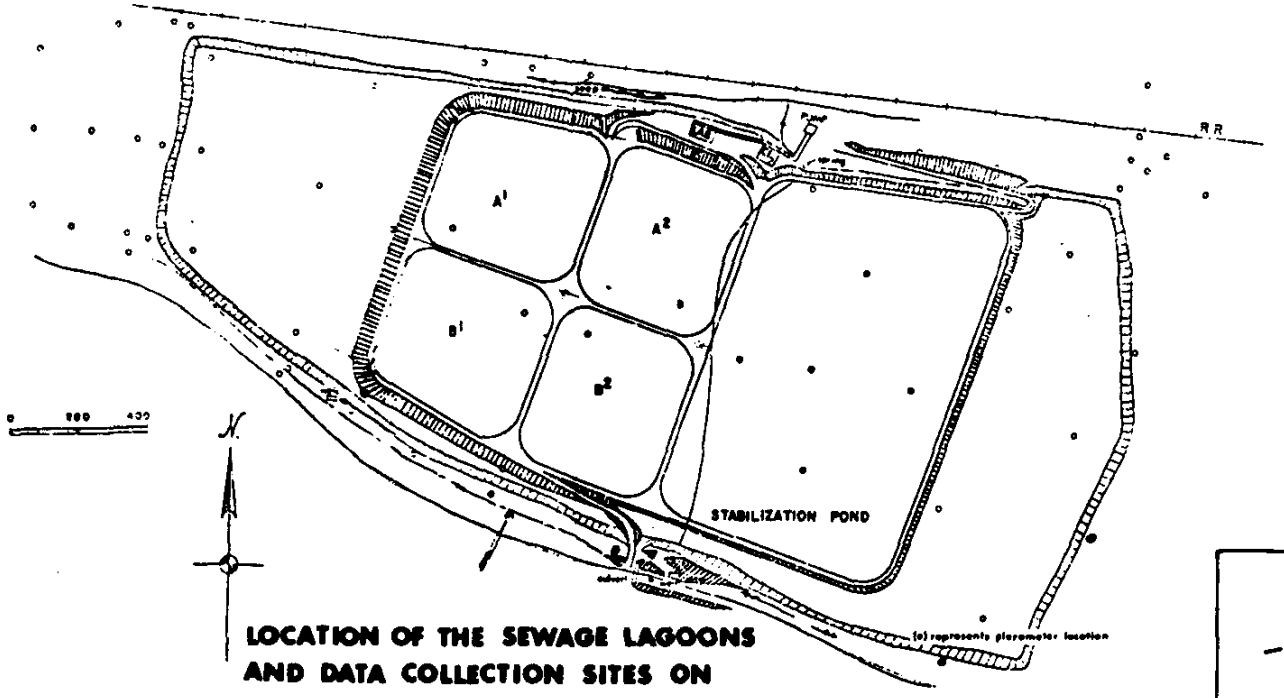


FIG. 11 CROSS-SECTION SHOWING LOCATION OF POTENTIAL LEVEL BEFORE, DURING, AND AFTER LAGOON FILLING



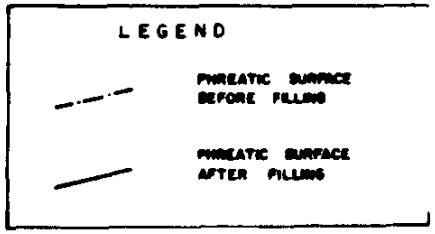


Figure 12 CROSS-SECTION SHOWING DISTRIBUTION OF PHREATIC SURFACE BEFORE AND AFTER LAGOON FILLING AS DETERMINED BY FINITE ELEMENT MODEL.

CONCLUSIONS

1. A sampling and testing program of the tailings pile showed that there are not distinct differences in the hydraulic conductivity of the tailings from various parts of the pile. The tailings pile may be treated as a homogeneous system.

2. Data from the mathematical model support the concept of the existence of a dynamic flow system in the pile.

3. Operation of the model showed that the location of the regional ground water table was an additional factor controlling the existence of the water mounded under the eastern part of the pile.

4. The finite element program allows calculation of the potential leakage from the sewage lagoon system to the tailings pile. This leakage was computed to be about seven percent of the total daily sewage input to the system.

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UNDERGROUND MINE DRAINAGE QUANTITY AND QUALITY GENERATION MODEL

Ricca, V. and Hemmerich, M.

ABSTRACT : This computer model is capable of simulating underground mine make-water ant its consequent discharge rates from adits. An additional feature is its ability to generate acid loads associated with the drainage. A hydrologic model using climatological data, watershed parameters, and mine operation information is used to calculate the amount of water passing to the geologic strata of the mine. As the water movement through the mine works is modeled, the acid generated is simulated by mathematical formulations describing the chemical productions and removal mechanisms. The component contributions are summed, with time preservation, and expressed as discharge rates and loads. The model is presented as a case study application to a coal mine in the U. S. A.

RESUME : Ce modèle est capable de simuler l'eau souterraine produite dans une mine et les vitesses des débits qui en résultent, en fonction des ouvertures des mines. Un aspect spécial de ce modèle est sa capacité pour générer les charges acides associées à ce drainage. Le modèle hydraulique utilise les données météorologiques, les paramètres de l'aquifère ainsi que les informations sur l'opération de la mine pour calculer la quantité d'eau qui s'écoule à travers les couches géologiques des mines. Pendant que le mouvement d'eau à travers la mine est modelé, l'acide générée est simulé par une expression mathématique qui exprime la production des acides et les moyens de s'en débarrasser. Les éléments qui contribuent sont ajoutés avec préservation du temps et exprimés comme vitesses des débits et charges. Ce modèle est présenté comme cas d'étude d'une mine dans la province d'Ohio aux Etats-Unis.

RESUMEN : Este modelo es capaz de simular el agua subterránea producida en una mina, y los caudales fluentes en función de la apertura de los huecos mineros. Una faceta adicional del modelo es su posibilidad de generar las concentraciones de ácido asociadas al drenaje. El modelo utiliza datos climáticos, parámetros del acuífero e información de las operaciones mineras, para calcular la cantidad de agua que fluye por las formaciones geológicas de la mina. Al mismo tiempo se simula la producción de ácido con formulaciones matemáticas que describen la producción y los mecanismos de remoción de sustancias químicas. Los diferentes componentes se suman, teniendo en cuenta el tiempo de contacto, y se expresan en forma de caudales y presiones. Se presenta una aplicación práctica a una mina de carbón en Ohio (U. S. A.).

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INTRODUCTION

Over the past decade researchers at The Ohio State University have developed high speed computer based mine drainage models. The models are designed to simulate the discharge flow rates and attendant acid loads emanating from coal mining operations. There are three models (underground, surface, and refuse pile) available and, although they are concerned with coal extraction and its associated pollution problems, they are basic enough to be altered to handle other mining operations. The underground or deep coal mine model is the topic of this paper.

The deep mine model is a hybrid of models for the hydrologic behavior of a watershed and for the generation of acid mine water. The basic operation of the model traces the disposition of water as it progresses through the hydrologic cycle with due concern for subsurface movements. The hydrology of the basin is simulated by a modified version of the Stanford Watershed Model. Climatological data, watershed and mine operation parameters are used to calculate the amount of water passing through the geologic strata associated with the mine complex. Through appropriate time delays controlled by the percolation of the makewater, quantities of water are transferred and stored in the aquifers bounding the active and/or abandoned mine. Subroutine hydraulics models then route the water into and through the mine complex, with attention given to travel lag times, to identified collection points for discharge through the adits.

If the water traversing the mine is subject to a pollutant loading, as is a common case in the coal mines of the Eastern U.S., it can be simulated by the acid generation portion of the model. The events of acid production and its removal are determined through the computer processing of their mathematical formulations applied to a prescribed micro-volume scheme representing the mine. The total minewater flow rate and quality is calculated by summing, with time preservation, the generation events in the micro-volume.

The quantities of water in any of the system components (surface runoff, infiltration, interflow, percolation, groundwater, mine aquifer storage, etc.) may be obtained via model output options. Standard outputs include flow rates in the receiving stream or watershed streamflow and mine adit discharges with their pollutant acid load rates, which occur in response to hourly precipitation input. Output may be tabular and/or graphical. The model continuously simulates the process and the more variable outputs, such as

streamflow, may be optioned down to one-minute time frames; however, the normal outputs are as average daily values.

Several reports and papers have been published that adequately describe in detail the technical aspects of the model's conception, structure and operation. Two comprehensive project reports, Shumate, 1976 and Ricca, 1978, presenting the contract research performed for the U.S. Environmental Protection Agency are recommended reading for a thorough understanding and operating the models. These reports contain an extensive bibliography as well as the computer program details and user manuals.

In this paper the concepts of the model will be indicated by a brief treatment of their technical aspects. The nature of the model and its utility will be stressed and exemplified by its application to a deep coal mine complex located in West Virginia, U.S.A. The particular mine site was chosen by the U.S. Environmental Protection Agency's Extraction Technology Branch for an intensive monitoring program on drainage pollution. It consequently affords one of the more complete compilations of data needed to develop and validate mine drainage models. All data sources peculiar to the U.S.A. will be de-emphasized and general statements as to their nature will be used instead.

HYDROLOGIC MODEL

The Ohio State University version of the Stanford Watershed Model is a high-speed, digital computer model which provides a versatile, reliable tool capable of simulating the hydrologic behavior of a basin. This is accomplished through the integrated use of mathematical statements describing the hydrologic activities which occur within the hydrologic cycle. The model is programmed to work toward a complete balance between the volume of water entering the basin and the amount of water leaving the basin plus the water remaining as storage. This balance, which is computed during each water year and displayed at the end of that year, uses precipitation and initial soil moisture conditions as the input, and generates transpiration, evaporation, overland flow, interflow and groundwater flow as the output. During the modeling process, a continuous account is kept of the amount of moisture in all the activities of the hydrologic cycle. Figure 1 shows a schematic diagram of the moisture accounting process in the Stanford Watershed Model (SWM).

Data requirements for the SWM consist of a variety of inputs involving: measurable and physical watershed parameters, trial and adjustment factors, selected or assigned values; along with basic recorded data on precipitation, pan evaporation, and daily streamflow. These parameters are listed in Table 1 in their assigned Fortran computer language names along with a brief description, units, and sample value associated with the case study application discussed later. There are also input-output control options available which may be used to improve, extend or analyze a simulation effort.

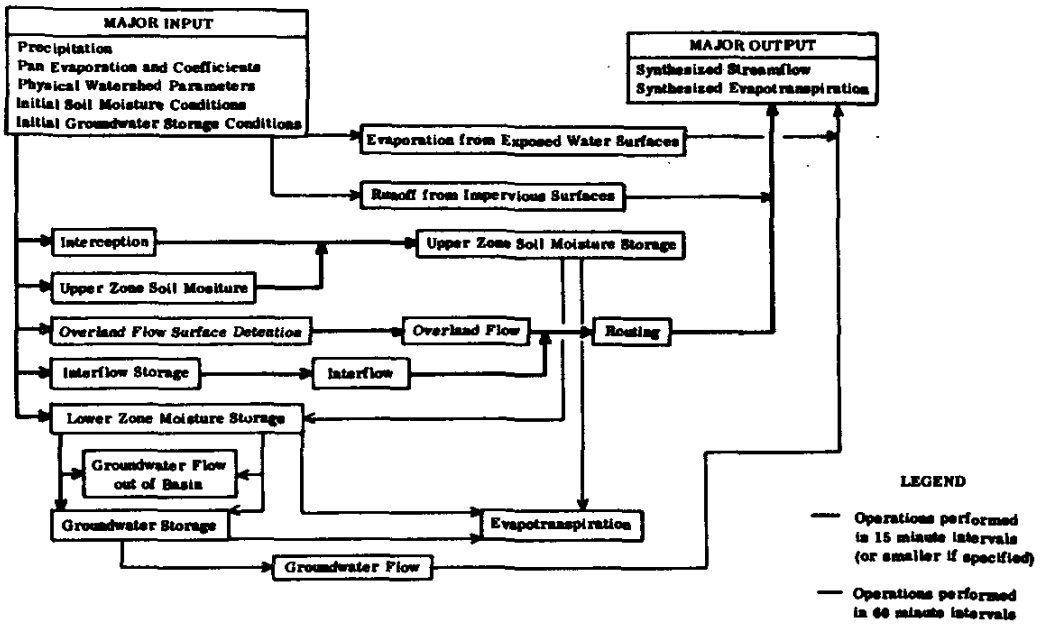

Figure 1 - Moisture Accounting in the Stanford Watershed Model

Of the two basic types of climatological data required by the SWM, precipitation is the more important and is usually more easily obtainable than evaporation because of the larger number of stations which record either hourly or storage gage daily values. Oftentimes precipitation data is not taken within the watershed, necessitating the use of records from stations outside the watershed. The decision on whether outside data may be used without modification depends on the proximity of the station to the watershed, how well the records reflect regional precipitation trends and how they compare with any incomplete records that may exist within the watershed. If unmodified records are inadequate or unrepresentative, then precipitation synthesizing techniques are employed.

The selection of evaporation data from an outside station is less critical than the choice of precipitation data due to the fact that daily evaporation varies to a lesser degree over a regional area. However, the fewer number of stations which record daily pan evaporation, combined with the suspension of daily recordings by many stations during the winter months, make adequate evaporation data difficult and sometimes impossible to obtain. This may be remedied in many instances by using records from outside stations and/or available local climatological data such as daily solar radiation, wind movement, dew point temperature, relative humidity and air vapor pressure to synthesize evaporation data.

Another type of climatological data is a grouping used as input to an optional subroutine that calculates snowmelt. The use of this subroutine improves the timing of runoff during winter and early spring; however, the amount of data needed to operate it is extensive and in most instances very difficult to locate or collect. Therefore, the subroutine is rarely used unless snowfall contributes a significant percentage to the total annual precipitation.

Table 1. Hydrologic Model (SWM) Input Parameters

MODEL PARAMETERS	PARAMETER DEFINITION AND UNITS*	CASE STUDY** VALUE
	Measurable Parameters	
TCONC	Time of concentration of watershed, min.	540.
TINC	Basin routing interval, min.	60.
A	Impervious fraction of watershed surface	0.0
AREA	Watershed drainage area, sq. mi. [$(km)^2$]	29.2
CHCAP	Index capacity of existing channel, cfs (m^3/s)	2000.0
ETL	Fraction of watershed in stream surface	0.002
IRC	Daily interflow recession constant	0.90
KK24	Daily baseflow recession constant	0.95
KSC	Streamflow routing parameter for low flows	0.85
KSF	Streamflow routing parameter for flood flows	0.934
L	Mean overland flow path length, ft. (m)	1900.0
SS	Average ground slope within watersheds, ft/ft. (m/m)	0.170
	Trial and Adjustment Parameters	
CB	Index controlling the rate of infiltration	0.50
CS	Index for estimating soil surface moisture storage	0.40
CY	Index for time distribution of moisture entering interflow	5.00
EDF	Index for estimating soil surface moisture storage capacity	1.00
EF	Seasonal factor adjusting infiltration and evaporation rates	1.00
EMIN	Minimum value of factor varying seasonal infiltration	0.50
GWS	Current value of groundwater slope index, in. (m)	0.10
LZS	Current soils moisture storage, in. (m)	2.00
LZSN	Soil moisture storage index, in. (m)	3.00
SGW	Groundwater storage increment, in. (m)	0.10
ATFLO	Parameter controlling adjustment of infiltration	10.0
ATCFS	Parameter controlling adjustment of infiltration	0.010
ATDR	Parameter controlling adjustment of infiltration	1.50
ATC2L	Parameter range adjustment of infiltration	0.250
ATC2U	Parameter range adjustment of infiltration	5.00
	Assigned or Selected Parameters	
EPXM	Maximum interception rate for a dry watershed, in./hr. (m/h)	0.18
K3	Soil evaporation parameter	0.20
K24E1	Groundwater evaporation parameter	0.80
K24L	Index for groundwater flow leaving the basin	0.20
KV24	Daily baseflow recession adjustment factor	1.00
NN	Manning's n for overland flow on soil surface	0.40
NNU	Manning's n for overland flow on impervious surface	0.012
RFC	Index for routing	6.00
UZS	Current soil surface moisture storage, in. (m)	0.0
VOLUME	Swamp storage and dry ground recharge, ac-ft. (m^3)	0.0
ETCORR	Adjustment factor for off-site evaporation data	1.0

Note: *The model is designed to operate with English units. SI units may be used with a data conversion program. The output units may be optioned in either system.
**English units values used for the Roaring Creek Study Watershed, West Virginia, U.S.A.

Physical data on the watershed concerning drainage areas, lakes, overland and stream flow characteristics, vegetative cover, etc. can be obtained from maps and aerial photographs. Soil moisture parameters are best evaluated with the aid of soil borings, well logs or a local geologic profile. Also a knowledge of the soil types and their moisture associated behavior is helpful.

Adequate streamflow data is essential to calibrate the hydrologic model. A minimum of three years of continuous average daily data is needed. Several isolated storm hydrographs are utilized to establish routing parameters and recession coefficients. Groundwater parameters describing percolation, water table fluctuations and slopes, and interbasin transfers, are determined through the aid of well records and boring logs.

Detailed instructions on how to acquire the requisite data, evaluate its suitability, and synthesize missing records are presented in the report and user manuals by Ricca, 1978.

The main role of the hydrologic model is to generate the quantities of moisture which will eventually percolate to supply the aquifers surrounding the underground mine workings. The daily amounts of percolation water are outputted as punched cards which in turn become input for the acid mine drainage model that follows.

DEEP MINE MODEL

The Deep Mine Model development began when the underground pyritic system was conceptualized by Smith, 1971 and has progressed to its current status as reported herein. The basic operation of the model consists of three steps. First, the model divides the deep mine system into micro-volumes. Then the events occurring in each micro-volume, involving oxidation and the mine's product removal mechanisms, are determined through the use of mathematical equations formulated to describe them. Finally, the total mine behavior is calculated as the sum of the events in each micro-volume. The schematic of Figure 2 offers a slightly more detailed description of the model operation.

The previous described hydrologic model (SWM) provides information to compute: water quantities involved in the mine drainage, the extent of inundation in the workings, and the generation and removal of pollutants, acid in this case, in the system. This information is linked to the acid production scheme and the combination produces a complete modeling system called the Deep Mine Model. The major point of linkage of the two is associated with subsurface flow and water storage in the region of the mine. The SWM calculates the amount of water infiltrating from the watershed above in 15-minute intervals. These values are processed through the subsurface and summed to yield the total daily moisture infiltrating to the groundwater, which is then transferred to the Deep Mine Model and used to determine daily aquifer moisture storage and daily mine-water flow. Once the amount of water flowing through the deep mine is defined, then the acid loads removed by leaching, inundation and gravity diffusion can be obtained. Program output consists of daily infiltration water reaching the

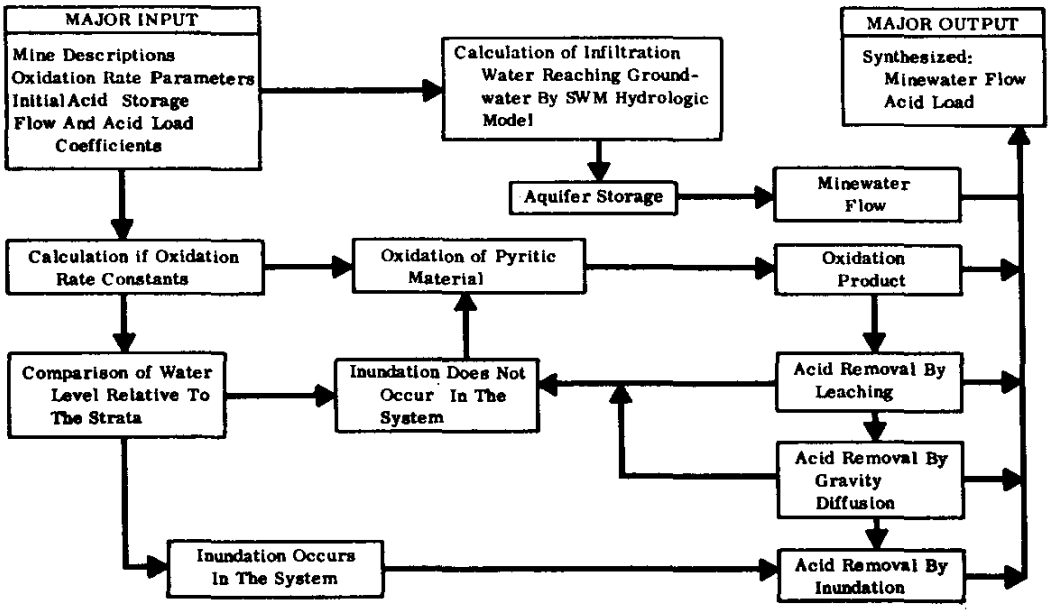

Figure 2 - Schematic of Acid Mine Drainage Model

mine aquifer, daily minewater discharge and acid load associated with each of the three removal mechanisms, and the total daily minewater discharge and acid load removal from the deep mine.

The potential complexity of a mining system and the use of elaborate models to simulate the mine flow and pollutant generation loads necessitate rather demanding data requirements or input information. This information should reflect the physical, climatologic, geologic and hydrologic characteristics of the watershed and should be representative of the coal mining system and its acid generation characteristics. Acid mine drainage sources have three basic characteristics to be simulated: the physical features of the pollutant source, the rate of pyrite oxidation in the source system, and the transport of acid products from the reactive sites by the mine drainage. The Deep Mine Model requires input information on: mine system descriptors, pyrite oxidation kinetics, pollutant products removal mechanism, and mine discharge quantity and quality records. Table 2 lists the input parameters by their Fortran Computer language names with a brief description, units, and a sample value used in the case study application presented later. In addition to these listed variables, there are parameters used to control the program and level of output generated.

Mine description data include information on areal extent and age of mine workings, surface watershed area, location of adits, mineral seam thickness and lengths, pyritic seams, and stratigraphy above the mine. Much of these data can be compiled from topographic and geologic maps, mine maps and operation records, and boring logs.

The pyrite oxidation data is of two types: those related to acid production characteristics of the strata, and those describing the mine conditions under which pyrite oxidation occurs. Values are needed for the chemical reaction

Table 2. Deep Mine Model Input Parameters

MODEL PARAMETERS	PARAMETER DEFINITION AND UNITS*	CASE STUDY** VALUE
	Deep Mine Physical Parameters	
IMINE	Number of mines involved	1
NFEET	Number of air-solid interface increments	1
NLAYER	Number of layers in the coal seam	20
NDEPTH	Number of depth increments in the model	15
DK	Length of depth increments, ft.(m)	1.0
DI	Length of air-solid interface increments, ft.(m)	1.473×10^6
ROCK TYPE	Liberal strata descriptors	COAL
TOP	Datum plane for top of coal seam, ft.(m)	98.57
ALT	Elevation of stratum relative to datum, ft.(m)	90.00-98.24
	Pyrite Oxidation Parameters	
REACT	Oxygen consumption rate of pyrite	2.55, 0.55
PYCON	Void fraction of the stratum	0.30, 0.005
STORE	Oxidation product storage array, lbs/ft^3(kg/m^3)	7.24, 1.95
TEMP	Mine temperature correction factor, none	0.15
FTGMOL	Volume occupied by one gram-mole of gas, ft^3(m^3)	0.79
CCPRFT	Constant to calculate pyrite reaction rate constant	28287.26
P	Mine pressure, psi (N/m^3)	454.0
GASC	Gas concentration under mine conditions	0.0856
DIFF	Gas diffusivity, ft^2/day(m^2/d)	99.965
	Oxidation Product Removal Parameters	
WSHED	Watershed area of the mine, sq. miles (km^2)	5.538
TANK	Mine aquifer storage, inches (cm)	0.158
CONFH	Constant relating minewater flow to aquifer storage	0.039
CONM, CONB	Constants in the relationship between head and storage	4.95, -2.013
FLOWMI	Min. flow for acid removal by flooding, gpd(ℓ/d)	2.268×10^6
HEADMI	Min. head needed for acid removal by leaching, in.(cm)	0.045
PER	Parameters for distance into coal strata that is inundated	1.50
WSLOPE	Hypothetical slope of the water level	0.10
COND	Constant in acid products removed by leaching	0.50
FRACT	Fraction of stored products removed by inundation	0.15
SOX	Initial total acid storage, lbs.(kg)	0.0
DIF	Base gravity diffusion constant	0.001
DTHETA	Time increment length, days	1.0

Note: *The model is designed to operate with English units. SI units may be used with a data conversion program. The output units may be optioned in either system.
**English unit values used for Roaring Creek Study Watershed, West Virginia, U.S.A.

rates, diffusivity and concentration of oxygen through the stratum, and temperature and pressure at the coal seam faces. Oftentimes many of these data are very difficult to obtain because of no prior evaluation of the coal seam. Two alternatives to ascertain the information arise; either collect samples of the coal and surrounding strata and perform a laboratory analysis for the values, or select intial values for the parameters and, by trial and error substitution, find values which provide the most suitable acid load simulation results.

Parameters associated with the pollutant product removal data fall into two categories: those in the minewater generation process and those that aid in the description of the removal mechanism. The major data of this group are: the daily water infiltrating and percolating to the mine aquifer (these values are generated by the hydrologic model), and the factors controlling the hydraulics governing the transfer of water from storage in the surrounding aquifers into the mine workings.

Minewater quantity and quality data is collected to provide some means of analyzing the success of the model simulations and to aid in determining a few of the input parameters. The more accurately and frequently the flows at the adits are monitored the more successful will be the application of the model.

Explicit instructions on data acquisition for the Deep Mine Model are available in the report and user manuals by Ricca, 1978.

CASE STUDY APPLICATION

The Hydrologic Model has been successfully applied to several watersheds in the Eastern coal fields of the U.S. The Deep Mine Model has had only a limited application due to the lack of sufficient hydrologic and mine drainage data being collected simultaneously. The only test site that the authors could find that had reasonably complete simultaneous data sets for the model pair was at Elkins, West Virginia. This site was under study by the U.S. Environmental Protection Agency from 1964 to 1969. One aspect of their investigation was to monitor the site climatology and drainage from the deep and strip mines within the Roaring Creek and Grassy Run Watersheds. Thus, partial records of minewater quantity and quality were available, as well as much of the other pertinent information on the physical and geological aspects of the area, and the nature and extent of the coal mining activities. Although not all the input data needed for modeling was collected either completely or consistently, enough information was compiled, by synthesizing missing pieces, to apply the models. The following describes the experiences of this application.

Location

The Roaring Creek watershed is located in east-central West Virginia, within the Eastern coal fields of the United States. It covers 75.6 square kilometers (29.2 square miles) and is drained by Roaring Creek, a stream within the drainage system of the Ohio River basin. Figure 3 shows the location of the study area in West Virginia and the continental United States, and also depicts

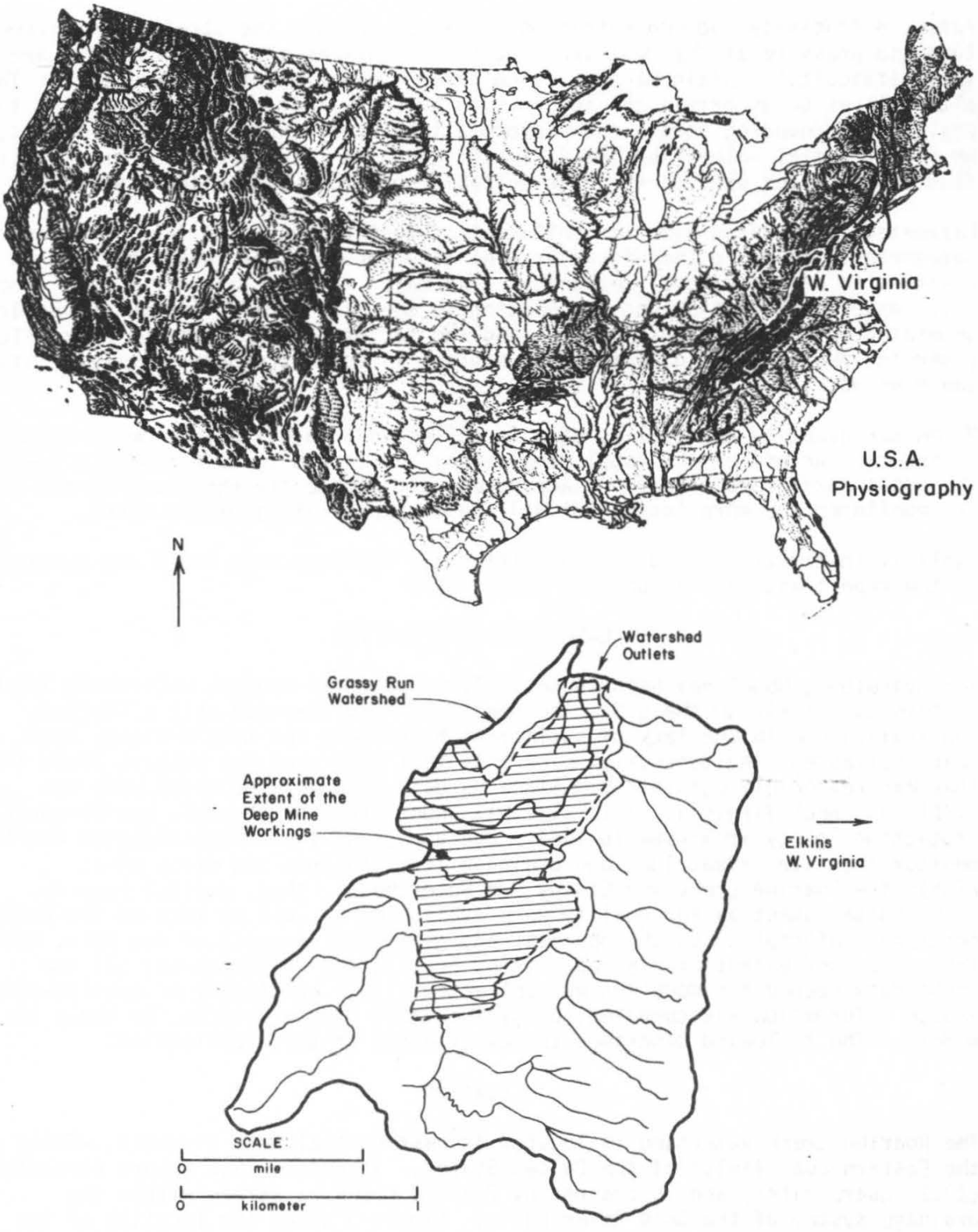

Figure 3 - Roaring Creek Watershed, Elkins, West Virginia

the Roaring Creek watershed. The topography of the area ranges from rolling foothills in the west to mountainous terrain in the east. The ground cover is predominantly hardwood forest, with a few clearings along Roaring Creek and at higher elevations.

Climate

The climate of the area is of the continental mountainous type, with cold winters and mild summers. Average yearly precipitation is 1.186 meters (45.92 inches). The vast majority of this precipitation comes from intense thunderstorms occurring during the summer months or large-scale, low-intensity, cyconic-type storms that occur in the early spring. Snowfall within the area is of moderate significance, with an average of about 1.19 meters (47 inches) per year, usually deposited during the period from November to April.

Geology

The Roaring Creek watershed is composed of two distinct physiographic areas which reflect the respective weathering characteristics and structures of the underlying rocks. The western two-thirds of the watershed consist of gently dipping beds of moderately resistive shales and sandstones which form broad, flat uplands separated by narrow V-shaped valleys carved by tributaries of Roaring Creek. The topography of the eastern one-third of the watershed is dominated by mountainous cliffs and flatiron-like ridges carved from moderately dipping sandstone and conglomerate sandstone. A syncline separates the two physiologic regions, and influenced the establishment of the main direction of drainage for the watershed.

Coal and Mining Activities

Three economically important coal seams exist within the Roaring Creek watershed; the Kittanning seam is the most extensively mined. Removal of this coal seam produced a massive underground mining complex of approximately 1200 hectares (3000 acres) which extends into the adjacent Grassy Run watershed. In addition, surface mining of the seam outcrop surrounding the deep mine complex resulted in more than 400 hectares (1000 acres) of additional disturbed land in the watershed.

Hydrologic Model Application

The Hydrologic Model (SWM) is first applied to generate input for the Deep Mine Model. After it is calibrated and is deemed to be successfully simulating the watershed hydrologic behavior, specific information on the water percolating to the aquifers surrounding the mine complex is generated as punched card output. Also, having established the watershed hydrographs permits one to see the impacts of the mine drainage pollutants on the receiving streams.

Climatological Data

Precipitation data for the Roaring Creek watershed was obtained through the use of statistical analysis techniques which combined the hourly precipitation records existing at two weather stations located outside the watershed with existing hourly and daily data taken in the watershed. Evaporation data was acquired through a combination of recorded and synthesized information.

Streamflow Data

Average daily streamflow data for Roaring Creek was taken from the United States Geological Survey (USGS) publication of Water Resources Data for West Virginia. The Environmental Protection Agency provided detailed hydrograph data needed to determine several streamflow routing and recession parameters required by the SWM.

SWM Inputs

The major SWM input parameters evaluated for the Roaring Creek watershed are listed in Table 1. Determination and/or adjustment of values for most of the parameters followed guidelines and procedures recommended by Ricca (1978). Special computational efforts were required in the case of K24L, the fraction of groundwater lost via inter-basin transfer. Due to the orientation of the mined coal seam and the extensive nature of the deep mine complex, some of the minewater originating in the Roaring Creek basin discharged into the adjacent Grassy Run watershed. Analysis of this moisture transfer involved the use of recorded minewater discharge data within the Grassy Run watershed and mine maps depicting the areal layout of the deep mine complex.

After the initial values of the input parameters were determined, the data was assembled for use. To enhance the utility of the model and reduce the cost of operation, the SWM source deck was compiled on disk and all evaporation, precipitation, and recorded streamflow data were transferred onto 9-track magnetic tape. These measures reduced the amount of input data on cards to essentially those parameters shown on Table 1, plus required program and input/output control data.

Hydrologic Modeling Experience

Analysis of the results of the initial run of the SWM for the Roaring Creek watershed indicated a severe undersynthesis of streamflow in the winter months followed by an equally harsh oversynthesis in the summer months. Also, a net increase in soil moisture storage occurred in each of the four water years being modeled. In order to improve these situations, different values of several trial and adjustment parameters were tried following the sensitivity guidelines. Parameter changes progressed until satisfactory simulation results were obtained.

Hydrologic Modeling Results

The simulation results for water-year 1968-69 represent the average success achieved during the hydrologic modeling effort. Results were generated in both tabular and graphical form. Selected output presented in this paper include plots of the recorded and simulated streamflow hydrographs for Roaring Creek (Figure 4), and the daily infiltration water reaching the groundwater (Table 3), which is the key hydrologic input data for the Deep Mine Model. Additional output includes tables of recorded and simulated daily streamflows; monthly and yearly summaries of streamflow, precipitation, and evaporation; end-of-the-month values for certain key parameters; the balance of all water in the basin throughout the water year; the daily correlation coefficient; and the daily soil moisture status.

The results of the hydrologic modeling effort indicated that the overall seasonal patterns of the recorded stream hydrograph were reasonably well simulated; however, only moderate success was met in reproducing individual streamflows. In general, monthly streamflow volumes indicated a moderate undersynthesis of streamflow occurring in the spring, and a definite oversynthesis in the summer and early autumn. From mid-autumn through the winter, an oversynthesis of streamflow volumes occurred, with most of the excess contained in peaks. Also, some simulated peaks in this period were out of phase with their recorded counterparts. The spring undersynthesis may be associated with annual spring thaw activities which cannot be completely modeled by the SWM. Possible explanations for the summer and fall oversynthesis are: the local rain storms which occur in the summer and fall seasons may not be properly represented by synthesized precipitation data; the evapotranspiration values used for this period are too small; and the actual soil moisture may be considerably lower than the modeled volume. In the winter, snowfall and its associated complications are the probable source of simulation difficulties. While the SWM is capable of handling this problem, the large amounts of data required to do so were not available. Lastly, the inability of the SWM to account for some of the total yearly moisture could be attributed to the interception of surface runoff by openings and fractures leading into the deep mine system, and the subsequent transfer of this moisture out of the watershed via underground routes not indicated on the old mine maps.

Deep Mine Model Application

A segment of the large deep mine complex extending beneath the Roaring Creek watershed and the adjacent Grassy Run basin served as the subject of modeling with the Deep Mine Model. This segment encompasses approximately one-third of the total complex area; it was selected after investigation of the EPA's minewater discharge monitoring network and the available deep mine maps which indicated that the majority of minewater emanating from this segment discharged into a tributary of Roaring Creek, and was not diverted through inter-basin transfer to Grassy Run. Figure 4 locates the underground mine in the watershed and Figure 5 shows the entire mine complex and the segment selected for modeling.

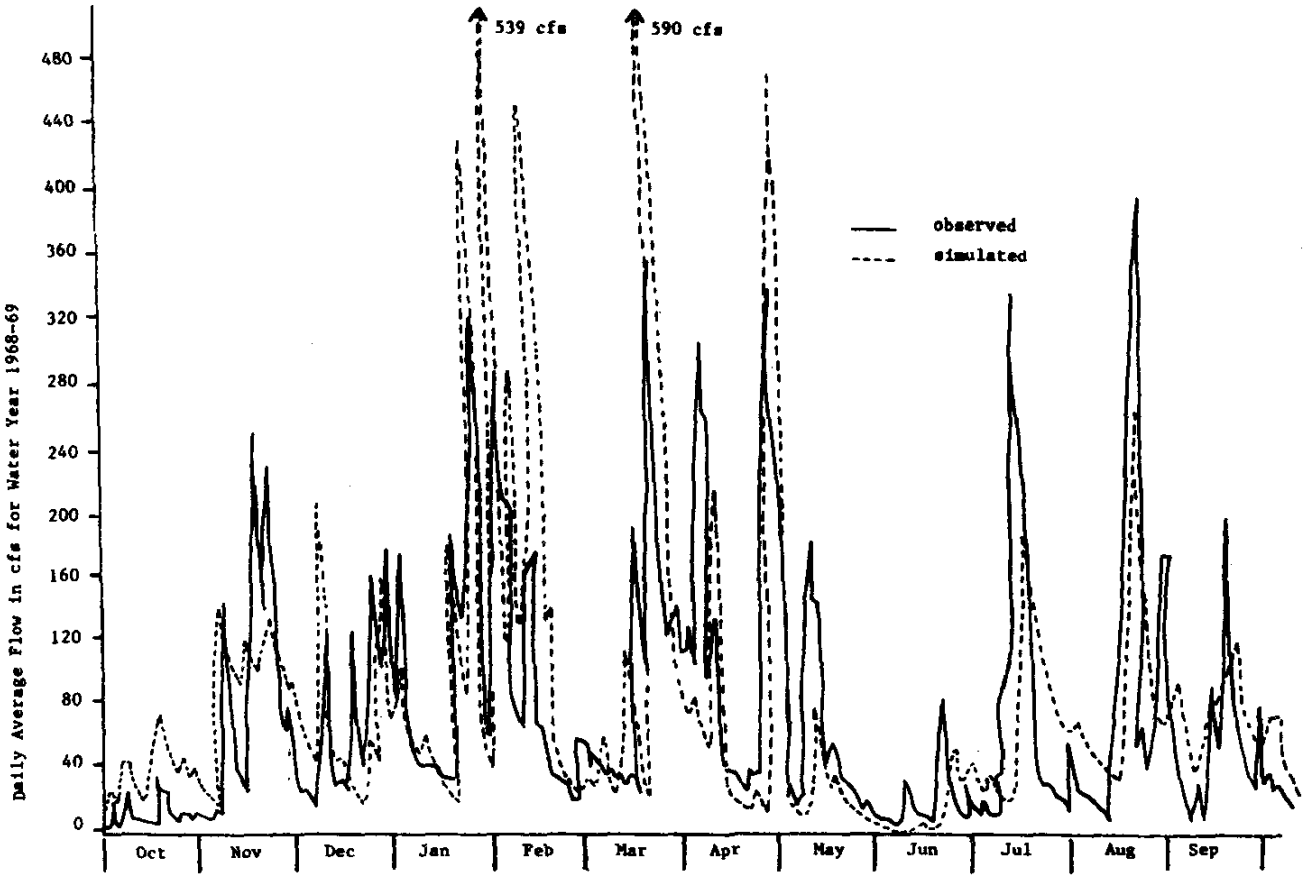

Figure 4 - Plot of Recorded and Simulated Streamflow

Table 3. Daily Infiltration Water Reaching the Groundwater, inches

DAY	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEPT
	DAILY INFILTRATION WATER REACHING GROUNDWATER											
1	0.0	0.000	0.000	0.032	0.026	0.011	0.001	0.0	0.0	0.0	0.0	0.0
2	0.0	0.000	0.000	0.023	0.010	0.006	0.000	0.0	0.0	0.0	0.0	0.0
3	0.0	0.000	0.013	0.016	0.018	0.003	0.001	0.0	0.0	0.0	0.0	0.0
4	0.0	0.016	0.155	0.010	0.024	0.002	0.001	0.0	0.0	0.0	0.0	0.0
5	0.0	0.000	0.001	0.008	0.013	0.001	0.001	0.0	0.0	0.0	0.0	0.0
6	0.122	0.106	0.001	0.007	0.008	0.000	0.016	0.0	0.0	0.0	0.0	0.205
7	0.007	0.244	0.000	0.005	0.014	0.000	0.009	0.0	0.0	0.0	0.0	0.000
8	0.000	0.046	0.000	0.013	0.013	0.0	0.004	0.0	0.0	0.0	0.0	0.193
9	0.000	0.004	0.0	0.242	0.033	0.010	0.001	0.0	0.0	0.011	0.0	0.081
10	0.000	0.002	0.0	0.019	0.022	0.000	0.000	0.0	0.0	0.022	0.197	0.003
11	0.0	0.001	0.113	0.012	0.039	0.000	0.000	0.0	0.0	0.029	0.021	0.002
12	0.0	0.000	0.000	0.009	0.018	0.012	0.001	0.0	0.0	0.057	0.014	0.001
13	0.0	0.0	0.000	0.007	0.010	0.037	0.000	0.0	0.0	0.147	0.008	0.000
14	0.0	0.251	0.000	0.003	0.007	0.009	0.000	0.0	0.0	0.008	0.004	0.000
15	0.0	0.001	0.000	0.001	0.004	0.010	0.0	0.0	0.013	0.003	0.001	0.000
16	0.0	0.000	0.000	0.001	0.002	0.001	0.0	0.0	0.009	0.001	0.060	0.0
17	0.0	0.040	0.000	0.000	0.001	0.000	0.0	0.0	0.000	0.000	0.002	0.0
18	0.076	0.100	0.039	0.016	0.000	0.000	0.0	0.0	0.000	0.0	0.024	0.0
19	0.089	0.002	0.038	0.032	0.000	0.0	0.0	0.0	0.0	0.0	0.190	0.0
20	0.000	0.001	0.002	0.006	0.000	0.0	0.0	0.0	0.0	0.0	0.395	0.171
21	0.000	0.000	0.001	0.024	0.000	0.0	0.0	0.0	0.0	0.0	0.028	0.232
22	0.000	0.000	0.023	0.006	0.000	0.0	0.001	0.0	0.0	0.0	0.015	0.001
23	0.0	0.000	0.002	0.004	0.001	0.0	0.012	0.0	0.0	0.0	0.007	0.000
24	0.0	0.039	0.001	0.003	0.004	0.0	0.006	0.0	0.0	0.0	0.003	0.000
25	0.027	0.001	0.001	0.002	0.018	0.013	0.000	0.0	0.0	0.0	0.001	0.034
26	0.004	0.000	0.003	0.002	0.008	0.007	0.000	0.0	0.0	0.0	0.000	0.000
27	0.014	0.000	0.035	0.001	0.011	0.005	0.0	0.0	0.0	0.0	0.000	0.000
28	0.027	0.000	0.057	0.000	0.030	0.001	0.0	0.0	0.0	0.0	0.0	0.012
29	0.000	0.0	0.022	0.028		0.000	0.0	0.0	0.0	0.0	0.0	0.001
30	0.000	0.0	0.014	0.019		0.003	0.0	0.0	0.0	0.0	0.0	0.000
31	0.000		0.013	0.015		0.000		0.0		0.0	0.0	

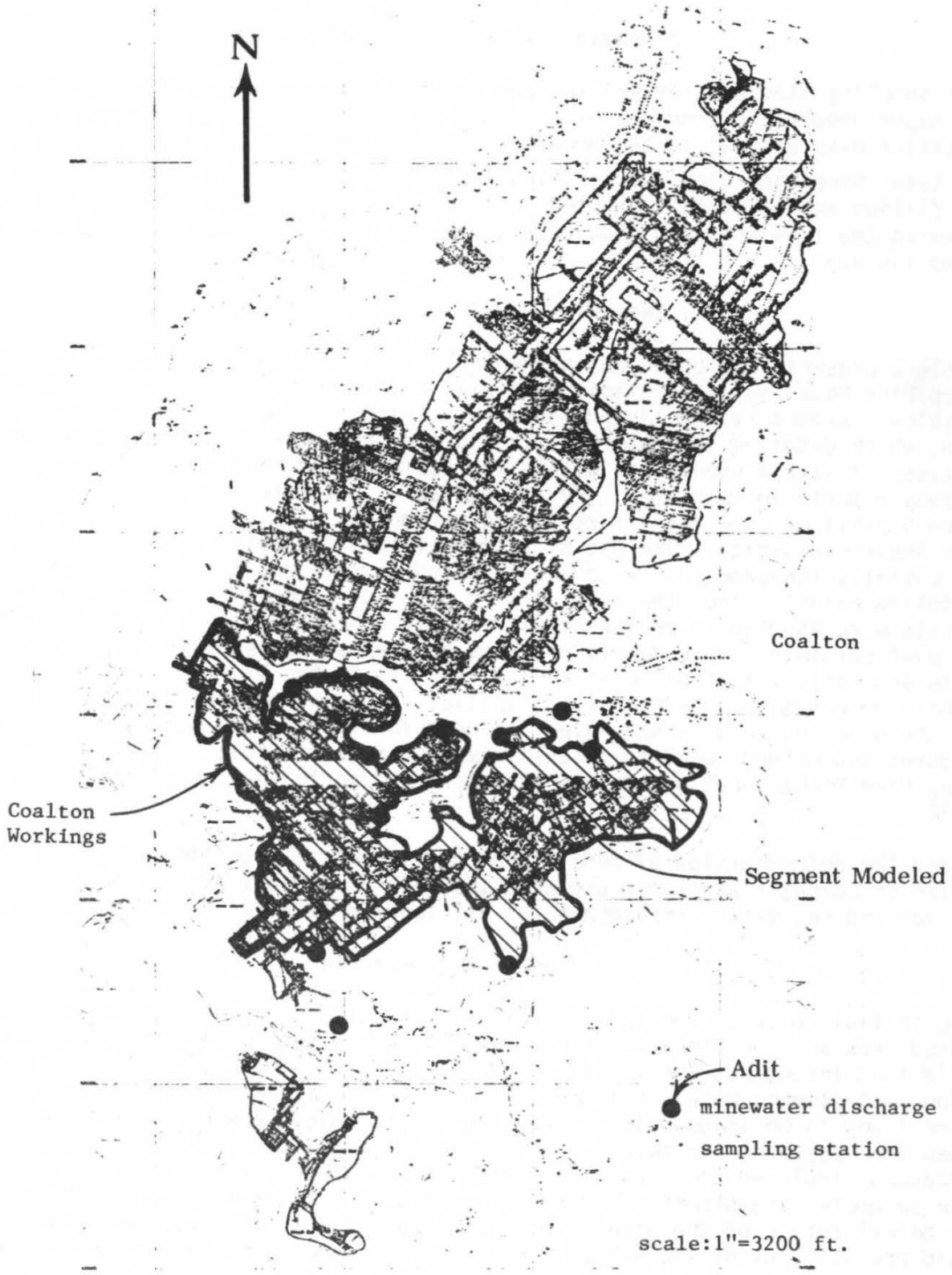

Figure 5 - Roaring Creek Deep Mine Complex

Minewater Quantity and Quality Data

Ten sampling stations, established by the EPA, monitored the minewater discharges of major proportion from the mine section of interest. These stations and their location with respect to the deep mine complex are shown in Figure 5. A record of total minewater flow and acid load data for the modeling area was obtained by filling any voids that existed in the individual station records, and then summing the flow and acid load data collected at each of the stations on each sampling day.

Deep Mine Model Input Parameters

Table 2 presents the principal parameter values used for the initial run of the Deep Mine Model in its application to the selected mine study area. Deep Mine physical parameters were evaluated with the aid of mine maps, furnished by the EPA, which detailed the internal structure of the mine, and documented observations of strata sub-layers within the unmined coal seam published in geologic survey reports for the area. Pyrite oxidation parameter values were taken from a previous modeling effort due to a complete lack of the information needed for their evaluation. The use of assumed values was based on the fact that essentially the same coal seam was involved in both earlier and present modeling efforts; thus the chemical and physical properties of the coal bearing strata were assumed to be reasonably similar. Several of the oxidation product removal parameters were determined through the use of recorded minewater flow data and daily values of moisture infiltrating to the mine aquifer. The remaining parameters were assigned initial values which were subject to change if the modeling results warranted the need for fine-tuning adjustments. Procedures and methods used to calculate and/or assign initial values to all the Deep Mine Model input parameters are described in further detail by Hemmerich, 1976.

After the determination of model input parameters, all required data was compiled on computer cards for use with the Deep Mine Model program deck. The format and sequential arrangement of all input data is presented by Ricca, 1978.

Modeling Experience

The initial computer run indicated poor correlation between recorded and simulated peak and low minewater flow occurrences and severe oversimulation of acid load throughout the entire modeled period. To improve acid load simulation, adjustments were first made to product removal parameters. When these were found to be inadequate, it was decided to adjust key pyrite oxidation and deep mine physical parameters. A reasonable reduction of these parameters produced simulated acid loads that were in the general range of recorded data. The parameter adjustments appear justified when considering the probable extent of deterioration of the mine's internal structure and the seasonal flushing of acid products out of the system during the more than half-century existence of the mines.

Further examination of the simulated minewater flow results leads to the identification of a time lag between simulated and recorded minewater discharges. It was theorized that the time lag was associated with the movement of moisture through the mine overburden and then through the mine system itself. Approximations of the moisture travel time through these two media were made using geologic field information and the results of minewater movement dye tests conducted by the EPA. Details of the computational techniques are discussed by Ricca, 1978; it is sufficient at this point to say that the total computed time lag was very close to the observed temporal displacement of simulated minewater flow events from the recorded data. After the justification of the time lag existence, a time delay feature applied to the infiltration to the mine aquifer and travel through the workings was incorporated into the Deep Mine Model.

Deep Mine Modeling Results

The simulation results for water year 1968-69 represent the better simulation success experienced during the modeled period. The total results of the modeling effort were generated in both tabular and graphical form. Output selected for presentation in this paper consists of the plots of recorded and simulated minewater flow and total acid load (Figures 6 and 7). Additional output includes tables of the daily infiltration water reaching the groundwater (as supplied by the SWM); the daily infiltration water corrected for a time delay; the average daily synthesized minewater flow and daily total synthesized acid load; the daily synthesized acid load removed by inundation, leaching, and gravity diffusion; and monthly and annual summaries of synthesized infiltration water, minewater flow, and acid load removal.

Examination of Figure 6 indicates that the simulated minewater flow follows the general trends established by the recorded data to a relatively adequate degree. Keep in mind that the simulated plot is on an average daily basis and the recorded flow spot points are instantaneous samples. Existing problems include the coordination of peak and low discharge periods and an undersimulation of total yearly flow volume. One possible explanation for these problems is the interception of surface runoff by the deep mine system. The Deep Mine Model in its present structure is unable to account for this additional moisture and the timing problems it creates. Another explanation involves a possible misrepresentation of natural minewater discharge conditions by combining the readings from all major effluent points into one lump sum discharge record. Because the Deep Mine Model can only simulate discharge from a single opening, unification of the flow records from each adit was necessary. However, this restriction in the model would tend to dampen out inherent timing differences between the effluent points, the result being that the recorded peaks of the cumulative record would be more drawn out, lower, and less sharp than the simulated, and less accentuated than the peaks recorded at each discharge point.

As in the case of minewater discharge, the simulated acid load follows the trends in recorded data reasonably well. However, total yearly acid removal was over simulated, and a few problems with the correlation of recorded and

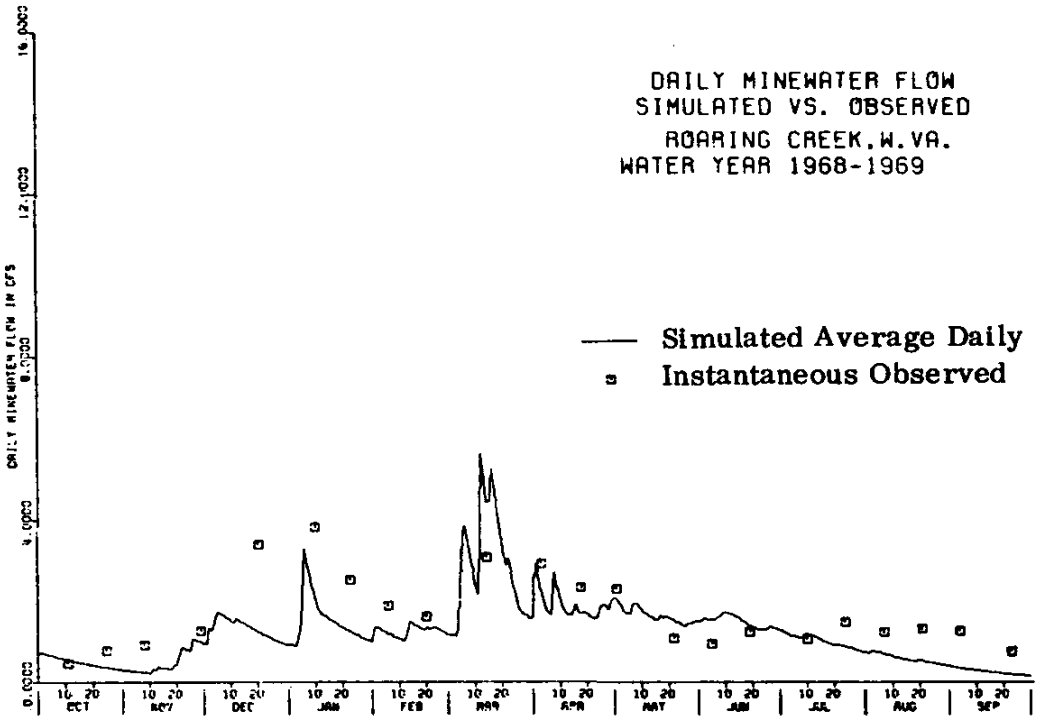

Figure 6 - Simulated and Recorded Minewater Flow Plots

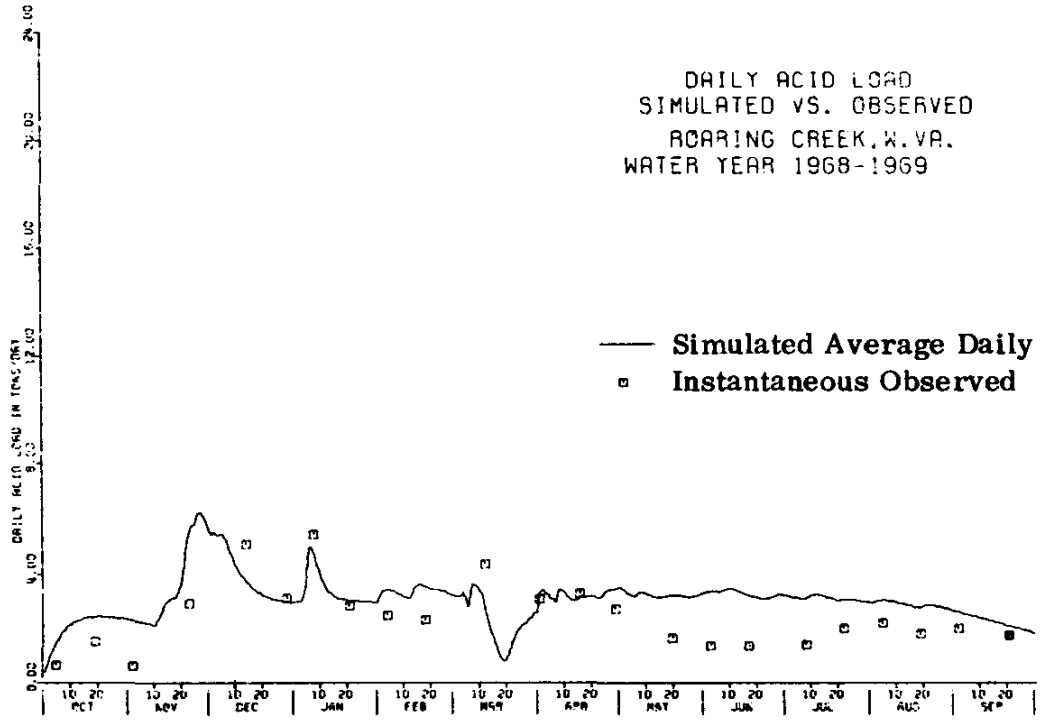

Figure 7 - Simulated and Recorded Acid Load Plots

simulated acid loads do exist. The tabular output indicated that leaching was the primary removal mechanism of acid products; since the main avenue of moisture entry into the mine is by infiltration through fractures in the overburden, the domination of leaching appears to be justified. Acid removal by inundation and gravity diffusion was relatively small throughout the entire water year. Causes or sources of the major problems in the simulated acid load results were difficult to interpret, basically due to the approximate nature of several input parameters describing acid production and removal. Efforts to further improve simulation were suspended once adequate results were obtained.

CONCLUSION AND RECOMMENDATIONS

The model application experience presented herein was only the second attempt at verifying its performance. The authors feel that the model is capable of performing well and that any shortcomings experienced can basically be associated with the completeness or consistency of the data employed. It takes several years of intensive effort to collect a complete data package on a deep mine drainage study. The one used herein was the best known to be available. As a consequence of this application, the deficiencies in data collection schemes surfaced. A paper was presented by Ricca, 1976 on data deficiencies in mine drainage modeling. Not only were the problems encountered discussed, but detailed recommendations were listed for future mine drainage data collection endeavors. Some of the salient recommendations are:

1) collect hydrologic and mine drainage quality and quantity data in the same time frame,
2) monitor climatologic events within the watershed; precipitation hourly and evaporation, daily. If snow is prevalent, collect snowmelt data,
3) gather information on the soil characteristics of the watershed and perform field tests on the overburden material,
4) analyze pyrite oxidation characteristics of the material comprising the mined coal seam and overburden,
5) acquire mine maps and operation techniques, and
6) locate and monitor major surface water diversions into the mines and/or transfers of water within the mine complex.

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COMPLEX CONTINUUM MODEL FOR DESCRIPTION OF THE SIMULTANEOUS SOLID-FLUID MOVEMENTS

Richter, R. and Bobok, E.

ABSTRACT : The purpose of this paper is to set out the fundamental concepts, definitions and laws of continuum mechanics applying for non-homogenous systems of rock, water and gas. In this treatise all scalar, vector and tensor fields are multiplied, as multiplied the number or the phases. On the basis of these simultaneous field concept the equations of change for mass, momentum and energy are presented.

RESUME : L'objectif de cet article est d'établir les fondements du mode d'exposé des mouvements simultanés des systèmes non homogènes de composants continus des solides et des liquides. Dans notre description, tous les champs scalaires, vectoriels et tensoriels se multiplient en fonction des nombres de phases. Nous avons introduit les équations de compatibilité, les équations des mouvements et les équations de l'énergie sur la base de la théorie des fonctions spatiales.

RESUMEN : El objetivo de esta comunicación es establecer los conceptos fundamentales, definiciones y leyes, relativos a los movimientos simultáneos en sistemas no homogeneos de roca, agua y gas. En esta descripción, todas las magnitudes, ya sean escalares, vectoriales o tensoriales, quedan multiplicadas por el número de fases. Sobre la base de la teoría de funciones espaciales hemos introducido ecuaciones de compatibilidad, del movimiento, y de transferencia de masas y energía.

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The need to investigate the mechanical properties of rock has especially stimulated research in a field where interactions occur between the solid rock and a fluid phase: water or gas. This paper sets out a new mathematical foundation of the mechanical behaviour of rock with water or gas in its pores. It does not attempt to give direct solutions for mining or engineering problems. We wish this paper could be a preliminary condition for the later applications.

The base of this work is the apparatus of continuum mechanics. Our fundamental assumption is that the multicomponent solid--fluid system can be considered as a complex continuum. The detailed fine structure of the rock and the fluid in its pores can be replaced by a continuous model of matter having to some extent continuum properties. The essential mathematical idea of continuum model is that the real properties of matter are substituted by continuously distributed field functions of these properties for example: density, stress, strain, displacement, gravity force etc. These physical fields are multiplied simultaneously for multicomponent mechanical systems as they follow:

$$\varrho_S = \varrho_S(\vec{r},t) \qquad \varrho_F = \varrho_F(\vec{r},t) \qquad \varrho_G = \varrho_G(\vec{r},t)$$

$$\vec{v}_S = \vec{v}_S(\vec{r},t) \qquad \vec{v}_F = \vec{v}_F(\vec{r},t) \qquad \vec{v}_G = \vec{v}_G(\vec{r},t)$$

$$\underline{\underline{F}}_S = \underline{\underline{F}}_S(\vec{r},t) \qquad \underline{\underline{F}}_F = \underline{\underline{F}}_F(\vec{r},t) \qquad \underline{\underline{F}}_G = \underline{\underline{F}}_G(\vec{r},t)$$

The indices S, F and G refer to the solid, liquid and the gas phases. Like the density, velocity and stress similar multiplication is obtained for other scalar, vector and tensor fields.

Thus the material derivatives are also multiplied:

$$\frac{d..}{dt_S} = \frac{\partial ..}{\partial t} + /\vec{v}_S \nabla / \ ...$$

$$\frac{d..}{dt_F} = \frac{\partial ..}{\partial t} + /\vec{v}_F \nabla / \ ...$$

$$\frac{d}{dt_G} = \frac{\partial ..}{\partial t} + /\vec{v}_G \nabla / \ ...$$

Assuming that the pores can be ignored individually it is convenient to imagine the infinitesimal volume of the complex continuum containing many pores to interpret a scalar field of porosity by definition

$$\Phi /\vec{r}, t/ = \frac{dV_{pore}}{dV} .$$

If $\Phi /\vec{r}, t/$ is a continuous field and the pores are filled perfectly with liquid and gas, we get for the infinitesimal volume

$$dV = dV_S + dV_F + dV_G = /1 - \Phi / \ dV + \Phi \ dV$$

Let the amount of the pore volume filled with fluid α_F and with gas α_G. Thus we get the following expression:

$$dV = /1 - \Phi / \ dV + \alpha_F \Phi \ dV + \alpha_G \Phi \ dV$$

Naturally:

$$\alpha_F + \alpha_G = 1.$$

Let V = V/t/ denote an arbitrary material volume of rock with fluid and gas in its pores. The mass of the matter in this material volume does not change as V moves. This is the principle of conservation of mass, which can be express by the equation:

$$\frac{d}{dt} \int_{V/t/} \left[/1 - \Phi / \varrho_S + \alpha_F \Phi \varrho_F + \alpha_G \Phi \varrho_G \right] dV = 0.$$

The substantial time derivative of the total amount of mass will be separated to local and convective terms

$$\int_V \frac{\partial}{\partial t} \left[/1 - \Phi / \varrho_S + \alpha_F \Phi \varrho_F + \alpha_G \Phi \varrho_G \right] dV +$$

$$+ \int_{/A/} \left[/1 - \Phi / \varrho_S \vec{v}_S + \alpha_F \Phi \varrho_F \vec{v}_F + \alpha_G \Phi \varrho_G \vec{v}_G \right] d\vec{A} = 0,$$

in which the first integral is taken through the control volume V and the second integral is taken over the closed surface /A/, bounding V. The first integral represents a time rate of accumulation of mass within the control volume, the second is the algebraic excess of the outflux of mass through the control surface over the influx of mass. The principle of conservation of mass is valid for both the rock, the fluid and the gas separately:

$$\int_V \frac{\partial}{\partial t} \left[/1 - \Phi / \varrho_S \right] dV + \int_{/A/} /1 - \Phi / \varrho_S \vec{v}_S \, d\vec{A} = 0$$

$$\int_V \frac{\partial}{\partial t} /\alpha_F \Phi \rho_F/ \, dV + \int_{/A/} \alpha_F \Phi \rho_F \vec{v}_F \, d\vec{A} = 0$$

$$\int_V \frac{\partial}{\partial t} /\alpha_G \Phi \rho_G/ \, dV + \int_{/A/} \alpha_G \Phi \rho_G \vec{v}_G \, d\vec{A} = 0$$

This scalar integral equations can be replaced by their differential forms, since V is arbitrary

$$\frac{\partial}{\partial t} \left[/1 - \Phi /\rho_S \right] + \operatorname{div} \left[/1 - \Phi /\rho_S \vec{v}_S \right] = 0$$

$$\frac{\partial}{\partial t} /\alpha_F \Phi \rho_F/ + \operatorname{div} /\alpha_F \Phi \rho_F \vec{v}_F/ = 0$$

$$\frac{\partial}{\partial t} /\alpha_G \Phi \rho_G/ + \operatorname{div} /\alpha_G \Phi \rho_G \vec{v}_G/ = 0$$

The equations are valid ignoring the interphase mass transport. Otherwise source terms must be occured on the right side of the equations. If $\Phi = 0$ we get the continuity equation for homogeneous rock. If $\Phi = 1$ and $\alpha_F = 1$ the second equation turns to the continuity equation for pure fluids. Finally the $\Phi = 1$, $\alpha_G = 1$ case refers to pure gases.

We consider now the dynamics of the motion. The fundamental principle of the dynamics the law of conservation of momentum. As it is known this statement means that the rate of change of momentum of a material volume V equals the resultant force on the volume.

The momentum of the infinitesimal volume of the complex continuum is

$$\left[/1 - \Phi / \rho_S \vec{v}_S + \alpha_F \Phi \rho_F \vec{v}_F + \alpha_G \Phi \rho_G \vec{v}_G\right] dV .$$

Its rate of change expanding to the whole volume V is

$$\frac{d}{dt} \int_V \left[/1 - \Phi / \rho_S \vec{v}_S + \alpha_F \Phi \rho_F \vec{v}_F + \alpha_G \Phi \rho_G \vec{v}_G\right] dV =$$

$$= \int_V \left[/1 - \Phi / \rho_S \vec{g}_S + \alpha_F \Phi \rho_F \vec{g}_F + \alpha_G \Phi \rho_G \vec{g}_G\right] dV +$$

$$+ \int_{/A/} \left[/1 - \Phi / \underline{\underline{F}}_S + \alpha_F \Phi \underline{\underline{F}}_F + \alpha_G \Phi \underline{\underline{F}}_G \right] d\vec{A} .$$

Here $\vec{g}$ is the extraneous force per unit mass. We tacitly assume that the force $\vec{g}$ is a known function of position and time. We also adopt the stress principle of Cauchy, which states that upon any surface exist a distribution of the surface forces depending at any given time on the position and the orientation of the surface. This relation between the stress vector $\vec{f}$ and the direction of the surface element can be expressed by the stress tensor $\underline{\underline{F}}$ as

$$\vec{f} = \underline{\underline{F}}\, d\vec{A} .$$

Applying Gauss' divergence theorem, since the volume V is arbitrary it follows that

$$/1 - \Phi / \rho_S \frac{d\vec{v}_S}{dt_S} + \alpha_F \Phi \rho_F \frac{d\vec{v}_F}{dt_F} + \alpha_G \Phi \rho_G \frac{d\vec{v}_G}{dt_G} =$$

$$= /1 - \Phi / \rho_S \vec{g}_S + \alpha_F \Phi \rho_F \vec{g}_F + \alpha_G \Phi \rho_G \vec{g}_G +$$

$$+ \mathrm{Div} \left[/1 - \Phi / \underline{\underline{F}}_S + \alpha_F \Phi \underline{\underline{F}}_F + \alpha_G \Phi \underline{\underline{F}}_G \right] .$$

This equation of motion is valid for any complex continuum, regardless of the form of the stress tensor and other particular properties of the material may take. This equation can't be reduced to three independent vector equation referring to the solid, liquid and the gas component. This is impossible in consequence of the interphase momentum transport. If we separate the terms referring to the rock, water and gas component, all others can be regarded as the interacting forces between the different phases. Thus we get

$$/1 - \Phi / \rho_S \frac{d\vec{v}_S}{dt_S} = /1 - \Phi / \rho_S \vec{g}_S + \mathrm{Div} \left[/1 - \Phi / \underline{\underline{F}}_S \right] + \vec{k}_S$$

This is the equation of motion for the solid component of the complex continuum, where $\vec{k}_S$ is the interacting force between the solid and the other two components. Similarly we can obtain that

$$\alpha_F \Phi \rho_F \frac{d\vec{v}_F}{dt_F} = \alpha_F \rho_F \Phi \vec{g}_F + \mathrm{Div} /\alpha_F \Phi \underline{\underline{F}}_F / + \vec{k}_F$$

and

$$\alpha_G \Phi \rho_G \frac{d\vec{v}_G}{dt_G} = \alpha_G \Phi \rho_G \vec{g}_G + \mathrm{Div} /\alpha_G \Phi \underline{\underline{F}}_G / + \vec{k}_G .$$

Now we have nine further scalar equations, but there are three unknown extraneous forces in them. To obtain any solution we must postulate any simplified relationship for the unknown force $\vec{k}$. Furthermore it is approximately true that certain terms of the equations are negligible depending on the accompanying circumstances. For example the density of the gas phase is negligible compared with the fluid or the rock. In other hand the effect of tangential stresses is small in many practical cases for fluids, expecially for gases, however and therefore it is not unreasonable to consider the idealized situation in which the tangential stresses are neglected. Thus we get

$$\underline{\underline{F}}_F = - p_F \underline{\underline{I}}$$

and

$$\underline{\underline{F}}_G = - p_G \underline{\underline{I}}$$

where $\underline{\underline{I}}$ is the unit tensor, p is the preasure.

It is frequently convenient to neglect the rate of momentum of the solid phase. These simplifying assumptions allow to obtain approximate solutions in certain cases.

The principle of conservation of angular momentum is guaranteed, if the stress tensor is symmetric, i.e.

$$F^{ij} = F^{ji}.$$

If the stress tensor is symmetric, the following equation is not independent of the equation of motion:

$$\frac{d}{dt_S}\int_V \vec{r}\times(1-\Phi)\rho_S\vec{v}_S\,dV+\frac{d}{dt_F}\int_V \vec{r}\times\alpha_F\Phi\rho_F\vec{v}_F\,dV+$$

$$+\frac{d}{dt_G}\int_V \vec{r}\times\alpha_G\Phi\rho_G\vec{v}_G\,dV=\int_V \vec{r}\times\left[(1-\Phi)\rho_S\vec{g}_S+\right.$$

$$\left.+\alpha_F\Phi\rho_F\vec{g}_F+\alpha_G\Phi\rho_G\vec{g}_G\right]dV+\int_{/A/}\vec{r}\times\left[(1-\Phi)\underline{\underline{F}}_S+\right.$$

$$\left.+\alpha_F\Phi\,\underline{\underline{F}}_F+\alpha_G\Phi\,\underline{\underline{F}}_G\right]d\vec{A}$$

Let the kinetic energy of an arbitrary volume V of the complex continuum

$$\int_V\left[(1-\Phi)\rho_S\frac{v_S^2}{2}+\alpha_F\Phi\rho_F\frac{v_F^2}{2}+\alpha_G\Phi\rho_G\frac{v_G^2}{2}\right]dV,$$

and let the deformation tensor

$$\underline{\underline{S}}=\frac{1}{2}(\nabla\vec{v}+\vec{v}\nabla).$$

Then the rate of change of kinetic energy is equal to the rate at which work is being done on the volume by external forces, diminished by a dissipation term involving the interaction of stress and deformation which is transformed into heat:

$$\frac{d}{dt_S}\int_V /1-\Phi/\varrho_S \frac{v_S^2}{2}\,dV + \frac{d}{dt_F}\int_V \alpha_F\Phi\varrho_F\frac{v_F^2}{2}\,dV +$$

$$\frac{d}{dt_G}\int_V \alpha_G\Phi\varrho_G\frac{v_G^2}{2}\,dV + \int_V\Big[/1-\Phi/\,\underline{\underline{S}}_S : \underline{\underline{F}}_S + \alpha_F\Phi\,\underline{\underline{S}}_F : \underline{\underline{F}}_F +$$

$$+\alpha_G\Phi\,\underline{\underline{S}}_G : \underline{\underline{F}}_G\Big]\,dV = \int_V\Big[/1-\Phi/\varrho_S\,\vec{g}_S\,\vec{v}_S + \alpha_F\Phi\varrho_F\,\vec{g}_F\,\vec{v}_F +$$

$$+\alpha_G\Phi\varrho_G\,\vec{g}_G\,\vec{v}_G\Big]\,dV + \int_{/A/}\Big[/1-\Phi/\,\vec{v}_S\,\underline{\underline{F}}_S + \alpha_F\Phi\,\vec{v}_F\,\underline{\underline{F}}_F +$$

$$+\alpha_G\Phi\,\vec{v}_G\,\underline{\underline{F}}_G\Big]\,d\vec{A}$$

This scalar equation can also be written in differential form:

$$/1-\Phi/\varrho_S\frac{d}{dt_S}/\frac{v_S^2}{2}/ + \alpha_F\Phi\varrho_F\frac{d}{dt_F}/\frac{v_F^2}{2}/ + \alpha_G\Phi\varrho_G\frac{d}{dt_G}/\frac{v_G^2}{2}/ +$$

$$+/1-\Phi/\,\underline{\underline{S}}_S : \underline{\underline{F}}_S + \alpha_F\Phi\,\underline{\underline{S}}_F : \underline{\underline{F}}_F + \alpha_G\Phi\,\underline{\underline{S}}_G : \underline{\underline{F}}_G =$$

$$= /1-\Phi/\varrho_S\,\vec{g}_S\,\vec{v}_S + \alpha_F\Phi\varrho_F\,\vec{g}_F\,\vec{v}_F + \alpha_G\Phi\varrho_G\,\vec{g}_G\,\vec{v}_G +$$

$$+\mathrm{Div}\Big[/1-\Phi/\,\underline{\underline{F}}_S\,\vec{v}_S + \alpha_F\Phi\,\underline{\underline{F}}_F\,\vec{v}_F + \alpha_G\Phi\,\underline{\underline{F}}_G\,\vec{v}_G$$

Finally the principle of conservation of total energy of a volume V of the complex continuum can be written. We define the total energy of a volume V as the sum of its kinetic energy and its internal energy E. The thermodinamical interpretation of the internal energy is different for compressible and incompressible media. For a compressible material for example gases we assume that internal energy is a thermodynamic state variable, which satisfies the following equation

$$TdS = dE + p\, d\left(\frac{1}{\rho}\right)$$

where T is the absolute temperature, S is the entropy per unit volume, p is the pressure. For incompressible materials the pressure is not a thermodynamical variable, because the density is constant we obtain a simpler equation:

$$TdS = dE$$

The principle of conservation of the total energy states that the rate of increase of the total energy of a material volume V is equal the rate at which work is being done on the volume plus the rate at which heat is conducted into the volume.

$$\frac{d}{dt_S}\int_V (1-\Phi)\rho_S\left(\frac{v_S^2}{2}+E_S\right)dV + \frac{d}{dt_F}\int_V \alpha_F\rho_F\Phi\left(\frac{v_F^2}{2}+E_F\right)dV +$$

$$+\frac{d}{dt_G}\int_V \alpha_G\Phi\rho_G\left(\frac{v_G^2}{2}+E_G\right)dV = \int_V\Big[(1-\Phi)\rho_S\vec{g}_S\vec{v}_S -$$

$$+\alpha_F\Phi\rho_F\vec{g}_F\vec{v}_F+\alpha_G\Phi\rho_G\vec{g}_G\vec{v}_G\Big]dV + \int_{(A)}\Big[(1-\Phi)\vec{v}_S\,\underline{\underline{F}}_S +$$

$$+ \alpha_G \Phi \vec{v}_G \underline{\underline{F}}_G \Big] d\vec{A} + \int_{/A/} \Big[\lambda_S \varepsilon_S \operatorname{grad} T_S + \lambda_F \varepsilon_F \operatorname{grad} T_F +$$

$$+ \lambda_G \varepsilon_G \operatorname{grad} T_G \Big] d\vec{A}$$

Applying the divergence theorem to the surface integrals we can get the differential form of the total energy equation, referring to the unit volume of the complex continuum:

$$/1-\Phi/\rho_S \frac{d}{dt_S} /\frac{v_S^2}{2} + E_S/ + \alpha_F \Phi \rho_F \frac{d}{dt_F} /\frac{v_F^2}{2} + E_F/ +$$

$$+ \alpha_G \Phi \rho_G \frac{d}{dt_G} /\frac{v_G^2}{2} + E_G/ = /1-\Phi/\rho_S \vec{g}_S \vec{v}_S + \alpha_F \Phi \rho_F \vec{g}_F \vec{v}_F +$$

$$+ \alpha_G \Phi \rho_G \vec{g}_G \vec{v}_G + \operatorname{div} \Big[/1-\Phi/ \underline{\underline{F}}_S \vec{v}_S + \alpha_F \Phi \underline{\underline{F}}_F \vec{v}_F +$$

$$+ \alpha_G \Phi \underline{\underline{F}}_G \vec{v}_G + \operatorname{div} \Big[/1-\Phi/\lambda_S \operatorname{grad} T_S + \alpha_F \Phi \lambda_F \operatorname{grad} T_F +$$

$$+ \alpha_G \Phi \lambda_G \operatorname{grad} T_G \Big]$$

The basic equations of the dynamics of complex continua are discussed.

The number of unknown variables is more in these equations than the number of independent scalar equations. Thus we need further equations to solve any problem. In general one may adjoin to these equations a thermodinamical relation, the equation of state:

$$p = f\,/\varrho\,,\ T/\ .$$

This equation may be different for gases, liquids or solids. The pressure and temperature dependence of density especially complex for multicomponent materials. With few exceptions, data are insufficient to make generalizations other than that methods suitable for pure materials may be applied with approximate results.

The other necessary equation is the so-called constitutive equation. It means the relation between the stress tensor and other kinematic variables. The stress tensor governs the dynamic response of the medium, by relating it to other kinematic and thermodynamic variables we can define the type of medium with which we work.

The most primitive form of the constitutive equation refers to inviscid liquids or gases

$$\underline{\underline{F}} = -p\,\underline{\underline{I}}\ .$$

Stokes derived a constitutive equation applicable to fluids which exert appreciable tangential stresses:

$$\underline{\underline{F}} = \left[-p + \varkappa \operatorname{div}\vec{v} + \varkappa^{*}/\operatorname{div}\vec{v}/^{2}\right]\underline{\underline{I}} + /2\mu + 2\mu^{*}\operatorname{div}\vec{v}/\,\underline{\underline{S}} + 4\nu\,\underline{\underline{S}}^{2}$$

which is simpler for incompressible fluids:

$$\underline{\underline{F}} = -p\,\underline{\underline{I}} + 2\mu\,\underline{\underline{S}} + 4\nu\,\underline{\underline{S}}^{2}\ .$$

Here μ is the viscosity, $\varkappa$, $\varkappa^{*}$, μ^{*}, ν are further constants.

The most general constitutive equation is presented by Asszonyi. He derived the following equation based on Onsager's law:

$$\underline{\underline{F}} = \underline{\underline{\Lambda}} \cdot / \nabla \bullet \vec{v} /$$

where $\underline{\underline{\Lambda}}$ is the conduction tensor of momentum. Its matrix is fourth-order, it has $3^4 = 81$ scalar elements. Regarding the symmetric properties of the tensor $\underline{\underline{\Lambda}}$, it has 21 independent elements in it. This is valid for perfectly anisotropic media.

If the conductive terms are dominated in the transport of momentum, the constitutive equation is the following

$$\underline{\underline{F}} = \underline{\underline{\Lambda}} \cdot / \nabla \bullet \vec{u} /$$

where $\vec{u}$ is the displacement vector.

The constitutive equation for the complex continuum can be the linear combination the above two equations:

$$\underline{\underline{F}} = /1 - \Phi / \underline{\underline{F}}_S + \alpha_F \Phi \underline{\underline{F}}_F + \alpha_G \Phi \underline{\underline{F}}_G =$$

$$= /1 - \Phi / \underline{\underline{\Lambda}}_S \cdot / \nabla \bullet \vec{u} / + \alpha_F \Phi \underline{\underline{\Lambda}}_F \cdot / \nabla \bullet \vec{v}_F / +$$

$$+ \alpha_G \Phi \underline{\underline{\Lambda}}_G / \nabla \bullet \vec{v}_G /$$

Thus the system of basic equations is complete. It includes many special cases.

If $\frac{\partial \Phi}{\partial t} = 0$, $\alpha_F = 1$, $\alpha_G = 0$, we get the equations for seeping flow through a rigid, porous media. The $\alpha_F = 0$,

$\alpha_e = 1$ case refers to the seepage of the gas. Allowing that $\frac{\partial \Phi}{\partial t} \neq 0$, the equations of consolidation can be derived.

The basic equation of hydraulic and pneumatic transport also can be deduced from this basic equation system.

We have the intention to introduce a general fenomenological treatment for the multicomponent systems based on continuum mechanics. We hope a better understanding of this very important topics for the practice of mining.

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AQUIFER PARAMETER IDENTIFICATION BY USING DIGITAL SIMULATION MODELS

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ABSTRACT : A digital model simulating unsteady-state radial flow to partially penetrating wells in unconfined anisotropic aquifers is applied to analyze test-data from an aquifer test carried out in the alluvial plains of Northern India. The parameters identified are the lateral permeability, anisotropy, specific storage and specific yield of the aquifer. Complex aquifer conditions can be simulated by digital models. It requires fewer number of observation wells for a digital model than those required for analytical methods.

RESUME : Un modèle digital qui simule un flux vacillant et radial aux puits qui sont partiellement pénétrés dans les aquifères illimités, est appliqué pour analyser les résultats d'une épreuve aquifère qu'on a faite dans les plaines du nord de l'Inde. Les paramètres qu'on a identifiés, sont la perméabilité latérale, l'anisotropie, l'emmagasinage spécifique et le produit spécifique de l'aquifère. A l'aide de ce modèle, on a pu simuler les conditions aquifères complexes. D'ailleurs, dans ce modèle, on a besoin d'observations moins nombreuses que celles qu'on doit employer dans les méthodes analytiques.

RESUMEN : Se aplica un modelo digital, que simula el flujo radial en estado transitorio con pozos parcialmente penetrantes en acuíferos libres anisótropos, para analizar los resultados de un bombeo de ensayo efectuado en las llanuras aluviales del norte de la India. Los parámetros identificados son : permeabilidad lateral, anisotropía, almacenamiento específico y caudal específico del acuífero. Los modelos digitales pueden simular las condiciones complejas de los acuíferos reales e incluso se requiere para ello menor número de piezómetros de observación que en el caso de interpretaciones analíticas.

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1. INTRODUCTION

In the conventional approach to analyze aquifer test-data we resort to type-curve methods which in turn are based on analytical solutions (WALTON, 1970). However, quite often the conventional methods do not yield consistent results, because existing type-curve model aquifers do not adequately cover and describe all prevailing aquifer conditions. In this context, the need for application of digital computer techniques to aquifer test-data analysis assumes importance (WALTON, 1976). The obvious advantage in this approach, is the facility to consider complex hydrogeologic conditions which have either to be idealized or ignored in the analytical type-curve techniques. The approach here would be to seek numerical solutions of the hydraulic heads in the aquifer in response to a test-well discharging at a constant rate, with assumed values of aquifer parameters, and then comparing those model responses with observed field data. In the present work the application of such techniques is illustrated in the specific context of aquifer test situations commonly encountered in the alluvial plains of Northern India.

The alluvial plains in Northern India are underlain by stratified formations with relatively more permeable strata interbedded between relatively less permeable strata which are however limited in their later extent. Tube-wells pumping from these aquifers are only screened against the relatively more permeable strata and are therefore in effect partially penetrating wells. While carrying out aquifer tests for identifying the parameters characterizing these aquifers, the test-well is usually run at a constant discharge and time-drawdown data at one or two observation wells screened in test-well screen interval are recorded. The time-drawdown data do resemble those under leaky artesian aquifer conditions. However piezometric tubes tapping water table depths show that there is a lowering of the water table. Due to this fact and along with the fact that the aquifer formations are stratified, it may be more appropriate to treat the aquifer system as a single anisotropic aquifer under water table conditions, and the well to be partially penetrating. The aquifer anisotropy will take care of the stratifications.

Analytical methods to analyze data of tests carried out in aquifer conditions similar to the one encountered in the alluvial plains of Northern India have been reported elsewhere (BENNET, REHMAN, SHEIK, AND ALI SABIR, 1967). These methods however ignore the compressibility of the aquifer and assume that the well discharge is derived entirely from water table storage. Also, to apply these methods time-drawdown data at a number of pairs of observation wells spaced over the full radius of influence of test-

well are required. Besides the above two limitations, the lateral permeability that is evaluated is also subject to errors since the flow to well is not strictly convergent in the depth interval and radial distances at which the drawdown are recorded (LAKSHMINARAYANA AND RAJAGOPALAN, 1977).

In the present study numerical solutions are sought for the unsteady-state radial flow to a partially penetrating well pumping at a constant rate from an unconfined anisotropic aquifer. The solution technique is the iterative alternate direction implicit method (PEACEMAN AND RACHFORD, 1955). The computing algorithm is a modified version of the one that has been earlier reported (PRICKETT AND LONNQUIST, 1971). The numerical solution presented here is an extension of an earlier work where the aquifer was treated to be isotropic and incompressible and the well to be fully penetrating (STRELTSOVA AND RUSHTON, 1973). The digital simulation model is applied to analyze data from an aquifer test carried out by the Water Resources Directorate, Punjab, India. The test-data comprises of time-drawdown data at two pairs of observation wells, one of the pair tapping well-screen depths and the other the water table depths. The aquifer parameters that are identified are the lateral permeability, anisotropy, specific storage and specific yield of the aquifer. These parameters were repeatedly adjusted in the model till the model response matched sufficiently well with the field data. An added advantage of such a digital simulation approach is that one quite often can get away with a fewer number of observation wells than are required for analytical methods and thereby, economizing on the aquifer test expenditure. The application of digital simulation technique in analysing aquifer test-data that has been reported here is an extension of an earlier work where the test-well was fully penetrating and the aquifer was treated to be isotropic and incompressible (RUSHTON AND BOOTH, 1976).

2. THEORY

In unconfined aquifers water is released by compaction of aquifer, expansion of water and gravity drainage at free surface (WALTON, 1960). Water release by gravity drainage at free surface has been characterised as delayed yield (BOULTON, 1954). Boulton gave the theory of unsteady flow to fully penetrating wells in unconfined isotropic aquifers (BOULTON, 1963). Based on this theory type-curve solution methods to analyze test-data have been reported (PRICKETT, 1965). Boulton's delayed yield theory is identified with the influence of flow in the unsaturated portion or in other words in the dewatered zone. However, later it was demonstrated

that the unsaturated flow plays a subordinate role in the delayed yield process, and a conclusion was arrived at that it is the free surface and flux which have an exponential change with time, and that specific yield remains a constant (STRELTSOVA, 1973). In line with this conclusion, analytical solution for unsteady flow to partially penetrating wells in unconfined anisotropic aquifers have been reported where, the aquifer is treated as a compressible system, the phreatic surface is treated as a moving material boundary, the specific storage and specific yield are treated constants, and the well is assumed to be of infinitisimal diameter (Neuman, 1974).

A schematic representation of a partially penetrating well pumping at a constant rate from an unconfined aquifer of infinite lateral extent, exhibiting simple two-dimensional anisotropy and resting over an impermeable horizontal bed is shown in Figure 1. If it is assumed that a) the flow exhibits radial symmetry, b) water is released by compaction of aquifer, expansion of water and gravity drainage at water table, c) radial hydraulic gradient is constant over the full length of well screen, d) water table drawdown is a small fraction of the saturated thickness of aquifer, and e) prior to pumping, the potential throughout the aquifer is same, the partial differential equation governing the unsteady-state flow and the boundary conditions may be written as,

$$P_r \left[\frac{\partial^2 h}{\partial r^2} + \frac{1}{r}\frac{\partial h}{\partial r}\right] + P_z \left[\frac{\partial^2 h}{\partial z^2}\right] = S_s \frac{\partial h}{\partial t} \qquad (1)$$

$$h(r, z, 0) = h_o \qquad (2)$$

$$h(\infty, z, t) = h_o \qquad (3)$$

$$\frac{\partial h}{\partial z}(r, m, t) = -S_y \frac{\partial h}{\partial t}(r, m, t) \qquad (4)$$

$$\frac{\partial h}{\partial z}(r, 0, t) = 0 \qquad (5)$$

$$\frac{\partial h}{\partial}(r_w, z, t) = 0 \text{ for } 0 < z < z_1 \text{ and } z_2 < z < m \qquad (6a)$$

$$= Q/(2\pi r_w P_r 1) \text{ for } z_1 \leq z \leq z_2 \qquad (6b)$$

where, P_r and P_z are lateral and vertical permeabilities; S_s and S_y are specific storage and specific yield; m is saturated thickness; h is hydraulic head; h_o is initial hydraulic head; r is radial coordinate; z is vertical coordinate; t is time coordinate; r_w is radius of well; l is screen length; and z_1 and z_2 are bottom and top elevations of well screen.

A numerical solution is sought for the Equation (1) under the above specified boundary conditions.

3. DIGITAL SIMULATION MODEL

A two-dimensional finite difference grid with the columns and rows co-axial with the z and r axes respectively is designed to model the aquifer. The radius represented by each node along rows is a constant multiple α, of that represented by the preceeding node. If radial distances are taken in the logarithmic scale, the nodes along rows occur at equally spaced intervals $\log_e \alpha$. The inner most column represents a radial distance r_w. Δz, the distance between nodes along columns was so chosen as to incorporate in the model identical well penetration as existing in the field test-well. To simulate effectively the infinite lateral extent of the aquifer, the number of columns in the model were made to be sufficiently large so that the potential heads in the aquifer were not affected by the presence of the boundary edge. In the finite difference grid, each branch is assigned a hydraulic conductance value, and each node is assigned a storage factor value. Hydraulic conductance to be assigned to branches along the rows is computed as the product of lateral permeability of the aquifer and the area of cross-section normal to the radial direction divided by the length of flow parallel to the radial direction. In the case of branches along columns, the hydraulic conductance is computed as the product of vertical permeability of the aquifer and the area of cross-section normal to vertical flow divided by the length of flow parallel to the vertical direction. The storage factor to be assigned to nodes in the internal rows and the bottom most row is computed as the product of the specific storage of the aquifer and the volume of aquifer segment represented by each of them. In the case of the top most row which represents the water table, storage factor to be assigned to nodes in it is computed as the product of the specific yield of the aquifer and the area of cross-section to vertical flow represented by each of those nodes.

Either from a formal mathematical treatment, substituting finite difference approximations for the derivatives, or from a physical standpoint involving Darcy's law and the principle of conservation of mass, the finite difference equation for each of the nodes can be derived in terms of the hydraulic head at the node under consideration, the hydraulic heads at the adjacent nodes the hydraulic conductance assigned to branches connecting the node under consideration and the adjacent nodes, and the storage factor assigned to the node under consideration. These equations are written in the implicit form where the space derivatives are replaced by their finite difference equivalent at the time level at which potentials are to be calculated and the time derivative is replaced by a backward difference approximation. This implicit approximation has been proved to be unconditionally stable regardless of the time increment (SMITH, 1965). At the nodes representing the well screen the well discharge is simulated, with discharge at each of them being proportional to the length of the well screen represented by them.

The finite difference equation written for all the nodes in the finite difference grid gives rise to a system of simultaneous equations which has to be solved for the hydraulic heads. The iterative alternate direction implicit method (PEACEMAN AND RACHFORD, 1955) was adopted to solve this system of simultaneous equations. An available computing algorithm for this method (PRICKETT AND LONNQUIST) was suitably modified and used. The computer program written in Fortran IV language was run on IBM 7044/1401 system.

4. APPLICATION OF THE MODEL

The digital model was applied to analyze data from an aquifer test carried out by the Water Resources Directorate, Punjab. The test well was a gravel pack well of radius 0.102 metres, and was screened against the relatively more permeable strata identified by electric logging of the test-well bore. Two pairs of observation wells, O_1; Pz_1 and O_2; Pz_2, at radial distances 18.44 metres and 61.26 metres respectively, were used to record the time-drawdown data during the test-period. O_1 and O_2 tapped well screen depths and Pz_1 and Pz_2 tapped water table depths. The test-well was run at a constant discharge of 0.032 metre^3 per second and duration of test was about 3000 minutes. The time-drawdown data recorded at the observation wells are shown in Table 1 and a graphical plot can be seen from Figure 2.

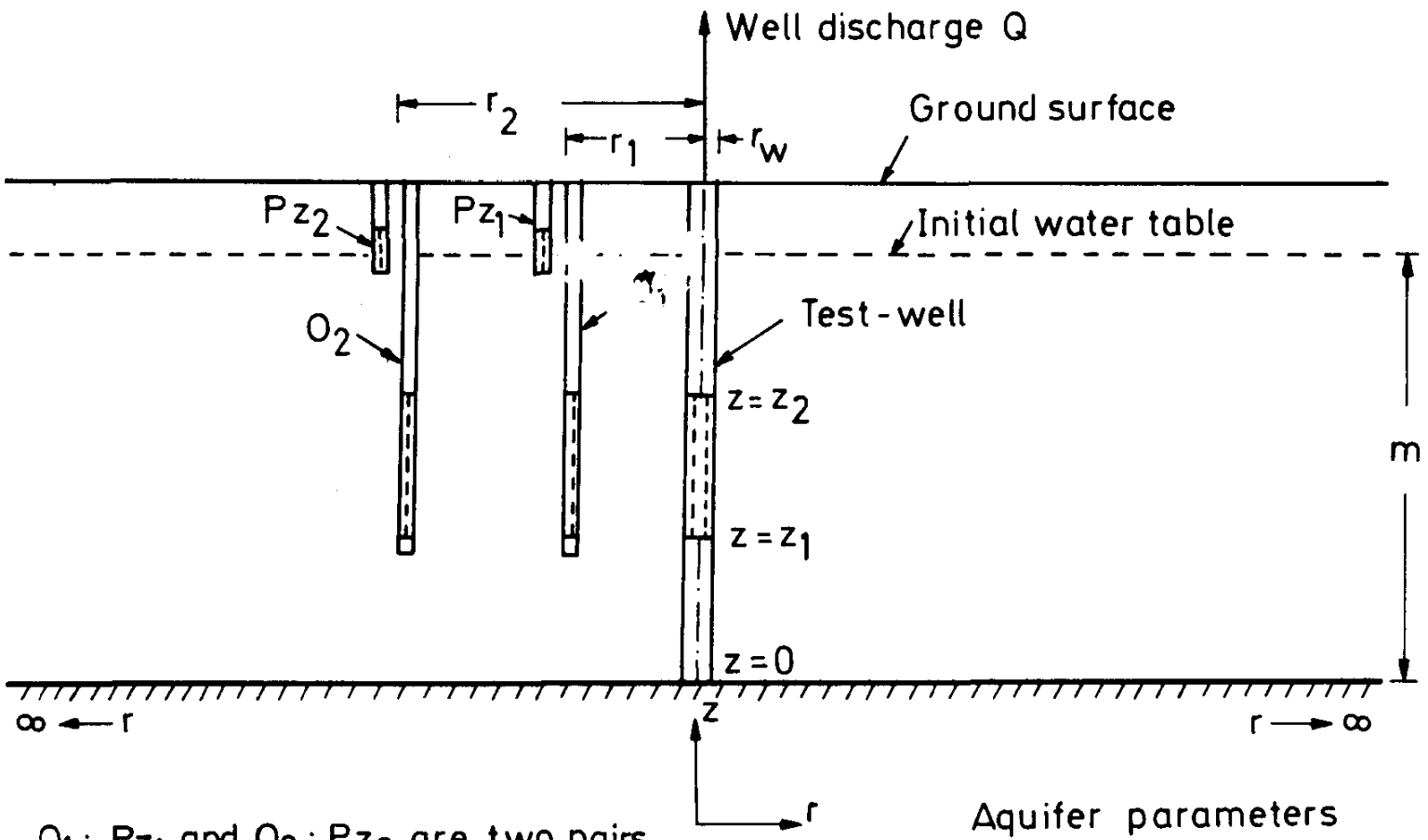

FIG.1 SCHEMATIC REPRESENTATION OF AQUIFER AND WELL CONDITIONS FOR THE UNSTEADY-STATE FLOW MODEL

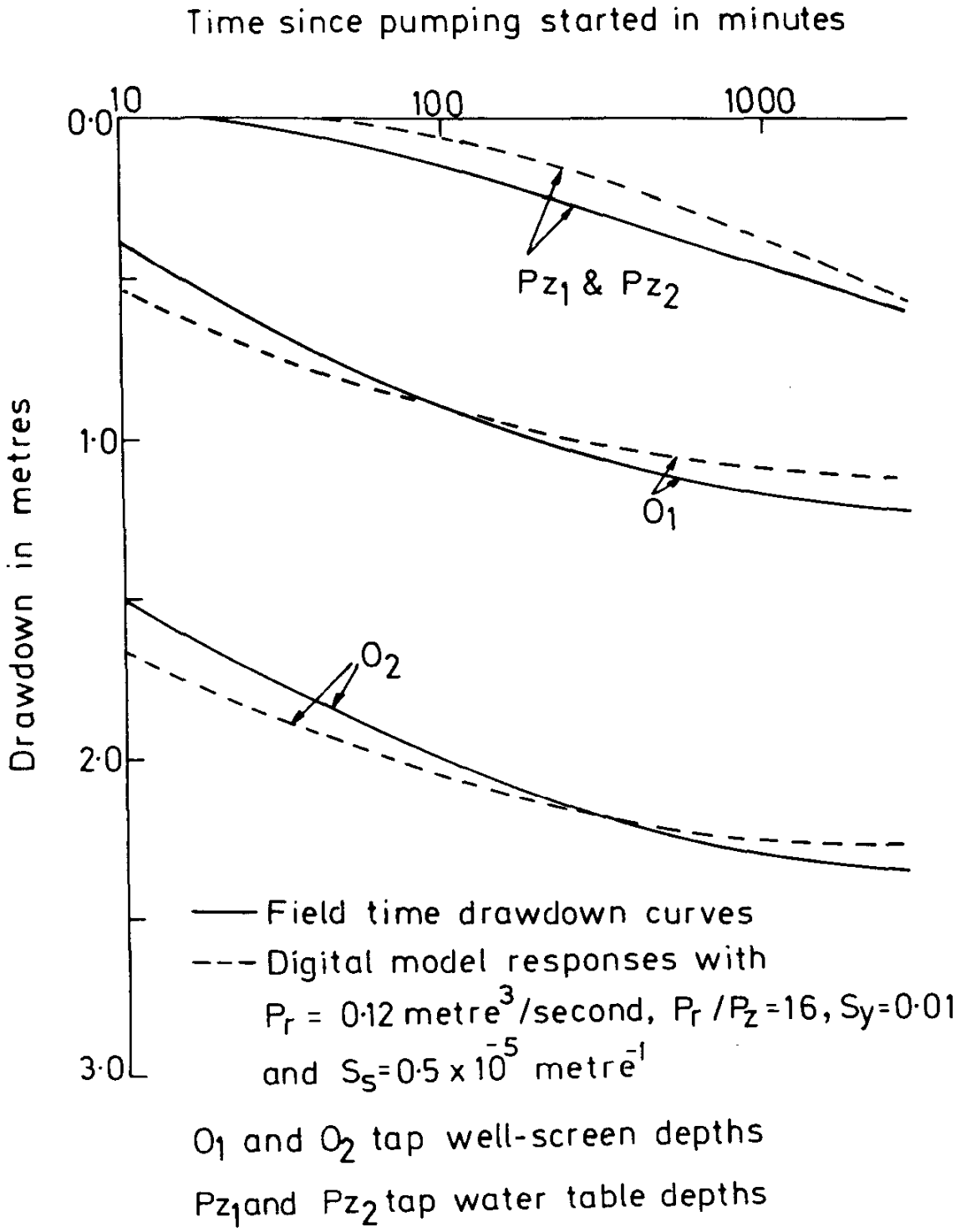

FIG.2 VARIATION OF DRAWDOWN WITH TIME

The drawdown data at well-screen depths do resemble that to be found in leaky artesian aquifers, and this suggests the possibility that the aquifer test-data may be analyzed by type curve methods available for the case of leaky artesian aquifers. Such an analysis would imply that the aquifer at test-well screen depths is under leaky artesian isotropic aquifer conditions and is overlain above the top of well-screen by a source bed at a constant potential. Observation wells Pz_1 and Pz_2 used in the field aquifer test, however show that there is a lowering of water table during pumpage. This aspect of the water table behaviour would have been missed if these two observation wells were not installed. The electric logging of the test-well bore also indicates that the aquifer is a stratified formation with relatively less permeable strata. The pattern of drawdown at well screen depths is probably explained by the fact that the test-well is screened at sufficient depths below the water table. Under these conditions it should be more appropriate to treat the aquifer test-situation as one where a partially penetrating well is pumping at a constant rate from a single anisotropic aquifer under water table conditions. The effect of stratifications will be taken care of by the anisotropy.

To model the aquifer the saturated thickness of aquifer was taken as 150 metres. A variable size z, was so chosen as to incorporate identical well penetration as existing in the field test-well. The radius represented by each node along rows was adopted as $\sqrt{2}$ times that of the preceeding node. The finite difference grid had 12 rows and 31 columns. At the nodes representing the well-screen in the innermost row of the model, the well discharge was distributed in proportion to the fractional screen length represented by each of them. The initial time increment chosen was 1 minute and for successive time steps, the time increment was increased by a factor of 1.2. Total number of time steps adopted was 35.

Repeated trial runs were made with various combinations of values for lateral permeability, anisotropy, specific storage and specific yield. Model response of time-distribution of drawdown at well-screen depths and water table drawdown were obtained for each trial run at the two radial distances at which the pairs of observation wells were located in the field aquifer test. The model response for each of these trial runs were compared with field drawdown data and the particular combination of P_r, P_r/P_z, S_s and S_y in the model for which this comparison gives the best possible match was identified. It was found that adopting lateral permeability P_r, as 0.12×1^{-3}, anisotropy P_r/P_z, as 16,

specific storage S_s, as 0.5×10^{-5}, and specific yield S_y, as 0.01 in the model, the model response agreed reasonably well with the field drawdown data at both the pairs of observation wells. The model response for this case is also reported in Figure 2.

5. CONCLUSION

The aquifer test-situations commonly encountered in the alluvial plains of northern India are partially penetrating wells pumping from unconfined anisotropic aquifers. Conventional type-curve methods of analysis of the test-data are often not appropriate. On the other hand the test-data may be analysed using a digital simulation model where the model parameters can be repeatedly adjusted till the model response matches sufficiently well with the field data. In the digital model a numerical solution is sought for the time distribution of drawdown at well screen depths and water table drawdown in response to a partially penetrating well pumping at a constant rate from unconfined anisotropic aquifer of infinite lateral extend and resting over an impermeable bed. The aquifer parameters that are identified are the lateral permeability, anisotropy, specific storage and specific yield. The field test-data requirements are time-drawdown data at two or three pairs of observation wells, one of the pair tapping well-screen depths and the other tapping water table depths. The digital simulation model has been applied to analyze data from an aquifer test carried out by the Water Resources Directorate, Punjab, India. An added advantage of such digital simulation approach over analytical methods is that a fewer number of pairs of observation wells are required thereby leading to considerable saving in aquifer test expenditure.

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TABLE 1. TIME-DRAWDOWN DATA

Time after pumping started t in minutes	Drawdown in Feet			
	O_1	Pz_1	O_2	Pz_2
2	3.59		0.49	
4	3.76		0.61	
6	3.93		0.90	
8	4.07		1.12	
10	4.34		1.28	
20	5.09		1.82	
30	5.42		2.13	
60	6.01	0.47	2.65	0.46
120	6.71		3.07	
180	6.90	0.83	3.28	0.81
300	7.04	0.95	3.44	0.94
420	7.22	1.10		1.06
540	7.27	1.18	3.64	1.14
720	7.29	1.30	3.71	1.26
900	7.30	1.35	3.74	1.33
1260	7.36	1.50	3.78	1.44
1500	7.55	1.58	3.88	1.54
1620	7.65	1.69	3.99	1.60
1860	7.80	1.77	4.07	1.69
2100	7.82	1.87	4.12	1.79
2340	7.84	1.92	4.20	1.83
3060	7.86	1.96	4.20	1.92

REDUCING WATER LEAKAGE INTO UNDERGROUND COAL MINES BY AQUIFER DEWATERING

Schubert, J. P.

ABSTRACT : Based on stratigraphic, structural, hydrogeologic, and mining data collected during a study in central Pennsylvania, a two-dimensional, finite-difference computer model was used to simulate groundwater flow in a sandstone unit (0.3 to 11 m thick) overlying an underground mine, and to evaluate the responses of the flow system and leakage rate into the mine when hypothetical dewatering wells are introduced into the system. Simulation of well dewatering, using 25 wells, showed that negligible reduction in leakage would occur if sandstone permeability was less than 0.30 m/day. When sandstone permeability equalled 3.0 m/day, 25 wells reduced leakage by 2.4 percent.

RESUME : On a utilisé un modèle bidimensionnel, de différences finies, pour simuler l'écoulement d'eau souterraine dans du grès (0,3 a 11 m d'épaisseur) qui recouvre une mine souterraine, et pour évaluer la dépendance du régime et la vitesse de drainance vers la mine, quand des puits hypothétiques de drainage s'introduisent dans le système. Ce modèle se base sur des données minières, hydrogéologiques, structurales y stratigraphiques. La simulation du drainage par puits, utilisant 25 d'entre eux, indique une réduction négligeable de drainance si la perméabilité du grès est inférieure à 0,30 m/jour. Pour des valeurs de 3 m/jour, 25 puits produiraient une réduction de drainance de 2,4 %.

RESUMEN : Se ha utilizado un modelo computacional bidimensional, de diferencias finitas, para simular el flujo de agua subterránea en una arenisca (0,3 a 11 m de espesor) que cubre una mina subterránea, y para evaluar la dependencia del caudal y la velocidad de los aportes hacia la mina, cuando se introducen en el sistema hipotéticos pozos de drenaje. Este modelo se basa en datos mineros, hidrogeológicos, estructurales y estratigráficos. La simulación del drenaje por pozos, usando 25 de ellos, muestra una reducción despreciable del aporte si la permeabilidad de la arenisca es menor de 0,30 m/día. Para valores de 3 m/día, 25 pozos producirían una reducción del aporte de 2,4 %.

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REDUCING WATER LEAKAGE INTO UNDERGROUND COAL MINES BY AQUIFER DEWATERING

Introduction

Pumping of water from wells has been used since the late 1920's to dewater ore-bearing rocks prior to and during the mining of iron ore, lead-zinc, and uranium deposits (Loofbourow, 1964). Well dewatering could also be a very useful means of reducing seepage of water into underground coal mines, thus reducing the flow of mine discharges. Several different dewatering schemes have been proposed (Ahmad, 1970; Parizek, 1970, 1971, 1972), but none have been tested or documented. Among the several proposed methods, gravity drainage, or connector, wells (Figure 1) appear to have the greatest potential for reducing seepage into active and abandoned underground mines for the least cost.

The purpose of connector wells is to reduce the hydraulic head and partially or completely dewater an aquifer above an active or abandoned underground mine. The wells would drain water away from the aquifer, allowing the uncontaminated water to flow by gravity past the mine and into a deeper aquifer (Fig. 1). By decreasing the hydraulic head in the upper aquifer, the leakage rate into the mine should also be reduced. Decreasing the flow rate from an abandoned underground mine will decrease the acidity and dissolved metal loading rates of the discharges (Schubert, 1978), thus reducing pollution problems. For an active mine, this would mean a reduction in costs of water handling and treatment. Connector well dewatering has many other desirable aspects as a mine drainage abatement technique (Parizek, 1970, 1972). Assuming the water being drained from the upper aquifer is good quality, it may serve to increase potable water supply yields from the lower aquifer. Only one aquifer above a mine needs to be dewatered (preferably the lowest one). Therefore, there should be little chance of depleting shallow domestic and farm groundwater supplies that occur commonly in coal mining regions. Pumping equipment is not used, so operating and maintenance costs are minimal. This is an important consideration because any abandoned mine drainage abatement program is a long-term endeavor. At any time in the future if local need for water became sufficient, wells could be converted to a water supply by installing pumps (assuming the quality of water in the wells is suitable for a desired use).

In 1973, the U.S. Environmental Protection Agency (U.S. EPA) approved and funded a demonstration project to test the engineering feasibility, evaluate the effectiveness as an acid drainage abatement technique, and develop cost estimates of connector well dewatering. Before drilling and installation of connector wells

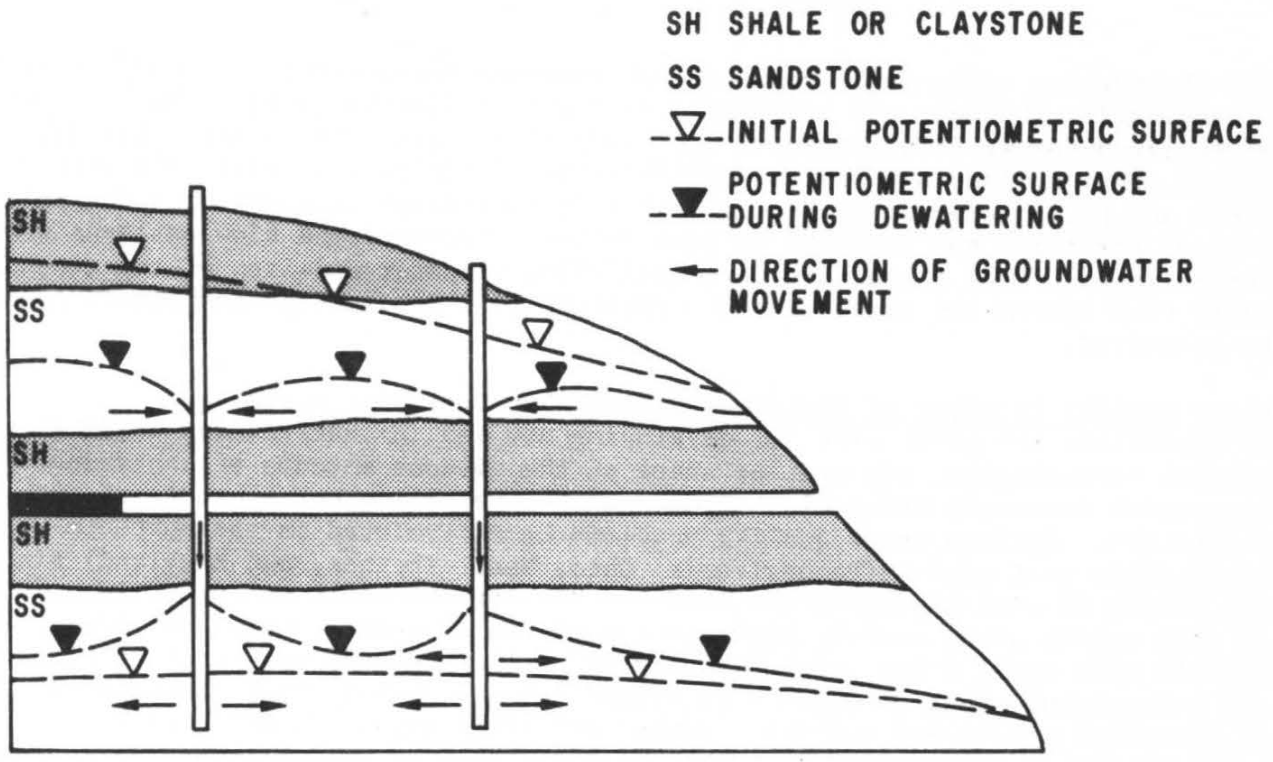

Figure 1. GRAVITY WELLS DISCHARGE WATER INTO UNDERLYING AQUIFER

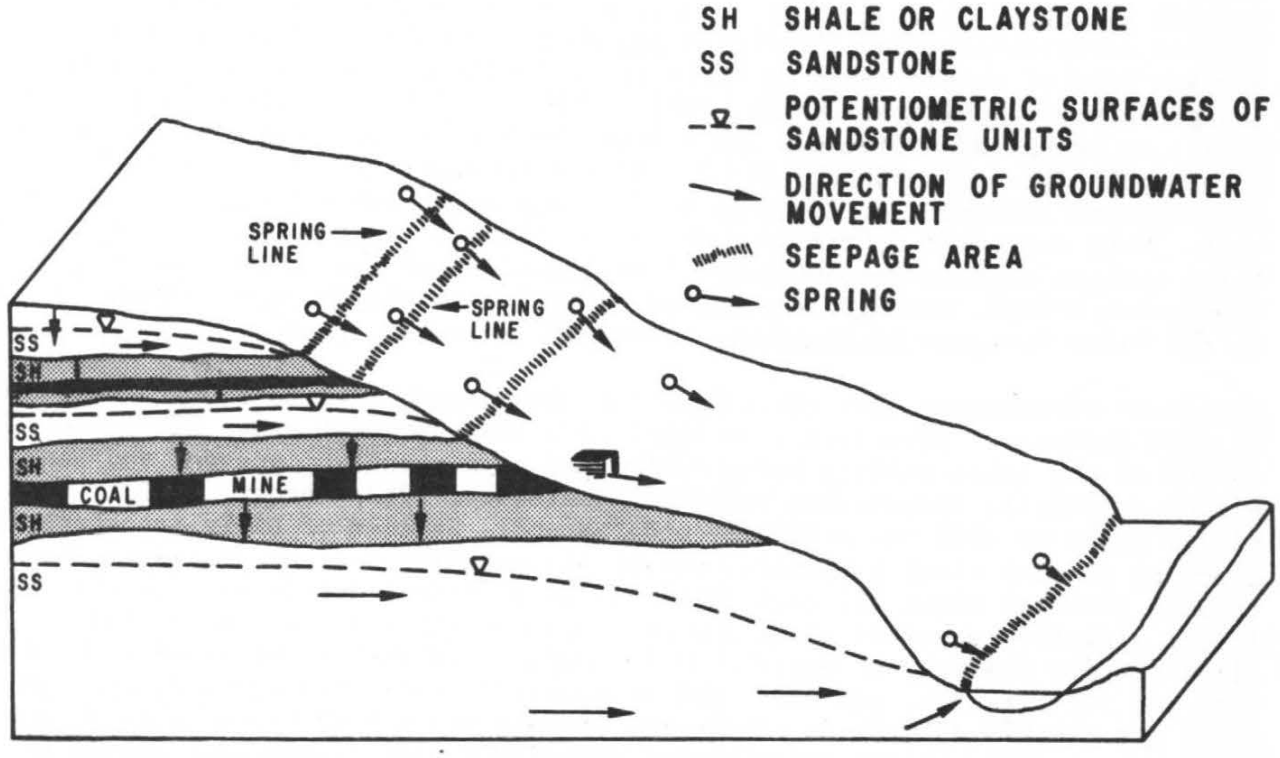

Figure 2. GENERALIZED GROUNDWATER FLOW SYSTEM OF THE APPALACHIAN BITUMINOUS COAL REGION (MODIFIED FROM PARIZEK, 1971)

and observation wells could begin at the proposed demonstration site in central Pennsylvania, geologic and hydrologic data were collected to determine the nature of the groundwater flow system in the study area. Areas of recharge and discharge, permeability and storage coefficients of lithologic units, the occurrence and potentiometric surface of water in individual aquifers, and the relation of fractures and jointing to groundwater occurrence and flow patterns were investigated. Once these data were available, computer modeling of the groundwater flow system and evaluation of connector well dewatering feasibility could be conducted.

Water Quality Problems of Abandoned Underground Coal Mine Discharges

To appreciate the great need for developing new and innovative mine-drainage control technologies, one must be aware of the awesome amounts of contaminated mine water currently being discharged, and the resulting consequences of such discharges. Serious water quality problems have developed in the northern Appalachian coal region (Pennsylvania, Ohio, West Virginia, and Maryland) since the advent of coal mining in the 1800's. Because the coals and overburden rock in this region often contain high concentrations of pyrite, water draining from surface mine spoil piles, coal refuse piles, slurry ponds, coal-washing plants, and underground mines is often very acidic and may contain high concentrations of dissolved metals and sulfate. Rivers and lakes seriously affected by acidic mine discharges can support only limited flora such as acid-tolerant molds and algae, and usually will not sustain fish. Ionic constituents from mine water, particularly dissolved metals, often occur in concentrations that are harmful or toxic to aquatic organisms. Streambeds can be coated with amorphous iron oxyhydroxide and other metal salt precipitates, which preclude the existence of benthic invertebrates and eliminates breeding areas for aquatic species. Subsurface flow of acidic water is known to seriously pollute springs, streams, and groundwater aquifers (Emrich, 1969; Schubert, 1978). In groundwater discharge areas, or seeps, mine drainage can destroy vegetation and cause the ruination of once productive soils (Brown, 1971). As the concentration level of mine drainage constituents increase, so do costs of treating water for potable or commercial uses. More extensive information on the biological, ecological, economic, and water quality impacts can be found in publications of the Appalachian Regional Commission (1969), the U.S. Army Corps of Engineers (1974), and references listed in the "Mine Drainage Bibliography 1910-1976" (Gleason, 1976).

Abandoned underground coal mines have long been recognized as the greatest source of acid drainage. Inventories in Appalachia indicate that these mines contribute 52-60% of the total acidity being discharged as a result of present and past coal mining activities (Appalachia Regional Commission, 1969; Warner, 1970). Coal mining prior to 1960 was primarily underground. Because of hilly topography and exposure of coal along hillsides, mining was normally done by placing mine entrances (drifts) along the coal outcrop. In addition, drifts were located at the lowest elevation possible along the outcrops and the coal was mined updip to allow gravity drainage of water from the mines. Because of high rates of rainfall and infiltration, and water tables generally near land surface, underground mines are usually subject to continuous seepage and act as groundwater drains. Subsequent surface mining and augering operations have intersected abandoned

underground mines and thereby created new routes of infiltration into, and drainage from, underground mines. Acid water continues to flow from abandoned mines and will continue to be a significant economic and environmental problem unless steps are taken to remedy the situation.

Methods of Controlling Mine Drainage

For abandoned underground mines, several methods or combination of methods have been utilized in an attempt to reduce acid discharges. Various ways to reduce infiltration of surface water were reviewed by Lorenz (1962). These include stream rechanneling, construction of stream channel liners, wooden sluiceboxes, and canals. Grading surface mines to facilitate surface water runoff away from underground mines is also very important. By causing surface water to avoid major avenues of infiltration, these methods can reduce sources of water entering underground mines.

In the 1930's, the federal government funded extensive sealing of mines with "wet" and "dry" air seals to prevent entrance of oxygen into mine atmospheres. Although some mines showed improvement in water quality, overall the method proved to be relatively ineffective. In several more recent and better documented cases, air seals were unable to reduce air entrance or pyrite oxidation (Moebs, 1970; U.S. EPA, 1977). Even though air may be prevented from entering a mine through sealed passages, it may still gain entrance through subsidence holes and fractured overburden.

Another method of isolating pyritic material from atmospheric oxygen is to flood a mine. Vanzandt (1933), for example, reported some naturally flooded mines discharged alkaline water, whereas nearby operating mines had acidic discharges. Thompson and Emrich (1969), Koppe and Thompson (1972), and Miller and Thompson (1974) make several recommendations to successfully seal and flood a mine. These designs are intended to make a mine relatively "leakproof" after cessation of mining. Prior to 1966, when mining companies in Pennsylvania were not responsible for discharges, mines were not designed for effective flooding after closure. Shaft mines below regional water tables will naturally flood, and sealing of these mines is not difficult. In recent years, efforts have been made to seal drift mines that are located in zones of perched or semi-perched water tables above major surface water drainage. These sealing projects have proven to be more difficult. Significant leakage from a mine can occur through 1) thin or fractured roof rock or mine perimeter, 2) thin coal barriers between adjacent mines, 3) locations where past surface mining or augering has intersected an abandoned mine and later backfilled, or 4) numerous types of undiscovered mine features, such as air shafts, boreholes, or water drains. As a consequence, these mines are extremely leaky and seldom attain complete flooding when sealed (Foreman, 1972, 1974; U.S. EPA, 1977), thus allowing pyrite oxidation to continue in unflooded portions of a mine. If a drift is located at the lowest elevation of a coal mine and mining has proceeded updip, several hundred feet of hydrostatic head due to flooding would make a safe, effective seal almost impossible.

Since 1967, over $100 million has been spent by the state of Pennsylvania for acid mine drainage abatement, including surface mine reclamation, mine sealing,

water treatment facilities, engineering designs, and administration costs. In addition, many millions of dollars of federal funds have also been spent for research, development, and demonstration projects. Despite this, Pennsylvania and other states of the Appalachian region are still plagued with mine water problems.

Groundwater Hydrology in the Appalachian Coal Basin

Coal-bearing strata in the Appalachian region generally consist of repetitious sequences of gently-dipping coal, shale, siltstone, sandstone, limestone, and clay of Pennsylvania age. The presence, thickness, and lateral continuity of these lithologic units, however, are quite variable. Many hydrogeologic investigations (Poth, 1963; Emrich, 1969; Brown, 1971; Schubert, 1978) have noted that sandstone and coal are the most permeable lithologic units; clay, shale, and siltstone are relatively impermeable. Groundwater flow systems are usually recharged in the upper and middle portions of hillsides, and discharge along streambanks at the base of the hills, thus contributing to base-flow of streams (Fig. 2). The presence of aquitards (e.g., clays, shale, and siltstones) at various levels within a hill, however, impedes downward flow of groundwater, thus causing some lateral flow through relatively permeable units, and surface discharge at springs located along hillsides where aquifers crop out. Each isolated hill has its own shallow groundwater flow system, which may or may not be linked to deeper regional groundwater flow.

Because groundwater can drain from a hillside by horizontal flow through a permeable stratigraphic unit or drift mine, confined aquifers are partially dewatered and several perched water tables may exist above the regional water table (Fig. 2). Observations by drillers and hydrogeologic investigators (Poth, 1963; Emrich, 1969; Brown, 1971) provide evidence that several free water surfaces may be present, and that large differences in hydraulic head can exist between upper and lower aquifers (Brown, 1971). The downward movement of water in these recharge areas is often enhanced by vertical faults or fracture zones. Fracturing and caving of shale roof rock in abandoned mines increase vertical permeability and vertical flow rates, and may significantly reduce groundwater storage and hydraulic head in the aquifer above the mines (Ward, 1968; Schubert, 1978).

Water may enter mine workings in several ways. Rapid infiltration of water can occur through boreholes, fracture zones, faults, and caved areas in the roof. The source of water may be surface water or an overlying aquifer. Methods of controlling surface-water infiltration are available (Lorenz, 1962), as discussed previously. Faults, fracture zones, and boreholes can be grouted, if accurately located. Most water entering a mine is usually due to slow, steady leakage through jointed shale roof rock from an overlying sandstone aquifer (Lovell, 1974). If a permeable sandstone is directly above a mine or if shale roof rock is thin, highly jointed, or extensively caved, then flow from the sandstone into the mine will be fairly rapid and the sandstone will be partially dewatered. If a shale roof is present and not extensively caved, then a difference in head will exist between the sandstone and the mine roof. Leakage into the mine will occur, according to the equation (Parizek, 1971):

$Q = k'\Delta hA/m'$, where

Q = leakage rate through the shale roof, in m^3/day,
k' = vertical permeability of shale roof, in m/day,
A = area of mine roof, in m^2,
m' = thickness of shale roof, in m,
Δh = head difference between overlying aquifer and the mine roof, in m.

A relationship has been shown to exist between mine roof areas and mine discharge rates for eighteen abandoned coal mines in western Maryland (Hollyday, 1973). This field evidence partially confirms the validity of the leakage equation.

The presence of large drift mines can drastically modify a groundwater flow system. The mine acts as an underdrain, intercepting most of the downward flow and discharging the water from drift entrances. Recharge to underlying aquifers is greatly reduced in the vicinity of the mine.

Hydrologic Considerations For Well Dewatering

Favorable hydrogeologic conditions are necessary to efficiently dewater an overlying aquifer (Parizek, 1971). For economic and practical reasons, it is desirable to have the least number of highly productive wells. A thin, very permeable aquifer, separated from the mine by a thin aquitard, and a high head difference between the aquifer and the mine roof are ideal conditions for connector well dewatering. Dewatering wells must significantly reduce the head in the aquifer over the entire area of the mine and eventually dewater the aquifer to some extent. If a highly desirable aquifer is not present above a mine, then effective dewatering may still be achieved by opening connector wells to more than one aquifer zone. Multiaquifer dewatering will be necessary where aquifers have low permeability and are not very productive; however, the potential success of dewatering becomes less if the permeability of the aquifers is low.

Fracture traces detected on aerial photographs are underlain by relatively narrow zones (3 to 13 m) of vertical fracture concentration, and permeabilities of these fracture zones have shown to be 100 to 1000 percent greater than adjacent unfractured rock (Lattmann, 1964; Koppe, 1972). For dewatering purposes, the most productive well sites should be located at fracture trace intersections.

The lower aquifer receiving water from the connector wells must be capable of transmitting maximum flows that could be drained from the upper aquifer (i.e., it must have adequate thickness and permeability), so that significant head build-up does not occur in the lower aquifer as a result of recharge. For downward flow through the wells to occur at all, the potentiometric level of water in the lower aquifer must be substantially less than the level in the upper aquifer; this conditions was shown to be the general case by Brown and Parizek (1971).

As with any mine drainage control method, problems may arise. The first concern is diversion of groundwater away from private wells and springs around the perimeter of the aquifer being dewatered. If a water supply is threatened, then

the yield of connector wells could be reduced, one or more of the connector wells closest to the water supply could be plugged, or one of the connector wells could be converted to a water supply by installing a pump. The second concern would be to locate the wells so that they penetrate coal pillars. In the event that a mine void is encountered, grout columns (gravel piles stacked to the mine roof and injected with cement or other sealant) can be constructed to prevent roof settlement from destroying the well casing and protect the well casing from corrosive attack by acid mine water. The possibility of well plugging is the third potential problem. If ferrous iron is present in the overlying aquifer, then precipitation of iron oxyhydroxide and/or growth of filamentous iron-oxidizing bacteria in and around the lower portions of the wells could reduce permeability of the lower aquifer and well efficiency over a period of time. High concentrations of iron in groundwater above a mine are not typical unless caused by other mining activities at a higher elevation.

Geology, Hydrology, and Mining Conditions of the Study Area

The study area (approximately 65 km^2) in Clearfield County, Pennsylvania, is dissected into discrete, steeply-sloping hills by Clearfield Creek and its tributaries. Many gently-dipping coal seams crop out on the hillsides and have been subjected to mining since the late 1800's. Several large underground mines near Madera are the major sources of the acidity, dissolved metals, and sulfate being discharged in the watershed. The initial phase of this project was to collect geologic, mining, and hydrologic information necessary to the formulation of a groundwater flow model and dewatering simulation.

The Burgoon member of the Pocono Formation (Mississippian) is about 107 m thick and consists primarily of massive, cross-bedded, fine- to coarse-grained, fairly well sorted, micaceous sandstone. It is resistant to erosion and forms steep slopes rising above Clearfield Creek in the northern part of the study area. The top of the Burgoon Sandstone is an erosion surface (Mississippian-Pennsylvanian unconformity) which dips toward the south and southeast. Above the Burgoon Sandstone lies the Mercer Formation (Pennsylvanian), a 21 to 30 m complex sequence of coals, clays, dark shales, and sandstones. The Lower and Upper Mercer coals are thin and have not been mined.

The Clarion coal (with minor clay partings) ranges in thickness from 0.76 to 1.78 m and has been extensively mined. The largest underground mine is the Mid Penn No. 4 Mine which had numerous drift and shaft entrances (Fig. 3). In addition, a large portion of the Clarion coal outcrop around the hill has been surface mined in the past, often intersecting or closely approaching underground mine workings. The underground mine is roughly 5.15 km along a NNW-SSE direction, 2.17 km at its widest point, and covers an area of about 8.34 km^2. The mine dips to the south at an average slope of 14 m/km, and because the long axis of the mine is parallel to the maximum dip direction, a total of 70 m of structural relief occurs in the mine. According to the local residents who worked in these mines, the coal was continuous, but occasionally offset up to 0.6 m by faults. Most coal pillars were mined near the Bucher No. 7 and Betz drifts in the 1940's.

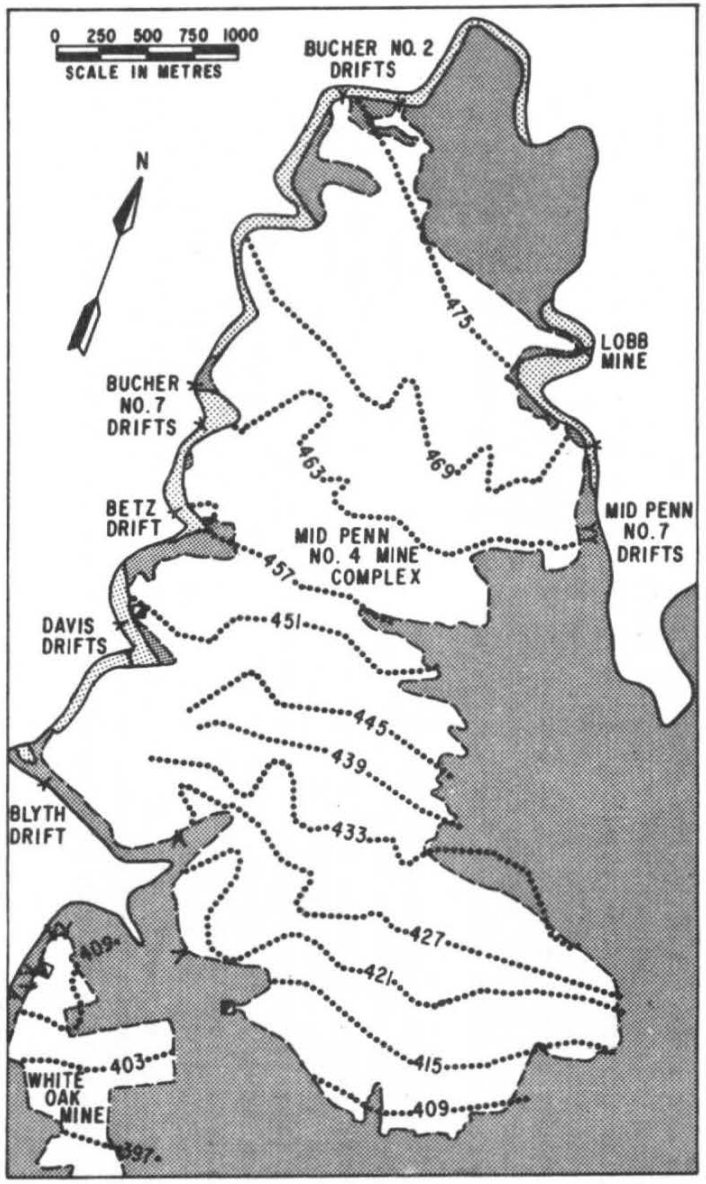

Figure 3. MINING AND STRUCTURAL GEOLOGY OF CLARION COAL

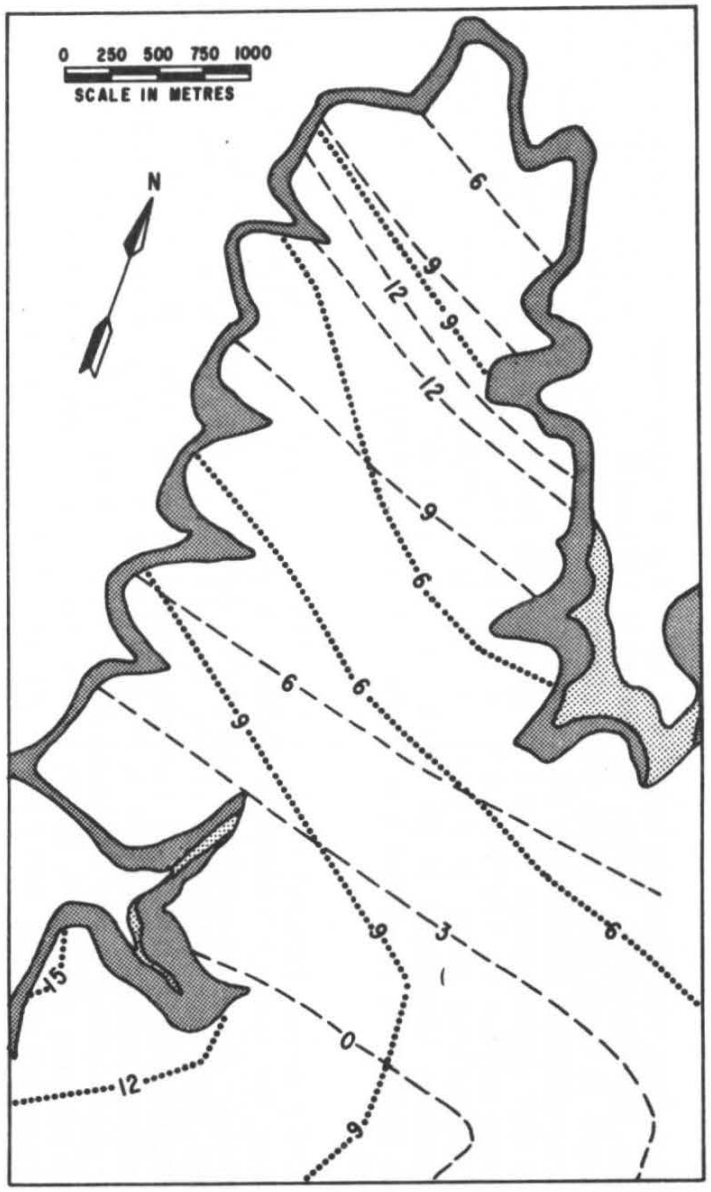

Figure 4. THICKNESS DISTRIBUTION OF CLARION SHALE AND SANDSTONE

Directly below the Clarion coal is 3 to 4.5 m of claystone. The Clarion coal is overlain, in ascending order, by variable thickness of 1) black, fissile, pyritic shale, 2) dark gray shale with siderite concretions, 3) silty shale, 4) fine- to coarse-grained sandstone, 5) siltstone, and 6) claystone. The combined thickness of the shales range from 6 to 12 m, with the thinnest shale roof occurring across the middle of the mine area (Fig. 4). The thickness is quite variable due to channeling prior to deposition of the overlying sandstone. In general, the sandstone thickness is inversely related to that of the shale, attaining a maximum thickness of approximately 12 m in the northern part of the mined area (Fig. 4).

Above the Clarion Formation is up to 107 m of additional coal-bearing strata, including the Lower Kittanning, Middle Kittanning, Upper Kittanning, and Freeport Formations. Small abandoned underground mines in the Lower Kittanning, Lower Freeport, and Upper Freeport coals, as well as surface mines in the Lower Kittanning, Middle Kittanning, and Freeport coals, overlie the Mid Penn No. 4 Mine. These mines have affected the hydrology in the hill to some extent. Surface mines, particulary Lower Kittanning and Lower Freeport contour mines, have impounded surface water and thereby increased groundwater recharge rate. Two Lower Freeport underground mines, located about 80 m above the Mid Penn No. 4, intercept some groundwater flow, resulting in small discharges to the surface.

The average annual precipitation in Madera is 1.04 m, with the greatest amounts falling from May through September. Using mean monthly temperatures, the annual evapotranspiration rate has been estimated to be 0.58 m, or 56 percent, using the Thornthwaite (1948) method. Studies (Reed, 1971) in coal basins of Pennsylvania have indicated that groundwater recharge may be in the range of 20 to 40 percent of total precipitation.

Once water enters the ground, its storage, flow rate, flow direction, and discharge are controlled by the geologic and hydraulic characteristics of the soil and rock units. Laboratory tests of small rock cores indicate that the intergranular porosity and permeability of coal-bearing strata are very low (Ward, 1968; Brown, 1971; Schubert, 1978); the porosities of shales, siltstones, and sandstones range from 5 to 11 percent and average permeability values are 4.82×10^{-5} and 4.15×10^{-5} m/day for shales and sandstones, respectively. Little difference exists between horizontal and vertical permeabilities.

Laboratory tests using rock cores do not reflect true field porosities and permeabilities because they do not taken into account all cracks, bedding planes, fracture zones, joints, and faults that occur in coal-bearing strata. Several studies (Poth, 1963; Ward, 1968; Brown, 1971; Koppe, 1972; Lovell, 1974) determined that fractures greatly influence the storage and flow of groundwater. Pumping tests, therefore, provide much better approximations of hydraulic characteristics. Fourteen drill holes in the study area were pumped at rates ranging from 6.3×10^{-5} to 37.9×10^{-5} m^3/s. Most drill holes were dewatered very quickly, since nearly all of the pumped water was derived from borehole storage. Residual drawdown was measured as a function of time after pumping each hole. The permeability values determined by the various recovery methods vary by two

orders of magnitude between holes. Several lithologic units were present in the saturated intervals of each bore hole tested, and the calculated permeabilities, therefore, represent a composite permeability of the various lithologic units. For three drill holes where no sandstone was present, permeabilities ranged from 1.10×10^{-3} to 2.53×10^{-3} m/day. The other drill holes had various percentages of sandstone present and permeabilities ranged from 2.65×10^{-3} to 8.80×10^{-2} m/day, suggesting that sandstone might be more permeable than shale and siltstone.

During the drilling of piezometers, Brown and Parizek (1971) observed that fractured coals and sandstones yielded the largest amounts of water, an observation that has been confirmed by many drillers in central Pennsylvania. In most surface mine highwalls in the study area where groundwater seepage was occurring, the water almost always flowed from the base of a sandstone unit or from a coal seam. Most springs in the area also emanated from sandstone or coal. The largest springs in the area flow from the Burgoon Sandstone along Clearfield Creek. In addition, owners of wells in the Burgoon Sandstone report that their wells have high yield capabilities and never go dry, indicating that this stratigraphic unit has the greatest permeability in the area's shallow groundwater flow system.

Water level elevations in 31 ponds, 166 springs, 83 residential wells, and 93 coal exploration drill holes were measured during January through July, 1975. Water levels in the dug wells and drill holes were generally less than 15 m below the ground surface and the water-table configuration is very similar to the surface topography. Greater depths to water existed in a few deeper drilled wells. Because most of the wells and drill holes in the study area were shallow, aerial configuration of potentiometric surfaces for groundwater in each aquifer could not be established. Two generalizations can be made, however: 1) in each hill, the hydraulic potential decreased with increasing depth and 2) for individual aquifers, potential is greatest in the interior of a hill and diminishes toward the edges, resulting in flow towards outcrop areas.

Digital Computer Model of Groundwater Flow

One of the primary objectives of this study was to estimate the potential reduction of leakage into the Mid Penn No. 4 Mine that could be achieved by connector wells. To do this, a digital computer model was essential to simulate the groundwater flow system and to evaluate the responses of the flow system and leakage rate into the mine when hypothetical dewatering wells are introduced into the system. A dewatering project would be most effective in reducing leakage into an underground mine if dewatering of the nearest aquifer above the mine was attempted (Parizek, 1971). In the case of the Mid Penn No. 4 Mine, the Clarion sandstone lies 6 to 12 m above the mine, ranges in thickness from 0.3 to 12 m, and is at least partially saturated, as evidenced by a few springs emitting from the Clarion Formation where it crops out around the hillside. For these reasons, simulation of groundwater flow and well dewatering of the Clarion sandstone was deemed most useful.

The computer program used for simulation is a two-dimensional, finite-difference model developed by Trescott and Pinder (1975) and recently modified (Trescott, 1976). Because the model is two-dimensional, only water flow into and out of

the Clarion sandstone could be simulated. A basic assumption of the model was that all water that flowed downward through connector wells could be adequately transmitted away from the wells by the receiving aquifer, the Burgoon Sandstone The capability of the Clarion sandstone to yield water to the connector wells is considered to be a much more critical factor affecting the usefulness of wel dewatering. To model a heterogeneous aquifer with irregular boundaries and boundary conditions, the model region was subdivided into a block-centered, finite-difference grid in which variable grid spacing is permissible (Fig. 5). Aquifer properties were assigned to each block based on actual or interpolated values available from field data collection. Assuming the coordinate axes of th grid are collinear with the principle components of the permeability tensor (when an aquifer is anisotropic), the partial differential equation governing groundwater flow in an aquifer in two dimensions may be expressed as:

$$\frac{\partial}{\partial x}\left(k_{xx} b \frac{\partial h}{\partial x}\right) + \frac{\partial}{\partial y}\left(k_{yy} b \frac{\partial h}{\partial y}\right) = S \frac{\partial h}{\partial t} + W(x,y,t), \text{ where}$$

k_{xx}, k_{yy} = are the principle components of the permeability tensor (L/T);
b = is the saturated thickness of the aquifer (L);
h = is the hydraulic head (L);
S = is storage coefficient for confined aquifers and specific yield for unconfined aquifers (dimensionless);
W(x,y,t) = is the volumetric flux of recharge or withdrawal per unit surface area of the aquifer (L/T);
L = constant unit of length;
T = constant unit of time.

Aquifer properties, aquifer boundary conditions, and initial head distribution in the aquifer are input data for the program. An equation of flow into and out of each aquifer block is calculated by the program using a finite-difference approximation of the above differential equation and the aquifer properties assigned to the nodes. An iterative equation-solving scheme then solves simultaneously the N unknowns (head values of the blocks) where N is the number of blocks representing the aquifer. The derivation and solution of the finite-difference equations and the theory of finite-difference aquifer models are discussed in more detail by Prickett and Lonnquist (1971) and Trescott et al. (1976).

The matrix of aquifer blocks is 35 (number of rows) by 51 (number of columns). The grid system is oriented so that it most efficiently fits the aquifer (and mine) boundaries within the grid, thus minimizing the number of blocks located outside the aquifer boundary (Fig. 5). The Δx and Δy dimensions of the individual blocks range from 122 to 427 m. Block sizes were generally made smaller near the boundaries of the aquifer and underground mine to better approximate their actual size and shape. Small block sizes were also arranged within the aquifer to help facilitate location of connector wells throughout the aquifer (accuracy of modeling is improved if wells are located in small blocks).

Data inputs that were fixed for all simulation studies were: aerial distribution of shale thickness, aerial distribution of the bottom elevation of the Clarion

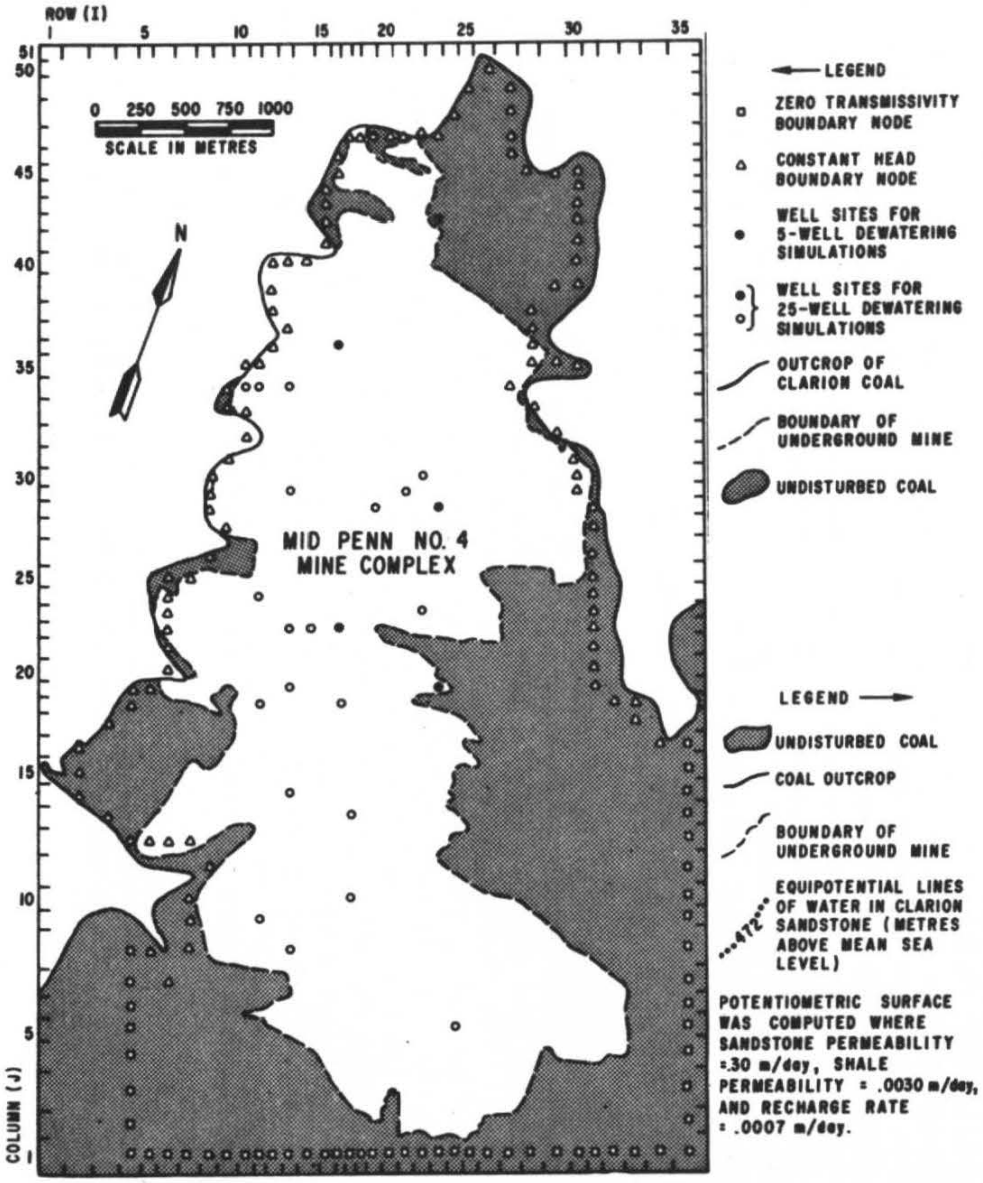

Figure 5. FINITE-DIFFERENCE GRID AND WELL LOCATIONS

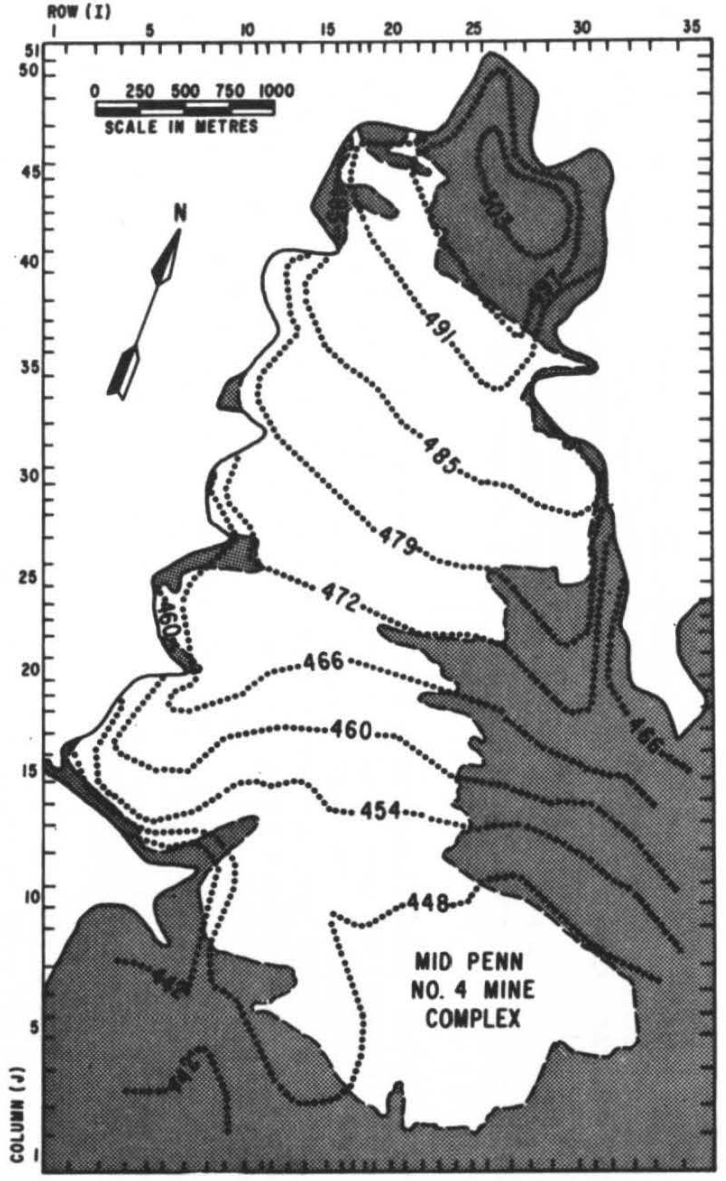

Figure 6. COMPUTED POTENTIOMETRIC SURFACE OF WATER IN CLARION SANDSTONE

sandstone aquifer, potentiometric surface at the base of the confining layer, initial potentiometric surface of the aquifer, and aquifer boundary flow conditions (constant head and zero flux boundaries). Parameters that were modified between simulation runs included: horizontal permeability of the Clarion sandstone, vertical permeability of the Clarion shale unit, constant recharge rate to the Clarion sandstone, and the number, location, and drainage rate of connector wells.

To estimate the potentiometric surface at the base of the shale unit, the following procedures were used: 1) where a mine pool exists, hydraulic potential was set equal to the mine pool elevation, 2) in unflooded portions of mine, hydraulic potential was set equal to the mine roof elevation, 3) where the Clarion coal cropped out along the western, northern, and northeastern edges, potential has been assigned a value of 0.9 m above the elevation of the top of the coal. Between the mined areas and the outcrop areas, hydraulic potential has been assigned an elevation higher than the top of the Clarion coal. The hydraulic potential distribution for the base of the shale unit is fixed and does not fluctuate during simulation.

Accurately estimating the initial potential distribution for the Clarion sandstone is not critical because the computer program will calculate a steady-state potentiometric surface of the aquifer based on other input data. The hydraulic potential values assigned to boundary blocks were, however, very important because they were fixed as constant head boundaries and influenced the horizontal movement of water into or out of the aquifer around its boundaries. Where the aquifer crops out along the western, northern, and eastern boundaries, potential was set to the bottom elevation of the aquifer plus 0.9 m, thus providing a seepage face for horizontal flow from the sandstone. Along the southern and southeastern blocks of the grid, the sandstone does not crop out and thins considerably (Fig. 4). For these blocks, aquifer thickness has been set to zero and therefore has zero transmissivity. These nodes form a no-flow boundary for the aquifer.

The horizontal permeability of the Clarion sandstone was varied between simulation runs; values used were 0.03, 0.15, 0.3, and 3.0 m/day. The values of 0.03 to 0.3 m/day are in the range of permeability values calculated from recovery test data. The Clarion sandstone above the Mid Penn No. 4 Mine is coarser-grained (contains a thin basal conglomerate at some locations) than the sandstone where recovery tests were conducted. Therefore, 3.0 m/day may be a reasonable value of permeability and has been included in some simulation runs. For each run, a single value of sandstone permeability was assigned to all nodes and permeability was assumed to be isotropic. The vertical permeability of the Clarion shale is another parameter that was varied for each modeling run; values used ranged from 10^{-5} to 10^{-3} m/day. These values lie in the range found from recovery test analyses and values given by Parizek (1971) and Brown and Parizek (1971).

Recharge rate to the entire hillside has been approximated as ranging from 0.21 to 0.42 m/year. Since some of this groundwater recharge leaves the hillside as

springs and seeps at elevations above the Clarion sandstone, 0.012, 0.178, and 0.254 m/year included a range of recharge rates to the Clarion sandstone that seems most reasonable.

The computer solution of the flow equation for all aquifer blocks, using the Strongly Implicit Procedure (SIP) of numerical analysis, yields the distribution of hydraulic head (potentiometric surface) in the aquifer and an accounting (water budget) of flow rates entering and exiting the aquifer. The water-budgeting feature enables a rapid evaluation of the flow system response to any change in aquifer parameters or the addition of dewatering wells.

Simulation of Natural Flow System (Model Calibration)

Three values of recharge rate, three values of horizontal permeability of the sandstone aquifer, and eight values of vertical permeability for the shale aquitard that appear realistic in terms of the data presented in previous sections were used to generate 54 possible steady-state potentiometric configurations, and are summarized by Schubert (1978). In the 54 simulations, the sandstone went mostly or completely dry in 24 runs when shale permeability was high or recharge rate was low. However, this condition is discounted by the fact that several springs emerge at many locations along the sandstone outcrop area. In 15 other simulations, completely unrealistic increases in potentiometric surfaces occurred when shale permeability was low or recharge rate was high. The possible combinations of variables that result in reasonable potentiometric surfaces are indicated in Table 1. The model was particularly sensitive to changes in shale permeability and recharge rate, and was only slightly affected by altering sandstone permeability.

In each case, the hydraulic head distributions show combined water table-artesian conditions in the aquifer with highest elevations occurring in the interior of the north end of the hill. The potentiometric surfaces decrease gradually toward the south and dip steeply toward the perimeter in outcrop areas, as shown in Figure 6.

Table 1. Calibration Tests of Natural Steady-State Flow Simulations

Shale Permeability (k') (m/day)	Recharge Rate (m/day)		
	0.00027	0.00049	0.00070
0.00305			
0.00152			
0.00091	aquifer goes dry		
0.00061			X
0.00046			X
0.00030		X	X
0.00015	X		
0.00003	excessive increase in potentiometric surface		

X = calculated potentiometric surface is similar to natural conditions when sandstone permeability equals 0.0305, 0.1524, or 0.3048 m/day.

Recharge rates, boundary flow rates, and leakage rates were calculated by the computer program for each simulation run. For the simulation runs having reasonable potentiometric surfaces (see Table 1), ratios of leakage rate to total discharge rate appeared to be quite high ($\geq$ 98.0%), indicating that nearly all of the water being recharged to the aquifer is leaking downward, and that very little water was flowing toward the constant head boundaries around the outcrop. Ratios with slightly lower values occur when the k/k' ratio is higher, which is to be expected. To account for the flow rate of springs from the aquifer around its boundary, the mean horizontal permeability of sandstone must be in the range of 0.152 to 3.05 m/day or higher. Estimates of discharge rate from the Mid Penn No. 4 Mine range from 4893 to 29359 m^3/day. Leakage rates calculated from computer simulations range from 15489 to 53972 m^3/day. This is a favorable comparison showing that the model is simulating downward leakage that corresponds very well with measured values. The calculated leakage rates are higher than the actual mine discharge rates, but this discrepancy can be accounted for because: 1) the mine does not extend under the full area of the aquifer being modeled; therefore, some water will leak into the undisturbed coal rather than the mine and probably continue downward into the underlying Mercer Formation, 2) leakage water is pooled within the mine for a period of time, thus allowing some vertical leakage into the underlying Mercer Formation.

Simulation of Connector Well Dewatering

Once the groundwater model was satisfactorily simulating the flow system, stress involving various hypothetical connector well systems (variable well numbers and spacing) was applied. The responses of the flow system and the leakage into the underground mine were determined for each hypothetical dewatering scheme, enabling the relative effectiveness of each scheme to be evaluated.

A dewatering well would hypothetically be screened the entire thickness of the Clarion sandstone. No well can completely dewater the aquifer immediately adjacent to the well bore, for there will always be a small seepage face existing. According to model design, wells must be assigned constant discharge rates. Thus, to simulate a connector well, the discharge rate for each well was gradually increased until the water level approached the base of the aquifer. Although wells were not placed on known positions of fracture traces or fracture-trace intersections, the potential hydraulic effects of wells placed on fracture traces were simulated by assigning the effective radii of wells to be 0.61 or 1.22 m. The large radii account for the increased storage and higher permeability of the fracture zones; the actual effective radii of such wells are not known, but are greater than the mechanical well radii.

The first dewatering simulations utilized 5 wells (locations are shown on Figure 5), and the following generalizations can be made: 1) very little water (<5.66 m^3 per day) can be drained by the wells on a continuous basis when the sandstone permeability is less than 0.16 m/day, 2) flow rates into wells were small (8.50 to 42.5 m^3/day) when sandstone permeability equalled 0.30 to 3.0 m/day, 3) flow rates into wells were much higher when sandstone permeability exceeded 3 m/day, and 4) the reduction in vertical leakage into the underlying mine ranged from 0.0 to 0.5 percent as a result of five dewatering wells.

Based on the preliminary simulations using 5 wells, more extensive dewatering systems of 25 wells were tested. Because only small sustained yields of water flowed to the wells where sandstone permeability was less than 0.305 m/day, the first 25-well test was run under the following conditions: well radii = 0.61 m, sandstone permeability = 0.305 m/day, shale permeability = 0.0003 m/day, and recharge rate = 0.00007 m/day. An aerial view of the potentiometric surface generated under these conditions (without connector wells) is shown in Figure 6. The maximum decline in water level for an aquifer block containing a connector well was only 4.02 m. The overall effects of the 25-well dewatering system is summarized in Table 2. The range of yields for the connector wells was 4.25 to 27.75 m^3/day and average 12.91 m^3/day. The decrease in leakage rate was only 0.49 percent.

Since connector wells in an aquifer where sandstone permeability = 0.30 m/day do not appear to produce a significant reduction in leakage, a second test was conducted where permeability was assigned the value of 3.0 m/day. The potentiometric surface generated under these conditions has a geometry very similar to the one presented in Figure 6, but is generally 0 to 4.6 m lower in elevation. The maximum decline in hydraulic head of a block containing a well was 6.6 m. The effects of this dewatering system is summarized in Table 2. The well yields ranged from 21.0 to 128.3 m^3/day and averaged 71.4 m^3/day. Vertical leakage from the aquifer was reduced by 2.44 percent (5 times more effective than the previous test where sandstone permeability = 0.30 m/day).

Both tests involving 25 wells were run again after one of the wells (location i=21, j=29) was removed. By comparing the potentiometric surfaces of the 25-well and 24-well systems, the extent of interference effects between dewatering cones of adjacent wells could be evaluated. When sandstone permeability = 0.30 m/day, removal of the well located at block (21,29) caused a recovery of 1.58 m in aquifer block (21,29), about 0.61 m in adjacent blocks (183 m distance), and about 0.15 m at 396 m distance. Water levels in wells (23,28; 22,30; and 19,28) located in the proximity of block (21,29) recovered from 0.61 to 1.22 m. When sandstone permeability = 3.0 m/day, water level in block (21,29) recovered about 1.83 m, water levels in adjacent blocks recovered 0.91 and 0.52 m at distances of 183 and 396 m respectively, and water levels in neighboring wells recovered 1.68 to 2.29 m when the well at block (21,29) was eliminated. From these tests it becomes evident that if additional wells were brought into the dewatering system and spaced less than 300 m apart, detectable interference effects of spreading cones of depression would likely occur. By adding additional wells, more water would certainly be drained from the aquifer, but placement of additional wells would slightly decrease the sustained yield and dewatering effectiveness of each individual well.

From the information obtained in the 25-well dewatering simulations, the following generalizations can be made.

1) Higher sandstone permeabilities ($\geq$3.0 m/day) are more conducive to dewatering; 25 connector wells could potentially decrease vertical leakage into the Mid Penn No. 4 Mine by 2.44%, if sandstone permeability = 3.0 m/day. Sustained

Table 2. Dewatering Simulations Utilizing 25 Wells

Before Dewatering	Sandstone Perm. = 0.30 m/day Shale Perm. = 0.0003 m/day Recharge Rate = 0.0007 m/day	Sandstone Perm. = 3.0 m/day Shale Perm. = 0.0003 m/day Recharge Rate = 0.0007 m/day
Recharge Rate	35469 m^3/day	35469 m^3/day
Boundary Flow	-573 m^3/day	-2423 m^3/day
Leakage Rate	-35939 m^3/day	-35285 m^3/day
During DeWatering		
Recharge Rate	35469 m^3/day	35469 m^3/day
Boundary Flow	-567 m^3/day	-2195 m^3/day
Leakage Rate	-35764 m^3/day	-34423 m^3/day
Range of Well Discharge Rates	-4.2 to -27.8 m^3/day avg. = -12.9 m^3/day	-21 to -128 m^3/day avg. = -71.4 m^3/day
Combined Well Discharge Rates	-322 m^3/day	-1784 m^3/day
Percent Decrease in Leakage Rate	0.49	2.44
Percent Decrease in Leakage Rate/Well	0.02	0.098
Percent Decrease in Boundary Flow	0.98	9.38

Negative numbers represent discharge from aquifer.

yields for wells where permeability equals 3.0 m/day are about 5 times greater than sustained yields of wells where sandstone permeability is equal to 0.30 m/day.

2) Dewatering of the sandstone aquifer decreased boundary flow (springs and seeps in outcrop area) by 9.38% when sandstone permeability equalled 3.0 m/day and by 0.98% when sandstone permeability equalled 0.30 m/day. Residential wells in outcrop areas would not be perceptibly affected by well dewatering.

3) A large number of wells will be required to affect significant reductions in leakage rate into the mine. The effectiveness of dewatering wells could be higher (thus requiring less wells) if the permeability of the Clarion sandstone is higher than estimated or if location and placement of wells in fracture zones can significantly increase sustained yields of the wells above that estimated.

Conclusions

Significant reduction of leakage rate of water into the Mid Penn No. 4 Mine by connector wells does not seem to be feasible under the geologic, hydrologic, and

mining conditions identified in this study. However, additional drilling would provide detailed data so that connector well dewatering could be more accurately simulated and evaluated. Based on the simulations where sandstone permeability was greater than 3.0 m/day, aquifer dewatering and reduction of leakage into a mine to reduce mine discharges could be very successful. The model used for this study was a two-dimensional, finite-difference model (Trescott, 1975), with slight modifications. Some additional modifications and experimentation, such as developing techniques to better represent aquifer boundary conditions, would be useful. Three-dimensional and finite-element models could also be utilized for mine drainage dewatering feasibility studies.

With great concern and money being spent on water quality control and mine drainage abatement projects at present, the technique of connector well dewatering merits consideration for additional studies and actual demonstration projects. Because of increasing legal responsibilities and costs for mine drainage treatment, this abatement technique also has great potential benefits for active underground mines. The costs of diverting good quality groundwater away from a mine will be much less than treating acid water discharges.

Acknowledgements

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PROBLEMES D'EXHAURE ET STABILITE DES PENTES DANS LA MINE A CIEL OUVERT DU MARQUESADO

par Streiff, C. et Ricci, G.**
avec la participation de Lamblin, J. M.** ; Ledoux, E.* ;
Medina Salcedo, F.*** et Mordos, R.*

ABSTRACT : In the open pit-mine of Marquesado (Spain), the iron-ore is extracted out of a lens of limestone, buried under alluvions, the depth of which reaches 200 m, and which contains an aquiferous sheet. Now, the presence of water under the alluvion-slopes is very bad for their stability. The problem which initially was to find practical means of drying the alluvions, lead the authors to build a mathematical model of the aquiferous sheet in a wide field around the pit, and then, to study the stability of the alluvion-slopes in presence of water, to know if it is better, from an economic point of vue, to dry the alluvions at any cost, or to give up drying them, but to reduce the declivity of the slopes accordingly, and to remove more alluvions.

RESUME : Dans la mine à ciel ouvert du Marquesado (Espagne), on extrait le minerai de fer d'une lentille calcaire, enfouie sous des alluvions dont l'épaisseur atteint 200 m, et qui contiennent une nappe phréatique. Or la présence d'eau dans les talus d'alluvions est très défavorable à leur tenue. Le problème posé, qui était au départ de trouver les moyens pratiques de dénoyer les alluvions, a amené les auteurs à construire un modèle mathématique de la nappe sur un vaste domaine autour de la mine, puis à effectuer une étude de la stabilité des parements en présence d'eau, afin de savoir s'il vaut mieux, sur le plan économique, dénoyer les talus quelqu'en soit le prix, ou bien renoncer au dénoyage, mais diminuer la pente des parements en conséquence et transporter plus de stérile.

RESUMEN : En la explotación minera a cielo abierto del Marquesado (España), el hierro se extrae de un lentejón de calizas, situado bajo aluviones cuyo espesor alcanza 200 m, que albergan un acuífero. La presencia de agua en los aluviones es muy desfavorable para la estabilidad de los taludes. Los autores, ante este problema cuya solución, en principio, consiste en encontrar medios prácticos para secar los aluviones, llegaron a la idea de diseñar un modelo matemático del acuífero, sobre un amplio área alrededor de la mina, para efectuar un estudio de estabilidad de los taludes, a fin de saber si es mejor, en el plano económico, secar los aluviones a cualquier precio, o renunciar al drenaje, con la contrapartida de disminuir la pendiente de los parámetros y, en consecuencia, extraer más aluviones.

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I - PRESENTATION ET HISTORIQUE DU PROBLEME

La compagnie Andalouse des Mines (filiale de la compagnie Mokta, du groupe IMETAL) exploite en découverture, sur le plateau du Marquesado, près de Grenade, un riche gisement de fer. Ce gisement est inclus dans une lentille calcaire, elle-même noyée sous des alluvions; si bien que pour une production annuelle de 3 à 4 Mt de minerai, on doit déplacer près de 20 Mt d'alluvions (10 Mm^3).

Ces alluvions présentent une granulométrie très étalée, qui va du gros bloc de quartzite d'un diamètre de 0,5 m jusqu'à l'argile, ainsi qu'une hétérogénéité très marquée. Les proportions des différents constituants (blocs, cailloux, graviers, sable, limon et argile) sont en effet éminemment variables, aussi bien à l'échelle régionale (affinement de la granulométrie vers l'aval), qu'à l'échelle de la dizaine de mètres, ou même du mètre. En effet, sur une même verticale se superposent, sans transition, des lits à gros grains, des lits limo-argileux, et des niveaux où les deux sont intimement mélangés. Et les variations suivant l'horizontale ne sont pas moins rapides.

Les calcaires, eux, sont les restes d'un ancien karst dans lequel s'est installé le gisement de fer, et ont été passablement métamorphisés (calcaires marmoréens). Leurs caractéristiques essentielles sont : leur grande dureté et leur intense fissuration.

L'ensemble alluvions-calcaires repose sur un substratum de micaschistes imperméables, et contient une nappe phréatique. (Voir figures 1 et 2).

Fig. 1 : Carte géologique simplifiée.

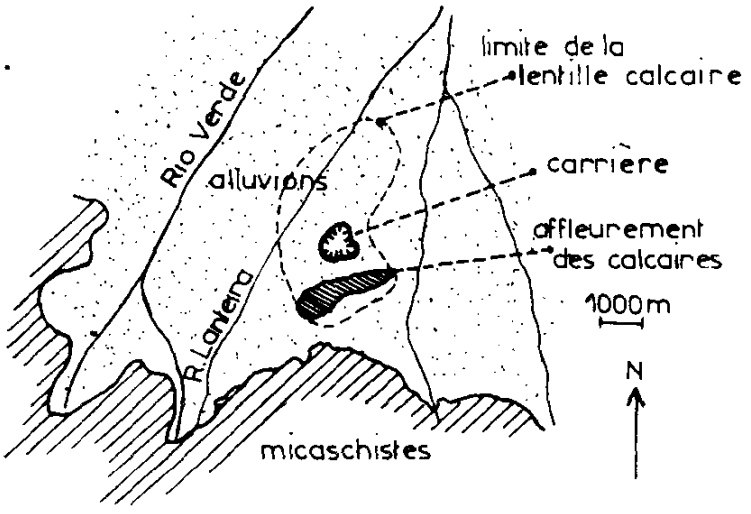
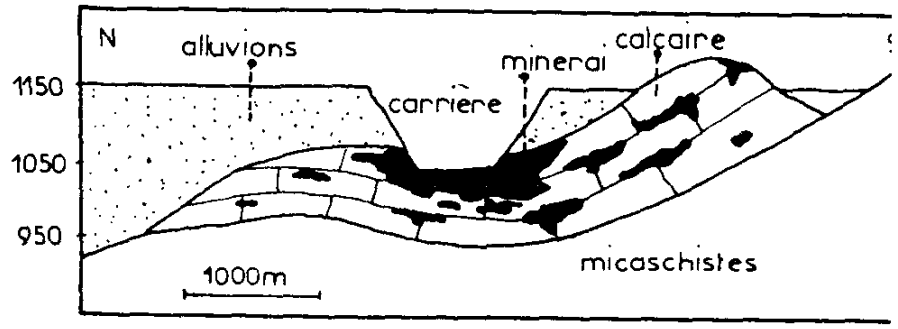


Fig. 2 : Coupe schématique Nord-Sud.



Depuis 1963, date à laquelle on a commencé à exploiter du minerai sous le niveau naturel de la nappe, on est donc confronté au problème du dénoyage. Au départ, le problème paraissait simple : la forte perméabilité des calcaires (perméabilité de fissures) permettait, au moyen de quelques travaux comme un réseau de galeries ou de puits de faible diamètre, de dénoyer le fond de la carrière. Ceci engendrait par drainance un rabattement de la nappe des alluvions, suffisant pour assécher les talus du même coup. Mais au fur à mesure de l'approfondissement et de l'élargissement de la carrière, on s'aperçut que les rabattements successifs du niveau d'eau dans les calcaires avaient de moins en moins d'influence sur le niveau dans les alluvions. La drainance naturelle ne remplissait pas son rôle, étant considérablement freinée par la présence, au contact entre alluvions et calcaires, d'un matériau argileux, le "rubial", ou de minerai, tous deux très peu perméables ($K \simeq 10^{-7}$ à 10^{-8} m/s). De plus, on exploitait des secteurs où les calcaires se trouvent de plus en plus profondément enfouis sous les alluvions, si bien que le rabattement nécessaire dans les alluvions pour assécher le talus devenait de plus en plus grand, et donc de plus en plus difficile. En fait, l'aquifère se scindait progressivement en deux; la nappe des calcaires devenait libre et son niveau baissait régulièrement; celle des alluvions atteignait un état d'équilibre. On aboutissait au schéma suivant : (figure 3)

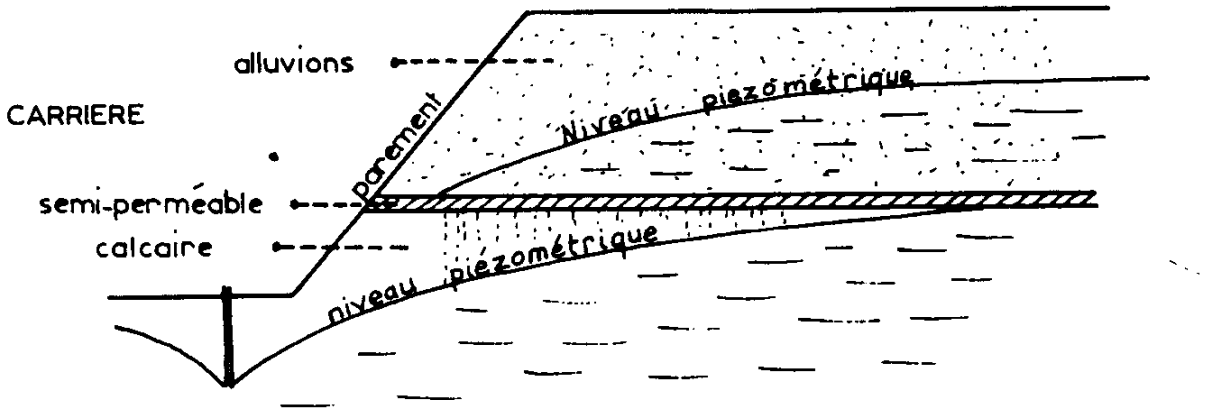


Fig. 3 : Schéma d'un talus montrant le décollement de la nappe des calcaires.

Si bien qu'en 1970, des reconnaissances firent craindre, pour les extensions futures de la carrière, des arrivées d'eau au pied des parements en alluvions.

Or, et c'est là le deuxième aspect du problème, la présence d'eau dans ces talus est très défavorable à leur tenue. Autrement dit, des parements mouillés demandent, pour être stables, une pente plus faible, ce qui signifie pour l'exploitant un volume d'alluvions plus important à déplacer. (voir figure 4)

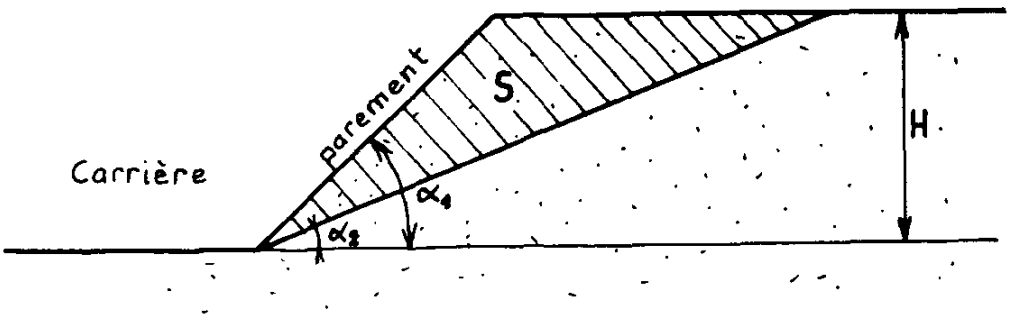


Fig. 4 : Volume supplémentaire d'alluvions à décaper :

$$\Delta V = L \times S = L.\frac{H^2}{2} (\text{cotg } \alpha_2 - \text{cotg } \alpha_1)$$: (L : longueur du talus).

Se pose alors la question suivante : faut-il dénoyer les talus en alluvions, quelqu'en soit le prix, ou bien au contraire renoncer au dénoyage, mais diminuer la pente des parements en conséquence et transporter plus de stérile ? S'y rajoute le fait que la stabilité de ces talus, lorsqu'ils sont mouillés, n'est guère connue que qualitativement, faute d'expérience et de modèles mathématiques pertinents. D'où, pour la deuxième solution, un risque de glissements de terrain plus élevé.

On a alors décidé de dénoyer le talus, en suppléant la drainance naturelle par l'installation de drains verticaux qui traversent alluvions, rubial et minerai, faisant ainsi jouer aux calcaires le rôle d'une crépine dans laquelle se déverse la nappe des alluvions. (Figure 5) Ce dispositif a résolu provisoirement le problème.

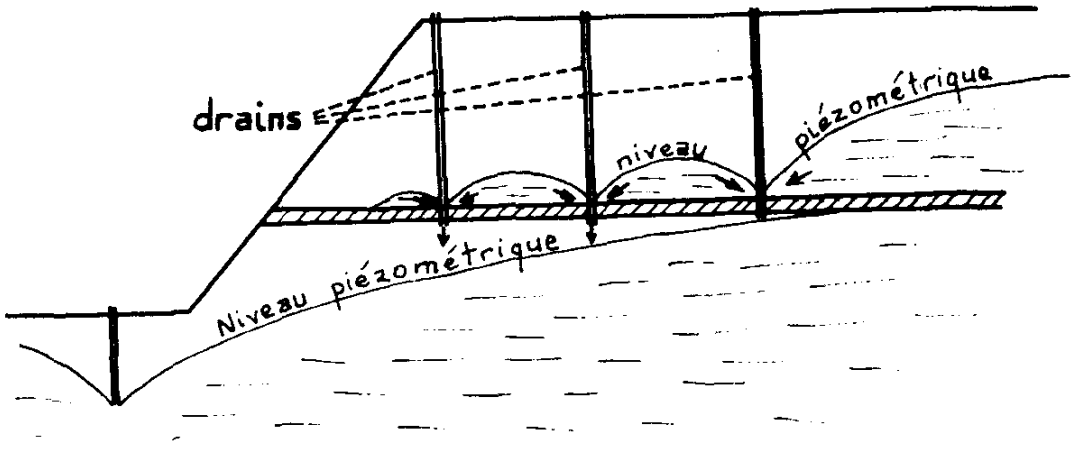


Fig. 5 : Fonctionnement des drains traversant le "rubial"

Mais les prochaines extensions de la carrière portent sur les secteurs où le calcaire est pratiquement inexistant : le système précédent ne s'applique plus, et le problème du dénoyage se repose de façon nouvelle. De même que se repose le choix entre dénoyer et déplacer davantage de stérile. Notre travail a été, précisément, d'apporter des éléments de réponse à ces questions.

II - LE DENOYAGE A MOYEN ET LONG TERME

1 - L'état initial - Nécessité d'un modèle régional

Pour des raisons liées à l'exploitation, on va prochainement détourner une rivière, la "Rambla de Lanteira", dont on pense que les infiltrations constituent une part inportante de l'eau qui arrive derrière les talus. Cette opération aura vraisemblablement un effet non négligeable sur le niveau de la nappe près de la mine. Notre premier problème a donc été de quantifier cet effet, c'est-à-dire de préciser l'état initial, l'état de référence de notre étude du dénoyage à moyen et long terme.

Or, le fait de détourner la rivière dépasse le cadre strict de la mine, et intéresse la nappe à l'échelle régionale; de plus, il est illusoire de vouloir quantifier ses effets en faisant appel à quelques calculs simples ou à des modèles analytiques : l'écart par rapport à la réalité est trop grand pour qu'on puisse en tirer quelque conclusion. La complexité du problème sur le plan géométrique, la multiplicité des apports à la nappe, la nécessité de tenir compte de la drainance entre alluvions et calcaires tout en étudiant la nappe dans le contexte régional, la recherche d'une connaissance précise du niveau piézomètrique aux abords de la mine : tout ceci nous amena à traiter le problème de façon numérique, c'est à dire à construire un modèle mathématique rendant compte du fonctionnement de la nappe des alluvions dans la région.

2 - Principe du modèle mathématique

La modélisation consiste à résoudre par discrétisation l'équation de la diffusivité sur un certain domaine, moyennant certaines hypothèses et certaines conditions aux limites. Rappelons tout d'abord l'équation de la diffusivité dans le cas général :

$$(1) \quad \mathrm{div}(\rho \bar{\bar{K}}.\mathrm{grad}\ h) = \frac{\partial}{\partial t}(\rho\omega) + \rho q$$

(ρ : masse volumique ; ω : porosité; q : débit prélevé par unité de volume; $\bar{\bar{K}}$: tenseur de perméabilité; h : charge hydraulique).

- Nous supposons tout d'abord que la verticale est une direction principale du tenseur de perméabilité. Plus précisément, nous supposerons que nos matériaux sont :

-soit isotropes : $\bar{\bar{K}} = \begin{pmatrix} K00 \\ 0K0 \\ 00K \end{pmatrix}$

-soit orthotropes : $\bar{\bar{K}} = \begin{pmatrix} K_h 00 \\ 0K_h 0 \\ 00K_v \end{pmatrix}$

- A cela, nous ajoutons l'hypothèse de Dupuit, qui dit que l'écoulement est plan et horizontal ($u_z = 0$, $\vec{v}$: vitesse d'écoulement), et que les vitesses sont égales sur une même verticale; comme on a $u_z = - K_v \frac{\partial h}{\partial z}$, on en déduit que la charge est constante sur une verticale. L'équation de la diffusivité s'écrit alors :

$$(2) \quad \frac{\partial}{\partial x}\left(K_h \frac{\partial h}{\partial x}\right) + \frac{\partial}{\partial y}\left(K_h \frac{\partial h}{\partial y}\right) = \frac{1}{\rho} \frac{\partial}{\partial t}(\rho\omega) + q$$

A ce stade, il nous faut distinguer deux cas, selon que la nappe est libre ou captive :

a - Nappe captive

Nous supposons la variation de porosité du milieu proportionnelle à la pression : ($\frac{\partial \omega}{\partial t} = \alpha \frac{\partial P}{\partial t}$, P: pression). L'intégration de l'équation (2) entre substratum et toit de la nappe donne :

$$(3) \quad \frac{\partial}{\partial x}\left(T \frac{\partial h}{\partial x}\right) + \frac{\partial}{\partial y}\left(T \left(\frac{\partial h}{\partial y}\right)\right) - Q = S \frac{\partial h}{\partial t}$$

(T : transmissivité de la nappe; S : coefficient d'emmagasinement; Q : débit surfacique prélevé).

b - Nappe libre

On considère cette fois la porosité comme constante dans le temps; on obtient :

$$(4) \quad \frac{\partial}{\partial x}\left(K_h (h-\sigma) \frac{\partial h}{\partial x}\right) + \frac{\partial}{\partial y}\left(K_h (h-\sigma) \frac{\partial h}{\partial y}\right) - Q = \omega \frac{\partial h}{\partial t}$$

(h : cote piézomètrique; σ : cote du substratum de la nappe).

- Le système hydraulique est alors schématisé par un ensemble d'aquifères horizontaux superposés, d'où le nom de modèle "multi-couche" séparés ou non par des niveaux semi-perméables, communiquant donc entre eux par drainance, et contenant des nappes qui sont soit libres, soit captives.

- On suppose que l'écoulement dans les niveaux semi-perméables est vertical, ce qui permet d'écrire : $u_z = - K \frac{\partial h}{\partial z}$, et, en régime permanent : $u_z = - \frac{K}{e} \Delta h$, Δh étant la différence de charge entre les deux aquifères séparés par le semi-perméable.

Dans notre cas, le schéma utilisé comportera 3 niveaux :

- un niveau aquifère pour les alluvions, où la nappe est libre
- un niveau semi-perméable, représentant l'ensemble du rubial et de la couche supérieure du minerai
- un niveau pour la lentille calcaire, où l'aquifère sera en partie libre, en partie captif. (Figure 6)

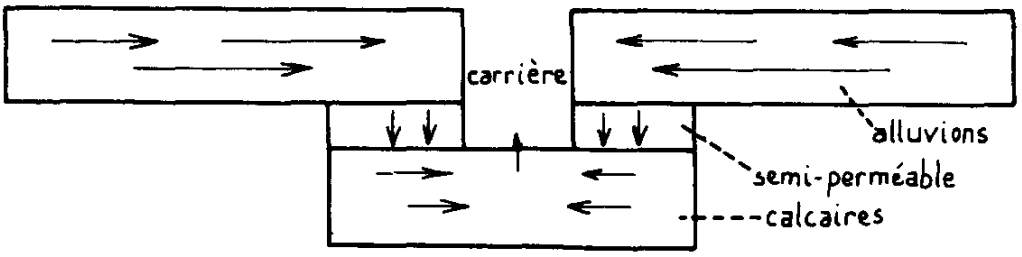


Fig. 6 : Schéma du modèle multicouche

La discrétisation consiste alors à "découper" chaque aquifère en mailles parallélépipédiques et à intégrer les équations (3) et (4), de façon approchée, sur ces volumes. Les premiers membres des équations représentant les débits volumiques entrant dans le volume que l'on considère, et les deuxièmes membres représentent les débits emmagasinés par ces volumes. Prenons l'exemple d'une maille de la nappe supérieure (alluvions) entourée de ses 5 voisines (on n'en a représentées que 4 sur la figure 7)

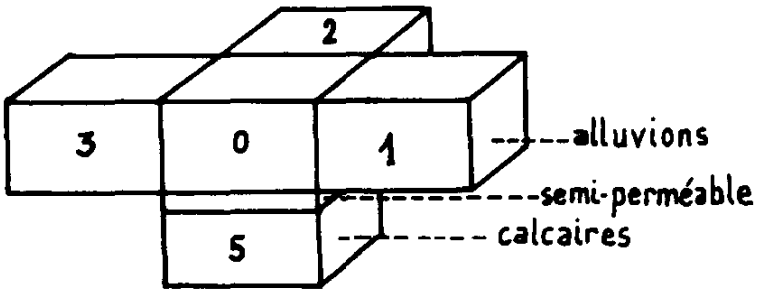


Fig. 7 : Une maille entourée de ses voisines (sauf une).

Le flux entrant par une des faces est : $\Phi = - \iint \vec{u}.\vec{n}\ ds$ ($\vec{n}$: normale à la face, orientée vers l'extérieur du volume).

$$\Phi = \iint_{\text{face}} K.\ \vec{\text{grad}}\ h.\ \vec{n}.\ ds = \int_{\text{largeur de la maille}} T.\ \vec{\text{grad}}\ h.\ \vec{n}\ dl.$$

La discrétisation se fait :

- en approximant la valeur de la transmissivité sur la surface de contact entre les mailles o et i :

$$T_{oi} = \frac{2\ T_o\ T_i}{T_o + T_i}$$

- en approximant $\vec{\text{grad}}h.\ \vec{n}$ par : $\dfrac{H_i - H_o}{a}$.

- en approximant le flux entrant par la face 0 - 5 par :

$$\Phi = K.\ \frac{H_5 - H_o}{e}\ \times a^2,$$ K étant la perméabilité du semi-perméable et e son épaisseur.

D'où l'équation de la diffusivité discrétisée :

$$(4)\quad \boxed{\sum_{1}^{4} T_{oi}\ (H_i - H_o) + \frac{Ka^2}{e}\ (H_5 - H_o) - Q = a^2\ S\ \frac{\Delta H_o}{\Delta t}}$$

On voit, que dans le cas d'un régime permanent et d'une nappe captive, on est ramené à la résolution d'un système d'équations linéaires, dont les inconnues sont les H_i. Dans le cas d'une nappe libre, la résolution est plus compliquée, du fait que les T_{oi} varient avec les H_i ($T = K\ (h - \sigma)$).

Dans la pratique, le modèle permet de juxtaposer, outre des mailles de même taille, des mailles de dimension double ou moitié. On pourra par exemple avoir la disposition suivante: (Figure 8).

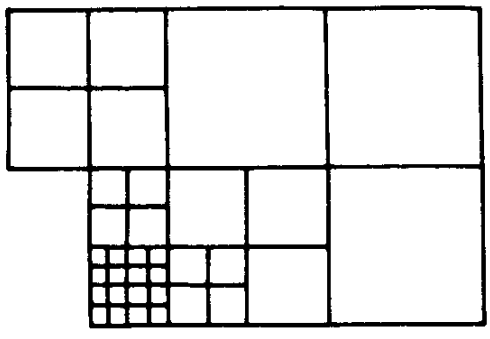


Fig. 8 : Exemple de maillage.

Dans notre modèle, le plus grand rapport possible entre les extrêmes des dimensions de mailles était de 8 (le coté de la plus grande maille du modèle égale 8 fois celui de la plus petite).

3 - Délimitation du domaine d'étude - Conditions aux limites

La finesse de calcul que nous recherchions aux abords de la mine nous a fait choisir comme dimension des plus petites mailles du modèle : 50 m de côté. Ceci nous interdisait de modéliser l'ensemble de la nappe, et il nous a fallu délimiter un domaine qui soit à la fois :

- assez grand pour pouvoir considérer que le fonctionnement de la nappe aux limites du modèle n'est pas influencé par le pompage dans la mine.
- assez restreint pour que le modèle ne soit pas trop lourd à manipuler.

Le domaine choisi est représenté sur la carte suivante : (Figure 9).

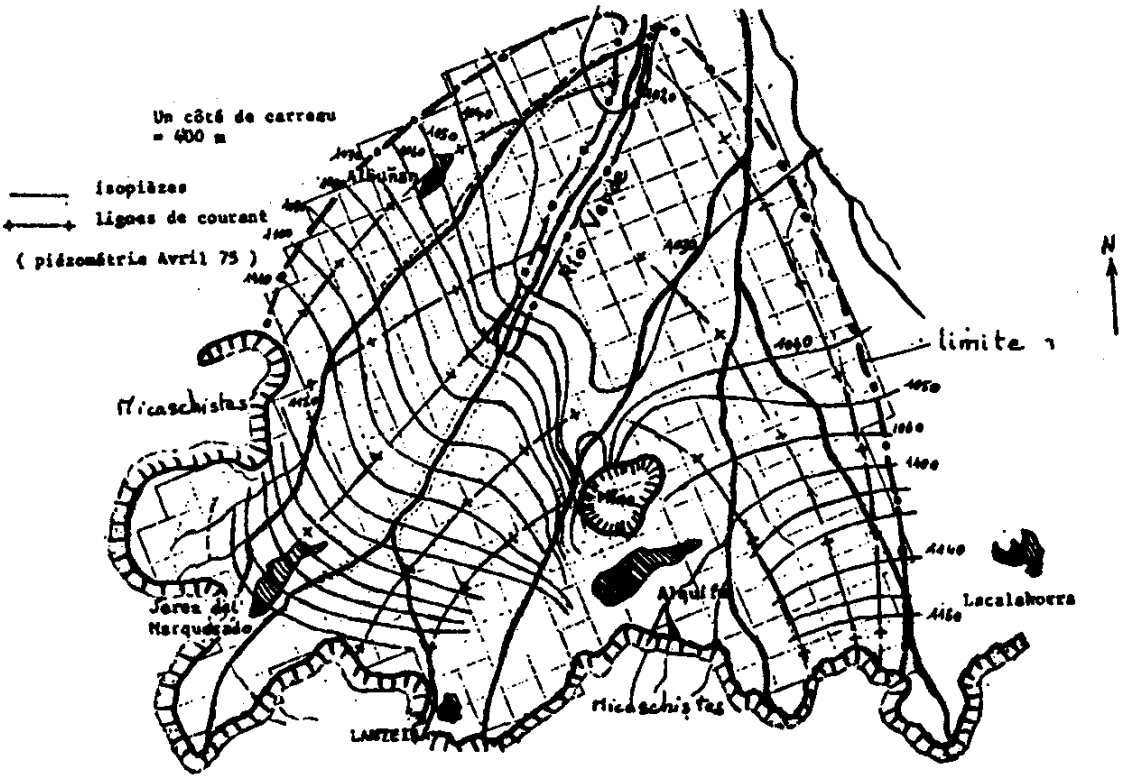


Fig. 9 : Carte du domaine d'étude

Nous sommes en présence de conditions aux limites de deux types :

* des limites à flux nul :
  - le long de la limite d'affleurement du substratum imperméable (ce qui est la limite physique de la nappe)

- le long des limites Est et Ouest, que nous avons tracées perpendiculairement aux lignes isopièzes, en supposant que la direction de ces lignes n'est pas influencée par le pompage dans la mine.

" une limite à potentiel imposé qui figure la drainance entre la nappe des alluvions et la nappe alluviale du Rio Verde, ainsi que les sources qui jalonnent la vallée de cette rivière, l'ensemble constituant l'unique exutoire naturel de la nappe dans notre domaine.

Nous avons donc établi un bilan de la circulation d'eau dans notre domaine, bilan dont nous avons ensuite réparti les différents termes sur les mailles de notre modèle (Figure 10).

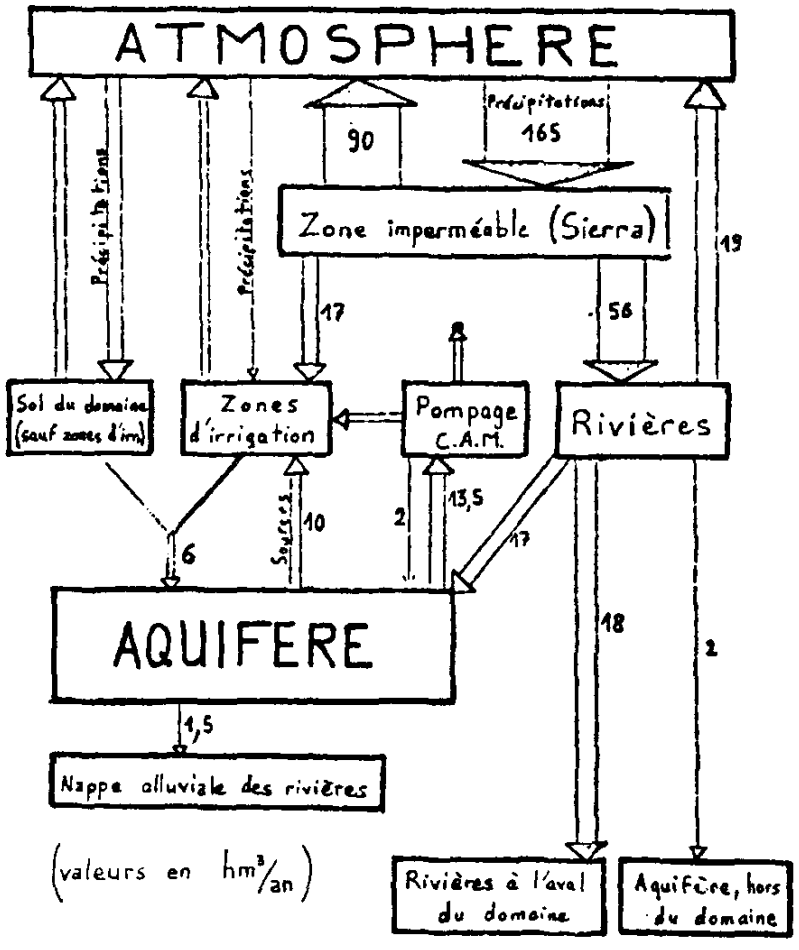


Fig. 10 : Les flux d'eau concernant le domaine choisi.

4 - Calage du modèle - Situation de référence - Solutions

L'étape suivante consistait à "caler" le modèle, c'est-à-dire à trouver les valeurs inconnues des transmissivités de la nappe, en simulant un état de la nappe dont la carte piézomètrique ainsi que les débits entrant et sortant sont connus. Ceci se fait "à la main" et est une opération assez longue. Le modèle

une fois calé, il restait à mettre en place notre état initial, c'est à dire à simuler le détournement de la Rambla de Lanteira en supprimant les apports à la nappe que représentent les infiltrations dans le lit de cette rivière. Il en est ressorti qu'à par un rabattement d'une dizaine de mètres, le détournement aura pour l'effet d'infléchir vers la mine l'écoulement de la nappe et d'y faire converger les infiltrations de zones assez éloignées. (La figure 11 montre cette inflexion des lignes de courant).

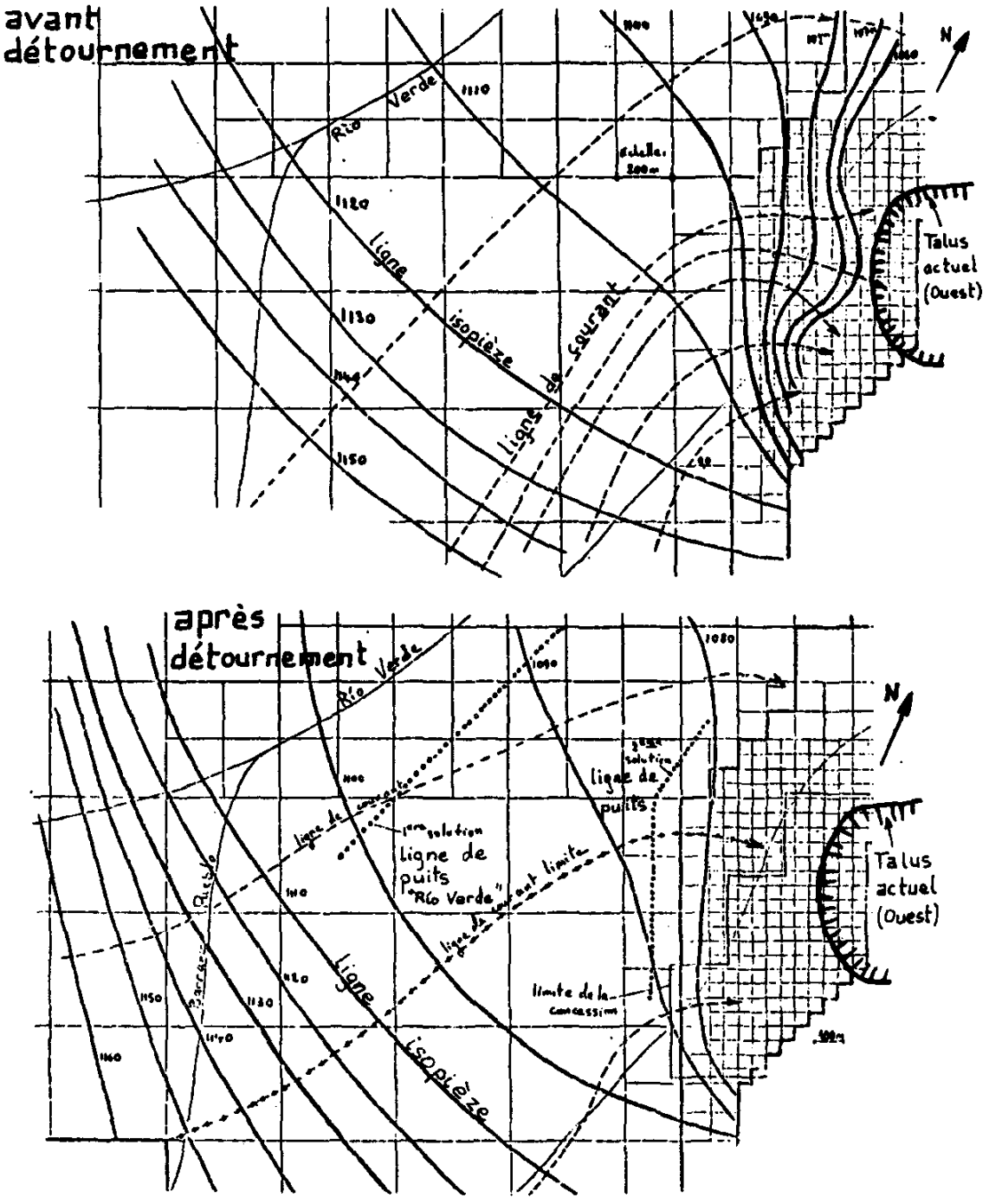
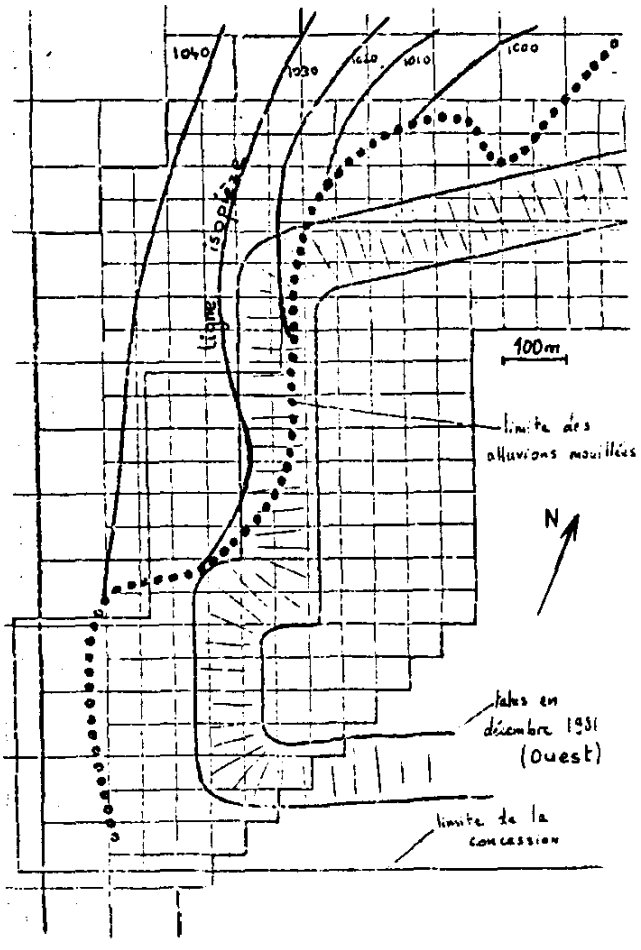


Fig. 11 : Carte piézomètrique avant et après détournement.

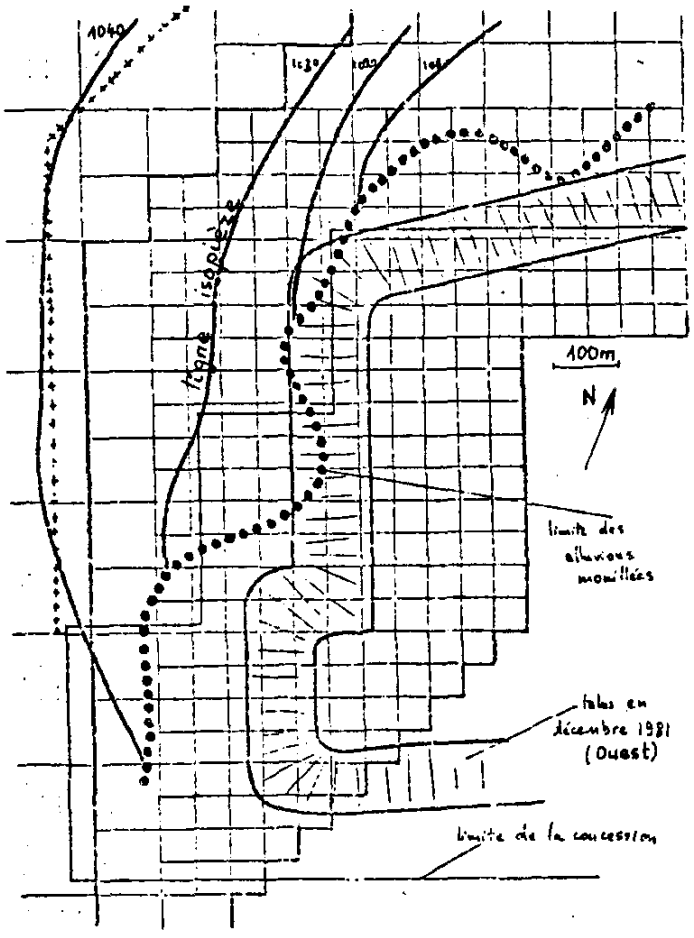
D'où deux idées pour la solution de notre problème :

- barrer la route à ces nouveaux apports en rabattant la nappe à la limite de la concession C.A.M., soit avec une série de drains verticaux se déversant dans une galerie, soit avec une ligne de puits équipés d'une pompe.
- rendre à l'écoulement sa direction initiale, en rabattant la nappe sur le chemin initial de ces apports, loin de la carrière, à proximité du Rio Verde par exemple. Pratiquement, cela reviendrait à installer une ligne de puits parallèle au Rio Verde, non loin de son cours.

Notre modèle nous permet de connaître, en fonction du débit retiré de tels ouvrages, l'ampleur du rabattement obtenu dans la zone de la mine, ainsi que la limite des alluvions mouillées. (Figure 12)



Résultat d'une exhaure de 180 l/s près de Rio Verde.



Résultat d'une exhaure de 180 l/s près de la carrière.

Fig. 12 : Deux exemples de résultats donnés par le modèle (Talus Nord-Ouest).

Nous avons ainsi calculé dans les deux cas les débits nécessaires pour le dénoyage des alluvions à moyen terme (échéance 1982) ainsi qu'à long terme, c'est-à-dire pour dénoyer les alluvions jusqu'à la limite de la concession. L'utilisation du modèle s'arrête ici : il s'agit ensuite de calculer le nombre de puits ou de drains nécessaires pour assurer le dénoyage à moyen et à long terme; d'évaluer le coût (installation, fonctionnement) d'un éventail de dispositifs pratiques afin de trouver l'optimum.

III - LE PROBLEME DE LA TENUE DES PAREMENTS ET LA SYNTHESE ECONOMIQUE

Pour donner à la C.A.M. les moyens d'une décision dans le choix entre dénoyer les alluvions et admettre une pente plus faible pour les talus, il fallait compléter les résultats de l'étude hydraulique par une étude de stabilité et faire une synthèse économique de l'ensemble.

Nous avons cherché à comparer 3 options pour l'exploitant :

- s'imposer de ne jamais exploiter avant d'avoir complètement dénoyé les alluvions, et en supporter les conséquences, tant en ce qui concerne l'avancement de la découverture que les dépenses nécessaires au dénoyage (c'est ce qu'on fait jusqu'à présent) : c'est la "solution 1", qui suppose qu'en fin d'exploitation, on tire de l'ordre de 300 l/s du dispositif d'exhaure.
- renoncer à dénoyer, ne réaliser aucun investissement en ce sens, mais diminuer suffisamment la pente des parements pour qu'ils restent stables malgré la présence d'eau, et décaper de ce fait plus d'alluvions d'ici la fin de l'exploitation : c'est la"solution 0".
- adopter un moyen terme entre ces deux extrêmes, c'est-à-dire laisser 7 à 8 ha non dénoyés dans la concession, avoir à installer un dispositif d'exhaure moins important (220 l/s) que pour la "solution 1", et décaper moins d'alluvions que dans la solution 0 : c'est la"solution 1/2".

Il fallait, dans chaque cas, déterminer l'angle du talus en fin d'exploitation, en tenant compte, pour les solutions 0 et 1/2, de la présence d'eau dans le talus, c'es-à-dire des changements de cohésion et d'angle de frottement entre alluvions sèches et alluvions mouillées, ainsi que des poussées d'écoulement et poussée d'Archimède.

Nous avons fait les calculs sur la base de données géotechniques provenant d'essais réalisés par le bureau d'études INTECSA, par l'école des Mines de Madrid, et par l'école des Mines de Paris, sur alluvions sèches et saturées (essais à la plaque et essais de cisaillement) suivant les 4 méthodes suivantes :

- méthode de Bishop
- méthode de Fellenius
- abaques de Hoek
- méthode des éléments finis, avec un modèle élastoplastique du matériau.

Ces calculs n'intéressent que d'assez loin ce symposium, et nous ne les exposerons pas. Disons simplement qu'ils nous ont permis de calculer l'angle à donner aux talus dans chacune des trois solutions, et, partant, le volume d'alluvions à décaper : par rapport à la solution 1, les quantités supplémentaires d'alluvions à décaper d'ici la fin de l'exploitation sont :

- pour la solution 1/2 : 10 Mt.
- pour la solution 0 : 50 Mt.

Un point est néanmoins à préciser : le choix d'un angle-limite pour un talus, qu'il soit sec ou mouillé, contient nécessairement une part d'arbitraire. En effet, la démarche est la suivante : on schématise le talus réel par un modèle où on fait les hypothèses simplificatrices (homogénéité, isotropie ...) et où on donne au matériau des caractéristiques déduites d'un petit nombre d'essais; puis on tient compte de l'écart entre modèle et réalité en prenant pour talus-limite, non pas celui où les contraintes calculées égalent les contraintes à la rupture, mais un talus où elles se trouvent à un niveau inférieur. Tout le problème consiste alors à situer ce niveau; or c'est vouloir quantifier ce qui par définition, nous est en partie inconnu; d'où le côté arbitraire de la méthode.

Ces remarques sont particulièrement vraies pour les alluvions mouillées : aux simplifications déjà assez grossières du calcul d'un talus sec :

- homogénéité, alors qu'il s'agit d'un des matériaux les plus hétérogènes qui soient.
- isotropie (Hoek; Fellenius, Bishop) alors que l'hypothèse d'orthotropie paraît bien plus pertinente,

s'en ajoutent d'autres, tout aussi grossières, concernant l'eau :

- écoulement unidirectionnel dans le talus, alors que toutes les expériences montrent une courbure tendant à lui faire tangenter le parement. (Figure 13).

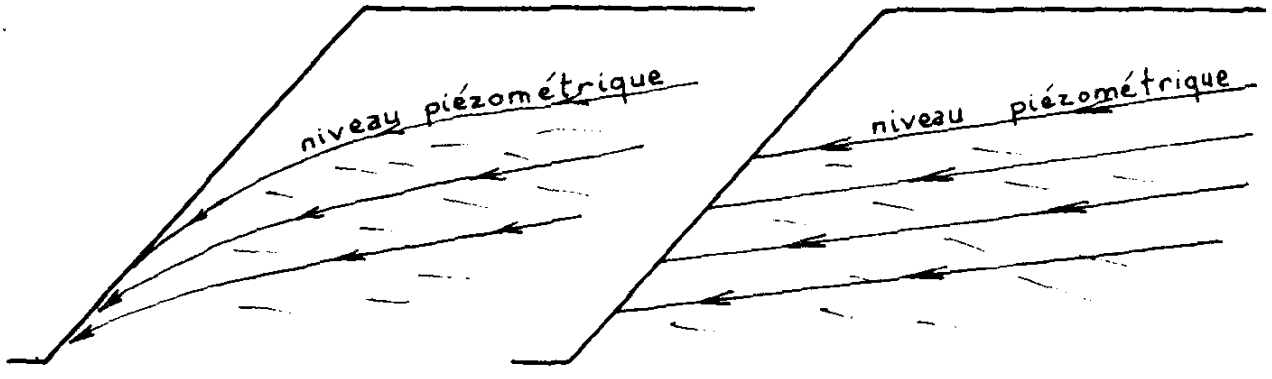


Fig. 13 : Ecoulement réel et modèle pris en compte dans les calculs.

- écoulement uniforme, alors que, de toute évidence, l'écoulement dans les alluvions suit préférentiellement les lits à gros grains, tend à colmater les lits à granulomètrie étalée, etc...
- changement de cohésion (dû à la présence d'eau) uniforme dans le massif, alors que certains lits - argilo-limoneux perdent toute leur cohésion et s'écoulent sous leur propre poids lorsqu'ils sont imbibés d'eau, tandis que pour d'autres la cohésion ne change guère.
- mêmes remarques pour la résistance à la traction.

Pour pallier à cet inconvénient, nous avons envisagé de modéliser le talus par un ensemble de plusieurs matériaux (3 dans notre exemple) disposés en lits horizontaux superposés, aux caractéristiques hydrauliques et géotechniques différentes. Il s'agit alors de distinguer dans le talus 3 (ou n) matériaux-types, aux couches granulomètriques bien différenciées, de définir par essais leurs caractéristiques avec et sans eau, ainsi que leur perméabilité verticale et horizontale, d'établir une coupe du talus en rattachant tous les lits trouvés à l'un de ces 3 (ou n) types, afin d'en calculer les proportions respectives et de les introduire dans le modèle. A titre d'exemple, nos 3 matériaux-types avaient les caractéristiques suivantes :

	TERRAIN SEC	TERRAIN SATURE
matériau 1 γ = 2,25 kg/l	c = 0,5 bar ϕ = 32° R_T = 0,4 bar	c' = 0,3 bar ϕ' = 32° R_T = 0,2 bar
matériau 2 γ = 2,2 kg/l	c = 0 ϕ = 30° R_T = 0	c' = 0 ϕ' = 30° R_T = 0
matériau 3 γ = 1,7 kg/l	c = 2 bars ϕ = 20° R_T = 0,5 bar	c' = 0 ϕ' = 20° R_T = 0

Il s'agit d'un modèle plan vertical aux éléments finis qui permet, dans un premier temps, de calculer les caractéristiques de l'écoulement dans le talus ainsi schématisé, d'en déduire les poussées d'écoulement et d'Archimède, et de les introduire dans un calcul de stabilité en élastoplasticité.

Malheureusement, le temps imparti à notre étude ne nous a pas permis de traiter suffisamment d'exemples de cette méthode, qui nous semble néanmoins prometteuse.

IV - CONCLUSION

Grâce aux résultats de l'étude hydraulique et de l'étude géotechnique, il devint possible d'esquisser une comparaison économique des solutions 0, 1/2 et 1. Elle nous fit retenir la solution 1 (dénoyage à tout prix) et le dispositif suivant, installé à la limite de la concession : (Figure 14)

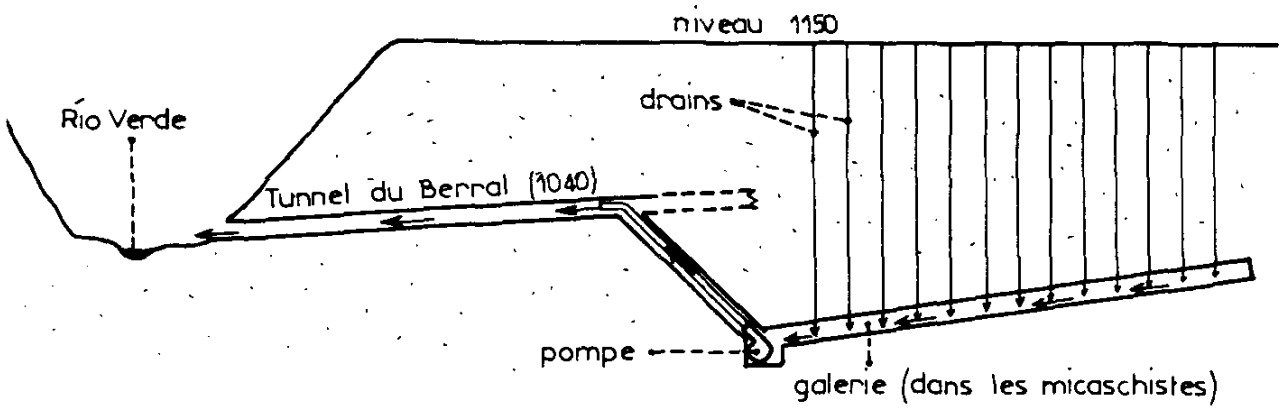


Fig. 14 : Schéma du dispositif d'exhaure préconisé. (Le tunnel du Berral a été construit en 1963 et sert déjà à l'exhaure.

Le coût de l'opération serait, en ordres de grandeur :

- investissement : 6,5 MF (1976)
- dépenses de fonctionnement : 0,8 MF/an

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APPLICATION OF A SIMULATION MODEL FOR A LARGE-SCALE KARSTIC WATER AQUIFER

Szilágyi, G.* ; Heinemann, Z.** and Bogárdi, I.*

ABSTRACT : Mining activity, water management and the environment is interrelated in a regional karstic water aquifer. A proper operation of existing works and the optimal development requires the knowledge of water movement in that aquifer. The simulation nethod elaborated is based on a finite difference solution of the two-dimensional, unsteady flow equation. Inputs of the model are rainfall over the area and water intakes at different points. Output yields the piezometric heights in grid points, and system outflow. The simulation model can be connected to a decision model which helps to choose the best alternatives for mine water management, water supply and environmental protection.

RESUME : Les travaux miniers, les conditions d'utilisation des eaux souterraines et l'environnement sont en inter-relation pour un aquifère régional d'eau karstique. Pour diriger convenablement les travaux entamés et pour assurer leur progrès optimal, on doit connaître les facteurs influant sur l'écoulement des eaux endedans de cet aquifère. La méthode de simulation décrite, mise au point à cet effet, est basée sur la solution par la méthode des différences finies d'une équation bi-dimensionnelle d'écoulement non-continu. Pour le modèle employé les entrées sont constituées par le volume des précipitations affectant le territoire et celui des eaux drainées en différents points, tandis que les sorties consistent en hauteurs piézomètriques dans les points de réseau et volumes écoulés à partir du système.

RESUMEN : En los acuíferos kársticos regionales las actividades mineras, y la gestión del agua y del ambiente están muy interconectadas. Para enfocar convenientemente todos los trabajos, y conseguir un desarrollo óptimo, se necesita conocer los factores que condicionan el flujo del agua subterránea. El método de simulación descrito se basa en una solución, en diferencias finitas, de la ecuación bidimensional de flujo transitorio. Las entradas son los valores de la precipitación sobre el área y las aguas infiltradas en diferentes puntos. Las salidas son los niveles piezométricos en los nudos considerados y los caudales transferidos a partir del sistema. Este modelo puede conectarse a un modelo de decisión, lo cual ayuda a encontrar las mejores alternativas para la gestión del agua de la mina, los abastecimientos y la protección ambiental.

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1. INTRODUCTION

The karstic aquifer in the Hungarian Transdanubian Mountain /TM/ has increasingly been affected by regional mining activity, water supply withdrawals and thermal water recharge. As a result of man-made interventions, level of the karstic water has been sinking. This paper underlines the necessity of using a simulation model /SM/ being capable of forecasting and interacting with a decision model. Several similar models are known from literature [De WIEST, 1965; WITHERSPOON and FREEZE, 1972; BREDEHOEFT and YOUNG, 1970; NEUMAN, 1975; KISIEL and DUCKSTEIN, 1976; HELWEG and LABADIE, 1977; PINDER and FRIND, 1972], though rarely are they used for karstic aquifers [YEVJEVICH, 1976]. The main idea of this paper is to show that the elaboration and direct application of such a model is not necessarily an end in itself but a further objective is to incorporate it into a multiobjective decision model /MDM/. This MDM would help decision-makers to find the best compromise among the various, sometimes conflicting goals of karstic water utilization, constrained by the physical laws governing karstic water movement. Thus, the model discussed in this paper is regarded as an integral part of a multiobjective analysis.

In Section 2 present situation of the aquifer is evaluated, as compared to the earlier, undisturbed state.

The aquifer system is described in Section 3, then Section 4 shows the development of the mathematical model.

Application and on-going improvement of the model is discussed in Section 5. Also, conclusions reached so far are summarized.

2. PRESENT SITUATION

A great part of Hungarian mineral resources /coal, bauxite, manganese, oil/ are located in the vicinity or directly in the main karstic aquifer system of the TM /Fig. 1/. Here, mining activity under water hazard requires karstic water withdrawals. As a result of mine water pumping /Fig. 2/ original hydrologic conditions in the aquifer have gradually distorted. Since the natural recharge volume is finite and rapidly approached by the withdrawals, further interventions should be carefully planned with the forecast of ensuing impact.

It is a further problem that world-famous thermal baths in Budapest and Héviz receive natural recharge also from that aquifer. As a consequence, further distortion of hydrologic conditions will result not only in a new /lower/ level equilibrium but the reduction of thermal water recharge.

As a third main factor, the use of karstic water for regional water supply complicates the situation. In fact, growing water demands /drinking and industrial/ call for the greater use of

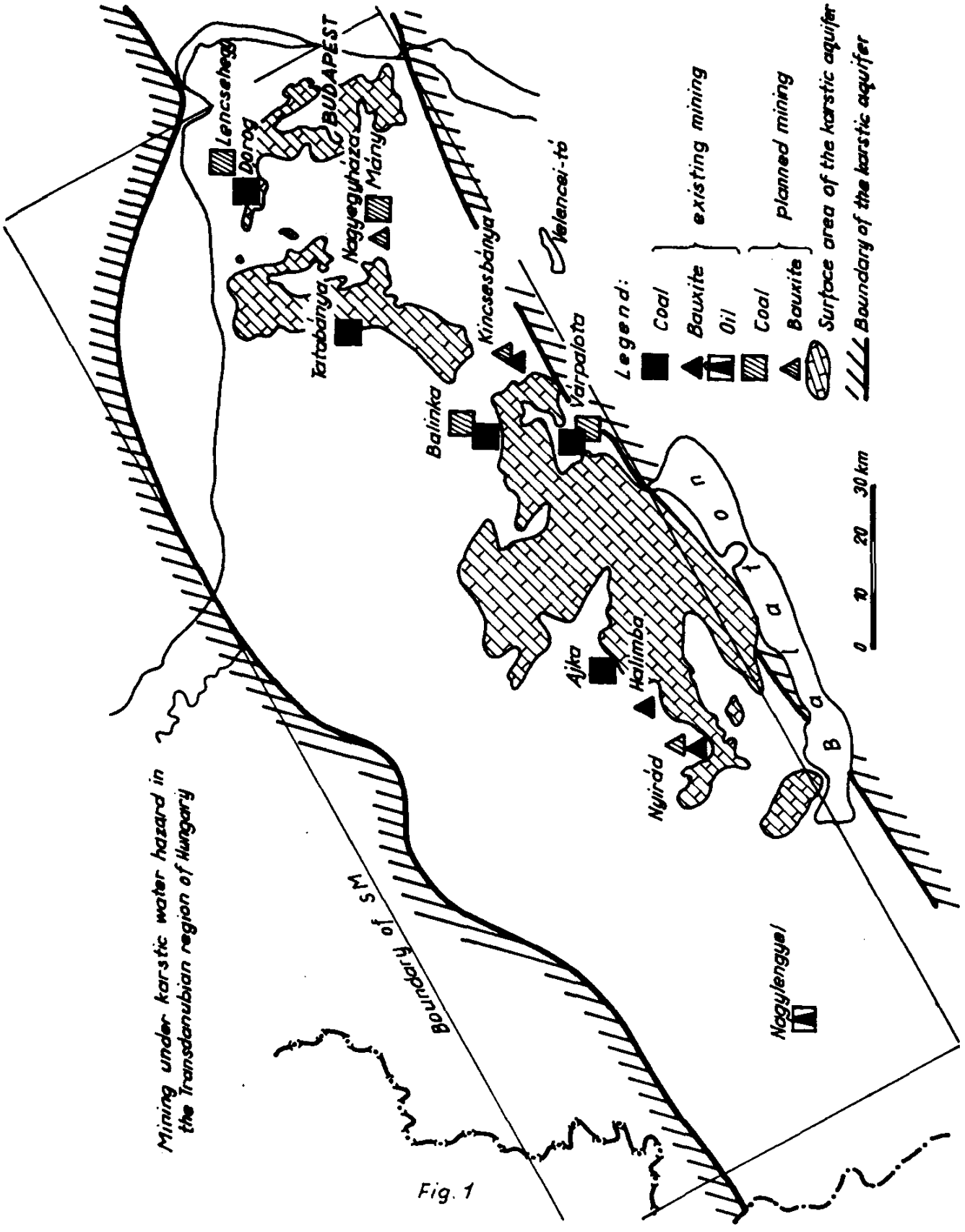
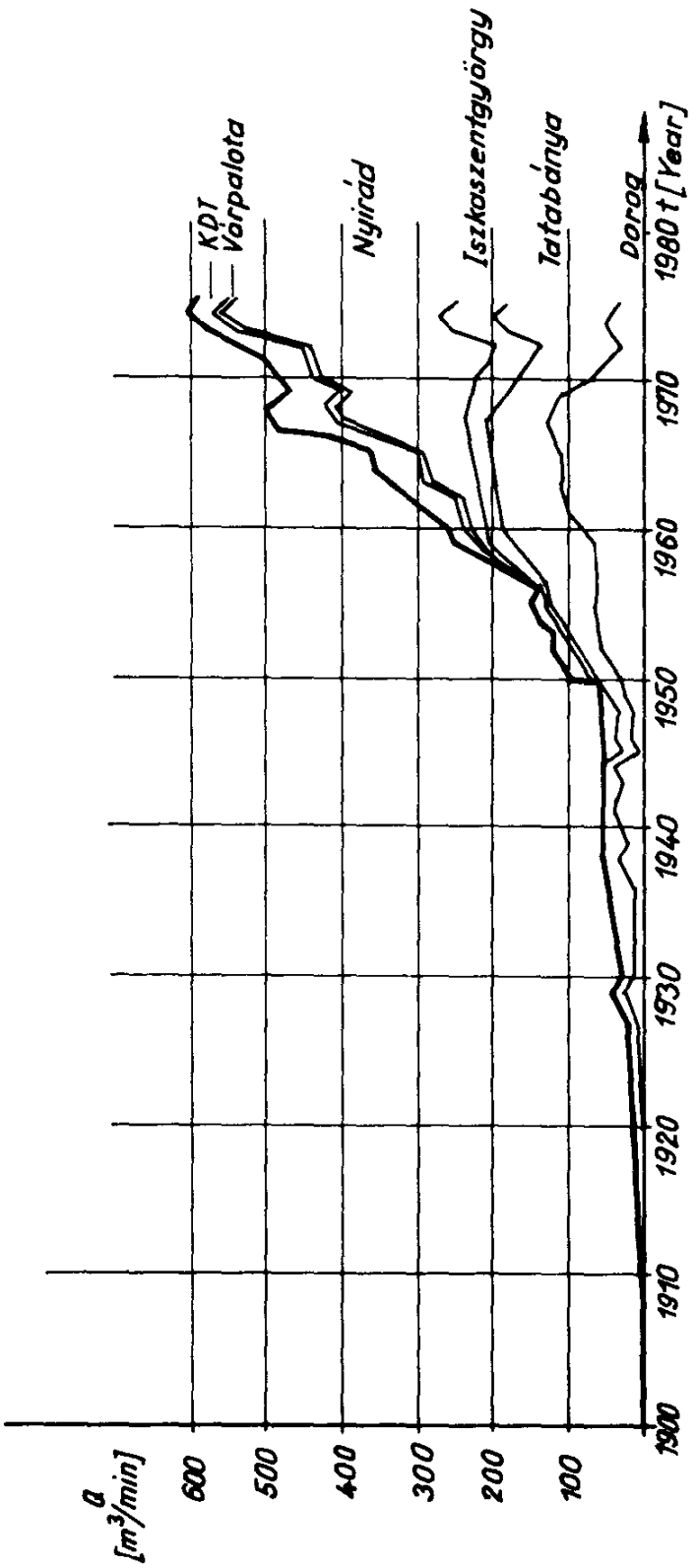

Mining under karstic water hazard in the Transdanubian region of Hungary
Boundary of SM
BUDAPEST
Lencsehegy
Dorog
Nagyegyházas
Tatabánya
Kincsesbánya
Velencei-tó
Balinka
Várpalota
Ajka
Halimba
Nyirád
Nagylengyel
Legend:
Coal
Bauxite
existing mining
Oil
Coal
Bauxite
planned mining
Surface area of the karstic aquifer
Boundary of the karstic aquifer
0 10 20 30 km

Fig. 1



Total mining withdrawals in the region

Fig. 2

the karstic aquifer, more or less the only potential water resources in the region.

Recently explored new mineral deposits in the region are located deeply under karstic water level. Mining of these minerals becomes necessary in order to meet national demands [KAPOLYI.1977]. As a result, further increase of mine water withdrawals is envisaged [SCHMIEDER et al., 1977], which may surmount natural recharge.

In this situation the need for a proper control is evident. Two main engineering alternatives are available to find a "satisfactum" among these conflicting effects:

a./ using such mining technologies under water hazard /e.g. sealing/ which result in smaller withdrawal;

b./ artificially recharging the pumped mine water into the aquifer in proper time and space in order to maintain the necessary underground recharge of thermal waters. A multiobjective analysis of these alternatives can be found in DUCKSTEIN et al. [1978]. Clearly, traditional design methods are not suitable for selecting the site, time and amount of artificial recharge. In order to find the "best" control strategy such a physical model is necessary which

a./ yields regional karstic water level fairly accurately, as affected by natural recharge /rainfall/, mining and water-supply withdrawals as well as artificial recharge;

b./ calculates underground outflow /recharge of thermal waters/ fairly accurately as related to the above variables;

c./ needs moderate computer time for the solution and can be connected to a decision model.

Model construction requires a proper knowledge of the system which is briefly described in the next Section.

3. SYSTEM DESCRIPTION

Average natural recharge of the karstic aquifer system in the TM is estimated as 700 m³/min [SZILÁGYI, 1976]. This recharge stems from rainfall over an area of 1500 km² and the average infiltration ratio is 36 %. In its original, undisturbed state, outflow from the system has the following distribution:
36 % cold-water springs, 28 % lateral seepage toward porous basins encircling the TM, 17 % surface waters, 10 % thermal water recharge and 9 % seepage into the Danube.

At present, outflow distribution has completely changed:

Average total outflow has increased to 850 m³/min, thus the average storage loss is 150 m³/min. Outflow distribution: withdrawals: 65 %, springs: 10 %, lateral seepage: 10 %, surface waters: 3%, thermal water recharge: 8 %, Danube communication: 4 %.

The loss of karstic water resources covers almost the entire TM, and there is a continuous sinking trend in regional karstic water level, with "noises" due to random hydrologic effects /e.g. rainfall/ and changing actual amount of withdrawals. Fig. 3 compares karstic water levels with and without man-made effects for a characteristic section.

These significant changes have contributed to acquiring more knowledge on the structure, hydraulic properties of, and main processes in the system as well as on the connection with adjacent aquifer systems [SCHMIEDER, 1975]. That knowledge is a prerequisite to construct any kind of system model with the criteria specified earlier. However, information should be increased on that part of the TM which is not yet affected by man-made interventions. It is hoped that the SM outlined in the next Section will also help in that respect.

4. THE MATHEMATICAL MODEL /SM/

The mathematical model is now described for the calculation of karstic water flow in the main aquifer of the TM. Details of the model can be found in HEINEMANN and SZILÁGYI [1976].

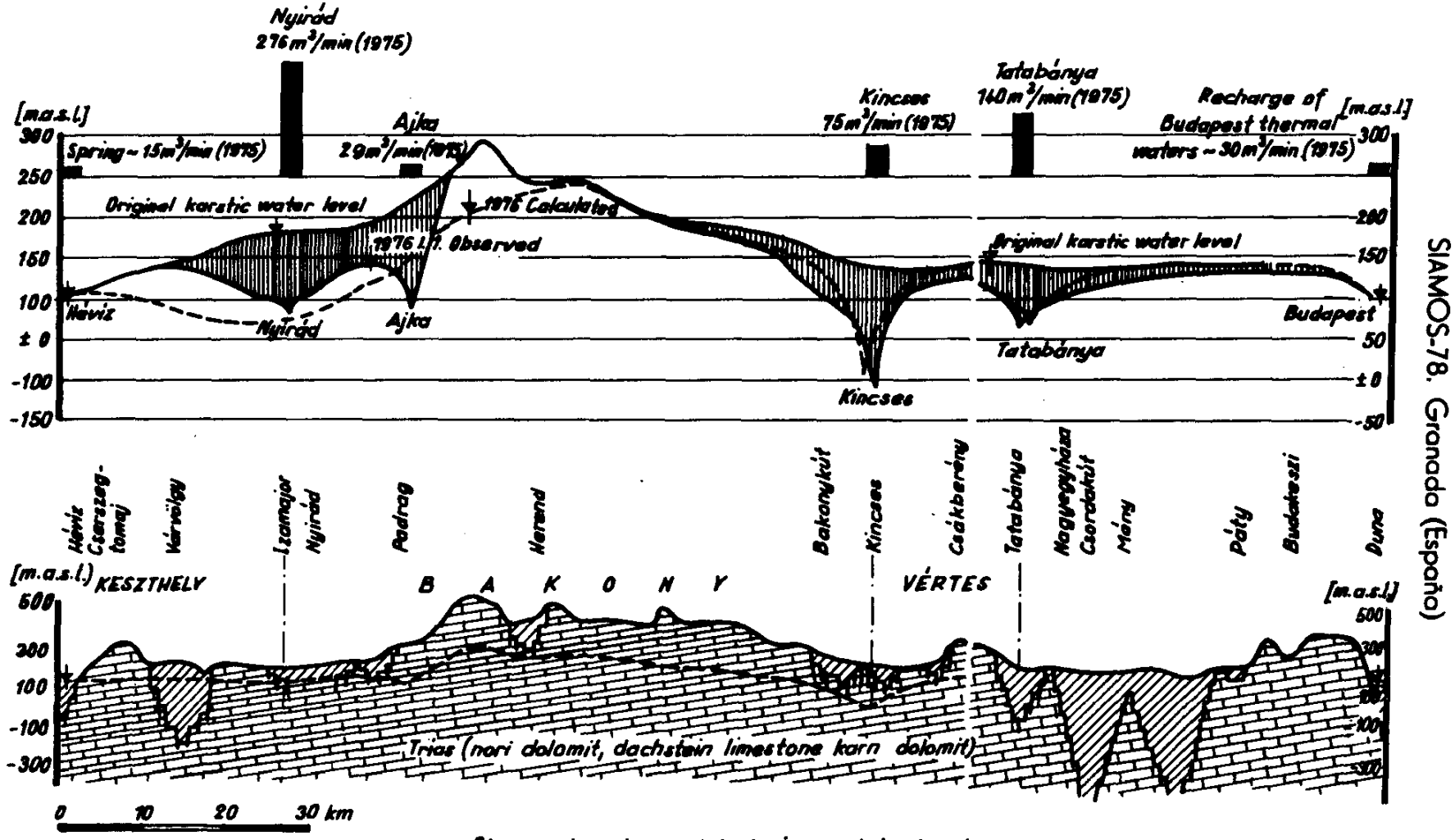
4.1. Principle of the model

It has been demonstrated that percolation in that regional karstic aquifer can be explained well by laminar motion and a mixed seepage /free surface and confined/ [SCHMIEDER, 1970]. Thus, seepage flow can be characterized in the karstic aquifer by the well-known two dimensional differencial equation for unsteady state [KISIEL and DUCKSTEIN, 1976]:

$$S \frac{\partial h}{\partial t} = \frac{\partial}{\partial x}\left(K_{xx} \frac{\partial h}{\partial x}\right) + \frac{\partial}{\partial y}\left(K_{yy} \frac{\partial h}{\partial y}\right) + q - w \qquad (1)$$

where: $h(x,y,t)$ is the piezometric level,
$K(x,y)$ is the aquifer conductivity,
$S(x,y)$ is the aquifer storage coefficient,
$q(x,y,t)$ is the natural recharge,
$w(x,y,t)$ is the withdrawal.

It is assumed that the aquifer is incompressible and anizotropic in the x and y directions. Along the vertical an average conductivity is taken into account. Water is regarded incompressible and of constant viscosity.



Observed and calculated piezometric level

Fig. 3

Equation 1 shows that seepage is induced in the aquifer by natural recharge $q(x,y,t)$ and withdrawal $w(x,y,t)$.

4.2. Initial and boundary conditions
Solution of equation 1 requires initial conditions $h(x,o) = \xi(x)$, and boundary conditions: $h(x,t) = \eta(x,t)$. For the calibration and validation of the model initial conditions were taken from BÖCKER and MÜLLER [1958-1974], who reported water level data in the karstic aquifer every year.

As boundary conditions two types are considered /Fig. 4/:

a./ erosion basis of the main karstic aquifer that is, springs and infiltration along the boundary as constant pressure points;

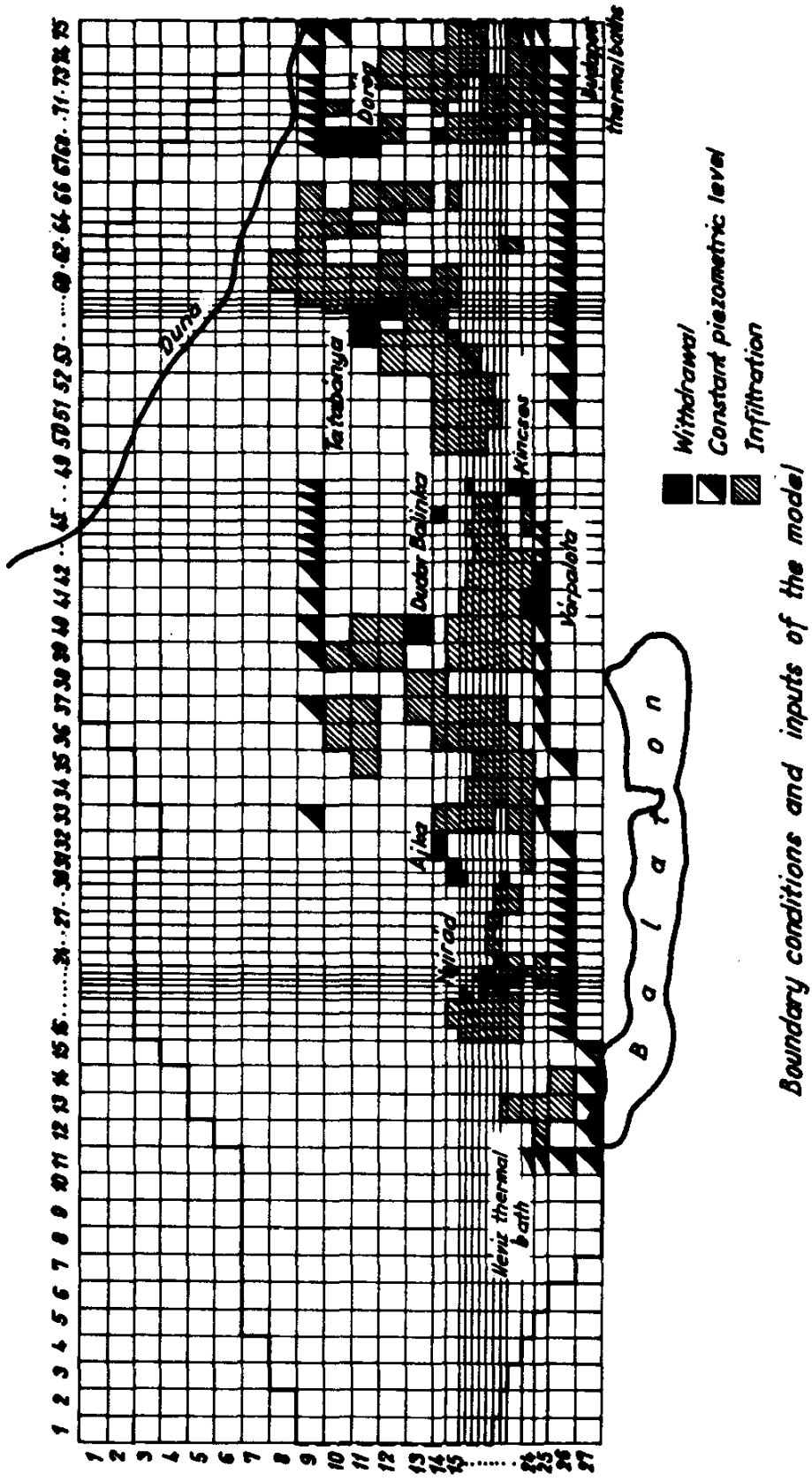
b./ other parts of the boundary are closed.

4.3. Model parameters
Determination of aquifer properties - conductivity and storage coefficient - for equation 1 means one of the most critical phases in such problems. Initial values of these parameters for different locations have been estimated on the basis of past mining withdrawal observations, numerous pumping tests and hydraulic analyses [SCHMIEDER, 1975]. Initial values were gradually improved during calibration.

4.4. Model solution and output
A finite difference method was used to solve equation 1 [HEINEMANN and SZILÁGYI, 1976]. Variable grid and a time step of 1 year was used. Solution yields for any time piezometric levels in the grid points, and system outflows along the boundary. The grid system shown in Fig. 4 has been gradually developed and contains 2025 blocks. First, smaller parts of the system was modelled [HEINEMANN and SZILÁGYI, 1976] and using that experiences the model was enlarged. In that present form of the modelled system such areas are also included which are, and probably will be undisturbed from mining interventions. The reason of such enlarging is that boundary conditions for the present system can be more truly modelled than for the smaller one.

4.5. Model calibration
Differences between model results - from different 1 year runs - and observations were reduced to ± 10 meters by iterative corrections of conductivity values. Over the best-explored areas, covering the vicinity of mine withdrawals, these corrections were relatively slight, less than one order of magnitude. However, in the northern part of the region, at some locations corrections of two orders of magnitude became necessary.

Not only corrections in conductivity values were required during the calibration process but modifications of inputs and boundary



Boundary conditions and inputs of the model

Fig. 4.

conditions were also necessary at some points, by
- transferring points with constant piezometric level along the boundary;
- changing inner impervious zones, and
- increasing infiltration surfaces.

4.6. Model validation
After the calibration with 1 year runs a further 20 year period was simulated and compared with observations. During this period already heavy mining withdrawals have been occurring. It was concluded that
- calculated values simulate the observed values in tendency fairly well,
- there are still considerable deviations at some places, and
- a quasi-steady state is reached with the calculated outflows equalling those determined by other methods.

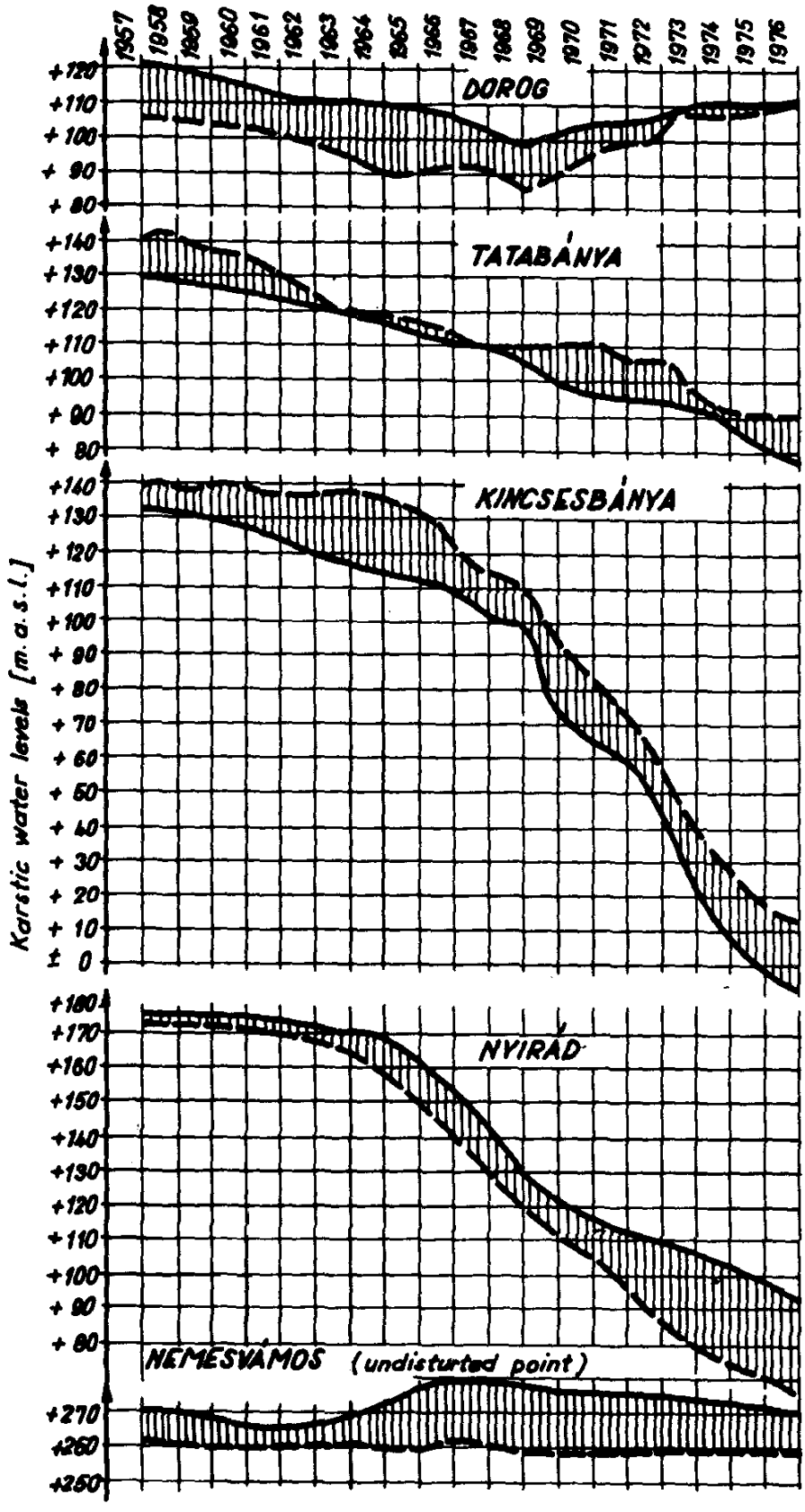
5. EXPERIENCES AND CONCLUSIONS OF SM APPLICATION
Results of the first application of SM are illustrated in Figs. 5 and 6. Direct check is possible when measured and calculated karstic water levels are compared. On the other hand, a part of outflows cannot be observed /lateral seepage, communication with the Danube/ but can be estimated by water-budget methods [SZILÁGYI, 1976; KOVÁCS, 1977]. As a result, comparison with the SM values is relative in these cases. An other part of outflows can be observed as discharge of springs and thermal wells.

Figs. 5 and 6 show that there are differences between calculated and observed or derived values. These differences are caused by model uncertainties and parameter uncertainties. Model uncertainties involve the inaccuracy of finite difference scheme /size of mesh/, and the modelled initial and boundary conditions. Parameter uncertainties involve the input error, mostly in rainfall infiltration, and the errors in rock properties. Further improvement in model accuracy can be expected if - in addition to the subjective calibration outlined - a calibration algorithm is used to estimate - in a sense at least - optimal initial and boundary conditions, inputs and rock parameters.

Finally, the implication of SM with MDM is briefly discussed. There are two possibilities to connect SM with MDM:

a./ For each decision alternative /amount of withdrawals and artificial recharges/ a system output /e.g. the amount of natural recharge toward thermal baths/ is calculated by the help of SM. Since MDM solution requires a number of such calculations, some efficient interactive operation of the SM is necessary as in HELWEG and LABADIE [1977].

b./ A regression model f is constructed by the help of a suffucient number of SM runs:



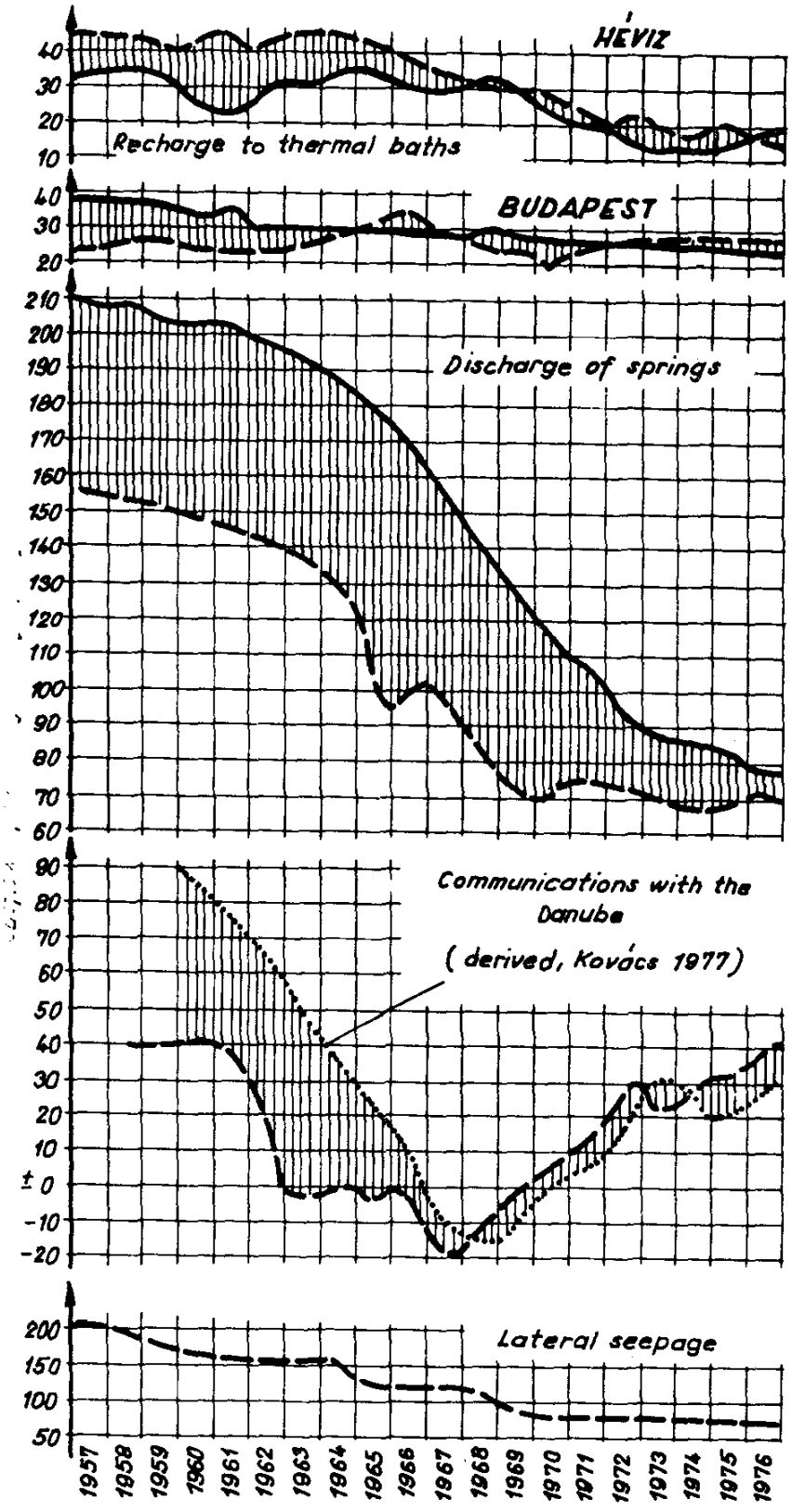
Legend:

Observed karstic water level

Calculated karstic water level

Observed and calculated Karstic water levels in specific points

Fig. 5



Observed and calculated natural outflows from the system

Fig. 6

$$y = f\left(x_1, x_2, \ldots x_n\right) + \underset{\sim}{\xi}$$

where $x_1, x_2, \ldots, x_n$ are values of the decision variables /withdrawals and artificial recharges/;
y is the system output in question, and
$\underset{\sim}{\xi}$ is the random error of the regression.
Then this regression equation can be directly inbedded into MDM as in DUCKSTEIN et al. [1978]. In that case there is no use to run SM in an interactive way with MDM, however the error ξ can be quite high unless a great number of SM runs is used for the estimation of regression model f. Also, the type of f - if it is not linear - may aggravete MDM solution.

In summarizing, the following conclusions can be drawn:
a./ In a regional aquifer, if several different objectives are present, a satisfactory water resources utilization policy can be found by the conjunctive use of an SM and a MDM.

b./ It is shown that an operational SM in this karstic aquifer can be based on the unsteady seepage equation elaborated for porous media.

c./ Due to model uncertainties and parameter uncertainties there are differences between observations and SM calculations, even after a proper calibration - on a subjective basis - and validation.

d./ SM can be connected to MDM by a direct interactive way or by the help of a regression model.

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THE ELEMENT BALANCE METHOD OF HYDROGEOLOGIC COMPUTATION

Wu, Ho

ABSTRACT : The element balance method is one of the numerical methods in solving the problem of planed non-steady flow of groundwater. The vadose region can be divided into several elements by this method. And then, based on the water balance theory the equations of every nodes could be set up directly using a physical model. The solution of two-dimensional partial differential equation of groundwater flow can be described using a discrete equation group which contains only limited numbers of unknown variables. The application of this method could not only give hydrogeologic parameters, but also predict the draining for mine and estimate the groundwater resource. The paper introduced briefly this method.

RESUME : La méthode du bilan d'éléments est une des méthodes numériques pour résoudre le problème d'écoulement transitoire horizontal, des eaux souterraines. Selon cette méthode, on divise la région des gués en plusieurs éléments. Puis, basés sur la théorie du bilan d'eau, et employant un modèle physique, on établit directement les équations des divers noeuds. La solution de l'équation différentielle partielle dimensionnelle de l'écoulement de l'eau souterraine, peut être décrite par un groupe discret d'équations, qui seulement contiennent un nombre limité de variables inconnues. Ainsi, non seulement on peut obtenir des paramètres hydrogéologiques, mais aussi on peut prédire le drainage des mines et calculer les ressources en eau souterraine. Le travail ci-après présente brièvement cette méthode.

RESUMEN : El método de balance de elementos, es uno de los métodos numéricos para resolver el problema de flujo transitorio horizontal, de las aguas subterráneas. Según este método, se divide la región vadosa en varios elementos. Luego, basados en la teoría del balance del agua, y empleando un modelo físico, se establecen directamente las ecuaciones de los diversos nudos. La solución de la ecuación diferencial parcial bidimensional del flujo del agua subterránea, puede ser descrita por un grupo discreto de ecuaciones, que sólo contienen un número limitado de variables desconocidas. Así, no sólo se pueden obtener parámetros hidrogeológicos, sino también predecir el drenaje de minas y calcular los recursos subterráneos. El presente trabajo expone brévemente este método.

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1. Introduction

The use of finite element method in hydrogeologic computation began in the late of 1960's. In recent years the study of the usefulness of finite element method in hydrogeologic computation has been carried out by many hydrogeologists at home and abroad (Chang Hun-ren, Li Tzuhn-ting and international well-known scientists S.P. Neuman; etc). They discovered many disadvantages contained in the finite element method, because of symmetric matrices of the storage. Therefore, the discussion of the usage of finite difference method in hydrogeologic computation for irregular grid became more and more fashion, but it is only limited in hydrogeologic computation of feeding water and the aquifer is considered isotropy. Refer to applying for the predict pit pumpage by these two methods of hydrogeologic computation in mining district, no papers have been published yet. The element balance method herein is based on the finite difference method for irregular grid and took in the reasonable form of finite - nent method. Thus, it is either used for hydrogeologic computation of feeding water or predict the pit pumpage. Also, it takes into account un-uniform medium of water-bearing bed and settles anisotropy of it.

2. Dividing the vadose region and establishing the model of water head.

Let's divide the vadose region and consider any plane element β and assume that its three point elements i, j, k, are anti-clockwise configuration (Fig 1). The value of surface h (x, y, t) of water head at the point element (L=i, j, k) is marked h_L (t). The coordinate of the point element L is (x_L, y_L). The linear interpolating function has to be done:

$$H^\beta (x, y, t) = ax + by + c$$

let

$$H^\beta (x_L, y_L, t) = h_L(t) \qquad L = i, j, k \qquad \cdots\cdots\cdots\cdots\cdots\cdots (1)$$

At the plane element β, the true function of water head can be approximated by the linear interpolation function H^β (x, y, t). The linear interpolating function H^β (x, y, t) is called the model of water head at β^{th} plane element.
From equation (1) we get:

$$H^\beta (x, y, t) = \sum_{L=i,j,k} \lambda_L h_L (t) \qquad \cdots\cdots\cdots\cdots\cdots\cdots (2)$$

Where:

$$\lambda_L = (\eta_L x - \xi_L y + \omega_L)/2\triangle_\beta, \quad L = i, j, k$$
$$\xi_i = x_j - x_k, \quad \xi_j = x_k - x_i, \quad \xi_k = x_i - x_j,$$
$$\eta_i = y_j - y_k, \quad \eta_j = y_k - y_i, \quad \eta_k = y_i - y_j,$$
$$\omega_i = x_j y_k - x_k y_j, \quad \omega_j = x_k y_i - x_i y_k, \quad \omega_k = x_i y_j - x_j y_i,$$
$$\triangle_\beta = (\omega_i + \omega_j + \omega_k)/2,$$

$\triangle_\beta$ is the area of the plane element β.

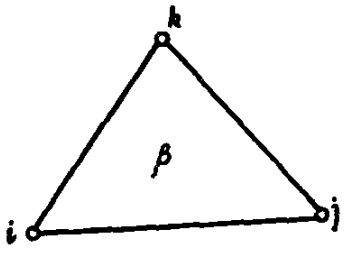

Fig 1

We assume the gradient of water head at direction x is $J_x^\beta(t)$ and one at direction y is $J_y^\beta(t)$, thus from equation (2) we obtain:

$$J_x^{\beta}(t) = \sum_{L=i,j,k} \eta_L h_L(t)/2\triangle\beta$$

$$J_y^{\beta}(t) = \sum_{L=i,j,k} -\xi_L h_L(t)/2\triangle\beta \qquad \cdots\cdots(3)$$

From this we can see that once we have determined the plane lements of vadose region, the model $H^{\beta}(x, y, t)$ of water head at any plane element β will uniquely be determined by true water head $h_L(t)$ (L=i, j, k) at its point element; the gradients along with directions x and y will be independent of variables x and y.

3. The establishment of the element balance equation

The establishment the element balance equation is closely correlated with the establishment of the balance element. The balance element R_i at an interior point element i will be set up in this way: the region R_i is consisted of connecting line between middle points appropriate line elements and the intersection of three perpendicular bisectors of the plane elements which have a common top the point element i (Fig 2). Actually, the establishment of the balance element is to subdivide the vadose region. After the subdividing, there must be no gap and superposition between the two neighboured balance elements. That is necessary condition which ensures that the approximate solution may converge to the precisional solution.

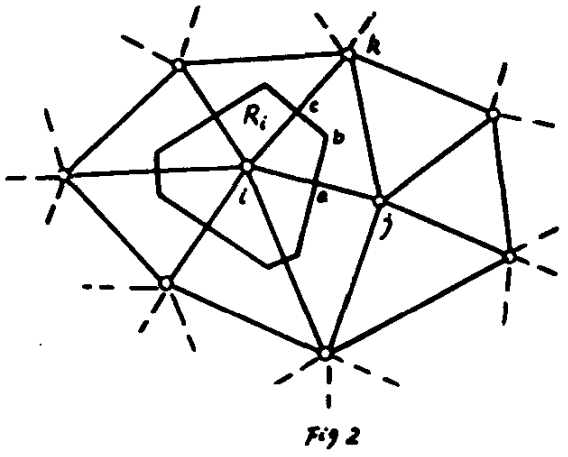

Fig 2

The mathematical expression of water quantity balance relation in the balance element is known as an element balance equation.

Now, let's deal with the main balance factors of a square iabc (a part of the plane element β) in the balance element R_i.

(1) Standing water storage or elastic water quantity

If we take $q_s^{\beta}(t)$ as the released standing water storage (for unconfined water) or elastic water quantity (for confined water) of the square iabc at time t, then it has:

$$q_s^{\beta} = \frac{S_\beta}{16\triangle\beta} \left| (\eta_i\eta_k + \xi_i\xi_k)(\eta_j^2 + \xi_j^2) + (\eta_i\eta_j + \xi_i\xi_j)(\eta_k^2 + \xi_k^2) \right| \frac{\partial h_i(t)}{\partial t}$$

Where

$$S_\beta = \begin{cases} \mu^* M & \text{for the confined region} \\ \mu & \text{for the unconfined region} \end{cases}$$

M is the average thickness of confined aquifer, μ^* is confined water specific storage, μ is uncontined water specific yield.

The standing water storage or the elastic water quantity being released by R_i is:

$$Q_s(t) = \sum_{\beta=1}^{P} q_s^{\beta}(t) \qquad \cdots\cdots(4)$$

Where the P is the numbers of all plane elements in the balance element R_i.

(2) Lateral flow

It is assumed that the lateral flow is positive when it is out from R_i, and negative when it runs into R_i (Fig 3). According to the Darsy theory, we take $q_u^{\beta}(t)$ as the lateral flow through the section abc, at a time t then it has:

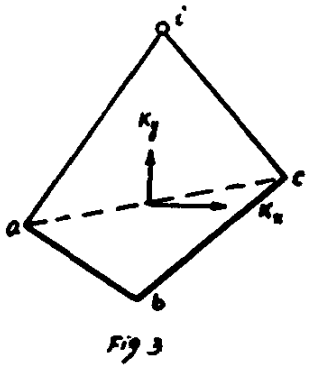

Fig 3

$$q_u^{\beta}(t) = -\frac{x_k - x_j}{2} J_y^{\beta} T_y - \frac{y_k - y_j}{2} J_x^{\beta} T_x = -\frac{\xi_i}{2} J_y^{\beta} T_y + \frac{\eta_i}{2} J_x^{\beta} T_x,$$

Substituting (3) in the above formula gives,

$$q_u^{\beta}(t) = \frac{\xi_i T_y}{4} \sum_{L=i,j,k} \xi_L h_L(t)/\Delta\beta + \frac{\eta_i T_x}{4} \sum_{L=i,j,k} \eta_L h_L(t)/\Delta\beta.$$

Where

$$T_x(t) = \begin{cases} K_x M & \text{in the confined region} \\ K_x [h_M^{\beta}(t) - B] & \text{in the unconfined region} \end{cases}$$

$$T_y(t) = \begin{cases} K_y M & \text{in the confined region} \\ K_y [h_M^{\beta}(t) - B] & \text{in the unconfined region} \end{cases}$$

B is average elevation of the base of plane element β, K is coefficient of permeability,

$$h_M^{\beta}(t) = \frac{h_i(t) + h_j(t) + h_k(t)}{3},$$

$Q_u(t)$ expresses the lateral flow of R_i, thus

$$Q_u(t) = \sum_{\beta=1}^{P} q_u^{\beta}(t) \qquad \cdots\cdots(5)$$

For the balance element R_i, according to the continuity theory we have:

$$Q_s(t) - Q_u(t) = 0 \qquad \cdots\cdots(6)$$

The equation (6) is called fundamental element balance equation of the balance element.

The equation (6) should be modified when the following yields take place on the plane element β in the balance element R_i

a. If there is a pnmping well with pumpage $q_w^{\beta}(t)$ in the plane element β (Fig 4), the $\frac{\Delta_i}{\Delta\beta} \times q_w^{\beta}(t)$

should be placed the right side of the equal symbol, where Δ_i is the area coordinate of the pumping well.

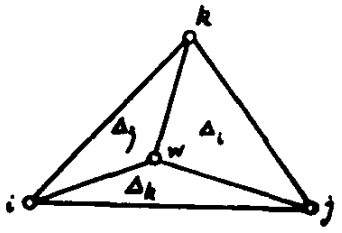

Fig 4

Let Q_w (t) be the total quantity of water given to R_i by the wells at all plane elements which have a common top point with the point element i, then the element balance equation is:

$$Q_s(t) - Q_u(t) = Q_w(t) \qquad \cdots\cdots(7)$$

b. If there is the infiltration water at balance element R_i and the infiltration in every plane elements is uniform. Let's use the symbol E_β to express the infiltration water quantity per unit area at per unit time, and the symbol $Q_E(t)$ to express the total infiltration quantity at the tlement R_i and we may have:

$$Q_E(t) = \sum_{\beta=1}^{P} \frac{E_\beta}{16\Delta_\beta} \left| (\eta_i \eta_k + \xi_i \xi_k)(\eta_j^2 + \xi_j^2) + (\eta_i \eta_j + \xi_i \xi_j)(\eta_k^2 + \xi_k^2) \right|,$$

and the element balance equation will be:

$$Q_s(t) - Q_u(t) = Q_w(t) - Q_E(t) \qquad \cdots\cdots(8)$$

c. When i is the point element of second boundary, the balance element R_i is shown in Fig. 5. we assume that the flow of unit width section at second boundary is $q_B(t)$, and the outflow of R_i is positive, the inflow of R_i is negative, it should be written:

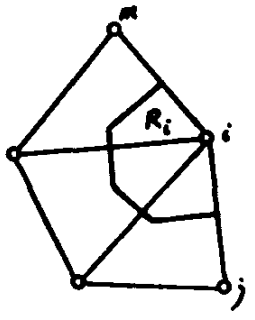

Fig 5

$$Q_E(t) = \frac{1}{2}(\overline{ij} + \overline{im})\, q_B(t)$$

Adding the above formula to the right side of equation (8) we have:

$$Q_s(t) - Q_w(t) = Q_w(t) - Q_E(t) + Q_B(t) \qquad \cdots\cdots(9)$$

The equation (9) is general form of the element balance equation.

When i is the point element of first boundary or the intersection between first and second bounderies we may not set up the equation. If there are N_1^{th} interior point elements and N_2^{th} point elements of second boundary, we may establish N^{th} ($N = N_1 + N_2$) element balance equations from equation (9) (Let $Q_B(t)=O$, when i is interior point element).

When the term $\frac{\partial h_i(t)}{\partial t}$ is obtained by the backward difference, the equation (9) will be the implicit equation. And then, all of the balance equations must be solved simulteneously. When it is obtained by the forward difference, the equation (9) will be the explicit equation. Therefore, the balance equations may be solved one by one. Because, various convergence rates may be produced in various balance elements for the same Δt, so in order to ensure the computational accuracy we should take the explicit equation in larger balance elements and the implicit equation in smallar balance elements. Thus, it may not only increase the computation speed, but also decrease the necessary memory capacity of a computer. In the unconfined water computation, we can properly

take the average value of non-off quántities of two moments before and after the time difference for Q_u (t).

4. Inversion of parameters and predict of water level

Generally, the optimal value is choosed by optimization method 1 and primary linear programming within the range of every hydrogeologic parameters' exchanges. If a certain group of parameters can make the sum of absolute values of the differences between the observed values and computational values which are the water head of all measured wells and the moments be less then the accuracy index which is given before hand, this group of parameters will be regarded as hydrogeologic parameters.

In the case of given initial conditions, boundary condition, pumping intensity and hydrogeologic parameters, we can obtain the water head of each interior point elements and second kind of boundary point elements at moment t_1, t_2, . . . t_n. The computational steps are as follows: first, compute the end-moment water head of first time interval. Second, take the result of the first step as the initial water head of second time interval to compute the end-moment water head of this interval. And then, use the same method sequently to compute the end-moment water head of the third, fourth . . . until the N^{th} time interval. We call this process water level predict. For paper limited specific steps about inversing the parameters and water level predict can not be given in detail.

In addition, the computation result is much dependent upon the boundary condition. In order to avoid subjectivity in general computational boundaries we must try our best to reach the natural boundaries.

Compared with the other numerical methods it has the advantages of clear physical concept and is easy for ordinary engineering technicians to master. Moreover, it may avoid disterbance of the local quality balance and the limitation of selected time intervals in the finite element method. We used this method in predicting the draining for mine and in estimating groundwater resource. As a rsult, we have received fairly good results.

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ANALISIS POR ORDENADOR DE LOS BOMBEOS EN LA MINA LLANO (TERUEL-ESPAÑA)

Yagüe Ballester, Antonio* ; Fernández-Rubio, Rafael**

ABSTRACT : The adequate analysis of the piezometric lowering, produced during long pumping period, is not easy, by the influence of several factors : lithological heterogeneities ; impervious boundaries ; constant head boundaries ; yield exchange ; variations of transmisivity with the time ; etc. To facilitate this analysis, we have developed a method of computer calculation, starting from the Jacob simplification ant the ratio method, that also increase the possibilities of application, and avoid the routine calculations. We present the FORTRAN code and an application to the drainage of the future iron mine of Llano (Teruel) is displayed.

RESUME : L'analyse correcte des baisses piézomètriques produites pendant de longues périodes de pompage, présente quelque difficulté, dû à l'influence de divers facteurs, tels que hétérogénéités du milieu, limites imperméables, barrières de recharge, changements de régime, variations de T dans le temps, etc. Pour faciliter cette analyse nous avons développé une méthode de calcul avec ordinateur, à partir de la simplification de Jacob et de la méthode du quotient, qui rend possible le travail d'interprétation et évite le temps de calculs routiniers. On présente le programme codifié en FORTRAN, et on l'applique au drainage de la future exploitation du minerai de fer appelée Llano (Teruel).

RESUMEN : El análisis correcto de los descensos piezométricos producidos en periodos largos de bombeo, presenta dificultades debido a la influencia de factores diversos, tales como heterogeneidades del medio, límites impermeables, barreras de recarga, cambios de caudal, variaciones de T en el tiempo, etc. Para facilitar este análisis hemos desarrollado un método de cálculo con ordenador, a partir de la simplificación de Jacob y del método del cociente, que potencia la labor de interpretación y evita el tiempo de cálculos rutinarios. Se presenta el programa, codificado en FORTRAN, y se aplica al drenaje de la futura explotación de mineral de hierro denominada Llano (Teruel).

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INTRODUCCION

Habitualmente, los datos hidrológicos dimensionales de los acuíferos se obtienen a través de los bombeos, ya sean de ensayo o de explotación : de su correcta interpretación depende la garantía de los resultados. El objeto de la presente comunicación es facilitar esa interpretación, mediante procesos automáticos de cálculo en ordenador.

La resolución de los datos obtenidos en bombeos ha pasado por diversas fases. Inicialmente se trataba de ajustar, mediante métodos gráficos, el problema a desarrollos analíticos previos que exigían mucho tiempo en operaciones rutinarias (THEIS, 1935 ; JACOB y LOHMAN, 1952 ; HANTUSH y JACOB, 1955 ; etc). Posteriormente se desarrollaron métodos más rápidos, sin variar prácticamente las técnicas interpretativas de los anteriores (JAEGER, 1959 ; OGDEN, 1965 ; SHEAHAN, 1967 ; etc). Más tarde han aparecido intentos -no exentos de polémicas- de traspasar con más o menos éxito, las operaciones rutinarias a los computadores (SALEEM, 1970 ; KORGANOFF y BESBES, 1973 ; LABADIE y HELWEG, 1975 ; HOLZCSCHUH, 1976 ; etc). En la actualidad hay una tendencia creciente a utilizar los procesos de modelación, que obligan a planteamientos más lentos y minuciosos, y que sitúan el problema en una esfera distinta al trabajo diario de los técnicos (WALTON, 1976 ; LAND, 1977 ; etc).

De manera general podemos decir que la principal dificultad estriba, en la actualidad, en la complejidad y variedad creciente de los esquemas que se proponen como casos tipo : en estas condiciones ¿ cómo escoger un método de interpretación que al mismo tiempo sea simple y resuelva con eficacia casos muy variados ? El método que vamos a exponer trata de abarcar el mayor número de condiciones, aprovechando la rapidez de cálculo del ordenador.

LA SIMPLIFICACION DE JACOB

En 1935 THEIS desarrolló la fórmula de no equilibrio, a partir de la analogía entre el flujo del agua subterránea y la conductividad del calor :

$$s = \frac{Q}{4\pi T} \int_u^\infty \frac{e^{-u}}{u}\, du = \frac{Q}{4\pi T}\, W(u) \qquad (1)$$

donde :

$$u = \frac{r^2 S}{4Tt} \qquad (2)$$

El cálculo de la integral en la ecuación (1) no se puede hacer directamente, pero su valor queda definido por la función W(u) :

$$W(u) = -0{,}577216 - \ln u + u - \frac{u^2}{2.2!} + \frac{u^3}{3.3!} - \ldots \qquad (3)$$

WENZEL (1942) tabuló esta función para valores de u comprendidos entre 10^{-15} y 9,9. JACOB (1950) observó que la suma de términos a partir del tercero, en la ecuación (3), es despreciable cuando u es suficientemente pequeño. De esta forma la ecuación (1) queda :

$$s = \frac{Q}{4\pi T} \; (-o,577216 - \ln u) \qquad (4)$$

Así se facilita la resolución, ya que los valores de s versus log t , se sitúan en una recta, que permite deducir gráficamente los valores de T y S.

EL METODO DEL COCIENTE

JAEGER (1959) y OGDEN (1965) dieron las bases del método, pero fueron SHEAHAN (1967) y NARASHIMAN (1968) quienes lo hicieron aplicable a la generalidad de situaciones posibles. El método presenta la ventaja de no basarse en ajustes gráficos para obtener el valor de los parámetros hidráulicos.

Si despejamos T de las ecuaciones (1) y (2) e igualamos queda :

$$T = \frac{Q}{4\pi s} \; W(u) = \frac{r^2 S}{4ut} \qquad (5)$$

$$u \; . \; W(u) = \frac{s}{t} \; . \; \frac{\pi r^2 S}{Q} \qquad (6)$$

Se define la función Z(u) de la siguiente forma :

$$2 \; . \; Z(u) = \frac{2u.W(2u)}{u.W(u)} \qquad (7)$$

Sustituyendo el valor de u.W(u) de la ecuación (6) en la (7) :

$$Z(u) = \frac{s_{1/2}}{s} = \frac{W(2u)}{W(u)} \qquad (8)$$

en donde :

s : descenso en el tiempo t
$s_{1/2}$: descenso en el tiempo t/2.

La resolución se basa en el cociente de los descensos en tiempos múltiplos de 2, y en el empleo de una tabla de valores correlativos Z(u), W(u) y u. Conocidos estos valores, los parámetros hidráulicos se calculan como en el caso de Theis.

EL ANALISIS MULTIPLE

Supongamos un bombeo en un acuífero, tal como indica la figura 1, en el que se distinguen dos partes denominadas *medio 1* y *medio 2*.

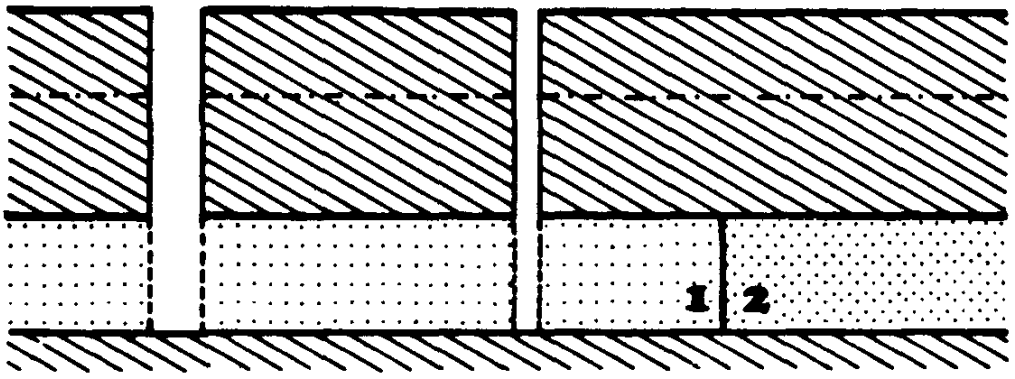


Figura 1

La figura 2 A muestra las variaciones posibles, al analizar el bombeo mediante la simplificación de Jacob. Existen tres tramos. El I corresponde al tiempo en que todavía no se cumple la simplificación. El II es recto y corresponde a los descensos debidos a la extensión del cono de bombeo en el *medio 1* y el II, que comienza curvo y continua recto, corresponde a la extensión del cono de bombeo por el *medio 2*.

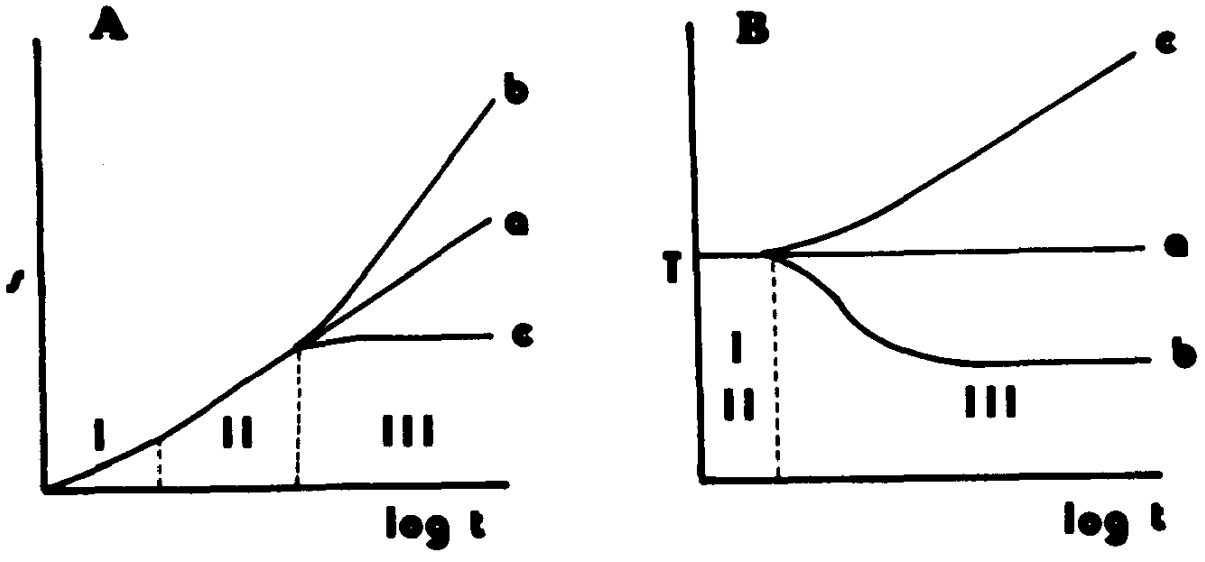


Figura 2

En el tramo III pueden ocurrir tres posibilidades fundamentalmente :

a) El *medio 2* tenga las mismas características hidráulicas que el *medio 1*. En ese caso los descensos continúan como en el tramo II, y el cálculo de T se hace mediante :

$$T_1 = \frac{2,3 \cdot Q}{4\pi m_a} \qquad (9)$$

donde :

m_a : pendiente de la recta a

b) El *medio 2* es impermeable. Los descensos en este caso son equivalentes a los producidos por el pozo existente, más el provocado por otro ficticio igual y simétrico en el *medio 2*, (FERRIS et al., 1962). La pendiente de la recta que se produce, tras un pequeño tramo curvo, es igual a :

$$m_b = 2 \cdot m_a \qquad (10)$$

y para calcular la transmisividad del *medio 2* :

$$T_2 = T_1 \left[\frac{2m_a}{m_b} - 1\right] \qquad (11)$$

El caso de impermeabilidad total es un caso límite. Si la permeabilidad es sólo diferencialmente menor, que en el *medio 1*, también sirve la fórmula (11) tomando m_b como la pendiente de la recta diferente a m_a.

c) El *medio 2* mantiene una recarga a nivel constante. En este caso los descensos permanecen constantes con el tiempo, y la pendiente m_c es igual a cero. Como en el caso anterior la recarga a nivel constante es un caso límite. Si la permeabilidad en el *medio 2* es sólo diferencialmente mayor se puede calcular la transmisividad en ese medio a partir de la ecuación :

$$T_2 = T_1 \left[\frac{2m_a}{m_c} - 1\right] \qquad (12)$$

Si en lugar de dos medios hay más, las fórmulas (11) y (12) sirven para hallar los parámetros hidráulicos, tomando m_a como la pendiente de la recta anterior a la que produce el medio buscado.

La figura 2B muestra el mismo análisis realizado para el método del cociente. A diferencia del anterior, en la escala normal está la transmisividad calculada para cada descenso. Como se ve, el tramo I y II son horizontales. En cuanto al tramo III el caso a) supone una simple continuación de los tramos I y II. El caso b), de total impermeabilidad, supone un tramo curvo seguido de otro horizontal y se calcula la T mediante la relación :

$$T_2 = 2\,T_b - T_a \qquad (13)$$

El caso c) supone una recta cuya pendiente depende del valor al que se estabiliza el descenso. Los casos intermedios de mayor o menor permeabilidad, terminan siempre en un tramo recto horizontal, y a partir de la ecuación (13) se pueden hallar los parámetros hidráulicos del segundo medio.

EL PROGRAMA DE ORDENADOR

El ordenador trabaja hasta confeccionar las gráficas de la figura 2, y dar los valores de los parámetros hidráulicos de los distintos medios existentes. A partir de ellos, y de acuerdo con los datos geológicos, el hidrogeólogo debe interpretar la correcta distribución espacial de las heterogeneidades existentes en ese acuífero.

El programa sigue el diseño estructurado de la figura 3. En las tablas 1 puede verse un listado del mismo. En su confección se han aprovechado las ventajas de las subrutinas. De esta forma se puede acoplar a cualquier programa principal que lea datos de tiempos, caudales y descensos en diversos piezómetros. Para facilitar el acoplamiento de las sentencias DIMENSION, del programa principal y de la subrutina, se comienza aquel con una sentencia PARAMETER en la que se dan los valores de NN, NI y NNM. El significado de las distintas variables utilizadas puede verse en la tabla 2.

TABLA 2

CAUDAL	:	Caudales acumulados (m^3)
HORA 2	:	Tiempos múltiplos de 2 interpolados (días)
QTIEMP	:	Caudales correspondientes a HORA 2
NN	:	Número de piezómetros de observación
NI	:	Número máximo de interpolaciones
NNM	:	Número máximo de medidas
XLONG	:	Distancia del pozo al piezómetro (m)
SNIVEL	:	Descensos correspondientes a HORA 2
TS	:	Transmisividad según el método del cociente
SS	:	Coeficiente de almacenamiento según el método del cociente
TJ	:	Transmisividad según el método de Jacob
SJ	:	Coeficiente de almacenamiento según el método de Jacob.

La subrutina AMY, ejecutora principal del análisis múltiple, comienza con la determinación de los tiempos múltiplos de 2, necesarios para el método del cociente. En caso de resultar el factor 2 demasiado grande, puede modificarse fácilmente.

De acuerdo con esos tiempos interpola entre los datos medidos los descensos y caudales correspondientes ; controla los ascensos, y calcula la función Z(u). A continuación busca los valores de W(u) y u mediante las funciones WENZEL y ZUCSH, y una vez hallados calcula la T y S para cada pareja de valores interpolada. En el caso de que se hayan cumplido las condiciones necesarias para que se cumpla la

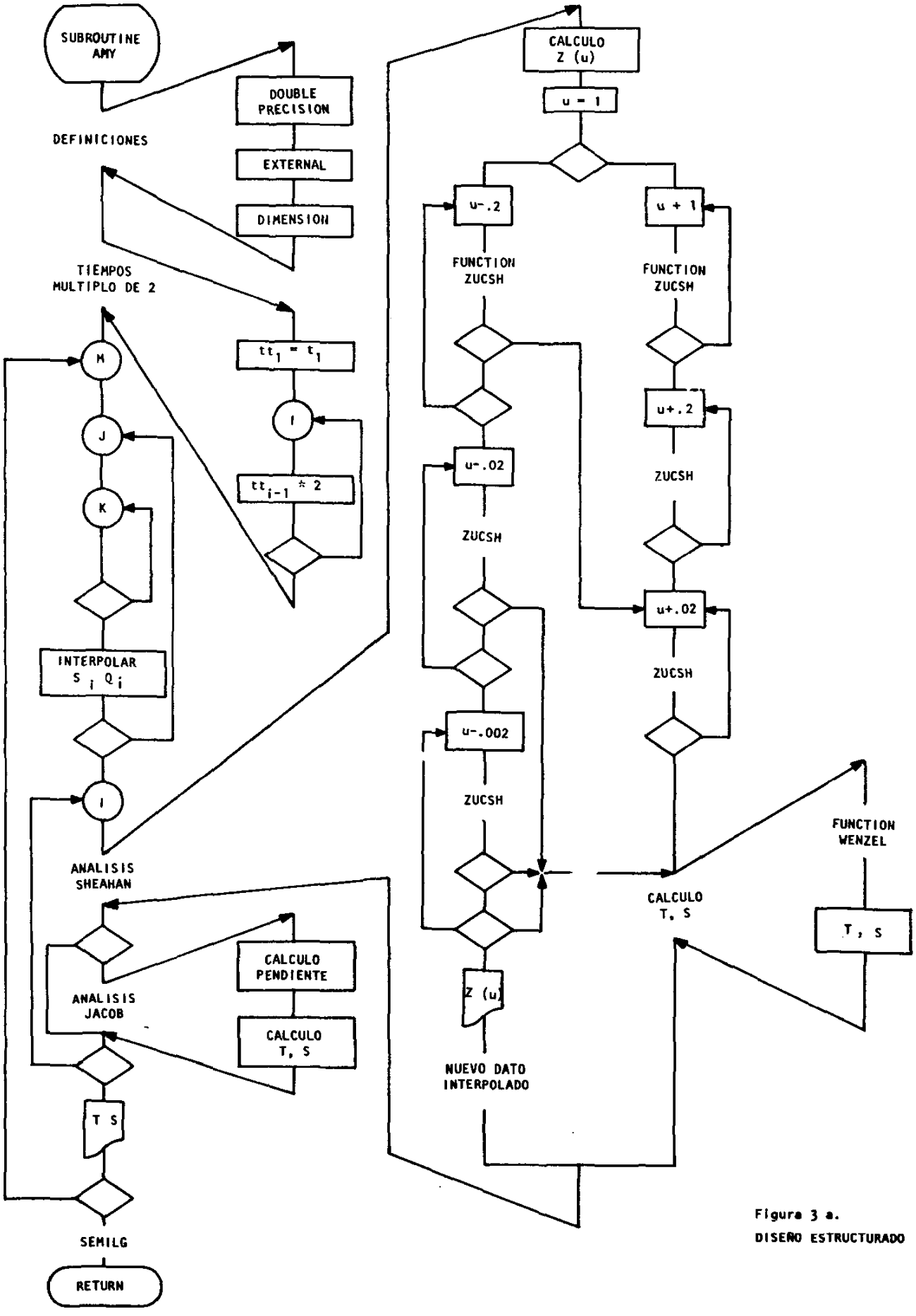
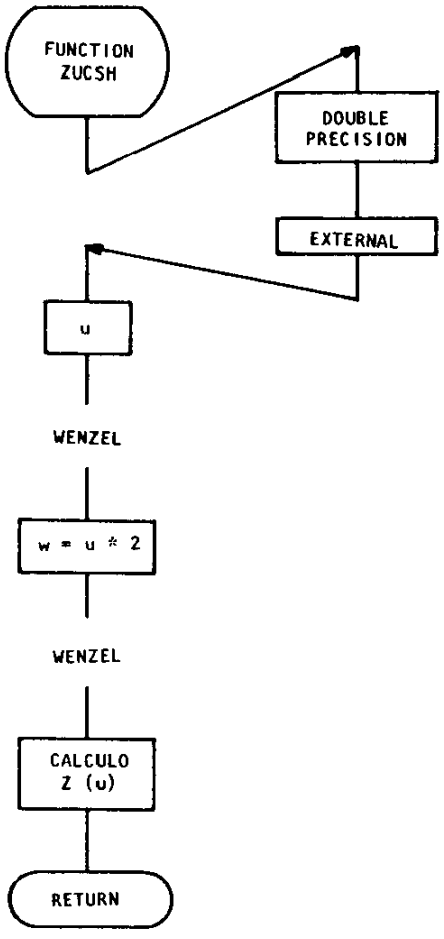


Figura 3 a.
DISEÑO ESTRUCTURADO



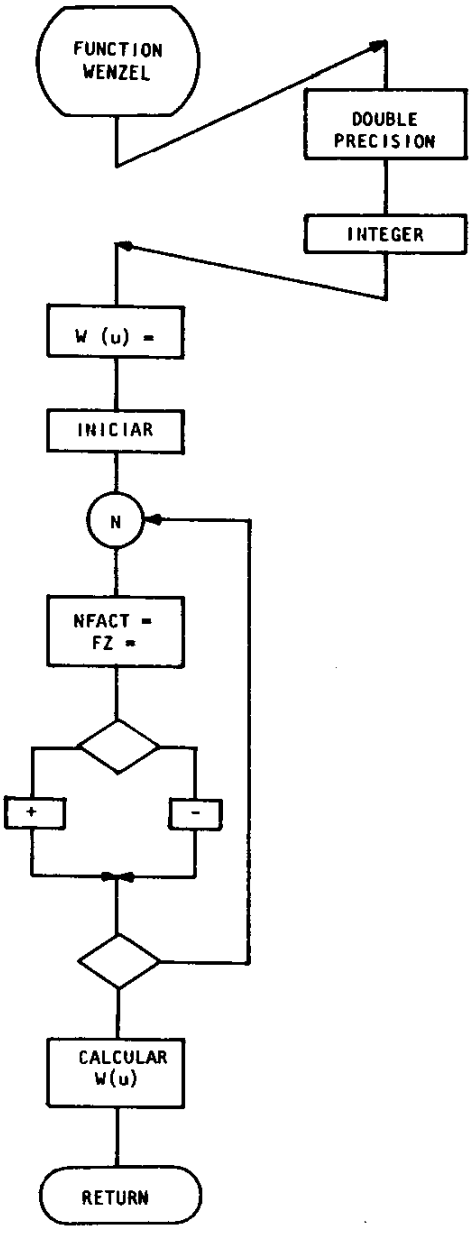


Figura 3 b.
DISEÑO ESTRUCTURADO

TABLA 1 a

```
1*      C       NN. ... .. .. .. .NUMERO DE PIEZOMETROS
2*      C       NI. ... .. .. .. .VALOR MAXIMO DEL NUMERO DE INTERPOLACIONES
3*      C       NMM ... .. .. .. .VALOR MAXIMO DEL NUMERO DE MEDIDAS
4*      C       XLONG(I) .. .. .DISTANCIA (METROS) DEL POZO AL PIEZOMETRO
5*      C       CAUDAL(J). .. .CAUDAL ACUMULADO (METROS CUBICOS)
6*              PARAMETER NN=5,NI=50,NMM=100
7*              DIMENSION  HORA2(NI),TIEMPO(NMM),XNIVEL(NN,NMM),CAUDAL(NMM),
8*             -SNIVEL(NN,NI),XLONG(NN),TS(NN,NI),SS(NN,NI),TJ(NN,NI),SJ(NN,NI),
9*             -PD(NN,NI),LSTEP(NN,100),ALINE(100),AMAXI(NN),NPUNTO(NN)
10*            -,C(NN),QTIEMP(NI)
11*     C       LECTURA DE LOS DATOS
12*             READ(5,1)(XLONG(I),I=1,NN)
13*     1       FORMAT(8F10.0)
14*             J=1
15*     2       READ(5,1,END=5)CAUDAL(J),TIEMPO(J),(XNIVEL(I,J),I=1,NN)
16*             DO 4 I=1,NN
17*             XNIVEL(I,J)=XNIVEL(I,J)*100.
18*             IF(J.EQ.1) C(I)=XNIVEL(I,1)
19*             XNIVEL(I,J)=XNIVEL(I,J)-C(I)
20*     4       CONTINUE
21*             J=J+1
22*             GO TO 2
23*     5       KK=J-1
24*             CALL  AMY( NN , CAUDAL,KK,HORA2,TIEMPO,XNIVEL,SNIVEL,XLONG,TS,SS,
25*            -TJ,SJ,PD,LSTEP,ALINE,AMAXI,NI,NMM,NPUNTO,QTIEMP)
26*             STOP
27*             END
```

TABLA 1 b

```
 1*            SUBROUTINE AMY (NN,CAUDAL,KK,HORA2,TIEMPO,XNIVEL,SNIVEL,XLONG
 2*           -TS,SS,TJ,SJ,PD,LSTEP,ALINE,AMAXI,NI,NMM,NPUNTO,QTIEMP)
 3*            DOUBLE PRECISION U,ZU,ZUC,ZUCSH,WENZEL
 4*            EXTERNAL ZUCSH,WENZEL
 5*            DIMENSION HORA2(NI),TIEMPO(NMM),XNIVEL(NN,NMM),CAUDAL(NMM),
 6*           -SNIVEL(NN,NI),XLONG(NN),TS(NN,NI),SS(NN,NI),TJ(NN,NI),SJ(NN,
 7*           -PD(NN,NI),LSTEP(NN,100),ALINE(100),AMAXI(NN),NPUNTO(NN)
 8*           -,QTIEMP(NI)
 9*
10*      C     LLENADO DE MATRIZ CON TIEMPOS MULTIPLOS DE 2
11*            HORA2(1)=TIEMPO(2)
12*            DO 10 I=1,100
13*            HORA2(I+1)=HORA2(I)*2.
14*            IF(HORA2(I+1).GT.TIEMPO(KK)) GO TO 11
15*      10    CONTINUE
16*      11    N2=I
17*
18*            DO 70 M=1,NN
19*      C     INTERPOLAR DESCENSOS EN LOS TIEMPOS MULTIPLOS DE 2
20*            N3=0
21*            B=0
22*            DO 20 J=1,N2
23*            DO 12 K=1,KK
24*            IF(HORA2(J).LT.TIEMPO(K)) GO TO 13
25*      12    CONTINUE
26*      13    L=K
27*            IF(XNIVEL(M,L-1).GT.XNIVEL(M,L)) GO TO 28
28*            SNIVEL(M,J)=((XNIVEL(M,L-1)-XNIVEL(M,L))*(TIEMPO(L-1)-
29*           -HORA2(J)))/(TIEMPO(L)-TIEMPO(L-1))+XNIVEL(M,L-1)
30*            IF(HORA2(J).EQ.TIEMPO(L-1)) L=L-1
31*            QTIEMP(J)=(CAUDAL(L)-B)/(HORA2(J)-HORA2(J-1))
32*            B=CAUDAL(L)
33*            N3=N3+1
34*      20    CONTINUE
35*
36*      28    WRITE(6,29) M,N2,N3
37*      29    FORMAT('1 EN EL PIEZOMETRO',I3,' DEBEN INTERPOLARSE',I4,
38*           -' DESCENSOS Y SE INTERPOLAN',I4//,' DATO',11X,'T SHEAHAN',13X,
39*           -'T JACOB',11X,'S SHEAHAN',13X,'S JACOB',/,84('-'))
40*            DO 50 I=2,N3
41*            IF(SNIVEL(M,I-1)-SNIVEL(M,I)).50,
42*            ZU=SNIVEL(M,I-1)/SNIVEL(M,I)
43*            U=1.
44*            IF(ZU.LT..2229) GO TO 41
```

TABLA 1 c

```
*15*
*16*      31    U=U-.2
*17*            ZUC=ZUCSH(U)
*18*            IF(ZUC.GT.ZU) GO TO 45
*19*            IF(U.GT.0.2) GO TO 31
*50*      32    U=U-0.02
*51*            ZUC=ZUCSH(U)
*52*            IF(ZUC.GT.ZU) GO TO 48
*53*            IF(U.GT.0.02) GO TO 32
*54*      33    U=U-0.002
*55*            ZUC=ZUCSH(U)
*56*            IF(ZUC.GT.ZU) GO TO 48
*57*            IF(U.GT.0.002) GO TO 33
*58*            WRITE(6,34) ZU,ZUC
*59*      34    FORMAT(2G18.6)
*60*            GO TO 50
*61*      40    U=U+1
*62*            ZUC=ZUCSH(U)
*63*            IF(ZUC.GT.ZU) GO TO 40
*64*      41    U=U-0.2
*65*            ZUC=ZUCSH(U)
*66*            IF(ZUC.LT.ZU) GO TO 41
*67*      45    U=U+0.02
*68*            ZUC=ZUCSH(U)
*69*            IF(ZUC.GT.ZU) GO TO 45
*70*      48    W1=WENZEL(U)
*71*            TS(M,I)=QTIEMP(I)*W1*(4*3.141592*SNIVEL(M,I))
*72*            SS(M,I)=4.*TS(M,I)*U*HORA2(I)*XLONG(M)**2
*73*      C     ANALISIS DE JACOB
*74*            IF(U.GT.0.02) GO TO 50
*75*            PD(M,I)=(SNIVEL(M,I)-SNIVEL(M,I-1))*ALOG10(HORA2(I)*HORA2(I-1))
*76*            TJ(M,I)=2.3*QTIEMP(I)*(4*3.141592*PD(M,I))
*77*            SJ(M,I)=4.*TJ(M,I)*U*HORA2(I)*XLONG(M)**2
*78*      50    WRITE(6,51)I,TS(M,I),TJ(M,I),SS(M,I),SJ(M,I)
*79*      51    FORMAT(I4,2F20.2,2E20.3)
*80*            NPUNTO(M)=N3
*81*      70    CONTINUE
*82*            CALL SEMILG(NN,NPUNTO,HORA2,LSTEP,SNIVEL,ALINE,AMAXI,NI)
*83*            CALL SEMILG(NN,NPUNTO,HORA2,LSTEP,TS,ALINE,AMAXI,NI)
*84*
*85*            RETURN
*86*            END
```

TABLA 1 d

```
1*      C       CALCULO DE LA FUNCION Z(U) DE SHEAHAN
2*              FUNCTION ZUCSH(U)
3*              IMPLICIT DOUBLE PRECISION (A-Z)
4*              EXTERNAL WENZEL
5*              WU1=WENZEL(U)
6*              W=U*2.
7*              WU2=WENZEL(W)
8*              ZUCSH=WU2-WU1
9*              RETURN
10*             END
```

TABLA 1 e

```
1*      C       CALCULO DE LA FUNCION DE POZO W(U) DE WENZEL
2*              FUNCTION WENZEL(U)
3*              IMPLICIT DOUBLE PRECISION (A-Z)
4*              INTEGER N
5*              WU=-0.577216-ALOG(U)
6*              NFACT=1
7*              SUMFZ=0.
8*              DO 8 N=1,50
9*              NFACT=NFACT*N
10*             FZ=U**N/(N*NFACT)
11*             IF(MOD(N,2)).6,
12*             SUMFZ=SUMFZ+FZ
13*             GO TO 8
14*     6       SUMFZ=SUMFZ-FZ
15*     8       CONTINUE
16*             WENZEL=WU+SUMFZ
17*             RETURN
18*             END
```

TABLA 1 f

```
1*            SUBROUTINE SEMILG(NN,NPUNTO,XMATR,LSTEP,R,ALINE,AMAXI,NI)
2*            DIMENSION R(NN,NI),XMATR(NI),LSTEP(NN,1000),NPUNTO(NN),
3*           -ALINE(100),AMAXI(NN)
4*            DO 50 L=1,NN
5*            NP=NPUNTO(L)
6*            PEQUE=XMATR(1)
7*            XMAX=XMATR(1)
8*            DO 2 I=2,NP
9*            IF(PEQUE-XMATR(I)) 1,1,
10*           PEQUE=XMATR(I)
11*     1     IF(XMAX-XMATR(I)),2,2
12*           XMAX=XMATR(I)
13*     2     CONTINUE
14*           NMIN=ALOG10(PEQUE)-1
15*           NMAX=ALOG10(XMAX)+1
16*           IF(NMAX.LE.0) NMAX=NMAX-1
17*           IF(NMIN.GE.0) NMIN=NMIN+1
18*           NCICLO=NMAX-NMIN
19*           NPASOS=100/NCICLO
20*           STPQUE=ALOG10(PEQUE)*NPASOS
21*           MINIMI=1000
22*           DO 3 J=1,1000
23*           MINIMO=MINIMI-J*NPASOS
24*           IF(MINIMO.LT.STPQUE) GO TO 4
25*     3     CONTINUE
26*     4     DO 5 I=1,NP
27*           STEP=ALOG10(XMATR(I))*NPASOS
28*     5     LSTEP(L,I)=STEP-MINIMO+1.5
29*           MM=NCICLO*NPASOS
30*           WRITE(6,9)
31*     9     FORMAT(1H1)
32*           AMINI=R(L,1)
33*           DO 10 I=1,NP
34*           IF(R(L,I).GT.AMAXI(L)) AMAXI(L)=R(L,I)
35*     10    IF(R(L,I).LT.AMINI) AMINI=R(L,I)
36*           X=AMAXI(L)
37*           TX=X-AMINI
38*           AL=TX/48.
39*           NCONT=0
40*           NLINE=0
41*     12    DO 15 I=1,MM
42*           ALINE(I)=' '
43*           IF(MOD(NLINE,5).EQ.0)ALINE(I)='-'
44*     15    IF(MOD(I,NPASOS).EQ.0) ALINE(I)=':'
45*           DO 30 I=1,NP
46*           IF((R(L,I).LT.X).OR.(R(L,I).GE.(X+AL))) GO TO 30
47*           J=LSTEP(L,I)
48*           ALINE(J)='*'
49*           NCONT=NCONT+1
50*     30    CONTINUE
51*           WRITE(6,35)X,(ALINE(I),I=1,MM)
52*     35    FORMAT(2X,F10.2,4X,'^',100A1)
53*           X=X-AL
54*           NLINE=NLINE+1
55*           IF(NCONT.GE.NP)GO TO 45
56*           GO TO 12
57*     45    WRITE(6,46)
58*     46    FORMAT(16X,102('>'))
59*           WRITE(6,48)NMIN,NMAX
60*     48    FORMAT(12X,'10**',I2,95X,'10**',I2)
61*     50    CONTINUE
62*           RETURN
63*           END
```

simplificación de Jacob, se realiza el cálculo de T y S para esos mismos valores. Por último, la subrutina gráfica SEMILG, realiza gráficos semejantes a la figura 2.

APLICACION A LOS BOMBEOS DE LA MINA LLANO

La Compañía Minera de Sierra Menera S. A. explota diversos yacimientos de hierro en las provincias de Teruel y Guadalajara (España). Dentro del próximo quinquenio se preveé iniciar las labores de la explotación denominada Llano.

Desde el punto de vista hidrogeológico, en una primera simplificación, este yacimiento puede asimilarse a un sistema acuífero kárstico, monocapa, limitado al techo por cuarcitas con intercalaciones arcillosas, y al muro por materiales margosos también impermeables. La disposición estructural, responde a un monoclinal que adopta longitudinalmente forma de sinclinal, con potencias que aumentan hacia el núcleo.

Con objeto de preparar, anticipadamente, el drenaje se ha comenzado el bombeo mediante un sondeo de captación en los carbonatos, ubicado en uno de los flancos del sinclinal, y se han instalado cuatro piezómetros para observar la evolución de los niveles en el paquete carbonatado. Los caudales extraidos en m^3, la distancia en metros, de los piezometros al pozo de bombeo, y los descensos medidos en cm, para los 70 primeros días de bombeo, se expresan en la Tabla 3.

TABLA 3

******** DATOS BOMBEO MINA LLANO ********

XLONS 1	2	3	4		
735	650	390	55		
CAUDAL	TIEMPO	XNIVEL 1.J	2.J	3.J	4.J
0	0	2420	2201	1640	1055
1670	1	2422	2210	1710	1148
4210	2	2429	2222	1779	1284
7610	3	2437	2236	1833	1341
10940	4	2446	2253	1880	1381
23210	8	2490	2317	2004	1468
34620	12	2517	2350	2090	1519
45230	16	2550	2377	2158	1566
53010	19	2577	2383	2200	1592
58650	22	2603	2397	2204	1606
66620	25	2630	2420	2268	1625
74720	29	2661	2463	2295	1634
81430	32	2684	2502	2330	1640
104860	36	2772	2557	2428	1691
110730	39	2795	2611	2447	1694
115980	43	2820	2629	2457	1695
121830	46	2844	2653	2481	1710
140650	60	2995	2785	2573	1856
144710	64	3035	2793	2588	1856
149340	67	3062	2800	2600	1874

A partir del método expuesto, y con el programa del ordenador de las tablas 1, hemos analizado los descensos provocados, y se han obtenido las gráficas de la figura 4. En ellas se pone de manifiesto lo siguiente :

- Al comienzo del bombeo, los piezómetros más cercanos (n° 3 y 4), acusan el menor espesor de materiales calizos, en forma de una menor transmisividad.

- Conforme avanza el bombeo se hacen más patentes, los límites impermeables a techo y muro de la formación carbonatada, disminuyendo en forma rápida la transmisividad.

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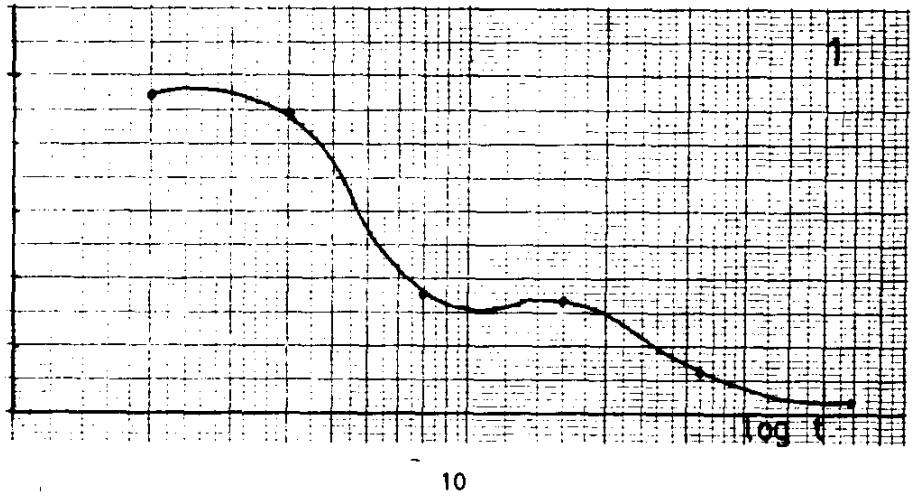
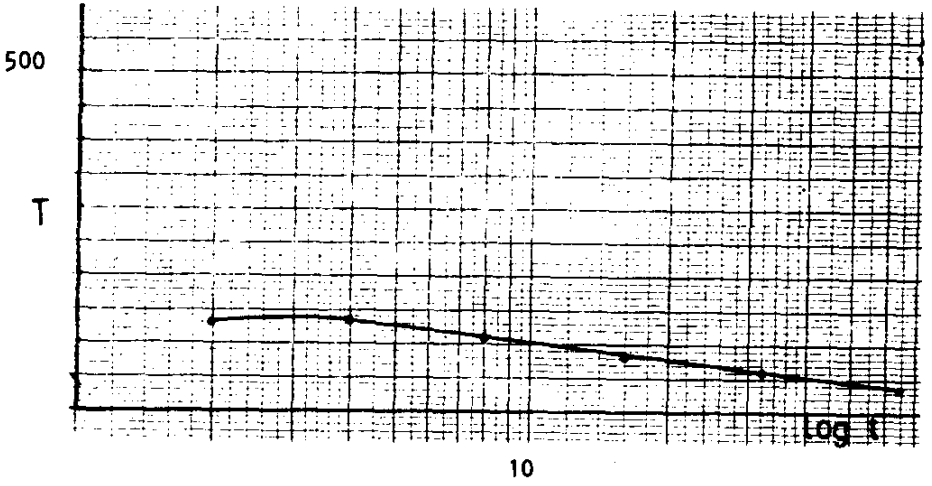
EN EL PIEZOMETRO 1 DEBEN INTERPOLARSE 7 DESCENSOS Y SE INTERPOLAN 7

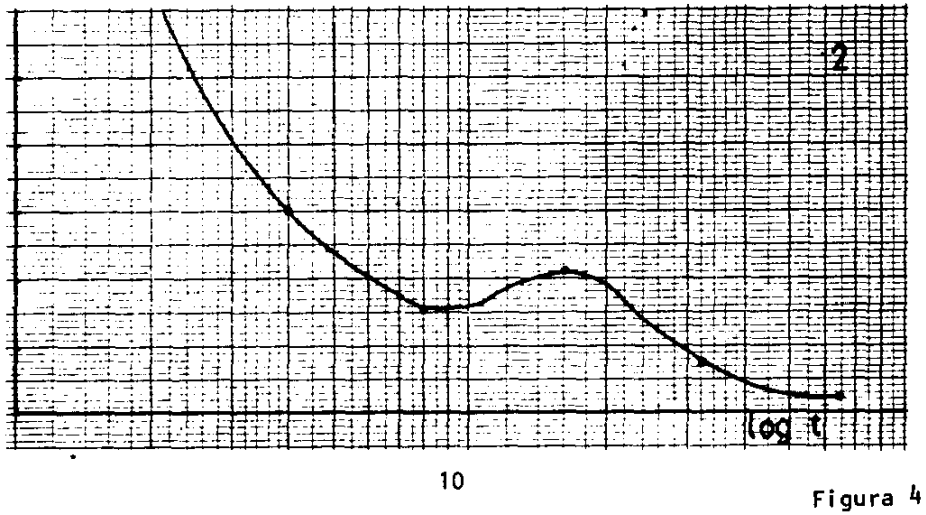
DATO	T SHEAHAN	T JACOB*	S SHEAHAN	S JACOB*
2	476.50	.00	.515-02	.000
3	449.62	.00	.706-02	.000
4	171.93	.00	.487-02	.000
5	170.93	.00	.450-02	.000
6	58.62	.00	.380-02	.000
7	16.36	.00	.292-02	.000

* El programa no calcula los valores mientras no se cumple la simplificación de Jacob

Tabla 4 : Ejemplo de la salida del programa







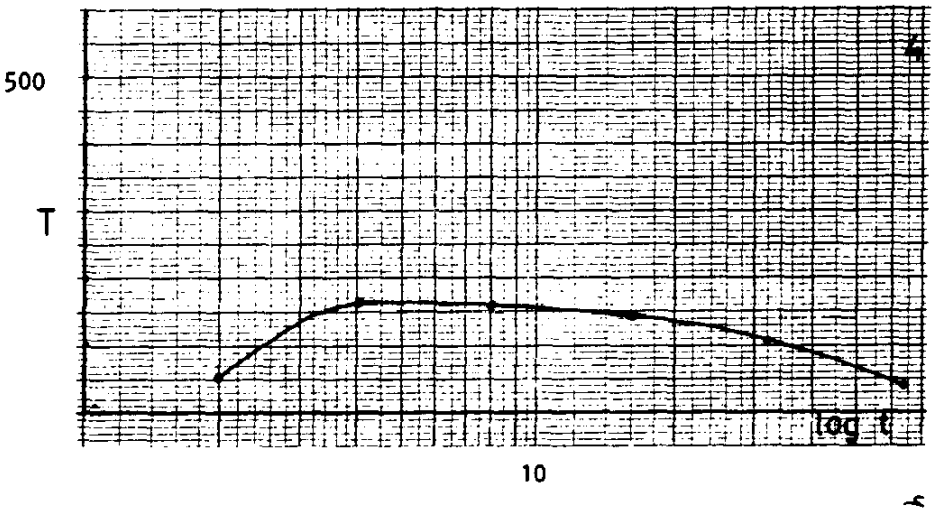


Figura 4

INFLUENCIA DE LA MINERIA POTASICA CATALANA EN LA CALIDAD DEL AGUA DEL RIO LLOBREGAT (BARCELONA)

Díaz, E.*; Custodio, E.**; Galofré, A.***

ABSTRACT : The exploitation of potash minerals in the central Llobregat River basin, produces the discharge of very saline waters, which increase noticeously the total disolved solids content of the surface waters downstream. The discharge reaches, and sometimes surpasses 5 kg/s in NaCl, in a river which mean flow near the mouth is about 20 m^3/s. The discharged salinity originates in the mineral treatment facilities, in the tips leachate, in the relief modification of the saline outcrops and in the mine water drainage. The origin and trend of the discharges in the different localities and the succesive improvements are studied, and the clear influence on the groundwater salinity is commented. Some possible ways to solve the problem are introduced.

RESUME : L'exploitation de minéraux potassiques dans le bassin central de la rivière Llobregat produit la décharge d'eaux très salines qui affectent sensiblement le degré de mineralización de las aguas superficiales. La décharge atteint et quelquefois dépasse 5 kg/s de ClNa, dans une rivière dont le débit moyen dans la partie inférieure est de l'ordre de 20 m^3/s. La salinité drainée provient des installations de traitement du minerai, du lavage des déblais, des altérations dans la topographie des affleurements salins et du drainage d'eau de mine. On étudie l'origine et l'évolution des décharges dans les différentes localités et les améliorations qui ont été introduites, ainsi que la nette influence des eaux souterraines. Les solutions possibles sont discutées pour combattre de type de contamination fluviale.

RESUMEN : La explotación de minerales potásicos en la cuenca media del río Llobregat produce vertidos de aguas muy salinas, que afectan sensiblemente al grado de mineralización de las aguas superficiales. El vertido alcanza y a veces rebasa la cifra de 5 kg/s de ClNa, en un río cuyo caudal medio en la parte baja es del orden de 20 m^3/s. La salinidad vertida procede de las plantas de tratamiento del mineral, del lavado de escombreras, de alteraciones en la topografía de los afloramientos salinos y del drenaje del agua de mina. Se estudia el origen y evolución de los vertidos en las diferentes localidades y las mejoras que se han ido introduciendo, así como la clara influencia de las aguas subterráneas. Se comentan las soluciones posibles para combatir este tipo de contaminación fluvial.

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1. - INTRODUCCION

El río Llobregat es uno de los pequeños ríos que desembocan al mar Mediterráneo entre la frontera francesa y el río Ebro (fig. 1). Nace en el Prepirineo y sale al mar en el límite S. de la ciudad de Barcelona. Su utilización para abastecimiento es intensiva y múltiple y de sus recursos superficiales y subterráneos asociados depende una importante concentración humana, industrial y agrícola. Los problemas de contaminación de las aguas superficiales y subterráneas son importantes, y entre los focos más singulares y cuantitativamente mayores están los vertidos de salmueras y aguas saladas de las explotaciones de sales potásicas en su cuenca media, ya sea sobre el propio río Llobregat como sobre su afluente el río Cardoner. El vertido salino ha ido creciendo desde 1925 a consecuencia de una cada vez mayor explotación de las minas, aunque las sucesivas mejoras tecnológicas, una mayor concienciación del problema de la contaminación y una más intensa labor de policía de aguas dentro de las posibilidades reales de actuación, han hecho que la salinidad vertida haya aumentado más lentamente que la producción.

El caudal medio del río en las cercanídas de Barcelona es de unos 20 m^3/s con una concentración media de ión cloruro de alrededor de 300 ppm de los cuales unos 250 ppm proceden de los vertidos mineros directos o indirectos. Si se consideran las variaciones de caudal medio de un año a otro se puede llegar a promedios anuales de 500 ppm en Cl^- y en los estiajes se pueden rebasar 1000 ppm. La Organización Mundial de la Salud señala un límite en el contenído de ión cloruro en el agua potable de 250 ppm en Cl; para el abastecimiento a Barcelona se ha fijado un límite de 350 ppm en Cl. Ambos límites se rebasan muy frecuentemente y aunque ello parece no ofrecer serios inconvenientes sanitarios, si que ocasiona problemas de sabor salino que afecta a la potabilidad, además de otros numerosos inconvenientes de cara a la corrosividad, usos industriales, calidad agrícola y depurabilidad, obligando a evitar determinados cultivos, a reponer más frecuente aparellajes metálicos y conducciones de agua, a mayores gastos en las plantas de depuración y a costosos tratamientos de desmineralización para determinados usos en procesos industriales. Es de destacar que a principios de siglo la salinidad del agua del río era de alrededor de 80 ppm Cl. La contaminación salina se extiende a los importantes acuíferos aluviales del Bajo Llobregat, que son una pieza clave en la regulación de la cuenca.

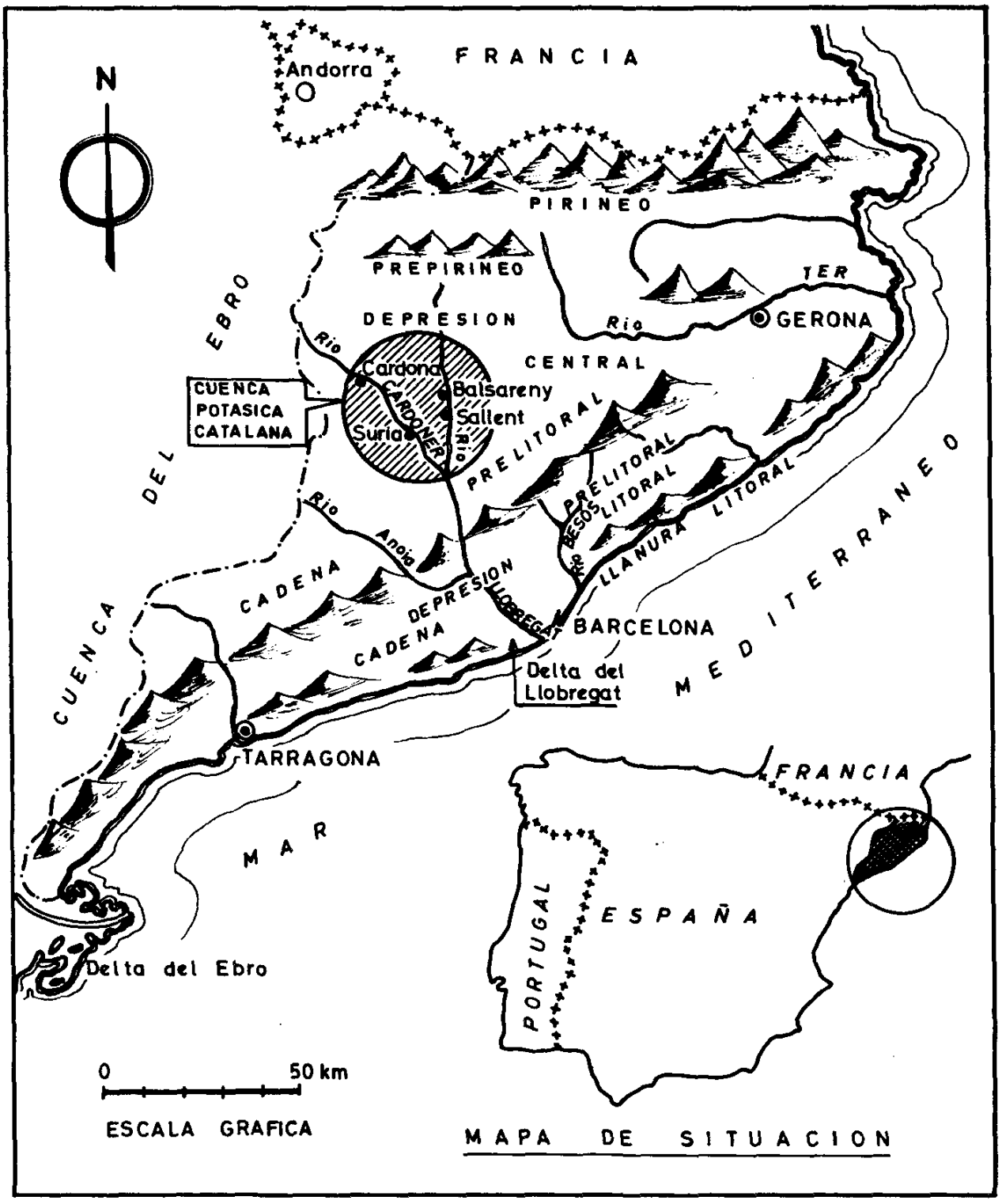

Fig. 1.— Esquema general del Pirineo Oriental con la situación de la cuenca potásica catalana

2. - EL ENTORNO GEOLOGICO

La cuenca potásica catalana se sitúa en la denominada clasicamente depresión del Valle del Ebro, formada por una enorme masa de materiales del terciario inferior (paleoceno a oligoceno) depositados sobre una formación preterciaria, posiblemente paleozoica, triásica, o cretácica, de acuerdo con los datos de los sondeos realizados.

Se trata probablemente de un yacimiento salino fromado por la evaporación de aguas cargadas en sales cloruradas y, en mucha menor proporción sulfatadas, originando capas o niveles horizontales originariamente paralelos o en forma de lentejones, que la actuación posterior de los esfuerzos tectónicos ha modificado de forma muy notable (fig. 2), originando espesores aparentes enormes, del orden de algunos centenares de metros en algún caso, y que a veces llegan incluso a perforar la cobertera aflorando en la superficie, tal como ocurre en los alrededores de Cardona.

Cuando no existe afección tectónica, los espesores son bastante menores y la disposición de los niveles salinos es perfectamente concordante con el techo y el muro de los mismos, estando estos formados por arcillas, areniscas y margas, aunque en algunos sondeos petrolíferos no aparecen por estar ya completamente laminados.

Desde el propio yacimiento hasta aguas abajo de Montserrat, el río Llobregat no discurre por materiales geológicos susceptibles de albergar acuíferos de cierta importancia. En efecto, desde el punto citado y hacia aguas abajo se encuentran la cubeta de Abrera, la cubeta de Sant Andreu de la Barca, el valle bajo del Llobregat,y finalmente su delta, constituídos todos ellos por acumulaciones de aluviones fluviales, conectados directamente con el río.

Estas cuatro unidades hidrogeológicas, de importancia capital para el abastecimiento urbano, industrial y agrícola del entorno de Barcelona,dependen casi exclusivamente del río Llobregat en cuanto a su recarga o alimentación a través de la infiltración del agua superficial,cuando el lecho del cauce lo permite, por lo que cualquier cambio en la calidad química del agua del río se transmite a las aguas subterráneas de esas unidades.

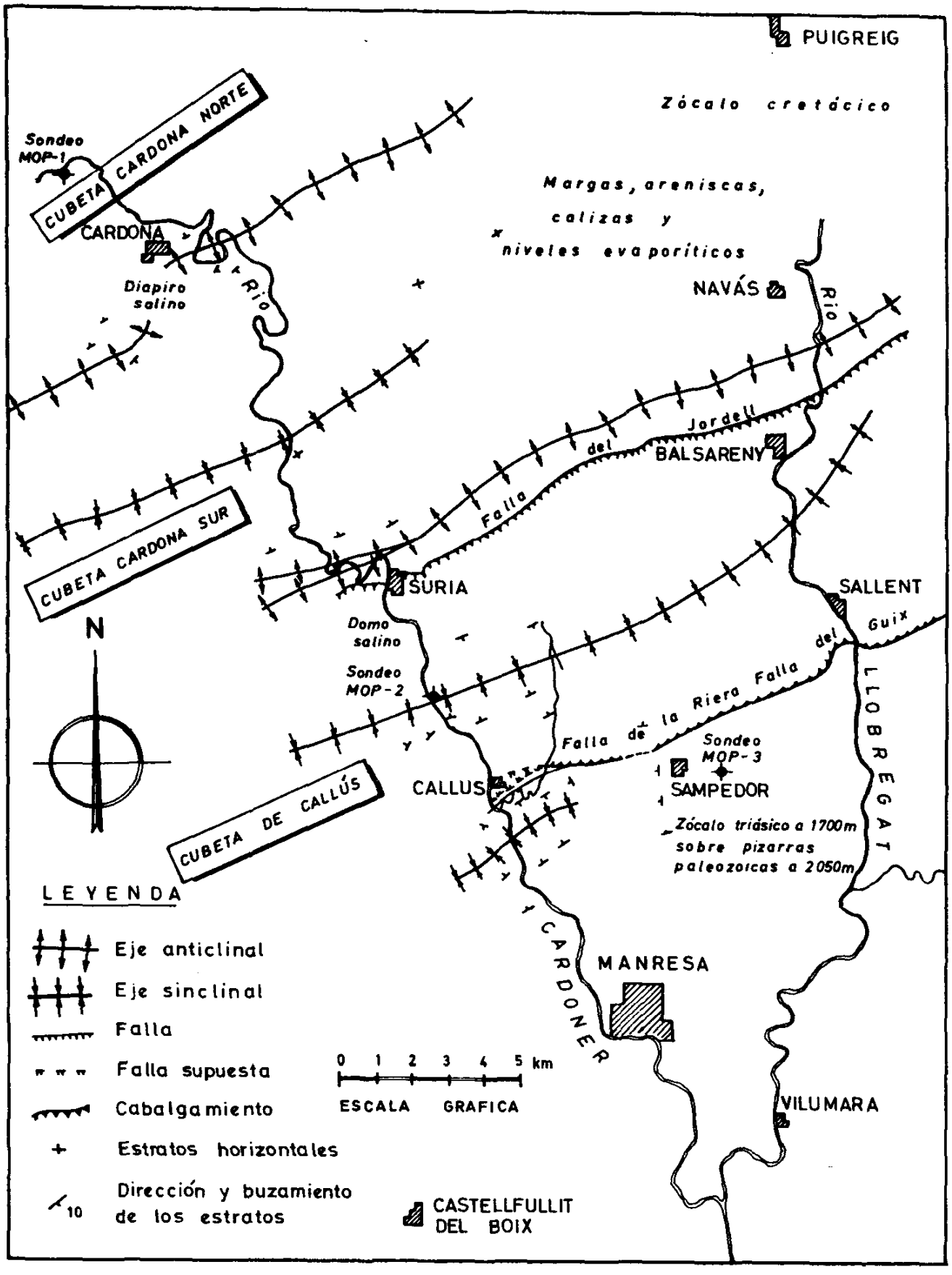

Fig. 2.— Esquema geológico-estructural de la Cuenca Potásica Catalana, según datos elaborados por Galotré y Custodio (1969)

3. - DATOS GENERALES DE LA CUENCA

En la figura 3 se indican los emplazamientos de las explotaciones potásicas en las cuencas de los ríos Cardoner y Llobregat, así como las casetas para el control automático de la salinidad y que corresponden a:

Explotación	Tno.munic.	Caseta control
Unión explosivos Rio Tinto,S.A.	Cardona	Malagarriga
Minas de Potasa de Suria, S.A.	Suria	Antius
Unión explosivos Rio Tinto ,S.A.	Balsareny	Torre Roca
Unión Explosivos Rio Tinto,S.A.	Sallent	Soler Vicenç
Sociedad General de Aguas de Barna.	Pallejà	Pallejà

En la actualidad las instalaciones de Balsareny y Sallent forman una sola unidad de explotación, aunque se mantienen las dos casetas de muestreo.

De los 5000 m^2 de la cuenca del río Llobregat, 2000 km^2 se encuentran aguas abajo de las explotaciones potásicas, lo que supone solo el 40% del total, pero con la particularidad de ser la zona con mayor población e industria, por lo que la influencia de las aguas residuales salinas supone un problema permanente a los aprovechamientos inferiores, entre los que se encuentra el abastecimiento de agua a Barcelona, con suministros del orden de 6 m^3/s , del propio río Llobregat, que se complementan con otros caudales procedentes principalmente del río Ter. Otras tomas para abastecimientos e industria, en gran parte de aguas subterráneas alimentadas por el río, suponen otros 5 m^3/s . Además se utilizan hasta 4 m^3/s de aguas superficiales para regadío.

En el río Cardoner existe, aguas arriba de la explotación minera, como única regulación,el embalse de Sant Ponç,de 24 hm^3 de capacidad, lo que permite mantener normalmente un caudal mínimo de estiaje en Manresa, del orden de 3 m^3/s .

En la cabecera del río Llobregat se ha construído recientemente la presa de La Baells de 115 hm^3 de capacidad, lo que permitirá regular un caudal mayor, del orden de 6 m^3/s más, con lo que se conseguirá una elevada regulación de la cuenca.

La aportación media del río Llobregat en Martorell es de 632 hm^3/año y 21,4 m^3/s su caudal medio, con grandes variaciones, desde 300

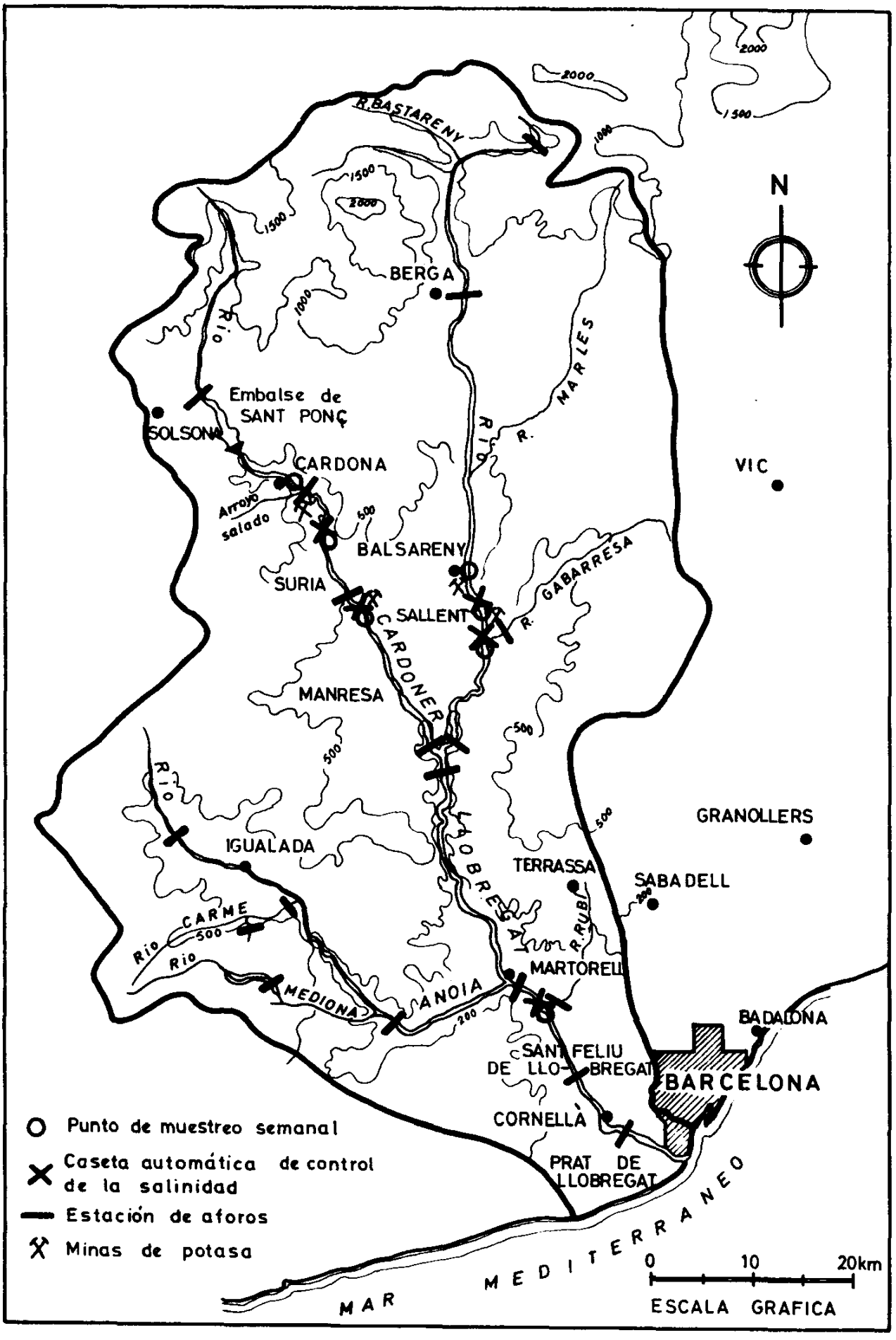

Fig. 3.— Cuenca del Llobregat y situación de las minas de potasa y puntos de control hidrométrico

hasta 1700 hm^3/año y caudales desde 3 hasta más de 1000 m^3/s en avenidas.

4. - PRODUCCION DE POTASA

La producción mundial en el año 1976/1977 fué de 40 millones de toneladas de cloruro potásico (ClK) equivalentes a 25 millones de toneladas expresadas como K_2O (potasa) y dedicadas casi en su totalidad a la obtención de abonos.

España ocupa el octavo lugar como productor, siendo exportador, según puede verse a continuación:

Miles de toneladas de K_2O

País	Producción	Consumo
1. URSS	8250	5356
2. Canadá	5655	261
3. Alemania Oriental	3161	624
4. USA	2189	5290
5. Alemania Occidental	2148	1196
6. Francia	1567	1508
7. Israel	600	19
8. España	560	304
9. China Continental	300	319
10.Congo	244	4

Los ingresos por venta de la potasa superan los tres mil millones de pesetas anuales. Las reservas probadas se estiman en 100 millones de toneladas de K_2O. Hasta el año 1971 la totalidad de la producción nacional procedía de la zona catalana en la cuenca del río Llobregat. A pesar de la puesta en marcha de las explotaciones de Navarra, dicha zona catalana supone el 90% de la producción española.

La silvinita es el mineral beneficiado en las explotaciones potásicas de la cuenca del Llobregat, permitiendo alcanzar concentraciones de hasta un 60%, riqueza mínima exigida en el mercado internacional para la fabricación de abono de calidad (el ClK puro supone una riqueza del 63,1 %).

5. - GENERACION DE APORTES SALINOS EN EL PROCESO DE ELABORACION DE LA POTASA

El mineral beneficiado casi exclusivamente en la cuenca potásica catalana es la silvinita. En la mina se encuentran unos 25 m de silvinita en dos capas separadas por halita, cuyo uso está formado por sal pura y cuyo techo es carnalita en una potencia de unos 50 m.

El proceso seguido en la actualidad para la obtención de la potasa a partir de la silvinita es el de flotación, que ha sustituído totalmente al anterior de disolución y cristalización selectivas.

La primera fase del proceso consiste en la molienda primaria para reducir hasta un máximo de 4 mm el tamaño del grano, que llega así a una instalación de tamices, separando la granulometría menor de 0,8 mm, que se envia a la celda de desquistaje y el resto a la sección de molienda húmeda. Con fuerte concentración, las espumas se conducen a los hidroseparadores, donde se separan regulando la dilución de los finos arcillosos que rebosan al decantador de lodos para su sedimentación y posterior secado hasta un 5 % de humedad, para ser transportados a la escombrera.

La pulpa de la parte inferior de los hidroseparadores se conduce a los clasificadores, desde donde con las adiciones necesarias (rebose espesadores mixtos, adyuvantes de floculación como aceite de pino y una amina) pasan a la flotación, basada en la separación del cloruro potásico al burbujear aire a través de la pulpa. La espuma de la flotación en las celdas primarias tiene un 56 % de riqueza en K_2O (89 % de ClK), pudiendo llegar en la celda cuaternaria hasta un 61 %.

El concentrado de las celdas se centrifuga hasta un 4-6 % de humedad y se seca en horno para ser almacenado y expedido con la riqueza deseada.

Los resíduos del proceso de fabricación pueden llegar a reducirse a los remitidos a las escombreras cuando las aportaciones de agua procedentes de distintas secciones compensan las pérdidas hidráulicas, pero también pueden haber excesos y reboses, más o menos incontrolados, que supongan un aporte directo de salinidad, además de las aguas sobrantes que por su contenído en otras sales (principalmente cloruro magnésico) no puedan cumplir tal cometido.

Los residuos obtenidos en los filtros rotativos, una vez eliminada el agua, tienen como composición 2,5% ClK, 91,7% ClNa y 5,8% impurezas y se transportan a las escombreras.

6. - OTROS APORTES SALINOS

Además del vertido directo de fábrica existen por lo menos tres otras fuentes de salinidad.

Una de ellas es el lavado y rezume de agua de las escombreras de sal, que en la cuenca del Cardoner se sitúan en barrancos y en la del Llobregat en parte sobre la llanura fluvial y en parte sobre los flancos del valle, tambien afectando a algunos barrancos. En algunas escombreras se recoge la salmuera que sale perifericamente y se conduce a la planta de producción, y en otras se derivan las aguas de tormenta para evitar en lo posible el contacto con el material salino, pero la eficacia de recogida no es completa y con seguridad existen fugas que se vierten de forma difusa en los ríos.

El agua de mina fue un grave problema en la mina de Sallent y fue origen de un vertido continuado hasta que la explotación del pozo afectado cesó y se inició recientemente una nueva explotación, produciéndose una mejora notable de la salinidad en el tramo próximo al río Llobregat.

No puede descartarse la existencia de un vertido natural de aguas salinas ya que en Cardona la sal gema y la carnalita afloran y la morfología delata un proceso diapírico aún activo, y en Suria llegan a estar a escasa profundidad bajo el río. No obstante, tal aporte de salinidad no era importante antes de establecerse las explotaciones ya que el río tenía una salinidad del orden de 80 mg/l de Cl^- de acuerdo con un análisis de 1912, y en especial de acuerdo con los datos de aguas del río infiltradas en los acuíferos del delta de Llobregat con edad superior a 100 años; se deduce una salinidad entre 80 y 100 mg/l de Cl^-. En la actualidad el vertido "natural" en el Arroyo Salado que recorre el alforamiento de Cardona es mucho mayor, aún después de recoger parte del agua superficial para ser incorporada al proceso de tratamiento del mineral, debido a la fuerte alteración morfológica que ha supuesto las instalaciones mineras y las escombreras próximas en las características de la cuenca receptora y curso del Arroyo, el cual posee tramos subterráneos. En este caso, el aporte "natural" es en gran parte una consecuencia indirecta de la minería.

7. - IMPORTANCIA DEL APORTE SALINO

En la década de 950 a 960 el aporte salino debido a la minería potásica se había estimado en 3 kg/s de Cl/Na. La descarga es variable de un día a otro, y a fin de evaluarla directamente, en 1969 se realizó una cuidadosa y prolongada campaña de medidas encontrándose un aporte medio de 5 kg/s de ión Cl (430 t de Cl por día aproximadamente, MOP 1969, 1970). Los vertidos directos de las plantas de tratamiento mineral eran de alrededor de 115 l/s (10.000 m^3/día)con una concentración media de 43 g/l de Cl, variable espacial y temporalmente entre menos de 30 g/l y 220 g/l de Cl.

Aunque en la cabecera de los ríos Llobregat y Cardoner existen pequeños aportes salinos procedentes de arroyos salados que drenan facies evaporíticas del garummiense prepirenaico, además de aguas de minas de lignito y de diversas poblaciones e industrias, las aportaciones de sal en la confluencia de los ríos Cardoner y Llobregat, aguas abajo de la zona minera, se originan en un 90 % en las zonas potásicas, y la adición aguas abajo a causa de población e industria (el regadío es casi inexistente) no altera esa proporción ya que hasta Martorell tiene poca importancia.

8. - CONTROL DE SALINIDAD

Debido al aumento previsible de salinidad en el río Llobregat a causa de las explotaciones potásicas, hace 50 años, casi simultáneamente al planeamiento y comienzo de las explotaciones, se creó la Comisión inspectora de la salinidad del río Llobregat (CISALL) que actuó ininterrumpidamente hasta el año 1967, en que sus funciones fueron absorbidas por la Comisaría de Aguas del Pirineo Oriental, sin interrupción de actividad. El control consiste en una sistemática de mediciones periódicas de parámetros químicos en relación con la salinidad.

En cada una de las casetas para la toma automática de muestras de agua del río indicada en la figura 3, cada hora se bombea una cantidad constante a unos depósitos en los que se acumula para su posterior análisis, cambiando cada 24 horas de depósito. Así, cada semana se tiene una muestra representativa de cada uno de los días de la misma.

De este modo se tiene el análisis diario de los cloruros de los puntos de control y el promedio semanal de MgO. Los datos correspon

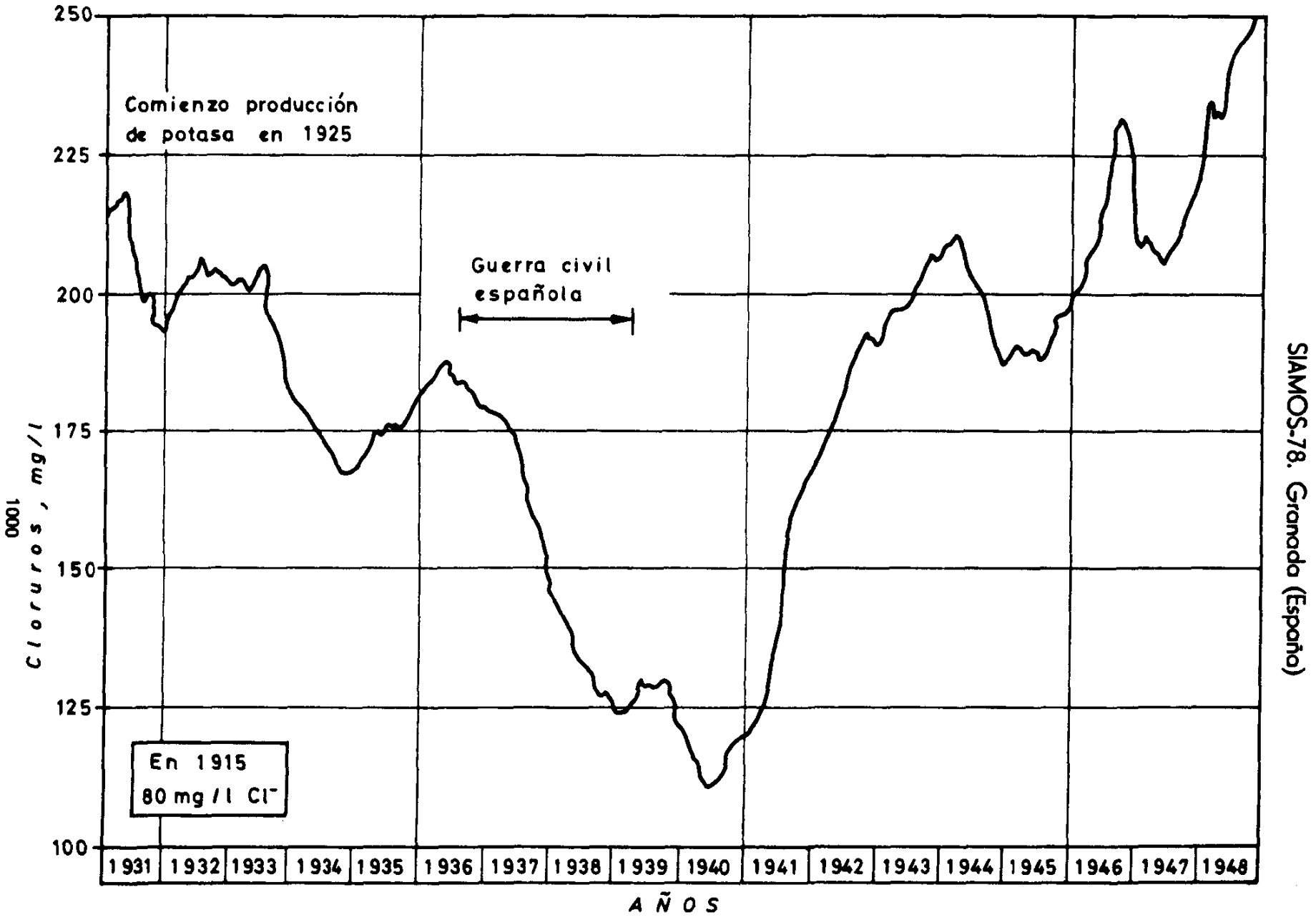

Fig. 4.— Evolución del contenido en cloruros en el acuífero aluvial de Cornellá (final del valle). Responde al contenido en el río con un desfase de 6 a 10 meses y suaviza las flutuaciones

dientes a la caseta de Pallejá permiten conocer las características del río Llobregat a la entrada de su delta.

Se confeccionan las correspondientes hojas de análisis y también en función de los caudales se calcula el aporte salino total en los distintos puntos.

Además cada semana se toma una muestra antes de las explotaciones potásicas en los ríos Cardoner y Llobregat.

Con carácter mensual se realiza una inspección más exaustiva, tomando 51 muestras del río Cardoner y 88 muestras del río Llobregat, en las que se determina el pH, alcalinidad, conductividad y cloruros, a fin de conocer las posibles variaciones en puntos estratégicos de la cuenca y al mismo tiempo tener un conocimiento estadístico de la calidad.

9. - RESULTADOS ANALITICOS

En la figura 4 se indica la evolución de la salinidad en Martorell entre los años 1930 y 1949, pudiéndose apreciar el descenso que supone el paro de las minas durante el periodo de la guerra civil española, durante el cual casi se retornó a las condiciones naturales. En ese tiempo la alteración morfológica del paisaje aún era pequeña.

El aporte hídrico guarda cierta relación proporcional al aporte salino total, y la concentración salina decrece al crecer la aportación de agua, aunque con notables desviaciones. En efecto, una lluvia tormentosa, frecuente en esta zona, si cae sobre terrenos salinos será origen de una importante contribución de sal, pero no lo será si cae localizada en el Bajo Llobregat. En la figura 5 se indica la evolución de la salinidad desde el año 1968 hasta el 1977, así como las aportaciones salinas en toneladas de Cl/día, y se aprecia claramente lo anteriormente expuesto.

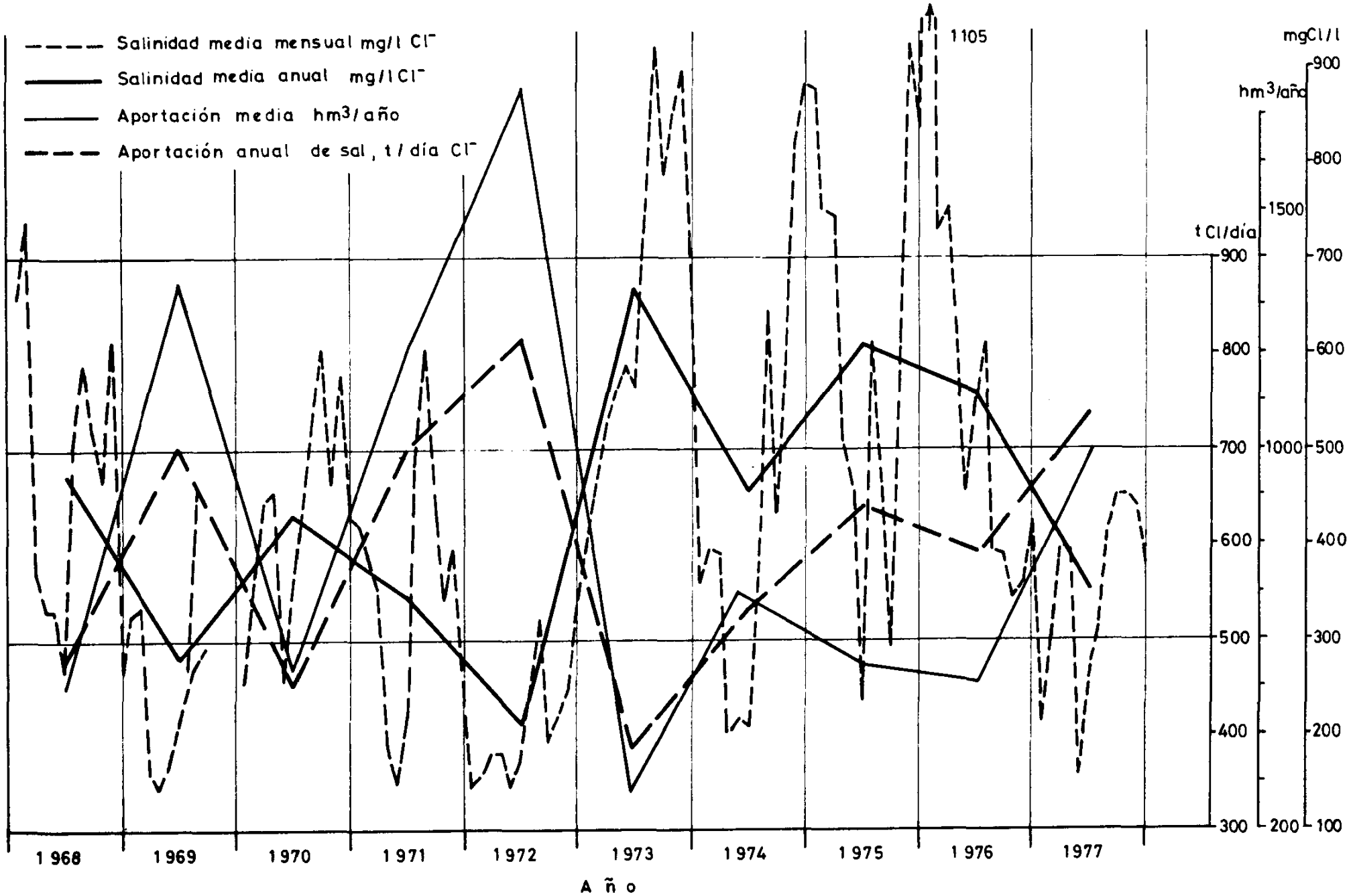

Fig. 5.—Evolución de las aportaciones y salinidad en el rio Llobregat entre 1967 y 1977

Si en medias mensuales, hasta el año 1949 no se superaron los 260 mg/l de Cl, en los últimos años se han llegado hasta alcanzar incluso valores superiores a 1000 mg /l, lo que supone que en algunos días se hayan encontrado concentraciones de 2000 mg /l en el agua superficial del río Llobregat, tal como se ha encontrado en algunos momentos de estiaje en la toma de abastecimiento de agua en Barcelona.

En años con aportaciones hídricas menores de 700 hm^3, la salinidad media anual ha superado los 400 mg /l con un maximo de 668 mg /l (año 1973), lo que ha supuesto medias mensuales de hasta 600 mg /l y puntuales de 1100 mg/l.

Con aportaciones de agua superiores a los 1000 hm^3/año la salinidad media anual se ha mantenído inferior a 350 mg /l.

Los valores medios antes relacionados muestran, no obstante, una gran irregularidad como se ve en la figura 5, irregularidad que concuerda en general, con el régimen de aportaciones de agua.

La figura 6 ilustra la evolución de las características químicas a lo largo del río. La influencia de la zona potásica en el incremento mineral es muy claro.

10. - INFLUENCIA SOBRE LAS AGUAS SUBTERRANEAS

Los únicos acuíferos interesantes y explotados en la cuenca del río Llobregat aguas abajo de la zona potásica son los aluviales del río, principalmente los del valle bajo y delta que, como se ha comentado, son una importante pieza en el aprovechamiento hídrico de los recursos del río.

Tal como se ha expuesto en el apartado 2, no existe ninguna conexión subterránea entre la cuenca minera y estos acuíferos, y la salinidad solo puede transmitirse a través de las aguas del río. Las extracciones en estos aluviales son muy intensas de modo que su recarga procede mayoritariamente de agua del río. Así, el agua extraída muestra con mayor o menor desfase y amortiguamiento la evolución de la salinidad del agua del río, como muestra al figura 7. El tránsito entre el acuífero libre de Cornellá, al final del valle del Llobregat, en facil conexión con el río (desfase entre 6 y 10 meses), y su extensión en profundidad como acuífero cautivo, hasta Prat de Llobregat, es de 10 años para un recorrido de 6 km. En este caso

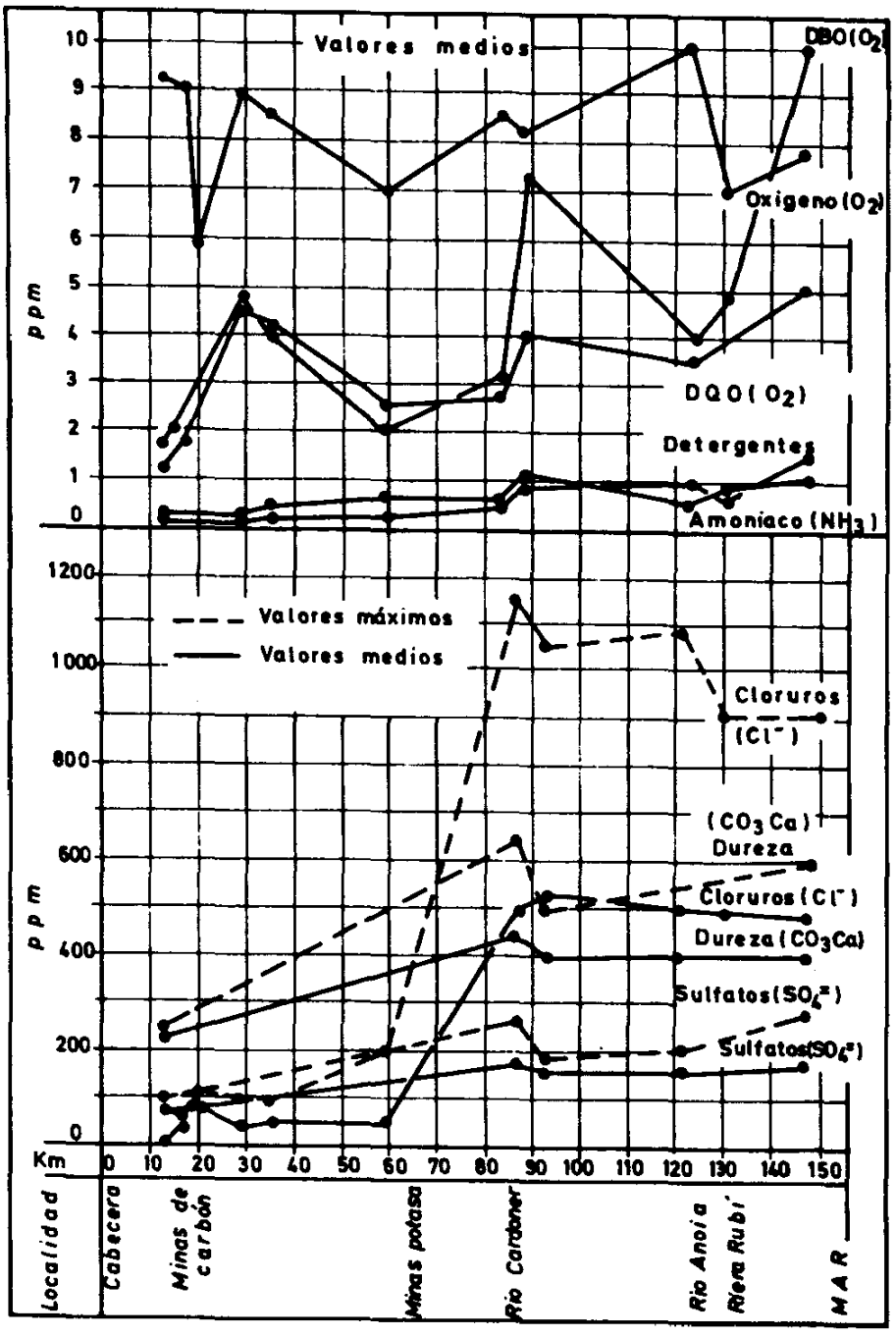

Fig.6.— Evolución de los parámetros químicos de calidad a lo largo del rio Llobregat. Año 1971 - 1972

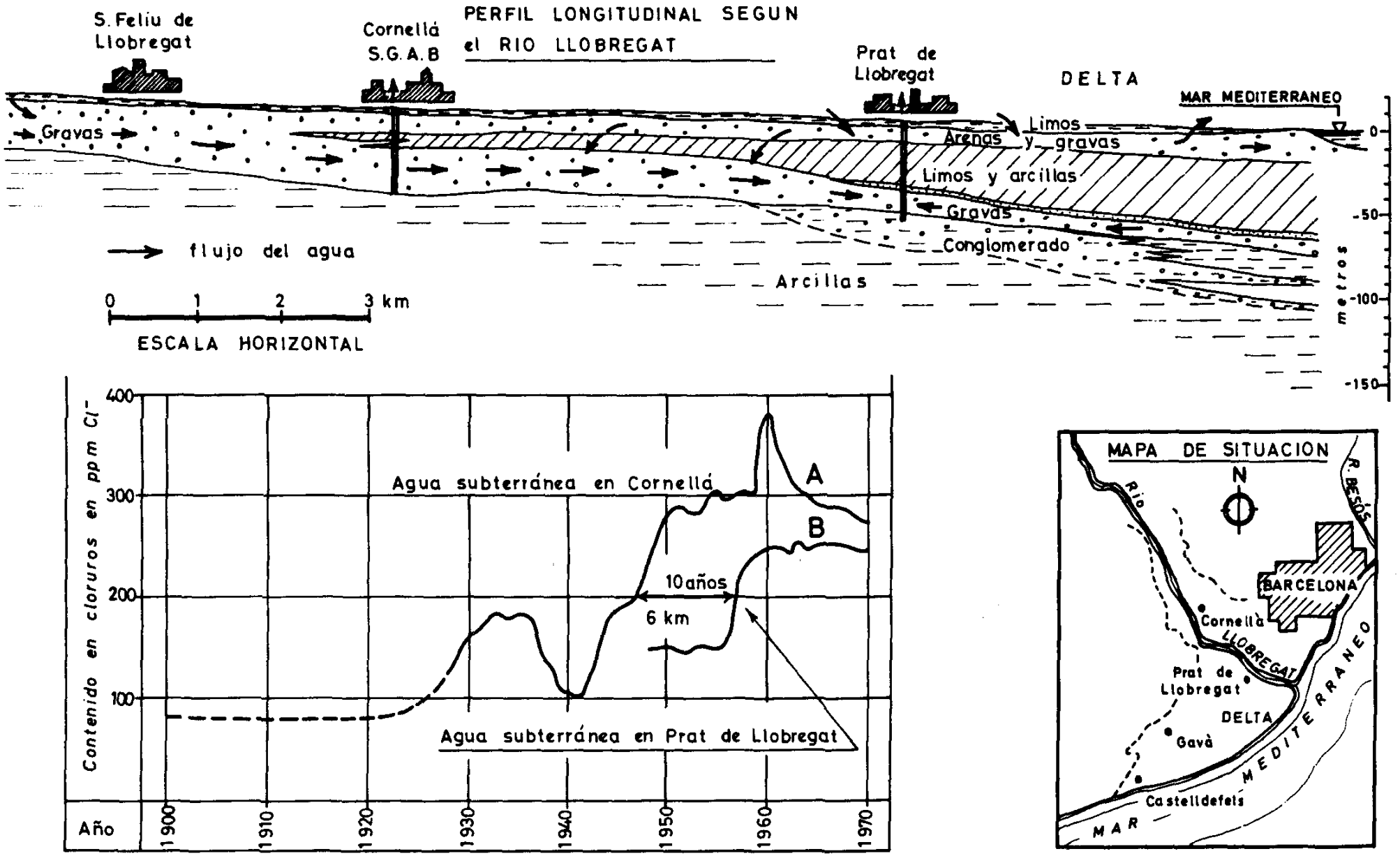

Fig. 7.– Perfil hidrogeológico longitudinal del Bajo Llobregat y gráfico que muestra el retraso de 10 años en la transmisión de las variaciones de salinidad entre dos puntos situados a 6 km en el sentido del flujo

las variaciones de salinidad del río se pueden utilizar a modo de trazado seminatural para obtener un mejor conocimiento hidráulico de los acuíferos. Con un estudio geohidroquímico de detalle es posible determinar el movimiento de las diferentes fuentes de agua (Custodio 1968 y 1968 b).

En la figura 8 se muestran las zonas del Bajo Llobregat principalmente afectadas por la salinidad del agua del río Llobregat. Esta salinización es causa de problemas industriales y de regadio , además de los que se producen en los abastecimientos públicos. Ciertas industrias deben utilizar agua desmineralizada, y esta salinización grava fuertemente los costes del tratamiento.

Un simple estudio geohidroquímico permite diferenciar las aguas procedentes de infiltración del río Llobregat, aún alteradas por el uso o por concentración de riego, de la contaminación salina de origen marino, ya que las relaciones iónicas y los diagramas químicos son notablemente diferentes.

11. - POSIBLES MEDIDAS PARA CORREGIR LA SALINIDAD

Los posibles sistemas para reducir el vertido salino son de dos tipos: a) en el propio origen, b) evacuando las salmueras sin producir salinización de las aguas continentales.

La reducción de salinidad en el orgien comporta:

- eliminar o reducir el vertido de las plantas de tratamiento del mineral. Con los nuevos procesos de tratamiento se ha conseguido aumentar la producción sin aumentar la descarga, pero no se han eliminado, a pesar de que en teoría sería posible hacerlo. La antigüedad de algunas instalaciones, el que los circuitos de agua no estan racionalizados y sea costoso modificarlos para conseguir el mayor reciclaje, y los vertidos fortuitos y accidentales sin suficiente protección del derrame, explican parte de ese vertido no eliminado, además del problema que supone la incorporación de sales de magnesio contenídas en el mineral a las aguas de proceso.

- proteger adecuadamente las escombreras y recoger efectivamente las aguas que escurren y que se infiltran, utilizandolas para el proceso del mineral.

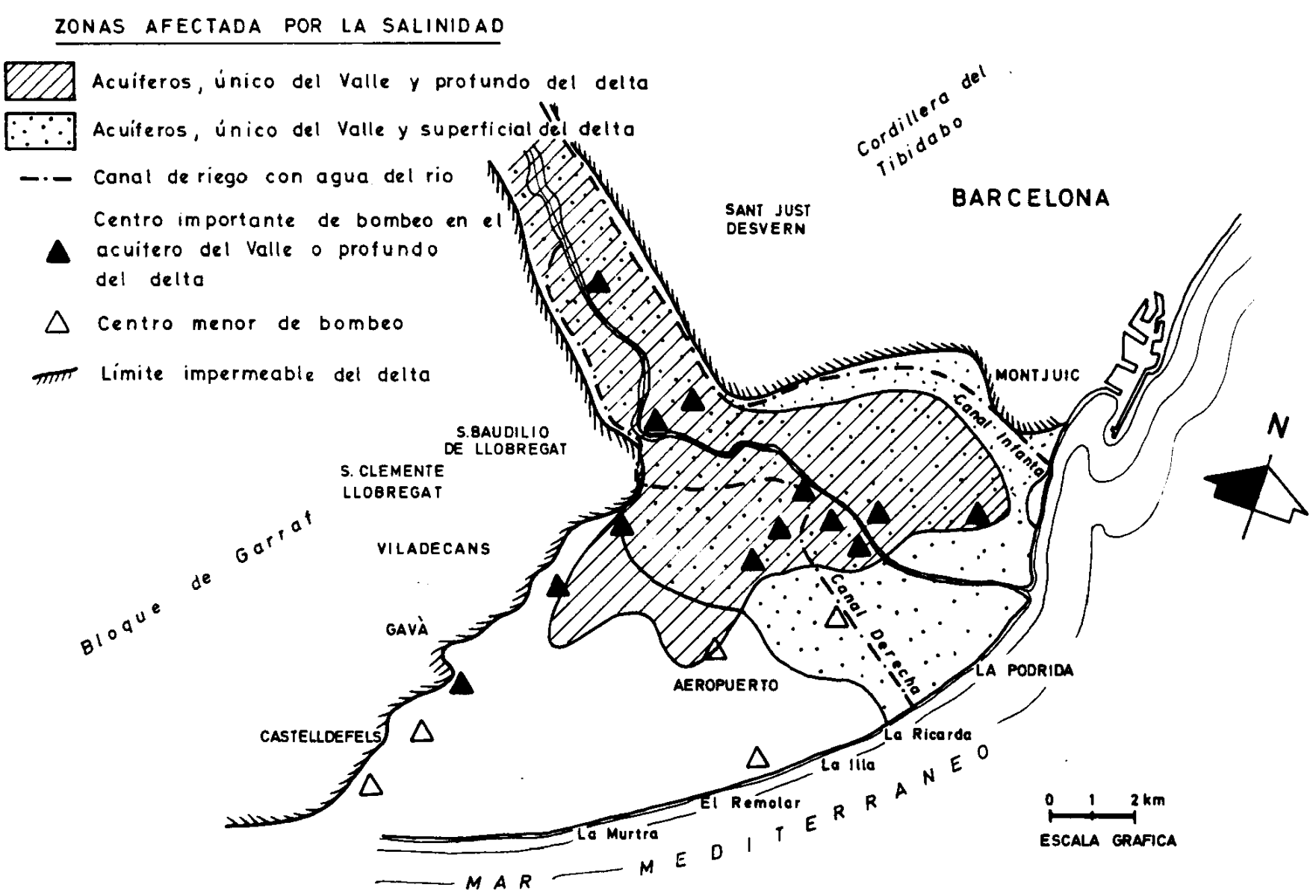

Fig. 8.— Zonas de los acuíferos del delta del rio Llobregat afectados por la salinidad del agua del rio Llobregat. En el acuífero superficial se producen por agua de canal con toma en el rio y en el profundo por los intensos bombeos, justo hasta los mismos, de acuerdo con la figura 7.

- recoger las salidas "naturales" de salmuera y aguas muy salinas, también para ser utilizadas en el proceso del mineral, compensando las pérdidas de agua por evaporación y por incorporación al mineral tratado residual (sal comun) que se devuelve a la mina como relleno.

- eliminar o utilizar las aguas de filtración en mina. Estas aguas, que se producían mayoritariamente en Sallent, han sido eliminadas al suprimir el pozo afectado.

El éxito de estos métodos radica en las propias empresas mineras. La evacuación de las salmueras sin producir salinización de las aguas continentales solo puede alcanzar un grado suficiente de eficacia si se recogen los vertidos difusos procedentes de escombreras y otros orígenes antes de su incorporación al río, lo cual no es sencillo y puede requerir obras y estudios de detalle.

Se han analizado diversas soluciones. Las que consistan en la evaporación forzada o solar en balsas de las salmueras no resultan apropiadas por su elevado coste en las circunstancias locales de coste de la energía, operación, bombeo previo y disponibilidad de terreno. La solución de vertido en avenidas naturales o provocadas por desembalse en presas aguas arriba es también costosa, insuficiente y con repercusiones graves en el funcionamiento de los aprovechamientos de agua superficiales y subterráneas, río abajo.

La solución de la inyección profunda es atractiva desde el punto de vista económico. Se han realizado tres sondeos de reconocimiento y ensayo para estudiar la viabilidad técnica de tal inyección (Custodio, Galofré, 1971; Fayas, 1972), en los emplazamientos señalados en la figura 2, pero solo se encontró un tramo permeable en el sondeo MOP-3, que después del ensayo resultó insuficiente (Custodio, 1976), decidiéndose abandonar esta posibilidad. En el momento actual la única solución externa que queda es la de un colector de salmueras a lo largo de los ríos Llobregat y Cardoner, para su vertido al mar. Se han realizado varios estudios de colector salino y en el último, en 1975, su coste se evaluó en 500 millones de pesetas que hoy deberían transformarse en algo más de 1000 millones de pesetas si se actualizan los precios y se añaden algunos complementos de obra y vertido necesarios para su aceptación. La viabilidad real de tal colector parece presentar algunos problemas técnicos, pero sobre todo económicos en cuanto al coste anual de mantenimien

to , que probablemente sería muy elevado dadas las condiciones del trazado, del líquido a transportar, y de las obras de recogída, regulación y tratamiento en origen necesarios.

La Administración sigue trabajando conjuntamente con las minas para reducir el vertido en origen.

Así, en la explotación de Cardona se aprovecha el caudal de unos 2 l/s, en concentraciones del orden de 100 g/l Cl^- del arroyo salado para introducirlo en el proceso de fabricación.

En la explotación de Suria se recogen las aguas de las escombreras, canalizándolas en lo posible e inyectando en mina resíduos de decantador. También para su aprovechamiento en otra fábrica se transportan resíduos, basicamente cloruro sódico, reduciendo así su aportación a las escombreras.

Y finalmente, en la explotación de Sallent, como ya se ha expuesto, se ha suprimido el vertido que procedía del constante vaciado de las filtraciones de una mina, dejándola inundar tras los estudios y autorizaciones necesarios.

También en esta explotación se recogen las aguas de una nueva escombrera, conduciéndolas separadamente al interior de la fábrica para su utilización en el lavado de mineral.

Durante años han existido unas balsas para acumular aguas salinas, evitando su vertido en épocas de estiaje y aprovechando las crecídas del río para lograr una dilución.

En todos los casos las mejoras se han obtenído conjugando las protestas de los usuarios aguas abajo, la insistencia de la Administración y la responsabilización de las propias explotaciones mineras.

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EL AGUA EN LA MINERIA DE LA HULLA, ESTUDIO DE UNA ZONA PILOTO

Fernández Aller, R.

ABSTRACT : In 1973 has been done a research about the water management in the coal mining area of Nalon river (Asturias coal basin in the nord of Spain). Now it is presented only a resume of such study mentioning the problematic created by the water drenage as well as the waste water from the coal preparation plants in operation during the period of the study. The most important polluants load is generated in the coal preparation plants. The drenage waters are alkalines so the pollution generated in the river by these waters is not very important, however it must be considered the great flow produced.

RESUME : En 1973 fut réalisée une étude de gestion de qualité de l'eau provenant des activités minières dont les déblais affectent la rivière Nalon. Cette communication est seulement un résumé de ladite étude et tente de représenter la situation créée aussi bien par les eaux de drainage des mines que par les installations de préparation du charbon, en fonctionnement pendant la période d'étude. En vue des résultats, le plus grand problème est présenté par les installations de préparation du charbon, tandis que les eaux de drainage, même si elles supposaient un grand volume, n'étaient pas très contaminantes étant donné leur caractère alcalin.

RESUMEN : En 1973 se realizó un estudio de gestión de calidad del agua proveniente de las actividades mineras cuyos vertidos afectaban al río Nalón. La presente comunicación es sólo un resumen de dicho estudio y trata de reflejar la situación planteada tanto por las aguas de drenaje de mina como por los lavaderos de carbón en funcionamiento durante el período de estudio. A la vista de los resultados, el mayor problema lo presentaban los lavaderos de carbón, mientras que las aguas de drenaje, si bien suponían un gran volumen no eran muy contaminantes, dado su carácter alcalino.

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INTRODUCCION

La importancia del agua como recurso básico en el desarrollo industrial y urbano en la planificación territorial, hace necesaria una adecuada gestión de su consumo, utilización y vertido de efluentes.

Al Norte de España se encuentra la región asturiana caracterizada por su gran potencial indutrial y minero, así como por su incesante evolución demográfica. En dicha región se ubica la cuenca carbonífera (hullera) más importante del país, circunstancia que viene a condicionar en gran medida sus recursos de agua.

La citada cuenca minera está formada por cuencas correspondientes a cuencas hidrográficas en su mayor parte, por lo que se seleccionó una de ellas; la Cuenca o Zona minera del Nalón, como zona piloto para el estudio de la gestión del agua en la minería de la hulla. El mencionado estudio fue realizado en y con datos relativos al periodo de Octubre (1971)-Diciembre (1972). La zona está explotada por minería subterránea y comprende nueve pozos de extracción y cuatro plantas de preparación de carbones o lavaderos, agrupados en una extensión de 26 Km^2.

METODOLOGIA

En las explotaciones de carbón y, en general, en la de cualquier mineral, cabe distinguir respecto al agua dos categorías de procesos, la extracción y las plantas de preparación, centrándose la problemática de los primeros en las aguas de drenaje, mientras que en las segundas la constituyen los vertidos de efluentes procedentes del flotado y lavado del mineral.

Para estas dos categorías de procesos, y mediante visitas y entrevistas, se obtuvo una detallada información sobre producción, consumos, usos del agua, caudales de agua de drenaje y diagradas de flujo de las aguas de drenaje en los pozos y de las aguas de proceso en los lavaderos, conociendo asimismo los cursos de las corrientes de agua residuales y sus puntos de vertido a los medios receptores.

Se realizaron dos campañas de toma de muestras de agua y se analizaron las características físicas y químicas determinantes de la calidad de las aguas de consumo y vertido.

CONSUMO DE AGUA

El consumo de aguas potables procedentes del sistema de distribución

se cifra en 3,5 l/persona-día, para las minas y de 12,3 l/t de carbón tratado y los costes (energía y tarifas) no presuponen ninguna incidencia económica sobre la tonelada extraída o lavada.

Los pozos de extracción del carbón precisan agua para usos industriales (maquinaría, talleres, etc.) y para servicios auxiliares (limpieza de vagones, casa de aseos, riego, etc.). En ambos casos no se necesita agua potable y por ello se utiliza, en general, aguas de río, o aguas de desagüe del propio pozo.

El consumo medio de este tipo de agua es de 0,21 m^3/t de carbón, siendo los consumos máximo y mínimo de 1,75 m^3/t y 0,23 m^3/t, respectivamente.

De estas aguas un 62%, aproximadamente, proceden del desagüe de mina, un 36% de los ríos próximos y un 2% de pozos de agua.

En las plantas de preparación la procedencia de aguas destinadas a fines industriales proviene, en un 6% de cursos fluviales y el resto del desagüe de pozos mineros próximos a las citadas plantas. El consumo medio es de 2,8 m^3/t de carbón lavado.

Cualquiera que sea su procedencia, las aguas para utilización industrial, no suponen ningún coste por pago de tarifas, pero deben considerarse los costes energéticos en aquéllos casos que sea necesario bombear el agua, o por las necesidades de mantenimiento. Los citados costes tuvieron una incidencia sobre el coste de la tonelada tratada de 0,02 pts, salvo para un pozo que fue de 2,04 pts (1972).

En cuanto a la calidad de las aguas a las que hace mención este apartado, y teniendo en cuenta los usos a los que son destinadas, hay que hacer constar las diferencias existentes entre las que proceden de río y las que proceden del drenaje de las minas. Las primeras, en base a las características físicas y químicas analizadas, son aguas de muy buena calidad para utilización undustrial o servicios.

Las segundas son aguas de bastente peor calidad que las anteriores, (dureza media de 42°f, contenido medio de sólidos disueltos de 71,4 mg/l, y un contenido de 100 mg/l, 55 mg/l y 4,9 mg/l en calcio, magnesio y sílice, respectivamente). El uso de este tipo de aguas en compresores, refrigeradores, calderas de vapor, etc. presenta graves inconvenientes dado que, por las condiciones de presión y temperatura a las que trabajan estas máquinas térmicas, se producen en ellas arrastres de partículas sólidas, incrustaciones y depósitos que alteran su correcto funcionamiento y que incluso pueden producir graves perjuicios en los pozos, si llegado el caso tales alteraciones produjeran una avería graves (parada del compresor, por ejemplo).

En las plantas de preparación la importancia del uso del agua radica en que interviene directamente en el procesado de lavado de carbón. Si

bien no existe una normativa específica que fije la calidad de las aguas en relación con la eficacia del proceso, las aguas de drenaje por su elevada dureza y contenido en sólidos en suspensión no resultan idóneas, siendo por ello aconsejable utilizar las aguas de los ríos a cuyas márgenes se encuentran ubicadas las plantas,después de un proceso de sedimentación (ver tabla n° 1).

AGUAS DE DRENAJE

Las aguas de drenaje constituyen en las minas el capítulo más importante debido a dos razones fundamentales:

- El volumen drenado ascendió a 25 Hm^3, lo que significa un volumen importante de agua, en su concepto de recurso hídrico.

- Bajo el punto de vista minero, estas aguas son un factor negativo en la explotación, precisándose para su eliminación: construcción de zanjas y canales, instalaciones de bombeo, personal de control y mantenimiento, todo lo cual se traduce en costes que gravan la producción

a) Aspectos hidrogeológicos

Atendiendo a las características del terreno y dado el sistema de explotación subterránea de estas minas cabe hacerse tres preguntas respecto a estas aguas:

- ¿Cúal es la procedencia de estas aguas en el interior de las minas?, ¿Cúales son las características de su circulación en el terreno? ¿Cúal es su mecanismo hidrogeológico?.

Por lo que respecta a la hidrogeología de la zona, dadas las características poco permeables de las formaciones que en ella afloran, se puede asegurar la inexistencia de ningún acuífero importante en la misma.

Sin embargo aunque no existan formaciones litológicas permeables que puedan dar lugar a acuíferos en el sentido hidrogeológico de esta palabra, los pozos mineros constituyen verdaderos acuíferos con características muy similares en su estructura, aunque no en su desarrollo a los acuíferos de tipo Kárstico.

La explotación de una mina de carbón en el subsuelo precisa en cualquier caso, bien sea mediante socavón en ladera de montaña bien mediante pozos de extracción, el ir abriendo continuamente nuevos frentes a través de talleres de explotación y construir asimismo galerias y transversales, necesarios para el transporte y los servicios a diferentes profundidades o plantas según el ritmo de la explotación.

Estas construcciones subterráneas y la extracción contínua del carbón y de la roca van dejando oquedades más o menos grandes.

Los frentes ya explotados son rellenados de nuevo, mediante relleno

o hundimiento en un periodo de tiempo variable según el sistema de explotación seguido en las labores mineras.

La zona que nos ocupa, el Valle del Nalón, y en general toda la cuenca carbonífera asturiana, está formada por materiales paleozoicos,que debido a los movimientos tectónicos hercinianos y posteriores está fuertemente plegada tanto en dirección NO-SE como NE-SO, siendo predominante la primera a causa de la citada tectónica herciniana.

Estos plegamientos han producido en las formaciones fracturas más o menos grandes y en ocasiones deslizamientos y fallas importantes,que han creado en los materiales una permeabilidad secundaria en mayor o menor grado que permite la infiltración del agua en el subsuelo a través de los terrenos de cobertura.

Partiendo de un terreno de estas características las explotaciones de carbón, según la forma someramente descrita anteriormente, alteran el subsuelo no sólo mecánicamente sino también hidrogeológicamente , ya que en los minados explotados, rellenos o hundidos, se produce un considerable aumento de la permeabilidad y de la capacidad de almacenamiento de agua, así como la infiltración a partir del suelo, bien sea a través de fracturas preexistentes o provocadas por la explotación si ésta no es demasiado profunda.

Tenemos así una superficie más o menos alterada por la que se infiltra el agua a través de las fracturas; unos minados explotados . por los que el agua circula con velocidad variable, y la explotación propiamente dicha, formada por una serie de plantas de labores a profundidades diferentes y comunicadas por los frentes de avance según el sistema de explotación, es decir, tenemos tanto en sentido horizontal como en sentido vertical, un gran número de oquedades comunicadas entre sí y por las que el agua circula con gran rapidez.

Buena muestra de ello es que las aguas de la planta primera de las explotaciones son por lo general aguas poco turbias que han sido recolectadas después de una infiltración producida en los terrenos de cobertura o de los minados hundidos y compactados sin haber tenido - tiempo de producir arrastres de sólidos.

Sin embargo, y a medida que se va profundizando y el agua va circulando por las labores a los minados recientes, aún sin compactar suficientemente, se observa un aumento considerable del contenido en sólidos.

El aumento que se aprecia en el contenido en sales disueltas con la profundidad, viene asimismo a demostrar la existencia de agua de velocidad lenta que permiten la solubilización de las sales del terreno.

Aun cuando el aumento del contenido en sólidos en suspensión pudiera

justificarse por el aumento producido en la conducción del agua por las galerías, y por tanto pensar que todas las aguas son de velocidad lenta no se justifica, por el contrario, dada la naturaleza geológica de los terrenos que se encuentran en la zona, ni los caudales desaguados, ni la correspondencia existente entre las puntas de precipitación y las puntas de caudal, cuyo desfase es del orden de uno a cinco días.

Todo lo anterior nos lleva a la deducción de que existen en la mina, aguas de velocidad lenta que circulan a través del terreno inalterado y aguas de velocidad rápida que transcurren a través del terreno alterado.

b) Aspectos cuantitativos y cualitativos

El volumen de agua drenada supone 10 m^3 por tonelada de carbón extraído y el caudal medio desaguado por todos los pozos estudiados, se cifra en 0,8 m^3/sg, siendo el máximo 1,3 m^3/sg. El caudal máximo por pozo (considerando la media anual), fue de 0,153 m^3/sg, oscilando en el resto de los mismos entre 0,042 m^3/sg y 0,099 m^3/sg. Hay que tener en cuenta que las variaciones de caudal son considerables de unas épocas a otras, así para uno de los pozos pudo establecerse entre 0,267 m^3/sg en Mayo y 0,070 m^3/sg en Septiembre, es decir en una relación de 4 a 1.

Sin contar con la amortización de las instalaciones de bombeo, canales, tuberías, etc., los costes de estas aguas son imputables al consumo energético producido por las bombas y a los costes del personal bombero.

El consumo energético evaluado para el periodo considerado de 31.087.920 Kwh, lo que representa un consumo de 1,18 Kwh por m^3 de agua desaguada.

Los costes del personal bombero ascendieron a 11.664.000 pts, es decir un 27% del coste total, que fue de 42.752.000 pts (costes de 1972).

El coste del m^3 de agua desaguada fue en dicho periodo de 1,62 pts y el coste por tonelada de carbón fue, de acuerdo con lo expuesto, de unas 16,3 pts.

Hay que reseñar que el coste que implica el desagüe, supone un 94,91% del coste total que ocasiona el agua en los pozos estudiados.

Los análisis de las muestras tomadas de estas aguas (tabla n° 2) indican:

1°) El contenido en sales disueltas es bastante alto (media de 1.199 μ ℓ 1 cm-1) y aunque por este concepto su calidad es mediocre, son perfectamente utilizables

TABLA Nº 2

Valores medios, máximos y mínimos de las características físicas y químicas en las aguas de drenajes de los pozos mineros y de las aguas de vertido de los pozos mineros y lavaderos de carbón

[illegible]		[illegible]								
determinación	Unidad	Valor medio	Valor máximo	Valor mínimo	Valor medio	Valor máximo	Valor mínimo	Valor medio	Valor máximo	Valor mínimo
Temperatura	º C	12	14	9	12,2	14	10,2	9,3	10,5	6,5
Conductividad	$\mu s \cdot l^{-1}$ cm^{-1}	1.199	1.475	450	1.242	1.562	962	839	1.275	372
[illegible]	mg/l	774,7	1.184,4	553,8	674,4	713,3	565,3	1.161,6	2.214,6	643,2
pH	--	7,7	8,2	7,2	7,9	8,3	7,4	7,9	8,3	7,4
Dureza	º f	47,6	65,6	29,2	41,4	64,1	26,9	23,6	38,2	6,7
[illegible]	mg/l	983	1.277	564	974	1.277	823	936	1.181	556
[illegible]	mg/l	64,9	121,2	43,8	58,5	1.084	37,5	38.526	87.953	864
[illegible]	mg/l	55,7	104,2	41,3	46,5	981	27,3	21.611	51.634	599
[illegible]	mg/l	252	439	59	256,8	392	77	267,3	564	775
[illegible]	mg/l	24,7	36	21	27,6	38	18	27	45	14
[illegible]	mg/l	143,6	158	36	85,7	132	36	49,3	144	14
[illegible]	mg/l	43,3	61	24	42	68	24	23,4	38	6
[illegible] (total)	mg/l	0,16	0,16	< 0,16	0,11	0,16	< 0,10	0,23	0,65	0,16
[illegible]	mg/l	< 0,16	0,16	-	< 0,16	0,16	-	0,10	0,16	-
[illegible]	mg/l	1,47	2,56	1,25	1,85	2,56	1,25	2,48	6,25	1,56
[illegible]	mg/l	5,2	14,8	3,2	5,3	10,8	3,7	4,6	9,7	1,6
[illegible]	mg/l de O_2	4,1	6	2,6	3,5	8,6	1,2	1.928	3.488	256
[illegible]	mg/l	2,8	4,6	1,3	1,9	3,6	1,5	1,7	3,9	< 0,5
[illegible]	mg/l	0,021	< 0,149	0,161	0,015	0,026	0,006	0,052	0,157	0,025
[illegible] amoniacal	mg/l	0,49	< 1,08	0,26	1,449	2,168	0,73	0,168	0,468	2,30
O_2 disuelto	mg/l	-	-	-	-	-	-	145,1	279,8	25,6
Fenoles	mg/l	-	-	-	-	-	-	0,05%	0,78%	0,009

2°) Su dureza es en general alta (media de 47,6°F), lo que las define asimismo de calidad mediocre, en cuanto a sus posibles usos, como ya hemos comentado en el anterior apartado. La dureza de estas aguas en algunos pozos las hace totalmente impropias para cualquier uso.

3°) El contenido en sólidos en suspensión, es contrariamente a lo que se podía suponer bastante bajo, siendo la media de éste, de 64,9 mg/l; (hasta un contenido de 70 mg/l las aguas pueden calificarse, en este sentido, como aceptables). Sólo en un pozo esta cantidad se rebasa llegando a 121,7 mg/l.

4°) Si bien en general el contenido en sulfatos es bajo (media de 150 mg/l) esta cifra se rebasa ampliamente en varios pozos, no pudiendo por ello establecer una característica común. Hay que tener en cuenta que por encima de los 100 mg/l de contenido en sulfatos, las aguas favorecen la corrosión y que en la mayoría de los usos industriales, el contenido de éstos debe estar comprendido entre 60 y 200 mg/l.

5°) Como se deduce de la elevada dureza de estas aguas, los contenidos en calcio y magnesio son también altos, lo que tiene el gran inconveniente de que por esta causa se produzcan depósitos en los circuitos de refrigeración.

6°) A partir de 3 mg/l de O_2 de valor al permanganato (materias orgánicas), las aguas pueden calificarse como dudosas, es decir, posiblemente afectadas por una contaminación de origen orgánico. En general esta cifra se supera con creces, llegando a valorarse de 5 ó 6 mg/l de O_2, que son ya signo inequívoco de este tipo de contaminación, a mayor abundamiento, los contenidos en nitritos y amoniaco (en N) con un valor medio de 0,021 mg/l y 0,64 mg/l respectivamente, refuerzan esta teoría.

Como conclusión podemos definir las aguas de desagüe con respecto a a su calidad química, como aguas de mediocre calidad para la mayoría de los posibles usos y afectadas de una ligera contaminación de origen orgánico.

VERTIDO DE AGUAS

a) Pozos de extracción

El volumen anual de aguas vertido a los emisarios por los pozos es de 26,5 Hm^3, de los que 90% proceden de las aguas de desagüe, correspondiendo el 10% restante a las aguas residuales de origen doméstico o industrial.

Cada pozo tiene un sistema de vertido diferente, mezclando, por lo general, aguas de diversa procedencia, si bien en la mayoría de ellos, las aguas de desagüe son vertidas de forma separada.

El caudal medio de vertido para todos los pozos de la zona es de 0,837 m^3/sg, este acudal varía considerablemente (teniendo en cuenta la incidencia de las aguas de desagüe) de unas épocas a otras, así mientras en el mes de Mayo el caudal medio total fué de 1,255 m^3/sg, aproximadamente, en el mes de Septiembre éste descendió a 0,485 m^3/sg.

Las características físicas y químicas de las aguas vertidas son sensiblemente iguales a las de las aguas de desagüe (ver tabla n° 2), si bien se observa en sus valores medios una ligera disminución de su conductividad y contenido en sales, debido a un cierto grado de dilución al mezclarse dichas aguas con otras de menor contenido en sales, y un aumento en materia orgánica, nitritos, N_2 amoniacal y N_2 orgánico, debido a aporte de aguas residuales de origen fecal.

Considerando unicamente los vertidos causados por las aguas de desagüe y sus características químicas, se ha confeccionado la tabla n° 3, en la que figura estimativamente la carga anual vertida.

b) Plantas de preparación

El empleo de agua en las diversas etapas de tratamiento y según los diferentes esquemas o procesos de éstos, precisa de la utilización de un número elevado de aditivos y en cantidades considerables, tal es el caso de la magnetita, utilizada en el proceso de separación por líquidos densos (entre 445 y 590 t/año en algunos lavaderos); de diversos reactivos o creosota, en el tratamiento de finos de flotación, floculantes utilizados en los filtros, etc.

Todos estos aditivos introducen en las aguas, a lo largo de todo el proceso, una gran cantidad de agentes contaminantes difíciles de determinar en la mayoría de los casos por la naturaleza química y comercial de los citados aditivos.

Los efluentes vertidos provienen principalmente de los reboses de estériles y flotación, así como de las balsas de decantación de finos y estériles, observándose que en el primer caso el contenido de fenoles y sólidos en suspensión es, aproximadamente, un 50% mayor que en el segundo.

Las plantas de preparación de carbones estudiadas trabajan en circuito abierto; todas ellas reciclan parte del agua, pero aún así los volúmenes de agua aportados y vertidos son sensiblemente iguales. El volumen medio anual de vertido fué de 4 Hm^3.

En la tabla n° 2 figuran las características físicas y químicas de estas aguas y en la tabla n° 3 los datos de cargas de contaminantes estimadas, vertidas a los medios receptores.

El perjuicio del vertido de estas aguas es doble, ya que de una parte se contaminan en gran medida los medios receptores, mientras que por otra parte, en el aspecto productivo, las toneladas vertidas de

TABLA Nº 3

CARGA MEDIA ANUAL VERTIDA (1972) A LOS EMISARIOS POR LOS POZOS MINEROS Y LAVADEROS DE CARBON

CARACTERIS.	UNIDAD	POZOS MINEROS	LAVADEROS DE CARBON	TOTALES
Sólidos en suspensión	t	1.234	129.370	130.704
Sulfatos	t	7.840	1.466	9.306
Cloruros	t	597	111	708
Calcio	t	2.356	202	2.558
Magnesio	t	1.109	96	1.205
Hierro (total)	Kg	2.906	985	3.891
Manganeso	Kg	2.377	369	2.746
Aluminio	t	19	10	59
Sílice	t	140	19	159
Materias orgánicas	t de O_2	103	7.012	8.015
Nitratos	t	50	7	57
Nitritos	Kg	396	336	732
N_2 amoniacal	t	26,5	17	43,5
N_2 orgánico	t		612	
Fenoles	Kg		394	

sólidos suponen un 6,2% del tonelaje bruto de carbón tratado y un 11,45% del tonelaje de carbón lavado, estimándose que de los citados sólidos un 32% son finos de carbón.

COSTES DEL AGUA

El coste total ocasionado por la gestión del agua en los pozos de la Zona del Nalón, ascendió a 45.000.000 pts (refiriendo todos los datos al año 1972), lo que implica una incidencia sobre la tonelada de carbón extraído de 0,30 $. El coste producido por el consumo energético ocasionado por el agua, fue para el mismo periodo de un 71% del coste total.

Por lo que respecta a los pozos, en particular, el coste del agua osciló entre 7.500.000 y 1.440.000 pts por año y pozo, siendo, aproximadamente, de 5.000.000 pts el coste medio.

Por tonelada de carbón extraído el coste del agua varía asimismo considerablemente, así mientras dicho coste fue para unos pozos de 9 pts para otros lo fue de 29 pts, según las diferentes circunstancias implicadas en cada caso.

El coste que supuso las aguas del desagüe fue por término medio, un 97,2% del coste total del agua.

Por lo que se refiere a los lavaderos el coste fue tan sólo del 4% de los costes del agua en toda la zona, y el coste de agua por tonelada tratada en éstos descendió a 1,20 pts y considerando el total de costes 40 pts.

POSIBLES PLANTEAMIENTOS EN LA PLANIFICACION DEL AGUA EN LA CUENCA MINERA

A la vista de los resultados del estudio del agua efectuado en los anteriores apartados, se puede llegar a establecer una serie de condiciones y alternativas de modo general:

La problemática actual del agua en las cuencas hulleras presenta tres características primordiales:

1°) Un gran volumen de aguas excedentes, procedentes del drenaje en el interior de las minas.

2°) Aguas de aporte para usos netamente industriales, de mala calidad cuando éstas provienen de las aguas de desagüe.

3°) Un considerable volumen de aguas de vertido, muy contaminadas en el caso de proceder a los lavaderos de carbón y ligeramente contaminadas si proceden de los pozos, pero que en su conjunto alteran gravemente los medios fluviales.

Las posibles soluciones técnicas que pueden arbitrarse para optimizar la gestión del agua, son de varios tipos y su elección vendrá rela-

cionada principalmente por factores socio-económicos y medio-ambientales.

En líneas generales y a modo de una posible alternativa técnica podría establecerse:

a) Eliminación de sólidos mediante decantación y reducción de dureza por un proceso de ablandamiento, para la utilización de aguas de drenaje en usos industriales.

b) Plantas de preparación de carbones con funcionamiento en circuíto cerrado y tratamiento mecánico y físico-químico de los efluentes residuales.

A este respecto, hay que hacer constar que en la zona minera estudiada ya se ha construído un lavadero con circuíto cerrado de aguas que irá sustituyendo a las actualmente en funcionamiento, con lo que el problema quedará en gran parte solucionado.

De otra parte y dado que los pozos de extracción y lavaderos se encuentran agrupados en un área de extensión reducida, pueden establecerse alternativas conjuntas, que si bien encarezcan los costes de conducciones y de bombeo en su caso, reducirán en gran medida los costes de instalación y mantenimiento de las necesarias plantas de tratamiento de agua.

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CONTROL Y TRATAMIENTO DE AGUAS RESIDUALES EN MINERIA

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ABSTRACT : Waste waters from mining activities are very polluant, specially those which have a low pH, because most of the metals are solubilized. The problem of mining waste waters (generally drenage waters) is solved in two different ways : a) reducing the infiltration of the waste water flow by means of control systems called "Mining Technics" and b) by means of waste water treatment "Water Technics" like oxidation or evaporation ponds, neutralization, desalation and aireation processes.

RESUME : Les eaux résiduelles provenant de l'activité minière sont contaminantes, spécialement si elles sont de caractère acide, étant donné que à pH acide, la plus grande partie des métaux deviennent solubles. La solution du problème posé par les eaux résiduelles (généralement eaux de drainage) se trouve en deux aspects : a) diminuer l'infiltration ou le volume d'eau dans la mine, les méthodes appliquées sont de contrôle et ont été appelées : "Techniques Minières". b) Traitement des eaux résiduelles ou "Techniques de l'Eau". Parmi ces dernières techniques on étudie les bassins d'oxydation et d'évaporation, la neutralisation, la désalinisation et l'aération.

RESUMEN : Las aguas residuales procedentes de la actividad minera son contaminantes, especialmente si son de carácter ácido, dado que a pH ácido, la mayoría de los metales se solubilizan. La solución del problema planteado por las aguas residuales (generalmente aguas de drenaje) tienen dos enfoques : a) disminuir la infiltración o el volumen de agua en la mina, los métodos aplicados son de control y se han denominado "Técnicas Mineras", b) tratamiento de las aguas residuales o "Técnicas del Agua". Entre estas últimas técnicas se estudian los estanques de oxidación y evaporación, la neutralización, la desalinización y la aireación.

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1. INTRODUCCION

La evolución progresiva del desarrollo industrial, urbano y agrícola mejora de forma constante el nivel de vida de la sociedad, por lo que este desarrollo es evidentemente positivo e irreversible.

Ahora bien, el desarrollo lleva consigo la producción acelerada de residuos que afectan de forma más o menos importante a los medios receptores, deteriorándolos y ocasionando con ello, entre otros, dos efectos graves: perjuicios de índole sanitario y reducción de los recursos naturales. A esta relación causa-efecto se la denomina contaminación.

De los tres medios receptores naturales, aire, suelo y agua, es sin duda el tercero, el agua, el recurso natural más escaso y el que en términos de balance se puede decir más desequilibrado con relación a la demanda.

Entre las actividades industriales contaminadoras del agua, la minería juega un importante papel, teniendo por otra parte unas peculiaridades propias que la diferencian del resto de las industrias en terminología práctica.

Con respecto al agua, en la minería cabe establecer dos fases, la primera fase son las operaciones de explotación bien sea ésta subterránea o a cielo abierto y la segunda fase son las operaciones de preparación del mineral, pudiéndose considerar a este fase como un proceso industrial clásico.

Las explotaciones mineras tienen un condicionante muy claro en relación con el agua y es que no pueden escoger su emplazamiento puesto que el yacimiento es un recurso natural que se encuentra ubicado en un área determinada bajo unas condiciones geológicas dadas. De otra parte, en contraposición a la industria, que es consumidora, la minería es productora de agua.

La producción de agua en las minas varía considerablemente según el tipo de terrenos perforados, tipo de explotación, método de explotación, etc., así en Canadá, por ejemplo, los volúmenes bombeados varían de 3 l/h/t a 160 l/h/t en minería subterránea y de 70 a 90 l/h/t en minería de superficie. En España, en minería subterránea del carbón el caudal de agua de drenaje puede estimarse en unos 10 m^3/t (Cuenca Asturiana).

Las principales fuentes de agua del drenaje de mina son la infiltración de las aguas subterráneas (acuíferos) y la percolación a través de los minados de aguas de lluvia. En menor grado; otras fuentes de agua son las derivadas del uso de la misma en las operaciones mineras (operaciones mecánicas, perforación, supresión de polvo, refrigeración y acondicionamiento de aire, aguas sanitarias y drenaje procedente del relleno hidraúlico).

En las explotaciones a cielo abierto la mayor incidencia la acusan las aguas de lluvia.

Los procesos industriales de preparación de menas son consumidores de agua y su consumo viene definido por el carácter del mineral o minerales a obtener y los tipos de procesos empleados para ello.

2. LA CALIDAD DE LAS AGUAS DE DRENAJE DE MINA

La calidad de las aguas de dranaje de mina (minería superficial o subterránea) se puede caracterizar por sus propiedades físicas o químicas.

Las propiedades físicas a considerar son:

- La temperatura (tiene una influencia muy escasa)
- Los sólidos en suspensión y disolución

Las propiedades químicas:

- pH
- Alcalinidad o acidez
- Dureza
- Iones metálicos en disolución
- Contenidos en sales
- Contenidos en sílice
- Contenidos en ión amonio
- Contenidos en aceites y grasas
- Radiactividad

Resulta evidente que no se puede generalizar sobre los parámetros o propiedades citadas y que cada yacimiento es un caso concreto que debe ser analizado y estudiado de forma particular, sin embargo, si se pueden establecer unas características comunes sobre los grandes grupos en que se pueden subdividir las explotaciones mineras.

A) Rocas industriales

Por las propias características geológicas, los yacimientos de rocas industriales, son, por lo general secos, y por tanto tienen poca influencia sobre los medios receptores acuosos, en todo caso, el único parámetro a tener en cuenta sería los sólidos en suspensión producidos por el arrastre del material al agua y el color aparente que estos sólidos ocasionasen.

B) Explotaciones de Sales

Como es fácil de suponer las características de las aguas provenientes de este tipo de explotaciones se caracterizan por su alto contenido en sales (cloruros, sulfatos, etc.).

El tratamiento de estas aguas es un problema espinoso, no tanto por las posibilidades técnicas del mismo, sino por sus aspectos económicos dados los grandes volúmenes de aguas a tratar y sus altos contenidos en sales. La solución técnico-económica más aceptable depende en gran manera de la localización del yacimiento, y de las características geológicas (hidrogeológicas) y geomorfológicas del entorno del yacimiento.

Uno de los sistemas hoy más utilizados en la minería de sales en relación con la evacuación de sus aguas de drenaje es la inyección profunda cuando existen condiciones favorables para la ejecución de esta técnica, salvaguardando en todo momento la calidad de las aguas subterráneas de los acuíferos explotables.

C) Minería metálica

Gran parte de los minerales brutos (mena + ganga) están asociados con sulfuros metálicos (inorgánicos) los cuales pueden ser clasificados - en términos generales como piritas. Algunas de las menas más comunes explotadas para la recuperación de metales como plomo, cinc, plata y molibdeno son asimismo sulfuros.

En general, la exposición de piritas y otros sulfuros inorgánicos a la atmósfera produce la oxidación de éstos a sulfatos. La disolución de los sulfatos en las aguas subterráneas o superficiales produce una degradación (variable) de las mismas a la vez que aumentan su acidez. El aumento de la acidez incrementa la solubilidad de los elementos - asociados y compuestos en el agua.

Los iones más comunes que se encuentran en las aguas de drenaje son: hierro (ferroso y férrico), aluminio, calcio y magnesio (dureza), manganeso, cobre, cinc, plomo, cadmio, níquel, arsénico, plata, cloruros, fluoruros, sulfatos, fosfatos y materiales radiactivos. La presencia y concentración de cada ión varía según el tipo de mineral, localización geográfica, período hidrológico, etc. Esta variación puede llegar a ser incluso significante entre las diferentes áreas, o, en su caso, galerías o pisos de una misma mina.

Por tanto, la minería metálica, produce unas aguas de drenaje que pueden reunir todas las propiedades contaminantes establecidas al comienzo del apartado.

D) Combustibles sólidos

La minería del carbón produce unas aguas de drenaje ácidas o básicas según que en el yacimiento existan piritas o no, en el primer caso, la acidez vendrá determinada como en el caso anterior por la oxidación

de los sulfuros (piritas) y en el segundo, al no existir condiciones que den lugar a la acidez, la posible existencia del ión bicarbonato facilitará el carácter alcalino de las aguas.

Además de lo dicho, en las aguas de drenaje de las minas de carbón - hay que considerar los sólidos en suspensión y el color aparente por ellas producido en las aguas.

En todos los casos citados la contaminación producida por las aguas de drenaje tiene carácter inorgánico, no obstante la presencia de ión amonio puede definir una contaminación de origen orgánico, al proveñir este ión de la descomposición de la urea, si bien en ocasiones - puede ser ocasionada su presencia por la degradación anaerobia de los nitratos.

3. METODOS PARA REDUCIR LA CONTAMINACION PRODUCIDA POR LAS AGUAS DE DRENAJE

Los métodos para reducir la contaminación producida por las aguas de drenaje se basan en los principios:

A) TECNICAS MINERAS
a) Eliminación de las condiciones favorables a los materiales potencialmente contaminantes (Métodos de Control)
b) Control de las aguas de infiltración

B) TECNICAS DEL AGUA
a) Control y reducción de vertidos
b) Tratamiento del agua

4. TECNICAS MINERAS

4.1.- METODOS DE CONTROL

4.1.1.- Minería de superficie

La contaminación de las aguas ocasionada por la minería de superficie proviene como ya se ha dicho de la exposición de los materiales potencialmente contaminantes a la oxidación. Antes de la explotación estos materiales se encuentran frecuentemente por debajo del nivel freático, estando por lo tanto aislados del ambiente atmosférico, no teniendo - por ello oportunidad de producir una contaminación significante. Se trata, pues, con los métodos de control de crear condiciones similares a las existentes previamente a la explotación, o si se prefiere, reducir la exposición de los materiales contaminantes al ambiente atmosférico.

Entre estos métodos cabe citar:

- Segregación de materiales de relleno y enterramiento: La cubierta vegetal, y los terrenos no mineralizados excavados se utilizan como relleno, formando unas pilas en las que "se entierran" los materiales (Fig. 1) contaminantes (método utilizado en explotaciones de carbón al Este de los EE.UU.).

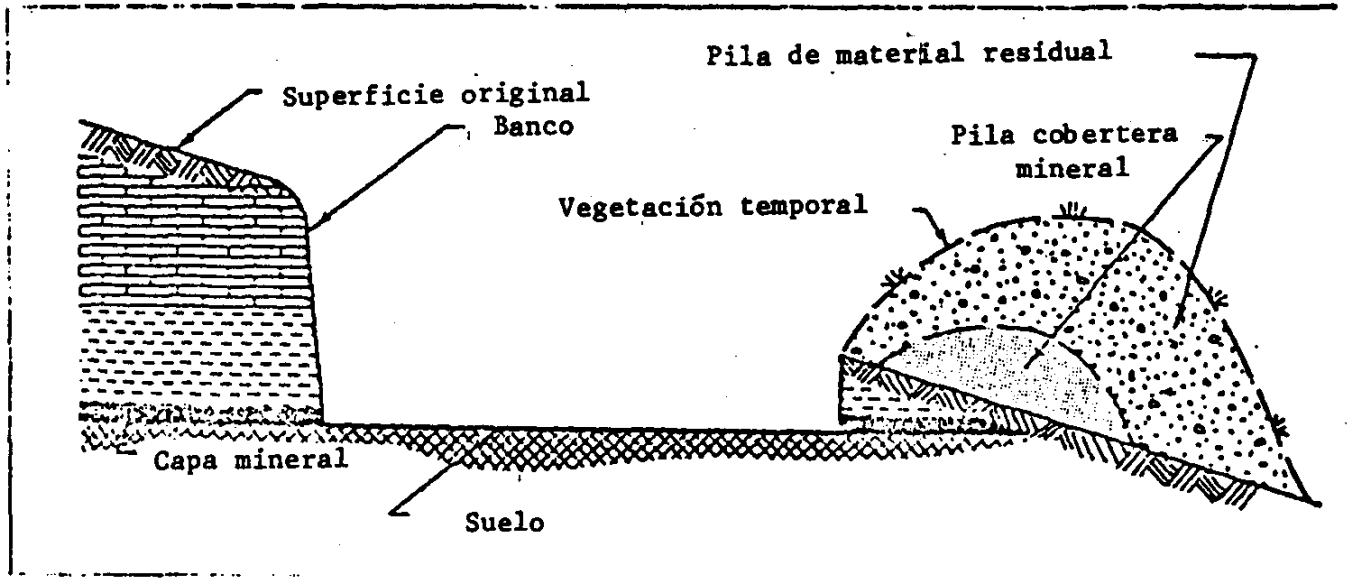


FIG. 1 - Segregación de materiales y enterramiento

- Barreras de mineral o explotación con frentes cortos (minería del carbón): Se deja sin explotar un frente de aproximadamente 9 metros de ancho a lo largo de la explotación, proveyendo así a ésta de un cierre natural reteniendo el agua de superficie o drenada durante las operaciones de laboreo.

- Explotación en frente largo (minería del carbón): Se trata de una aplicación de este mismo sistema de laboreo en minería subterránea o minería de superficie, mediante la maquinaria adecuada.

- Otros métodos: Se basan en la forma de apilar los materiales excavados aprovechando el sistema de explotación de la roca, la topografía de la zona o ambas circunstancias.

4.1.2.- Minería subterránea

En el caso de la minería subterránea, la contaminación de las aguas por disolución de iones metálicos subsiste y por tanto la solución solo tiene dos vías, o inundación o descarga nula de aguas de drenaje. Evidentemente estas soluciones son en gran medida teóricas y en todo caso complejas de llevar al terreno práctico.

Los métodos que hemos llamado de control serían:

- Inundación: Este sistema desde el punto de vista hidrogeológico requiere condiciones adecuadas y no siempre es posible, precisa socavones o galerías con pendiente hacia abajo en sentido de la explotación y dejar frentes de mineral a modo de cierre. El sistema es costoso por cuanto que durante la explotación exige el bombeo del agua infiltrada y dejar como se ha dicho unos frentes (barreras) de mineral no recuperable.

- Control de fracturas: la mayor parte del agua que penetra en las labores proviene del techo después de haber atravesado verticalmente los estratos suprayacentes. Durante las operaciones de laboreo se producen fracturaciones en el techo de las galerías que facilitan el paso del agua. Los sistemas de reducir este efecto son: a) dejar pilares de mineral como soporte, b) entibación como soporte adicional, c) galerías circulares para reducir los esfuerzos normales que actuan de forma más intensa en las zonas angulares de las galerías rectangulares y d) relleno de las fracturas.

- Otros métodos: son a) control de la extracción del mineral teniendo en cuenta que dentro de la explotación pueden existir materiales más contaminantes que otros, b) implantación de una atmósfera libre de oxígeno, utilizando la mina como almacén de hidrocarburos líquidos y gaseosos.

4.2.- CONTROL DE LAS AGUAS DE INFILTRACION

4.2.1.- Escombreras

El control de la infiltración superficial requiere o bien aislar las escombreras de las zonas de alimentación de agua, o bien disminuir la permeabilidad del suelo. Generalmente no es viable eliminar las grandes cantidades de materiales excavados en las operaciones mineras que además por otra parte son necesarios en ocasiones para los sistemas de relleno. Bajo estas condiciones, si la infiltración del agua es causa de formación de contaminantes, no cabe otra solución que un control "in situ" de la infiltración.

El sistema de control requiere la utilización de alguno de estos sistemas: capas impermeables, implantación de una cobertura vegetal, constitución de una red de drenaje o por último la combinación de estos sistemas.

Las capas impermeables pueden ser constituidas a base de arcilla, asfalto, hormigón, latex, plástico o materiales especiales y deben ser compactadas suficientemente. Este sistema de impermeabilización resulta muy costoso.

Una densa cobertura vegetal pueden en algunos casos reducir la infiltración. La vegetación tiende a reducir la velocidad del agua y aumenta la retención de ésta en el suelo facilitando así una mayor evaporación.

El establecimiento de una red de drenaje en las escombreras o explotaciones de superficie, es quizás el sistema más ampliamente utilizado. Este sistema no es suficiente "per se", para eliminar el problema de la contaminación de las aguas, debiéndose complementar, en su caso, con el tratamiento adecuado de las aguas drenadas.

4.2.2.- Minería subterránea

Las presentes técnicas tienen por efecto reducir la cantidad de agua que se infiltra al interior de la mina y subsecuentemente reducir el bombeo de las aguas de drenaje y en su caso, su posterior tratamiento.

La infiltración proviene de la recarga por precipitación de los acuíferos. La zonas de fracturación o fallas tienen una gran influencia sobre las líneas de flujo y frecuentemente sirven como canales interconexionados que conducen grandes cantidades de agua, estas zonas por tanto y en la medida de lo posible deben ser evitadas durante la explotación.

Algunos de los métodos para reducir la infiltración son los siguientes:

- Aumento de la escorrentía superficial: Esta técnica requiere eliminar las depresiones y explanar la superficie del suelo para incrementar la velocidad del agua, estableciendo asimismo una red de drenaje adecuada.

- Cierre de pozos de agua existentes y relleno de fracturas o fallas.

- Extracción de agua mediante bombeo de los acuíferos interceptados. El estudio hidrogeológico establecerá el tipo de acuífero o acuíferos de la zona de explotación determinando:
  - Las características de los niveles piezométricos
  - Las características del bombeo (bombeo de extracción o inyección del agua de acuíferos más superficiales en acuíferos más profundos, etc.).
  - Colocación de las rejillas en los pozos.

5.- TECNICAS DEL AGUA

Si bien los métodos anteriormente descritos tienen por objeto reducir la contaminación de las aguas de mina, son métodos que requieren típicamente un planteamiento de ingeniería minera. Sin embargo con frecuencia no son suficientes para reducir el problema a sus justos límites o no son viables técnica o económicamente hablando, por ello es preciso acudir a métodos que entran de lleno en el ámbito de la ingeniería sanitaria, o suponen una combinación de ambas técnicas minera y sanitaria.

5.1.- CONTROL Y REDUCCION DE VERTIDOS

Los presentes métodos son aplicables cuando las técnicas de control en las labores mineras para reducir la contaminación son inefectivas o muy costosas o bien como alternativa de tratamiento de las aguas, en aquellos casos en que éste resulte prohibitivamente costoso.

5.1.1.- Reutilizaciones de los vertidos

Esta técnica es conocida también como minería en circuito cerrado, porque el agua generada durante los procesos de laboreo y preparación de minerales, no es vertida a los medios receptores.

Los elementos básicos del circuito son:

a) Un sistema de recogida y transporte de las aguas drenadas de la mina hasta los estanques de almacenamiento.

b) Un sistema de bombeo y transporte del agua, de los estanques de almacenamiento hasta la planta de preparación.

c) Un sistema de transporte de estériles a presa de estériles.

d) Un sistema bombeo y transporte de agua de presa de estériles a estanques de almacenamiento.

e) En el caso de que una parte del agua almacenada en los estanques deba ser vertida a los medios receptores, habrá que estudiar su tratamiento adecuado.

Este método de control de las aguas, es aconsejable en regiones secas con escasez de agua. En regiones húmedas y especialmente en el

caso de minería a cielo abierto para el dimensionamiento de los estanques se deberá considerar las condiciones climatológicas y expresamente los períodos punta (Fig. 2).

El coste del sistema varía para cada caso particular.

5.1.2.- Estanques de evaporación

Grandes presas de retención pueden ser utilizadas con el fin de conseguir una alta evaporación y una mínima descarga a los medios receptores. En suelos permeables se requiere un revestimiento de materiales impermeables como arcilla. Los materiales arcillosos tiene a su vez la ventaja de adsorber elementos químicos contaminantes tales como los compuestos de arsénico.

El sistema debe ser diseñado para el máximo de flujo con la mínima evaporación. Resulta evidente que este sistema solo es viable en zonas áridas o semiáridas en las que la pluviometría sea menor que la evaporación. El requerimiento de un mantenimiento periódico es una limitación para el uso del presente método.

5.1.3.- Otros métodos

Además de los métodos ya citados cabe citar otros de aplicación muy particular debido a que su utilización es función directa no solo de la calidad de las aguas a verter, sino también de las características específicas de los medios receptores.

Entre estos métodos se pueden indicar los siguientes:

a) Irrigación por aspersión
b) Inyección profunda
c) Regulación de vertido

La irrigación por aspersión requiere que el agua no contenga elementos tóxicos para la vegetación y que no contenga asimismo altas concentraciones de sodio o sales solubres que puedan afectar la calidad de los suelos, especialmente a largo plazo. Es indudable que este sistema presenta graves problemas y por tanto su aplicación requiere detallados estudios.

La inyección profunda es un método muy conocido, que viene condicionado principalmente por las características hidrogeológicas del terreno y por la necesaria protección de la calidad de las aguas subterráneas.

Por último la regulación de los vertidos viene definida por las características de regulación de los cursos fluviales, lo que implica asimismo un detallado análisis para utilizar los puntos de vertido en los ríos o tramos del río más favorables.

5.2.- TRATAMIENTO DE AGUAS

5.2.1.- Tratamiento de neutralización

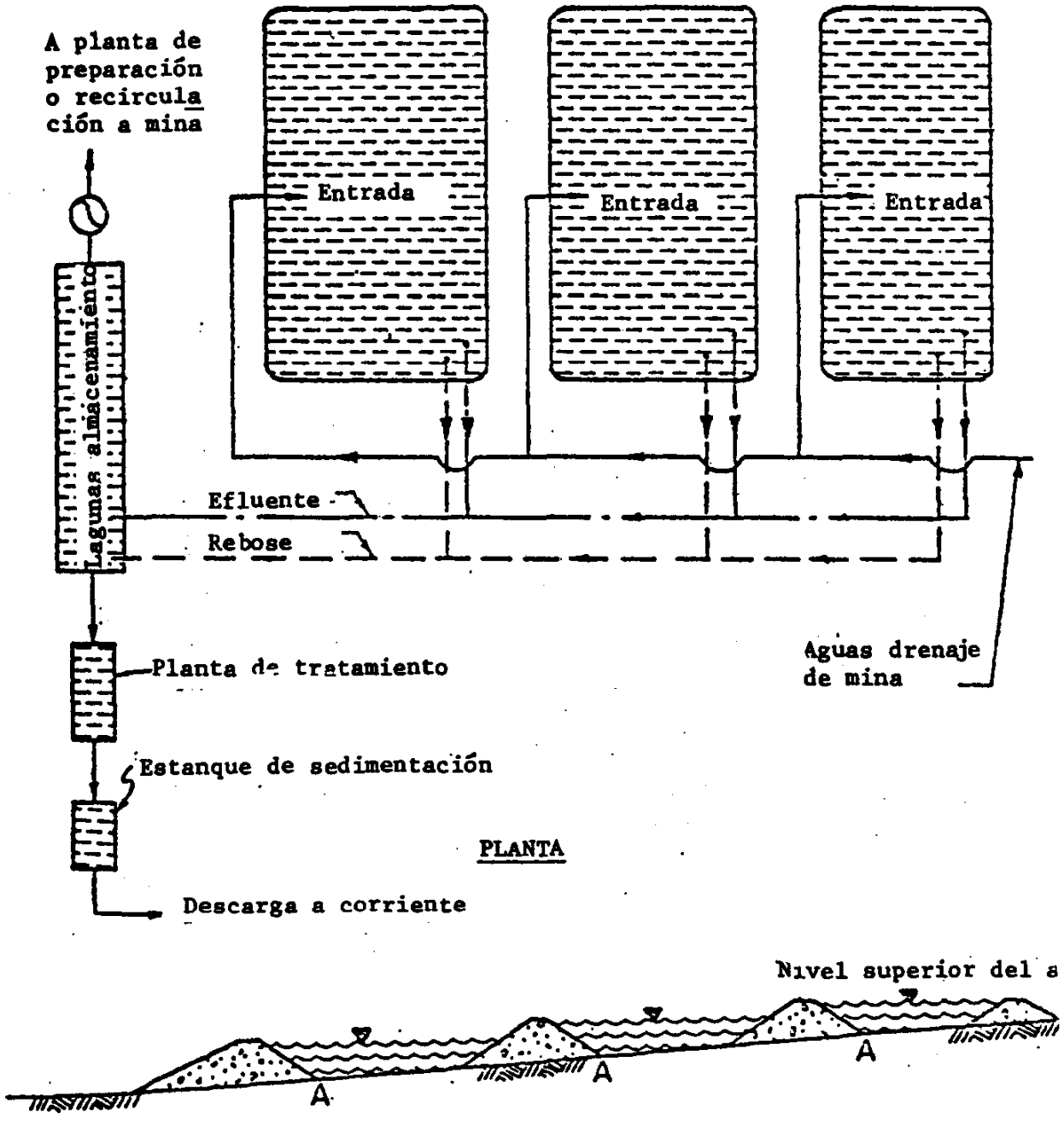


FIG. 2 - Reutilización de vertidos

Cuando las aguas de drenaje de mina son ácidas, la acidez puede ser neutralizada por la adición de agentes alcalinos. Con una adecuada elección del agente alcalino, muchos metales (cationes) pueden ser eliminados como hidróxidos insolubres. Los aniones como fosfatos , fluoruros y sulfatos pueden ser también precipitados mediante los citados agentes teniendo en cuenta el principio de insolubilidad.

De los estudios realizados en laboratorio (ver tabla 1) se puede ver la evolución de los agentes alcalinos más comunes, de acuerdo con su factor de basicidad, es decir la disponibilidad alcalina de cada material medida en gramos de carbonato cálcico por gramo de agente alcalino.

Los agentes alcalinos más usados son: cal rápida, cal hidratada, roca caliza, caliza en polvo, dolomía, magnesita, sosa caústica, cenizas de sosa e hidróxido de amonio; sin embargo, teniendo en consideración los grandes volúmenes de agua a tratar el factor coste es definitivo, por ello en la práctica los agentes más utilizados son cal, cal hidratada y caliza (ver tabla 1).

Las concentraciones de metales pesados en soluciones acuosas pueden por lo general ser reducidos por precipitación a hidróxidos insolubles, llegando así a concentraciones del metal en disolución de 1 mg/l o menos.

El pH al que se produce la precipitación es diferente para cada metal (ver valores típicos en tabla 2). En el caso de los metales anfóteros como cinc y aluminio, el metal se resolubilizará en la solución si ésta se hace demasiado alcalina.

TABLA 2

VALOR MINIMO DEL pH PARA LA PRECIPITACION DE IONES METALICOS COMO HIDROXIDOS

ION	pH	ION	pH	ION	pH	ION	pH	ION	pH
Sn^{+2}	4,2	Al^{+3}	5,2	Ca^{+2}	7,2	Ni^{+2}	9,3	Cal^{+2}	9,7
Fe^{+3}	4,3	Pb^{+2}	6,3	Zn^{+2}	8,4	Fe^{+2}	9,5	Mn^{+2}	10,6

Un caso especial y por otra parte frecuente, lo constituye la presencia de ion ferroso. El ion ferroso no alcanza el mínimo de insolubilidad a menos que la solución alcance un pH de 9,5, por el contrario el ion férrico es eliminado fácilmente por precipitación como hidróxido a pH próximo a 5. El sistema de eliminar el ion ferroso puede ser por tanto en base a condiciones alcalinas previa oxidación (aireación) a ion férrico o combinando ambos métodos. En presencia de ion ferroso, el tratamiento con caliza es difícil por cuento que en el proceso se forma CO_2 que tiende a bajar el pH de la solución.

TABLA 1

COMPARACION DE COSTE DE ALGUNOS AGENTES ALCALINOS UTILIZADOS PARA LA NEUTRALIZACION DE LAS AGUAS DE DRENAJE DE MINA

AGENTE	Composición	Factor de basicidad (1)	Relación de coste	AGENTE	Composición	Factor de basicidad (1)	Relación de coste
Cal rápida	CaO	1,786	1,60	Magnesita	CO_3Mg	1,186	2,65
Cal hidratada	$CaO(CH)_2$	1,351	2,30	Sosa caústica	50% NaOH	1,250	7,60
Caliza, roca	CO_3Ca	1,000	1	Ceniza de sosa	50% CO_3Na_2	0,943	4,75
Caliza polvo	CO_3Ca	1,000	1,25	Hidróxido amonio	$NH_4(OH)$	1,429	5,65
Dolomía	CO_3CaMg	0,543	5,40				

(1) Gramos de CO_3Ca equivalente por gramo de agente alcalino

En los casos en que las aguas de drenaje sean alcalinas, prácticamente no subsistirán los problemas arriba expuestos.

En la tabla 3, se comentan algunas de las características más importantes de los procesos de neutralización más comunes. Asimismo en las Figs. 3, 4 y 5 pueden verse algunos de los esquemas clásicos del sistema de neutralización utilizados en minería.

5.2.2.- Tratamiento de lodos

Los procesos de neutralización produce una gran cantidad de lodos que es preciso tratar (reducir su volumen por secado) y depositar en un lugar idóneo.

Sobre los métodos de secado no existen evaluaciones de casos concretos en la literatura, sin embargo, a modo de orientación y según diversos autores los costes pueden estimarse de acuerdo con la tabla 5.

Los métodos de secado y deposición son:

- Implantación de grandes depósitos (presas) de sedimentación.
- Almacenamiento de los lodos en minas profundas abandonadas.
- Utilización de lechos de secado porosos (0,3-0,6 m de espesor del lecho con un espesor de lodos de hasta 0,45 m).
- Filtración a vacio (se consiguen lodos secos con concentraciones de sólidos de hasta el 30%).
- Deposición de lodos en el terreno mediante el sistema de vertido controlado.

5.2.3.- Tratamientos de desalinización, desmineralización o ablandamiento

5.2.3.1.- Procesos de evaporación

Los procesos de evaporación son métodos convencionales de desalación de aguas salinas o salobres, produciendo aguas de alta calidad para usos potables o industriales.

En la actualidad hay tres diferentes procesos: 1) evaporación súbita en múltiples etapas (E.S.) o (MFE), 2) evaporación en tubos largos con multi-efecto (E.T.L.) o (ME-L.T.V.), 3) compresión de vapor (C.V.) o (V.C.).

El proceso de (E.S.) se basa en el hecho de que el agua hierve a temperatura más baja cuando es sometida progresivamente a presiones más bajas. El agua de alimentación es calentada a 93°C y se introduce en una cámara a presión reducida que causa una evaporación súbita (flash) de las partículas de agua. El vapor asciende en la cámara, se condensa en los tubos y el agua destilada es recolectada en un separador. Los sólidos disueltos permanecen con el agua de alimentación (salmuera). La salmuera fluye a una segunda cámara a presión más baja que la primera y una cantidad adicional de agua es destilada, continuando así el proceso repetidas veces. Cuando la salmuera sale de la cá

AGENTE ALCALINO	PROCESO Y VELOCIDAD DE REACCION	EFICIENCIA	VENTAJAS	DESVENTAJAS
Caliza	Aumenta cuando se reduce el contenido en dolomía. Aumenta - cuando el tamaño del grano es menor	Depende del contenido de ión ferroso, en altas concent. de éste, es impracticable - (Fe>100 mg/l). Sistema relativamente poco utilizado	Lodos densos y generalmente sedimentables. Fácil manejo del material. Alimentación no requiere un control muy sensitivo	Reacciona lentamente requeriendo largos tiempos de retención. Usando grandes cantidades es más costosa que la cal
Cal	Neutralización con agitación durante 1 ó 2 minutos. Aireación posterior durante 15 a 30 min para oxidar Fe^{++} Sedimentación. Tratamiento de lodos	Muy alta pH = 3 → pH = 7 F total = 360 mg/l → 7 mg/l	Facilidad de operación	Requiere un depósito o presa de almacenamiento de aguas a tratar para trabajar en contínuo.
Caliza y Cal	La caliza se utiliza para neutralizar hasta pH 4. Tiempos de reacción: 20 - 30 - min. para la caliza y 10 - 15 min para la cal.	A pH mayores de 6 la caliza es poco eficaz, por lo que se utiliza la cal	Lodos más densos - que utilizando la cal	Producción de más - cantidad de lodos que utilizando sólo caliza

DESCRIPCION GENERAL DE LOS PROCESOS DE NEUTRALIZACION. TABLA 3

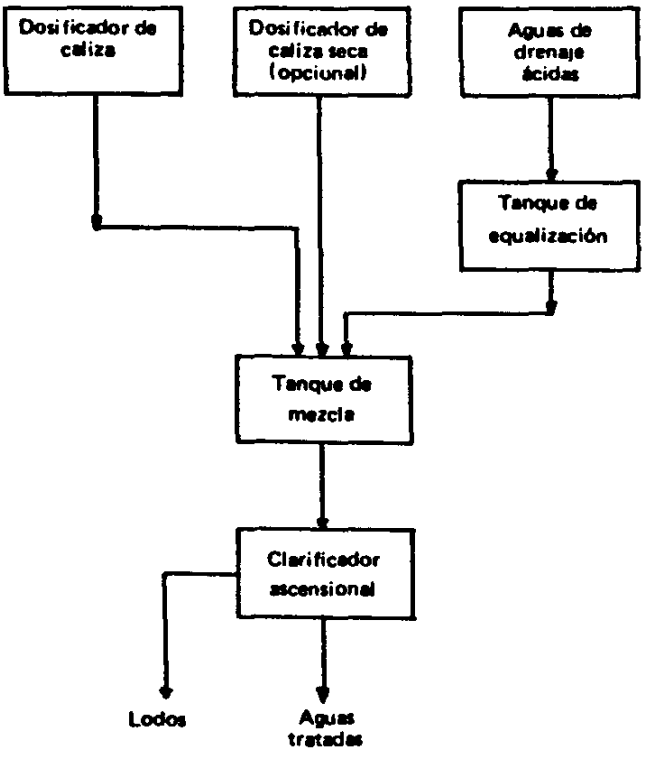


Fig. 3.— Neutralización con caliza pulverizada. Sistema F.W.Q.Q. "Norton Field Site" West Wirginia (U.S.A.)

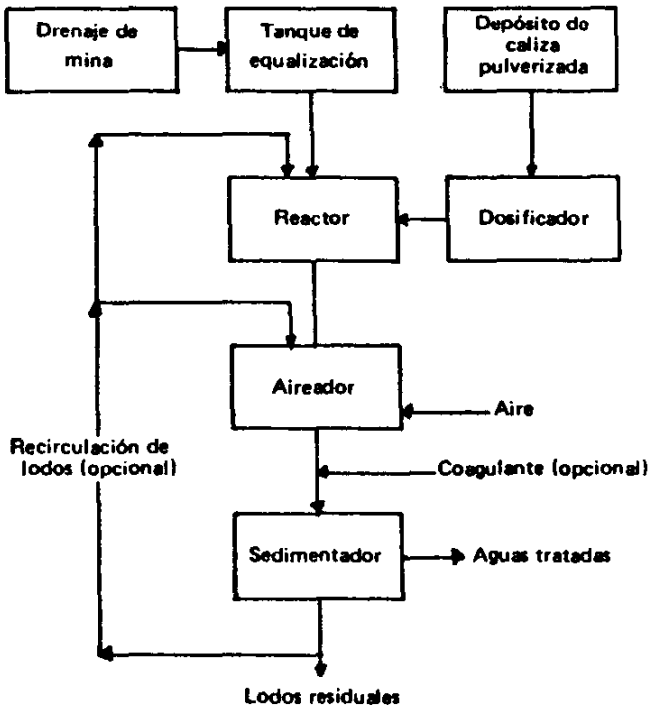
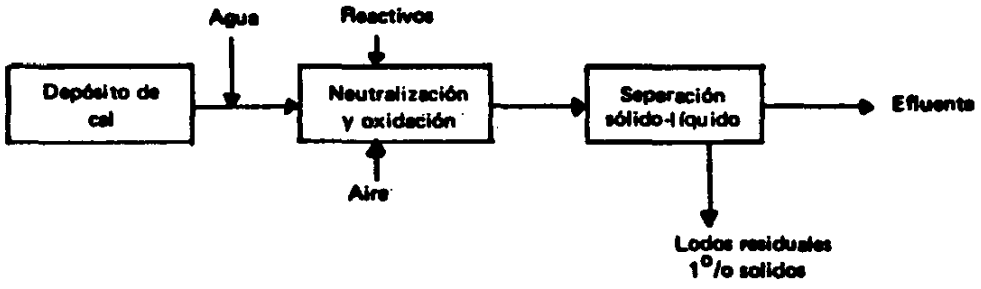


Fig. 4.— Neutralización con caliza pulverizada. Sistema B.C.R.

PROCESO CONVENCIONAL



PROCESO DE ALTA DENSIDAD DE LODOS

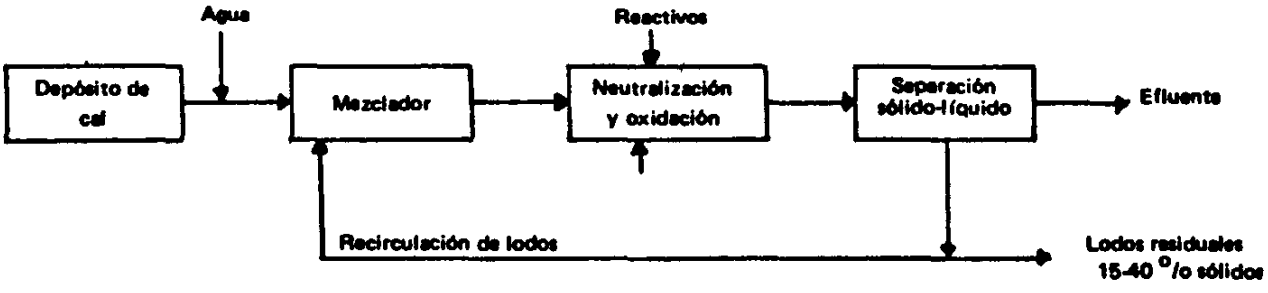


FIG. 5 - Comparación de los sistemas de neutralización con cal convencional y de lodos de alta densidad.

mara final, se utiliza para calentar el agua de alimentación. El equipamiento del proceso requiere la utilización de costosas aleaciones para evitar la corrosión. De otra parte el aumentar la concentración de sulfato de calcio aparecen frecuentemente problemas de depósitos de las tuberías. La salmuera requiere cuando se extrae del proceso un método adecuado de deposición en el terreno para evitar la contaminación de las aguas.

Un proyecto realizado por Westinghouse Electric Corporation concluyó que el proceso de (E.S.) puede ser aplicado a las aguas de drenaje para producir aguas potables en casos muy específicos. De acuerdo con los datos del citado estudio, los costes de operación y los costes de amortización (suponiendo una amortización del 6% en 20 años) supondría para el proceso con producción a vapor (energía) "in situ" un coste de 130,3 pts/m^3 (153,3 cents/m^3, coste a todas luces prohibitivo.

Las aguas producidas en este proceso tienen un contenido en sólidos disueltos menor de 50 mg/l. Esta calidad es mucho mayor que la usualmente requerida para fines domésticos o industriales.

5.2.3.2.- Otros procesos

Tanto el proceso anteriormente descrito como el resto de los procesos de desalinización hoy utilizados (osmosis inversa, electrodiálisis e intercambio iónico) resultan en la práctica económicamente inaceptables, para tratar grandes caudales de agua, a la vez que técnicamente ocasionan problemas graves.

Así, según los estudios piloto, en el proceso de osmosis inversa la formación de sulfato de calcio y la presencia de los iones ferroso y férrico reducen considerablemente la eficacia de la membrana (acetato de celulosa). La calidad de las aguas obtenidas por osmosis inversa no es aceptable para aguas potables debido a su pH, acidez y a los contenidos en hierro y manganeso, su potabilización requeriría un tratamiento posterior de neutralización química, coagulación, filtración y esterilización.

El proceso de electrodiálisis sólo ha sido estudiado a escala de laboratorio, observando que las membranas se obturan con facilidad y que una mejora de la eficacia del proceso requiere una coagulación y filtración previas.

Los procesos de intercambio iónico por medio de resinas intercambiadoras es un método muy conocido en la técnica del tratamiento de aguas para reducir la dureza, desmineralizar o eliminar un ión específico.

Las diferentes resinas intercambiadoras son utilizadas generalmente como sigue:

- Resinas catiónicas de ácidos fuertes:
  De sodio: para eliminar dureza (Ca^{++} y Mg^{++})
  de hidrógeno: para eliminar todos los cationes

- Resinas catiónicas de ácidos débiles:
  De hidrógeno: para eliminar cationes asociados con aniones alcalinos
  para eliminar dureza bicarbonatada

- Resinas aniónicas de base fuerte:
  Eliminación de todos los aniones

- Resinas aniónicas de base débil:
  Eliminar eficientemente ácidos como clorhídrico y sulfúrico.

En la tabla 4 figura el orden de selectividad de iones de los diferentes tipos de resinas citadas. La combinación de las resinas hoy disponibles en el mercado pueden ser empleadas en el diseño de un tratamiento para cada caso específico.

Los procesos actualmente más utilizados de intercambio iónico son:

a) El proceso Sul-bisul (Dow Chemical): Combinación de resina catiónica de ácido fuerte (de hidrógeno) o resina de ácido fuerte - ácido débil con resina aniónica de base fuerte. El proceso requiere además: decarbonatación, filtración, ajuste de pH, neutralización y clarificación.

b) El proceso Desal. Combinación de tres lechos, resina aniónica de ácido débil (de hidrógeno) -resina aniónica de base débil.Requiere además: decarbonatación, aireación, sedimentación, ablandamiento y filtración.

c) El proceso de Dos Resinas (Culligan International Company): resina catiónica de ácido fuerte (de hidrógeno)-resina aniónica de base débil (de hidróxido).

Los procesos arriba descritos tienen por objeto producir aguas potables a partir de aguas de drenaje o de río contaminados por estas aguas. Entre los proyectos de este tipo conocidos figuran: la planta Smith Towhship que trata agua de río contaminadas por el proceso Sul-bisul y la planta de Hawk Run para tratar aguas ácidas de mina por el proceso Desal. Ambas plantas se encuentran en Pennsylvania y producen aguas potables.

5.2.4.- Oxidación del hierro

Como ya se ha indicado, las aguas ácidas de mina contienen frecuentemente hierro ferroso, que es muy soluble. Para su precipitación como hidróxido indisoluble requiere pH muy básicos, es decir, entre 9, 3

y 12. La oxidación del hierro ferroso a férrico, es rápida a partir de pH 8, moderada entre pH 6-8, lenta entre pH 4-6 y muy lenta para pH menor de 4. El ion férrico es mucho menos soluble que el ferroso y comienza a precipitar a pH con una solubilidad mínima a pH 9,5.

La necesidad de alcanzar pH muy básicos para la precipitación del hierro ferroso, impone serias desventajas el proceso de neutralización, por ello se recurre a otros sistemas, clásicos también en el tratamiento del agua.

Tales sistemas son:

- Aireación
- Precipitación electroquímica
- Ozonización

TABLA 4 - SELECTIVIDAD DE ANIONES CON RESINAS INTERCAMBIABLES

TIPO DE RESINA	ORDEN DE SELECTIVIDAD
Cationes de ácidos fuertes (Tipo ácido sulfúrico)	$Ba^{++} > Ag^{+} > Pb^{++} > Hg^{++} > Sr^{++} > Ca^{++} > Cu^{+} > Ni^{++} > Cd^{++} > Zn^{++} > Fe^{++} > Mg^{++} > Mn^{++} > K^{+} > Na^{++} > H^{+}$
Catiónicas de ácidos débiles (tipo ácido carboxílico)	$H^{+} > Cu^{++} > Ca^{++} > Mg^{++} > K^{+} > Na^{+}$
Amónicas de base fuerte (Tipo I amonio cuaternario)	$I^{+} > HSO_4^{-} > NO_3^{-} > Br^{-} > Cn^{-} > HSO_3^{-} > NO_2^{-} > Cl^{-} > HCO_3^{-} > CH^{-} > F^{-}$
Aniónicas de base débil	$CH^{-} > SO_4^{=} > CrO_4^{=} > NO_3^{-} > PO_4^{3-} > I^{-} > Br^{-} > Cl^{-} > F^{-}$

La aireación se efectúa en tanques con una profundidad de 3 á 4,5 m y la transferencia de aire se realiza bien mediante difusores, o bien mediante agitadores mecánicos.

La acción del aire (oxígeno) puede ser resumida de forma teórica por la siguiente reacción:

$$4\ Fe\ SO_4 + O_2 + 10\ H_2O \rightarrow Fe(OH)_3 + 4\ H_2\ SO_4$$

El período de retención del agua en el tanque oscila entre 15 y 30 minutos. La eficiencia de la transferencia de oxígeno depende de diversos factores, valores frecuentes en los diseños son una eficiencia del 10% para aireación con difusores y un consumo de 1,5 km de O_2 para Kwh. Por este sistema 1 kg de Fe^{++}, requiere 5,5 m^3 de aire para su oxidación. En aireación mecánica un caudal de 1 kg Fe^{++}/h - requiere 0,11 kw para su oxidación.

El método de precipitación electroquímica ha sido puesto en funcionamiento por Tico Laboratories Inc. Los estudios realizados en células con ánodo de caolón y cátodo de acero inoxidable con aguas de drenaje ácidas (sintéticas) a pH 2,7 han demostrado que las concentraciones de ion ferroso pasan de 2.10^{-2} M a 5.10^{-4} M en una solución 0,2 M de ácido sulfúrico.

La utilización de ozono en combinación con el tratamiento posterior de neutralización ha sido estudiado por el Brookhaven National. Laboratory de la Comisión de Energía Atómica de USA. La producción de ozono por descarga eléctrica tiene un grave inconveniente, su alto coste.

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L'EPURATION DES EAUX ACIDES D'EXHAURE DE LA MINE PICCALINA (SARDAIGNE) ET LA RECUPERATION DES METAUX SEPARES

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ABSTRACT : After a brief review of the most common methods of treatment in use today to resolve the problem of water pollution posed by acid pumping waters of certain metallic mines, the paper examines, with experimental methods, the possibility of purifying acid pumping waters from the Piccalinna mine (Montevecchio) having an exceptionally high heavy metal ions content. The technical possibility of purification is demonstrated by applying the well-known processes of precipitation and sedimentation of hydroxides, and adopting the following alternative different flowsheets :
- single stage treatment, with the sole object of complying with the severe limitations imposed by the provisions of Italian law regarding water pollution ;
-multistage treatment with the additional aim of recovering one or more of the commercial products precipitaded.

RESUME : Après une brève revue des méthodes les plus communes de traitement actuellement utilisées pour résoudre le problème de pollution des eaux posé par les eaux acides d'exhaure de certaines mines métalliques, le mémoire examine, avec méthode expérimentale, la possibilité d'épuration des eaux acides d'exhaure de la mine Piccalinna (Montevecchio) caractérisées par un titre exceptionnellement élevé d'ions de métaux lourds. On démontre la possibilité technique de l'épuration, en appliquant les procédés de précipitation et de sédimentation d'hydroxides, suivant différents diagrammes alternatifs :
- par traitement monostade, dans le seul but de satisfaire les limites sévères imposées par les lois italiennes en vigueur en matière de pollution des eaux ;
- par traitement multistade dans le but supplémentaire de récupérer sous forme marchande un ou plusieurs produits métallifères.

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RESUMEN : Después de pasar revista, rápidamente, a los métodos más comunes de tratamiento, actualmente utilizados, para resolver el problema de polución, provocado por las aguas ácidas de drenaje de ciertas minas metálicas, se examina, con método experimental, la posibilidad de depuración de las aguas ácidas del drenaje de la mina Piccalinna (Montevecchio), caracterizadas por un contenido iónico excepcionalmente elevado de metales pesados. Se demuestra la posibilidad técnica de depuración, mediante aplicación de los procedimientos de precipitación y sedimentación de hidróxidos, según diferentes diagramas alternativos :
- por tratamiento monoestado, con el fin de satisfacer los severos límites impuestos por las leyes italianas en vigor, en materia de polución de agua ;
- por tratamiento multiestado, con el fin suplementario de recuperar, en forma comercial, uno o varios productos metálicos.

1. INTRODUCTION

Jusqu'il y a quelque temps le seul problème apparent de pollution des eaux à la attention de l'industrie minière était la séparation des solides en suspension dans les eaux d'exhaure et dans les effluents des installations d'enrichissement.

Depuis quelques années toutefois, l'attitude va se modifiant en conséquence des plus amples connaissances acquises à l'égard des effets que la présence des différents éléments en traces exerce sur l'équilibre écologique du système des eaux.

En même temps on a adapté et renforcé la législation qui concerne la prévention de la pollution des eaux.

Les eaux d'exhaure des mines métalliques, et de celles à sulfures en particulier, ainsi que les eaux de rejet des installations d'enrichissement constituent certainement les sources principales de pollution des eaux dans l'industrie minière.

Se référant au premier type d'eaux, les problèmes de pollution y associés sont essentiellement à attribuer à la formation d'acide. Dès que l'eau entre en contact avec des sulfures, de petites quantités de soufre se dissousent et ceci donne lieu à la formation de thiocomposés qui peuvent s'oxyder produisant de l'acide sulfurique.

La lixiviation lente des minéraux de la part des eaux acides qui se forment de cette façon détermine ensuite une augmentation progressive de la concentration d'ions de métaux lourds dans ces eaux.

Par conséquent, si l'on considère les limites sévères imposées par la législation italienne et par la loi régionale de la Sardaigne en matière de décharge des eaux industrielles (loi du 10 mai 1976; decret régional de la Sardaigne, 14 décembre 1976), on peut bien comprendre la nécessité d'un traitement préliminaire des eaux de ce type avant de les amener dans les systèmes extérieurs, dans le but d'en neutraliser l'acidité et d'en réduire la concentration en éléments nuisibles au dessous des limites prescrites.

2. EPURATION DES EAUX D'EXHAURE DANS LES BASSINS DE DECANTATION DES STERILES

L'épuration des eaux acides d'exhaure est basée généralement sur le rapport entre la concentration des ions métalliques et le pH de l'eau (FAIR, 1968).

La méthode de traitement la plus commune consiste à envoyer les eaux dans le bassin des stériles où le pH est réglé à un niveau capable de neutraliser l'acidité et de précipiter les métaux sous forme d'hydroxydes insolubles (BELL, 1974).

Le bassin permet aussi la sédimentation et l'oxydation des eaux. La méthode est simple seulement en apparence car souvent un seul bassin doit accomplir simultanément plusieurs tâches:
- enlever les solides stériles par sédimentation;
- neutraliser l'acidité;
- précipiter les métaux sous forme de composés insolubles;

- sédimenter ces précipités;
- retenir indéfiniment les stériles et les précipités sédimentés;
- stabiliser les éléments oxydables;
- régulariser les fluctuations de qualité et de quantité de l'effluent, emmagasinant en plus l'eau de pluie et régularisant son flux.

L'utilisation presque universelle des bassins de stériles comme méthode principale de traitement des eaux polluées dans l'industrie minière dérive sans doute de la capacité effective de ces bassins d'accomplir plusieurs des tâches mentionnées et en plus du fait qu'ils représentent un mode de traitement relativement peu couteux. Les phases du traitement les plus critiques sont sans doute la sédimentation, l'oxydation et le stockage des solides sédimentés. Dans beaucoup de cas, la disponibilité d'une zone étendue renfermée permet de réaliser efficacement ces trois procédés à condition que le bassin soit bien projeté et géré. Dans de autres cas, toutefois, cela est empêché pour deux raisons fondamentales : en premier lieu à cause des dimensions et des caractéristiques naturelles du système défavorables à un contrôle soigné de chacun de ces procédés, et en second lieu, en raison des exigences contrastantes dont on doit tenir compte en vue de leur optimisation.

D'autre part, les mines qui utilisent des bassins de stériles bien contrôlés obtiennent facilement des effluents qui respectent largement les limites imposées par la législation en vigueur (BELL, 1974, BUSS, 1974, OKO, 1974).

3. EPURATION DES EAUX D'EXHAURE DANS LES UNITES MECANISEES

Dans le cas où la structure des activités minières est articulée de sorte qu'il n'existe pas la disponibilité de bassins de sédimentation des rejets provenants de l'installation d'enrichissement, la solution alternative peut consister dans l'utilisation d'unités mécanisées où on réalise les différentes phases de mélange des réactifs alcalins, de contrôle de pH, floculation, clarification et stockage des boues. En cas de nécessité l'installation peut comprendre un stade de épuration ultérieure de l'effluent dans le but d'obtenir des standards d'épuration particulièrement poussés.

L'utilisation de ces unités mécanisées de traitement des eaux d'exhaure représente aussi une solution alternative dans le cas où, tout en disposant d'un bassin de stériles bien adapté, on croit qu'il soit opportun de traiter l'ensemble des eaux d'exhaure et de celles de drainage superficiel séparément des eaux de rejet de l'installation d'enrichissement. Dans le cas, par exemple, où ces dernières soient recyclées, après ou non leur épuration, la solution citée représente aussi la seule possible.

D'autre part, puisque le recyclage des eaux de rejet des installations de valorisation représente sans doute pour l'industrie minéralurgique l'objectif plus avancé dans la lutte contre la pollution des eaux, et étant donné le nombre croissant des installations qui tendent à le réaliser, il est évident que dans l'avenir prochain l'utilisation d'installations mécanisées pour traiter les eaux de exhaure deviendra plus répandue.

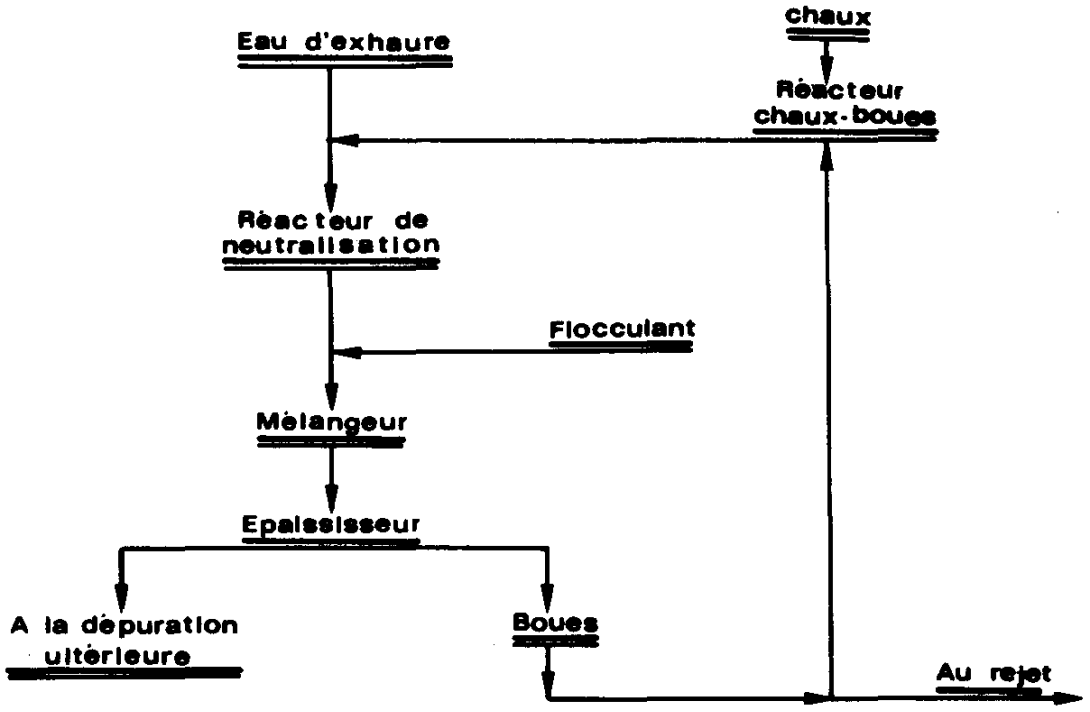

Fig.1 - Schéma d'une unité mécanisée pour le traitement des eaux d'exhaure

On trouvera à la Fig.1 le schéma classique d'une unité mécanisée de traitement des eaux d'exhaure et de celles provenant du drainage de surface, tel que celui réalisé actuellement dans de nombreuses mines canadiennes (BELL, 1975). Il consiste d'un stade de préparation des réactifs, d'un stade de floculation, de clarification des eaux, de recyclage des boues quand nécessaire, d'épuration ultérieure de l'effluent et de séchage des boues.

On utilise cette dernière phase d'épuration supplémentaire lorsqu'il faut obtenir des taux d'épuration très poussés. En principe elle est constituée d'un procédé de sédimentation supplémentaire dans un bassin artificiel, bienqu'ils aient été expérimentés, mais avec un succès discutable, des procédés de filtration en lit sableux ou de cémentation sur oxyde de magnésium (BELL, 1975).

En ce qui concerne l'application industrielle, ces installations ont donné de bons résultats et il en résulte que, entre autres, quant à la qualité de l'effluent, le dégré de pollution de l'effluent n'a pas d'influence, tandis que de

ce point de vue, l'utilisation de floculants et le recyclage partiel de la boue sont d'une importance fondamentale.

4. LES EAUX D'EXHAURE DE LA MINE PICCALINNA

La mine Piccalinna (Montevecchio) est une mine relativement humide, puisque moyennement le débit d'exhaure des eaux souterraines est de l'ordre de 3000 m^3/j.

Le tableau 1 montre la concentration en mg/l des ions contenus dans un échantillon d'eau d'exhaure prélevé pour l'exécution de l'étude expérimentale, mais qui reproduit substantiellement la composition moyenne de ces eaux.

Tableau 1 - Composition ionique des eaux d'exhaure de la mine Piccalinna

Ions	Concentration (mg/l)	Ions	Concentration (mg/l)
Cu^{2+}	35	Ca^{2+}	206
$Fe^{2+} + Fe^{3+}$	235	Al^{3+}	97
Mn^{2+}	186	SO_4^{2-}	6170
Zn^{2+}	2426	Cl^-	82
Mg^{2+}	186	pH	2,64

Soit l'acidité soit le titre d'ions nuisibles très élevés, obligent, en concomitance avec la pleine application de la loi italienne pour la prévention de la pollution prévue d'ici le 13/6/1979, de traiter ces eaux avant leur décharge.

Nous avons donc effectué une étude expérimentale ayant pour but principal de réduire l'acidité et la composition ionique au dessous des limites prévues par la législation en vigueur. La technique de traitement adoptée était celle à chaux. De plus, dans le but de réduire la charge économique du traitement, aussi par rapport à la concentration exceptionnellement élevée en zinc de ces eaux, on a étudié la possibilité d'application d'un procédé de précipitation en deux stades en mesure de fournir au moins un produit marchand à haute teneur en zinc. La méthode, déjà appliqué à l'échelle industrielle dans la République Fédérale Allemande, aux eaux d'exhaure de la mine de Meggen, permet en principe de réduire aussi le prix élevé de stockage des boues précipitées(BERGMANN, 1971).

5. PRECIPITATION PAR CHAUX DES HYDROXYDES METALLIQUES

Le relèvement des courbes de précipitation des hydroxydes des différents métaux contenus dans l'eau d'exhaure de la mine Piccalinna a été effectué ajoutant à des échantillons de 500 ml d'eau quantités croissantes de solution N/10 de $Ca(OH)_2$. Dans chaque essai on a puis pourvu à la filtration des hydroxydes précipités, séchant à 110°C, pesant ensuite ces précipités, et déterminant le pH de l'eau avec un pH-mètre avant et après précipitation. Pour l'eau telle quelle et pour l'eau filtrée de chaque essai, on a déterminé, par le moyen d'analyses au spectrophoto-

mètre, la concentration des ions Cu^{2+}, Zn^{2+}, Fe^{2+} + Fe^{3+}, Mn^{2+}, Al^{3+} et par analyse chimique directe la concentration de SO_4^{2-}.

Les courbes de précipitation des différents hydroxydes métalliques en fonction du pH sont indiquées dans la Fig.2. Ces courbes montrent que la précipitation est presque complète dans le champ de pH entre 8 et 9. Par contre, la concentration de l'ion sulphate se révèle seulement faiblement réduite.

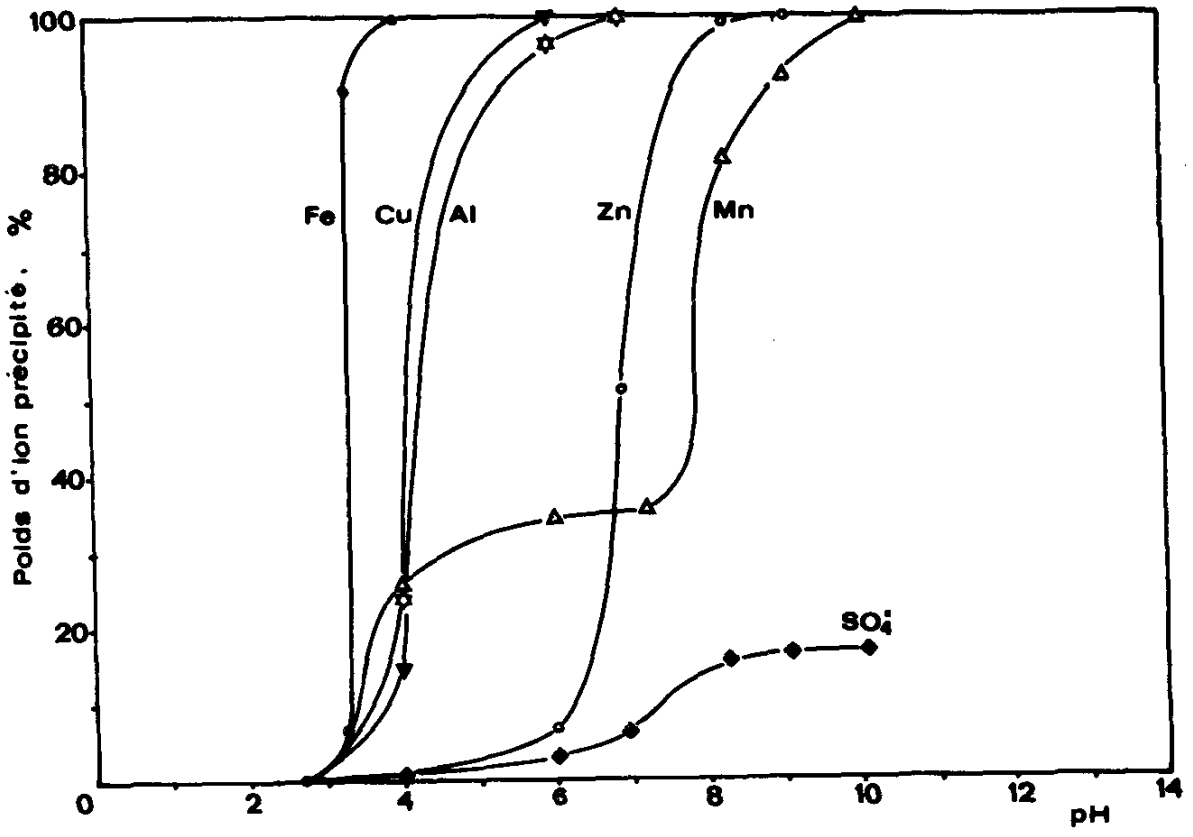

Fig.2 - Courbes de précipitation des ions contenus dans l'eau d'exhaure de la mine Piccalinna. Modificateur du pH : $Ca(OH)_2$

L'application de la technique de précipitation utilisant la chaux, aux eaux de la mine Piccalinna, permet donc, dans le cas où les eaux résultantes sont déchargées dans la mer, de respecter les limites imposées par la législation actuelle. Au contraire, ce n'est pas le cas pour les eaux à décharger dans le système intérieur (fleuve ou lac), leur concentration résiduelle d'ion SO_4^{2-} étant supérieure à la limite maximum admissible en ce cas (1000 mg/l).

6. EPURATION AVEC $BaCl_2$ ET $Ca(OH)_2$

La réduction de la concentration de l'ion SO_4^{2-} au dessous des limites requises rend nécessaire le recours à un réagent spécifique en mesure de précipiter l'excès. Les sels solubles de barium sont certainement des réagents indiqués pour ce but. Pour le chlorure de barium, par exemple, la quantité stoechiométriquement nécessaire pour précipiter 1 g de SO_4^{2-} est de 2,168 g.

Le procédé a été essayé sur l'eau d'exhaure de la mine Piccalinna observant la méthodologie suivante. A 1 litre d'eau d'exhaure telle quelle on a ajouté 13,3 g de $BaCl_2$ et 3,7 g de $Ca(OH)_2$ (1)

Après brève agitation le précipité, filtré et seché à 110°C, a été pesé et analysé, tandis que pour l'eau filtrée on a déterminé par voie spectrophotomètrique la concentration résiduelle des métaux principaux, et par voie chimique directe la concentration résiduelle d'ion SO_4^{2-}.

Le résidu pesant 20,108 g, titrait 64,62% $BaSO_4$, 1,17% Fe, 0,92% Mn, 11,97% Zn, 0,018% Cu, tandis que dans l'eau filtrée on a relevé seulement de traces des différents ions.

Il ressort donc que le processus se révèle efficace pour l'épuration des eaux de la mine Piccalinna, avec l'élimination soit des métaux soit du sulphate, quoique on doive avancer des réserves en ce qui concerne le prix de revient élevé des réagents nécessaires et le contenu élevé de chlore dans les eaux épurées en raison de l'emploi du chlorure de barium.

Cependant, on souhait que pour ces eaux, en dérogation des lois en vigueur, soient tolérées des concentrations résiduelles d'ions SO_4^{2-} et/ou Cl^- supérieures aux limites actuellement prévues, en accord, du reste, avec la pratique suivie dans quelques pays hautement industrialisés.

Pour les eaux d'exhaure des mines canadiennes de New Brunswick, par exemple, dont la concentration de SO_4^{2-} peut excéder les 10.000 mg/l, il n'existe aucune limite en ce qui concerne la concentration de ce composant dans les eaux qui proviennent de l'épuration (BELL, 1975).

7. EPURATION AVEC RECUPERATION DE METAUX

Etant donné le prix de revient élevé de l'épuration des eaux très polluées telles que celles d'exhaure de la mine Piccalinna, il serait opportun de vérifier, comme d'autre part c'est la pratique dans la mine de Meggen déjà citée, la possibilité de réduire la charge économique du traitement. Pour atteindre ce but on a adopté un procédé de précipitation multistade, qui est en mesure de fournir au moins un produit marchand ayant une teneur en zinc relativement élevée.

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(1) *Quantités calculées d'après la composition ionique de l'eau d'exhaure.*

Les diagrammes de Fig.2 montrent que la précipitation des métaux tels que Fe, Cu et Al, est presque complète à un pH autour de 6, tandis que plus que 90% du zinc est encore contenu en solution.

Cependant, la réalisation d'un schéma d'épuration en deux stades, le premier à un pH compris entre 5 et 6 et le deuxième à un pH autour de 8, peut permettre de obtenir une boue, précipitée pendant le deuxième stade, ayant une teneur en zinc plus élevée. Une amélioration ultérieure de la qualité de ce produit est possible précipitant la totalité du manganèse au cours du premier stade. L'effet est, en principe, possible au moyen d'une oxydation de la solution, par example, employant hypochlorite de sodium, lequel, comme il est bien connu, est un agent oxydant à température ordinaire, soit en solution alcaline, soit en solution acide. Plusieurs hydroxydes métalliques sont transformés, par oxydation, dans d'autres hydroxydes dont le métal se trouve dans un état d'oxydation plus élevé.

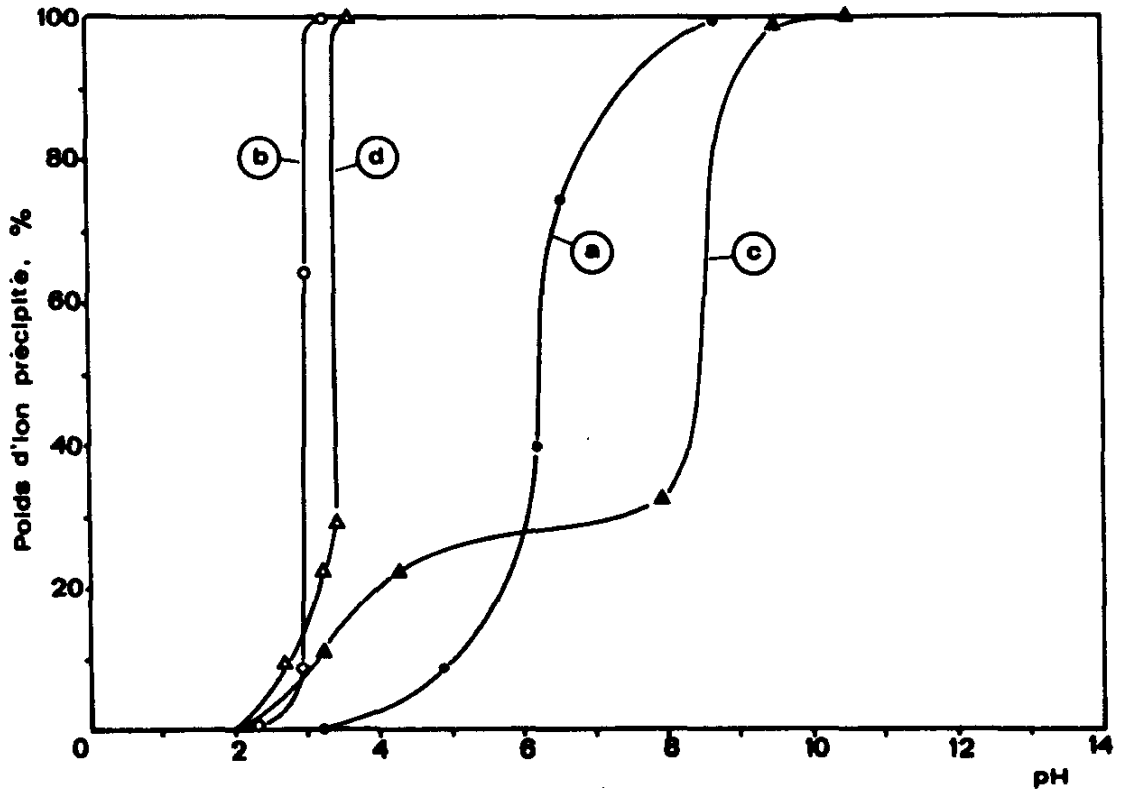

Fig.3 - Influence de l'oxydation sur la précipitation du fer et du manganèse
a) précipitation du fer d'une solution de $FeSO_4$ avec NaOH;
b) précipitation du fer d'une solution de $FeSO_4$ avec NaOCl;
c) précipitation du manganèse d'une solution de $MnSO_4$ avec NaOH;
d) précipitation du manganèse d'une solution de $MnSO_4$ avec NaOCl.

Par exemple, pour le fer ferreux $2Fe(OH)_2 + OCl^- = Cl^- + 2Fe(OH)_3$ et pour le manganèse $Mn(OH)_2 + OCl^- = Cl^- + MnO(OH)_2$.

Il est de conséquent que le fer ferreux et le manganèse peuvent précipiter sous la forme d'hydroxydes dans un champ de pH plus bas. Les résultats de ces expériences de laboratoire sont indiqués dans la Fig.3.

D'après ces considérations théoriques et contrôles expérimentaux on a effectué la précipitation en deux stades des impuretés contenues dans les eaux d'exhaure de la mine Piccalinna, suivant la méthodologie schématisée en Fig.4.

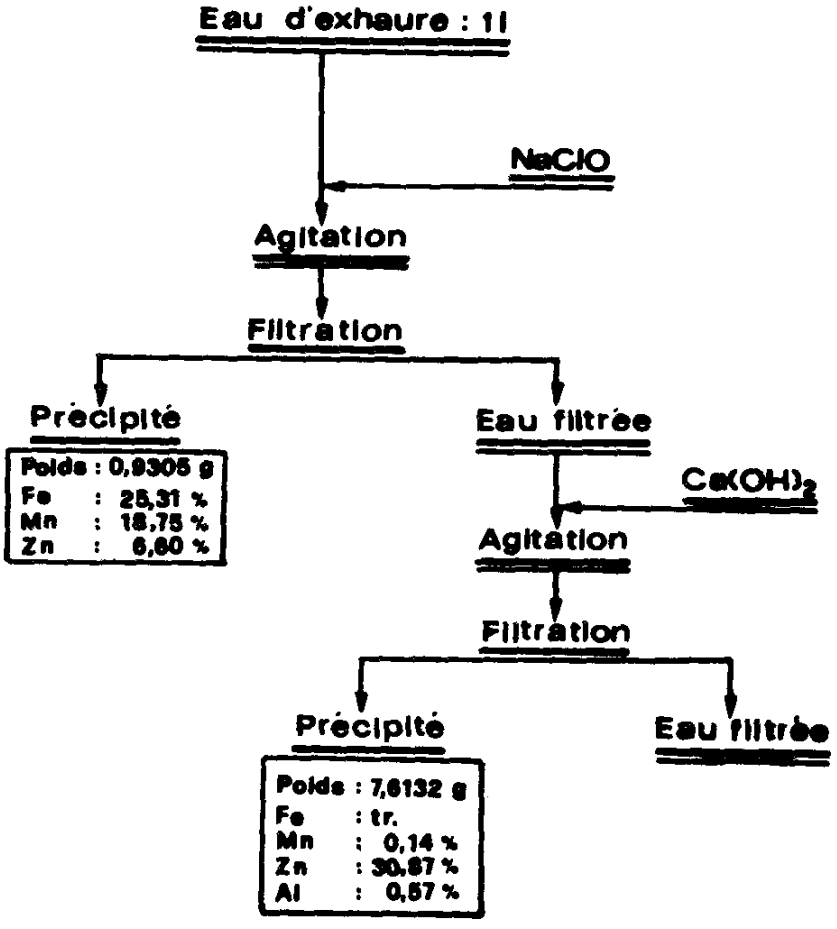

Fig.4

A 1 litre d'eau d'exhaure telle quelle on a ajouté 10 ml d'une solution 6% de chlore de hypochlorite de sodium, filtrant, après brève agitation, le précipité. A l'eau filtrée on a ajouté encore 3,7 g de chaux, filtrant de nouveau. Les précipités filtrés ont été séchés à 110°C, pesés et analysés. Les résultats obtenus sont montrés dans la même Fig.4.

Comme l'on constate, le procédé permet l'épuration de l'eau, obtenant dans le deuxième stade de précipitation une amélioration relative des caractéristiques qualitatives de la boue précipitée. La précipitation d'une certaine proportion d'ion sulphate sous forme de sulphate de calcium, au cours du deuxième stade susdit, limite la teneur en zinc de cette boue. Par conséquent, l'élimination préalable de l'ion sulphate, par exemple au moyen d'une précipitation avec $BaCl_2$, peut obvier à l'inconvénient.

La vérification expérimentale relative a été conduite suivant le procédé schématisé dans la Fig.5.

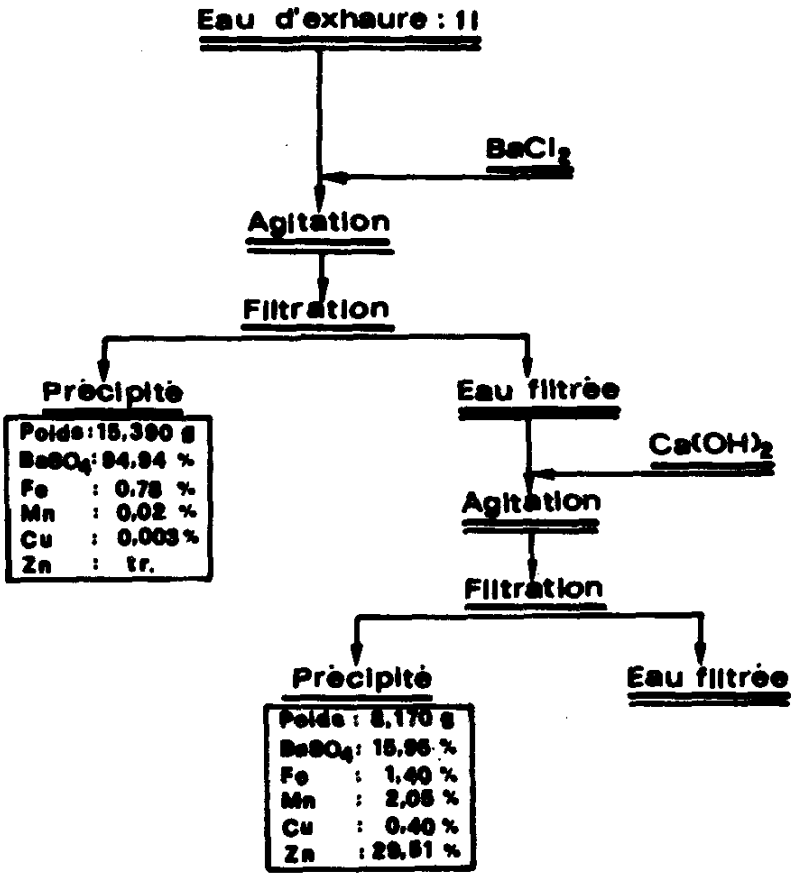

Fig.5

A 1 litre d'eau telle quelle on a ajouté 13,3 g de $BaCl_2$, filtrant après brève agitation. A l'eau filtrée on a ajouté encore 3,7 g de $Ca(OH)_2$. Les précipités, filtrés, ont été séchés à 110°C, pesés et analysés.

Les résultats, indiqués dans la même Fig.5, soulignent la validité du procédé et l'intérêt économique de l'utilisation d'un diagramme à trois stades, à savoir, précipitation des métaux dans le premier, des sulphates dans le deuxième et du zinc dans le troisième.

On trouverà à la Fig. 6 un schéma du procédé susdit et les résultats expérimentaux obtenus.

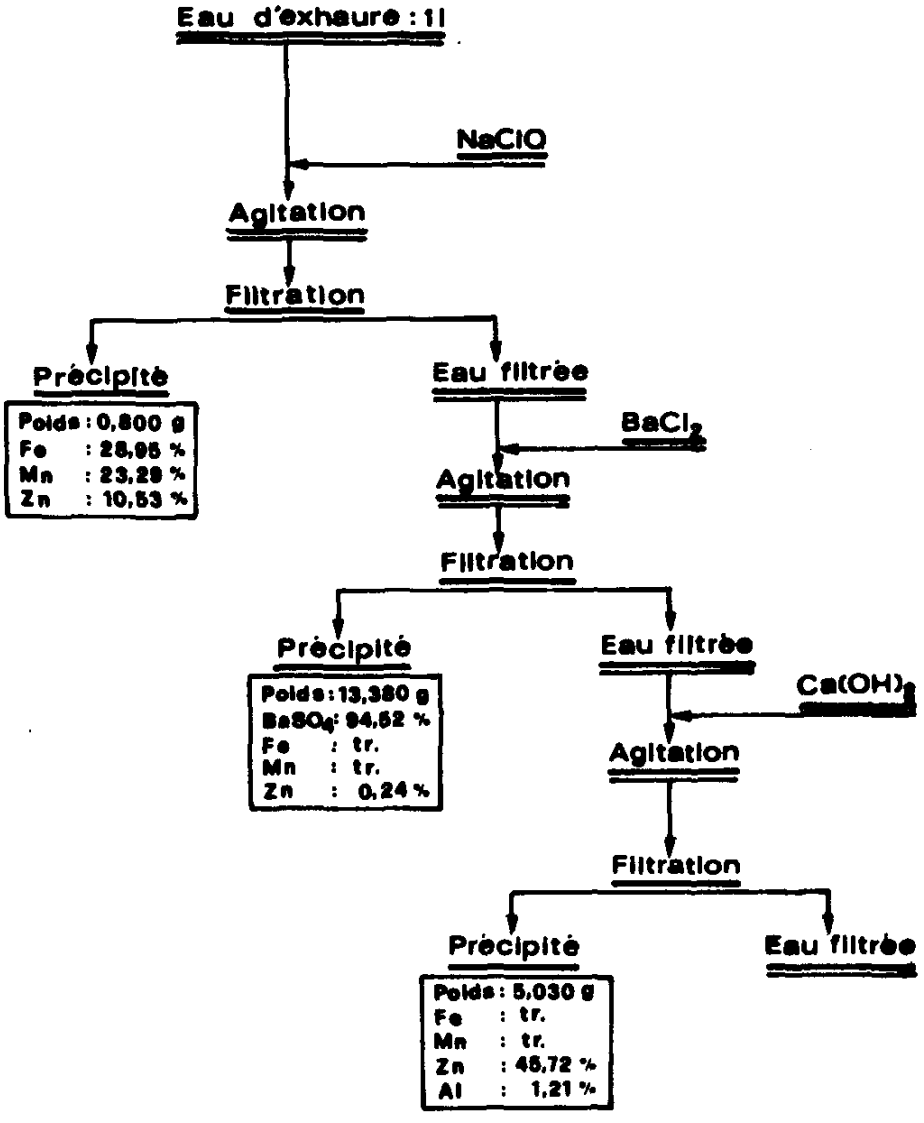

Fig.6

A 1 litre d'eau telle quelle on a ajouté successivement 10 ml d'hypochlorite de sodium, 13,3 g de $BaCl_2$ et 3,7 g de $Ca(OH)_2$, agitant et filtrant après chaque addition. Les précipités, filtrés, ont été séchés à 110°C, pesés et analysés.

Les deux résidus provenant de la filtration du deuxième et du troisième stade, constituent des produits marchands, lesquels pourtant peuvent valablement contribuer à la limitation des charges économiques de l'épuration.

Evidemment le procédé peut, en alternatif, être simplifié, conglobant le premier et le deuxième stade dans une seule phase de précipitation.

8. CONCLUSIONS

On a démontré expérimentalement la possibilité d'épuration des eaux acides d'exhaure de la mine Piccalinna tout en respectant les limites imposées par les termes de la loi en matière de pollution des eaux.

On peut atteindre le but utilisant différentes méthodes et diagrammes alternatifs, à savoir:
- traitement monostade dans le bassin de décantation des stériles;
- traitement monostade en unité mécanisée spécialement préparée;
- traitement multistade en unité mécanisée avec récupération d'un ou plusieurs produits marchands.

Du point de vue économique le choix optimal doit, naturellement, être évalué comparant la valeur des produits obtenus dans un diagramme multistade avec le prix de revient supplémentaire d'investissement, de réagents et de main d'oeuvre lié à cette dernière solution.

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TENEUR EN SULFATES DES EAUX STOCKEES DANS LES MINES DE FER DE LORRAINE (FRANCE)

Hervé, D.

ABSTRACT : Iron mining waters of Lorraine (France), especially in former drowned mines, show an amount of calcium sulphate, which is awkward as far as their use is concerned. We are looking for the origin of this mineralization, which may reach 800 mg/l in SO_4, by observing it "in situ", by analysing its isotopes, and by systematically leaching the rocks. The results show that the sulphates come from the wheathering of pyrite, which is spread in the marly top of the ore bed. The wheathering starts again whenever the level of the water changes. Accumulating information is necessary for a mathematical modeling of chemical exchanges between water and rock.

RESUME : Les eaux d'exhaure des mines de fer de Lorraine (France), présentent notamment dans les anciennes mines noyées, des teneurs en sulfate de calcium qui sont gênantes lors de leur utilisation. La recherche de l'origine de cette minéralisation, qui peut atteindre 800 mg/l en SO_4, est menée par l'observation "in situ", par analyses isotopiques, et par lixiviation systématique des roches. Les résultats montrent que les sulfates proviennent de l'altération de pyrite diffuse dans les toits marneux des couches de minerai. Il y a reprise de l'altération à chaque variation du niveau d'eau. L'accumulation des renseignements permettra la réalisation d'un modèle mathématique des échanges chimiques roche-eau.

RESUMEN : Las aguas de desagüe de las minas de hierro de Lorena (Francia) presentan especialmente en las antiguas minas inundadas, tenores en sulfato de calcio, que crean problemas en su utilización. La investigación del origen de esa mineralización, que puede alcanzar 800 mgr/l en SO_4, se realizó por medio de observaciones "in situ", análisis isotópicos y por lixiviación sistemática de las rocas. Los resultados muestran que los sulfatos provienen de la alteración de la pirita diseminada en los techos margosos de las capas de mineral. Existe una reanudación de la alteración a cada variación del nivel de agua. La acumulación de informaciones permitirá realizar un modelo matemático de intercambio químico roca-agua.

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1. Introduction

Les couches ferrifères exploitées en Lorraine font partie du Lias supérieur. Ce sont des marno-calcaires, caractéristiques d'une mer épicontinentale.

Les mines situées sous le réservoir aquifère des calcaires du Dogger ont une exhaure considérable (125 à 250 Mm^3/an pour 50 Mt de minerai extrait). Une partie importante de ces eaux est utilisée par l'industrie et pour l'alimentation en eau potable (35 Mm^3). L'irrégularité saisonnière des débits d'exhaure est telle qu'un stockage des eaux est nécessaire pour satisfaire les besoins des utilisateurs. Mettant à profit la fermeture de certaines mines, plusieurs réservoirs souterrains de grande dimension (de 0,8 Mm^3 à 6 Mm^3) ont été constitués par ennoyage des exploitations abandonnées. Cependant, il a été constaté, lors du remplissage de ces réservoirs, une dégradation importante de la qualité de l'eau, essentiellement par élévation de la dureté de l'eau (jusqu'à 100° F) et de la teneur en sulfates (jusqu'à 800 mg/l en SO_4). Ces eaux ne sont pas utilisables par l'industrie et il n'est pas souhaitable de les distribuer, bien que les sulfates ne présentent pas de danger pour la santé des utilisateurs aux teneurs observées.

L'Agence Financière de Bassin Rhin-Meuse, en liaison avec le Ministère de l'Environnement, a confié l'étude de ce phénomène au Laboratoire d'Hydrogéologie et d'Hydraulique de l'E.N.S.Géologie de Nancy. De nombreux renseignements avaient été acquis antérieurement par l'Agence de Bassin. Son service technique a, en outre, participé activement à l'interprétation des observations.

Ces travaux, non encore achevés, n'ont pu être réalisés qu'avec la collaboration des mines concernées et de l'Institut de Recherche de la Sidérurgie.

2. But de l'étude

L'étude a évidemment pour but de rechercher le moyen d'améliorer la qualité des eaux stockées dans les mines de fer. Deux approches différentes ont été menées simultanément :

a) synthèse d'observations réalisées dans certaines mines, à l'occasion de remplissages ou de vidanges de réservoirs souterrains, notamment à la mine d'Angevillers ;

b) étude expérim[illegible]e dans les laboratoires de l'E.N.S.Géologie axée sur le cas du [illegible]ervoir (Valleroy), comprenant :

- recherche de [illegible]e du soufre par les rapports isotopiques $^{34}S/_{32}S$
- lixiviations systématiques des différentes roches noyées dans le réservoir

3. Observations en mine

Les eaux d'exhaure des mines contiennent en général de 150 à 250 mg/l de sulfates. On peut stocker ces eaux soit dans des galeries, creusées à cet effet ou dont l'exploitation est terminée (albraques), soit dans des secteurs dépilés, les eaux baignant alors les roches effondrées du toit des couches.

Lors du stockage de l'eau dans des albraques, le phénomène d'augmentation de la salinité n'a jamais été constaté. L'eau y a toujours un temps de séjour faible, mais des mesures ont été réalisées sur des eaux ayant séjourné pendant des années dans des puits ou des galeries noyées, situées en aval pendage : la quantité de sulfates ne s'est pas révélée importante.

Emplacement	*Site*	*Durée de séjour*	*TH (°F)*	*SO_4 (mg/ℓ)*	*PH*
Quartier 02		1 année	14	190	7,6
Vers le déversoir n° 4	dans un puits	secteur abandonné	31	239	7,6
	en galerie noyée	depuis 20 ans	34	251	7,9
			31	239	7,6

Tableau 1 : analyses d'eau dans la mine du Paradis

A l'inverse, des teneurs importantes ont été constatées dès le premier ennoyage de zones dépilées. Le tableau 2 illustre ces constatations.

Réservoirs	*Volumes (Mm^3)*	*Dépilages*	*SO_4 (mg/ℓ)*
Hayange Nord	2,2	oui	815
Angevillers	0,1	oui	700
Valleroy	6,0	oui	540
Hayange Sud	0,8	oui	150
Sainte-Marie	1,6	oui	340
Roncourt	-	non	230
Nonkeil	-	non	120

Tableau 2 : salinité des réservoirs

L'évolution du phénomène dans le temps est observée par la Société SACILOR à Hayange-Nord et par l'Agence de Bassin, en liaison avec les mines concernées, à Sainte-Marie, Valleroy et Angevillers. Une amélioration sensible de la qualité de l'eau a été observée.

Réservoirs	SO_4 (mg/ℓ)	
	1975	1977
Hayange Nord	815	420
Valleroy	540	375
Sainte-Marie	340	195

Tableau 3 : évolution de la teneur en sulfates

Hayange Sud :

On peut expliquer la faible teneur en sulfates du réservoir d'Hayange-Sud, pourtant situé dans des zones dépilées par le fait qu'il a été noyé il y a une trentaine d'années, au cours desquelles le niveau a été maintenu constant par des apports d'eau relativement peu sulfatée des mines d'Hayange et de La Paix : les sulfates ont pu être éliminés depuis la mise en eau de ce réservoir.

Hayange Nord :

Le réservoir d'Hayange-Nord est maintenant exploité à niveau constant, mais une circulation d'eau est assurée par des apports d'eau de la mine de La Paix (150 mg/l de sulfates) en exploitation, et cela se traduit par une amélioration de la qualité de l'eau. Il semble donc qu'en l'absence de fluctuation du niveau d'eau, l'augmentation de la salinité ne se produit pas.

Angevillers :

Un grand nombre de mesures ont été réalisées sur le réservoir d'Angevillers : on s'était proposé de suivre l'évolution de la minéralisation de l'eau de ce réservoir lors de remplissages et vidanges successifs ; mais le percement, par le fond de ce réservoir, lors de travaux dans une mine voisine, a mis fin à ces mesures après la première vidange.

Toutefois, lors de cette vidange, il a été possible d'établir une corrélation linéaire significative entre le temps de séjour et la teneur en sulfates de l'eau dans le réservoir (RAMON S., 1976).

4. Etudes en laboratoire

Parallèlement à l'observation de l'extension et de l'évolution dans le temps de la salinité des réservoirs dans les mines de fer, s'est déroulé un programme d'étude dans les laboratoires de l'E.N.S.Géologie, qui avait pour buts de mieux comprendre ce phénomène et d'essayer de trouver une solution.

4.1 - Isotopes du soufre

La pyrite, susceptible de s'altérer en oxydes de fer hydratés et sulfates, peut se trouver à l'origine de la minéralisation sulfatée rencontrée dans les eaux de mines. Cette pyrite est notée à certains niveaux des couches de minerai, en particulier au pied du minerai de "couche verte", dans les toits marneux des couches et les marnes micacées, situées au-dessus de la formation ferrifère.

Or, le rapport des isotopes 34 et 32 du soufre, est caractéristique d'un milieu de dépôt. Ce rapport est modifié lors des réactions chimiques, par action bactérienne (^{34}S est moins "mobile" que ^{32}S). Par contre, l'oxydation de la pyrite en sulfate, à température ordinaire, et la mise en solution de celui-ci, n'affectent pas la composition isotopique du soufre. Des calculs thermodynamiques permettent d'estimer les rapports isotopiques dans le minerai (+ 7°/$_{oo}$ par rapport à l'étalon de Canyon Diablo) (ARNOLD M., 1978) et dans les niveaux marneux de l'encaissant (-25 à -40°/$_{oo}$) (SCHWARCZ H.P., 1973), à partir de la composition isotopique des sulfates marins contemporains (+ 20°/$_{oo}$) (PILOT J., 1973).

Les résultats obtenus sur les sulfures isolés des marnes micacées sont conformes à ceux prévus par la théorie, pour un milieu marneux euxinique (tableau 4).

Echantillon	S^{34} S°/$_{oo}$
1a	- 33,3
3	- 39,1
4a	- 36,3
4b	- 36,0
4d	- 39,3
5	- 37,3

Tableau 4 : composition des pyrites dans les marnes micacées

Les résultats obtenus sur le minerai de la "Couche Verte" sont éloignés des résultats prévus ; toutefois, les échantillons 15 et 17 ont une composition isotopique proche de celle qui peut être calculée pour un milieu marin littoral (tableau 5).

Echantillon	S^{34} S°/₀₀
12	+ 36,4
13	+ 32,4
14	+ 26,5
15	+ 10,7
17	+ 11,6
18	+ 39,6
22	+ 31,8

Tableau 5 : composition isotopique des pyrites du minerai de la "Couche Verte"

Les résultats obtenus sur des sulfates tirés d'eau ayant percolé à travers les marnes micacées et la "Couche Verte", sont en accord avec la théorie, l'eau étant initialement déminéralisée (tableau 6).

Echantillon	S^{34} S°/₀₀
Marne micacée	- 39,3
Couche Verte	+ 12,8

Tableau 6 : composition isotopique des sulfates des eaux de percolation

Des eaux ayant lessivé les toits marneux des couches ont donné des valeurs compatibles avec celles de sulfures extraits de niveaux marneux réducteurs (tableau 7).

Echantillon	S^{34} °/₀₀
Toit de Rouge	- 39,3
Toit de Grise	- 38,7
Toit de Verte	- 35,8

Tableau 7 : composition isotopique des sulfates des eaux de percolation

L'eau arrivant au toit des marnes micacées a une composition isotopique voisine de -5°/₀₀ (eau de la nappe du Dogger). En comparant les valeurs données aux tableaux 4, 5, 6 et 7, et les résultats obtenus sur des eaux de mines (tableau 8), il est exclu que les sulfates proviennent de la dissolution des sulfates contemporains, gypse ou anhydrite, du dépôt des formations marneuses ou des couches minéralisées, qui auraient une composition isotopique supérieure à +20°/₀₀ (PILOT J., 1973).

Mine	Date	S^{34} S°/₀₀	
La Paix	20.03.1978	- 25,7	eau d'exhaure
Moutiers	01.06.1978	- 29,3	
Giraumont	03.06.1978	- 25,8	
Hayange Nord (niveau 3)	20.03.1978	- 23,8	eau de réservoir
Hayange Nord (niveau 0)	20.03.1978	- 27,7	
Hayange Nord (niveau 0)	25.04.1978	- 26,6	
Valleroy	27.02.1978	- 33,4	
Valleroy	10.03.1978	- 32,0	
Valleroy	20.03.1978	- 32,9	

Tableau 8 : composition isotopique des sulfates des eaux de mine

L'explication la plus plausible est que les sulfates des eaux proviennent pour l'essentiel de l'oxydation de la pyrite, finement disséminée dans les niveaux marneux, déposés dans des milieux euxiniques.

4.2 - Essais de lixiviation

Parmi les couches étudiées, nous présentons deux exemples significatifs : marnes micacées (toit du minerai) et minerai de "Couche Verte", celui-ci étant le seul niveau exploité testé à fournir des sulfates en quantité notable.

4.2.1 : Matériel utilisé

La colonne de percolation utilisée à l'E.N.S.Géologie est constituée par un tube de plexiglas de 1 m de haut et 0,19 m de diamètre. Une toile de tamis repose sur le fond.

L'eau est recyclée par une pompe de type péristaltique, aspirante et refoulante, travaillant par écrasement de tuyaux souples enroulés autour d'un rotor en forme de prisme droit. On peut placer sur le rotor plusieurs tuyaux à la fois. Pour un tuyau donné, le débit est constant (figure n° 1).

L'eau utilisée est de l'eau d'une source du Dogger (nappe qui alimente l'exhaure des mines de fer).

4.2.2 : Marnes micacées

Essai A

Cet essai a eu lieu avec des roches fraîches, quinze jours après leur prélèvement sur un dépilage récent, avec 4 litres d'eau et 15 kilos de roches, dont l'analyse est reportée au tableau 11. Les courbes se rapportant à cet essai sont représentées à la figure n° 2.

Il a été extrait 264,5 mg de sulfates par kilo de roche.

Au cours de cet essai, l'extraction du calcium est restée très faible. Dans le diagramme triangulaire Mg-Ca-Na (figure n° 4), le point A correspond à une eau sulfatée sodique.

Essai B

Une série d'essais a eu lieu sur des marnes micacées altérées, provenant d'un dépilage ancien.

L'analyse de ces marnes figure au tableau 11. Le premier essai (B) a eu lieu avec 23 kilos de roches et 7 litres d'eau. Les figures n° 2 et n° 3 permettent de comparer les résultats obtenus avec des roches fraîches (essai A) et des roches altérées (essai B).

Il a été produit 733,7 mg de sulfates par kilo de roche. L'extraction du calcium est importante et l'eau a une composition sulfatée sodique et calcique (point B des figures n° 4 et n° 6).

Figure n° 1

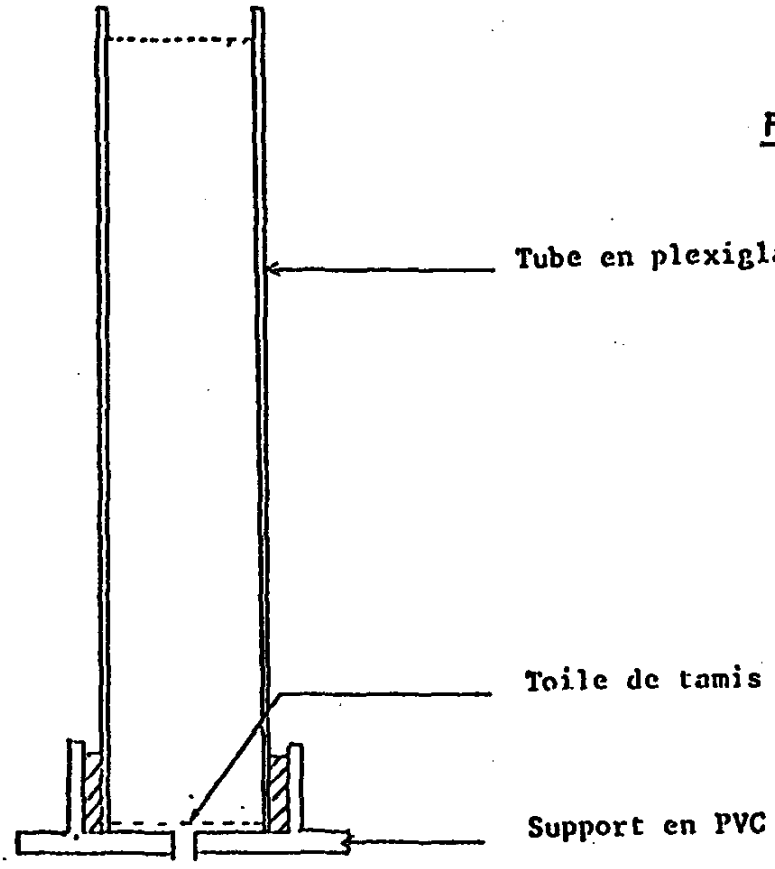


Schéma de colonnes employées à l'E.N.S.G.

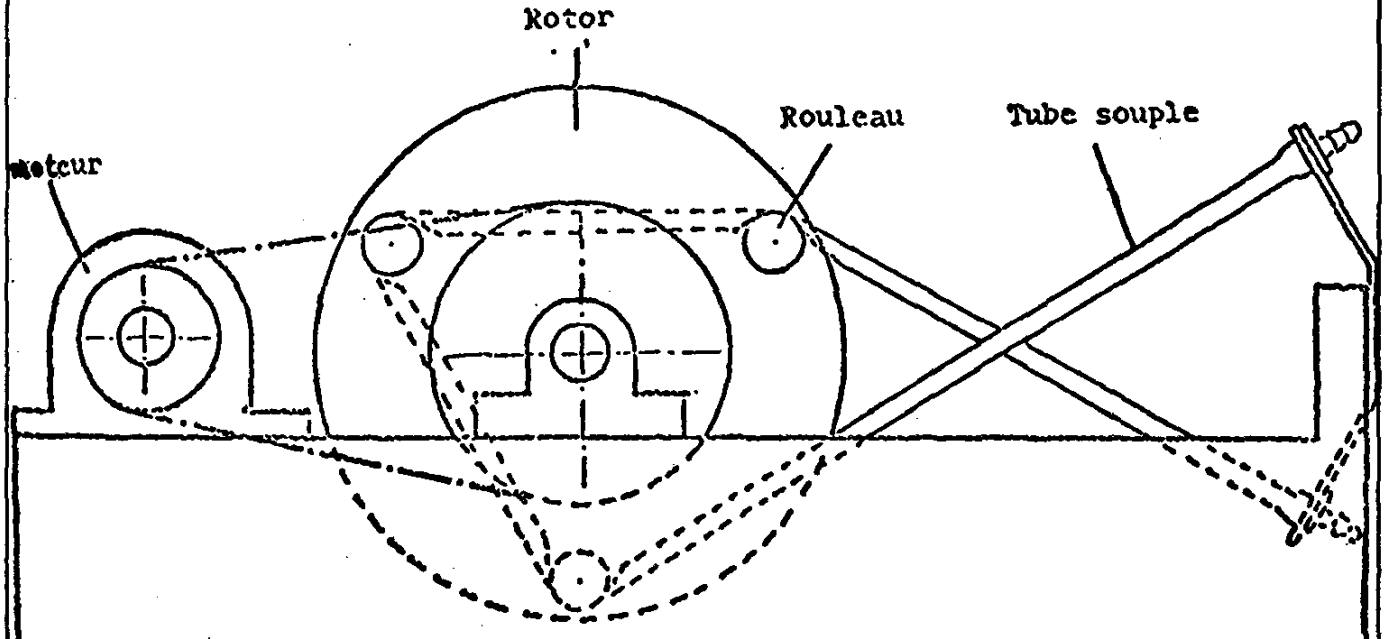
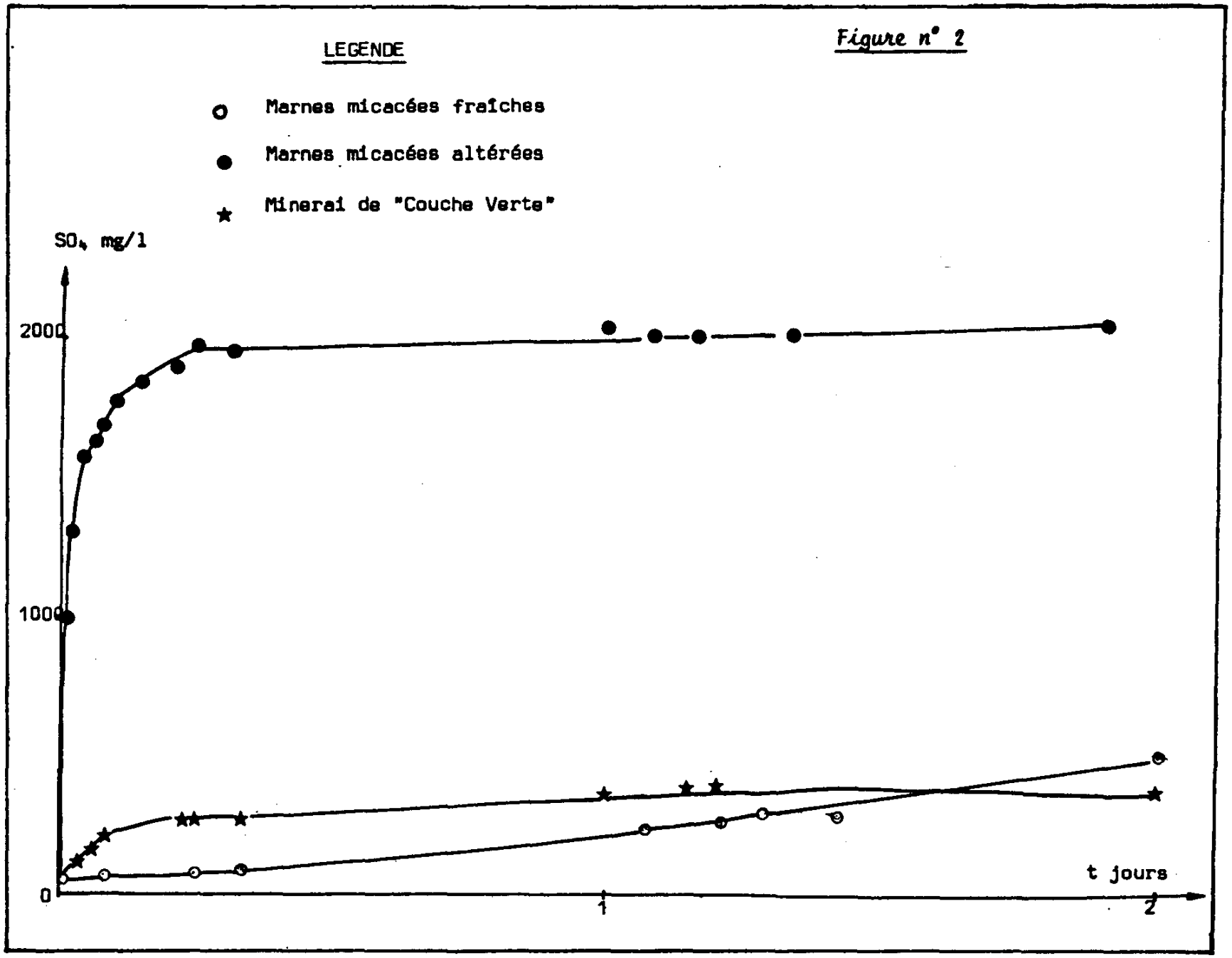


Schéma d'une pompe péristaltique de type "Delta"

Figure n° 2


LEGENDE
Marnes micacées fraîches
Marnes micacées altérées
Minerai de "Couche Verte"
SO4 mg/l
2000
1000
0
1
2
t jours

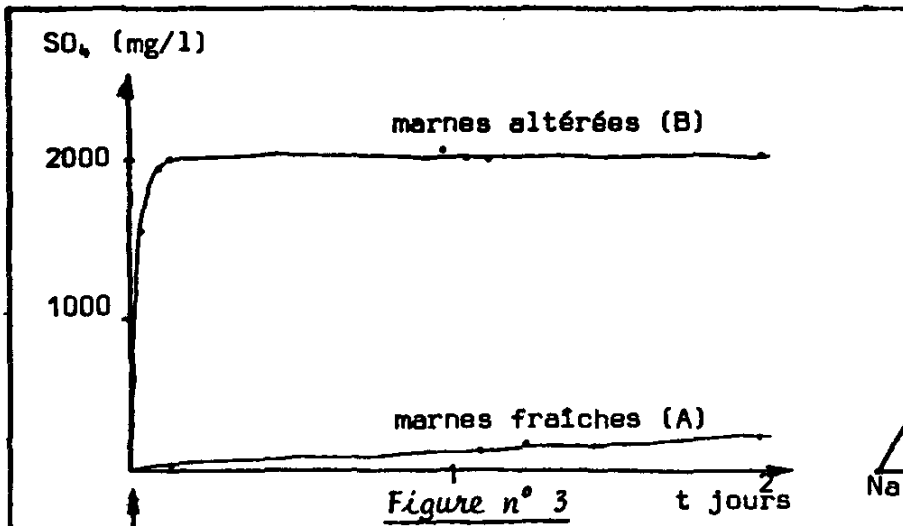


Figure n° 3

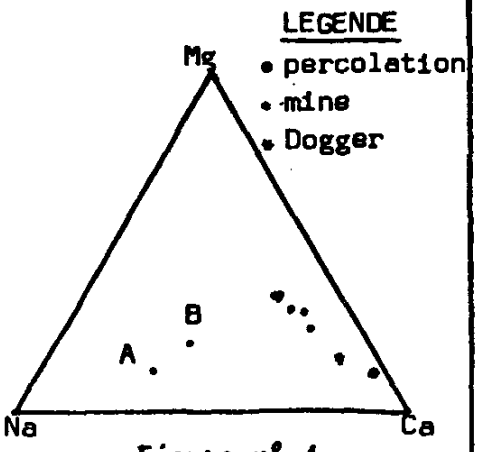


Figure n° 4

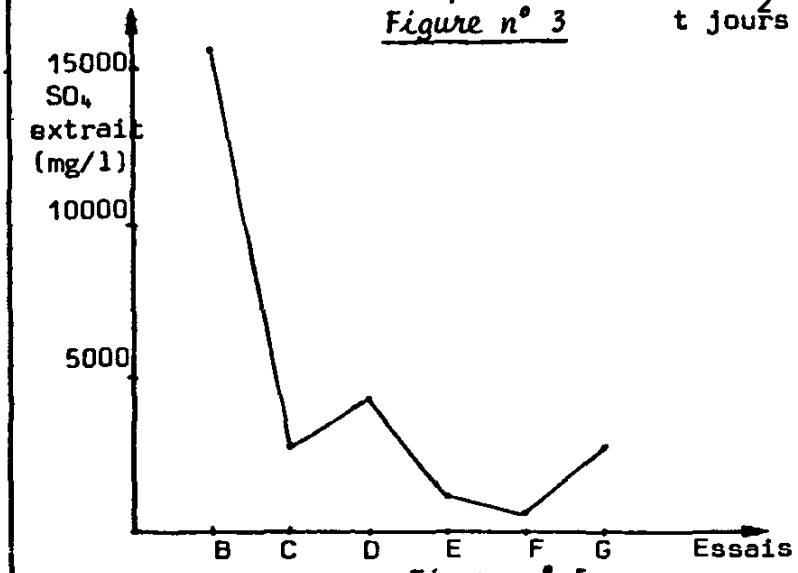


Figure n° 5

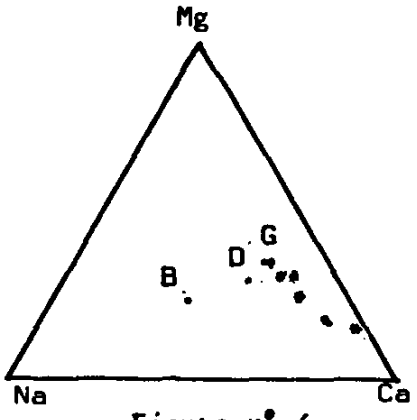


Figure n° 6

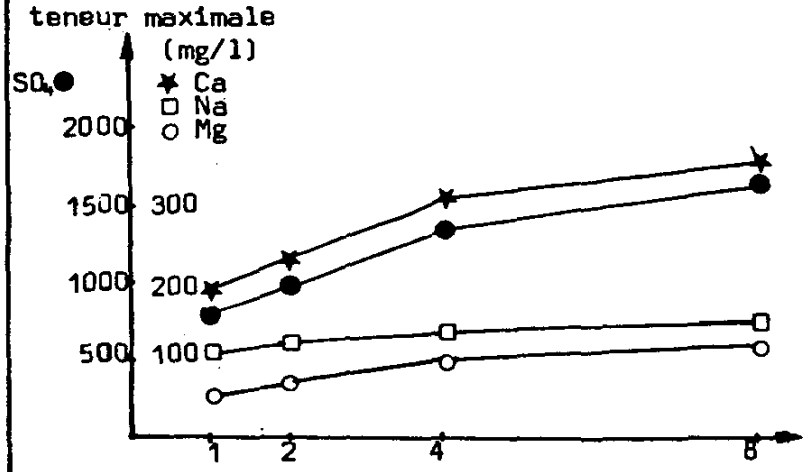


Figure n° 7

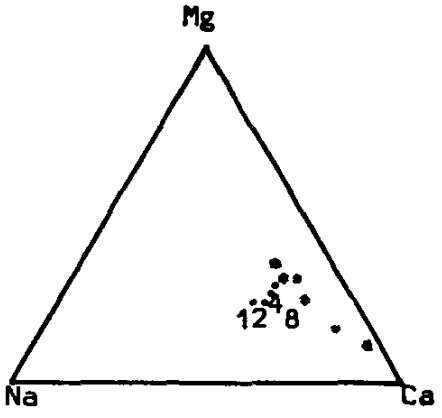


Figure n° 8

Essai C

A la fin de l'essai, toute l'eau a été retirée et remplacée par 5 litres d'eau neuve. La poursuite de la lixiviation sur le même échantillon, a cette fois fourni 126 mg de sulfates par kilo de roche. La colonne a été à nouveau vidée de toute son eau.

Essai D

Ces marnes ont été conservées huit semaines dans la colonne avant d'être à nouveau reprises pour une percolation avec 5 litres d'eau neuve. On a obtenu alors 183,7 mg de sulfates par kilo et une eau plus calcique (figure n° 6).

Essais E et F

Sur le matériau précédent, deux essais ont suivi immédiatement avec 5 litres d'eau renouvelée à chaque fois. Il a tout d'abord été extrait 57,8 mg de sulfates par kilo de roche, puis 26,5 mg.

Essai G

Suite aux essais E et F, les marnes ont été retirées de la colonne et conservées dans des sacs plastiques pendant huit semaines avant d'être reprises en percolation avec 5,5 litres d'eau. Il a été à nouveau extrait 125,5 mg/kg d'échantillon. Dans un diagramme triangulaire (figure n° 6), on note encore une fois que l'eau devient de plus en plus calcique, se rapprochant de la qualité d'une eau de mine.

L'ensemble des résultats de ces essais est résumé par le tableau 9.

Essai	*Volume d'eau*	*Poids de l'échantillon*	*Temps d'altération*	*Sulfates extraits (mg/kg d'échant.)*
A	4,0 l	15 kg	2 semaines	264,5
B	7,0 l	23 kg	plusieurs années	733,7
C	5,0 l	23 kg	0	126,0
D	5,0 l	23 kg	8 semaines	183,7
E	5,0 l	23 kg	0	57,8
F	5,0 l	23 kg	0	26,5
G	5,5 l	23 kg	8 semaines	125,5

Tableau 9 : essais de percolation sur les marnes micacées

Cet ensemble d'essais a permis d'étudier l'influence de la durée de percolation (figure n° 2), du renouvellement de l'eau (figures n° 5 et n° 6), sur la qualité de l'eau.

Essai d'altération

Parallèlement, il a été procédé à une autre série d'essais, ayant pour but d'étudier l'influence de l'altération des marnes micacées.

Après avoir lessivé les sulfates contenus dans un échantillon de 84 kilos de marnes micacées, l'échantillon a été quarté de manière à obtenir quatre échantillons de 21 kilos qui ont été conservés pendant des durées variables, avant d'être à nouveau repris pour un essai de percolation. Le premier échantillon a été repris au bout d'une semaine d'altération, le deuxième au bout de deux semaines, le troisième au bout de quatre semaines et le quatrième au bout de huit semaines.

Les résultats de cet essai sont représentés par les figures n° 7 et n° 8 : ils montrent que l'altération des marnes micacées reprend rapidement et que l'eau devient de plus en plus calcique, lorsque le temps d'altération augmente.

4.2.3 : Minerai de "Couche Verte"

Les résultats obtenus au cours d'un ensemble d'essais réalisés sur le même échantillon de 23 kilos de minerai sont reportés dans le tableau 10.

Essai A

Ce premier essai a eu lieu avec 6,5 litres d'eau. La courbe représentant l'évolution de la teneur en sulfates de l'eau de percolation au cours de cet essai, est fournie par la figure n° 2.

L'extraction du calcium a été négligeable.

Essais B, C, D

Ces essais ont été effectués sans temps d'arrêt, en renouvelant l'eau de la colonne avant chaque essai (5 litres).

Essai E

Après l'essai D, le matériau a été conservé et une nouvelle lixiviation a donné, huit semaines plus tard, une extraction de 37,4 mg de sulfates par kilo d'échantillon, avec 5 litres d'eau. L'altération n'a donc pas repris de façon sensible.

Essai F

Il a suivi immédiatement l'essai E, après renouvellement de l'eau.

Essai G

L'essai F a été suivi d'une nouvelle période d'arrêt de huit semaines ; il a été extrait 29,3 mg/kg au cours de l'essai G.

La figure n° 9, qui représente l'évolution de l'extraction des sulfates au cours de ces essais, montre que la mise en solution des sulfates est importante lors du premier essai et devient par la suite très réduite, même après des périodes d'arrêt. Sur un diagramme triangulaire (figure n° 10), l'eau obtenue reste toujours sulfatée sodique, très éloignée du pôle calcique et d'une composition d'eau de mine, même après une période d'altération.

Essai	Volume d'eau	Poids de l'échantillon	Temps d'altération	Sulfates extraits (mg/kg d'échant.)
A	6,5 l	23 kg	1 année	126,9
B	5,0 l	23 kg	0	50,0
C	5,0 l	23 kg	0	49,1
D	5,0 l	23 kg	0	36,3
E	5,0 l	23 kg	8 semaines	37,4
F	5,0 l	23 kg	0	32,2
G	5,0 l	23 kg	8 semaines	29,3

Tableau 10 : essais de percolation sur le minerai de "Couche Verte"

4.2.4 : Conclusions de ces essais

Le tableau 11 permet une comparaison des résultats obtenus avec des marnes micacées et le minerai de "Couche Verte".

Echantillon	Teneur (% poids)				Extraction après 24 h de percolation (mg)			
	S	CaO	Na_2O	MgO	SO_4	Ca	Na	Mg
Marnes micacées fraîches	0,549	2,70	-	1,24	1 000	225	129	49
Marnes micacées altérées	0,491	2,18	0,345	1,49	15 050	2 506	3 409	952
Minerai de "Couche Verte	0,060	7,15	0,152	2,05	2 509	260	2 100	39

Tableau 11

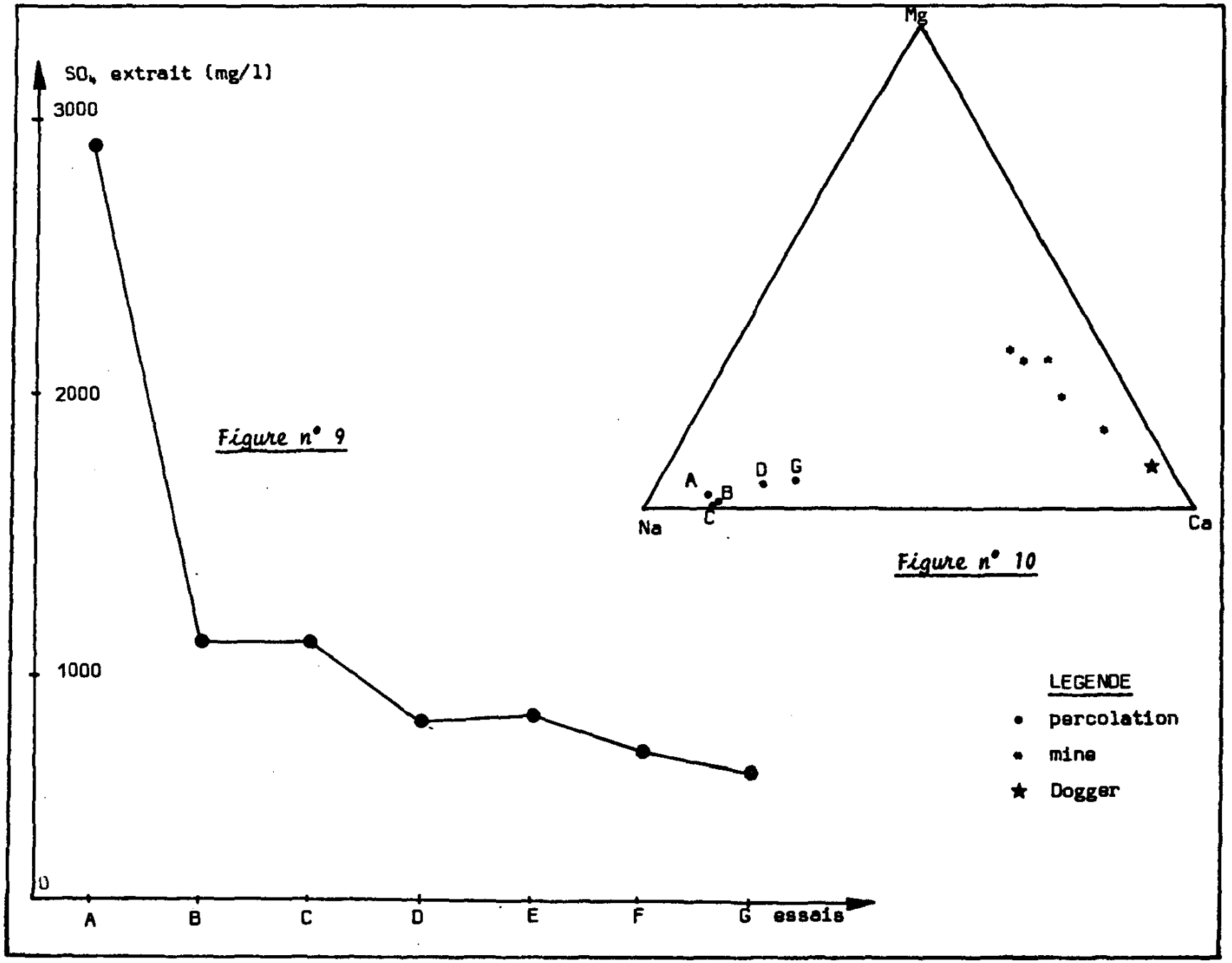


Figure n° 9

Figure n° 10

LEGENDE

- percolation
- mine
- ★ Dogger

Le minerai de "Couche Verte" donne une quantité de sulfates relativement importante, mais la dureté de l'eau reste faible (11° F) et l'eau est sodique. En outre, la quantité de sulfates extraits diminue rapidement, jusqu'à une valeur faible, et l'altération n'est pas sensible après une période d'arrêt prolongée.

Au contraire, les marnes micacées altérées, qui fournissent davantage de sulfates lors du premier essai, provoquent une dureté de l'eau très élevée (121,5° F). Leur altération est sensible après une période d'arrêt, et l'eau devient alors plus calcique, comparable à une eau de mine.

Ces essais nous montrent bien que l'essentiel des sulfates provient des marnes micacées.

Les essais entrepris avec d'autres couches de minerai et d'autres toits marneux, nous ont conduits aux mêmes résultats.

5. Conclusion

Les observations en mine, l'analyse des isotopes du soufre et les résultats des essais de lixiviations, montrent clairement la provenance des sulfates rencontrés dans les eaux de mine. Ils résultent de la mise en solution des sulfates issus de l'altération de la pyrite contenue à l'état diffus dans les niveaux marneux des toit des couches et des marnes micacées, noyés dans les zones foudroyées.

Deux solutions sont envisageables pour tenter de réduire la salinité excessive de ces réservoirs :

- maintenir un niveau constant, mais en assurant un apport d'eau peu chargée en sulfates (d'une mine voisine). Ce procédé permet à la fois de délivrer une eau de qualité moyenne et d'espérer une amélioration de la qualité de l'eau par élimination progressive des sulfates qui ont pu se former pendant l'exploitation de la mine. Par ce procédé, la mine d'Hayange a réduit la dureté de l'eau de son réservoir Nord.

- vider le réservoir à plusieurs reprises, de façon à lessiver la plus grande partie des sulfates. Cette façon de procéder, qui devait être mise en application à Angevillers, aurait permis, en outre, de comparer les résultats obtenus dans les colonnes de lixiviation sur les vitesses d'altération et les observations "in situ".

L'étude va se poursuivre par l'élaboration d'un modèle mathématique sur les échanges chimiques roche-eau, grâce à l'ensemble des informations recueillies lors de cette première phase des travaux.

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GROUND WATER HYDROLOGY - A TOOL FOR MINE PLANNING, OPERATION AND ABANDONMENT

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ABSTRACT : Most hydrogeologic problems from mining activities are related to the modification of existing ground water flow systems or the creation of new flow systems. Hydrogeology has not generally been included in past mineral resource development decisions because of : 1) the historical lack of economic incentive, 2) the lack of hydrogeologic understanding and data base, and 3) the lack of expert hydrogeologists. We believe that hydrogeologic planning for mining today is less than fully effective because it is either responding to the specific short term stress on the part of industry or it is impact oriented planning done by public agencies. For Hydrogeology to be an effective input into the mine decision process, it must be included in the economic consideration of mine planning.

RESUME : La plupart des problèmes hydrogéologiques dérivés des activités d'exploitation des mines présentent quelque relation avec la modification des systèmes d'écoulement existants des eaux telluriques et aussi avec la création de nouveaux systèmes d'écoulement. L'Hydrogéologie n'a généralement pas été incluse dans les décisions se rapportant au développement des ressources minérales du passé en raison de : 1) l'absence historique de ressources économiques, 2) l'absence de connaissances hydrogéologiques et de données de fond, 3) le manque d'hydrogéologues expérimentés. Nous croyons que la planification hydrogéologique pour l'exploitation des mines à présent est loin d'être totalement effective parce que d' un côté elle obéit à la pression de l'industrie avec ses demandes à court terme ou bien elle est aussi une planification orientée vers l'impact préparée par les agences publiques. Afin que l'Hydrogéologie soit d'une portée efficace dans les processus d'étude des décisions se rapportant à l'exploitation des mines, elle doit être comprise parmi les considérations économiques de la planification.

RESUMEN : La mayoría de los problemas hidrogeológicos, derivados de las actividades mineras, están relacionados con la modificación de los sistemas de flujo de las aguas subterráneas, o con la creación de nuevos sistemas de flujo.

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En el pasado la Hidrogeología no ha sido tenida en cuenta, generalmente en las decisiones relacionadas con el desarrollo de los recursos mineros debido a la falta de : 1) incentivos económicos históricos, 2) conocimientos hidrogeológicos y de información de base, y 3) hidrogeólogos expertos. Creemos que la planificación hidrogeológica en minería, hoy día, no es totalmente efectiva porque, o responde a las presiones de la industria, con sus demandas a corto plazo, o es una planificación orientada al impacto realizada por las agencias públicas. Para que la Hidrogeología sea incluida efectivamente en los procesos de decisión de la mina, deberá ser incluida en las consideraciones económicas de la planificación minera.

INTRODUCTION

Some of the most serious ground water quality problems faced today are associated with mining. The basis for these historical problems was largely a lack of understanding and consideration of ground water in the decision process of mineral resource development. The purpose of this paper is three-fold: 1) to examine past mining decisions that lead to pollution problems, 2) to consider the role of hydrogeology in todays mining operations, and 3) to suggest the role that ground water hydrology should play in future mineral resource development.

HYDROGEOLOGY OF PAST MINING DECISIONS

Most hydrogeologic problems resulting from mining activities are related to modification of existing ground water flow systems or creation of new flow systems. Mining can make complex chemical reactions possible that with water movement can create a discharge of poor quality water. Two examples are discussed in this paper to illustrate the actual and potential problems resulting from modification of ground water flow systems.

BLACKBIRD MINE

The Blackbird Mine in north central Idaho is an example of a site where a lack of hydrologic planning precipitated a major water quality problem upon closure (Figure 1). The mine was developed to remove a copper cobalt ore body from a mountainous site that receives an average of about 23 inches of precipitation per year. The Blackbird mining complex consists of 12 underground levels, 8 portals, an open pit, 3 major waste piles, a tailings pile, a mill and concentrator and support facilities. The mine was operated from 1917 to 1967. The present mine owner has maintained a small crew at the site since 1967 for maintenance and exploration.

The primary water resource problem at the Blackbird Mine is the discharge of low pH water with high concentrations of dissolved iron, copper and cobalt (Baldwin, Ralston and Trexler, 1978). About one-half of the metal load can be traced to point discharges such as portals. The remainder of the metal load in

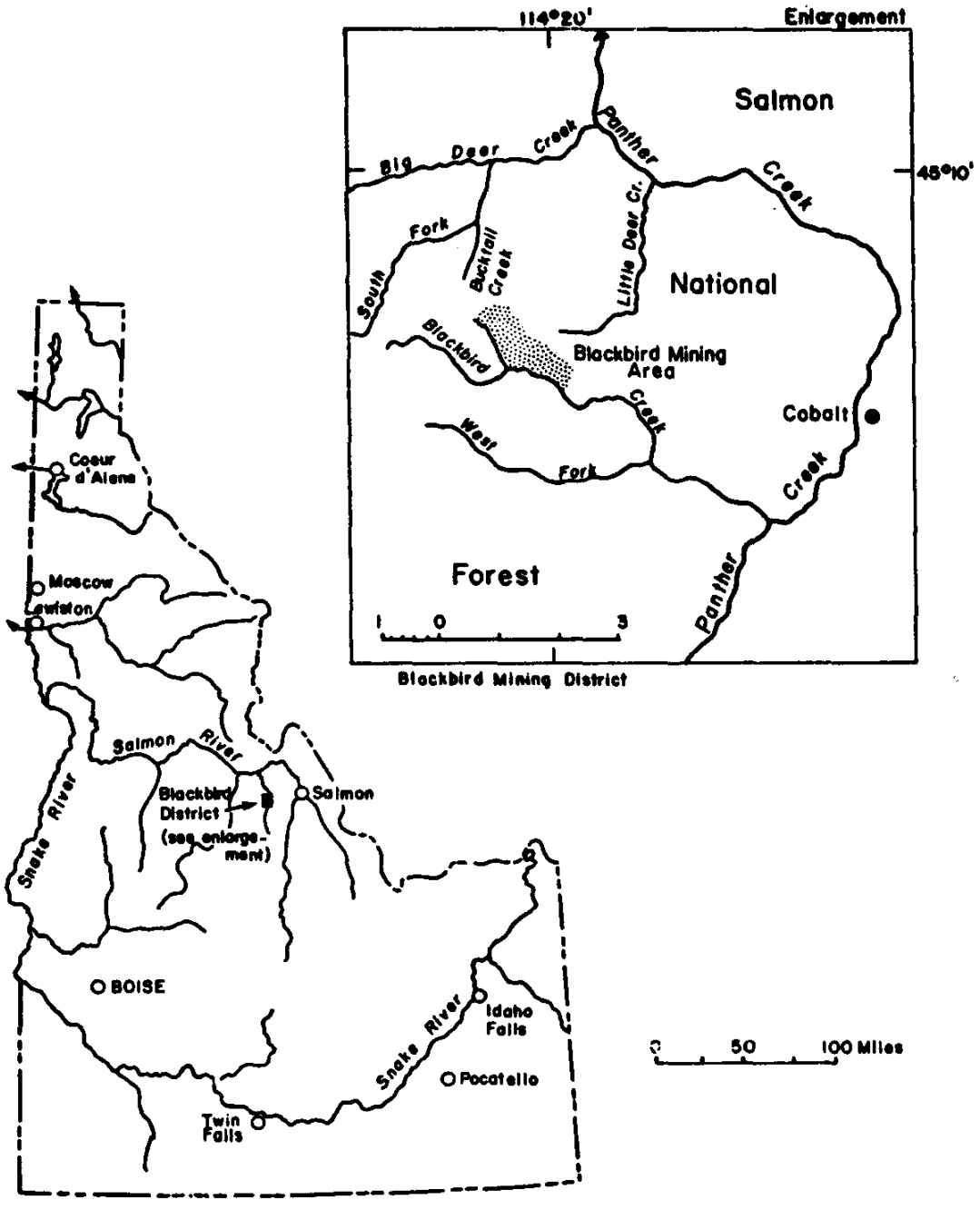

Figure 1. Map of Idaho showing location of the Blackbird District and the Blackbird Mining area.

the streams is from non-point discharge sources such as waste piles, the tailings pile and wastes along the stream channels.

Two small streams, Blackbird Creek and Bucktail Creek, receive the poor quality drainage from the mine area. Both of these streams are tributary to Panther Creek which discharges into the Salmon River. This portion of the Salmon River has been designated as a National Wild and Scenic River. The total discharge from the Blackbird mining area for the 1976 water year was about 7.3 million cubic meters (5,900 acre-feet) with about 7.2 million cubic meters (5,800 acre-feet) from Blackbird Creek.

Less than 740,000 cubic meters (600 acre-feet) of the discharge in the two streams during the 1976 water year was acid drainage. The remainder of the water was from Blackbird Creek above the mill and from small tributaries unaffected by mining. Although the total volume of acid drainage is small, the impact on downstream water uses is great.

Stream discharge and water quality are closely interrelated in the Blackbird mining area. Heavy metal concentrations are: 1) low during winter months, 2) increase sharply during the initial spring runoff period, 3) are very low during the latter part of spring runoff, and 4) rise gradually during the later summer months (Figure 2). As much as 75 percent of the total metal discharge from the mine area occurred during April and May of 1976. This large output of heavy metals occurs during a short time period each year.

Acid production from the underground workings and surface wastes is controlled by: 1) oxygen, 2) availability of pyrite and other heavy metals, 3) moisture in the mine atmosphere, 4) availability of water to transmit oxidation products, 5) mine or waste pile characteristics, and possibly 6) iron bacteria. Variables 4 and 5 (availability of water and mine or waste pile characteristics) can be managed to reduce acid drainage from the underground workings. Recharge to the underground workings occurs primarily from water movement into raises, stream loss to mine levels and seepage from the open pit to the underlying levels. Movement from level to level is primarily through man-made openings. The factors controlling acid production in surface waste piles are the same as those listed above for the underground workings. Recharge to the waste piles is mostly from precipitation; however, some piles are recharged by acid water from upstream mine features.

The April-May peak in metal production from both surface and subsurface waste features results from flushing or washing of oxidation products from the metal rich wastes. Reclamation techniques to reduce the acid drainage are centered on limiting the movement of water through the mine features. Techniques that have been suggested to reduce acid drainage include: 1) sealing of raises at and near the surface, 2) sealing or regrading the mine pit, 3) maintenance of selected drainage ways underground to minimize ponding and to route water around stopes and raises containing oxidized ore, 4) isolation of the point sources of acid water from the large volumes of good quality water,

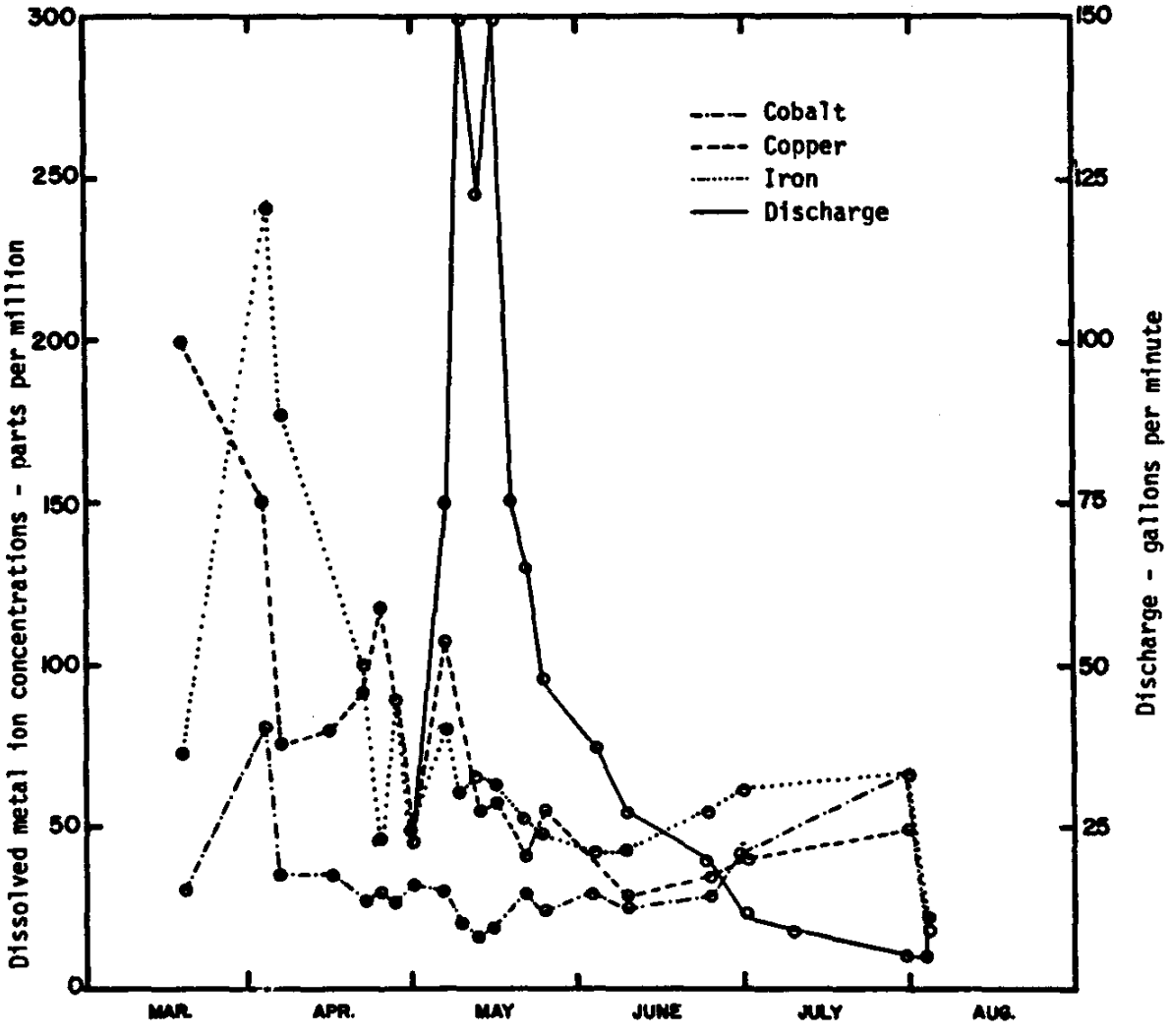

Figure 2. Dissolved cobalt, copper, and iron concentrations at station 10 (7100 waste pile) for 1976.

5) revegetation of surface waste piles to limit erosion and deep percolation and 6) construction of lined surface diversion channels to prevent recharge of water to waste piles and underground workings.

Water quality problems associated with any future mining activities in the Blackbird area can be minimized by designing the mine facilities to limit water movement through pyrite rich areas. Some discharge of poor quality water is probably unavoidable in this mining area. However, the magnitude of the problem and the ease of treatment of any drainage both benefit by hydrologic input into the decision process of mine design and construction.

BUNKER HILL MINE

The Bunker Hill Mine consists of lead-zinc mining properties located along the Coeur d'Alene River near Kellogg in northern Idaho (Figure 3). Initial mining began in 1885 in the form of many shallow, small tunnels and stopes around the original discovery site. The mine now includes more than 241 kilometers (150 miles) of workings to a depth of almost 1.6 kilometers (one mile) below land surface (Figure 4). The block of ground disturbed by mining includes a volume of approximately 21 cubic kilometers (five cubic miles). The development averages about 6.4 kilometers per year (four miles per year) of drifting and about 18,300 meters per year (60,000 feet per year) of diamond drill holes.

Acid drainage from the Bunker Hill Mine is discharged into the mine's tailings pond and constitutes a major portion of waste water that must be treated prior to discharge into the South Fork of the Coeur d'Alene River. The drainage from the Kellogg Tunnel of the Bunker Hill Mine averages about 158 liters per second (2,500 gallons per minute) of flow with a pH of about 4.0 to 4.7. The pH occassionally dips to about 3.3. The mine discharge is a major contributor of acid, heavy metals and suspended solids to the treatment plant.

The discharge from the Kellogg Tunnel includes surface and ground waters intercepted by mining facilities and water introduced during mining activities such as drilling and sand backfilling. The largest contributors to this flow are: 1) surface recharge through mine features, 2) water pumped into the mine with sand backfill and 3) water intercepted from the regional ground water flow system by the underground workings.

The quartzites and argillites that host the mineralized zones in the mine area have extremely low hydraulic conductivities except where faulted or fractured. Water from the regional ground water flow system discharges into the mine through drill holes and drift and stope areas. A total of about 850,000 cubic meters per year (690 acre-feet per year) discharges from flowing drill holes. Some of this water is used for drilling. Much of the excess flow from these drill holes could be eliminated by effective plugs and valves.

The water needed to transport sand fill into the mined stope areas amounts to about 530,000 cubic meters per year (430 acre-feet per year). This water is the major source of suspended solids for the mine discharge. The water used

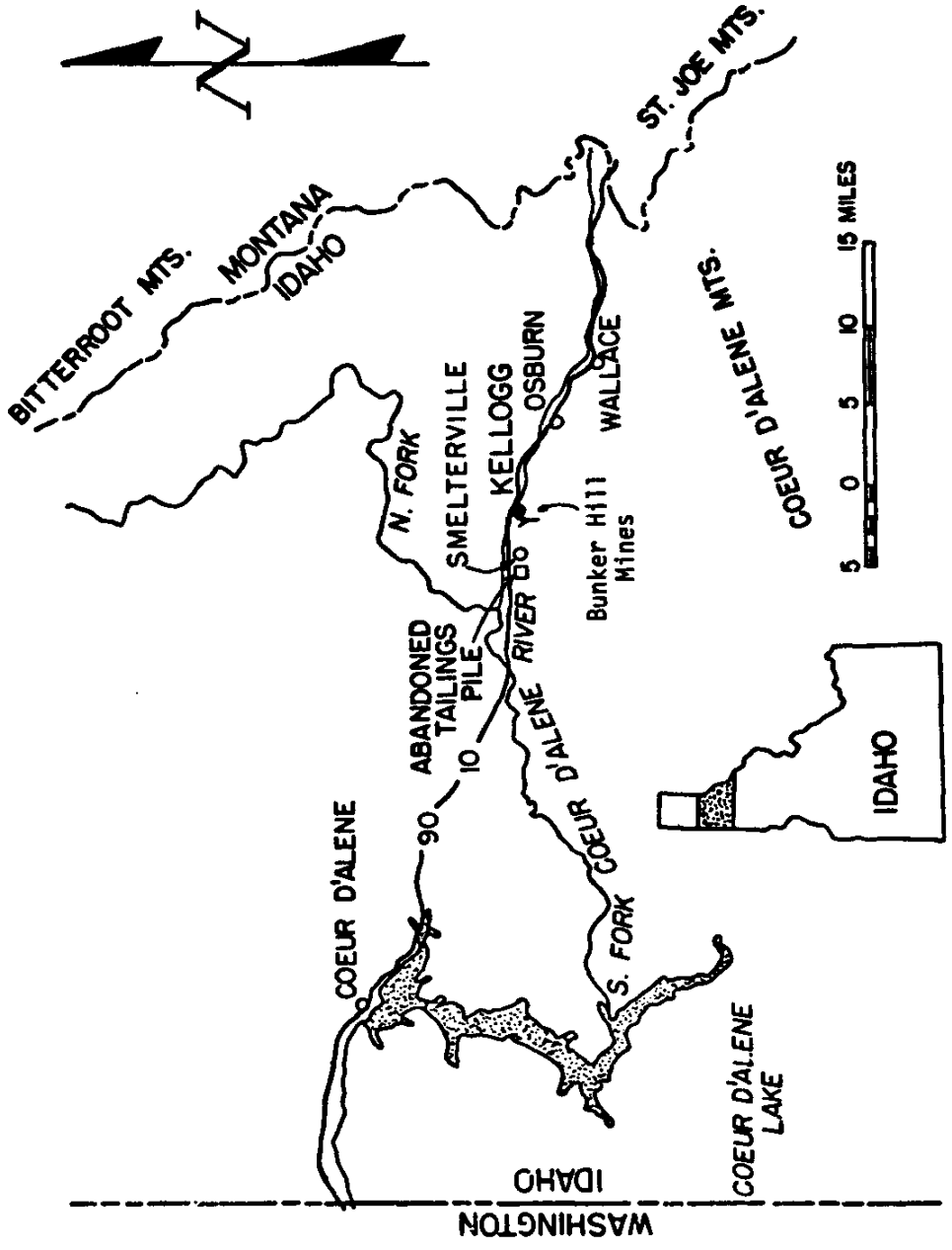

Figure 3. Map of part of northern Idaho and adjacent areas showing the location of the Bunker Hill Mine and Page tailings pile.

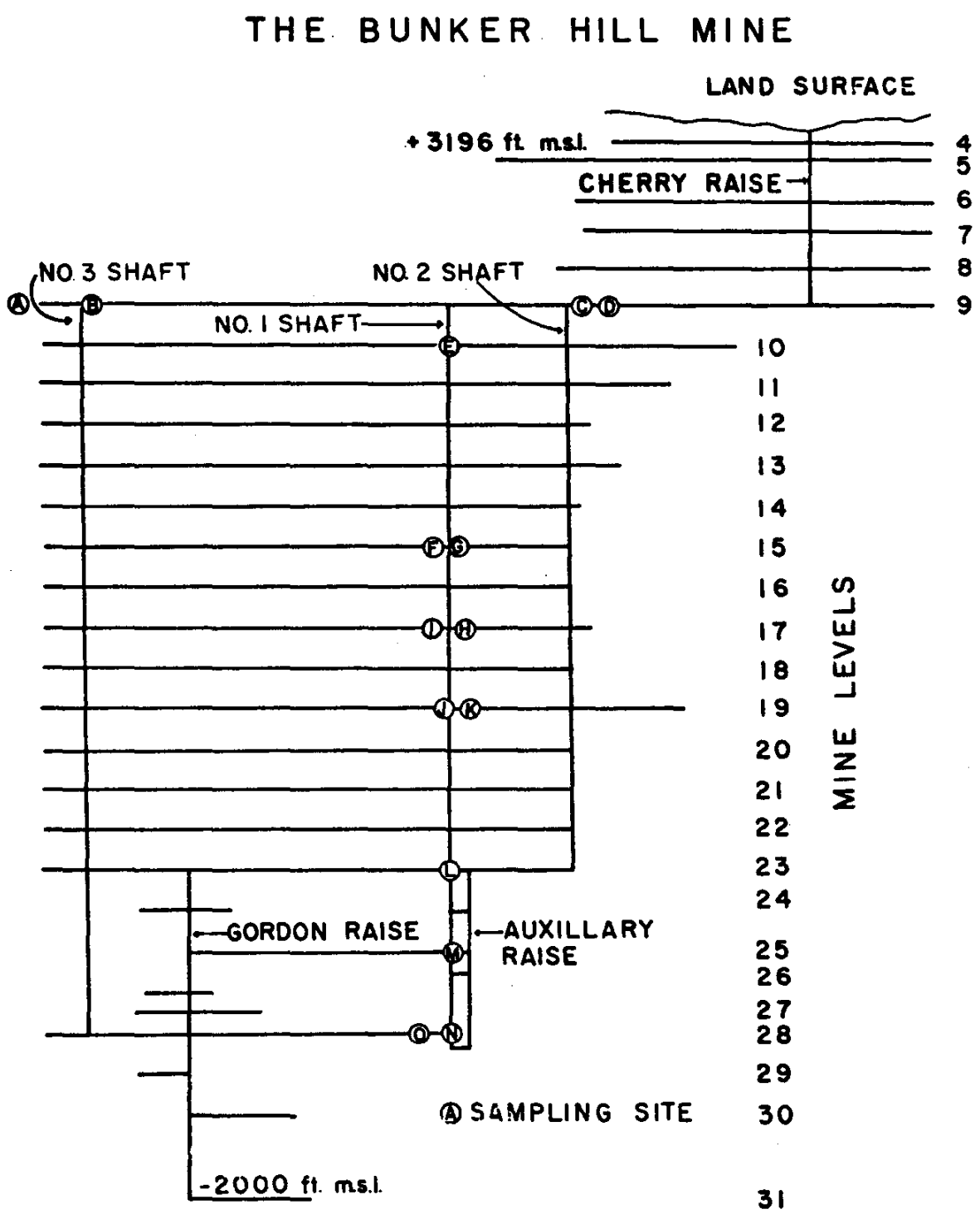

Figure 4. Generalized diagram of the Bunker Hill Mine showing location of sampling stations in the Mine's pumping system.

for sand fill and the drill hole drainage total about thirty percent of the total drainage in the Kellogg Tunnel.

The largest source for the mine drainage is surface water which recharges the mine. As the mining extended downward along the ore bodies, vertical zones of high permeability were created. Local ground water flow systems developed along these vertical permeable zones. Four areas of recharge to this local flow system within the mine were detected: 1) leakage from a small reservoir on a small creek which is located near some of the shallow stoped areas, 2) losses in an upper reach of the same stream which parallels the fault along which the mine is located, 3) an area where underground caving has resulted in major surface depressions in the bottom of several small drainages and 4) an area where stoping extended upward to intercept the channel of a small intermittent stream. The four areas have a total potential of 5.9 million cubic meters of recharge per year (4,800 acre-feet per year). The movement of water down through the mine provides the transport mechanism for acid salts formed in the old stoped areas.

The acid drainage problem in the Bunker Hill Mine is formed by the movement of water from surface recharge down through old stope areas that have been backfilled with waste that is rich in pyrite. The same variables control acid production here as were described for the Blackbird Mine (Figure 5). The ore was hand sorted during the early years of mining to insure that the highest grade of concentrates was shipped to the lead smelters. The hand sorting led to a waste rich in ore by present standards with some areas having high concentrations of pyrite. Acid production is mainly in the upper levels of the mine containing the ore rich wastefill. Surface water enters the mine and then moves downward through the interconnected stopes between the mine levels. The water washes acid salts from the highly oxidized ore and wastefill that is rich in pyrite. An examination of water quality throughout the mine shows that the acid production occurs primarily along one ore body in the upper portion of the mine (Figure 4).

Two reclamation techniques may be applied to reduce the large discharge of poor quality water from the Bunker Hill Mine. The first alternative is to reduce recharge to the mine. This involves diversion of water around the localized areas of surface recharge. The Bunker Hill Company and the University of Idaho are continuing to evaluate alternative diversion plans to achieve a reduction in water movement into the mine. The second alternative is to reduce water movement through areas of pyrite rich wastes in the underground workings. This reclamation technique is difficult because of the complex pattern of man-made openings in the upper portion of the mine.

The case histories show the magnitude and extent of problems that result when consideration of the ground water resource is not incorporated in the decision process of mineral resource development. We believe that there are three basic reasons for the lack of consideration of hydrogeology in past mine decisions.

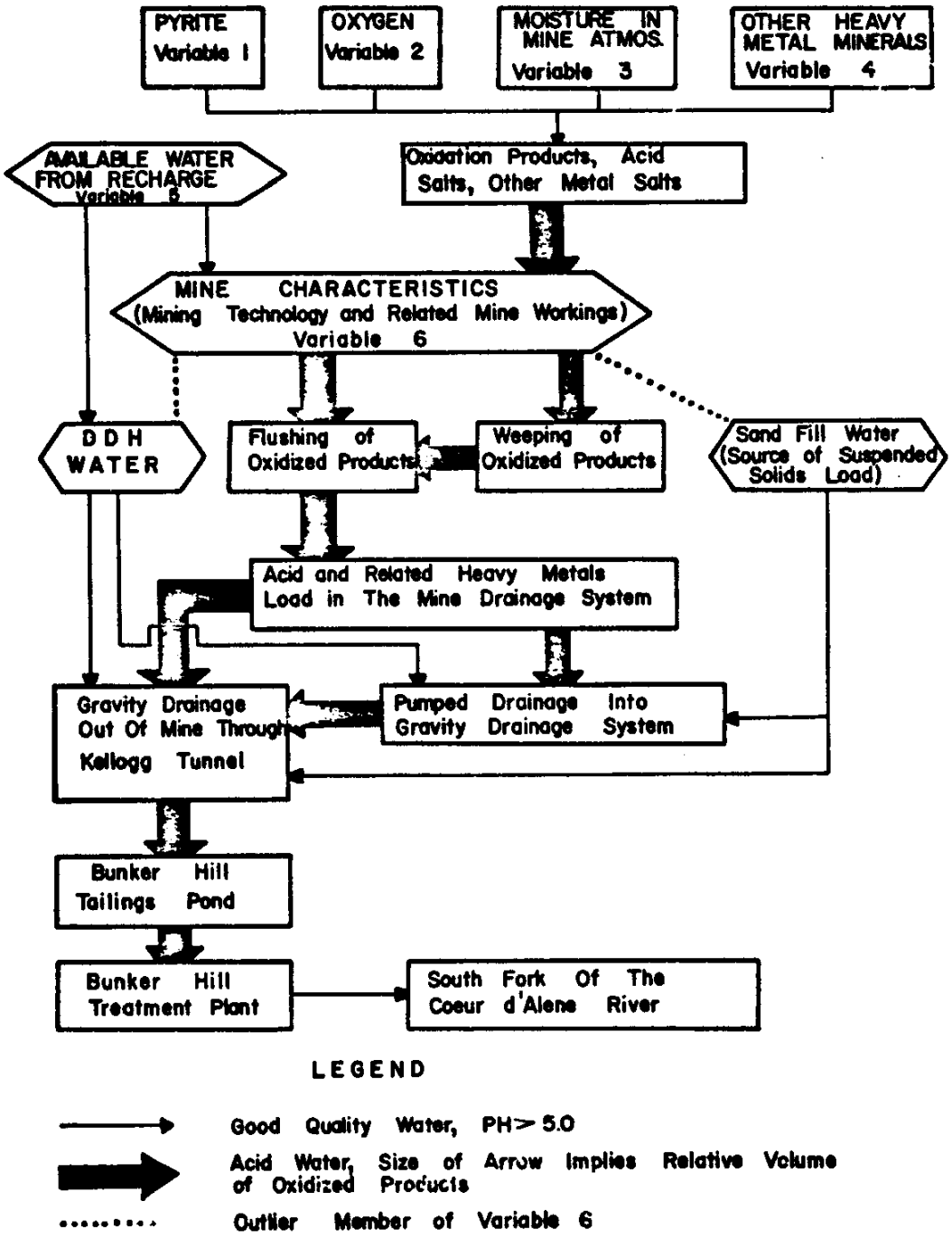

Figure 5. The variables' roles in the production of acid mine drainage in the Bunker Hill Mine.

First, there has been a historic lack of economic incentive to include hydrogeology because of the low monetary value of subsequent water resource impacts. Second, there has been a lack of hydrogeologic data upon which to base a decision. Third, there has been a lack of hydrogeologic expertise to guide either data collection or to make hydrogeologic input into the decision process. Much work is being done today to find ways to reverse the impacts from this historic lack of consideration of hydrogeology in mine development. The reclamation alternatives for problems such as those described above are limited by both physical constraints and economic constraints. The past mining activities have often created complex networks of underground openings which make physical reclamation alternatives difficult. Many of these sites also have been mined to the extent there is no longer an economic mineral base upon which to support the costs of reclamation activities. The reclamation of many abandoned sites thus becomes a federal or state problem.

HYDROGEOLOGY OF MINING DECISIONS TODAY

The hydrogeology of mining decisions today can be divided into two major parts: the hydrogeologic input into planning and operating an existing mine, and the hydrogeologic input for planning of future mineral resource development.

We will take another look at the Bunker Hill Mine as an example of the type of input that hydrogeology can make in a continuing mining operation. The Bunker Hill Company is continuing to develop new sections of the mine. Mineralogical studies have shown that the pyrite to carbonate ratio is a good indicator of potential acid production from any given ore body (Trexler and others, 1974). Any ore body where this ratio is greater than 0.6 must be considered as a potential acid producing area. This tool can be used in continuing mine development to allow the potential for hydrogeologic problems to be entered into the decision process. Once a potential acid producing area is identified, then this potential can be included in the economic decisions of mining activities. For example, if the area is economically marginal, knowledge of the potential of acid production may favor not mining. If the area is to be mined, then care should be taken to minimize the water drainage in the area and to limit transport of the acid salts.

One of the major problems with the inclusion of hydrogeology into the decisions of mine operation is the relatively low internal cost of quality degradation The cost of acid production within the Bunker Hill Mine was estimated for the years 1972 through 1974. These figures show that significant costs are associated with acid drainage; however, these costs are small in comparison to the operating costs of the total mine.

HYDROGEOLOGIC PLANNING FOR FUTURE MINERAL RESOURCE DEVELOPMENT

Hydrogeologic planning for future mineral resource development may be divided into two broad groups. We note these as: 1) internal industrial planning and 2) external public planning.

Internal industrial planning has been generally in response to public regulations and pressures. Requirements for environmental impact statements and other such documents have often resulted in the hiring of short-term consultants who often turn out reports based upon generally inadequate data and poor understanding of the hydrologic system in the area. The data are inadequate because of a lack of long term data collection by the mining companies. Short term of the employment of the consultant prevents him from developing a detailed understanding of the mine environment. This type of planning is a responding type. Often a decision is made before an adequate hydrogeologic basis for the decision can be established. Only a few companies have hired full time staff hydrologists who can make day-to-day input into the decisions of mine planning and operation.

The second kind of planning is what we call external public planning. External public planning is evident from the local level to the federal level. This type of planning may be technically excellent but generally represents only boundary or impact planning. External public planning is largely based upon some sort of regulatory function. The regulatory function is normally based upon the prevention or monitoring of impacts. The monitoring of possible impacts for planning purposes is a major problem in hydrogeology. Once a hydrogeologic problem is identified, it is usually major and difficult to reverse. The external public planning is not a direct input into the day-to-day decisions of mine planning and operation.

WHAT SHOULD BE THE ROLE OF HYDROGEOLOGY IN FUTURE MINING RESOURCE DEVELOPMENT

The life cycle of a mine includes a number of distinct steps. Hydrogeologic decisions are required in each part of the life of a mine from exploration to development, operation and abandonment. Each decision requires a different level and type of hydrologic understanding and associated data base. The level of the understanding of the created or modified ground water systems must grow with continued mine development. This cannot be achieved by either responding type of ground water planning or an impact type of planning. Hydrogeological planning is best accomplished by a full time mine hydrologist. The water resource factors must be entered into the economics of mining engineering in a cost effective manner. This is possible only with adequately trained and equiped mine hydrologist.

CONCLUSIONS

Most hydrogeologic problems from mining activities are related to the modification of existing ground water flow systems or the creation of new flow systems. This has been shown graphically by the examination of the Blackbird Mine and the Bunker Hill Mine in Idaho. Hydrogeology has not generally been included in past mineral resource development decisions because of: 1) the historical lack of economic incentive, 2) the lack of hydrogeologic understanding and data base, and 3) the lack of expert hydrogeologists. We believe that hydrogeologic planning for mining today is less than fully effective because it is either responding to the specific short term stress on the part of industry or it is impact oriented planning done by public agencies. For hydrogeology to be an effective input into the mine decision process, it must be included in the economic consideration of mine planning. It is our contention that a mine hydrogeologist can be and must be a cost effective component of continued mineral resource development if the past hydrogeologic problems associated with mining are to be avoided in the future.

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FLUIDIZED BED ELECTROLYSIS FOR THE REMOVAL OR RECOVERY OF METALS FROM DILUTE SOLUTIONS

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ABSTRACT : Akzo Zout Chemie has developed a new electrolysis technology based on the fluidized bed principle. As scaling-up problems have been solved the process is suitable for industrial application. Features of fluidized bed electrolysis (FBE) are a high specific active cathode area and a high mass transfer rate due tu turbulence created by the moving metal particles, which function as the cathode. Therefore FBE is particularly suited for the selective recovery or removal or metals which are present in low concentrations in large volume process streams. FBE can be applied both in solving waste water problems (e. g. mine water) and in hydrometallurgical processes. An important advantage of FBE over conventional techniques is the economic and simple one-step conversion of a dilute metal solution into a rather pure metal product.

RESUME : Akzo Zout Chemie a développé une technologie nouvelle d'électrolyse basée sur le principe de couche fluidifiée. Etant donné que les problèmes de "scale-up" ont été résolus, le processus est propre à être appliqué. Les caractéristiques de ces cellules électrolytiques de couche fluidifiée (FBE) sont : une grande surface de cathode active spécifique, et un grand taux de transfert de masse dû à la turbulence créée par les particules de métal en mouvement, qui fonctionnent en tant que cathode. Par conséquent, la cellule de FBE est particulièrement adaptée à la récupération sélective ou l'extraction de métaux qui se trouvent présents, en faible concentration, dans des eaux de cours à grand débit. Le FBE peut s'appliquer tant pour résoudre des problèmes d'eaux résiduelles (par exemple eaux de mine) que dans des processus hydrométallurgiques. Un avantage important du FBE sur les techniques conventionnelles est la conversion économique, par un simple passage d'une solution métallique diluée à un produit métallique très pur.

RESUMEN : Akzo Zout Chemie ha desarrollado una nueva tecnología de electrólisis basada en el principio del lecho fluidizado. Como sea que los problemas del "scale up" han sido resueltos, el proceso resulta apropiado para aplicaciones. Las características de estas células electrolíticas de lecho fluidizado (FBE)

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son : un gran área de cátodo activo específico, y una gran tasa de transferencia de masa debido a la turbulencia creada por las partículas de metal en movimiento, que funcionan como cátodo. Por consiguiente, la célula de FBE está particularmente adaptada para la recuperación selectiva o extracción de metales que se hallen presentes, en baja contracción, en aguas de procesos con gran caudal. El FBE puede aplicarse tanto a resolver problemas de aguas residuales (por ejemplo aguas de mina) como en procesos hidrometalúrgicos. Una importante ventaja del FBE sobre las técnicas convencionales es la conversión económica, por un simple paso, de una solución metálica diluida en un producto metálico muy puro.

Introduction

Akzo Zout Chemie is a producer of salt, chlorine and chlorinated hydrocarbons. Therefore, many of its process streams and waste waters are highly concentrated chloride solutions. Some of them also contain metal ions in low concentrations, as is illustrated in figure 1 for a process stream of a mercury cell chlor-alkali electrolysis plant and for a waste water stream from a chlorinated hydrocarbons plant.

A) Cl^- = 160 g/l pH = 2-3	Hg = 1-2 ppm
B) Cl^- = 5-30 g/l pH = 0-1	Cu = 30-100 ppm

Figure 1. Composition of a Mercury Chlorine Electrolysis Process Stream (A) and a Waste Water Stream from a Chlorinated Hydrocarbons Plant (B)

Lowering of the metal concentration in these solutions can be necessary because of technical requirements or because of environmental regulations. Especially, environmental regulations are expected to become more strict in the future. Therefore, it appeared necessary to develop a process to lower these metal concentrations.

Suitable processes for the removal of metals in low concentrations from streams with a high flow rate are ion exchange, solvent extraction, precipitation and fluidized bed electrolysis. Fluidized bed electrolysis was chosen because of our electrolysis experience. It was soon realized that fluidized bed electrolysis (FBE) can also be applied to the separation and winning of metals from hydrometallurgical process streams. This extended the scope of our research and development work.

Principle of fluidized bed electrolysis

The FBE cell (figure 2) consists of an anode and cathode compartment, which are separated from each other by a diaphragm. In the cathode compartment metal particles are fluidized by the process stream or waste water to be treated. The bed expansion, which is defined as increase in bed height x 100/ static bed height is varied between 10 and 30%.

The diameter of freshly added particles is about 0.5 mm. By deposition of the metal the particles grow to a diameter of about 1 mm. In consequence the particles become heavier during electrolysis and sink gradually to the bottom of the cathode compartment. The process can be operated continuously by adding small particles at the top of the cathode compartment and discharging grown particles from the bottom.

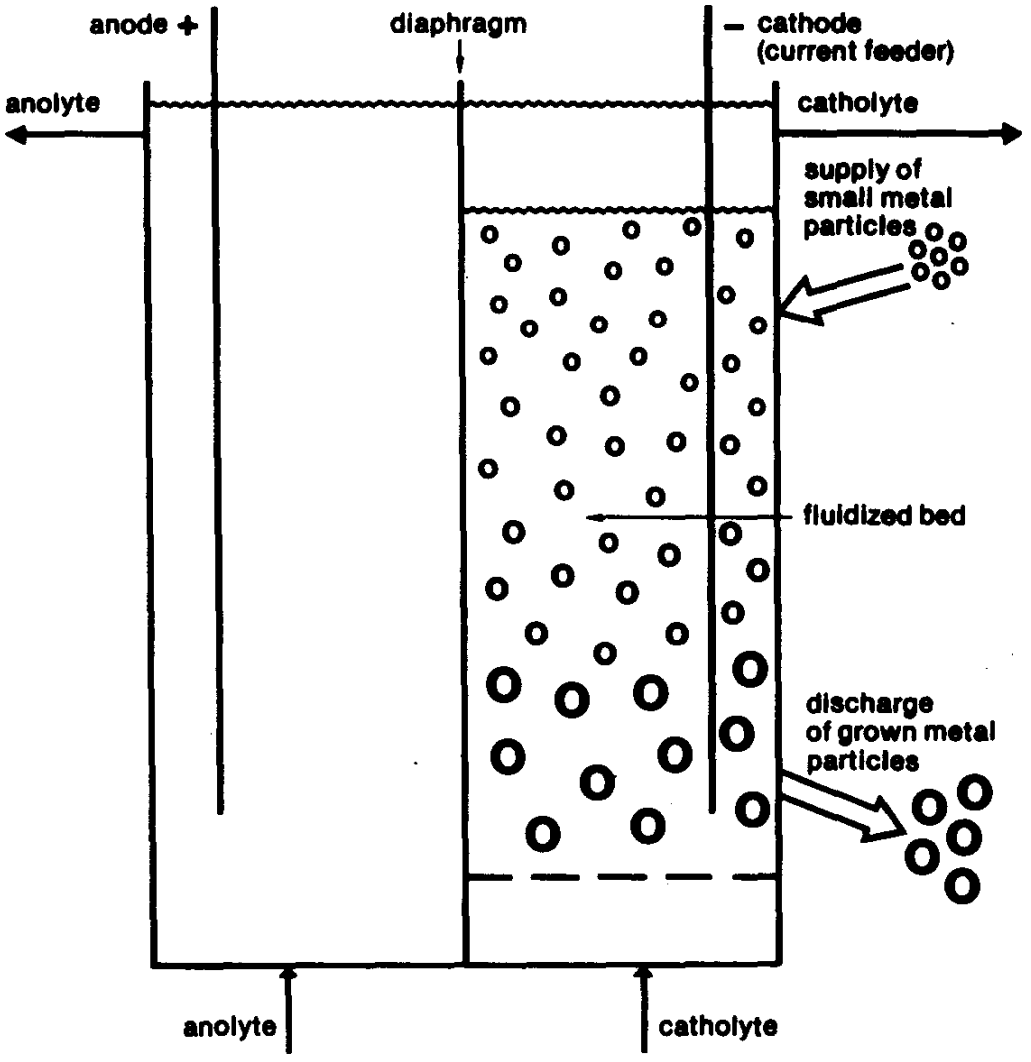

Figure 2. Schematic View of an FBE Cell

In general the material of the fluidized particles will be the same as the metal to be deposited. The electrical charge which makes the particles function as a cathode, result from their contact with the current feeders as well as from contact of particles with each other.

Features of fluidized bed electrolysis

The one-step conversion of a diluted metal solution into a rather pure metal product is an important general advantage of FBE. Other techniques, such as precipitation, cementation, ion exchange and solvent extraction, result in derived problems. The product of precipitation or cementation is a sludge or cake that requires quite some further processing before a salable metal product is obtained. The same is more or less true of the concentrated metal solutions produced by ion exchange or solvent extraction.

In comparison with other electrolytic processes, such as porous bed electrolysis or conventional planar electrolysis, the FBE cell has a high specific active cathode area.

Another characteristic feature of fluidized bed electrolysis is the high mass transfer rate due to the turbulence of the moving particles. Therefore, as compared with planar electrodes, higher local current densities can be applied without approaching the limiting current density range.

The high specific active cathode area results in low cathodic current densities at high current intensities, where current intensity is defined as the ratio of current load to cell volume. The combination of the high specific active cathode area and the high mass transfer rate, make FBE particularly suited for electrochemical reactions with a low reaction rate per unit of electrode area. The reasons for a low reaction rate can be either a high activation energy required or a low metal concentration. As a consequence, even at low metal concentrations, an FBE cell can operate at high current efficiencies.

Comparison of FBE with conventional electro-winning cells (figure 3) shows that the specific cathode area of a fluidized bed electrode is some 200 x as high as that of a conventional planar electrode. Therefore, local current densities in an FBE cell are about 40 x as low, even though the current intensity in an FBE cell is 5 x as high! With planar electrodes metals can be recovered at acceptable current efficiencies from solutions containing 30-150 g/l of metal. Because of the high specific cathode area and the high mass transfer rate in an FBE cell this concentration range is extended downwards to concentrations as low as 1 mg/l! However, planar electrolysis will in general be the most economical electrolysis process to recover metals from concentrated solutions. In case of solutions containing less than several g/l of metal FBE will be more attractive.

Characteristics	Type of Electrode	
	Planar Electrode	FBE
Cathode area/m^3 cell	16 m^2/m^3	3500 m^2/m^3
Current intensity	2.5-10 kA/m^3	15-50 kA/m^3
Metal concentration	30-150 g/l	0.001-150 g/l

Figure 3. Comparison of FBE with Conventional Electro-winning Cells

Further advantageous operating characteristics of FBE are:

— There is no clogging of the bed by solid impurities, which may be present in the process stream.

— Due to the short contact times, agglomeration of particles does not occur.

— As already mentioned before, the process can be operated continuously supplying fresh and discharging grown particles at respectively the top and the bottom of the cell.

The diaphragm

Up to now, scale-up of FBE to industrial size has been troubled by problems with diaphragms or membranes, which have to meet many requirements. The diaphragm developed by Akzo Zout Chemie solves many of the usual problems, as can be seen from the following list of properties.

a. The mechanical strength is high. The diaphragm withstands the pressure of the fluidized bed and is resistant to the erosion of the moving particles. In addition the high mechanical strength makes it possible to subject the anode compartment to overpressure in order to prevent contamination of the anolyte by the catholyte. An example is chloride transport from the cathode compartment through the diaphragm resulting in chlorine evolution at the anode. This is undesirable in many circumstances.

b. The diaphragm is made of materials with a high chemical resistance. Therefore, the diaphragm is not attacked by aggressive media, such as chlorine gas and concentrated acids, like sulphuric or hydrochloric acid.

c. The hydrodynamic permeability is very low: approximately 10^{-2} m^3 water per hour and per m^2 of diaphragm at a pressure difference of 1 atmosphere. The low hydrodynamic permeabillity is required in order to prevent a substantial loss of catholyte to the anode compartment due to pressure differences over the diaphragm of up to 0.5 atmosphere.

   The very low permeability of this diaphragm allows operation with a high bed-height so that a high depletion per pass can be achieved. Moreover, the anolyte can be chosen irrespective of the catholyte. In general, sulphuric acid is chosen as anolyte because of its high electrical conductivity and the absence of chlorine gas evolution.

d. The structure of the diaphragm is such that the electrical resistance factor is low, i.e. about 2.5, whereas the permeability is very low (c). In consequence the contribution of the diaphragm to the cell voltage is small.

e. The smooth diaphragm surface prevents adhesion of particles, wich causes scaling that may spread over the entire cell.

Cell construction

The following targets were aimed for during the development of the cell:

1. Scale-up should be economically advantageous. For many chemical processes the ratio of plant costs of different capacities is roughly proportional to the ratio of the capacities to a power between 0.6 and 0.7. A much lower scale-up advantage is available in electro-chemical processes because of the dependence of capacity on electrode area and the need to locate these electrodes accurately in the equipment. The FBE cell developed shows a scale-up advantage comparable to that of chemical plants: the ratio of plant costs is proportional to the ratio of the capacities to the power 0.731

2. Cell construction and process control should be simple.
3. Particle agglomeration should be prevented. This objective requires round corners, smooth surfaces and uniform fluidisation.
4. The cell should be closed. One reason is the desired possibility to subject the anode compartment to overpressure. Another reason is chlorine evolution at the anode in case of metal removal from concentrated chloride solutions. Therefore, in the case of chloride hydrometallurgical applications, a closed cell construction is a necessity.

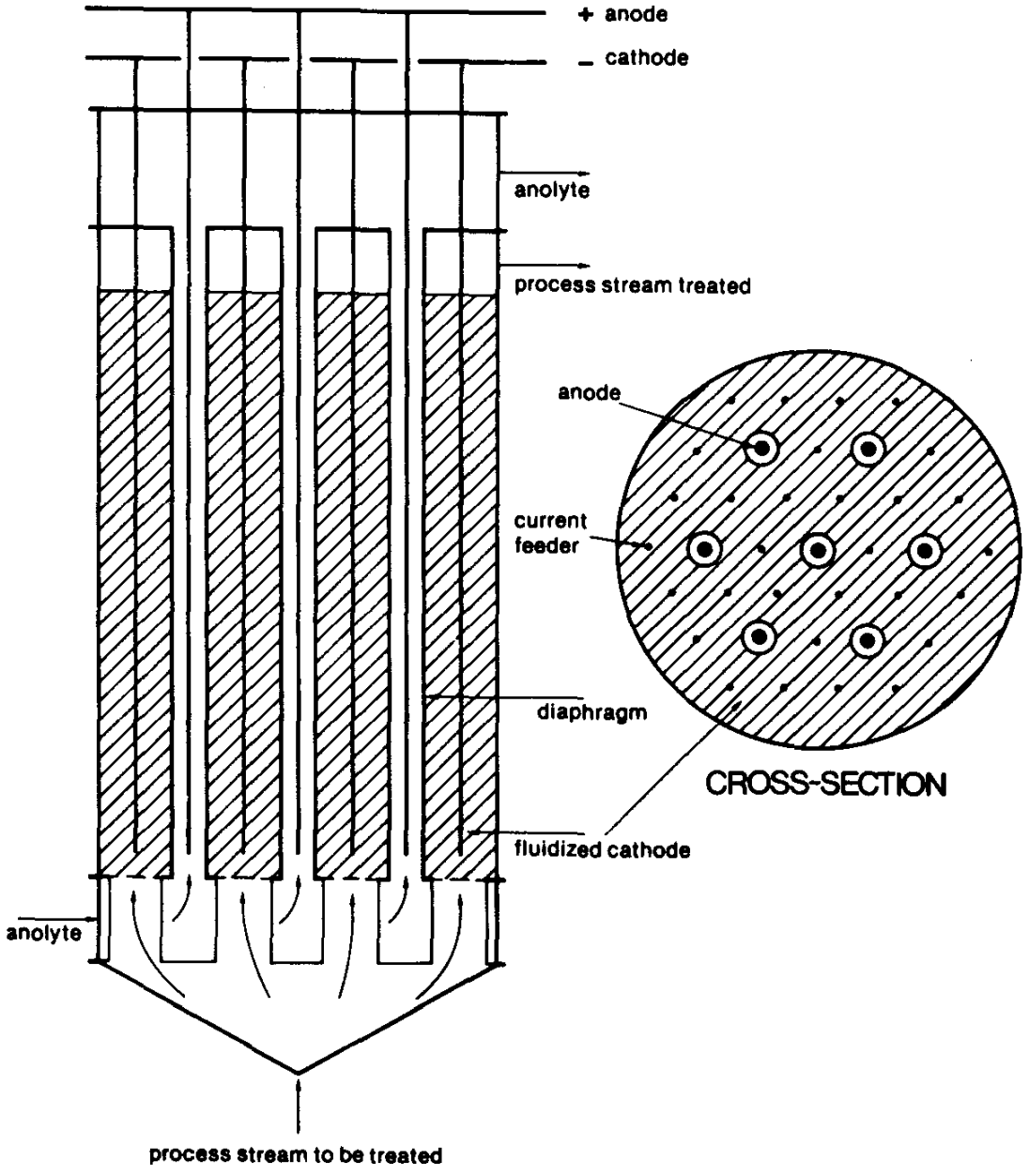

Figure 4. Cell Configuration of FBE Cell

The final result of our cell development, which meets the aforementioned requirements, is a cell configuration as shown in figure 4. The main characteristics of this configuration are the cylindric diaphragm construction and the possibility to install many diaphragms in one cathode compartment. This type of cell construction allows simple enlargement of units, while both large and small units require only one catholyte supply. This is an important advantage over filter press type FBE cells.

The anodes are placed in the centre of the diaphragms. In consequence, the anode area needed is small, which makes it possible to apply stable, but relatively expensive anode materials. Around each diaphragm six bar-shaped contact electrodes or current feeders are installed to supply the fluidized bed with current.

Figure 5 shows the largest FBE test cell of the Research Department of Akzo Zout Chemie. This cell contains 7 diaphragms, while the height of the fluidized bed is about 1.2 m. The diameter of the cell is 0.35 m. In general the operational conditions are a current load of 1000-2000 A and a flow rate of 10-20 m^3/h.

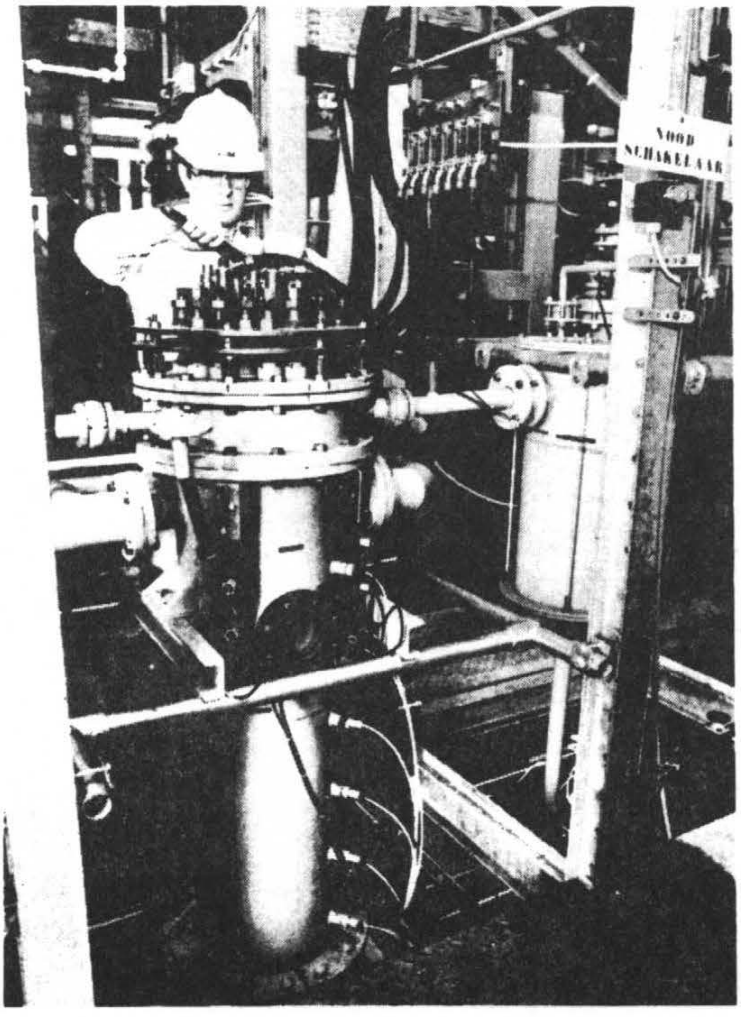

Figure 5. FBE test cell, containing 7 diaphragms, at the research laboratories of Akzo Zout Chemie

The process flow diagram of a complete fluidized bed electrolysis unit is given in figure 6. The installation is composed of an FBE cell, which makes the heart of the unit, an anolyte circuit and a catholyte circuit. The two circuits are separated from each other and due to the low diaphragm permeability the anolyte can be chosen irrespective of the catholyte. Moreover, the anode compartment can be subjected to overpressure in order to prevent a flow of catholyte through the diaphragm. Recirculation of catholyte is necessary to keep the fluidization velocity constant, even when the supply varies. Both the anolyte and the catholyte recirculation tank function also as a gas-liquid separator.

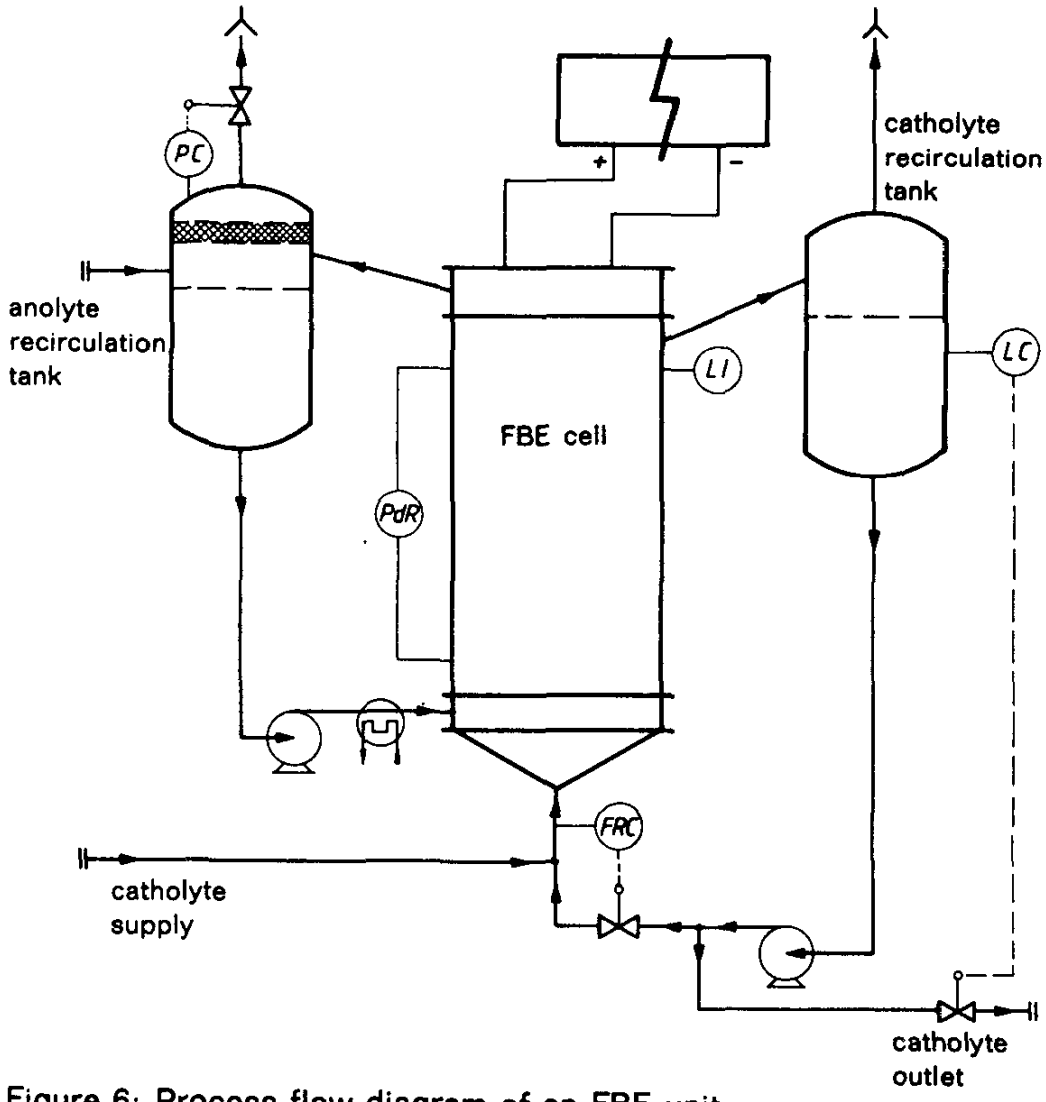

Figure 6: Process flow diagram of an FBE unit

Test results

The first process flow, which we examined and subsequently treated in a test unit in situ, was a copper containing waste water stream from a plant for chlorinated hydrocarbons (figure 7). In this test unit with a capacity of about 1 m^3/h, the copper content was lowered from 100 to less than 1 mg/l in one pass. The vast quantities

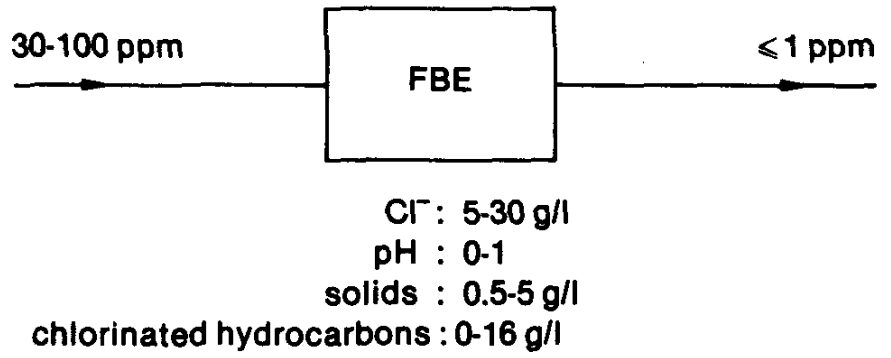

Figure 7. Copper Removal from a Process Stream in a Plant for Chlorinated Hydrocarbons

of solid particles and chlorinated hydrocarbons present in this stream did not interfere. A current efficiency of some 70% has been obtained. Chlorine evolution at the anode was greatly suppressed by the use of overpressure in the anode compartment. The copper particles were allowed to increase in weight by a factor 10, which corresponds to an increase in particle diameter of about 2.

Till now this copper removal has been performed by precipitation/filtration. However, the filtration of the precipitated $Cu(OH)_2$ sludge is rather troublesome. Moreover, cost comparisons we made for the treatment of 15 m^3/h demonstrate that:

- the investment costs of FBE are about $ 200,000 lower than in the case of precipitation, while
- the operational costs of FBE are $ 200,000 per year lower.

An FBE-unit has also been tested in a brine stream of the mercury cell chlor-alkali electrolysis plant at our Hengelo (Netherlands) site. This unit (figure 8) has already been in operation for almost two years and up to now we did not meet any major problem. The FBE cell, removing mercury from this process stream, contains 1 diaphragm and the height of the fluidized bed is 1.2 m. The mercury content of the brine is reduced from 5 mg/l to about 0.05 mg/l (figure 9). Both the ionic and the metallic mercury species are deposited onto the copper particles, with which they amalgamate. Recovery of mercury is obtained by distillation of the amalgamated particles.

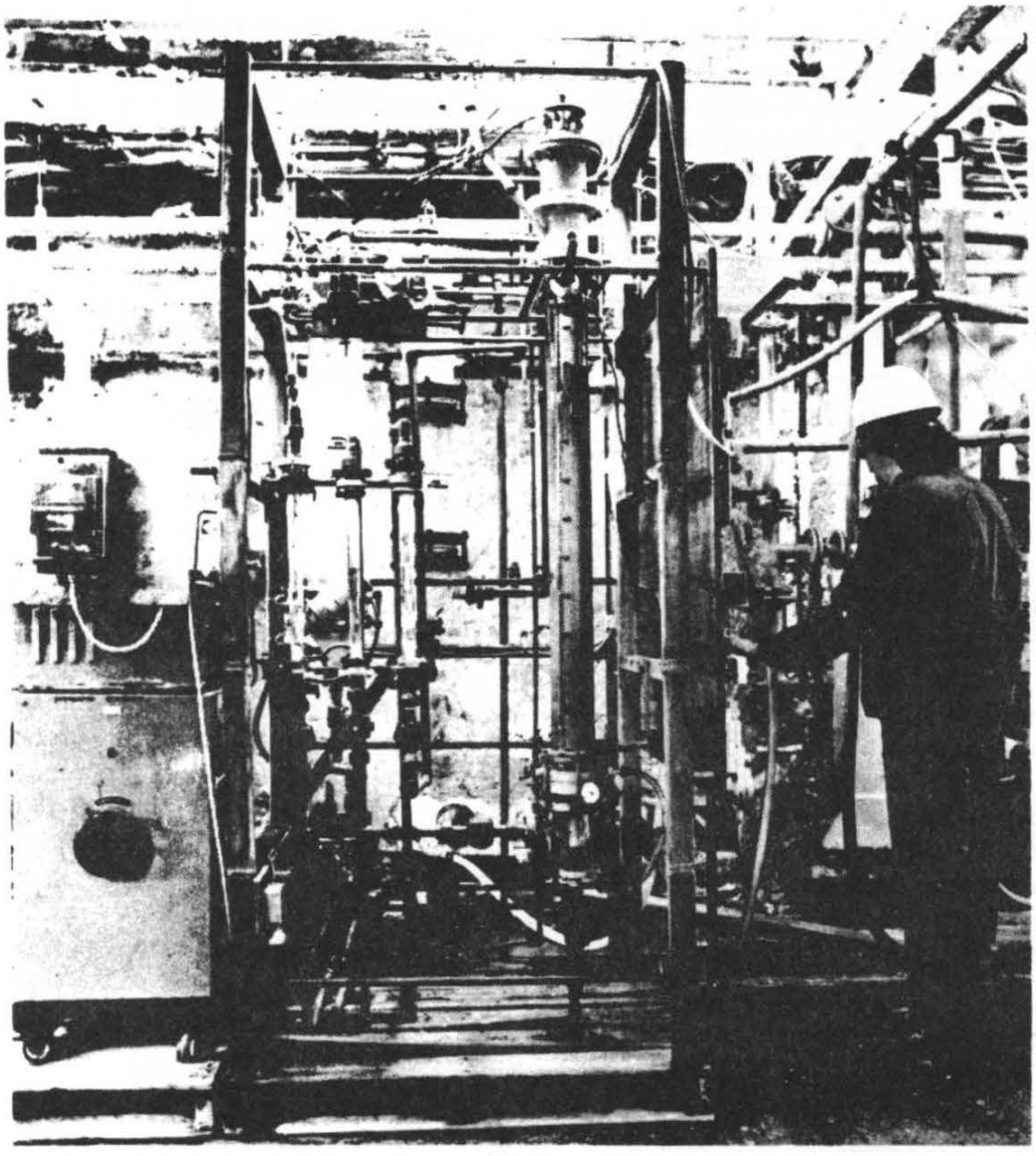

Figure 8. FBE-unit removing mercury from a brine stream

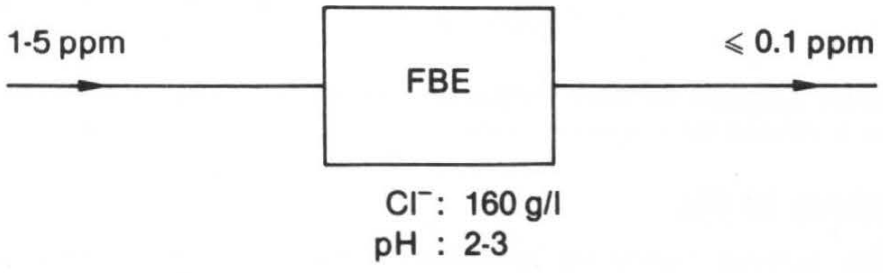

Figure 9. Mercury Removal from a Process Stream in a Mercury Cell Chlor-Alkali Electrolysis Plant

Another example of the technical possibilities of fluidized bed electrolysis is the separation of copper and nickel in a nickel electrolyte (figure 10). At a nickel concentration of some 50 g/l the copper concentration was lowered from 2 g/l to less than 0.1 mg/l. The copper content of the copper deposit was 99%. This example, just as the next one, proves that FBE can be an attractive technique to solve hydrometallurgical problems!

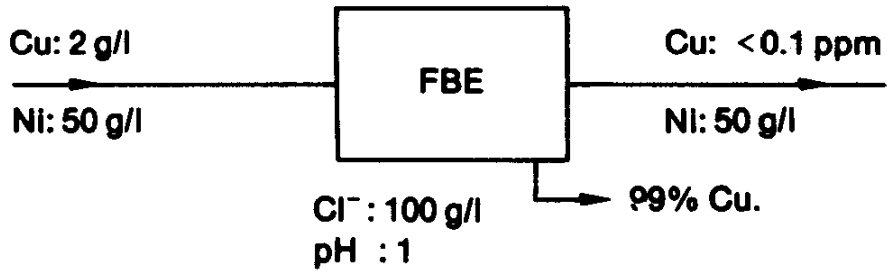

Figure 10. Copper-Nickel Separation

The presence of arsenic in copper electrolyte can be a great problem in copper hydrometallurgy because of arsenic hydride evolution during copper electrolysis of a tankhouse bleed. FBE, however, appeared to be capable of separating copper from arsenic without arsenic hydride evolution. In an electrolyte containing 5 g/l of both copper and arsenic FBE reduced the copper concentration to 10 mg/l (figure 11). Evolution of arsenic hydride did not occur until the copper concentration was further lowered to less than 0.1 mg/l. Up to copper concentrations of 10 mg/l the deposited copper appeared to be rather pure: the arsenic content was less than 0.3%

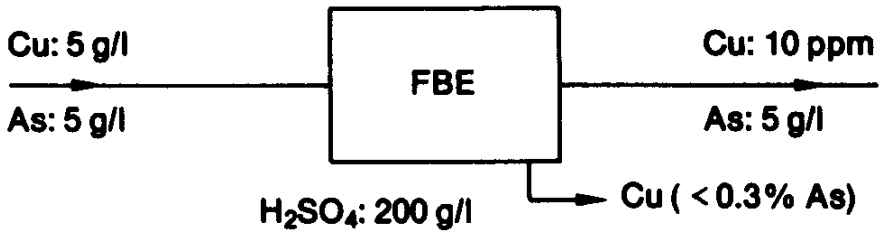

Figure11. Copper-Arsenic Separation without AsH_3 Evolution

Feasibility of FBE

Besides technical aspects the feasibility of a technique is also determined by economics. The potential attractiveness of FBE in waste water treatment will be demonstrated by a case of copper removal from a sulphuric acid solution. The feasibility of FBE in hydrometallurgical applications will be illustrated by the removal of copper and cadmium from a zinc electrolyte.

Two FBE cells, each containing 7 diaphragms, will be installed in a plant at the Wuppertal (West Germany) site of Enka Glanzstoff. Each unit will have to remove about 1 kg/h of copper from 3 m^3/h of a 100g/l sulphuric acid solution. By reducing the copper concentration from some 300 mg/l to less than 5 mg/l re-use of the sulphuric acid solution will be possible. Up to now this copper has been removed by ion exchange. However, ion exchange requires neutralisation of the solution, for instance by ammonia, and therefore re-use of the sulphuric acid is impossible. Due to a decrease in consumption of sulphuric acid and ammonia, replacement of ion exchange by FBE will result in annual savings in operational costs of some $ 200,000 per FBE unit. Investment costs of FBE and ion exchange are about the same.

The second example of the feasibility of FBE is the removal of copper and cadmium from a zinc electrolyte.
Previously cementation with zinc dust has been used to perform this removal (figure 12). The outlet of a copper-cadmium cementation in an electrolytic zinc plant contains 1 mg/l of copper and 10 mg/l of cadmium. The addition of 3 g/l zinc dust results in an increase of the zinc concentration with about 2 g/l leaving part of the zinc in the copper-cadmium cement. Treatment of this cement results in copper and cadmium products of low quality and therefore of low value. Higher proceeds can be obtained by further costly refining.

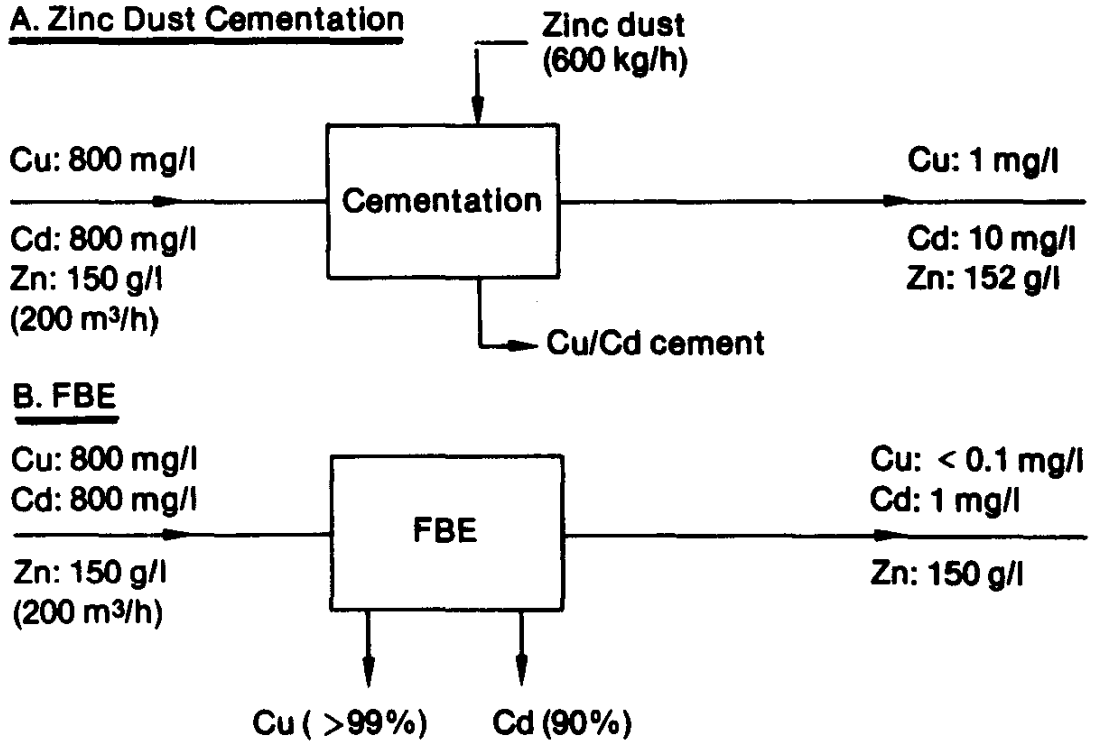

Figure12. Copper-cadmium Removal from Zinc Electrolyte by Means of Zinc Dust Cementation (A) and Fluidized Bed Electrolysis (B)

Fluidized bed electrolysis results in lower copper and cadmium concentrations (figure 12), namely less than 0.1 mg/l of copper and 1 mg/l of cadmium. The copper deposit has a copper content of more than 99%, while the cadmium deposit contains some 90% of cadmium. Current efficiencies of copper and cadmium deposition are 70 and 40% respectively.

	Zinc Dust Cementation	FBE
Investment	— $8 million	— $6 million
Operational Costs (excluding Depreciation)	— $0.8 million/year	— $1.3 million/year
Increase in Zinc Output		+ $2 million/year
Increase in Copper and Cadmium Quality		+ $1.2 million/year

Figure 13. Cost Comparison between Zinc Dust Cementation and FBE for Copper-Cadmium Removal from Zinc Electrolyte in a Zinc Electro-Winning Factory of 150,000 MTPA of Zinc

From these technical results a cost comparison between FBE and zinc cementation has been made for a zinc electro-winning factory of 150,000 MTPA of zinc (figure 13). It appears that the estimated investment of FBE is substantially lower, whereas the operational costs of cementation are about $ 500,000 per year lower at an energy price of $ 0.02 per kWh. Besides the lower investment FBE offers some further advantages over the conventional zinc dust cementation.

Firstly, there is an increase in marketable zinc output, since FBE does not require zinc dust. In consequence the part of the zinc electrolysis, required for zinc dust production, can now also yield a salable product! At an addition of 3 g/l of zinc dust to an electrolyte flow of 200 m^3/h application of FBE results in an extra yield of 5000 MTPA compared to cementation. This extra zinc output requires only an equivalent amount of zinc concentrate and a part of the low variable costs. Therefore, the profit margin on these 5000 MTPA is about equal to the added value, which is estimated at some $ 400 per metric ton of zinc.

A second advantage of FBE over cementation is the better quality of the copper and cadmium products. It is estimated that the value of the 99% copper produced by FBE is about $ 400 per metric ton higher than the copper cake of the cementation. At a copper production of 1000 MTPA this results in extra proceeds of $ 400,000 per year. It is also estimated that refining of the 90% FBE cadmium will cost some $ 2000 per metric ton less than in the case of the cementation cadmium cake. At a cadmium output of some 400 MTPA the 90% FBE cadmium will result in profits $ 800,000 per year higher than the cementation cadmium cake.

Furthermore, the cadmium concentration in the FBE outlet is 9 mg/l lower than in the case of cementation. This means that FBE will give a 15 MTPA higher cadmium output and hence extra proceeds of about $ 90,000 per year at a cadmium price of $ 6000 per MT.

Cost comparison between FBE and zinc dust cementation for copper-cadmium removal in a zinc electro-winning factory results in the following conclusion: the investment of FBE is $ 2 million lower, while the net annual operating profits of FBE are some $ 2.8 million higher!

Conclusions

We have demonstrated that the FBE technique developed can be an attractive solution to many environmental and hydrometallurgical problems. Cost comparisons showed that for removal of metals from waste waters, in many cases FBE is cheaper than alternative processes such as precipitation or ion-exchange. The data presented on the copper-nickel separation and the copper-arsenic separation without arsenic hydride evolution, as well as the feasibility study of the copper-cadmium removal from zinc electrolyte illustrate that FBE can also be a very attractive process for separation and recovery of metals from hydrometallurgical process streams.

HOT WATER IN UNDERGROUND MINING

Anderson, D. L.

ABSTRACT : This paper will discuss high temperature water in underground mining at a location in Central America. The water referred to shows a maximum temperature of 77° C and owes its origin or is directly related to recent volcanic action. The case history here presented will review methods of control and working under such adverse conditions.

RESUME : Cette communication discutera des eaux à haute température dans les mines souterraines d'une localité d'Amérique Centrale. Cette eau atteint une température maximum de 77° C et doit son origine ou est directement liée à une action volcanique récente. Le cas historique ici présenté passera en revue des méthodes de contrôle et de travail sous de telles conditions adverses.

RESUMEN : Esta comunicación se refiere a la presencia de aguas con elevada temperatura, en las minas subterráneas de una localidad de América Central. Este agua alcanza una temperatura máxima de 77° C, y tiene su origen o está directamente ligada a una acción volcánica reciente. El caso histórico aquí presentado revisará métodos de control y trabajo bajo tales condiciones adversas.

Prof. D. L. Anderson, Dept. of Mining, Metallurgical, and Ceramic Engineering, University of Washington, Seattle, Washington, U. S. A.

HOT WATERS IN UNDERGROUND MINING

Hot waters in subterranean workings is not something that is new. Such waters have been encountered and dealt with at numerous locations, and the mere fact of a geothermal gradient means that temperatures increase automatically with depth.

Temperature gradients vary from about 0.9°C per 100 meters to about 5.1°C per 100 meters of depth (Loofbourow, 1966). In general, older sedimentary and metamorphic rocks exhibit the lowest gradients, while younger intrusives and volcanics show much higher gradients. Of the latter, typical examples are found at Butte, Montana and Magma, Arizona. At the last location, the temperature gradient is reported to be about 2.8°C per 33 meters of depth (Hartman, 1961), hence for workings that are located some 1500 meters below the surface (where the surface temperatures are abnormal), this condition obviously becomes severe, and such mines can operate only with the benefit of air conditioning (Short, 1957). Finally, although investigated to the relatively shallow depth of only about 400 meters, a record or near record thermal gradient is found in the Furnace Creek Formation of Death Valley, California (1977) where a logged drill hole shows a temperature increase of 5.7°C per 100 meters of depth.

Examples of other mining areas that have moderate gradients are the Coeur d'Alene District of Idaho and the gold reef mines of South Africa. Despite the moderate temperature increase with depth, both of these regions suffer from extreme depths (especially the latter) and limited size of openings which create major ventilation problems.

Elevated rock temperatures mean elevated water temperatures and hence higher than normal air temperatures when air is introduced into openings developed. The water, being highly migratory, can have a severe direct and indirect effect on working conditions and worker efficiency.

Where orebodies and mine workings or potential mine workings occur above a potential adit tunnel level, or for some engineering construction projects, then such a drainage tunnel is an obvious way of removing waters irrespective of whether they are neutral, acidic, hot, or cold. This is illustrated by the Tecolote Tunnel near Santa Barbara, California where water at 47°C was removed by gravity flow and shallow-head pumping (Dominy and Bloodgood, 1959). On the other hand, if water must be pumped from underground workings, then several options may be considered including (for moderate depths) large diameter drill holes with deep well pumps, special pumping shafts, and combinations of such.

When flat-lying water-bearing geologic members occur above a bedded orebody, such as in the potash mines of Saskatchewan, the aquifier may be isolated

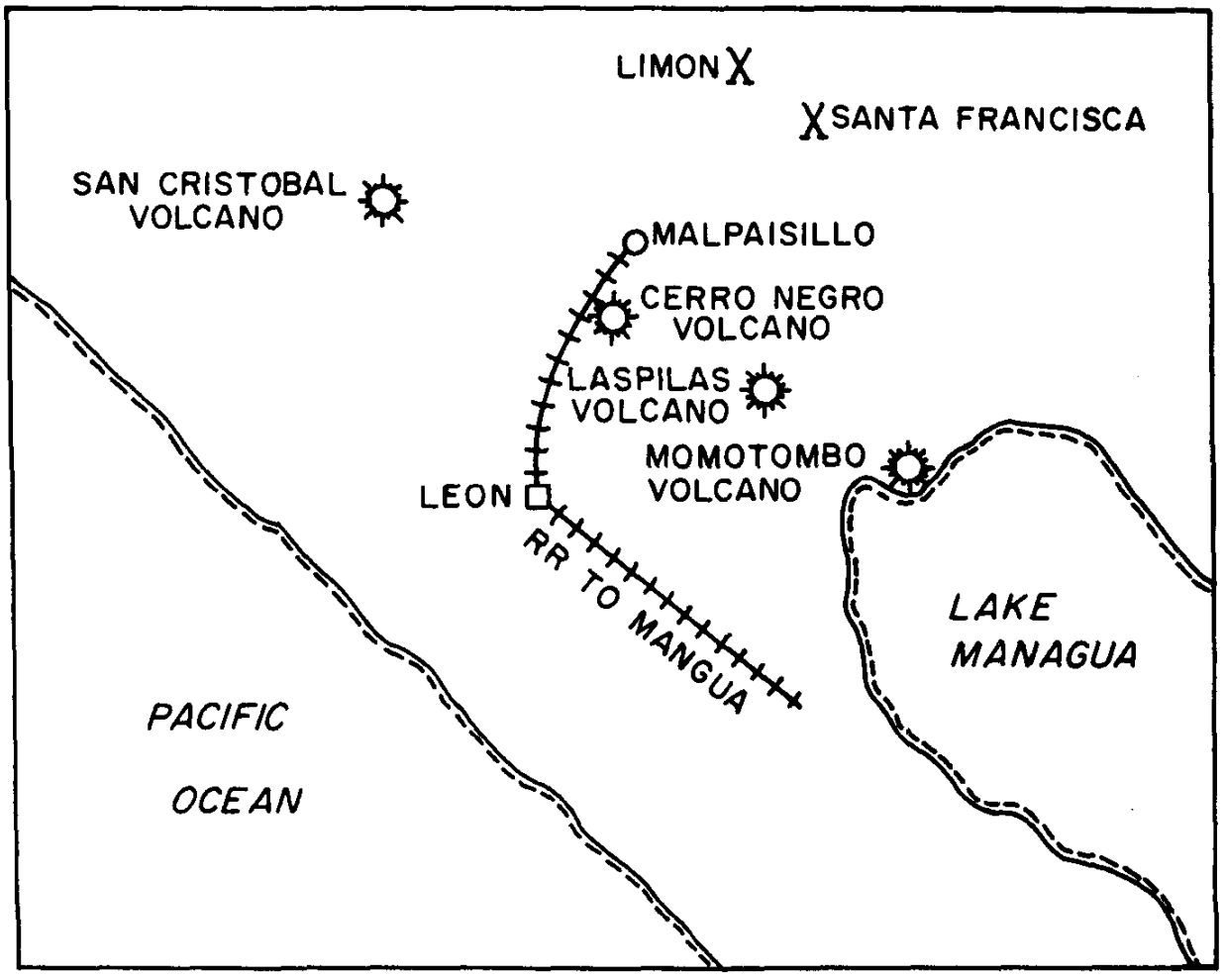
LIMON
SANTA FRANCISCA
SAN CRISTOBAL VOLCANO
MALPAISILLO
CERRO NEGRO VOLCANO
LASPILAS VOLCANO
MOMOTOMBO VOLCANO
LEON
RR TO MANGUA
LAKE MANAGUA
PACIFIC OCEAN

FIG. I

by penetrating it with a concrete or steel lined shaft. Such an orebody is continuously isolated by controlled mining on a horizontal plane. However, when an orebody is on an inclined or vertical plane and if the orebody is semi-fractured and contains water, then the problem can be severe, and the water level must be coned and lowered to or below the bottom working level. Should the waters encountered be neutral and cool, then it becomes a matter of standard pumping, either under low or high heads and low or high volumes. As opposed to the above, if the waters are hot and corrosive, then problems are compounded, and special measures must be taken to cope with the situation.

In summation, warm waters and severe pumping conditions exist in many areas throughout the world, but only in a few locations such as Casapalca, Peru (Editor, ENR, 1970) and Mina Limon, Nicaragua, do waters exist that are directly related to volcanic or late volcanic action. It is this latter location that is dealt with in this instance.

The Limon Mine of Empresa Minera de el Setentrion is a 400-ton gold-silver mine that was brought into production in the early 1940's. The mine is located some 100 kilometers northwest of Managua, Nicaragua, Central America (see Fig. 1). Limon is further described as being relatively close to a chain of active and semi-active Tertiary volcanoes. At the mine, the rock mass is predominantly andesite with accompanying rhyolite. The volcanic mountains are not particularly high in this section of Central America with San Cristobal showing the highest local elevation of about 1800 meters. From the volcanic peaks and ridges, there is a sloping trend to the northeast and at the mine, the original main shaft collar shows an elevation of less than 100 meters.

Although not the highest, the most prominent volcanic cone is Momotombo. It has an elevation of 1200 meters and is a geographical landmark for the surrounding countryside and the air approaches to Managua. Momotombo has been quiescent for decades, although it is common to see slight vapors rising from its crater. Current investigations are now underway to determine whether elevated water temperatures along the flanks of the cone can be developed as a source of geothermal energy.

The most active volcano is the lowly Cerro Negro whose history of eruptions include 1947 and 1950. Over this period, the cone increased in altitude by over 100 meters and during the eruptions spread volcanic ash over a wide area towards the Pacific Ocean. Fortunately, the mine was on the windward side of Cerro Negro (see Fig. 1).

Apart from isolated reservoirs, hot water was first encountered on the 634 (Anderson, 1959) level of the main shaft, about 20 meters below sea level, when the main vein was intersected by crosscutting from the shaft. A diamond drill hole penetrated the ore vein, a collar was grouted to the hole, and a pressure gage installed. Hydrostatic pressure was only about

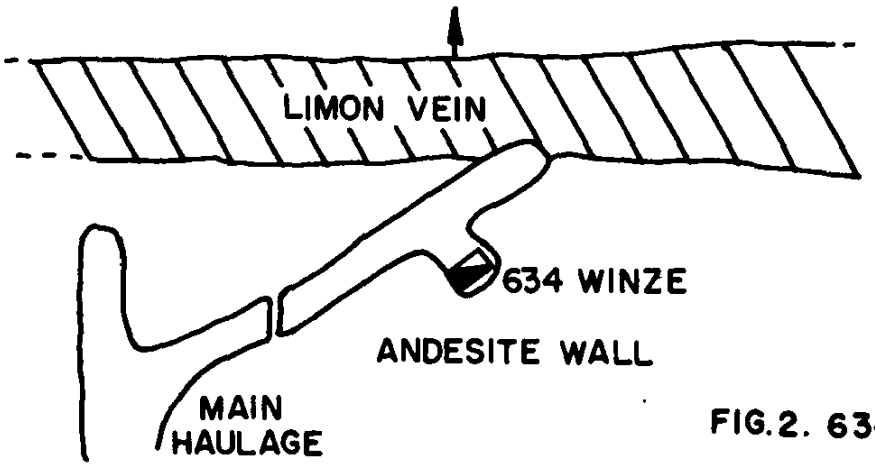

FIG. 2. 634 LEVEL

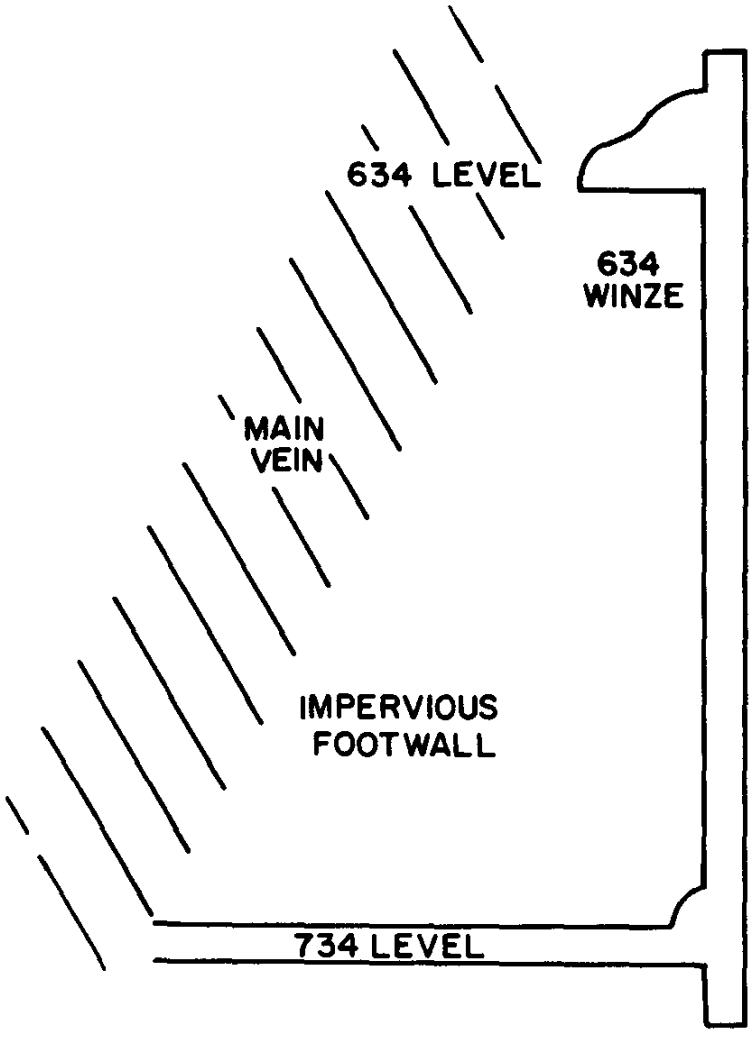

FIG. 3. WINZE AND 734-LEVEL CROSSCUT

10 meters of head, but the water showed a temperature of about 48°C. Since this was to be the main haulage level, it was decided to sink a shaft in the impervious footwall andesite and crosscut to and through the vein at 30 meters below the main level. The objective was to lower the water level and permit drifting on the 634 level under more favorable conditions. The small winze was sunk as planned with provision for a sump at the bottom to be serviced by a low-head deep well pump. The crosscut towards the footwall of the ore yielded a few isolated quartz stringers with a nominal amount of water in the range of 49°C (see Figs. 2 and 3).

The vein, about 6 meters wide, was crosscut with difficulty, but fortunately the inflow of water was limited and with controlled drilling and blasting did not exceed 300 liters per minute. Water occurred mostly in fractures which paralleled the strike and dip of the vein, and after one or two days of draining, it diminished enough to permit a further advance. Water at 49°C can be endured on the hands, but care had to be exercised to keep it off the backs and feet of the miners. Adequate ventilation was maintained by a 15 h.p. axial flow fan located near the winze collar that forced air through a 45-centimeter diameter flexible vent tube down the small shaft and up to the face. After the vein had been crosscut, the water level on the upper level dropped, permitting normal operations on that upper horizon.

The hot water so far released was clear in appearance, had a maximum temperature of 50°C, and had a slight odor of hydrogen sulfide.

Shortly after completing the 734 winze, work was initiated on a new vertical shaft to the southeast of the main orebody and somewhat nearer the volcanic chain. This area includes the Santa Francisca and San Luis veins (see Fig. 4), and hot water was known to exist here from verbal reports of early 20th century mining and from hot drill rods during exploration drilling. Diamond drilling showed irregular values, but little was known of any underground workings and even less about potential waters at elevated temperatures.

Prior to shaft sinking, consideration was given to placing one or more large-diameter drill holes in the hangingwall of the orebody, then installing deep well pumps to lower the water table. This tentative plan was not put into effect because of concern that the water might not migrate to the drill hole, and hence draw-down would be limited.

The Santa Francisca shaft was collared at an elevation of 75 meters above sea level and was sunk 237 meters to an elevation of 163 meters below sea level. It was a two-compartment vertical opening (see Fig. 5) with standard timber sets; the andesite rock was relatively firm with bearer sets at 20-meters, and most of the timber sets were unlined. The rock was cool, and nominal amounts of water in sinking were cool; drilling was done with lightweight sinkers and all mucking by hand. In effect, this was a routine shaft sinking job, and the final level was cut on the 212-meter level (700 level).

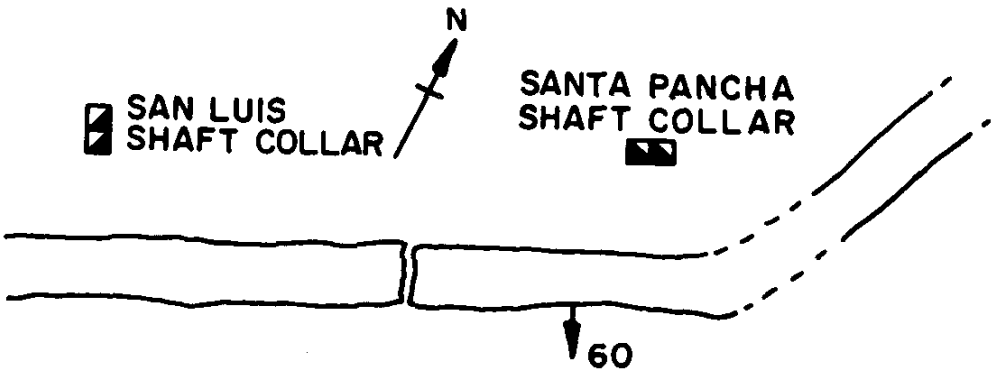

FIG. 4. SANTA FRANCISCA SHAFT

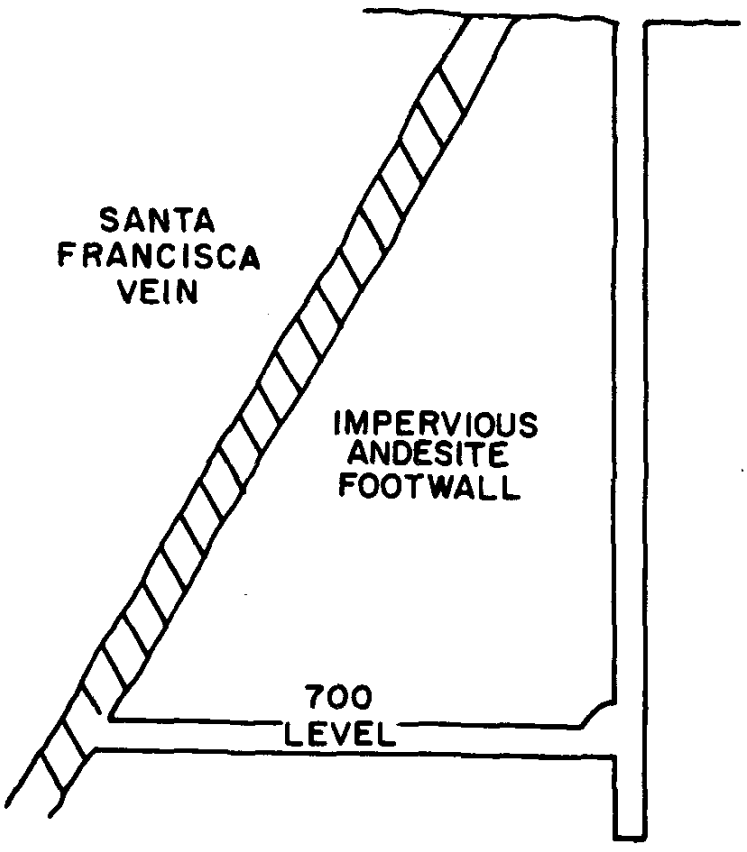

FIG. 5. SANTA FRANCISCA SHAFT AND 700 LEVEL

During shaft sinking in the cool footwall rock, no ventilation problem existed, but after the stations and loading pocket were completed, provision was made for ventilation in anticipation of adverse conditions ahead. One 15 h.p. axial flow fan was installed on surface and delivered 225 cubic meters per minute of air through a 45-centimeter diameter metal vent tube to the crosscut below. A second similar fan was installed on the level in series with the first one. This combination delivered fresh air through a flexible vent tube to within 5 meters of the face.

Upon completion of a conventional below-grade type of sump with a centrifugal pump installed, the crosscut was pushed towards the ore zone. As in the main Limon shaft, a deep ditch was carried to drain off and keep any water below track level.

In this instance, a diamond drill pilot hole was not drilled in advance of the face. It was feared that such a hole, if it encountered hot water under high pressure, would prohibit work at the face. In retrospect, these fears were justified, and if such a hole had been drilled, it would have been utterly impossible to approach the face until and if the hole so drilled was able to drain off enough water to lower the pressure sufficiently to gain access to the tunnel heading.

Each round on the 212 level of the Santa Francisca shaft was drilled with caution to avoid any sudden inflow of water. A few stringers of quartz were cut as the level progressed. These were parallel in strike and dip to the main Santa Francisca vein. Nominal amounts of hot water were developed at a temperature in the range of 50°C. There was thus every indication that water temperatures and flows were following a pattern similar to the main Limon shaft.

The footwall of the quartz vein was entered, and crosscutting of the mineralized body proceeded with no change in water flow or temperature. This sitation persisted until the entire 20 meters of vein was traversed. (It proved to be unusually wide at this point.) On a final round that presumably broke into the hangingwall, hot water at 75°C and at an estimated flow of 2200 liters per minute gushed from the face. The temperature was measured at a later date from pump discharge at the surface, and hence it is safe to say that initially, this water approached a record 77°C. The quantity of water in itself was not too great a problem. The mine staff was experienced in handling water, and the sister mine La India was at that time draining 40,000 liters per minute through its San Lucas drainage tunnel. However, water at only 23°C below the boiling point presented special difficulties that proved impossible to cope with at the time, and the shaft flooded almost immediately to within 25 meters of the collar.

When the initial impact of these events had subsided, a deep well pump was installed at the shaft collar for draw-down tests. Over a period of time

additional pumps were placed in operation until a cluster of four were concentrated in one of the shaft compartments. This permitted lowering the water to within 30 meters of the bottom level. Insofar as only limited values were found on this bottom level and in this shaft as a whole, no working re-entry was made, and the Santa Francisca shaft in effect became a high cost sump even though its drainage characteristics were only indirectly related to subsequent adjacent underground workings.

Origin of this water is a matter of some speculation. Corrosion tests showed it to be high in sodium and calcium sulfates and contrary to what might be expected, a pH in the range of 5. The presence of boron, carbon dioxide, and hydrogen sulfide was established with the inference from these latter of a magmatic source (Lindgren, 1933). A probable safe assumption is that it is a mixture of magmatic and meteoric waters. However, it needs only a glance across the broad valley floor to Cerro Negro to verify the cause of the high temperatures. In addition, at the base of Cerro Negro, a hot water "spring" yields water at 93°C.

Following the extreme conditions cited at Santa Francisca, a new vertical shaft was sunk in the footwall of the San Luis vein some 3-400 meters to the west. Here again, shaft sinking was routine and temperatures normal. However, upon intersecting the vein about 75 meters above the bottom of the previous shaft, water at 60°C was encountered. To excavate an opening under such conditions was, to say the least, difficult. A procedure adopted was to drill a bottom hole first to relieve some pressure and to effect some drainage. The rest of the round was then drilled and blasted with less water and under less pressure than originally found. As previously done, a deep drainage ditch was excavated prior to reaching the vein, and this drained water to a below-drift sump. The sump had two standard centrifugal pumps, one operating and one standby. The pump motors were kept from burning out by the discharge from a 45-centimeter duct blowing air directly from the surface.

The vein was crosscut using a technique of placing sheet metal on stulls along the back of the tunnel to keep hot water from coming in contact with the workers. After driving the crosscut into the hangingwall, a vertical raise was driven to surface as a main exhaust shaft. Hence this operation was now set up for mine development with the shaft as an intake and the ventilation raise as an upcast. Lateral development of the mine then proceeded with the lowest level (at water level) being driven with some difficulty, but the upper level development was routine.

(At Santa Francisca, the water was concentrated in the hangingwall and not within the quartz vein. As a contrast, when the vein in the San Luis shaft was crosscut, hot water was confined to the ore vein and not in the hangingwall andesite. The inference here is that there is a cross-strike feeder channel that has conduited the water selectively to the Santa Francisca shaft area.

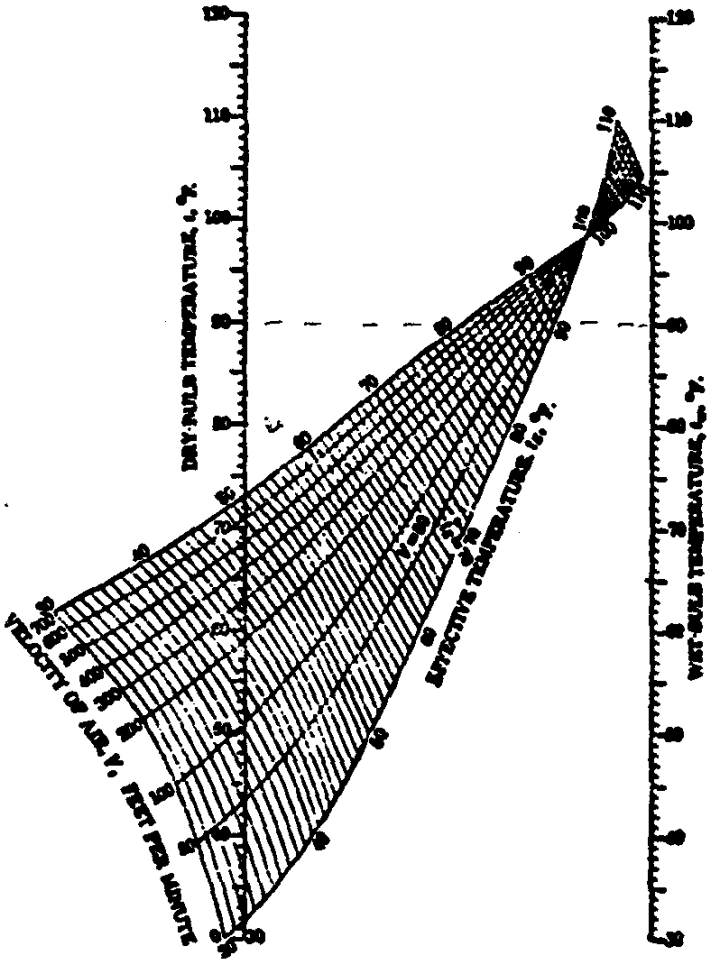

FIG. 6. EFFECTIVE TEMPERATURE CHART
(U.S. BUREAU OF MINES BULL. 385 (1935), p. 33)

This has never been directly confirmed, since more hot water was the last thing in the world that any staff member wanted to develop.)

On opening a new hot zone, an early objective was to establish a ventilation raise. The raises driven were conventional with vent tubes close to the face or in some instances with a drop cage. (The raise borers developed in the 1960's would be excellent in certain conditions for this type of development.) Upon completion of the ventilation shaft, an exhaust fan was placed at the surface over the vent shaft, and the operating shaft became a downcast. From there, auxiliary fans forced the air to the working faces throughout the mine. As mentioned, vent tubing had to be carried to within 5 meters of the face to make working conditions tolerable. Ventilation in raises was particularly difficult because of their inherent tendency to capture the hot and lighter air. This called for flexible expendable ducts to be carried close to the face, and constant maintenance and repair were required.

Hot waters at 100% or near 100% humidity produce working conditions that are difficult to cope with. Since the body loses heat by radiation, evaporation, and convection, if temperatures approach body temperature and if the humidity is 100%, then convection must play an important part. For the situation here cited, this is why large volumes of air had to be carried to within a short distance of the face.

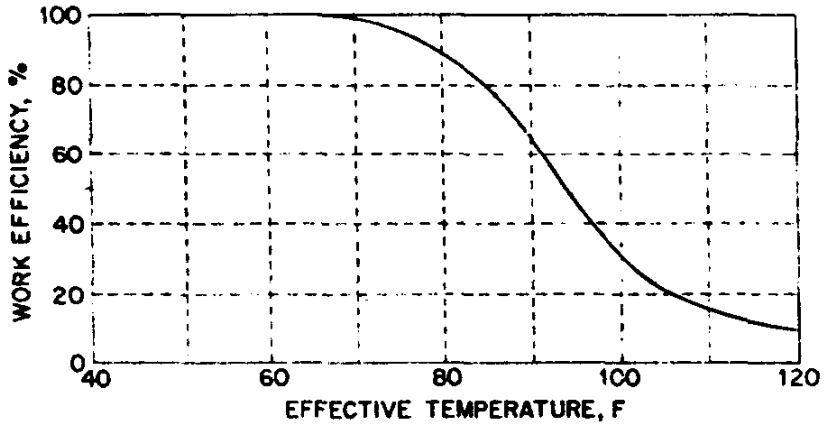

FIG. 7. EFFECTIVE TEMPERATURE AND WORK EFFICIENCY. (W. H. CARRIER, "PRINCIPLES OF AIR CONDITIONING," HEATING, PIPING AIR COND. (AUGUST, 1950): 108)

It is apparent from the foregoing charts that efficiency is low when opening a new hot water area. With air blowing at the face, temperatures were kept to about 90°F. This means that efficiency falls to about 50%. Fortunately,

this situation is found only during initial development directly within the hot water zones before any draw-down is effected. Nevertheless, the effect of the hot water is felt continuously throughout the mine, and the overall efficiency is considerably less than what would be expected under normal temperature conditions.

Mining on the San Luis vein was followed by the development of other veins including Las Mercedes which fortunately did not produce hot water. At the present time (1978), most production comes from El Panteon vein. This ore zone was first found by an exploration drill hole put down in 1962.

Hot water at Panteon approaches 76°C, and volumes have now increased to about 3000 g.p.m. Production shafts at this part of the mining complex are sunk in the hangingwall, since the HW rock is more competent and requires less shaft support and maintenance. At Panteon, a combination of horizontal drill holes and crosscuts below the lowest working level are used for dewatering.

In 1953, about 20% of all power produced by the diesel electric plant was used for ventilation; today this figure has increased. As mentioned, fan installations consist of main exhaust fans on surface coupled with auxiliary fans underground. Axial flow fans are principally used, since emphasis is on large volumes of air rather than high heads.

In conclusion, hot water of volcanic or near volcanic origin in underground mining makes for disagreeable workings and high costs, including lower than normal efficiencies and excess ventilation. Technically, however, the problem can be handled by sinking initial openings either in hanging or footwall formations, whichever is more impervious. From such impervious host rock, bottomed below the lowest operating level, crosscuts or drill holes can tap the hot waters and by special pumping techniques, lower the water level and drain the orebody. However, like most unusual problems in mining, it can best be overcome by proper advance planning coupled with beforehand knowledge of conditions that may be encountered.

ACKNOWLEDGMENT

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THE CONTROL AND DRAINAGE OF WATER IN MINE TAILINGS DAMS

BAUER, G. E.

ABSTRACT : Mining is one of the largest industries in Canada. The costs of controlling water in tailings structures represents a major share of the total cost of tailings disposal. Furthermore, the control and drainage system adopted will have an influence on the stability of the dam and on the pollution potential of the environment. Monitoring of water pressures and water flow within these structures will supply valuable records during construction and for future design considerations.

RESUME : L'industrie minière est la plus importante industrie du Canada. Les coûts de contrôle de l'eau dans les structures faites de déchets miniers représentent la plus importante partie des dépenses totales d'un tel projet. En outre, le contrôle et le système de drainage adoptés auront une influence sur la stabilité du barrage et sur l'éventuelle pollution de l'environnement. La connaissance des pressions intersticielles et de l'écoulement au sein de ces structures fourniront des éléments précieux d'information pendant la construction et pour de futurs projets.

RESUMEN : La industria minera es una de las más importantes de Cánada. Los costos de control del agua en los balses de decantación de rechazos de mina, representan la partida más importante de los gastos totales de tales proyectos. Además, el control y el sistema de drenaje adoptados son fundamentales, en cuanto se refiere a la estabilidad de las presas, y a la eventual polución del medio ambiente. El conocimiento de las presiones intersticiales y del flujo dentro de estas estructuras aporta elementos básicos de información, durante la construcción y para proyectos futuros.

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1. INTRODUCTION

Mining is the largest industry in Canada and the expanding demand for mineral ores has resulted in the production of solid and liquid wastes at an accelerated rate. Solid wastes are usually dumped on waste piles and they can also be used to fill in the mined-out slopes or open pits. In the wet process of extracting minerals considerable quantities of water are added and the resulting waste is leaving the plant in the form of a slurry. This mixture of liquid and solid constituents is generally impounded for sedimentation in tailings dams. The design engineer is faced with the task to design a water-control system which will guaranty an adequate factor of safety for stability at minimum cost.

2. STABILITY OF TAILINGS DAMS

In Canada tailings dams are generally designed using the basic principles from Soil Mechanics, in particular, principles applicable to the design of conventional earth dams. There is, however, a fundamental difference between these two structures. The earth dam will only be subjected to water and seepage forces after the structure has been completed. In contrast the construction of a tailings dam is an ongoing process over the life of the structure and is exposed to water flow right from the start.

Once the strength properties of the tailings material has been established from laboratory and/or field tests, the stability of the dyke can be adequately determined using conventional slope stability analyses, such as proposed by Bishop (1965) or by Morgenstern and Price (1965). Mittal and Morgenstern (1974) concluded from their studies that the shear strength and compressibility of tailings sand were generally comparable or better than natural sands and hence are not a problem in stability considerations of tailings dykes.

Therefore, in the following discussion the attention will be focussed on the other performance requirement: seepage control and the factors affecting seepage control.

3. METHODS TO CONTROL WATER INFLOW

Site Selection

A great degree of control to regulate the amount of runoff which enters the tailings pond can be exercised by proper site selection. A tailings structure located on flat ground has no runoff inflow, whereas a cross-valley dam will experience a maximum volume of runoff inflow. A side-hill dam will have a moderate water inflow due to runoff. In cold and wet climates, inflow from direct precipitation, such as rain and snow, will be high, and the water control system has to be designed to handle these peak volumes. In Canada 50 per cent of the precipitation is lost again by evaporation (Bragg, 1973). During the winter months special precautions have to be taken to ensure that the inlet and outlet areas are kept free from ice to prevent overtopping. The excess free water, that develops primarily from the surface runoff or direct precipitation can be handled by several methods as follows:

(1) The excess free water is allowed to flow over an emergency spillway,

(2) The water is directed around the tailings dams through diversion channels, and,

(3) The excess water flows into the pond area but is removed again through a decant system.

Particular care has to be taken to locate the decant outlets a relatively far distance from the slopes of the dams to avoid erosion and piping. Also, the beds of the diversion channels should be lined to minimize erosion. Spillways are generally relocated as the height of the dam increases.

Water Content of Slurry

The amount of water accompanying the solid particles of the tailings depends on the type of the ore mined and the milling process used. Depending on the thickening procedures, the tailings slurry may contain anywhere from 50 to 85 percent water by weight. For example, the Sherman Mine of Northern Ontario, an open pit iron ore mine, uses 150,000 liters of water per minute. Ten percent, or 15,000 liters per minute is pumped to the tailings pond. The remaining 90 percent is reused in the milling process. This pumped tailings contains 50 to 55 percent solids by weight. In special cases, for example, in tailings from oil shales which have been dried for easier handling, the water content may drop to about 20 percent.

Once the site and type of the tailings structure has been selected, the amount of water inflow from the milling process, the surface runoffs and the frequency at which the peak runoffs will occur can be estimated.

4. METHODS TO CONTROL WATER OUTFLOW

Permeability

The subject of permeability is of paramount importance in the safe and economical design of tailings structures. The quantity of water seeping through the soil pores is directly proportional to the pressure gradient, that is, the difference of the water levels of the pond area and the downstream end, divided by the length of the seepage path. This is expressed by the well known Darcy's law

$$Q = ki\,A \qquad (1)$$

where Q = quantity of seepage water
k = coefficient of permeability
i = hydraulic gradient
A = area through which water flows

It is generally agreed that the coefficient of permeability is influenced by the void ratio, grain size distribution and the amount of fines (#200 sieve) of the tailings material.

Figure 1 shows the relationship of the coefficient of permeability to the amount of fines present in the tailings sand. This figure can be used as a first approximation to estimate the volume of water lost through the dyke and/or through the underlying foundation material. Hazen (1892) has established the following empirical relationship between the effective grain size, D_{10}, and the coefficient of permeability, k:

$$k = D_{10}^2 \qquad (2)$$

when D_{10} is expressed in millimeters, k will be in cm per second. In Figure 2 this relationship is presented graphically. Superimposed are the test results of different researchers which indicate a close agreement with Hazen's equation. The above discussion holds true only for completely saturated soils, in the case of tailings dams, these soils will be located below the phreatic seepage line. When it becomes necessary to establish the permeability of partially saturated soils above the water table, the engineer can resort to an infiltration test and the coefficient of permeability can be determined by the method proposed by Palubarinova-Kochina (1962).

Density

The density of the sand mass, though less important than the grain size, has an important influence on the permeability. The denser the soil, the smaller the porosity or the void ratio and the smaller will be the corresponding permeability. Cedergreen (1966) reported that for a particular sand permeabilities varied from 1 to 20 times from the loosest to the densest state. For sands of more uniform particle sizes the permeability is influenced to a lesser degree. The placement density of the embankments are controlled to some degree by the method of construction, such as the upstream, downstream or centre-line methods and construction with imported materials. These methods of construction are illustrated in Figure 3.

Embankment Height

The factor of safety against sliding for a dry cohesionless slope is independent of the height of the embankment. Submergence or seeping water will reduce the stability by one half or more. As a general and wise rule the top seepage line (phreatic surface) should be located away from the downstream face of the dyke. The seepage force, which acts in line with the direction of the flowing water, can cause erosion of soil particles where the seeping water exits the embankment. This toe or slope erosion can lead to piping or tunnelling and eventually to collapse of the tailings structure.

Kealy and Bush (1971) have shown the dependency of the factor of safety against failure with regard to the location of the top seepage line. This relationship is shown in Figure 4. Another reason to locate the seepage or flow lines away from the face of the dam is to decrease the potential against liquefaction as reported by Smith (1969).

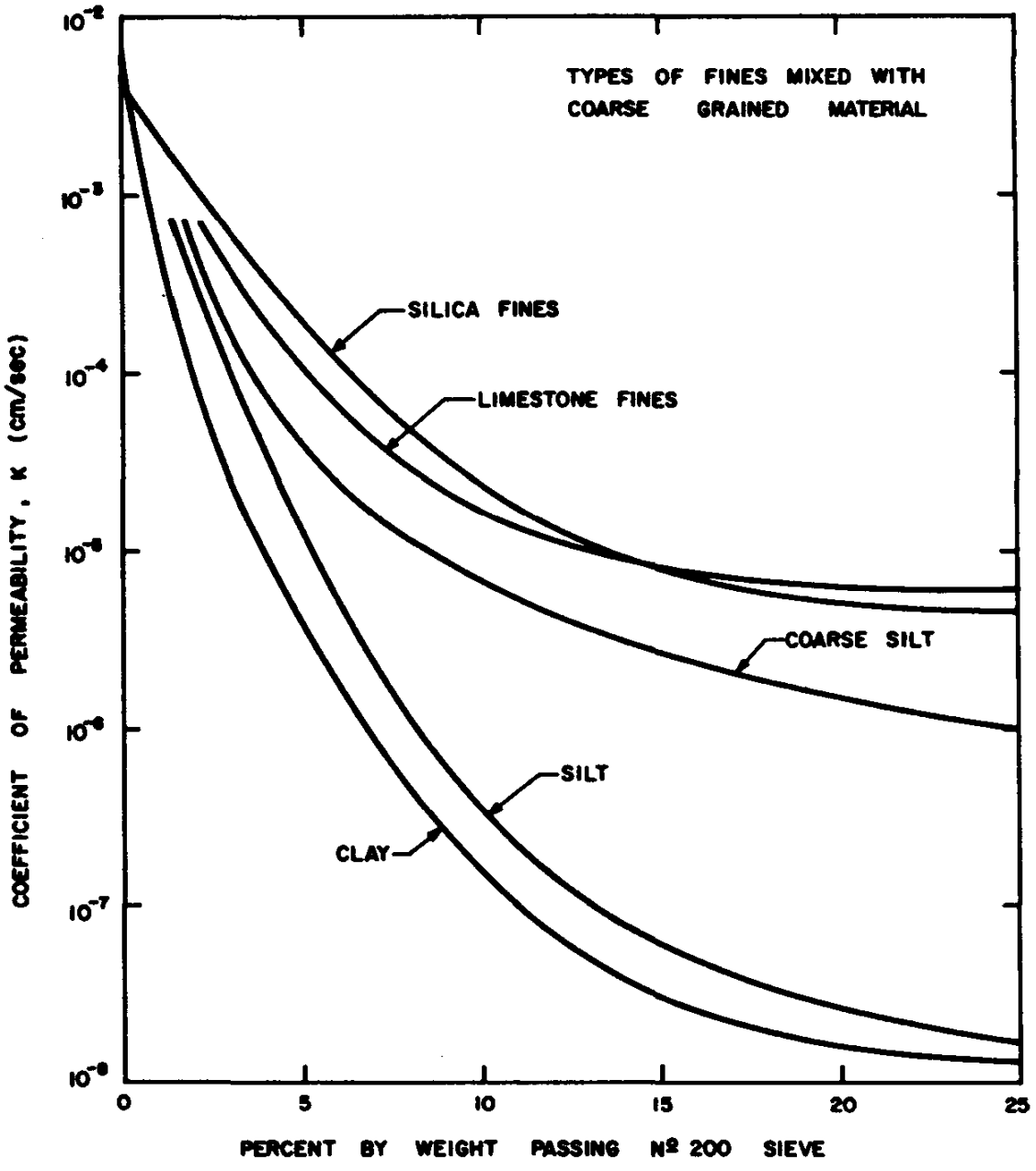


Figure 1. Permeability versus Fines Content

(EMR, 1972)

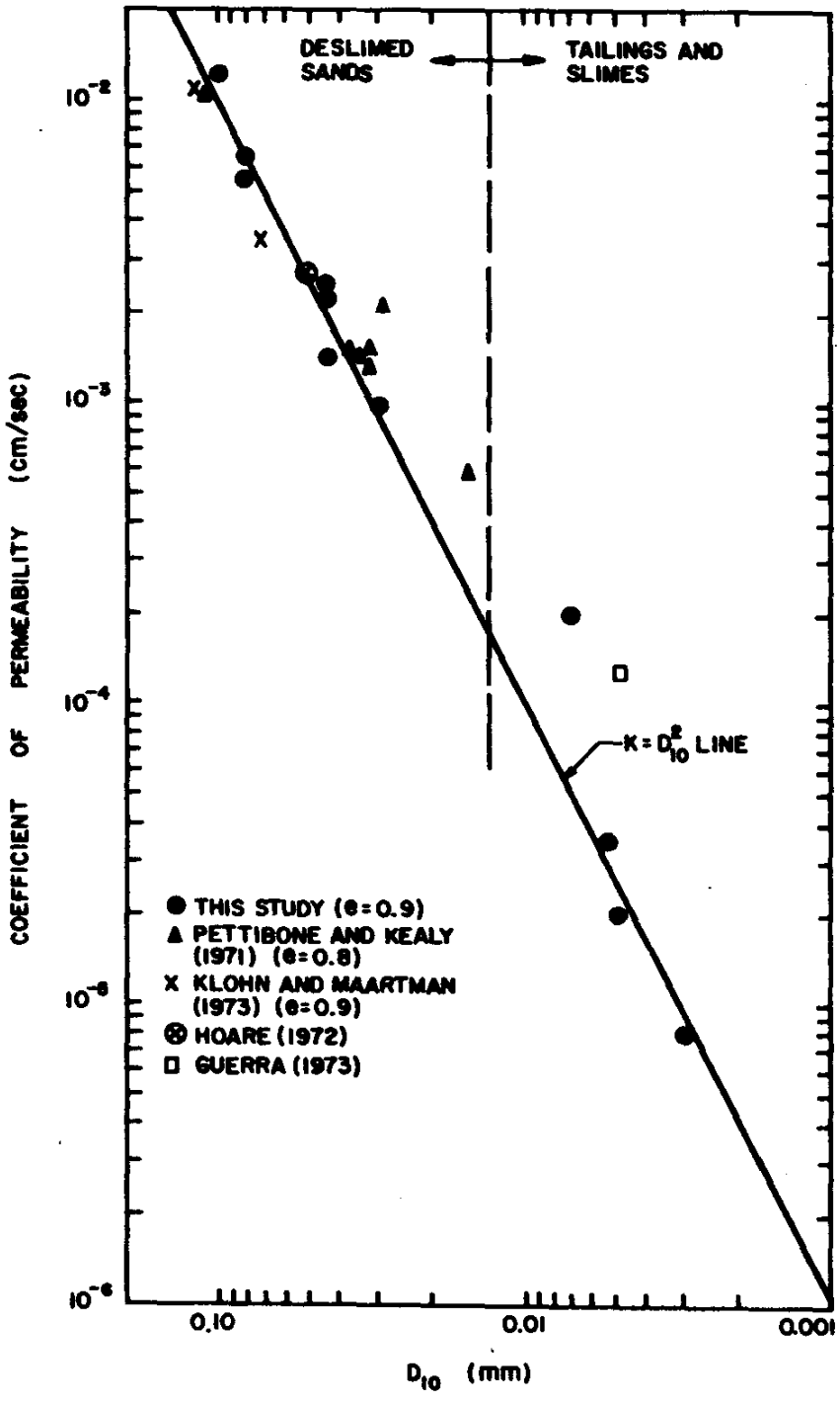
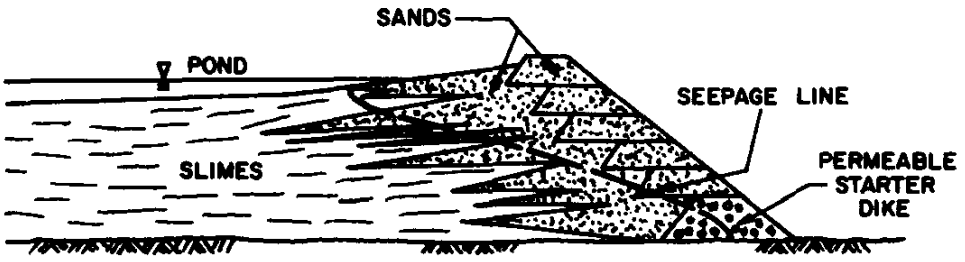
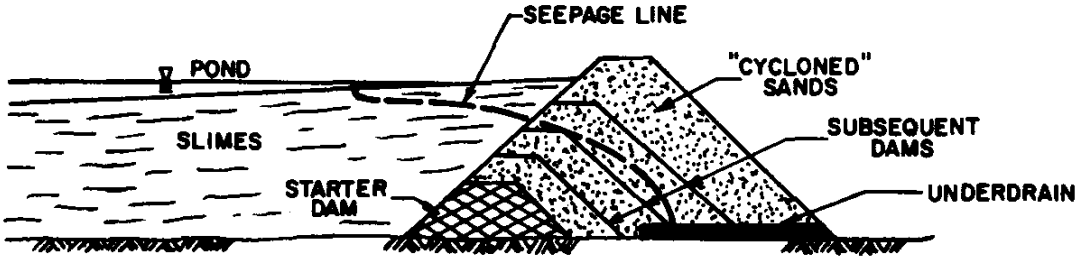


Figure 2. Permeability versus D_{10}

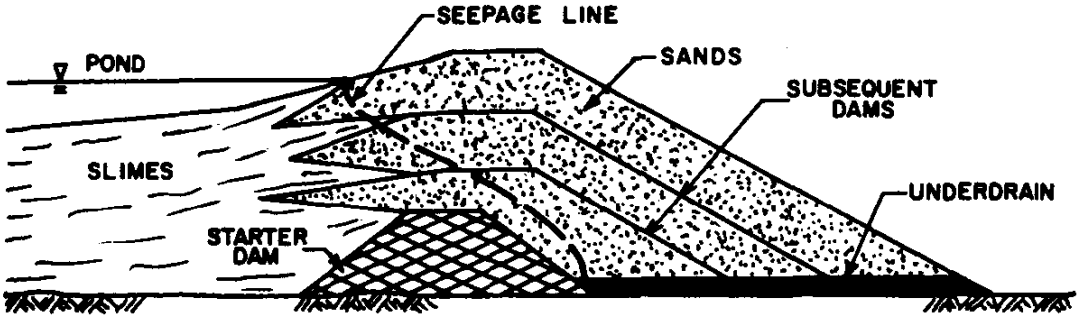
(Mittal and Morgenstern, 1976)



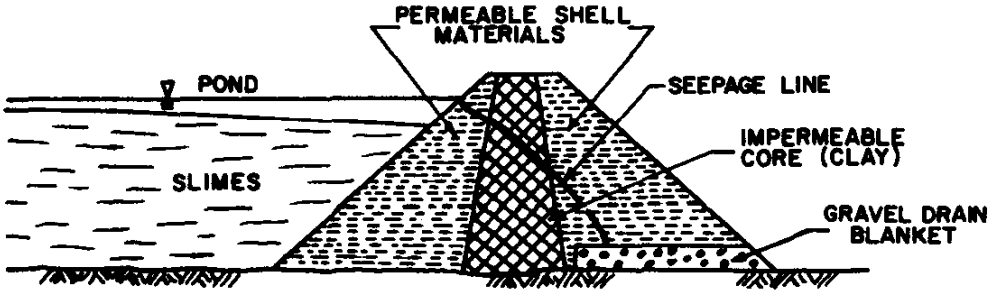
a) Upstream Method



b) Downstream Method



c) Centreline Method



d) Earth Dam Method

Figure 3. Methods of Construction of Tailings Dams

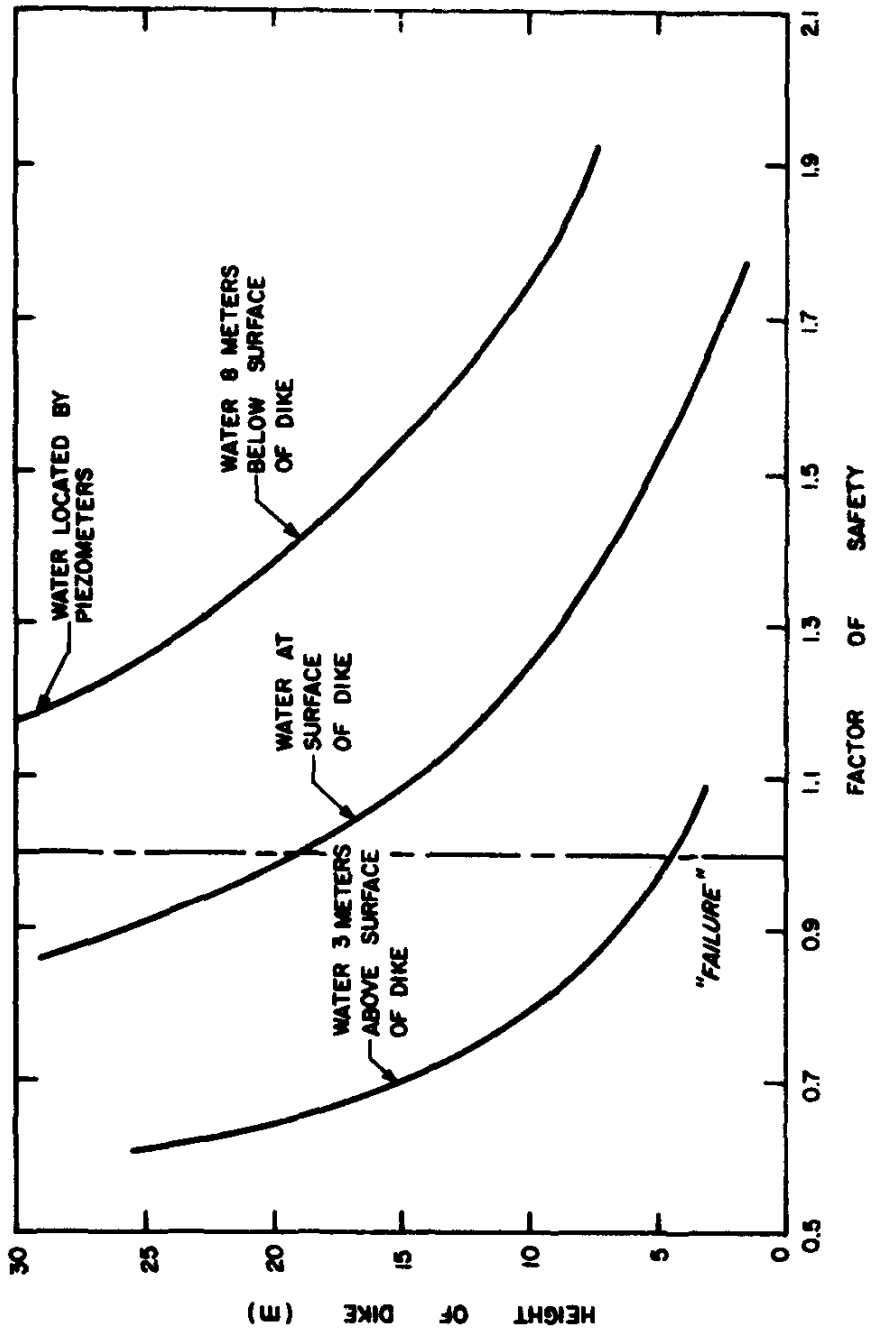


Figure 4. Factor of Safety in relation to Location of Top Seepage Line (Kealy and Bush, 1971)

Drainage Filters

Filter drains can most effectively be incorporated in the construction of the embankment in order to suppress the seepage lines and to effect better control of the water leaving the structure. The grading or sizing of the filter material is of paramount importance in order that the filter fulfills its two main requirements, (1) the filter material should be course enough to prevent any build-up of hydrostatic pressures and to allow easy passage of water, and (2) the material must be fine enough to prevent washing through or erosion of the tailings sand. These two criteria have been expanded by the Department of Energy, Mines and Resources of Canada (1972) into the following six rules:

1. $\frac{\text{The 50\% size of the filter material}}{\text{The 85\% size of the protected soil}}$ should be less than 5

2. $\frac{\text{The 50\% size of the filter material}}{\text{The 50\% size of the protected soil}}$ should be less than 25

3. The filter material should be smoothly graded; gap graded materials should be avoided.

4. $\frac{\text{The 15\% size of the filter material}}{\text{The 15\% size of the protected soil}}$ should be greater than 5

5. The filter material should not contain more than 5 percent of particles, by weight, finer than the No. 200 sieve, and the fines should be cohesionless.

6. The coefficient of uniformity of the filter material should be equal to or greater than 20.

These requirements are more readily illustrated in Figure 5. The location of the filter drains will dramatically affect the position of the seepage lines as shown in Figure 6. Very seldom is it economically justified to prevent seepage through the tailings dam completely. Reduction of the amount of water through the dyke by grouting, the use of slurry trench walls and cutoff curtains have been used, but if impervious materials, such as clays and silts are readily available at the site, they could be used advantageously to construct an impervious upstream blanket or an impervious core. The application of clay and bentonite to seal tailings ponds has been described by Monea (1977). For environmental considerations, a secondary dam can be constructed downstream of the main tailings dam to impound the outflow from the main dyke. This water can be treated, if necessary, and then returned to the milling plant for reuse.

5. INSTRUMENTATION

Most problems which occur in tailings structures are due to seepage forces and liquefaction. Therefore, as a requirement to a safe performance of the structure it should be made compulsory to monitor the water pressures within and underneath the dam. Installation of simple piezometers to monitor water pressures and reference point to observe horizontal and vertical movements, can provide valuable information during the life of the tailings dam. Hardy (1974)

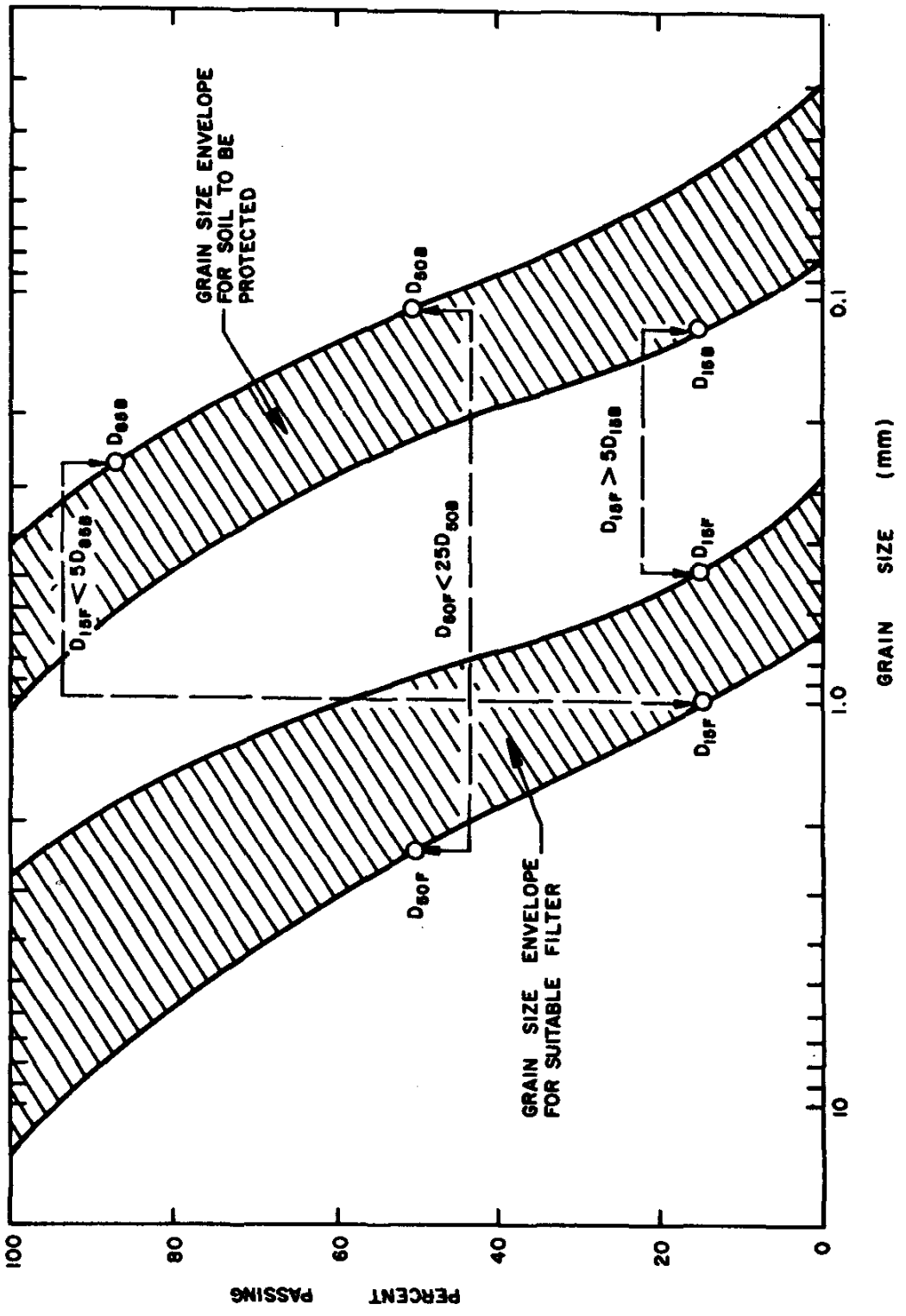
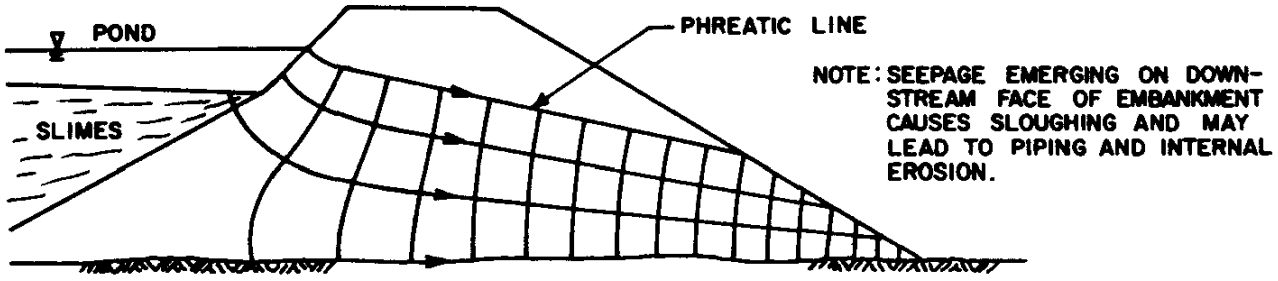
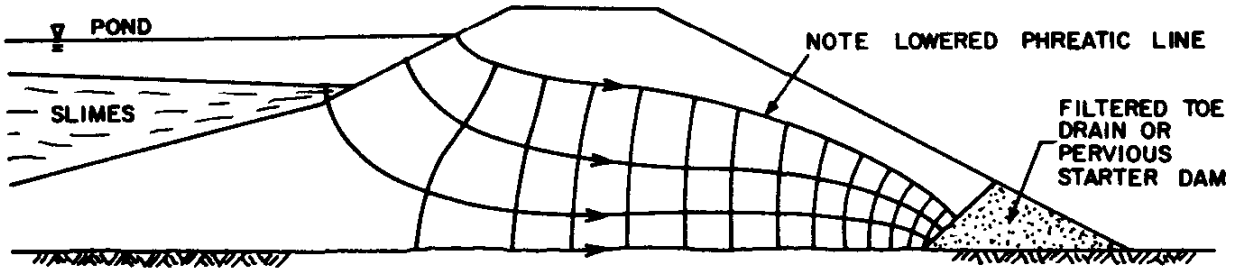


Figure 5. Gradation Requirements for Drainage Filters

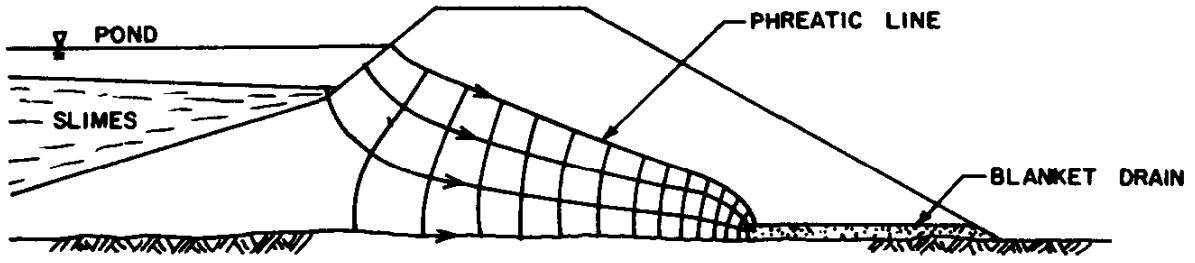
(EMR, 1972)



(a) No Drainage Filter



(b) Toe Drainage



(c) Blanket or Under Drainage

Figure 6. Effect of Drainage Filter on Phreatic Line

reported that 75 piezometers were installed in the 70 m high tailings dam at Great Canadian Oil Sands since its construction was started in 1967 and that they are frequently read in particular before a decision to raise the height of the dyke is made, these monitoring results are reviewed.

In the upstream method of construction some of the material for the successive dykes will be placed over the saturated slimes. This could generate high enough porewater pressures to cause localized failure of the underlying material. The placement of piezometers in the susceptible material could dictate the rate of construction in order to keep the porewater pressures within safe limits.

Since most of the coarser tailings material is quite pervious, simple and inexpensive non-corrosive piezometers, such as the Casagrande type, are quite adequate to monitor the water pressures and seepage quantities. These records could either be used to modify the construction procedures or to help in future design considerations of tailings structures. Also, observation data are frequently used to prove or disprove statements in conflicts or lawsuits.

6. CONCLUSIONS

Proper control of water flowing into and out of tailings dams is absolutely necessary to guaranty the safe performance of these structures. The design engineer has a number of control devices, as discussed in this paper, at his disposal and the decision which ones to use depends on many factors, the two most important ones are cost and concern for the environment.

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GROUND WATER PROBLEMS IN THE MINING DISTRICT OF IGLESIENTE (SARDINIA, ITALY)

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ABSTRACT : In Iglesiente (SW Sardinia) there is an important orebearing district. The ores occur in the carbonate rocks of the Lower Cambrian. The Cambrian sequence was folded and faulted by the Caledonian, Hercynian and Alpine orogenies, with remobilization of the mineral deposits and the development of various karstic cycles. Large quantities of ground water have posed problems for mining. At present mining extends down to - 300 m. The 1500 l/s pumped off at present are slightly briney. Research is now underway to allow deepening of the mines and of the water-table level.

RESUME : A Iglesiente (Sud-Ouest de la Sardaigne), il y a un important district métallifère constitué surtout de gisements sulfurés encaissés dans des roches carbonatées ("métallifère") du Cambrien inférieur (Formation de Gonnesa). Les formations cambriennes, qui constituent l'essentiel des terrains d'Iglesiente, ont été affectées principalement par des phases de plissement de l'orogenèse hercynienne et alpine. Par conséquent des phénomènes de remobilisation et de reconcentration de la minéralisation préexistante dans des fractures et cavités karstiques sont très répandus. L'exploitation minière a rencontré une quantité d'eau considérable. Actuellement, à - 300 m, il faut pomper 1500 l/s d'eau en partie salée. Des études sont en cours pour approfondir les travaux miniers et par conséquent le niveau de pompage de l'eau.

RESUMEN : En Iglesiente (SW de Cerdeña) existe un importante districto metalífero, constituido sobre todo por yacimientos sulfurados que arman en rocas carbonatadas del Cámbrico inferior (formación de Gonnesa). Las formaciones cámbricas, que integran la mayor parte de las formaciones de Iglesiente, se han visto afectadas, esencialmente, por fases de plegamiento correspondientes a la orogénesis herciniana y alpina. En consecuencia, son muy frecuentes los fenómenos de removilización y reconcentración de la mineralización preexistente, en fracturas y cavidades kársticas. La explotación minera ha encontrado cantidades considera-

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bles de agua. Actualmente, a -300 m, hay que bombear 1500 l/s de agua (en parte salada). Se están realizando estudios para profundizar las labores mineras, y por consiguiente el nivel de bombeo del agua.

1 - INTRODUCTION

Southwestern Sardinia is the site of Italy's most important Pb-Zn ore-bearing district. It has been estimated that this district contains 2.5 million tons of metal, and at least twice as much has already been mined.

In Arburese (north of Iglesiente) there are the hydrothermal veins of the Montevecchio - Ingurtosu - Gennamari area, which branch off from the granite stocks of Arbus and are surrounded by the Silurian and Carboniferous (= Postgotlandian) slates. The stocks are part of the Hercynian Sardinian batholith.

In Iglesiente, there are sedimentary strata-bound mineral deposits in the Cambrian carbonate rocks (Fig. 1). The main group of mines is found in the Iglesias Valley: Monte Agruxiau, Monteponi, Campo Pisano, Genna Luas, San Giovanni, Monte Onixeddu and Barega. To the west and northwest of Iglesias, there are the mines of Nebida, Acquaresi, Masua, Buggerru and Candiazzus, while to the east and northeast there are the mines of Arenas, Tiny, San Benedetto and Reigraxius.

In Iglesiente, mining goes as deep as 300 meters below sea level, where the problem of ground water is a serious one. During a century of pumping off ground water, a great deal of information on the subject has been gathered.

However, it has not been enough to unravel the complex hydrogeological situation of the zone. Studies at present underway aim to clarify the problem in order to permit deepening the mines, which are now in serious difficulties.

These studies will certainly increase our knowledge of the hydrogeology of carbonate rocks as well as contribute to the development of techniques for digging galleries in the freatic zone and for pumping off ground water.

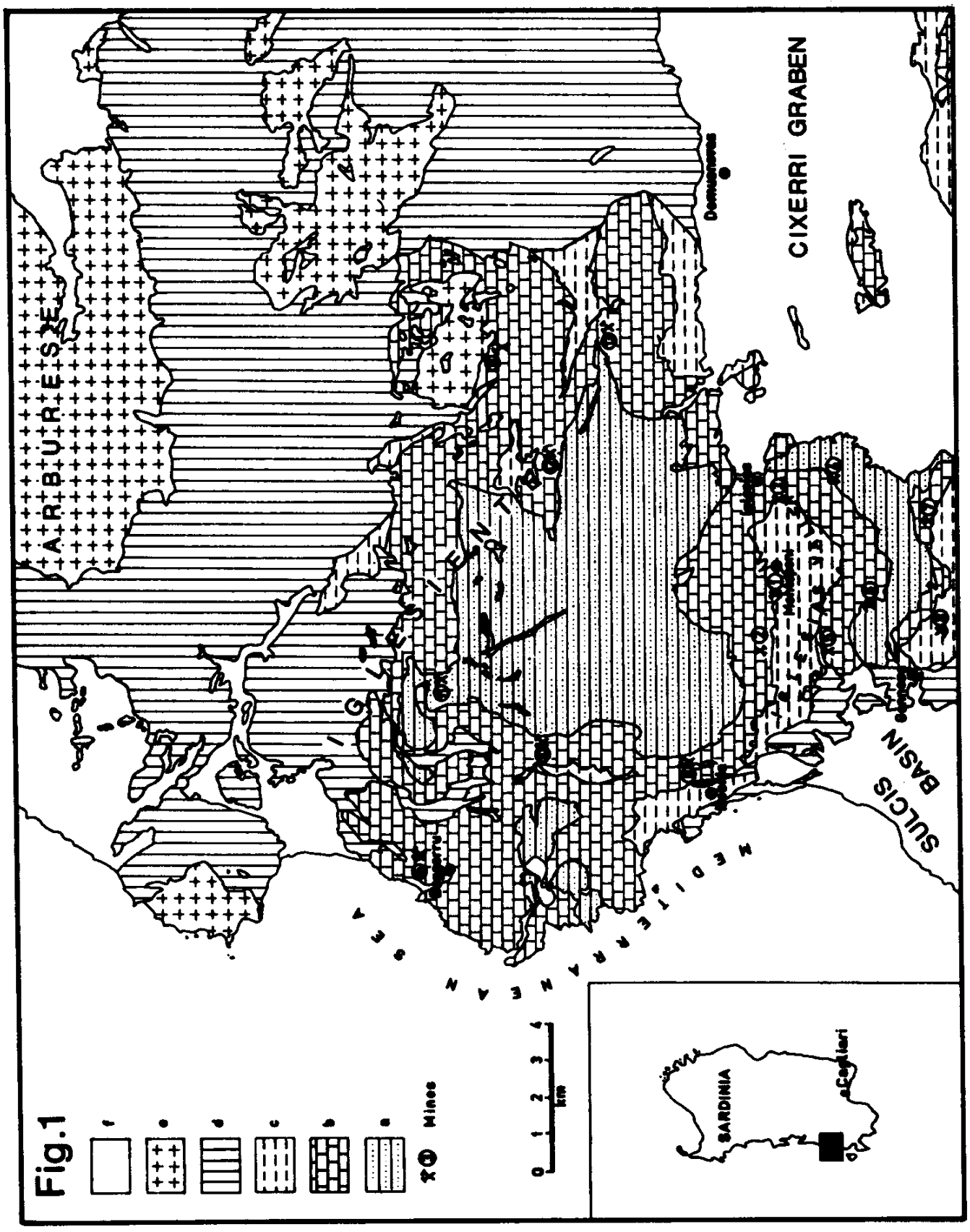
2 - GEOLOGICAL SETTING

The ores of southwestern Sardinia (Iglesiente and Arburese), which have been mined for centuries, occur in the Cambrian sediments.

The Cambrian succession, whose base is not exposed, attains a maximum thickness of about 2000 m and consists of three formations (Fig. 2 and Table 1). From bottom to top, they are:

i) Nebida Formation ("Arenarie" AUCT.): trilobite-bearing shallow-water sandstone, well bedded, with archeocyathid-bearing limestone lenses and dolomite beds in the upper part (known as "alternanze") of the Lower Cambrian.

The "alternanze" beds grade into the overlying formation.


CIXERRI GRABEN
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km

Fig.1

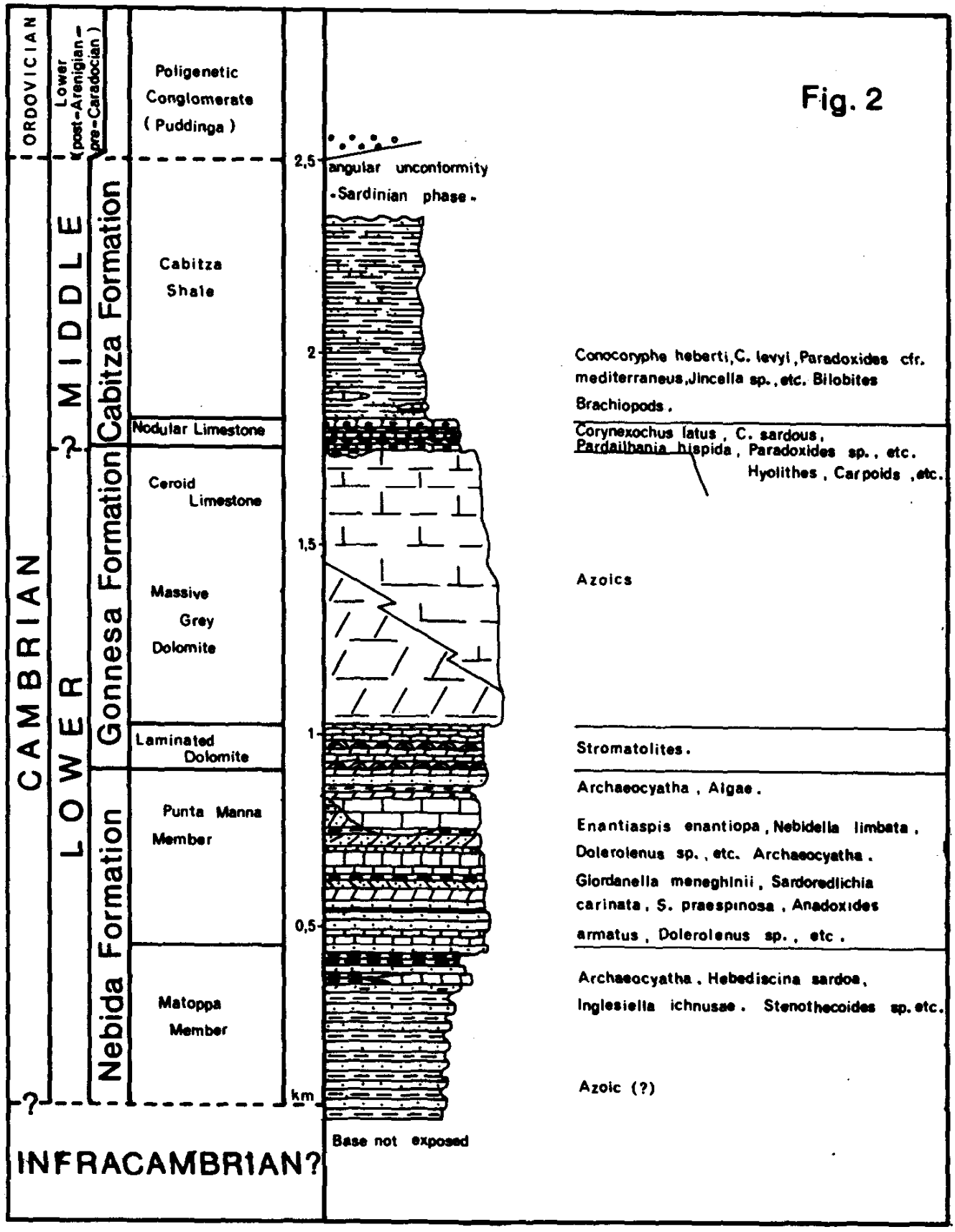
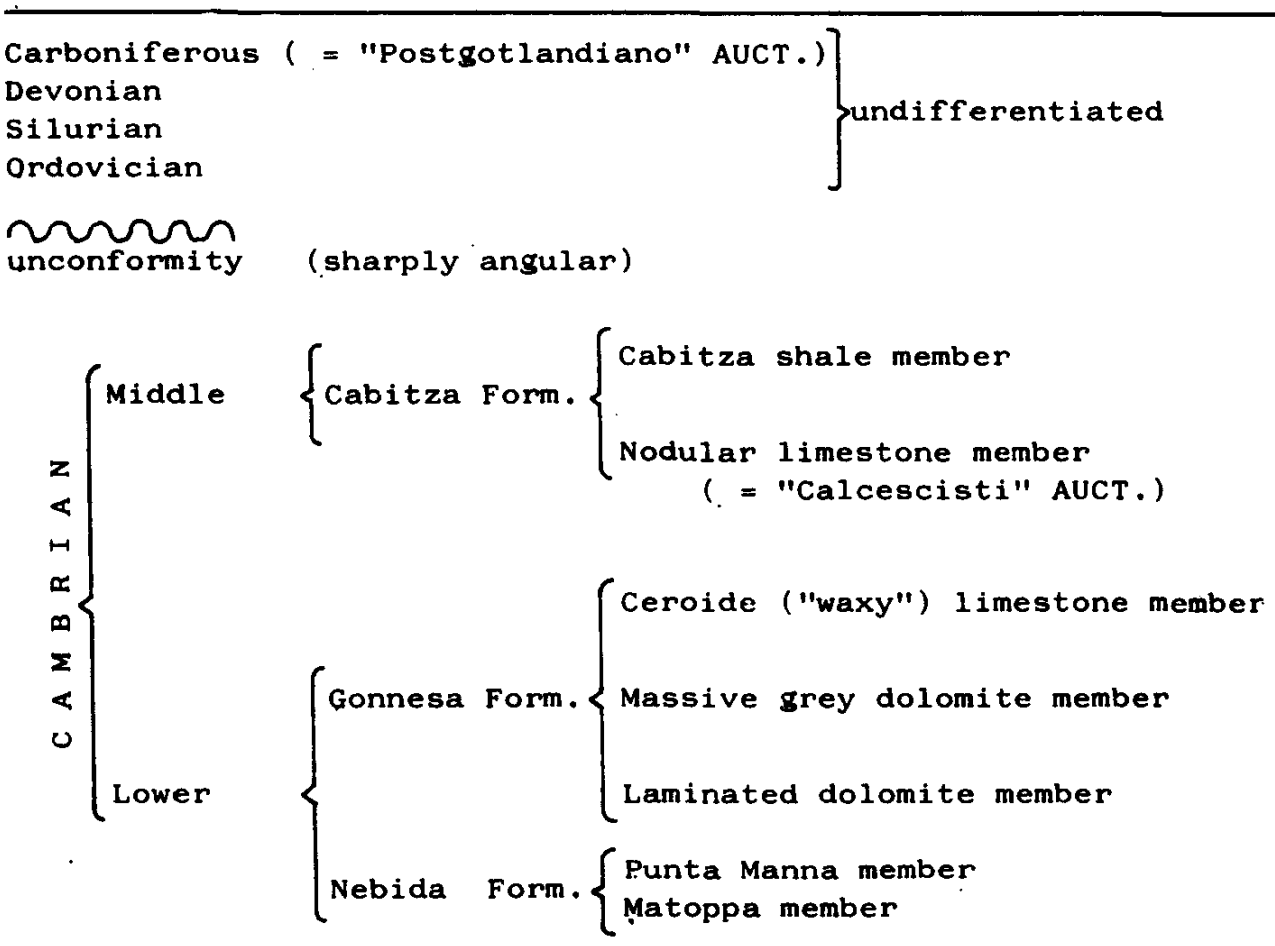

ORDOVICIAN
Lower (post-Arenigian - pre-Caradocian)
CAMBRIAN
MIDDLE
LOWER
?
?
Poligenetic Conglomerate (Puddinga)
Cabitza Shale
Cabitza Formation
Nodular Limestone
Ceroid Limestone
Massive Grey Dolomite
Gonnesa Formation
Laminated Dolomite
Punta Manna Member
Matoppa Member
Nebida Formation
2,5
angular unconformity
-Sardinian phase-
2
1,5
1
0,5
km
Base not exposed
INFRACAMBRIAN?
Fig. 2
Conocoryphe heberti, C. levyi, Paradoxides cfr. mediterraneus, Jincella sp., etc. Bilobites
Brachiopods.
Corynexochus latus, C. sardous,
Pardailhania hispida, Paradoxides sp., etc.
Hyolithes, Carpoids, etc.
Azoics
Stromatolites.
Archaeocyatha, Algae.
Enantiaspis enantiopa, Nebidella limbata, Dolerolenus sp., etc. Archaeocyatha.
Giordanella meneghinii, Sardoredlichia carinata, S. praespinosa, Anadoxides armatus, Dolerolenus sp., etc.
Archaeocyatha, Hebediscina sardoa, Inglesiella ichnusae. Stenothecoides sp. etc.
Azoic (?)

Table 1



ii) Gonnesa Formation (known as "metalliferous" because of its lead and zinc sulphide and barite ore deposits): a thick succession of mainly biochemical (but also chemical) azoic dolostone and limestone of Lower Cambrian age. This formation was deposited in a shallow-water environment varying from subtidal-supratidal to shelf lagoon. The contact between the limestone and dolostone ranges from gradational to sharply defined. In the Trias, after the post-Hercynian peneplanation, a widespread surgene yellow dolomite formed as a surface phenomenon.

iii) Cabitza Formation: overlying the Gonnesa Formation in local unconformity and composed of nodular limestone with trilobite, carpoid and hyolithid fragments at the base, grading

upward into a series of trilobite-bearing shales and siltstones. At about the end of the Middle Cambrian, deformation and uplift were brought about by the first movements (Sardic phase) of the Caledonian orogeny (or late movements of Assintic orogeny ?). No sedimentation occurred in the Upper Cambrian. After a long time-lapse, at roughly the Arenigian-Llanvirnian, the sea began to flow back over most of this eroded landscape. The Ordovician conglomerate ("puddinga"), sandstone and slate, is overlain with graptolite-rich black shale and lenses of reticulate limestone rich in cephalopod and other fossil remains of Silurian age. The black slate and limestone of the Devonian are poorly represented, and a complete stratigraphic sequence of the Devonian has not been found anywhere. The relationship of the Devonian with the underlying Silurian (also generally incomplete) is unclear because of the fragmentation of the outcrops and their truncation at various stratigraphic levels, probably due to the action of Hercynian imbricate structures.

Other Caledonian movements probably occurred between the Ordovician and Silurian and perhaps between the Silurian and Devonian. However, they are essentially pre-Caradocian epeirogenic movements with the extrusion of basic volcanic rocks in Iglesiente and also large quantities of acid volcanic rocks in the other Sardinian regions The major phase of the Hercynian orogeny involved all pre - Westphalian sediments including the so-called "Postgotlandian" (Lower Carboniferous in age) that underwent a more or less intense metamorphism, generaly increasing from the southwest to the north-east of the island. This formation unconformably overlies the older terrains down to the Ordovician. The contact therefore suggests a probable overthrust or other Hercynian structures. During the post-Hercynian distensive phase, and probably after an isothermic syntectonic uplifting, many plutons of the Sardinian batholith were emplaced, such as the intrusive granitic cupolas and dikes of Arburese.

In the Upper Carboniferous - Lower Permian, lacustrine basins formed in the depressions of the mountains, where erosion products accumulated (S. Giorgio basin, near Iglesias). The continental phase of the island, continuing into the beginning of the Mesozoic, involved the peneplaining of large areas.

In the Middle Trias, lagunal and evaporitic conditions obtained in the plains close to the sea, and thin layers of germanic facies were deposited (Campumari plateau, east of Iglesias).

It was not until the beginning of the Middle Eocene that the sea began to cover the land again. But even so, it was only for a

short while and only affected the margins of the land (Sulcis basin, southwestern part of the island).
In contrast with what happened during most of the Paleozoic, during the Mesozoic, Iglesiente, along with the rest of the island, behaved as a relatively stable plateau subject only to minor epeirogenic movements.

3 - MINERAL DEPOSITS

Most of the Pb-Zn and Ba is in the upper part of the carbonate sequence of the Gonnesa Formation; much less is in the lower part (Fig. 1). Fe ores are quite widespread in the laminated dolomite member.
The primary mineral deposits are synsedimentary and are thought to derive from the weathering of a pre-Cambrian crystalline relief, probably situated to the west-northwest of present-day Sardinia. The richest orebodies are the product of mobilization and reconcentration along fractures and karst cavities in the carbonate sequence, which were formed during various orogenic phases. The resulting orebodies are in columns and irregular masses in the upper part of the "metalliferous" formation (Monteponi, Acquaresi, Nebida-Masua and San Giovanni). In the lower part (Candiazzus, Genna Luas and probably Campo Pisano) pyrite and blende predominate mostly in lens-shaped bodies and, to a lesser extent, in irregular ones.
A further type of depcsit is the result of the filling of karst cavities with both oxidized ores and barite and silver-rich galena (Arenas, Barega, "Ricchi argento" in San Giovanni mine).
These orebodies vary greatly in size, sometimes exceeding a million tons of ore with 4 to 8% Zn-Pb.

4- KARSTIC FEATURES

During its long geological history, Iglesiente was involved in various karstic cycles, brought to light by geological studies of the surface and by mining activities.
Ist cycle. It corresponds to a period of local emersion that took place between the deposition of the "metalliferous" limestone and that of the Cabitza shale. Its traces consist in local erosion pockets filled with breccia and ores, but the karst features are poorly developed. During the deposition of the nodular limestone in some places (e.g. in the +8 level drainage tunnel at Monteponi mine), an emersion occurred with the formation of small cavities and breccia.

2nd cycle. It is related to the movements of the Sardinian phase, way before the conglomeratic Ordovician transgression. It is characterized by cavities of various sizes that are almost always filled by breccia and silica "hard ground" of the Arenas mine type (known as "quarzite").
3rd cycle. Mainly during the Permian-Trias, the weathering of the partially emerged carbonate sequence produced karst cavities with epigenetic dolomitization ("yellow dolomite"). The cavities were subsequently filled (Trias) with collapse breccia. The breccia is highly silicified and contains barite and silver-rich galena in its cement. Some of the ore-filled cavities are eroded and covered by transgressive conglomeratic sediments with barite and silver-bearing galena in the cement (Sa Bagatta deposits, cf. BRUSCA et al., 1967).
4th cycle. Large vertical fissures opened, were widened by karstification and filled by conglomerates, coarse-grained sand and Eocene carbon ("Moken slate" at the - 100 m level of Monteponi mine).
5th cycle. Present-day karstification with the formation of both horizontal and vertical cavities along fractures and, sometimes, old karst structures. Mines have run into large active caverns in the upper levels, like the Santa Barbara cavern (167 m above sea level) in San Giovanni mine and the Gran Sorgente (6.5 m above sea level) to the west of Monteponi along the drainage tunnel.
Downwards the number of the karst features diminishes, and at -100 m the gound water flows in fissures of previous karst cycles. Superficial karst (whether macro-or microform) is slight in the Cambrian carbonate reliefs. This is due, on the one hand, to the structural situation (subvertical strata) and, on the other, to the widespread presence of primary dolomite and the extensive secondary dolomitization (yellow dolomite) related to the pre-Triassic peneplanation. Instead, karstic caverns and wells are especially common in the Cambrian limestone northeast of Iglesias.

5 - HYDROGEOLOGICAL FRAMEWORK AND RESEARCH

The Cambrian calcareous and dolomite sequence (Gonnesa Formation), in which the mineral deposits are located, is also the main aquifer of the district. High ingestion and seepage capacity characterize the aquifer, although the seepage capacity varies widely with the degree of fracture and the karst ratio.
The complicated geological structure of Iglesiente affects the hydrogeological structure: the carbonate aquifer is almost

everywhere bound by other less permeable elements of the Cambrian and post-Cambrian sequences. Thus the carbonate complex contains a high-yield hydric network, the original water table of which was soon reached by mining excavation. In fact, the Monteponi mining activities, which date back to the Carthaginian period, had already reached the water table at the time of the Pisan miners during the 13th and 14th centuries, but the first reliable information is of a mine shaft running into a slight yield of ground water at 70 m above sea level in 1870. This was kept under control with small pumps. In 1874, at 43 m a.s.l. a mine gallery under construction ran into a fracture filled with water under pressure. The water table rose again to +70 m and it was necessary to pump 3C0 l/s just to draw the water table down to +61.5 m. In order to solve the ground water problem, it was decided to dig a drainage tunnel about 5 km long, starting at +3 m near the sea and running towards Monteponi mine at +13.5 (Fig. 1.3, 4 and 5). The tunnel began in the impermeable Silurian conglomerates and Cambrian shale, and after 4161 m, cut across the contact with the Cambrian carbonate rocks. The ensuing leak, initially yielding 380 l/s, lowered the ground water level to +49.5 m in a year.

Excavation of the tunnel was then resumed, and, after another 103 m, it ran into a karst cavern with an initial yield of 2400 l/s, which diminished to 1400 l/s after 5 months. After three years, the water table at Monteponi had settled to +13.5 m with a yield of 1000 l/s. The water table had also been lowered in all the other mines of the basin, following the drawndown cone resulting from the new base level. Subsequent drawdown was obtained by lower and lower pumping plants: at - 15 m in 1927, at - 60 m in 1935 and at - 100 m in 1955. The yield is now 1350 l/s at Monteponi, where the main plant is located, with an additional 42 l/s drawn off at Masua, 100 l/s at Campo Pisano, and 22 l/s at San Giovanni.

At present the ground-water level has settled at -100 m at Monteponi, - 72 m at San Marco, -45 m at Nebida sud, - 80 m at Campo Pisano, - 79 m at Seddas Moddizzis, and - 80 m at San Giovanni. Mining activities are nevertheless possible below the water table in unfractured zones without a large water yield: the B shaft of Monteponi is at - 150 m, the main shaft at Campo Pisano is at - 120 m, and mining proceeds at - 94 m at Seddas Moddizzis, -150 m at Masua, and - 300 at San Giovanni. This is evidence of the unisotropy of the aquifer and of its considerable subdivision. In fact, it should be noted that despite the long period over

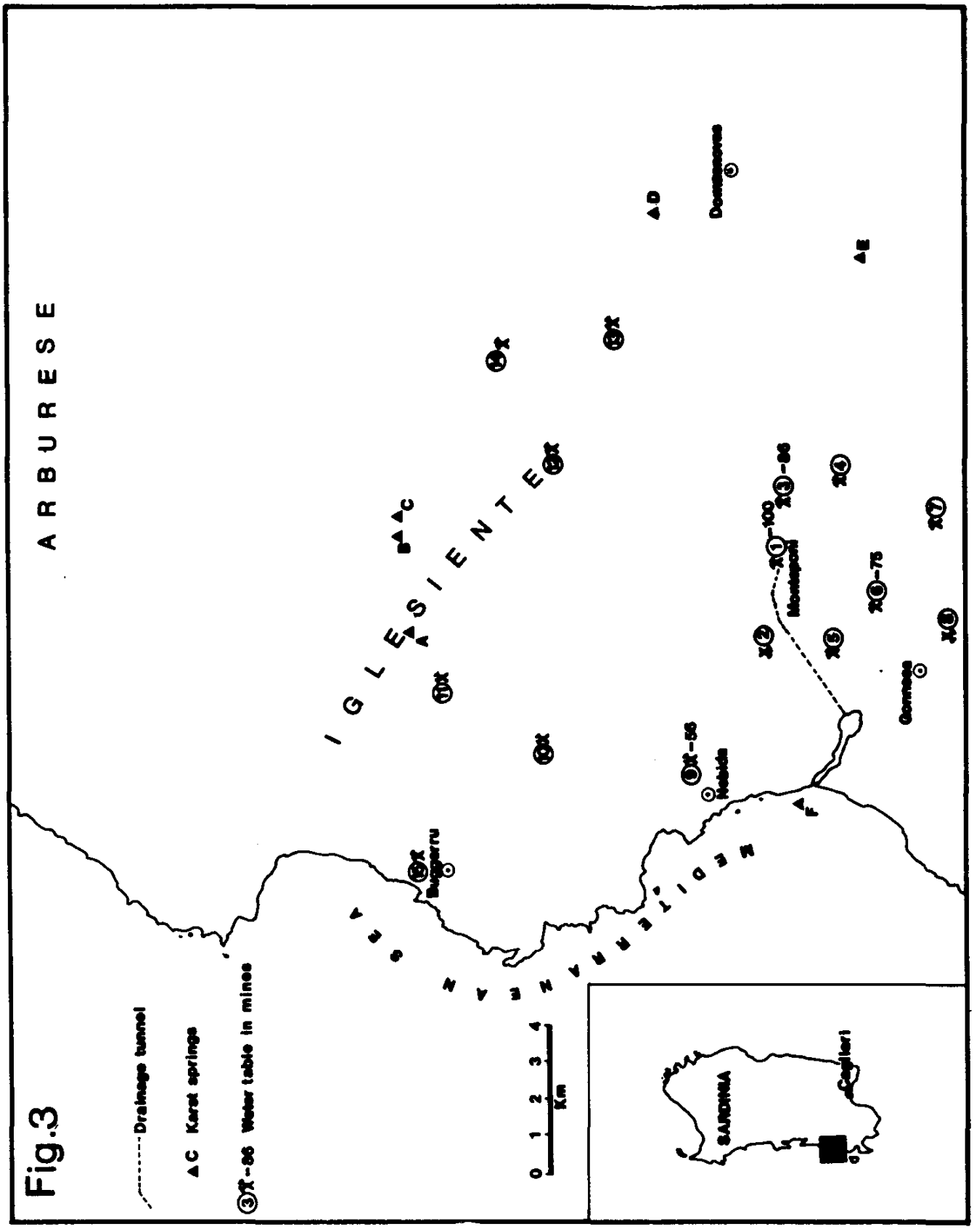

ARBURESE
IGLESIENTE
Drainage tunnel
Karst springs
Water table in mines
Km
SARDINIA
Cagliari

Fig.3

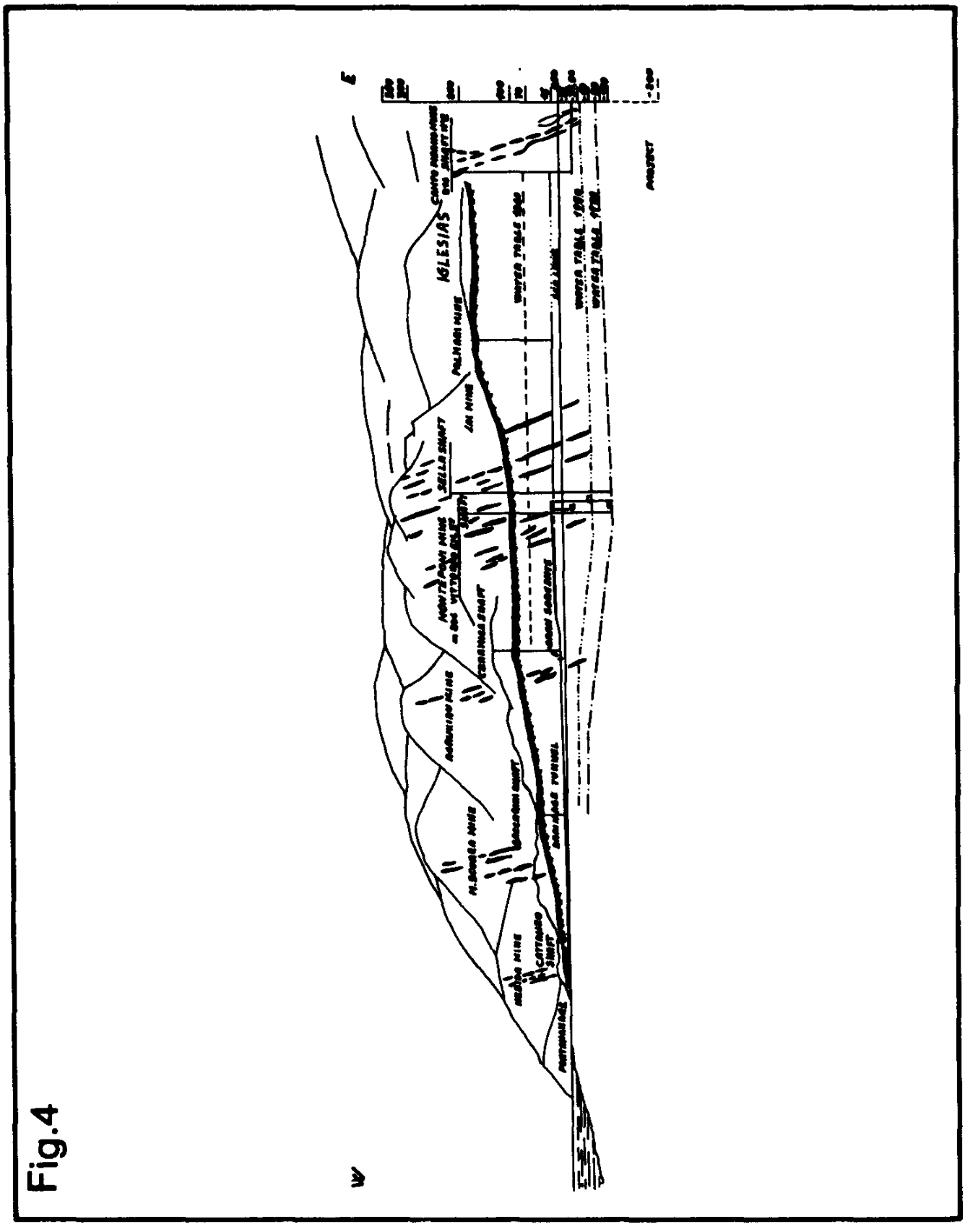


Fig.4

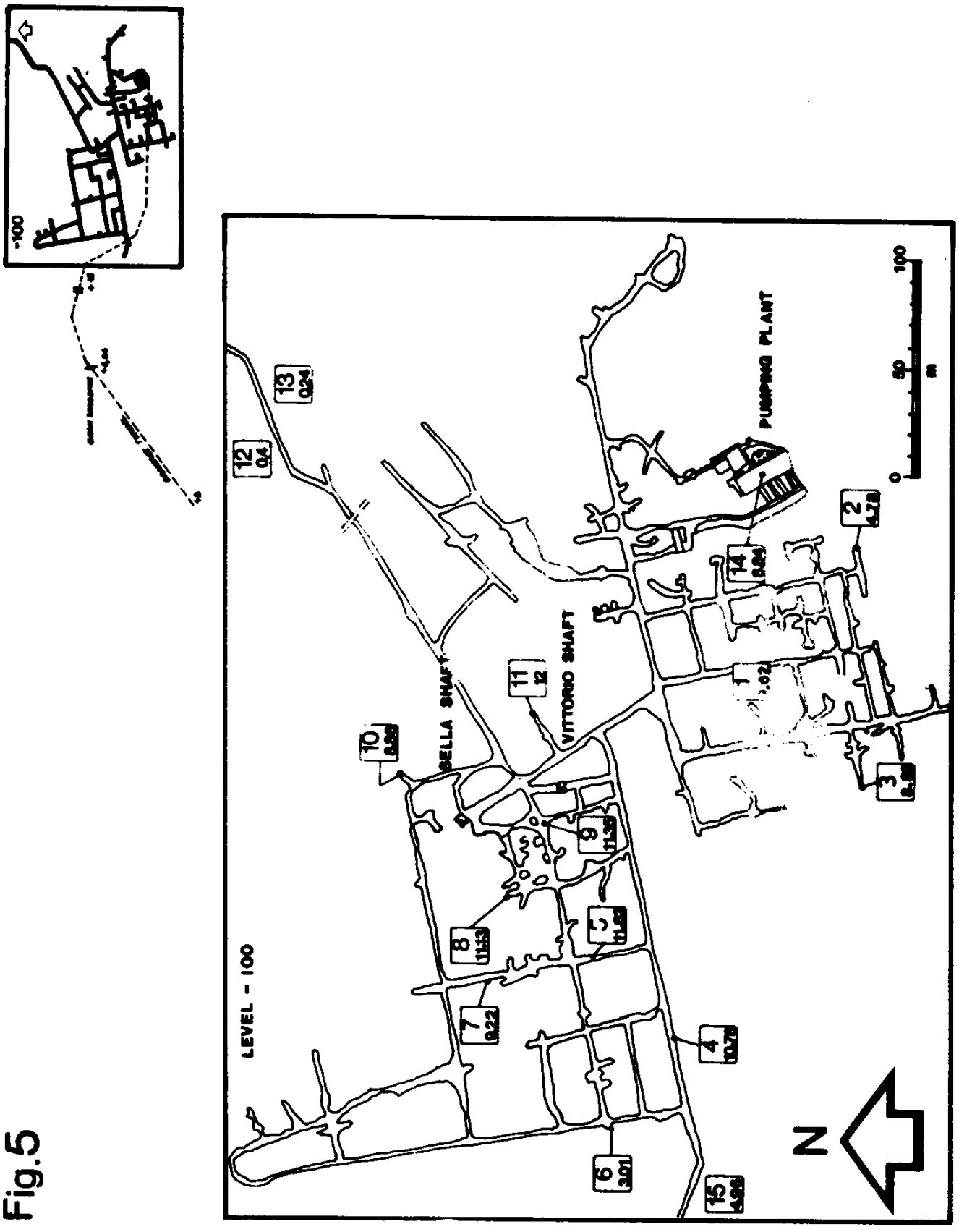

-100
LEVEL - 100
BELLA SHAFT
VITTORIO SHAFT
PUMPING PLANT
N
0
50
100

Fig.5

which the hydric network has been drawn down, yield has not diminished and the springs of the zone have not disappeared. Some of the springs (definitely karst-fed) have a substantial yield. A submarine spring at Fontanamare, near Nebida, is the only one to have dried up after the drainage tunnel was constructed.
The water pumped out at Monteponi has a high concentration of Cl^-, while the salinity of the leaks in the mine varies greatly from one to another, ranging from fresh to highly chlorinated water (Fig. 5). The NaCl content of the drainage-tunnel water was 0.13 g/l in 1889 and had risen to 0.4 g/l in 1898. The leaks at - 60 m and - 100 m have been analysed for their Cl^- over a long period of time. Despite irregularities, the general tendency is for the Cl^- to increase as the depth increases and with the passage of time.
The present need to further lower the water table in order to mine the underlying thick zone of mineral deposits has inspired the complex hydrogeological study involving the entire potential intake area of the hydric network, which is now underway. The edges of that area are almost everywhere marked by structural permeability sills formed by the juxtaposition of the aquifer with aquiclude layers of the Cabitza Formation and other Paleozoic formations. But, in some zones, the position of the boundary is not so clearly marked: the ground watershed east, north and northwest (seawards) of the metalliferous district is still undetermined.
In addition to defining this boundary, which is essential to the estimate of the hydric potential of the aquifer, the study aims to investigate the source of the ground waters, their quality and their hydrodynamics in order to be able to plan adequate pumping plants at -200 m and - 300 m.
The first phase of the study, which is already well along, includes a complete review of the vast accumulation of past data regarding metereology, piezometry, hydrometrical gauging, hydrogeochemistry, etc.; a detailed geological survey for the purpose of clarifying the paleogeography, stratigraphy and tectonics of the entire area, in order to reconstruct existing hydrogeological structures; a complex program of hydrogeochemical and isotopic analyses to clarify the origins and age of the pumped waters; and lastly, the elaboration of all the information in one or more digital models in order to reliably calculate all the parameters involved in the recharge-discharge system of the aquifer (CIVITA, 1975; CELICO et al., 1978).
The drain-off data that has been collected over 100 years of

pumping has been plotted and tabulated. A model based on the calculation of the specific drawdown (CHAUMET, 1965; ROCHE, 1970) has led to the estimation of the regimen drain-off yield at -200 m as 1750 l/s.
A period of 18 months has been calculated as sufficient to lower the water table of the hydric network from - 100 m to - 200 m, with an initial drain-off yield of 2500 l/sec.
The second phase of the study will be carried out during the excavation of service and pumping shafts and the pump chamber. Direct and indirect methods will be employed to detect water - bearing fractures and avoid water leakage during the digging of tunnels.
Once the pumps have been installed, tunnels will be dug to cut across the principal fractures and begin the pumping. The information gathered during this phase will be used to study the feasibility of a drain-off plant at - 300 m.
The shafts and the pump chamber, which will be below the water table at pressures as high as 10 atmospheres, will be excavated in a zone of compact limestone ("waxy limestone") without major fractures.
During the third phase of hydrogeological research, after employing various methods including geophysics to calculate the flow direction of the ground water towards the mining district, an attempt will be made to tap ti before it reaches its destination in order to utilize the fresh water to increase the water supply of the region, which is poor in water. Hydraulic techniques will be used to diminish the seepage of the run-off water that gets lost in the fractures of the carbonate formation.

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EL DESAGUE EN LA MINERIA ESPAÑOLA

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ABSTRACT : This paper covers the draining systems in the spanish mining. For that purpose the study is based on the most important mines, both the underground, aswell the open-cut works of the most representative minerals. The broad coverage of different minerals and working systems makes the paper a close indicator of national average data. The conclusions are considered as the result of standard values in the daily flow, upper values at high load conditions and the actual power used in the service, taking also in account the percentage of it related to the total power capacity of the mine.

RESUME : On présente le drainage dans les mines espagnoles. Pour cela, on a amassé les données des principales exploitations minières, aussi bien souterraines qu'à ciel ouvert, des substances minérales les plus représentatives. Aussi bien par les mines étudiées, que par les substances minérales comprises, l'article résulte indicatif de la moyenne nationale. Le drainage examine le débit moyen journalier, le maximum prévu dans des époques de pointe, la puissance attachée à ce service et le pourcentage de celle-ci sur celle totale installée.

RESUMEN : Se contempla el desagüe en las minas españolas. Para ello se han acopiado los datos de las principales explotaciones mineras, tanto subterráneas como a cielo abierto, de las sustancias minerales más representativas. Tanto por las minas estudiadas, como por las sustancias minerales comprendidas, la comunicación resulta indicativa de la media nacional. El desagüe examina el caudal medio diario, el máximo previsto en épocas puntas, la potencia adscrita a este servicio y el porcentaje de ésta sobre la total instalada.

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En el muestreo realizado sobre desagüe se excluyeron, desde el - primer momento, las canteras habida cuenta
- que gran parte de ellas trabajan de manera intermitente evitando la actividad en los períodos de climatología desfavorable durante los que, generalmente, se presentan los problemas de desagüe.

- que otra parte importante limitan su actividad a las coyuntu-/ ras favorables de mercado.

- que como consecuencia de los bajos precios de los productos de gran número de las mismas, se desplazan paralelamente a las - -/ obras que absorben su producción.

- que el elevado número de ellas, cercano a las 4.000 imposibilitaba un análisis que resultara representativo sino se llevaba a/ cabo un trabajo cuya extensión y duración rebasaba ampliamente a la de la presente comunicación.

Por otra parte, las canteras solo representan, respecto al total de la minería, el 19,5% con relación al pueblo laboral, el 18% - con relación al valor bruto de la producción y el 25% con rela-/ ción a la potencia instalada.

Los sectores mineros estudiados representan, sobre el total de - la minería, excluidas las canteras, el 90,4% respecto a las plantillas, el 85% sobre la potencia instalada y el 76,9% con rela-/ ción al valor de la producción, según se pone de manifiesto en - los cuadros que siguen.

Estos porcentajes reflejan que los resultados obtenidos sobre desagüe en minería pueden tomarse como indicativos de la media nacional.

De los cuadros correspondientes se deduce:

1º) Que el desagüe medio diario de la minería española rebasa -/ los 700.000 m.c. aproximadamente el máximo diario previsto a - - 1.500.000 m.c.

2º) Que la potencia instalada para el desagüe asciende a 138.500 C.V., que representa el 11,7% de la total instalada, correspon-/ diendo 1 C.V. instalado por cada 5 m.c. de desagüe medio diario, y por cada 10,5 m.c. de desagüe máximo previsto.

Se distribuye así: el 63,5% para la minería de los productos energéticos, el 27,7% para la de los minerales metálicos y el 8,8% - para la de los no metálicos.

El número de horas de bombeo efectivo varía sustancialmente de -/ unas explotaciones a otras.

3º) Que el desagüe máximo corresponde a la hulla que se explota - en su casi totalidad, mediante labores subterraneas ubicadas mayoritariamente en la región norte.

Hay 1 C.V. instalado por cada 3,5 m.c. de desagüe medio diario - y por cada 7,4 m.c. de desagüe máximo previsto.

4º) El lignito adquiere también importancia por la corta de Puentes de García Rodriguez con pluviometría media anual de 1.771 -/ l/m^2 y máxima diaria de 127 l/m^2 lo que obliga a previsiones de/ desagüe de 150.000 m.c. en 24 horas con 3.600 C.V. instalados a/ tal fin. Como la explotación es casi superficial la altura manométrica resulta pequeña.

5º) El segundo, en orden cuantitativo, es el hierro cuya explotación se realiza, en su mayoría, a cielo abierto por lo que hay - 1 C.V. instalado por cada 16,5 m.c. de desagüe medio diario y -/ por cada 24,3 m.c. de máximo previsto.

El Marquesado tiene un desagüe medio de 1.400 m.c./hora durante/ las 24 horas del día ya que el acuífero superior condiciona la - estabilidad de taludes y el inferior la profundidad útil de la - corta.

6º) Sigue el plomo-cinc por la influencia de Reocin con un desagüe máximo previsto de 144.000 m.c. día y una potencia instalada de 9.670 C.V. que representan, respectivamente, el 76,6% y el -/ 65%, de los totales de este sector minero.

7º) Por el contrario, el desagüe es insignificantemente en la minería del cobre ya que en las dos principales explotaciones, Cerro Colorado y Arinteiro, ambas a cielo abierto, se realiza el -/ drenaje por gravedad.

8º) Igual consideración para el beneficio de las sales potásicas/ en el que la mayor parte del agua procede de la recuperación del/ relleno hidraúlico reduciéndose el desagüe propiamente dicho a pequeñas cantidades de salmuera saturada procedente de infiltraciones por explotaciones antiguas en la zona catalana. En Potasas de Navarra el desagüe es nulo.

ESTADISTICA MINERA DE ESPAÑA AÑO 1976

Substancia	Personal	Potencia instalada C.V.	Producción Toneladas	Producción Valor 10^6 ptas.
HULLA	35.004	270.000	6.970.000	17.456
ANTRACITA	11.762	77.000	3.548.000	7.300
LIGNITO	6.697	72.000	4.150.000	4.282
HIDROCARBUROS	100	4.000	1.775.000	5.440
URANIO (U_3O_8)	237	4.000	228	442
PRODUCTOS ENERGETICOS...	53.800	427.000	-	34.900

Substancia	Personal	Potencia instalada C.V.	Producción Toneladas	Producción Valor 10^6 pts
Hierro	2.675	125.000	8.200.000	4.150
Pirita	2.304	91.000	2.400.000	2.200
Cobre (Cu)	1.384	91.000	35.000	2.700
Plomo (Pb)	2.330	61.000	66.400	2.260
Zinc (Zn)	2.090	67.000	82.500	2.300
Estaño (Sn)	402	13.000	750	290
Wolframio (WO_3)	372	13.000	400	280
Mercurio (Hg)	1.319	7.000	1.400	350
Otros Minerales	124	16.000	-	1.370
MINERALES METALICOS	13.000	484.000	-	15.900

ESTADISTICA MINERA DE ESPAÑA AÑO 1.976

Substancia	Personal	Potencia instalada C.V.	Producción Toneladas	Producción Valor 10^6 pts
Espato fluor	1.023	24.000	293.000	1.340
Magnesita	563	16.000	134.000	1.050
Sales sódicas	1.092	32.000	3.158.000	1.620
Sales potásicas(ClK)	4.443	143.000	890.000	3.760
Arcilla refractaria	600	12.000	380.000	360
Baritina	204	4.000	91.000	200
Caolín	706	14.000	250.000	400
Bentonita	78	4.000	108.000	190
Cuarzo	216	7.000	597.000	210
Sepiolita	53	5.000	71.000	160
Glauberita	116	2.000	71.000	210
Otros minerales	696	9.000	-	800
MINERALES NO METALICOS.....	10.600	272.000	-	10.300

Substancia	Personal	Potencia instalada C.V.	Producción Toneladas	Producción Valor 10^6 ptas
Arcilla	1.644	74.000	13.200.000	880
Basalto	177	12.000	7.100.000	200
Caliza	6.445	70.000	82.500.000	6.100
Dolomia	272	15.000	1.850.000	220
Granito	1.523	56.000	5.420.000	680
Mármol	1.831	39.000	692.000	63C
Pizarras	1.686	36.000	1.134.000	89C
Yeso	1.192	34.000	4.908.000	500
Otros productos	4.030	64.000	-	3.100
PRODUCTOS DE CANTERA.....	18.000	400.000	-	13.200

ESTADISTICA MINERA DE ESPAÑA AÑO 1976

Substancias	Personal	Potencia instalada C.V.	Valor 10^6 ptas	Nº de explotaciones
Productos energéticos.....	53.800	427.000	34.900	172
Minerales metálicos.....	13.000	484.000	15.900	138
Minerales no metálicos.....	10.600	272.000	10.300	438
Productos de cantera.....	18.800	400.000	13.200	3.808
TOTAL MINERIA ...	96.200	1.583.000	73.300	4.556

PORCENTAJE DE LOS SECTORES ESTUDIADOS SOBRE PUEBLE MINERO (PLANTILLAS)

		%	
Hulla	35.004	45,2	
Antracita	11.762	15,2	69,1
Lignito	6.697	8,7	
Restantes	337	0,4	
PRODUCTOS ENERGETICOS ...	53.800	69,5	
Hierro	2.675	3,4	
Pirita	2.304	3,0	
Cobre	1.384	1,8	15,6
Plomo	2.330	3,0	
Cinc.	2.090	2,7	
Mercurio	1.319	1,7	
Restantes	898	1,2	
MINERALES METALICOS	13.000	16,8	
Sales potásicas	4.443	5,7	5,7
Restantes	6.157	8,0	
MINERALES NO METALICOS ...	10.600	13,7	
TOTAL MINERIA ..... (Sin canteras)	77.400	100,0	90,4

PORCENTAJE DE LOS SECTORES ESTUDIADOS SOBRE POTENCIA INSTALADA.

		%		
Hulla	270.000 C.V.	23,0		
Antracita	77.000 "	6,5	35,6	
Lignito	72.000 "	6,1		
Restantes	8.000 "	0,6		
PRODUCTOS ENERGETICOS	427.000 C.V.	36,2		
Hierro	125.000 C.V.	10,6		
Pirita	91.000 "	7,7		
Cobre	91.000 "	7,7		
Plomo	61.000 "	5,1	37,3	
Cinc	67.000 "	5,6		
Mercurio	7.000 "	0,6		
Restantes	42.000 "	3,5		
MINERALES METALICOS	484.000 C.V.	40,8		
Sales potásicas	143.000 C.V.	12,1	12,1	
Restantes	129.000 "	10,9		
MINERALES NO METALICOS.....	272.000 C.V.	23,0		
TOTAL MINERIA ..... (sin canteras)	1.183.000 C.V.	100,0	85,0	

PORCENTAJE DE LOS SECTORES ESTUDIADOS SOBRE VALOR PRODUCCION MINERA.

		%		
Hulla	17.456 MM ptas	28,8	)	
Antracita	7.300 "	12,0	(	47,8
Lignito	4.282 "	7,0	)	
Restantes.	5.862 "	9,0		
PRODUCTOS ENERGETICOS	34.900 MM ptas	56,8		
Hierro	4.150 MM ptas	6,9	)	
Pirita	2.200 "	3,6	(	
Cobre	2.700 "	4,4	)	
Plomo	2.260 "	3,7	(	23,0
Cinc.	2.300 "	3,8	)	
Mercurio	350 "	0,6	(	
Restantes	1.940 "	3,2		
MINERALES METALICOS	15.900 MM Ptas	26,2		
Sales potásicas	3.760 MM ptas	6,1	)	6,1
Restantes	6.540 "	10,9		
MINERALES NO METALICOS	10.300 MM Ptas	17,0		
TOTAL MINERIA	61.100 MM Ptas	100,0		76,9

DESAGUE MINERIA PRODUCTOS ENERGETICOS

	DESAGUE DIARIO m.c.		POTENCIA INSTALADA C.V.		
	Medio	Máximo	Desagüe	%	Total
Hulla.....	229.600	483.100	65.600	24,3	270.000
Antracita.....	47.000	100.000	13.600	17,6	77.000
Lignito.....	27.800	164.700	8.100	11,2	72.000
Hidrocarburos.....	-	-	-	-	4.000
Uranio.....	2.100	4.400	600	15,0	4.000
TOTAL PRODUCTOS ENERGETICOS.....	306.500	752.200	87.900	20,6	427.000

DESAGUE MINERIA METALICA

	Desagüe diario m.c.		Potencia instalada C.V.		
	Medio	Máximo	Desagüe	%	Total
Hierro.....	165.000	242.000	10.000	8	125.000
Pirita.....	18.300	48.200	9.100	10	91.000
Cobre.....	5.000	22.000	500	0,5	91.000
Plomo.....	42.700	84.500	6.700	11	61.000
Cinc.....	52.300	103.400	8.200	12,2	67.000
Mercurio.....	600	2.900	600	8,5	7.000
Otros minerales....	27.000	48.000	3.300	8	42.000
TOTAL MINERALES METALICOS.....	310.900	551.000	38.400	8	484.000

DESAGUE MINERIA NO METALICA

	Desagüe diario m.c.		Potencia instalada C.V.		
	Medio	Máximo	Desagüe	%	Total
Potasa - Navarra	-	-	-	0	69.800
" - U.E.R.T.	30	40	16	0,08	20.000
" - "	36	36	125	0,49	25.500
" - Suria	660	1.000	2.216	7,5	27.700
SALES POTASICAS.	726	1.076	2.216	1,5	143.000
RESTANTES....	82.574	143.524	9.984	7,75	129.000
TOTAL MINERALES NO METALICOS...	83.300	144.600	12.200	4,48	272.000

DESAGUE TOTAL MINERIA

	Desagüe diario m.c		Potencia instalada C.V	
	Medio	Máximo	Desagüe	Total
Hulla.....	229.600	483.100	65.600	24,3 %
Antracita.....	47.000	100.000	13.600	17,6 %
Lignito.....	27.800	164.700	8.100	11,2 %
Restantes.....	2.100	4.400	600	7,5 %
PRODUCTOS ENERGETICOS....	306.500	752.200	87.900	20,6 %
Hierro.....	165.000	242.000	10.000	8,0 %
Pirita.....	18.300	48.200	9.100	10,0 %
Cobre.....	5.000	22.000	500	0,5 %
Plomo.....	42.700	84.500	6.700	11,0 %
Cinc.....	52.300	103.400	8.200	12,2 %
Mercurio.....	600	2.900	600	8,5 %
Restantes.....	27.000	48.000	3.300	8,0 %
MINERALES METALICOS.....	310.900	551.000	38.400	8,0 %
Sales potásicas.	726	1.076	2.216	1,5 %
Restantes.....	82.574	143.524	9.984	7,7 %
MINERALES NO METALICOS...	83.300	144.600	12.200	4,5 %
TOTAL MINERIA...	700.700	1.447.800	138.500	11,7 %

LAS MINAS DE CAPTACION DE AGUA EN EL MACIZO VOLCANICO DE FAMARA (LANZAROTE, ISLAS CANARIAS)

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ABSTRACT : The water abstraction mines in the Famara volcanic massif draft mainly on the water reserves contained in a low permeability formation. They are uncommon abstraction works, from which many observation data have been cummulated. The study of these data coverts them in some kind of pilot works in massifs alike. The study of discharge and water chemistry yieds a mean permeability less than 1 m/day and frequently less than 0,01 m/day, but the storage coefficient after a slow drainage may exceed 10 per cent, inclusive in tabular basalts of miocene age and until 600 metres below ground surface. The anisotropy is very important. The water is saline and its composition is a function of climate and reflects a very high underground residence time.

RESUME : Les mines de captation d'eau du massif volcanique de Famara, exploitent principalement des réserves d'eau emmagasinée dans un milieu de faible perméabilité. Il s'agit de travaux singuliers de captation, desquels on possède un nombre élevé de données qui les rendent de grand intérêt en tant que travaux pilotes pour l'étude du drainage et des travaux souterrains dans des massifs similaires. L'analyse détaillée de l'évolution des débits, et de la composition chimique des eaux permet de calculer que les perméabilités moyennes sont inférieures à 1 m/jour et fréquemment de l'ordre de 0,01 m/jour, mais avec une porosité à drainage lent qui peut être supérieure à 10 %, même en basaltes tabulaires datés comme d'âge miocénique et jusqu'à des profondeurs de 600 m au-dessous de la surface du sol. L'anisotropie est très grande. L'eau est notablement saline, montrant un clair effet climatique, en outre d'indiquer un temps élevé de séjour dans le terrain.

RESUME : Las minas de captación de agua en el macizo volcánico de Famara, explotan principalmente reservas de agua almacenada en un medio de escasa permeabilidad. Se trata de obras de captación singulares, de las que se posee un número elevado de datos que las hacen de gran interés como obras piloto para el estudio del drenaje y de obras subterráneas en macizos similares. El análisis detallado

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de la evolución de caudales, y de la composición química de las aguas permite calcular que las permeabilidades medias son inferiores a 1 m/día y con frecuencia del orden de 0,01 m/día, pero con una porosidad a drenaje lento que puede superar el 10 %, aún en basaltos tabulares datados como de edad miocénica y a profundidades de hasta 600 m bajo la superficie del terreno. La anisotropía es muy grande. El agua es notablemente salina, mostrando un claro efecto climático ademas de reflejar un elevado tiempo de residencia en el terreno.

1. - INTRODUCCION

La isla de Lanzarote, con alrededor de 45 000 habitantes y 796 km^2 de superficie, es la más septentrional y oriental de las Islas Canarias, y dista solamente 125 km de la costa sahariana (fig. 1). Su forma es alargada, de NE a SW, y está separada de la vecina isla de Fuerteventura por solo 12 km de un mar poco profundo.

Toda la isla es volcánica, con importantes efusiones históricas, siendo las de 1724 unas de las más duraderas y voluminosas de las conocidas mundialmente.

El área más elevada es el macizo basáltico miocénico de Famara, en el norte, donde se alcanzan los 670 m de altitud a solo 2 km de la costa. El macizo de Famara, alargado de Norte a Sur, está formado por coladas de basalto tabulares. Acaba en un impresionante acantilado sobre el mar en el lado Oeste, y es algo más suave al Este, donde alcanza el mar a través de formaciones volcánicas más recientes, las cuales también cubren en algunas zonas deprimidas el interior del macizo.

Dicho macizo de Famara está disecado por profundos barrancos, en especial hacia el lado Este, que permanecen secos a lo largo del año y solo muy efimeramente conducen agua como consecuencia de tormentas importantes, en general de más de 50 a 60 mm en 24 h (en realidad en unas pocas horas).

Aunque el macizo de Famara es la zona más húmeda de la isla, no por ello pierde el caracter de aridez acentuada, ya que la pluviometría media en su cumbre, donde es mayor, es del orden de 180 mm/año y en la periferia es de solo 110 mm/año, e incluso de solo 80 mm/año en el límite llano del SE. Existen notables variaciones pluviométricas de un año a otro, entre 1/3 y 2 del valor medio. Se producen unos 40 días de lluvia al año, pero sólo unas pocas tienen alguna importancia, aunque en ocasiones pueden ser tormentas muy violentas (hasta 50 mm/hora).

La evaporación media del agua libre es de alrededor de 3000 m/año y la evapotranspiración potencial puede alcanzar 1500 mm/año.

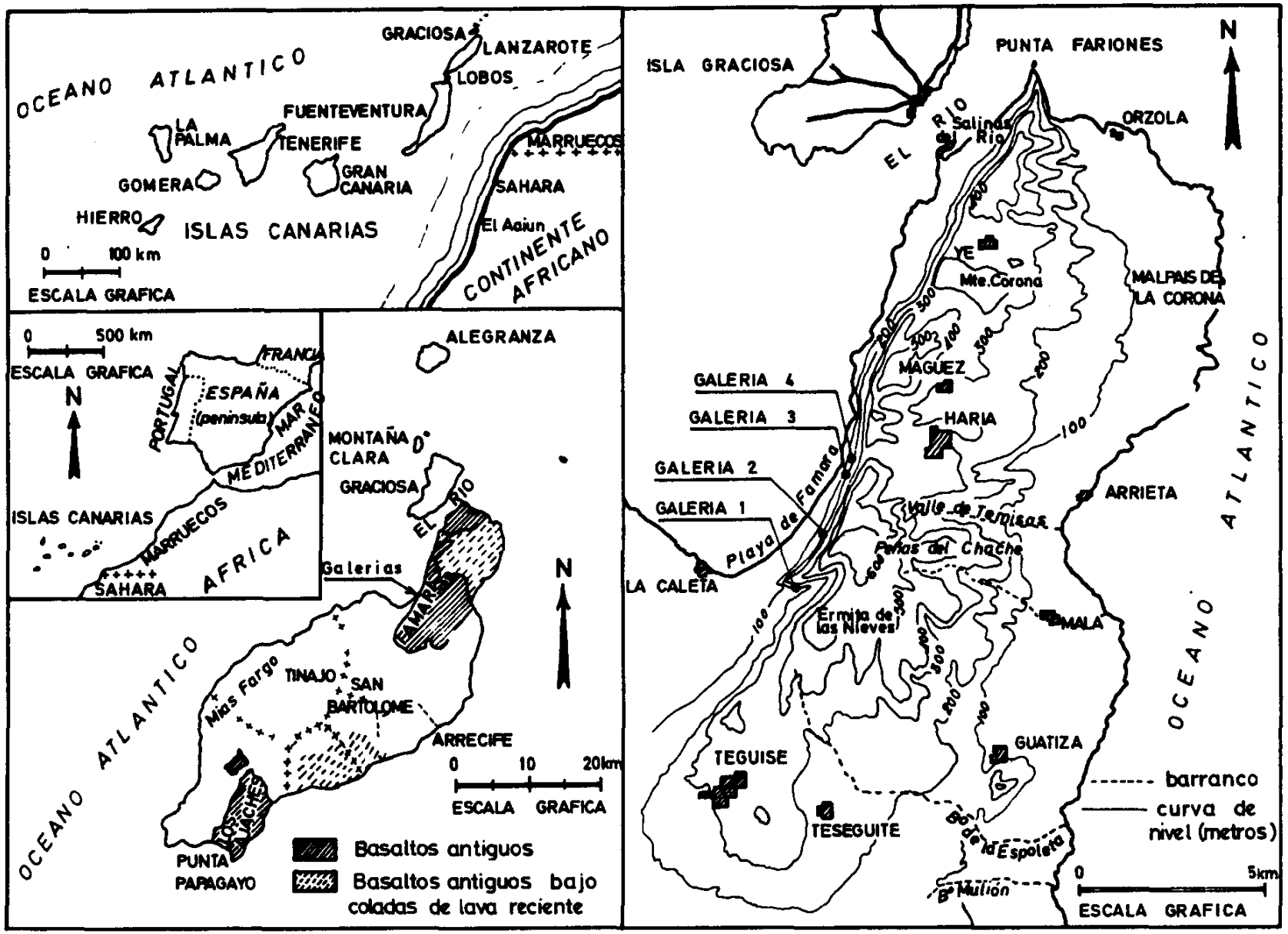

Fig. 1._ Situación general y detalle del extremo norte de Lanzarote, con el macizo de Famara y situación de las galerias

2. - NATURALEZA DE LOS MATERIALES

Los basaltos del macizo de Famara, que son de caracter alcalino, corresponden a zonas inmediatas a grandes erupciones fisurales que se extendieron sobre una superficie mucho mayor que la actual, acompañándose de erupciones centrales simultáneas. Son frecuentes los cuerpos intrusivos tales como diques, sills y lacolitos, más frecuentes cuanto mayor es la profundidad.

La disposición de los basaltos tabulares muestra cierto grado de imbricación y alternan capas de lavas masivas, de unos pocos metros de espesor por lo general, con escorias y acumulaciones de piroclástos, en un conjunto que regionalmente buza unos 5º hacia el ESE. Son relativamente frecuentes los niveles rojizos o almagres, de hasta 2 metros de espesor, en ocasiones con un grado de alteración notable, aunque su continuidad y homogeneidad es cuestionable a pesar de que se manifiestan claramente en largos trechos; no obstante pueden jugar un papel no despreciable en la existencia de pequeños niveles acuíferos colgados.

Los basaltos más modernos corresponden a emisiones centrales con conos que mantienen ciertas alineaciones preferenciales, y que en ocasiones están asociados a una notable acumulación de productos piroclásticos.

La zona preferente de emisiones se sitúa en el lado occidental y corresponde a fisuras de orientación aproximada N-S. Es probable una fisuración distensiva paralela asociada, así como fisuras de descompresión que llegan hasta el nivel freático en los valles más profundos del lado oriental (valle de Temisas). No existen evidencias de fallas.

La edad de los materiales influye en las características de permeabilidad propia de la roca, ya que cuanto más antigua sea mayor es el grado de alteración, de relleno de cavidades y de aparición de materiales poco permeables, si bien en ocasiones puede aumentar el grado de conexión entre poros. No obstante, al aumentar la edad puede aumentar la permeabilidad secundaria asociada a fisuras, en especial en zonas descomprimidas próximas a la superficie, aunque este efecto es menos acusado que el anterior.

En el macizo de Famara, los basaltos miocenos resultan poco permeables, pero la debil permeabilidad se mantiene a profundidades de

más de 500 m bajo la superficie del terreno. Los diques no suponen ningún obstáculo importante al movimiento del agua, e incluso en ocasiones son más permeables que la roca encajante. En general aumentan solo la anisotropía, principalmente en función de la fisuración vertical asociada (Custodio, 1978).

3. - RECARGA Y DESCARGA

La aridez climática hace que la recarga media anual sea muy pequeña o nula, tanto menor cuanto más retentivo sea el escaso suelo existente. Donde la roca aflora y está visiblemente fracturada es más probable la recarga del agua de lluvia y quizás de la condensación nocturna (aunque su contribución parece despreciable), ya que el agua infiltrada alcanza con cierta facilidad profundidades desde las que ya no es devuelta a la atmósfera. Esta recarga media anual, según datos basados en el balance diario del agua en el suelo y en el balance de ión cloruro entre el contenido en la lluvia y el contenido en el agua freática, puede llegar a 10 mm/año en zonas elevadas, y desciende hasta menos de 3 mm/año, e incluso cero en las áreas marginales.

Entre el nivel del mar y cota 500 m aparecen manantiales y rezumes permanentes, pero de caudal insignificante, con un aporte total máximo de 1 a 2 m^3/día ($\sim$ 0,1 mm/año).

En el entorno de la ciudad de Haria, existen más de 100 pozos someros excavados en un recubrimiento de toba volcánica moderna. La extracción total no rebasa los 20 m^3/día (menos de 1 mm/año en el area de recarga probable). Extracciones del orden de 50 m^3/día, en parte agua salada marina, existen en el área de La Caleta, ya marginal al macizo de Famara, en el SW.

Las fuentes más importantes de agua subterránea son la galería de Famara (Fig. 1), en especial las cuatro construídas por el Cabildo Insular de Lanzarote para el abastecimiento a la capital, Arrecife, con la asesoria y colaboración del Servicio Geológico de Obras Públicas. Estas galerías son el objeto principal de esta comunicación.

En el conjunto del macizo se estiman unas reservas del orden de 250 hm^3 por encima de la cota +25m, de las que en 1973 las galerías solo habían extraído menos del 1 %, si bien la extracción de una

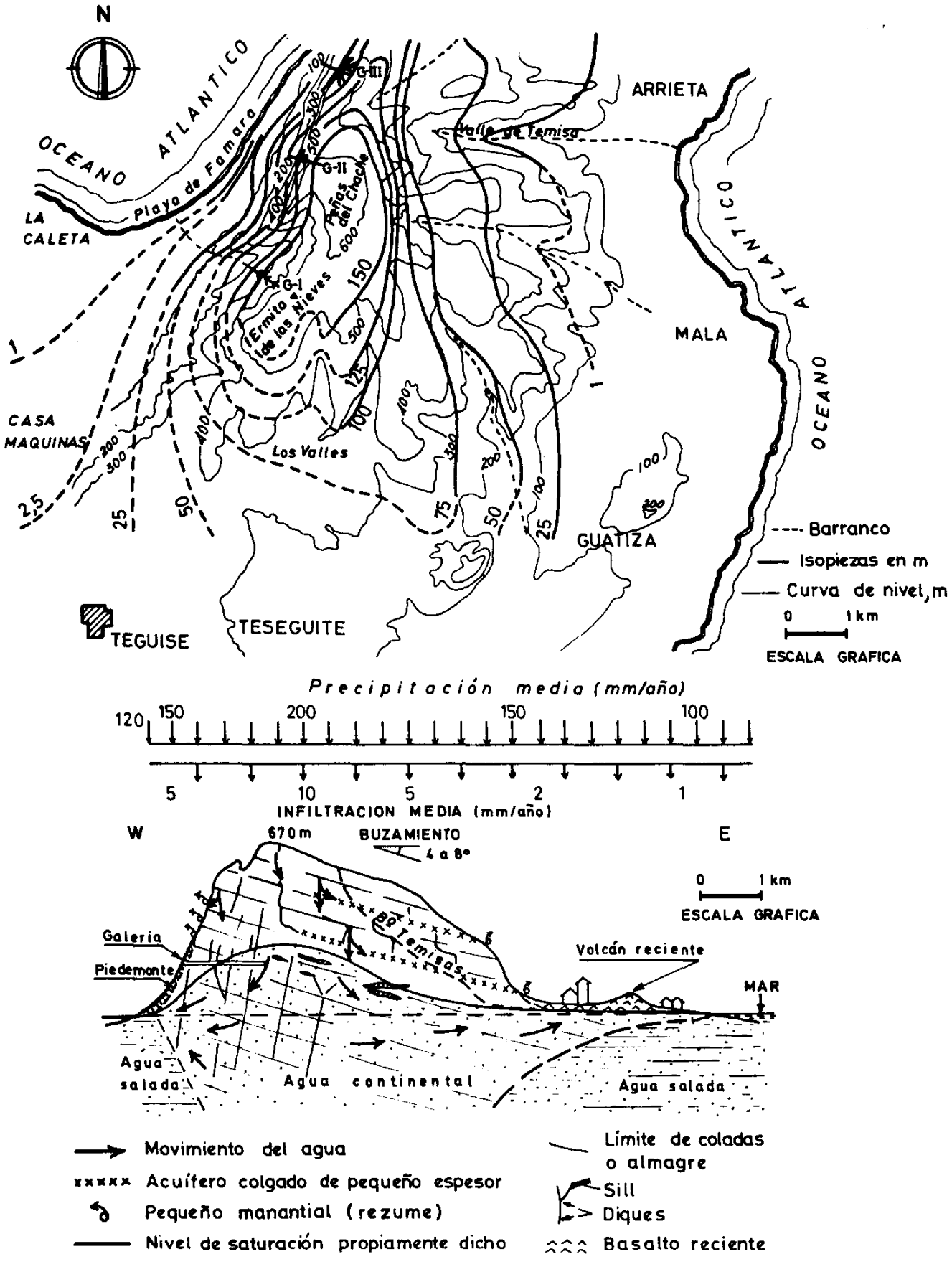

Fig. 2.— Superficie freática y sección simplificada

parte importante de las mismas, dada la escasa movilidad del agua, requeriría numerosas nuevas obras de captación. En 1972 la producción de las galerías era de 900 m^3/día, en su mayor parte procedente de las reservas, pero después ha declinado hasta menos de 200m^3/d.

4. - EXISTENCIA DE UN NIVEL DE SATURACION

Aunque en varias ocasiones se ha cuestionado la existencia de un nivel de saturación, la relativamente importante información piezométrica existente en el macizo de Famara en los sondeos y galerías permite dibujar una superficie freática (fig. 2) bien definida, con niveles estables, si bien los gradientes piezométricos son muy elevados (hasta 0,13) a pesar de la escasa recarga, dada la pequeñísima transmisividad en dirección al mar. Debido a la existencia de componentes verticales del flujo, los niveles piezométricos varían con la profundidad (fig. 3), pero ello no se contradice con la existencia del nivel de saturación profundo, sino que lo confirma.

De hecho la superficie freática debe considerarse virtual en zonas en que coincide con roca impermeable, pero con perforaciones apropiadas puede demostrarse su existencia. Ello no excluye la existencia de pequeños niveles colgados, de pequeña importancia (fig. 2).

La continuidad y regularidad aún a pequeña escala, de la superficie de saturación, queda demostrada por un estudio piezométrico con perforaciones realizado a lo largo de la galería III de Famara - (fig. 6), a pesar de que esta galería tiene extensos tramos no productivos e intersecta varios diques, almagres, capas de basalto masivo, capas escoriáceas, etc. La normal respuesta de las propias galerías y de los sondeos de drenaje en ellas practicados, a las - pruebas hidráulicas que serán comentadas más adelante, es una - prueba más.

5. - CARACTERISTICAS DE LAS GALERIAS DE FAMARA

Las primeras obras de captación en galerías en Famara construídas en la década de 1950, sirvieron para recoger las aguas de algunos rezumes de escaso caudal, en general colgados sobre el nivel freático regional.

Tras un estudio de reconocimiento general y de perforaciones realizado por el Servicio Geológico de Obras Públicas, el Cabildo Insular de Lanzarote inició la construcción de galerías profundamente

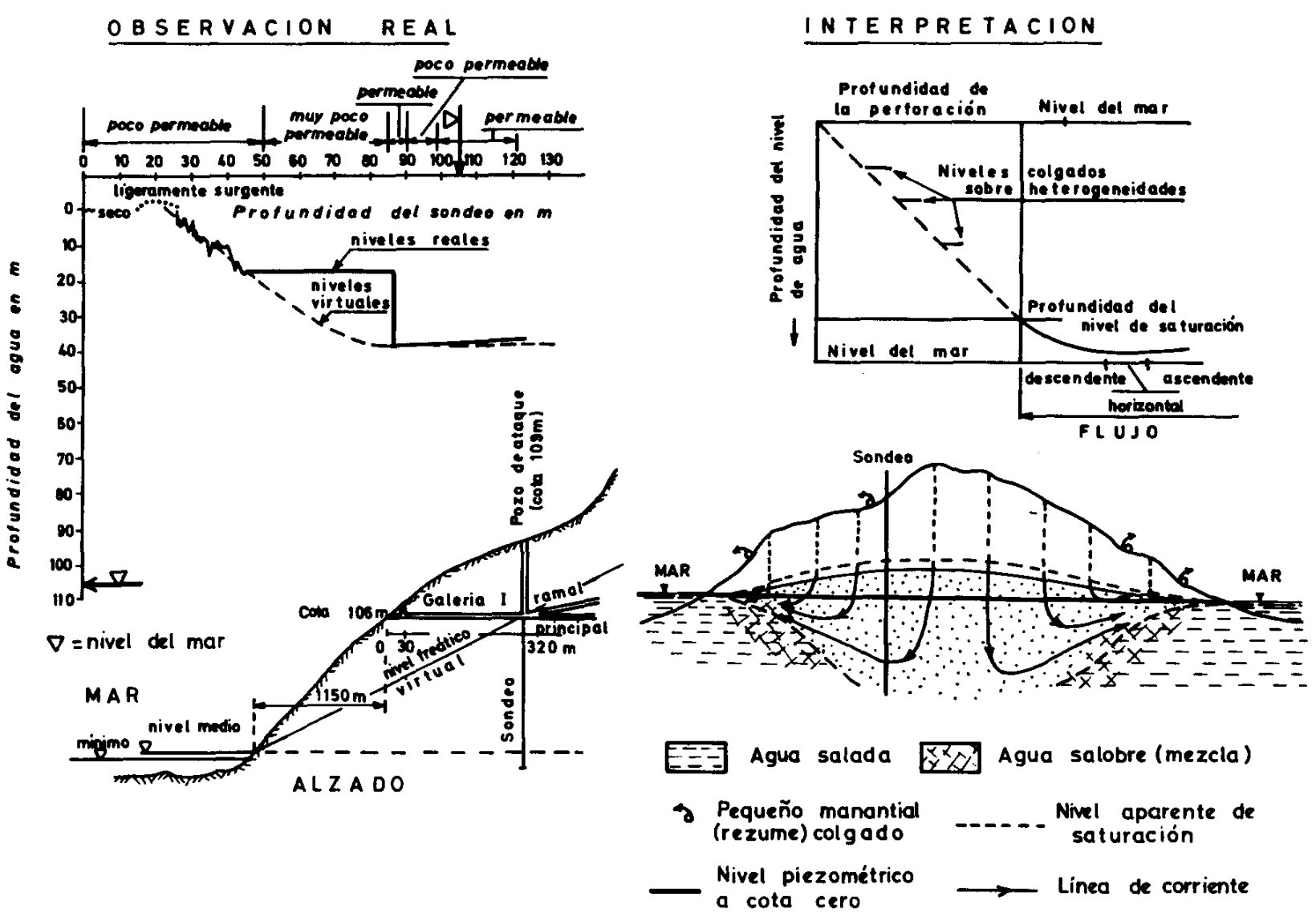

Fig. 3.— Evolución del nivel en sondeos, real e interpretada, en el Macizo de Famara, cuando los gradientes piezométricos son muy grandes

penetrantes, hasta alcanzar el nivel de saturación regional. En las galerías III y IV, el mencionado Servicio Geológico ha realizado sondeos y estudios de detalle para aumentar y mejorar las características de producción, así como para orientar las nuevas obras a realizar.

Las galerías I y II se emboquillaron a altura relativamente elevada para evitar la perforación del piedemonte poco estable adosado al farallón de Famara. La III se emboquilló a menor cota en un barranco, pero se tuvo que efectuar un notable revestimiento de su acceso para combatir los efectos de movimientos de ladera y caida de piedras. La IV fué emboquillada casi a nivel del mar, directamente en un afloramiento del basalto por entre el recubrimiento.

En la figura 4 se muestra la disposición en planta de cada una de las galerías en 1972 y en la tabla 1 sus principales características. Mientras en las galerías I y II apenas se ha realizado nuevas obras, las III y IV han sido prolongadas y se han establecido nuevos sondeos de drenaje frontal entre 1973 y 1975, con longitudes entre 100 y 200 m por dren. Estas galerías III y IV son de mayor sección que las otras y es posible recorrerlas con facilidad.

Mientras el caudal de los sondeos de la galería I no puede ser controlado, si es posible en los de la galería III ya que disponen de un tubo de emboquillado cementado y de una válvula de regulación. Ello permite adaptar el caudal de producción de la galería a la demanda, dentro de ciertos límites y evitar así desperdiciar agua en momentos de menores necesidades. En la galería III la mayoría de los sondeos son horizontales, en buena parte en el frente de avance de la galería y con disposición radial, pero además existen otros verticales en el tramo saturado, y por lo tanto surgentes. Su caudal puede incrementarse por encima del máximo natural, mediante el bombeo con aire comprimido, aprovechando la instalación existente para lós trabajos de perforación en curso, y lo mismo puede efectuarse en otros sondeos verticales de caracter piezométrico perforados en la zona no saturada de la galería. Estos modos de operación resultan muy interesantes y permiten regular el caudal obtenído, de forma sencilla y eficaz.

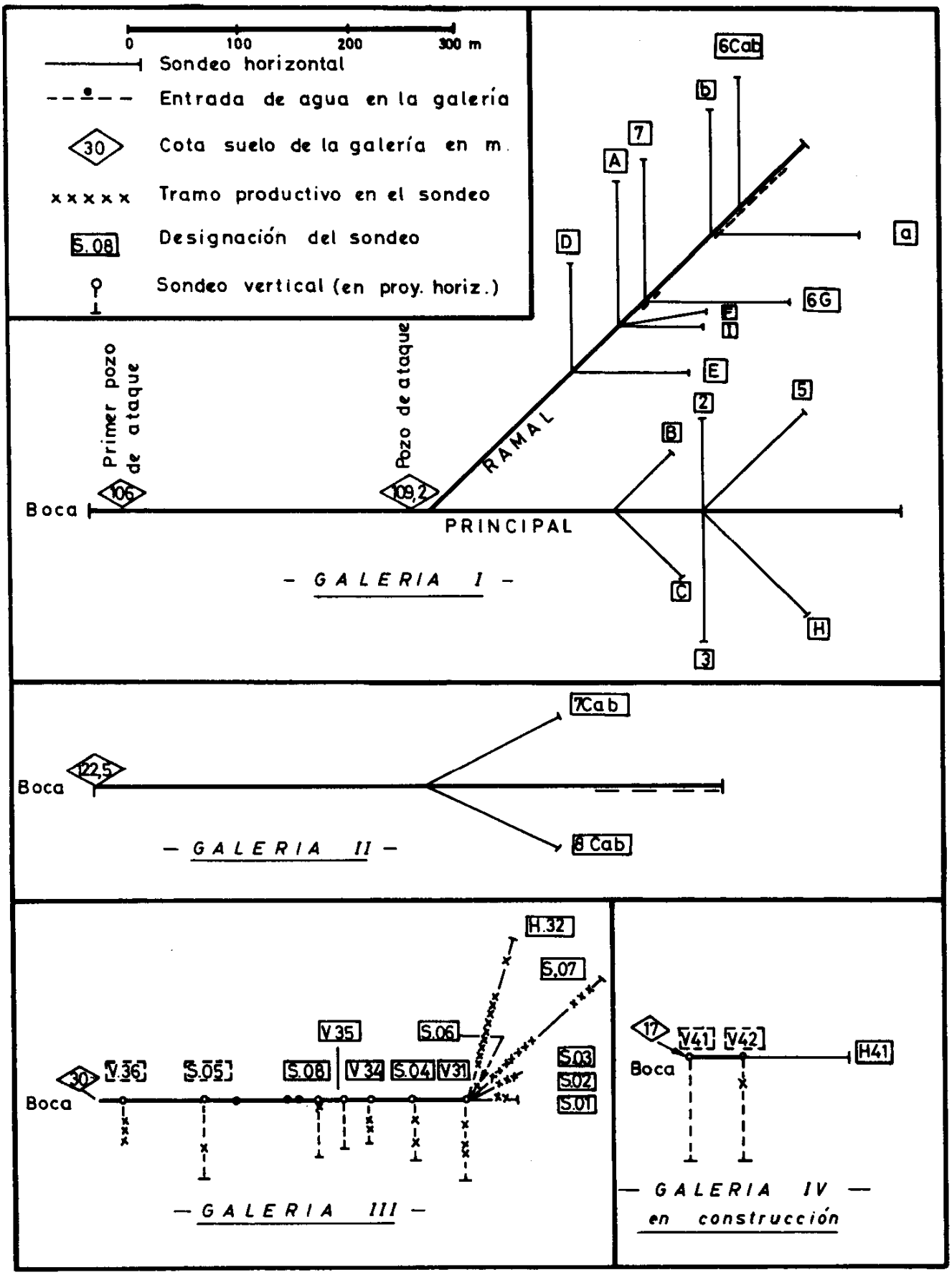

Fig. 4.— Plantas de las galerías de Famara, mostrando sus sondeos. Los sondeos verticales están proyectados. Situación a finales de 1972.

Tabla 1 - Características de las galerías de Famara (datos referidos a finales de 1972)

Características	Galería I	II	III	IV
Longitud en metros	1373	819	357	100
Metros desde la boca al mar	1150	500	70	39
Metros hasta la galería anterior	-	1700	1100	507
Cota de la boca (metros)	106	122	30	17
Rumbo	N 100 E	N 100 E	N 100 E	N 125 E
Longitud de la galería	768+505	820	356	55
Caudal en 1960 (l/s)	2,5	-	-	-
en 1966 (l/s)	2,1	1,2	-	-
en 1969 (l/s)	1,5	2,1	1,2	-
en 1972 (l/s)	1,25	1,4	8	0,0
Nº de sondeos (productivos)	16(5)	2(0)	14(10)	3(1)
Longitud de sondeos (m)	1838	286	1000	208
Caudal de los sondeos (l/s)	0,5	0,0	8(máx 20)	0,0
Presión máxima en sondeos (atm)	0,2	-	2,1	-
Metros productivos de galería	400+500	200	150	0
Inicio de construcción	1955	1963	1968	1971
Fin de obras principales	1964	1971	en obras	en obras
Volumen de agua hasta fin 1972 (hm^3)				
- total	1,3	0,37	0,25	0,00
- de los sondeos	0,26	-	0,13	0,00
m^3/m de sondeo	140	-	130	-
Cloruros, ppm Cl^- en salida	650	1000	2100	650

Observaciones:

Galería I: Galería principal de 768 m y ramal de 505 m que se inicia a 320 m de la boca, inclinado 45º a la izquierda.

Galería II: Sondeos perforados en zona no saturada.

Galería III: El caudal se regula con las válvulas de los sondeos según demanda. Hay sondeos horizontales, inclinados, verticales y piezométricos. Los caudales descienden rápidamente con el tiempo.

Galería IV: En construcción. Sondeos horizontales en zona no saturada. Los verticales son piezométricos. Se cortan algunos niveles con un poco de agua colgada.

6. - ANALISIS DE LA EVOLUCION DE LOS CAUDALES DE LAS GALERIAS

El análisis hidráulico de los caudales de las galerías de Famara debe tener en cuenta:

- El efecto de los trabajos de prolongación de las galerías.
- El efecto de perforación de sondeos de drenaje.
- La existencia de una carga de agua variable a lo largo del tiempo y de la galería.
- La penetración parcial de la galería en el domo de agua.

La galería se asimila, o bien a un manantial que drena en macizo rocoso, o bien a un dren, suponiendo que:

- Existe una red tridimensional de zonas permeables y fisuras conductoras de agua, tal como se ha observado a partir de ensayos y controles de interferencia entre sondeos.

- Aunque el medio es en detalle muy heterogéneo (en los drenes de la galería III se ha observado que en promedio existen 2 a 3 tramos permeables por cada 100 m lineales, abarcando del 10 al 20 % de la longitud penetrada), cuando el volúmen de influencia es grande (por ejemplo con una dimensión de 100 a 500 m), el medio es de características hidráulicas suavemente cambiantes (quasihomogéneo), tal como después se confirma.

- El nivel freático virtual cambia suavemente, aunque su pendiente es grande a causa de la baja permeabilidad de los materiales.

- La irregularidad en la reducida recarga no afecta pues es promedida en el tránsito por el gran espesor de medio no saturado (de 100 a 500 m, con un tiempo de tránsito de varias decenas de años por lo menos).

- Los cambios en la presión atmosférica solo inducen pequeñas variaciones en la producción de las galerías y en sus drenes.

- Los avances de las galerías o la instalación de drenes no se hace a saltos instantáneos, aunque ello puede admitirse después de algunos meses o años después, pero no para los primeros tiempos.

- En los primeros tiempos después de la construcción de un tramo de galería o de un sondeo se produce un rápido drenaje de los tramos

más permeables, pero al aumentar el volúmen de influencia con el tiempo, el drenaje se realiza según las características medias y según leyes asintóticas.

Las galerías drenan preferentemente reservas en el macizo, aunque también captan parte del debil flujo de agua de recarga que circula hacia el mar, después de haber descontado las salidas por los manantiales y rezumes. Este caudal de base permanente puede estimarse a grosso modo calculando la disminución porcentual de gradiente hidráulico entre la zona activa de la galería y el mar, suponiendo que todo el macizo es permeable y que los gradientes aguas arriba no se alteran sensiblemente.

Puede estimarse, como primera aproximación, que (Custodio, 1974b):

$$Q_b = \frac{4}{3} q_o D \left(1 - \frac{H}{2Li_o}\right)$$

Q_b = caudal de base de la galería
D = distancia de influencia lateral de la galería
H = altitud de la zona productiva de la galería
L = distancia efectiva entre la costa y la zona productiva de la galería
q_o = caudal de salida natural al mar por unidad de longitud de costa, deducide de la recarga
i_o = pendiente natural de la superficie freática

Para las diferentes galerías, con un error probable entre triple y tercio, se obtiene:

Galería	I	II	III	IV
Q_b, m^3/día	4	2	2	2

Las diferentes leyes de estudio del agotamiento de caudales, con las que se han obtenído resultados aceptables en base a un ajuste suficiente a las observaciones reales, son:

a) Ley exponencial $Q - Q_b = (Q_o - Q_b).e^{-\alpha t}$

Q = caudal en el tiempo t
Q_b = caudal de base (residual para $t = \infty$)
Q_o = caudal inicial teórico (diferente del caudal inicial observado)

α = constante que depende de las características hidráulicas y dimensiones del sistema

Es válida cuando $t > 0,5\alpha$ (Llamas, 1976)

La representación de log $(Q-Q_b)$ en función de t debe ser una recta para $t > 0,5\alpha$, cuya pendiente es $-2,3/\alpha$ y cuya ordenada en el origen es Q_o-Q_b.

En un macizo ideal se puede establecer que:

$$\alpha = \frac{\pi^2}{4d^2}\ \frac{T}{S}$$

$$Q_o = 2\frac{\Delta h}{d}\ T.L$$

T = transmisividad = k.b = permeabilidad x espesor saturado
S = coeficiente de almacenamiento
T/S = difusividad hidráulica
d = distancia a la divisoria de aguas en el macizo
L = longitud efectiva drenante de la galería
Δh = descenso piezométrico efectivo producido por la galería

Utilizando valores apropiados para las galerías, I (fig. 5) y II, en las que la serie de datos es suficientemente larga (los valores entre paréntesis pueden tener un error considerable):

	α años^{-1}	Qo m^3/día	T/S m^2/día	(T) m^2/día	(S) -	(k) m/día	Qb mejor m^3/día
Galería I	0,10	250	100-150	15-25	0,13-0,20	0,1	0-20
Galería II	0,2	150	200-500	75-110	0,2 -0,3	0,5	0

b) Ley de la raiz inversa $\quad Q-Q_o = (Q_o-Q_b)\ (1+\beta t)^{-2}$

β = constante que depende de las características hidráulicas y dimensiones del sistema.

Es apropiada al análisis del agotamiento del caudal de manantiales que drenan un acuífero libre (Llamas, 1976). Coincide con la Ley exponencial para $\beta = \alpha/2$ cuando $\alpha . t \ll 1$. Ambas funciones tienden a $Q = Q_b$ para $t \rightarrow \infty$, aunque la exponencial tiende más rapidamente. La ley de la raiz inversa, al igual que la exponencial, no es válida durante los primeros tiempos del drenaje.

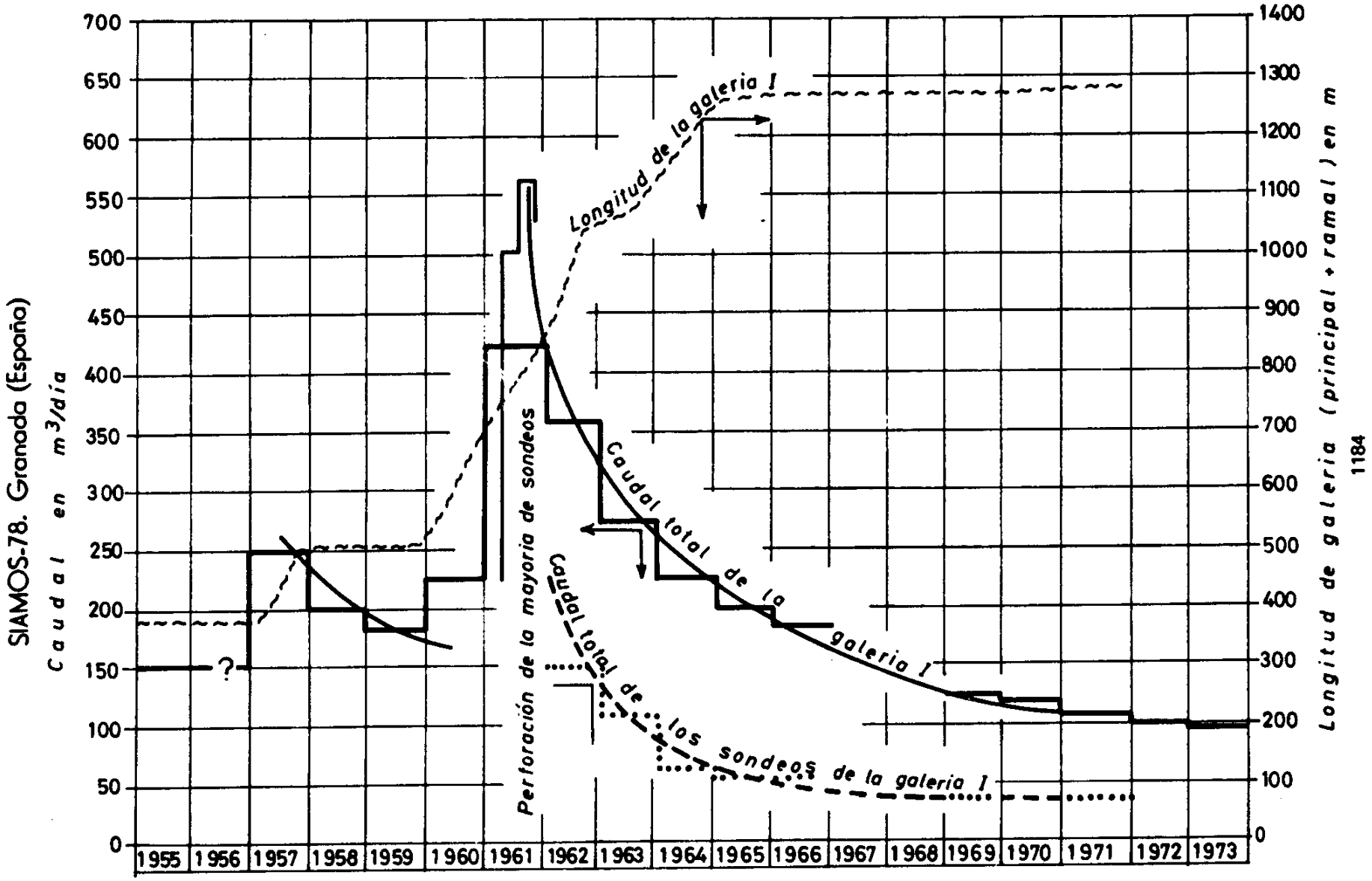

Fig. 5.— Evolución de los caudales de la Galeria 1 y de las obras de prolongación.

Si se escribe:

$$(Q-Q_b)^{-\frac{1}{2}} = (Q_o-Q_b)^{-\frac{1}{2}} + \beta.(Q_o-Q_b)^{-\frac{1}{2}} \cdot t$$

la representación de $(Q-Q_b)^{-\frac{1}{2}}$ en función de t debe ser una recta de pendiente $\beta.(Q_o-Q_c)^{-\frac{1}{2}}$ y ordenada en el orgien $(Q_o-Q_c)^{-\frac{1}{2}}$

En un macizo ideal puede establecerse que:

$$\beta = \frac{2\,\Delta h.k}{S.d^2} \qquad H_o = \text{espesor saturado inicial}$$

$$Q_o \simeq 4\,\frac{\Delta h.k.Ho.L}{d}$$

Utilizando valores apropiados para la galería I, se obtiene:

	β años^{-1}	Q_o m^3/día	k m/día	S -	Q_b mejor m^3/día
Galería I	0,068	320	0,22-0,17	0,11-0,16	20

c) Ley de la inversa de cuadrados

Es preferentemente válida para los primeros tiempos de drenaje, para los cuales $Q \gg Q_b$ de modo que $1/Q^2 \simeq \gamma.t$. Debe cumplirse que $t < 0{,}3\ d^2S/T$ (Custodio, 1976), lo cual supone que la depresión de drenaje no alcanza la divisoria de aguas del macizo. Representando $1/Q^2$ en función de t debe obtenerse una recta de pendiente γ, que pase por el origen de coordenadas.

En un macizo ideal, con poca variación del espesor saturado,

$$\pi/\gamma = 4\,\Delta h^2.L^2.S.T$$

Otros tipos de tratamiento (Custodio, 1976b) hacen variar el factor 4 entre 0,25 y 3, aunque los valores más adaptados al problema planteado varian entre 1 y 3.

Utilizando valores apropiados para las galerías I y II, se obtiene:

	γ día^2/m^6/año	T.S m^2/día
Galería I	$(5\div10).10^{-6}$	1÷4
Galería II	$(0{,}35\div0{,}4).10^{-4}$	~10

Como $S < 1$, los valores de T son mayores a los obtenídos anteriormente, como en principio cabía esperar dada la mayor influencia de la zona descomprimida y fracturada en el entorno de la galería a consecuencia de su excavación, drenaje de fisuras mayores y efecto de los explosivos. Se han ensayado otras leyes sin justificaciones teóricas razonables (Custodio, 1972, 1974), pero no se comentan por solo ser útiles para la extrapolación, y de ellas solo la de la inversa de caudales resulta bien ajustada ($1/Q = \omega + \delta.t$).

El agotamiento del caudal de los sondeos de drenaje practicados en las galerías puede estudiarse de forma similar al de las galerías, si bien existen mayores dificultades debido a:

- Interferencia entre sondeos y con la galería (salvo que se considere la producción conjunta de sondeos y galería)

- Efecto de los sondeos en construcción sobre los en producción

- Efecto de la regulación del caudal de los sondeos mediante la válvula de cabeza.

Por ello este estudio está aún por realizarse y dependerá de la representatividad de los datos que se obtengan. La aplicación de la ley exponencial en uno de los sondeos de la galería I (sondeo D) permite calcular, con valores apropiados de los valores geométricos del macizo afectado por el sondeo:

$\alpha = 0{,}42$ años^{-1}; $Q_o = 11{,}5$ m^3/día; $T/S = 15\text{-}40$ m^2/ dia;
$T = 0{,}8\text{-}1{,}1$ m^2/día; $S = 0{,}03\text{-}0{,}04$; $k \simeq 0{,}01$;

Sin un análisis más profundo no pueden tomarse estos datos como fiables, que parecen dar valores de T y S demasiado bajos.

El estudio de la superficie piezométrica proporciona valores de la permeabilidad mucho menores, del orden de $T = 0{,}2$ m^2/día y $k \simeq 0{,}001$ m/día. No es de extrañar la disparidad de estos valores con los anteriores, ya que los deducidos de la superficie piezométrica son solo en dirección E-W, hacia el mar, y unicamente ponen de manifiesto la elevada anisotropía. El elipsoide permeabilidades tiene un eje menor en dirección E-W y las permeabilidades mayor y media están en un plano vertical de orientación N-S.

Durante la prolongación del ramal de la galería I se observó que el caudal de los sondeos de drenaje próximos al frente de avance disminuía temporalmente para luego aumentar de nuevo. Además de un efecto de interferencia a causa de la producción de agua del nuevo tramo de galería, existe un efecto de brusco aumento de la porosidad local a causa de la perforación (uso de explosivos y descompresión bajo algunos cientos de metros de montera). Dada la escasa permeabilidad del material, el aumento del volumen de poros y fisuras produce una pérdida de potencial hidráulico que tarda varios meses en restablecerse, afectando a la producción de los sondeos y tramos de galería próximos.

7. - ENSAYOS HIDRAULICOS EN LOS SONDEOS DE LA GALERIA III

En varios de los sondeos de drenaje de la galería III (horizontales y verticales) se han realizado ensayos para obtener mayor información sobre las características hidráulicas de los materiales. En esos ensayos de variación de la presión con el tiempo después del cierre tras un largo periodo de producción, la valoración se ha realizado suponiendo válida la aproximación logarítmica de Jacob, por lo menos durante los primeros minutos, calculando con ello T (transmisividad) y una estimación del valor aproximado de S (coeficiente de almacenamiento) si se admite que el radio del sondeo es asimilable al radio eficaz de cálculo. Dada la corta duración de los ensayos (algunas horas) y debido a que solo son interpretables los datos de los primeros 5 a 15 minutos, el valor de S no representa la porosidad eficaz del medio, sino un valor en relación con la reacción elastica del medio.

En las diferentes curvas se observa:

- La puesta en presión de los sondeos verticales surgentes es inmediata, pero en los sondeos horizontales, a menos que el caudal sea elevado, existe un periodo de llenado de la porción vacía del tubo que enmascara los resultados.

- El aumento de presión registrado con el tiempo se amortigua progresivamente por diferentes causas:

  a) Interferencia con el caudal de otros sondeos y con la producción directa de la propia galería, tanto más acusado cuanto más sondeos próximos existan y/o más productiva sea la galería en el entorno.

b) Efecto de penetración parcial de los sondeos.

c) Posible efecto de variación de la apertura de las fisuras al aumentar la presión.

Los resultados obtenidos son:

Sondeo		Caracter zona	Transmis. T m^2/día	Perm. m^2/día media	Perm. m^2/día tramos produc.	Coef. almacen. S
S.04	Vert.	No product.	2,6-3,3	0,05	0,25	0,0005
S.31	Vert.	Productiva	7-9	0,11	0,8	0,009
S.06	Ascend.	Productiva	0,5	0,05	0,015	0,02

Se realizaron ensayos de inyección para estudiar la relación caudal/presión, encontrándose que es aceptablemente constante, lo cual demuestra que el flujo es lineal y por lo tanto que es aplicable la ley de Darcy. De los valores de esta relación se obtiene:

Sondeo	S.02	S.01 por interferencia con S.02	S.06
Transmisividad m^2/día	6,3	3,2	0,8
Permeabilidad media m/día	0,15		0,048
Tramo productivo	0,75		0,11

En ensayos similares de producción se obtuvo la misma relación lineal (fig. 6) y los valores:

Sondeo	SH.32	S.07	SV.31	S.03	S.02	S.01	S.04
Presión cierre atm.							
medida	2,0	1,8	1,9	2,1	2,2	2,1	1,7
máxima	2,9	2,8	2,3	2,5	2,5	2,5	1,9
Producción l/s/m	0,21	0,22	0,13	0,09	0,06	0,045	0,03
Tranmisiv. m^2/día	18	19	11	8	5	4	2,5
Permeabilidad m/día							
media	0,11	0,12	0,14	0,15	0,12	0,09	0,04
tramo productivo	0,24	0,27	1,0	0,30	0,7	0,5	0,21

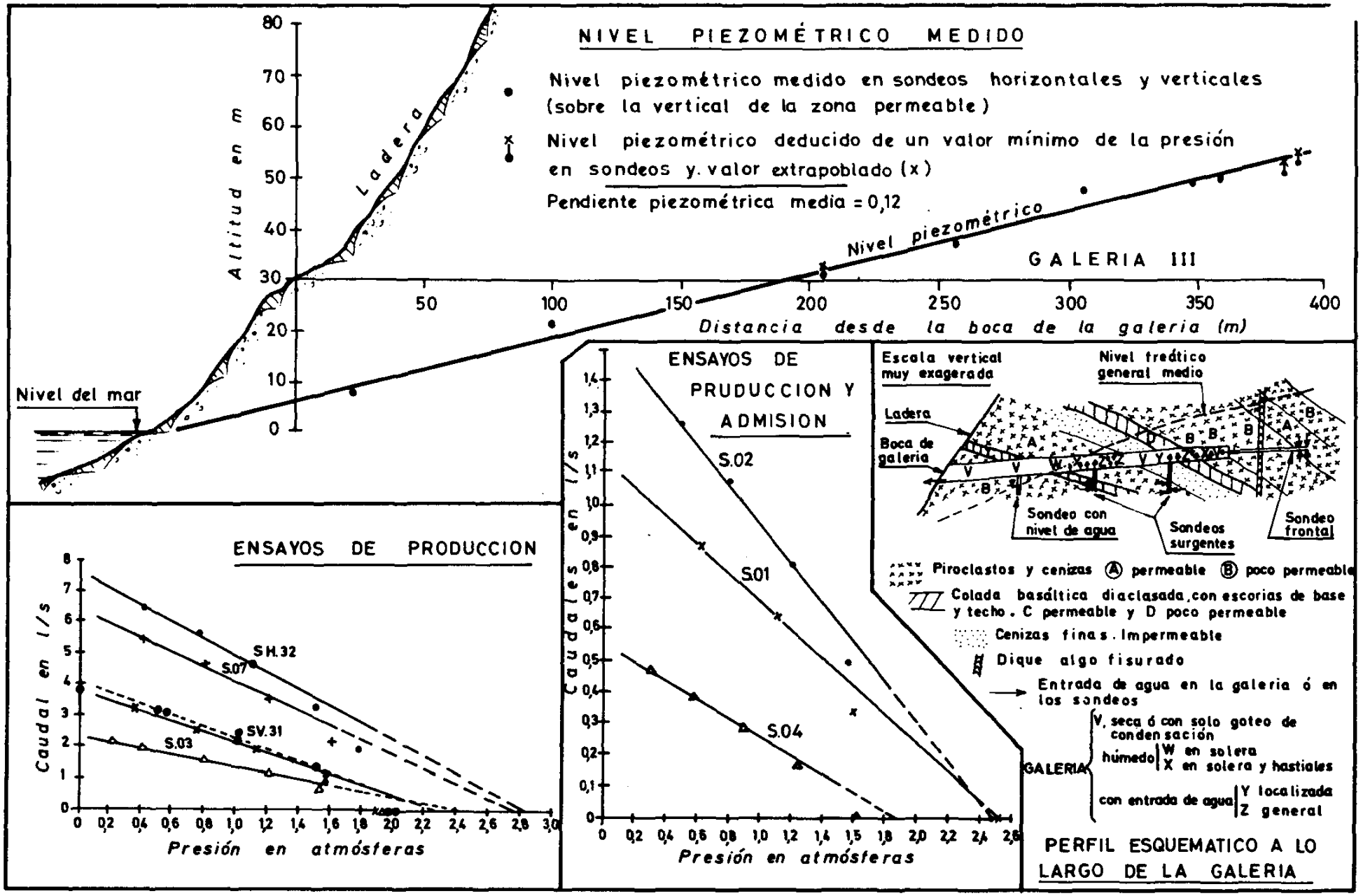

Fig. 6.— Ensayo de producción y admisión en los sondeos de la Galería III de Famara y perfil piezométrico medido con los datos de los sondeos

Dado que cuando el sondeo está cerrado o próximo a estar cerrado se producen pequeñas fugas por los cierres o a través de fisuras, no se llega a medir la presión máxima, pero esta puede deducirse de la extrapolación de la recta caudal-presión hasta el caudal nulo.

En principio no existe una relación clara entre orientación y posición del sondeo y la permeabilidad media. Los sondeos verticales se comportan de forma similar a los horizontales, si bien parece existir una tendencia no suficientemente confirmada a que aquellos aparezcan algo menos permeables. Es de destacar que a pesar de la notable heterogeneidad de materiales atravesados, distribución de zonas permeables y posición de los sondeos, la dispersión de valores de permeabilidad es muy pequeña.

8. - ASPECTOS GEOHIDROQUIMICOS DE LAS AGUAS DE LAS GALERIAS DE CAPTACION

Según Fuster, Fernández Santin y Sagredo (1968), los basaltos tabulares de Famara son de carácter alcalino con caracter frecuentemente de oceanitas, aunque hacia la base del macizo se presenta una mayor frecuencia de basaltos olivínicos. Se presentan inclusiones más alcalinas que corresponden a gabros y a veces a monzonitas nefelínicas.

El agua obtenída de las galerías es notablemente salina y su contenido en sales varía entre 1,4 y 8 g/l, aunque su composición iónica es similar, lo que demuestra una génesis común. Se trata de aguas cloruradas sódicas, con relación rNa/rCl (r = meq/l) > 1 para las menos salinas y < 1 para las más salinas. La relación rMg/rCa es notablemente mayor que 1. Al aumentar la salinidad, la concentración de Ca se mantiene aproximadamente constante y tiende a aumentar la relación rMg/rNa. La relación rSO_4/rCl se mantiene en el entorno del valor 7,2, menor que en el agua marina.

La salinidad es incluso elevada para los manantiales y rezumes de los acuíferos colgados, entre 0,8 y 1,3 g/l. Esta salinidad es de origen climático por fuerte concentración de un agua de lluvia que contiene entre 10 y 15 ppm de Cl^- en las zonas altas.

La salinidad aumenta rapidamente por debajo del nivel freático y también lateralmente al pasar a zonas menos permeables como se ha constatado en el estudio del agua de los sondeos de las galerías I

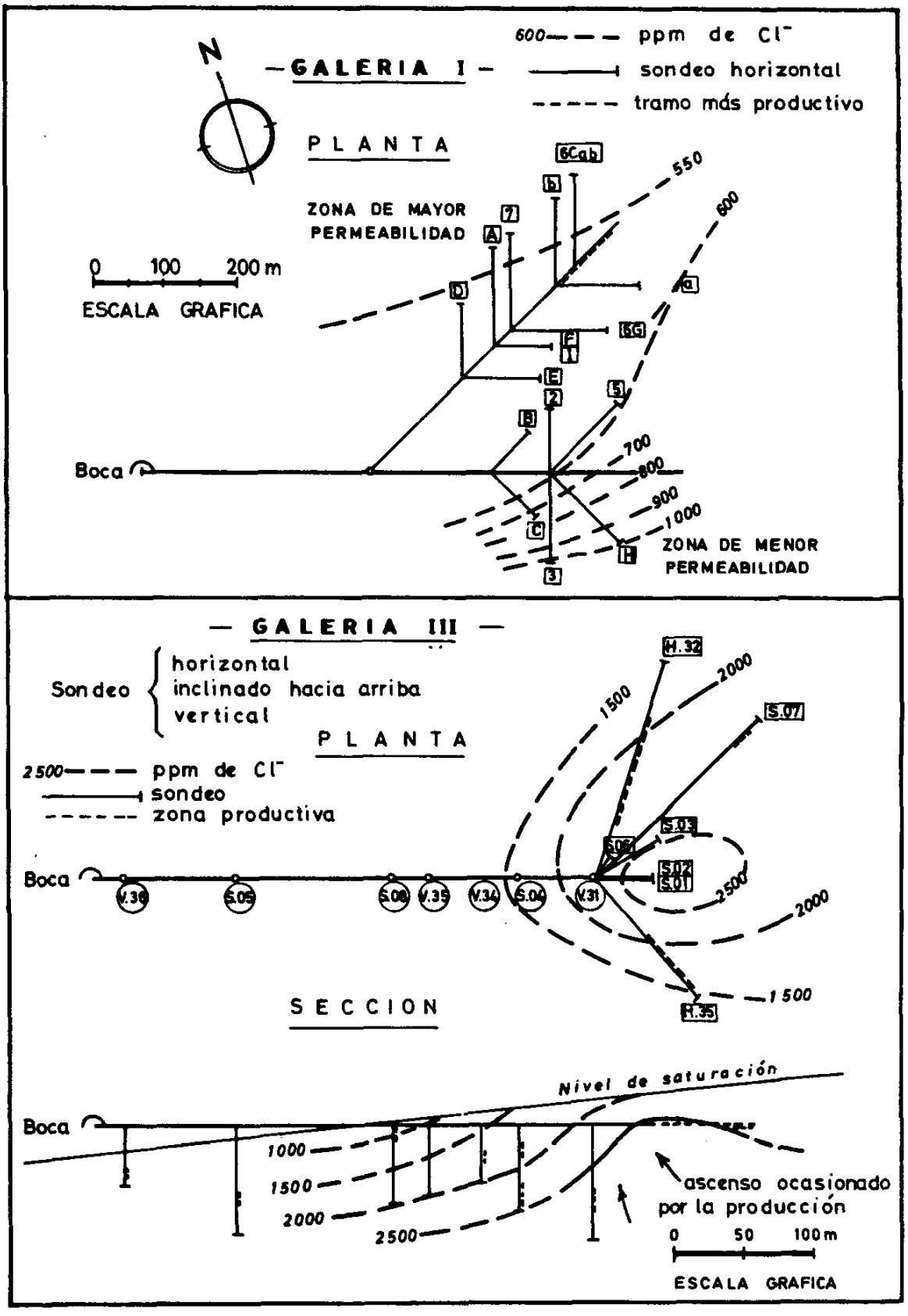

Fig. 7.—Distribución de la salinidad, en ppm de ion cloruro, en el entorno de las galerias I y III de Famara

(un lado es menos permeable que el otro) y III (existen sondeos verticales y la estratificación es clara a pesar de la alteración producida por la propia producción de la galería en su frente), tal como se muestra en la figura 7.

El contenído en tritio de estas aguas es nulo (los valores no nulos corresponden a contaminación atmosférica de las muestras) y está prevista una campaña de medidas de radiocarbono, cuya interpretación parece posible en las circunstancias geohidroquímicas del macizo.

Este incremento de salinidad parece que solo puede atribuirse a una lentísima velocidad de circulación del agua hacia el mar (Custodio, 1974c; 1978), con tiempos de residencia media en el terreno seguramente superiores a varios centenares de años y probablemente superiores a decenas de miles de años en las capas profundas o menos permeables.

El lavado de muestras de polvo permite determinar que su contribución a la salinidad es pequeña, en especial en ión cloruro. El lavado de muestras de roca quebrantada, utilizando agua neutra y acidulada, muestra igualmente un pequeño aporte de Cl^- y $SO_4^=$, lo cual señala que el incremento de salinidad antes mencionado en ausencia de aportes de esas sales por actividad volcánica, solo puede explicarse por una muy lenta circulación y un reducido número de renovaciones del agua en áreas profundas, a pesar del elevado gradiente piezométrico.

El contenído en fosfatos procedentes de la roca, es algo elevado, entre 0,3 y 1,2 ppm, y tiende a disminuir al aumentar la salinidad. La relación rCa/rSr que vale entre 500 y 1000 para las aguas menos salinas, disminuye hasta 100 para las aguas más salinas. La relación rCl/rBr vale del orden de 200 a 1000 para las aguas menos salinas, de acuerdo con el contenído en el agua de lluvia y también en el agua marina, que es la fuente de sales solubles que se incorporan al agua de lluvia. En las aguas más salinas la relación rCl/rBr crece hasta valores del orden de 10000, lo cual parece confirmar que el aumento de ión cloruro no es por aporte de sales marinas, en conformidad con lo que se puede deducir por otros caminos, sino de aportes del terreno.

El estudio de iones metálicos menores no aporta ningún nuevo dato de interés.

9. - AGRADECIMIENTOS

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WATER IN MINING

Chaturvedi, A. C.

ABSTRACT : The run off data for several parts of our watershed in the mining areas has estimated computerisation of automatic stream gages. It has found that flood peaks were considerably reduced by the reduction of the contributing drainage areas while the low flow was increased from 15 to 50 percent. The river basin management has vast scope with use of modern innovations for detailed investigations with remote sensind and satellite based systems for watershed management. The importance and utility of collection and utilisation of vast amount of hydrogeological and pumping data is brought out in the computer oriented study of these problems.

RESUME : On a estimé, par computation, à partir des enregistrements automatiques de jaugeage, les ruissellements de nos bassins versants dans des zones minières. On a trouvé que les maxima des crues diminuent sensiblement, par réduction des surfaces versantes de drainage, tandis que les débits de base augmentent de 15 à 50 %. La gestion du bassin hydrographique a un large champ d'action, avec les apports des satellites et des télédétections. On met en évidence l'importance et l'utilité de la collecte et de l'emploi de la grande quantité de données hydrogéologiques et de sondage, par le traitement en ordinateur de ces problèmes.

RESUMEN : Se han estimado, por computación a partir de los registros automáticos de aforo, las escorrentías superficiales de nuestras cuencas vertientes, en áreas mineras. Se ha encontrado que los máximos de las crecidas disminuyen sensiblemente, por reducción de las superficies vertientes de drenaje, mientras que los caudales de base se incrementan entre el 15 y el 50 %. La gestión de la cuenca hidrográfica tiene un ámplio campo de acción, con las aportaciones de los satélites y sensores remotos. Se pone de manifiesto, la importancia y utilidad de la recolección y empleo de la gran cantidad de datos hidrogeológicos y de bombeo, mediante el tratamiento en ordenador de estos problemas.

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INTRODUCTION

Mining prospecting has a four fold increase in the National Fifth Five Year Plan of India and surface water hydrology has a significant role in this development. In the Eastern coal-field areas of the states of West Bengal, Bihar and Assam along with the Western coal field areas of Maharastra, Madhya Pradesh and Rajasthan, extensive measurements were undertaken of the seepage flow in these mines. Automatic stream gaging was undertaken and the data computerised to afford valuable results on the surface water contribution to excavation and underground works. Mathematical models were simulated to afford studies on drainage in the basins. Strip mining is found to have increased the flooding in mountain areas. Reclaimed and unreclaimed mines have been found to produce acid water containing heavy metals, sulphates and iron. Radioactive elements are found to have been produced in many of our coal mines. The productivity of the strip mined land is found to have been reduced. Application of methematical models to drainage irrigation is discussed. The positioning of mine in the western regions of our vast country which offers vast scope for exploitation of new and valuable minerals is detailed with special emphasis on rehabilitation potential. (CHATURVEDI 76)

Study of groundwater drawdown and recovery are important in the surface mining of coal in Jharia area. The effect of surface mining on two watersheds in the Eastern and Western belt of India is reviewed. The measurements indicated loss in water resources of the region to be greater in the Bihar state than in the state of Maharastra where new studies are to be forged ahead in the coming years. The criteria for selection of mine site is laid down detailing the importance of rehabilitation potential.

SHAPE OF CATCHMENT AREAS

The catchment or drainage area from which water is collected in the Indian rivers varies widely from river to river. The catchment of the cauvery river is 87000 sq.km. The water from the catchment flows into the main river through a number of tributaries and subsidiary streams. The patterns of the tributary system depend on the physical characteristics of the area, nature of rocks and their erodibility. The shape of the catchment influences the run off pattern of the river. (CHATURVEDI 77).

The western ghats watershed is located in Southern India. The peninsular rivers fall into two categories namely the coastal and the inland rivers. The coastal rivers are comparatively small streams. While 600 of such rivers drain into the sea near the West Coast, they are of a great importance. Although only 3% of the areal extent of the basins of India is drained by these rivers, as much as 14% of the country's water resources are contained in them. The Narmada and Tapi, between flanking mountain ranges have narrow elongated catchments. Near the valley the till is covered by rolling sandy hills of wind blown deposits bottom lands consist of unconsolidated fluvial sands and gravels or silts and clays. These deposits are overlain in places by recent silty loam alluvium.

There were 20 stream flow gages in this watershed. Five gages were installed in 1940 and the remaining 15 during 1950-70. The effects of strip mining on hydrology of Periyar river were determined from analysis of stream flow records for sub basins having different exposures to strip mining. Variations in flow characteristics were related to areas stripped in the sub-basin after accounting for differences which might be attributed to other causes.(CHATURVEDI 77).

The annual run off and seasonal distribution are given in Table 1 and 2 below:

Table 1. Area drained by the rivers and their average annual run off.

Sl. No.	Name of the river	Drainage Area Sq/km.	Area within Kerala Sq/km.	Average Annual Run off.(TMC/M.Cum.
1.	Periyar	5200	1996	428/12120
2.	Bharatpurha	3800	1492	221.5/6270
3.	Chabiyar(Beypore)	2400	940	160.5/4540
		1950	753	
4.	Paruba	1950	753	
		1540	600	222.8/6310
		1500	548	145.0/4110
5.	Kalibari(East flowing)	1710	456	76.0/2150
6.	Kallada	1140	456	93.7/2650
7.	Muvattupurha	1100	430	92.3/2610
8.	Meenachil			76.0/2150
9.	Acherkoil			77.3/2190
10.	Kadalundi	1000	391	75.9/2150

Table 2. Seasonal Distribution of the yield of Principal rivers.

Sl. No.	Name of the river	Percentage Yield June-Sept.	Oct.-Dec.	Jan.-May	Year for which average has been worked out.
1.	Periyar	77.5	18.5	4.0	1969
2.	Bharatpurha	72.0	26.0	2.0	1968
3.	Chabiyar (Baypore)	81.5	7.0	11.5	1970
4.	Pemba	74.5	24.5	1.0	1967
5.	Kabbaru	22.5	77.5	-	1966
6.	Kallada	51.5	44.0	4.5	1968
7.	Muvathupurha	68.0	27.0	5.0	1962
8.	Meerachil	72.0	26.0	2.0	1960
9.	Achenkoil	59.1	39.7	1.2	1967
10.	Kadalundi	62.0	36.0	2.0	1964
11.	Chalakudy	79.0	19.0	2.0	1970

Periyar gage and Bharatpurha record flows were from large contributive areas with varied run off conditions. These records are representative of the effects of strip mining as well as structural changes which have taken place in these water sheds. (CHATURVEDI 77).

ANALYSIS OF STREAMFLOW DATA

Records for the gaging stations formed the principal basis for comparisons to determine the effects of strip mining. The two sub basins are similar to size and most physiographic features and reinfall was found to be uniform over the water shed. The average monthly and annual discharges data for the coincident period for the six gages in the watershed indicates significant difference in annual unit discharge from some of the sub-basins. The Periyar sub-basin and the Bharatpurha sub basins show average annual unit discharge 15% greater and 15% less than the third river. Average annual discharges for the other stations show minor differences. The differences in the average monthly run off is greatest in the driest months. For the sub-basins affected by the strip mining, higher unit discharges are obtained during the low flow periods. Graphs of rainfall discharges and evapotranspirations for 10 days spell have been drawn and the resulting changes in the water storages for 10 days and for the cumulative periods were also

plotted to demarcate water surplus and water deficit periods The study has revealed that sufficient change in water storage is available for consumptive use during the lean period. (CHATURVEDI 77).

These differences in the temporal distribution of flow are attributed to storage resulting from mining operations. Water is impounded in many small ponds and in the void area in the cast spoil. Much of the water is released over long periods as ground flow. Use of differential analysers, analogue and digital computors was traced for model studies. Ground water movement in different acquifers of varying geological strata called for use of isotopes and computors. The study of statistical dependencies and common mode failure provided interesting analysis. All these water resources problems were solved with availability series and parallel structures composed by repairable, unrepairable, and tested series. These studies were purposely designed to help us combat the absolescent malady offered a wide range of practical creative indepth learning the problem solving programme. The most probable effect of mining on water quality would be to reduce ground water discharge to Periyar Creek. Reduced discharge would provide less fresh waterto the lower reaches of the creek to dilute the saline water that is discharged to the surface water system in the North Central part of the basin. Based on current estimates the concentration of dissolved solids would be less than 5000 ml. per litre. Performance of the reservoir system design was evaluated by using historical stream flow data sequence with the implied assumption that performance under the past hydrologic input was the time indicator of the future performance. To design the system under the hydrological uncertainly therefore it was preferred to generate a number of synthetic streamflow sequence that were statistically indistinguishable from the historical sequence and use them in computor simulation of system design. The conjunctive use of the synthetic data and simulation hastened the decision process by providing a more thorough evaluation of the alternative designs.

The deficit index used herein was tantamount to a linear time invariant loss function in which water deficit were penalised proportionately, with a little modification in the computor programme, it became possible to use more complicated functions capable of measuring non linear and corelated economic losses. The suitable model for flow synthesis was first searched. For simulation of reservoir operations, it was considered necessary to use atleast 10 day period as the minimum time unit. Accordingly to use model for generation of 10 day period flow sequence was evolved or selected. From the detailed analysis of the data, Fierings multiseasonal model was adopted. Flow duration curves were plotted for unit mean daily discharges for each of the stream flow records. These show creeks having lesser flow at lower exceedence levels than does Bharatpurha creek. The

Periyar creek having larger flows at greater exceedence level (higher sustained flows). This was also attributable to the effects of storage in the watersheds which retard flood flows and increased dry season base flows by realeasing water from the storage when streams were effluent.

To efficiently allocate water to meet developing energy resources and energy conversion facilities required giving attention to the interlocking energy and water resource system. The water needs of the system already constituted the major demand for water in the state of Kerala. The optimisation model was used as an example of a preliminary screenining model to examine trends and trade offs associated with selected planning alternatives. An important characteristics of these optimisation models was that they could be used to select alternative development patterns from among the almost infinite number of solutions . Any one mathematical model however took into account only a few aspects of major policy questions and was used in conjunction with other quantitative and qualitative analysis procedures withi the planning activity.(CHATURVEDI 77).

Hydrographs exhibited both time and space changes resulting from the strip mining. Drastic changes in both stream chemistry and sediment load usually occurred downstream of the miped area thereby causing a significant alteration of the stream biota. Not all effects were negative in several observations increased ground and surface water storage resulted. The water availability in the Kerala State is the criterion that had to be considered first in any development supported for controlled by water. Because of the critical water balance in the future decades, the rules for water allocation and the constraints for water use were clearly defined at each stage of the plan. Preliminary values of future storage required under a 75% significant level were as given in table 3.

Table 3. Storage Requirements.

Decades	Years	Required storage (Cum./Sec.)
1	1970-79	50
2	1980-90	60
3	1991-2000	70
4	2001-2010	85
5	2010-2020	105
6	2021-2030	140

The hydraulic structures required to ensure this storage rep-

resented one of the main economic inputs of the whole plan. Finally the loss of water availability due to non predicted pollution was not considered nevertheless if the appropriate pollution controls were not established from the begining, then the balance was subject to change. The equilibrium between the annual total surplus and the total annual deficit was obtained, when the requirement function intercepted the long term mean of the water availabilities and this point gave the total duration of the plan under the hypothesis of self sufficiency. The assumption of annual balance was a restricted one. (CHATURVEDI 77).

MATHEMATICAL MODELS

Development and construction of systems for control of drainage basin waters for useful purposes is very essential in our state of Uttar Pradesh. The use of these analytical techniques required a close relationship with the data analysis activities. A mathematical model often aids in the determination of the amount, location and type of the data to be collected. But thinking in terms of the rivers model which very significantly from one to the other, collection of the real data is necessary These data help to shape the model and through verification analysis, indicate the appropriateness of the model. After verification some basis exists for making projections, on the effect of control procedures. The Jharia water shed is a system where the input is also the amount of the rainfall falling on the drainage area which transformed the input rainfall to an output namely runoff. In our dynamic model, the forcing function was time variable. Spatial gradients existed in the system. The fundamental idea in the deterministic relationship was to write a mass balance equation for the elemental volume which gave an account of all the materials entering and leaving the basin. Mathematically the system remained linear under the first order assumption which was useful as superposition was applied. For higher grder urn-linear type reactions simultaneous type of reaction simulative analysis became necessary.((CHATURVEDI 77).

Simulation models were effectively used for the determination of flow conditions in acquifers in the case of recharge for supply of ground water. Some important differences were found in the study of the model. Flood frequency curves of unit discharges above selected bases, partial duration series were plotted for Chebiyar, Bharatpurha, indicating widely diverse flood peak producing conditions. The resulting analytical expression was :

$Y = 34.2046\ e.^{0268}$ where Y represented the water demand in cubic metres/sec. and X was the time variable in years with X=1 for 1974. The annual water availability was considered as a normal variable with mean X= 701.10 cubic metres/sec. 1

and substandard deriv.tion S= 132,06 cubic metres/sec. For interpretation purposes the time period from the begining of the plan was divided into decades, the first one starting in 1950.

The probability distribution of the dry and wet years respectively was plotted for different stages of the rivers. The probabilities of dry and wet years for different decades were worked.out. The probability of having a drought or wet period of two years duration at the b.:.ining of the 6th decade was 0.7 x 10 and 1.26x10 respectively, lag times from the centroid of rainfalls to peak discharges were determined for the three creeks /br recorded coincident floods. Expected time lags were calculated using the lag time in hour values for the non mined east drainage areas, the coefficient was calculated to be 2.15. Disregarding the effects of strip mining, the Pamba and the Minachil creek drainage areas were hydrologically similar to the Chabiyar creek area. That is characteristics such as soil type, vegetal cover, geographical area and climatic exposures were the same. The variable factors such as sub-basin shape and drainage areas were accounted for.(CHATURVEDI 77).

The system stimulated by the model (38864 sq/ms.) was divided into 3440 model areas. Two dimensional non steady flow in non homogenous isotropic acquifer was simulated. The model was calibrated by comparing computor output heads in nodes with known water level elevations in 48 observations points. No attempt was made to achieve an absolute correspondence between the model and nature. A total of 60 separate computor runs of the model were made, reducing the average error from 5.7 to 3.8 m. The small number of calibration runs in this model was commensurate with the quality and quantity of the control points. Input parameters to the model were: infiltration rate, transmissivity and storage coefficient of the acquifer fixed surface water levels and selected nodes.

The computor yielded preliminary water balance steady state water level map, corrected transmissivity distribution modified recharge, infiltration to the acquifer. Twelve exploitation schemes were tested. Testing time was from 5 to 20 years with a cycling pumping pattern 6 months of pumping followed by 6 months of pumping followed by 6 months of idling. The amount of withdrawal was 17 to 30 cum. /sec. Several initial schemes were tested on a 40x26 array of model points and the last six on a 40x30 array. The model grid was plotted on the horizontal coordinate directions. The nodes were numbered from I=1 to I=401 and from J=1 to J=3.

The inclusion of the simulation by the model phase between the two actual investigation phases was of special importance. There were manifold benefits: confirmation of the conclusion

of the previous investigations in a regional sense, provision of guidelines for future data selection and preliminary location of well fields in an optimal manner for future operating conditions.

Acquifers were operated with increasing frequency in recent years as a sole source of large amounts of water in a conjunctive manner with surface water system. It has been more fully realised during the last decade that refined quantitative answers were need concerning available basin resources. Proper planning and management of basin water resources required the testing of all possible schemes and appraising of the merit of various alternatives. The fast speed hardware provided the basin hydrology and the resource engineers with an efficient too for evaluation of surface water resources and the consequences of the utilisation of the acquifers. The sequence of investigation followed in these models was adopted in other basins of the various states of the Indian Republic for evaluation and valuation of surface water potentialities in similar areas of hydrological structures size and economic status. The water balances, calculated by the computor at the end of every section in every tested scheme was plotted and gave the resultsfor the entire system in the exploitable conditions. (CHATURVEDI 77).

REHABILITATION

Surface rivers have led to environmental economic and social degradation of most of these locations. Land development programme have been developed and implemented in Kerala with extensive,coordinated successful effort in freezing injection and cementing. It is quite likely therefore that some perhaps, most future surfaces mines will affect surrounding areas in a similar manner. The conditions of the site at the time of disturbance were restored afterwards. The site was habitable to organisms either originally present or similar to the original inhabitants. After rehabilitation the land was returned to a farm and productivity that conformed with a prior land use which accounted for establishing a stable, environmentally acceptable ecological state. Surface mines in the water abundant (typical mean annual precipitation of 88 cm.) western Kerala affected with water quality and quantity within the stream channel. In using the water allocation model, the change on the limits of the total. Withdrawals from the given river was shown and the different model solutions offerred the impact on the water allocation pattern of the region. The different solutions specified different locations on the water facilities. The model dealt with the water source points which included the major river reaches groundwater acquifers and potential reservoir sites,municipal, and non power industrial water demand points and load centres for electric-power demands.

The linear programming model minimised the sum of the transport-

ation cost of all of the commodities and the site dependent costs of supplying water. These costs depended on the flow paths selected by the model for the four commodities and these flow paths depended in part on the locations selected for the energy facilities. The site dependent cost for energy facilities were readily incorporated into the cost coefficients of the model. (CHATURVEDI 76).

CONCLUSION

The control of ground water pressure, is an important component activity for mining in South India. Since the commencement of mining operations a lot of hydrological and pumping data has been collected since 1940. With the availability of the data and the likelihood of more severe hydrological conditions being encountered in mining in the coming decade, a computor oriented study of the problem of mine water control was initiated.

Reservoir systems are to be designed in the face of hydrologic uncertainty. An investigation into the effect of this uncertainty on the performance of a system design is the basic objective. Synthetic streamflow data were generated by evolving a flow model and alternate sequences were used in simulation of reservoir operations. The performance of the design was measured by the deficit index. Variat on in the deficit index when alternate synthetic data sequences were used reflected the hydrologic uncertainty involved. An objective approach to the determination of the adequacy of synthetic data sequences was worked in terms of the desired accuracy of the estimate of deficit index. The extreme low flow volume for Bharatpurha creek were 5 to 15 times greater than the corrêsponding values on the Chabiyar creek. In each case the comparison of the least square feet time and the equal yield line shows for the sub basins affected by strip mining, higher unit discharge during low flow periods.

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SECADO DEL MINERAL LIMONITICO DE LOS YACIMIENTOS DE LA SIERRA MENERA (TERUEL Y GUADALAJARA, ESPAÑA)

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ABSTRACT: The iron ore mined in the "Ojos Negros" mines retains a certain quantity of moisture, despite on the spot drainage by means of pumping in boreholes. This moisture can give rise to problems in the manipulation and treatment of the ore. In this paper the percentages of the different types of water content (retention, capillary and gravity) have been studied in different hydrogeological conditions of the ore deposit: below or above the original water level; before and after drainage in the mines; and in areas exposed to the air. We describe several standardized series of experiments conducted on industrial scale. Then we analyse the effects on the moisture content of removal of the mineral, pile depth, orientation, also with respect to wind and sun radiation, atmospheric pressure, rainfall, temperature and relative humidity of the air.

RESUME: Le minerai de fer exploité dans les mines de "Ojos Negros", malgré le drainage in situ, au moyen de pompages en sondages, conserve un pourcentage d'humidité qui peut poser des problèmes dans leur manipulation et traitement. On étudie les pourcentages des différents types d'eau (rétention, capillaire et gravifique) dans les différentes conditions hydrogéologiques du gisement (au-dessous ou au-dessus du niveau piézometrique, avant et après le drainage in situ; aéré). On décrit une serie d'essais d'aération à echelle industrielle et on analyse les effets sur l'humidité, en conséquence du remuement; profondeur dans la pile; localisation en ce qui concerne le vent et l'insolation; pression atmosphérique; pluie; température et humidité relative de l'air.

RESUMEN: El mineral de hierro explotado en las minas de "Ojos Negros", no obstante el drenaje in situ, mediante bombeos en sondeos, conserva un porcentaje de humedad que puede plantear problemas en su manipulación y tratamiento. Se estudian los porcentajes de los diferentes tipos de agua (retención, capilar y gravífica), en las distintas condiciones hidrogeológicas del yacimiento (bajo o sobre el nivel piezométrico; antes y después del drenaje; oreado). Se describen una serie de ensayos de oreo, a escala industrial, y se analizan los efectos sobre la humedad, como consecuencia del removido; profundidad en el apile; localización con respecto al viento e insolación; presión atmosférica; lluvia; temperatura y humedad relativa del aire.

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PLANTEAMIENTO

La Compañía Minera de Sierra Menera S. A. explota un conjunto de minas, que se localizan a cotas superiores a 1.300 m, a lo largo de la alineación montañosa, de directriz NNW-SSE, que marca la divisoria hidrológica e hidrogeológica entre la cuenca atlántica y la mediterránea, en las provincias de Teruel y Guadalajara (figura 1)

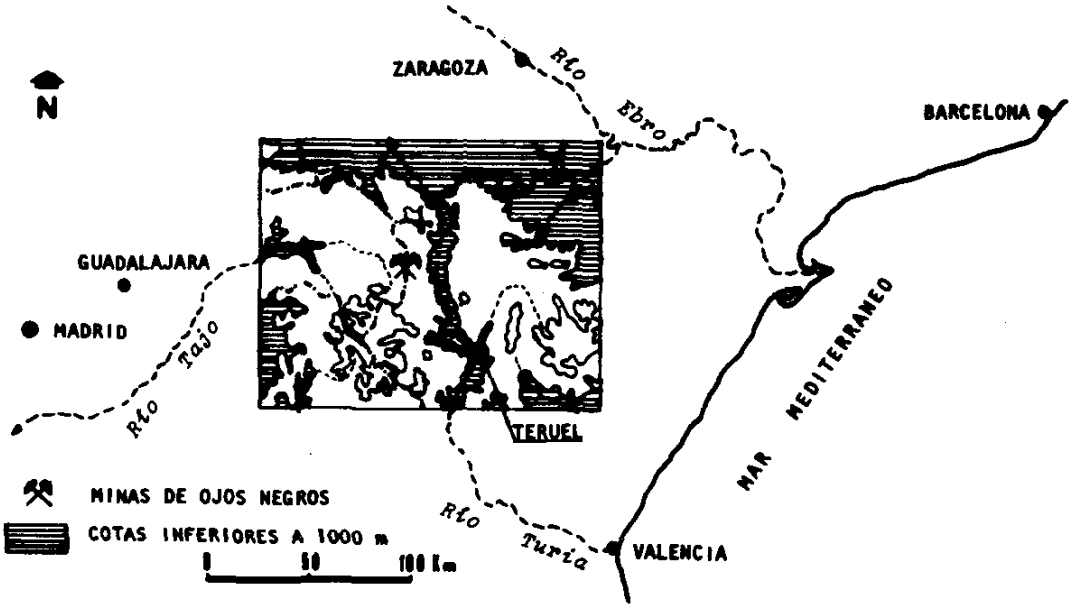

Figura 1. Plano de situación.

Estos yacimientos se ubican en el Asghilliense (Ordovícico), y están constituidos, fundamentalmente, por goethita (con ley media superior al 50 % de Fe), que cambia lateralmente a arcilla o a formaciones carbonatadas (con todos los términos entre la dolomía y la magnesita). Al techo se localizan cuarzoarenitas (y localmente pizarras) del Valentiense, mientras que al muro lo hacen carbonatos y margas briozóicas, que descansan sobre areniscas y grauwacas del Caradociense. El conjunto tiene a su techo pizarras del Ludlowiense-Wenlokiense, y al muro también pizarras del Llandeiliense (Figura 2).

Grandes masas de este mineral, compartimentadas por fallas, se encuentran bajo nivel piezométrico, lo que ha exigido la puesta a punto de un dispositivo de drenaje, mediante bombeos en sondeos, cuya eficacia está suficientemente contras-

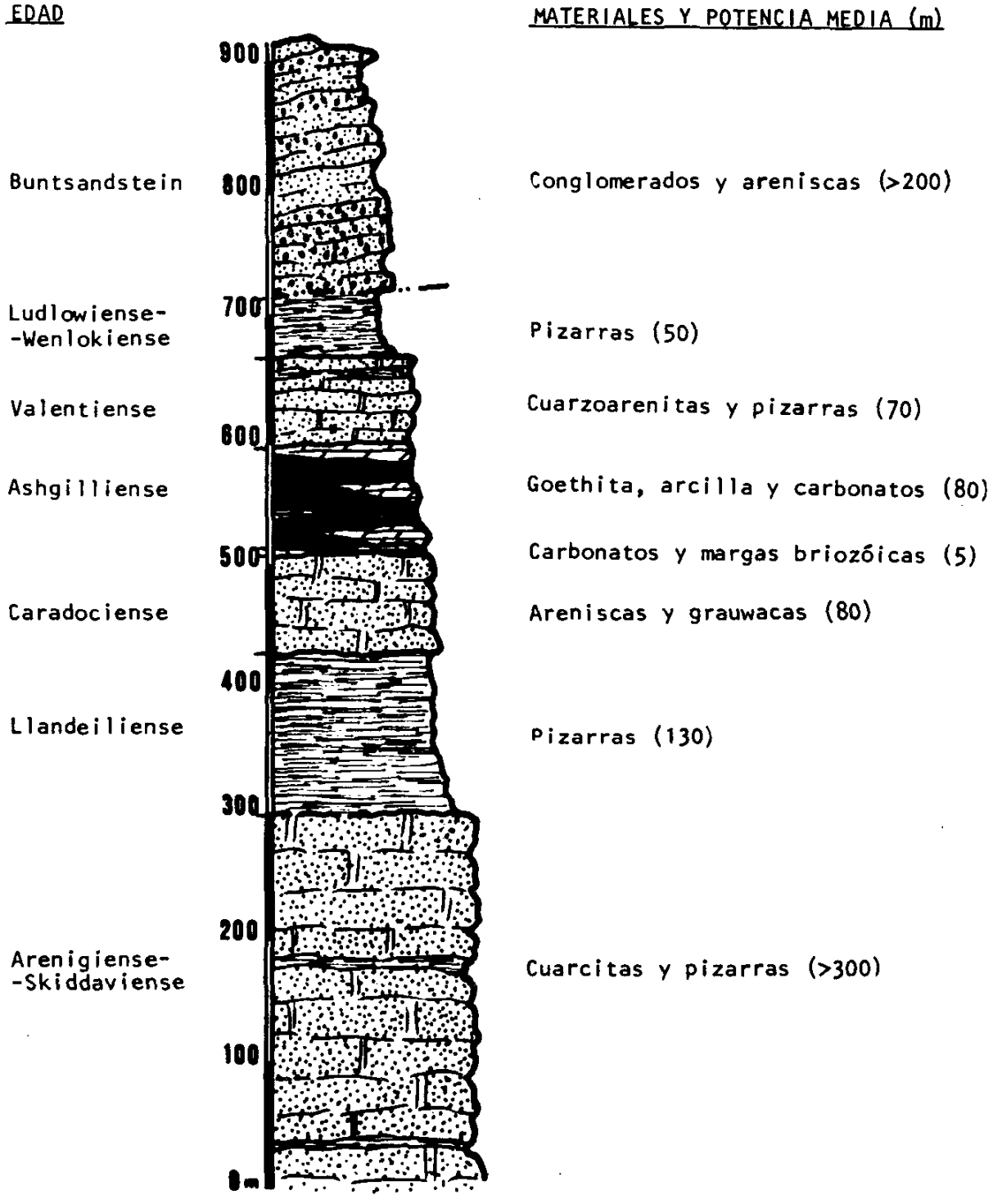

Figura 2. Columna estratigráfica

tada por varios años de explotaciones.

No obstante este drenaje, el mineral retiene un porcentaje de humedad que, cuando supera al 19 %, puede plantear problemas de manipulación en la Planta de Homogeneización y Cribado.

CARACTERISTICAS HIDROGEOLOGICAS DEL MINERAL

Los hidróxidos de hierro que nos ocupan tienen textura pulverulenta, con acusada heterometría y abundancia de tamaños finos (Figura 3), de tal manera que el mineral presenta porosidad capilar y subcapilar (BOGOMOLOV, 1958), lo que condiciona una notable retención de agua en los poros, y una porosidad total muy superior a la eficaz.

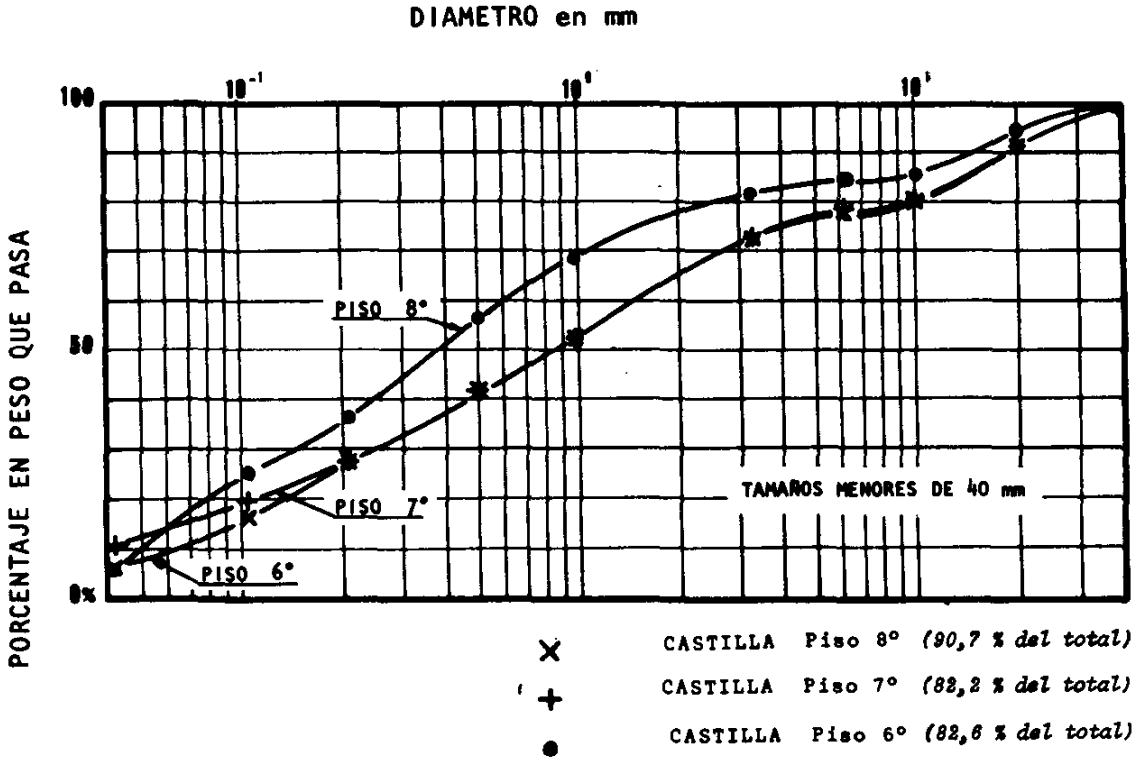

Figura 3. Curvas granulométricas acumulativas (medias de 12 muestras por curva).

Coherentes con estos datos son los resultados de los ensayos de permeabilidad, realizados in situ, y que han proporcionado para la conductividad hidráulica de esta goethita valores medios de 10^{-5} cm/seg (FERNANDEZ-RUBIO, 1974), los cuales

corresponden, en la clasificación de CASAGRANDE (1940), a *drenaje pobre*.

En estas condiciones, y de acuerdo con la información acumulada en las explotaciones, y las medidas que hemos realizado en laboratorio, podemos establecer el siguiente cuadro (FERNANDEZ-RUBIO, 1975):

	PORCENTAJES DE AGUA EN PESO			
Tipo de agua en la goethita	in situ bajo nivel piezométrico	in situ drenada	drenada y oreada	sobre nivel piezométrico natural
Retención Capilar Gravífica	10 - 15 8 - 10 15 - 18	10 - 15 8 - 10	10 - 15 3 - 6	10 - 15
Total	33 - 43	18 - 25	13 - 21	10 - 15

El *agua de retención*, higroscópica y capilar, es beneficiosa al evitar problemas ambientales en la manipulación del mineral (formación de polvo).

Por lo que respecta al *agua capilar*, y especialmente en los minerales de grano más fino, es necesaria una desecación, para evitar los problemas que ocasiona el tratamiento del mineral (tupido de cribas y atasco de molinos).

El *agua gravífica* del mineral se consigue eliminar, en general, mediante los bombeos en sondeos de captación, ubicados en los carbonatos karstificados del Ashgilliense, en contacto con el mineral (con un bombeo específico, en 1977, de 325 litros/día/tm de mineral, para una explotación de 8.000 tm/día de mineral). En todo caso, y dada la baja conductividad hidráulica de esta goethita, y la consiguiente lentitud del *goteo*, se requiere efectuar este drenaje con suficiente antelación en el tiempo, antes de la explotación.

A consecuencia de estas distintas categorías de agua, se establece una diferenciación muy neta, en las explotaciones, entre el mineral que se localiza encima del nivel piezométrico natural (*zona no saturada*), con humedad comprendida, en general, entre el 10 y el 15 %, de aquel otro mineral que se encontraba bajo dicho nivel piezométrico, y que si bien ha sido drenado por los bombeos, retiene cierta cantidad de *agua capilar*, por lo que su humedad in situ alcanza valores entre el 18 y el 25 %.

Así pues, una vez conseguida la puesta a punto del dispositivo de drenaje del *agua gravífica*, mediante los referidos bombeos desde los carbonatos, se planteaba la necesidad de reducir el *agua capilar* residual.

ENSAYOS DE SECADO EN TROMEL

Con objeto de estudiar, desde el punto de vista técnico y económico, la posibilidad de secado del mineral en tromel, con una contracorriente de aire caliente, se realizaron siete ensayos en una planta experimental (KOERTING, 1976).

En estos ensayos, a partir de un mineral con humedad media del 20,6 %, se obtuvo un producto final con humedad del 10,8 %, lo que exigió un consumo de 970 kcal por kilo de agua evaporada.

La conclusión de estas pruebas fué negativa, debido a que el tratamiento origina un importante aumento de tamaños finos, supone un coste elevado de combustible, y la producción es reducida en comparación con el tonelaje que se debería tratar.

ENSAYOS DE SECADO POR OREO

Ubicación

Con el doble objetivo de rebajar la humedad, y de favorecer la homogeneización del mineral, se decidió la realización de ensayos controlados de oreo, en condiciones ambientales (FERNANDEZ-RUBIO y SASTRE, 1976).

Estos ensayos se han efectuado con mineral drenado por el bombeo, procedente de las explotaciones de la Mina Castilla (piso 8°) y de la Mina Corral (piso 5°), sobre un tonelaje de unas 1700 toneladas en el primer ensayo, y de 350 en el segundo.

Los ensayos se realizaron sobre una ámplia superficie explanada, en una escombrera de estériles, a la cota 1.300 m, en un sector sin obstáculos para los vientos del Norte, Este y Sur.

Manipulación del mineral

El primer ensayo se realizó con cinco apiles (números 1 a 5), de 64 x 5 m de base y de 2 m de altura. El segundo ensayo se efectuó sobre un apile semejante (número 6). Estos apiles, con superficie de oreo de aproximadamente 400 m^2, se dispusieron con el eje mayor en dirección Este-Oeste.

En el primer ensayo los apiles fueron sometidos, en diferentes fechas, a un removido mediante pala, mientras que no se alteró la situación del apile 6

Número de apile	1	2	3	4	5	6
Fecha de colocación	26/10/76					07/11/76
Fecha de manipulación	27/10	28/10	02/11	03/11	04/11	no
Días transcurridos	1	2	7	8	9	no

Control de humedad

En los cinco primeros apiles (primer ensayo), se efectuó una toma de seis muestras diarias, para cada uno de los cuatro costados, a profundidades de 0 - 30 cm, 30 - 60 cm y 60 - 90 cm. Igualmente, en el sexto apile (segundo ensayo), se tomaron seis muestras diarias sobre los costados Norte y Sur, y cuatro sobre los Este y Oeste, a profundidades de 0 - 10 cm, 10 - 20 cm y 20 - 30 cm.

Los resultados de los análisis de humedad se expresan en las tablas 1 y 2.

Control meteorológico

Para analizar la posible incidencia de las condiciones atmosféricas se instaló, en el área de ensayo, el siguiente instrumental de observación:

- Pluviómetro Hellman
- Termómetro de máxima y mínima
- Termohigrómetro
- Barómetro

Los datos registrados, durante los ensayos, se expresan en las tablas 1 y 2.

Estas observaciones suplementaron a las que se realizan, de manera sistemática, para el control de las precipitaciones en las áreas de explotación (Figura 4).

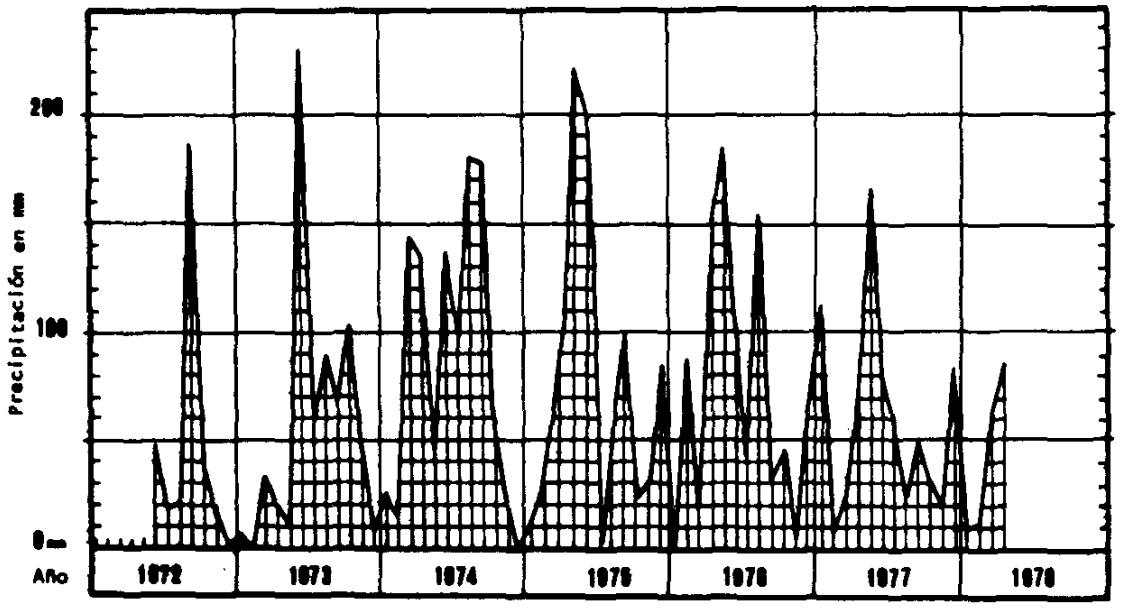

Figura 4. Precipitaciones mensuales en Sierra Menera (Mina Castilla)

DISCUSION DE RESULTADOS

De acuerdo con los factores controlados, vamos a considerar, sucesivamente:

- *Efectos antrópicos*
  - . *removido del apile*
- *Efectos de localización*
  - . *orientación*
  - . *profundidad*
- *Efectos climáticos*
  - . *presión atmosférica*
  - . *lluvia*
  - . *temperatura*
  - . *insolación*
  - . *humedad atmosférica*

No podemos considerar el efecto de la velocidad y dirección del viento, por no disponer de dicha información en el área de ensayo.

Aunque discutiremos por separado cada uno de estos factores, es lógico pensar en un efecto combinado de los mismos, con mayor o menor intensidad de cada uno de ellos, y así lo señalaremos en su momento.

Efectos del removido del apile

Según hemos expuesto anteriormente, los apiles del primer ensayo (1 a 5), fueron sometidos a removido en las fechas que se indican .

En la tabla 1 se expresan los análisis diarios de humedad, para cada apile de este primer ensayo, y se identifican en letra cursiva los que corresponden al día siguiente a mezclar el apile con pala.

En el apile 1 la remoción coincidente con un día de lluvia no supuso aumento de humedad, si no todo lo contrario. En el apile 2, en día sin lluvia, baja la humedad 1,7 unidades. En los apiles 3 y 4, removidos durante la lluvia, se advierte un aumento de humedad de 2 y 2,5 unidades, superior a la de los apiles no removidos durante dicha lluvia.

Para interpretar estos datos, esenciales, pensamos que hay que tener en cuenta que, en el caso del apile 1, se partía de una humedad media del mineral alta (22,3 %), mientras que en la segunda lluvia, a pesar de ser más escasa, el mineral se encontraba ya más seco (18,8 % en el apile 3, y 17,8 % en el apile 4).

Si bien son insuficientes los datos disponibles para establecer conclusiones definitivas, sobre el efecto de deshumectación por removido del apile, si parece atestiguarse la conveniencia de no remover el mineral en días de lluvia.

Efecto de la orientación

Como se ha indicado anteriormente, los apiles se dispusieron con su eje mayor en dirección E - W, lo cual condiciona el máximo desarrollo superficial en los costados Norte (*umbría*) y Sur (*solana*).

También hay que indicar la escasa superficie de las extremidades Este y Oeste, lo que implica una menor representatividad del muestreo en ellas (toma de sólo cuatro muestras, frente a seis en los otros costados), especialmente después de realizar tomas en varios días sucesivos, ya que la reducida superficie de dichas extremidades se encuentra agujereada por la introducción del tomamuestras.

Hecha esta salvedad, y por lo que al segundo ensayo se refiere, único en el que se ha muestreado independientemente en cada costado del apile, se observa con nitidez (Figura 6), un secado más eficaz en el flanco orientado al Sur, que en el orientado al Norte, ya que en el primero la humedad media pierde 3,6 unidades, y en el segundo sólo 0,7 unidades (para la media total ha supuesto un descenso de 2,6 unidades).

Este secado del costado Sur del segundo ensayo, es semajante al obtenido para la media del primer ensayo, con los cinco apiles (2,2 unidades), que se muestrearon siempre sobre el costado Sur.

Se confirma, por tanto, el secado más intenso del borde de los apiles orientados al Sur (*solana*).

Efecto de la profundidad

Uno de los aspectos más concluyentes, en los dos ensayos realizados, es el relativo al aumento del secado en las capas más superficiales (tablas 1 y 2 y figuras 5 y 6).

Con referencia al efecto de profundidad hay que tener en cuenta que, según DAVIS y DE WIEST (1971), la acción evaporante es inferior a un metro en suelos arenosos, e inferior a tres metros en suelos arcillosos. Para otros autores esta acción evaporante se reduce, como promedio, a un metro en regiones templadas, y a dos metros en zona árida. Aquí tendríamos que tener en cuenta una mayor penetración del efecto de secado, por el aumento de porosidad, consecuencia del arranque, transporte y apile del mineral.

Efecto de la presión atmosférica

Durante el periodo de observación las variaciones de la presión atmosférica han sido, realmente, muy reducidas, ya que sólo han oscilado entre 768 mm y 778,8 mm.

Los resultados adquiridos no permiten establecer una relación con el efecto de secado, que puede estar más condicionado por otros factores climáticos.

TABLA 1

PRIMER ENSAYO DE OREO EN GOETHITA (APILES 1 a 5) Año 1976										
% HUMEDAD / DIAS		27/10	28/10	29/10	30/10	02/11	03/11	04/11	05/11	12/11
APILE 1	00 - 30 cm	22,1	*22,0*	19,7	18,7	16,2	14,2	14,1	14,4	14,7
	30 - 60 cm	22,3	*20,3*	21,5	20,5	17,2	18,6	19,2	17,2	16,5
	60 - 90 cm	22,5	*22,0*	21,3	20,2	18,2	20,0	20,8	19,8	18,5
	media general	22,3	*21,4*	20,8	19,8	17,2	16,6	18,0	17,1	16,6
APILE 2	00 - 30 cm	21,6	23,6	*21,7*	19,8	16,2	17,1	17,6	17,5	15,0
	30 - 60 cm	21,8	23,4	*21,6*	21,1	16,3	18,9	20,1	20,1	16,2
	60 - 90 cm	22,1	23,0	*21,5*	21,7	20,3	20,5	20,4	20,4	19,6
	media general	21,8	23,3	*21,6*	20,9	17,6	18,8	19,4	19,3	16,9
APILE 2	00 - 30 cm	19,9	23,5	21,6	19,4	17,3	*19,6*	18,0	18,7	16,4
	30 - 60 cm	21,7	23,0	21,8	20,2	18,1	*21,6*	20,0	20,8	16,5
	60 - 90 cm	21,8	23,2	22,6	22,3	20,9	*21,3*	20,2	16,4	16,1
	media general	21,1	23,2	22,0	20,6	18,8	*20,8*	19,4	18,6	16,3
APILE 4	00 - 30 cm	20,6	23,7	21,3	18,0	18,3	15,5	*20,7*	18,9	14,6
	30 - 60 cm	19,6	23,0	21,6	20,4	19,6	18,4	*19,0*	20,9	17,8
	60 - 90 cm	18,9	23,4	22,3	22,0	21,3	19,4	*21,3*	21,0	18,1
	media general	19,7	23,2	21,7	20,1	19,7	17,8	*20,3*	20,2	16,8
APILE 5	00 - 30 cm	21,6	23,5	22,2	18,1	16,6	17,0	16,9	*19,9*	20,2
	30 - 60 cm	22,4	23,7	22,5	21,6	18,2	20,6	19,6	*21,3*	20,7
	60 - 90 cm	22,9	23,4	22,6	22,3	22,3	21,7	21,5	*21,5*	21,1
	media general	22,3	23,5	22,4	20,7	19,0	19,8	19,3	*20,9*	20,7
DATOS GLOBALES	media 00 - 30 cm	*21,2*	*23,3*	*21,3*	18,8	16,9	*16,7*	*17,5*	*17,9*	16,2
	media 30 - 60 cm	*21,6*	*22,7*	*21,8*	20,8	17,9	*19,6*	*19,6*	*20,1*	17,5
	media 60 - 90 cm	*21,6*	*23,0*	*22,1*	21,7	20,6	*20,6*	*20,8*	*19,8*	18,7
	media general	*21,5*	*23,0*	*21,7*	20,4	18,5	*19,0*	*19,3*	*19,3*	17,5
DATOS EXCLUIDO REMOVIDO	media 00 - 30 cm	21,2	23,6	21,2	18,8	16,9	16,0	16,7	17,4	16,2
	media 30 - 60 cm	21,6	23,3	21,8	20,8	17,9	19,1	19,7	19,8	17,5
	media 60 - 90 cm	21,6	23,3	22,2	21,7	20,6	20,4	20,7	19,4	18,7
	media general	21,5	23,4	21,8	20,4	18,5	18,5	19,0	17,8	17,5
Temperatura máxima (°C)		*	*	13,0	8,0	10,0	11,0	9,0	11,0	11,0
Temperatura mínima (°C)		*	*	3,5	1,0	4,0	4,0	5,0	7,0	5,0
Pluviometría (mm)		0,0	3,6	0,0	0,0	0,0	0,0	0,1	0,3	0,0
Humedad relativa (%)		*	*	66,5	98,0	98,0	82,3	83,5	90,5	81,0
Presión atmosférica (mm)		*	*	772,0	768,0	772,0	771,2	770,5	772,2	770,0
Hora de toma		9^h30	9^h30	9^h30	9^h30	9^h30	9^h30	9^h30	9^h30	9^h30

* sin datos

en cursiva valores excluidos del cálculo

TABLA 2

SEGUNDO ENSAYO DE OREO EN GOETHITA (APILE 6) Año 1976

% HUMEDAD \ DIAS		08/11	09/11	10/11	11/11	12/11	13/11	15/11	16/11	17/11	18/11	19/11
COSTADO NORTE	00 - 10 cm	21,5	20,7	19,1	21,7	23,3	21,4	21,9	21,6	18,3	19,7	20,6
	10 - 20 cm	21,4	21,3	19,1	21,8	23,1	20,5	22,3	19,8	20,6	20,4	21,1
	20 - 30 cm	21,2	22,0	21,2	17,5	22,6	21,3	21,9	20,6	21,9	20,2	20,4
	media general	21,4	21,3	19,8	20,3	23,0	21,1	22,0	20,7	20,3	20,1	20,7
COSTADO SUR	00 - 10 cm	21,1	21,0	18,4	19,1	19,0	14,1	17,0	13,0	12,8	13,8	15,1
	10 - 20 cm	21,4	20,4	19,3	18,6	21,2	19,1	19,5	18,9	17,8	17,6	18,3
	20 - 30 cm	22,0	20,5	19,3	19,3	21,1	20,1	20,7	19,4	19,9	19,8	20,3
	media general	21,5	20,6	19,0	19,0	20,4	17,8	19,1	17,1	16,8	17,1	17,9
COSTADO ESTE	00 - 10 cm	18,5	21,7	20,8	20,5	19,9	19,3	14,7	18,0	14,4	17,8	18,4
	10 - 20 cm	21,2	21,5	19,5	19,9	20,2	18,9	18,1	15,1	18,9	20,3	18,5
	20 - 30 cm	22,5	21,4	18,9	20,6	20,3	19,6	19,9	19,1	18,9	20,4	20,6
	media general	20,7	21,5	19,7	20,3	20,1	19,3	17,6	17,7	17,4	19,5	19,2
COSTADO OESTE	00 - 10 cm	23,3	22,3	20,0	22,4	21,4	19,0	18,8	16,4	15,9	11,7	16,2
	10 - 20 cm	22,5	22,3	18,3	18,7	21,8	21,5	20,9	20,0	19,2	17,6	18,5
	20 - 30 cm	22,4	*	17,1	17,3	21,1	21,0	20,9	20,9	20,0	19,1	20,0
	media general	22,7	22,3	18,5	19,5	21,4	20,5	20,2	19,1	18,4	16,1	18,2
MEDIA GLOBAL	00 - 10 cm	21,1	21,4	19,6	20,9	20,9	18,5	18,1	17,3	15,3	15,8	17,6
	10 - 20 cm	21,6	21,4	19,5	19,7	21,6	20,0	20,2	18,5	19,1	19,0	19,1
	20 - 30 cm	22,0	21,3	19,1	18,7	21,3	20,5	20,9	20,0	20,2	20,0	20,3
	general	21,6	21,4	19,3	19,8	21,3	19,7	19,7	18,6	18,2	18,2	19,0
Temperatura máxima (°C)		15,5	12,0	11,0	8,0	11,0	11,0	11,0	9,0	11,5	11,0	7,0
Temperatura mínima (°C)		2,0	6,5	3,5	0,5	5,0	2,0	1,0	-1,5	-0,5	0,0	-2,0
Pluviometría (mm)		2,9	0,8	1,3	1,1	3,3	0,0	0,0	0,0	0,0	0,0	0,0
Humedad relativa (%)		84,0	95,0	99,5	99,5	81,0	96,0	91,0	66,0	72,0	71,0	84,0
Presión atmosférica (mm)		770,5	769,5	768,5	770,0	770,0	773,0	773,5	778,8	774,5	774,0	769,0
Hora de toma		11^h00	10^h00	10^h00	10^h25	12^h25	11^h00	10^h45	17^h00	16^h00	16^h00	16^h00

* sin datos

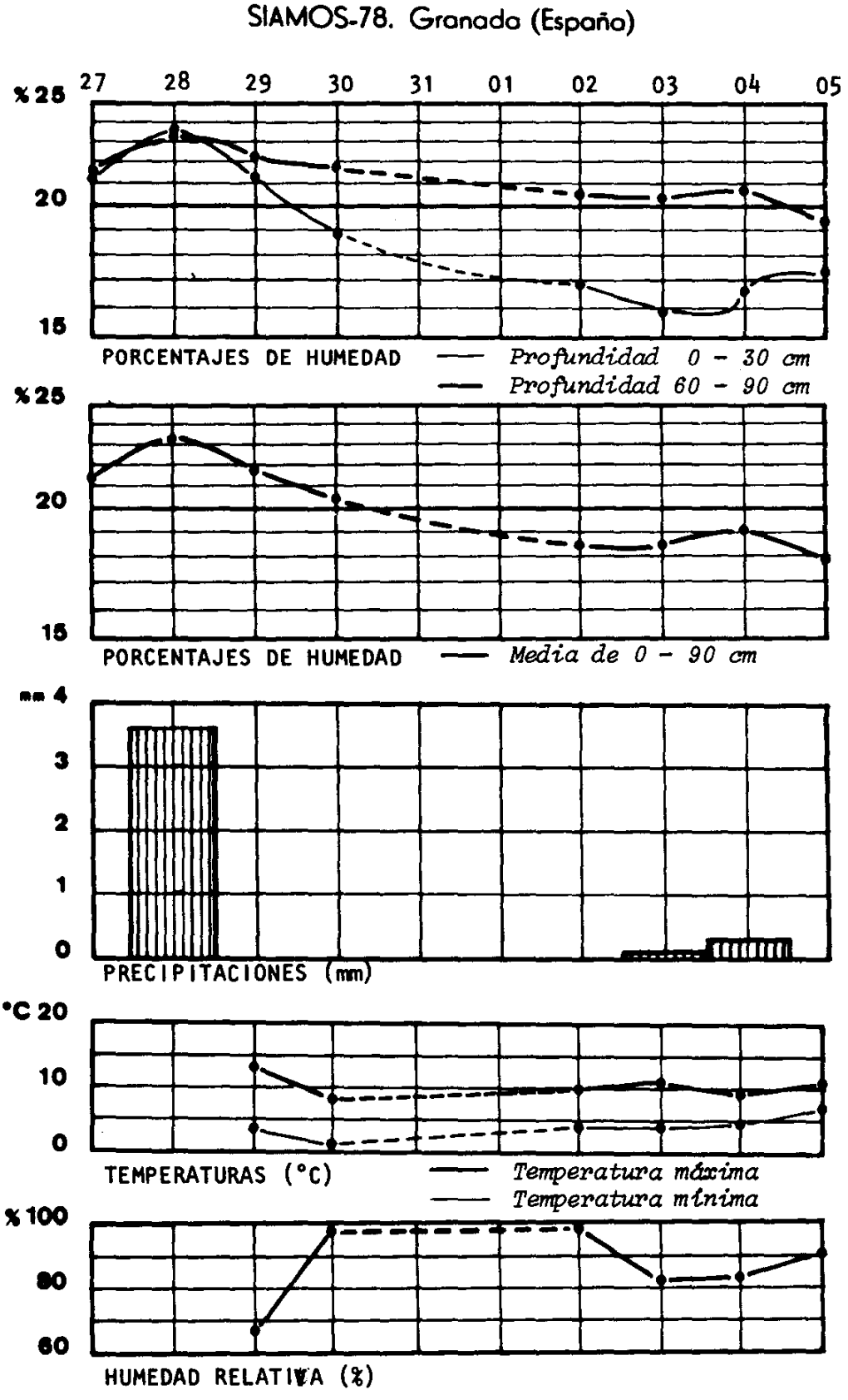

Figura 5. Resultados y observaciones del primer ensayo.

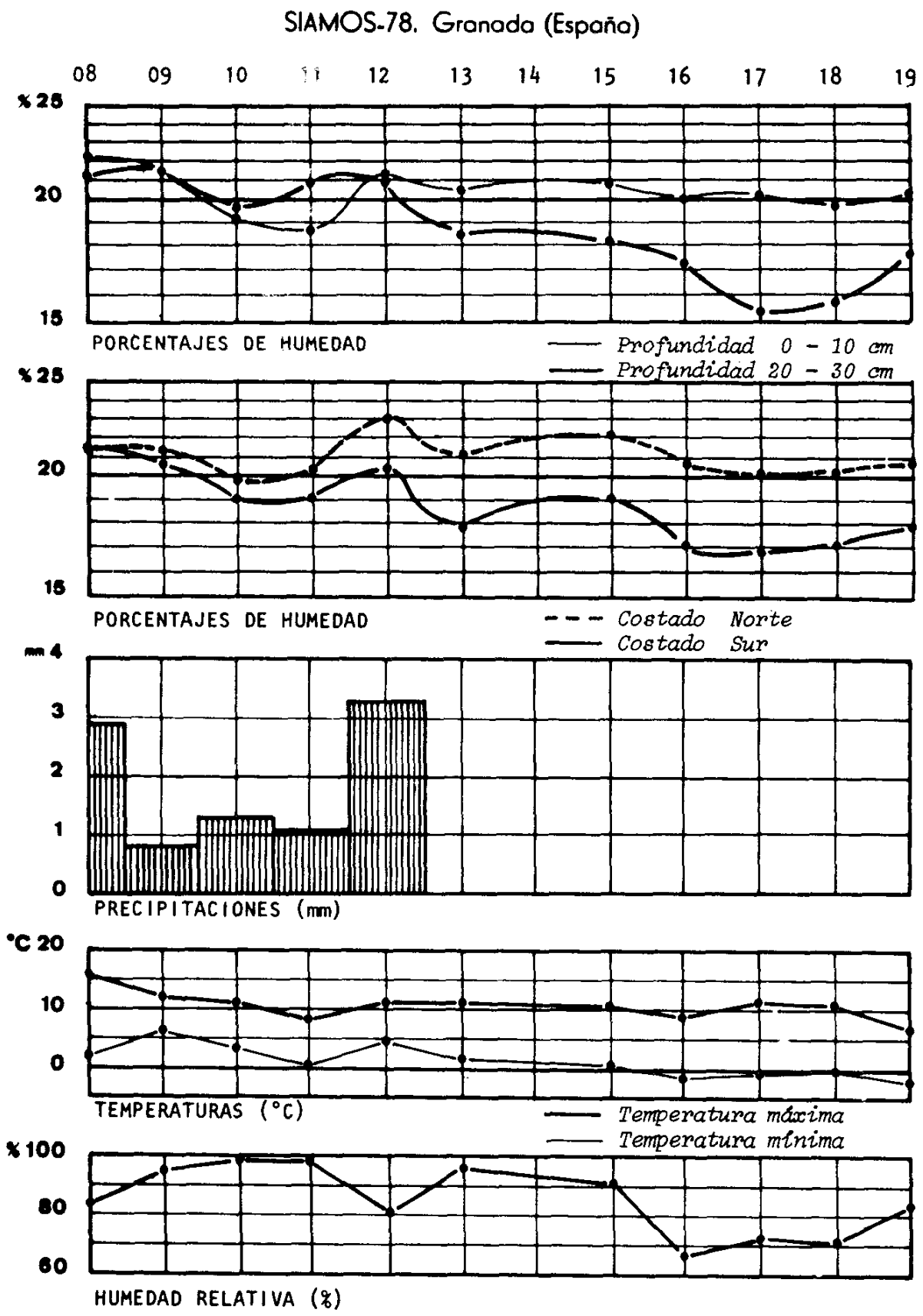

Figura 6. Resultados y observaciones del segundo ensayo.

Efecto de la lluvia

Indudablemente es uno de los factores cuyo control ofrece mayor interés.

Ya hemos señalado, anteriormente, el efecto desfavorable del removido de los apiles durante los días de lluvia, y hay que considerar aún que éste debe ser más acentuado en lluvias cuya intensidad supere a la capacidad de infiltración del mineral en el apile, ya que en este caso favoreceríamos la mezcla del agua con el mineral.

Pero posiblemente el efecto más interesante que se pone de manifiesto (Figura7), es que si bien las lluvias se acusan, en general, por un aumento de humedad del mineral, también se constata que con lluvias como las que han tenido lugar durante algunos días del primer ensayo (aisladas y poco intensas), la curva de secado, en escala semilogarítmica, muestra una recuperación al trazado que se definiría de no haber tenido lugar dicho aguacero.

Puesto que no han tenido lugar lluvias intensas durante los ensayos realizados, no podemos definir los efectos que estas ocasionarían.

Efecto de la temperatura

La temperatura debe tener una incidencia doble, ya que, por una parte, el aumento de temperatura supone un incremento del poder evaporante de la atmósfera y, por otra parte, la disminución de temperatura (especialmente durante la noche), incrementa la condensación en el mineral de la humedad ambiente (efecto del que nos ocuparemos en el epígrafe siguiente).

Con respecto a la evaporación por temperatura hemos de señalar que, durante la realización de los ensayos, las temperaturas medias han oscilado entre 2,5° y 8,75 °C, que hemos de considerar como bajas, aunque normales para la época en la que se han realizado los ensayos.

En estas condiciones la temperatura, como factor evaporante, no es decisiva, aunque se puedan observar ciertos descensos de humedad, correlativos a aumentos de temperatura (por ejemplo día 13/11).

Efecto de la insolación

Si bien no se ha controlado la insolación que, como factor de radiación solar, condiciona la evaporación, no obstante, a partir de los datos publicados por el Centro del Ebro del Servicio Meteorológico Nacional (1975 y 1976), se puede inferir que, en la época de los ensayos las horas de insolación han estado por debajo de la media anual (Figura 8).

Efecto de la humedad atmosférica

La humedad atmosférica juega un papel muy destacado para la evaporación, tanto por condicionar la tasa de evaporación (*humedad relativa*), como por el efecto de

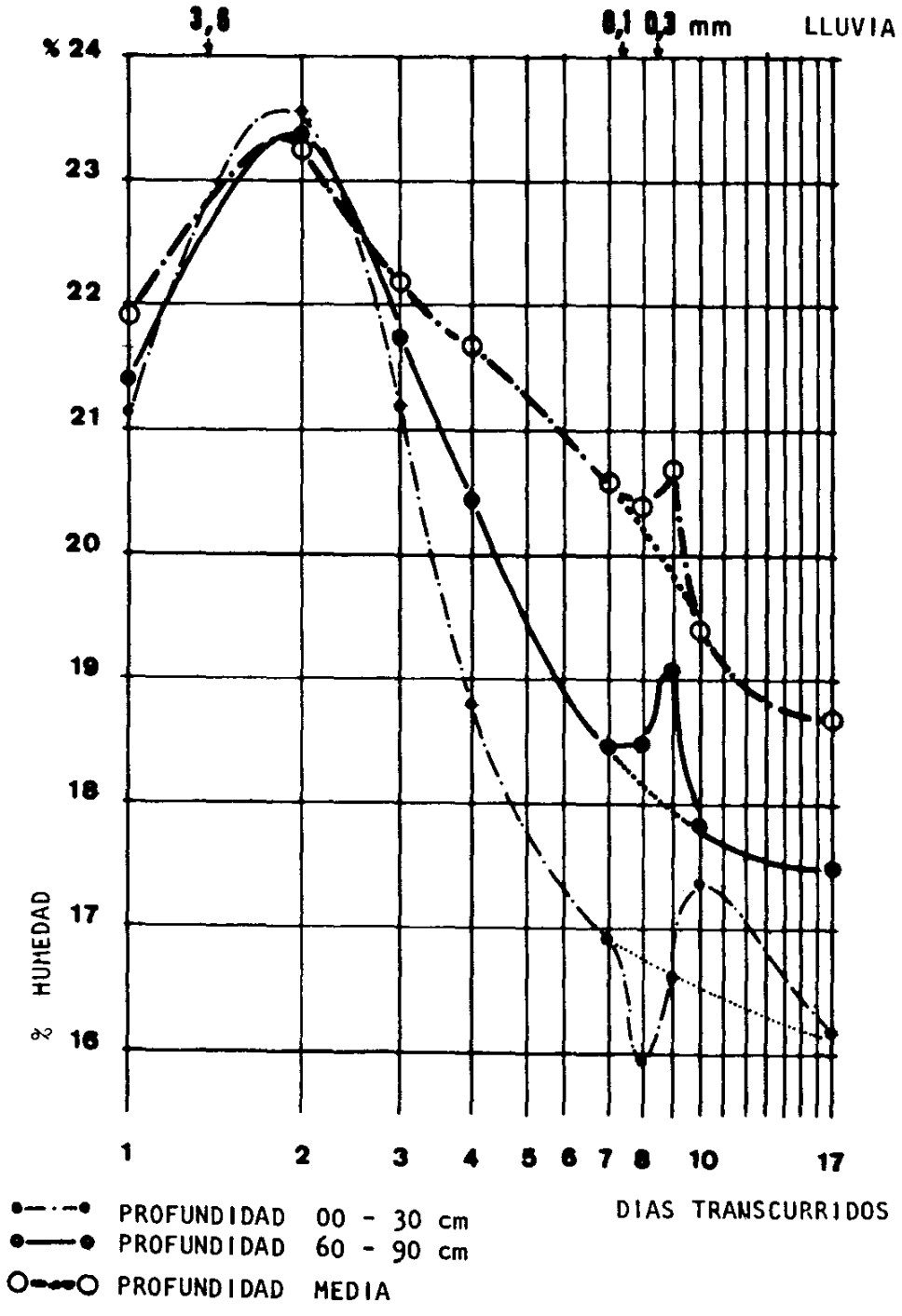

Figura 7. Evolución del secado por oreo en el tiempo

de *aportación oculta* de dicha humedad (durante la noche), al producirse la condensación en la superficie del mineral (rocío; escarcha; helada).

Hay que tener en cuenta que este efecto de condensación, en contacto con el suelo frío, se produce sobre toda la superficie de los poros del mineral, por lo que esta aportación oculta hay que multiplicarla por un factor de superficie, que en la bibliografía se estima entre 6 y 7, y que aquí podría incluso ser mayor, por la esponjosidad de este mineral removido.

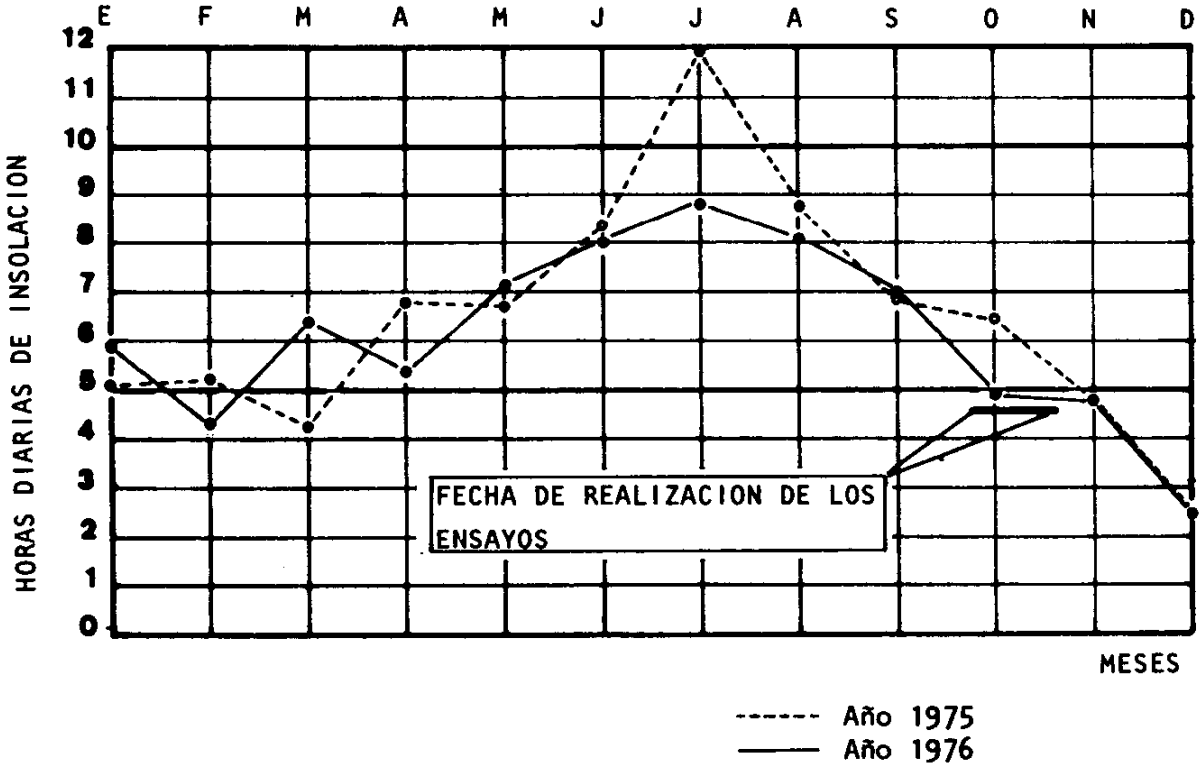

Figura 8. Horas diarias de insolación.

En estas condiciones, el efecto combinado de un aumento de humedad relativa atmosférica, y un descenso de temperatura (especialmente cuando esta es inferior a 0°C), debe implicar un incremento del contenido de humedad del mineral, y una reducción de la evaporación, que prácticamente se anula durante dicha condensación Esto se comprueba, por ejemplo, el 19/11, con un aumento de humedad, y el 16/11, con un descenso de humedad ambiental.

La medida de esta *aportación oculta* ofrece ciertas dificultades y requeriría de un control mediante un drosómetro.

CONCLUSIONES DE LOS ENSAYOS

En estos ensayos no se controló la evaporación, ni la dirección e intensidad del viento, y dada la importancia de dichos factores, se obtuvo del Servicio Meteorológico Nacional, en su Centro del Ebro, un equipamiento muy completo, incluidos evaporímetros y anemocinemómetros, que permitirá en un futuro estudios más completos. No obstante, consideramos que, de los ensayos ya realizados, parece confirmarse la eficacia del método, para conseguir un descenso de humedad que en este caso ha oscilado entre 1.6 y 5.8 unidades (para los 6 apiles).

Por otra parte, y con respecto al tiempo conveniente de mantener el apile, no creemos se justifique prolongarlo (sin lluvia), durante más de dos semanas, ya que si bien la tasa de evaporación potencial diaria puede ser de 1 a 4 mm en invierno, y de 3 a 12 mm en verano, el proceso de evaporación supone una disminución de la humedad del mineral, y en consecuencia parece amortiguarse logarítmicamente la deshumectación (Figura 7).

También se pone en evidencia el interés del removido del mineral, siempre que no coincida con las lluvias, muy irregularmente distribuidas, con precipitaciones copiosas, aunque dispersas.

En consecuencia la Dirección Técnica de Compañía Minera de Sierra Menera S. A. ha programado introducir, en el Plan de Explotación, a partir de julio de 1978, un circuito de secado por oreo, para un ciclo de renovación de 4.000 tm/día de mineral, que serán extendidas y removidas sobre una superficie de 24.000 m^2, con una altura de 0,6 m. En dicho programa se incluye el control riguroso de los factores condicionantes, de acuerdo con la experiencia ya recogida.

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CONCURSO DE LA HIDROGEOLOGIS SUBTERRANEA EN LA VALORACION DE YACIMIENTOS

García-Caro, P.

ABSTRACT: For the establishment plan, about investment and exploitation of the ore deposit, necessary must be know the parametric measure that, conjugated with the market position, permit cipher the final expectancy of profit. So more sure will be, as more bigs is the grade of knowledge of the ore deposit.Do not consider the incidence of the underground water action, in the stage of evaluation, will be possible to carry at the collapse project.

RESUME: La programmation d'un plan d'investissements pour la mise en exploitation d'un gisement minier, prend, comme éléments de base, des paramètres mesurables qui, conjugués avec des expectatives des marchés, permettent de chiffrer quelques espoirs de bénéfice, d'autant plus sûrs que le degré de connaissance du gisement est grand. Ne pas considerér judicieusement l'incidence des eaux souterraines sur l'étape d'évaluation, peut conduire à l'échec du projet.

RESUMEN: La programación de un plan de inversiones para la puesta en explotación de un yacimiento minero, toma, como elementos de base, unos parámetros medibles, que, conjugados con unas espectativas de mercados, permiten cifrar unas esperanzas de beneficio, tanto más ciertas cuanto mayor es el grado de conocimiento del criadero. No considerar ponderadamente la incidencia de las aguas subterráneas, en la etapa de evaluación, puede llevar a la ruina del proyecto.

Sociedad Minera y Metalurgica de Peñarroya-España, S. A.

La estimación de la rentabilidad de un negocio minero, se realiza generalmente sobre unas bases o niveles de información , afectados de un índice de incertidumbre, tanto mayor cuanto más escasa sea dicha información.

El costo de los equipos, la evolución del coste de los factores de producción, e incluso la evolución del valor de la producción, pueden ser estimados con gran exactitud, para todo el período de tiempo que dure la explotación.

Análogamente, puede obtenerse un alto grado de certidumbre en la evaluación de las reservas o tonelajes a extraer y sus correspondientes calidades.

Pero, junto a estos parámetros fiables, intervienen otros, cuya determinación es frecuentemente incierta, y cuya incidencia en el balance final es siempre decisiva. Dilatar el período de implantación, es equivalente a una mayor inversión inicial. Obtener un mayor coste de explotación del inicialmente previsto, equivale a reducir en esa proporción el beneficio esperado.

De las diferentes causas que pueden incidir en el sentido negativo antes apuntado, destacamos la presencia del agua subterránea que, en determinadas circunstancias, puede incluso hacer inviable un proyecto minero.

Si, en la evaluación del proyecto, se ha realizado una valoración cierta de la incidencia de las aguas subterráneas, habremos mejorado nuestra posición al reducir el índice de incertidumbre, y no sólo eso, pues el conocimiento de las posibilidades de aguas subterráneas en los terrenos de asiento de la proyectada explotación o en sus proximidades, puede incidir fuertemente en la inversión programada, sustituyendo costosas soluciones.

Tratamos aquí de hacer una llamada de atención, pues que resulta extraordinariamente frecuente presenciar el inicio de ambiciosos proyectos mineros que, basados en un complejo acúmulo de datos obtenidos como consecuencia de prolijas investigaciones y elaborados con técnicas de cálculo muy sofisticadas, dejan en el olvido lo que puede, en su momento, representar un aspecto fundamental para la viabilidad del proyecto en cuestión.

Así, hemos presenciado que, tras exhaustivas campañas de geoquímica y geología de superficie, se han implantado y ejecutado cerradas mallas de sondeo, impuestas por consecuencia de dudosas aplicaciones, que tratan de representar, por un determinado algoritmo o modelo matemático, cuál es la evolución de las mineralizaciones, estableciendo correlaciones de contenidos, zonas de influencia, etc., que ofrecen elegantes justificaciones para determinadas anomalías y que, eso sí, suelen acompañarse de espectaculares programas y listados de ordenador. Pero en los que en ningún momento se hace mención, siquiera remota, a la existencia o no de aguas subterráneas, ignorando que, en general, los terrenos sirven, o pueden servir, de asiento a corrientes subterráneas.

El diseño de los taludes en una explotación a cielo abierto, se realiza sin considerar la influencia que sobre las características mecánicas resistentes de las formaciones rocosas, tiene el desplazar el nivel freático; cuando, conociendo los parámetros hidrogeológicos de las mismas, transmisibilidad, permeabilidad, etc., pueden preverse los efectos de la presión de corriente, que aparece por el flujo del agua, ejerciéndose sobre los granos de terreno en el sentido del flujo y que puede, en determinadas circunstancias, provocar la ruina del talud.

Recientemente, un importante proyecto minero ve cómo se ha desfasado su fecha de entrada en explotación, ocasionando una pérdida de producción del orden de 50.000 Tm. de concentrados de cinc y 15.000 Tm. de concentrados de plomo. La causa principal: el desconocimiento de las características hidrogeológicas de la zona. La supuesta inexistencia de recursos hidráulicos para atender las necesidades del taller de concentración, indujo a construir un embalse de regular capacidad, en el que acumular los caudales suficientes para el proceso, el cual se realizaría en circuito cerrado, esto es, recuperando, tras su decantación y clarificación, las aguas contenidas en la salida de estériles, los que, a su vez, serían utilizados como relleno de la mina.

El terreno, asiento del embalse, resultó estar constituido por una caliza con abundante fisuración. Sin entrar en más detalles sobre el particular, significamos que el vaciado del embalse se produjo en el momento en que las aguas filtrantes descolmataron las grietas anteriormente rellenas de arcillas y acarreos de superficie. Tras esto, hubo de procederse a la impermeabilización del vaso, mediante costosas operaciones de inyección de cementos.

Tal vez, en la base de estas calizas, pudieran haberse implantado captaciones de caudal suficiente para atender a las necesidades del proceso.

Para el siguiente ejemplo, del que disponemos de una información más completa, presentamos una solución que, a buen seguro, hubiese sido la más eficaz y de menor costo.

Una formación mineralizada en Plomo/Zinc/Cobr[illegible]enía siendo objeto de beneficio por una modesta explotación, que alcanzaba a -180 m. medidos desde la bocamina y sobre una extensión de 350 m. en dirección.

Se realizó una intensiva investigación geológica, complementada con sondeos mecánicos desde el exterior, detectándose la existencia de mineralización económica sobre un yacimiento o criadero que, al contacto con una formación de pizarra, se encuentra sobre una caliza; en posición subvertical, con potencias de 2 a 5 m., presenta una extensión lateral de varios kilómetros, habiéndose constatado su extensión en profundidad a 600 m.

Los niveles portadores de mineralización se extienden bajo una sucesión de concesiones y permisos, de las cuales, en dos, se habían realizado las labores ya mencionadas. En las proximidades de esa explotación, la serie estratigráfica es cortada de través por un río, cuyo cauce se encuentra a -192 m. de la bocamina. Es característica del entorno aflorante la existencia de cavernas y fracturas del tipo karst.

No resulta difícil suponer que, precisamente las labores más profundas de la mina, se encontraban a la cota correspondiente a la superficie piezométrica del acuifero soportado en la caliza fracturada. El desagüe establecido a ese nivel alcanzaba a un orden de 3 metros cúbicos/hora; es decir, bastante reducido. Pero, y por bajo de ese nivel, en la zona del acuifero saturado?

Determinada la macroestructura del yacimiento, se estableció un programa de trabajos mineros, que pretendían el acceso a los niveles profundos, confirmación de resultados de investigación, preparación, si procede, y puesta en explotación.

De las diferentes opciones, se adoptó la profundización de rampas de sección de 4'5 x 3'7 m., al 17 % de pendiente, aptas para la utilización de maquinaria y equipos actuales. Los rendimientos de avance se estimaron en 100 metros/mes, en organización de trabajo a tres turnos de siete horas, sobre los que se realizarían los ciclos de arranque / carga y transporte a exterior.

La progresión del avance en rampa, hasta el antiguo nivel 180, se realizó efectivamente al ritmo previsto, siendo las desviaciones imputables a causas tales como formación del personal, dificultades de ventilación, etc.

Se constató la existencia de abundante fracturación, consecuencia de la intensa tectónica, grietas de disolución y karst, algunas de regulares dimensiones, constatándose asimismo que el agua de infiltración, debida a las lluvias, discurría por la red de fracturas, hasta sumirse a cotas aún no alcanzadas.

El programa de trabajos proyectados dedicaba un capítulo a desagüe, pero en un tratamiento superficial, estimándose unas necesidades mínimas de potencias a instalar, equipos de bombeo de reducido caudal, conducciones de pequeño diámetro, etc., cuya incidencia en el futuro coste resulta de muy escasa entidad. Y, en cuanto a la incidencia sobre la inversión prevista en primera etapa, no alcanzaba al uno por mil. Es decir, se ignoró totalmente la posibilidad de aguas subterráneas.

En noviembre de 1976, ya en el entorno de la cota -180, la efusión de agua en el avance entorpecía las operaciones de barrenado, carga de explosivo y

desescombro; resultando insuficientes los equipos disponibles, hubo de procederse a la sustitución de los mismos, improvisar instalaciones, etc. Aun disponiendo de equipos de potencia suficiente, los caudales a evacuar hacían sentir su acción, pues alcanzaba a 80 metros cúbicos/hora, con máximos ocasionales de 250 metros cúbicos, pero ello en circunstancias de pluviosidad intensa. Las mayores precipitaciones observadas, se han cifrado en 270 litros/m^2. /mes.

El deterioro de los rendimientos resultó escalofriante y, consecuentemente, el aumento de los costos unitarios se produjo hasta cotas insoportables. El rendimiento por mes del avance pasó, de una media de 79'2 m. a 36'2 m.

Se suspendieron los avances a la cota 235, encontrándose el caudal estabilizado en 80 metros cúbicos/hora. Si tenemos en cuenta que transcurrieron 11 meses, desde la aparición del agua hasta la suspensión del avance, resulta una pérdida total de 11 x 43 = 473 m. en el avance descendente, que representan 80 m. en altura. Es decir que, a la fecha de suspensión, se hubiesen alcanzado cotas próximas al nivel 315, en lugar del 235, lo que permitiría un reconocimiento más extenso de la formación mineral. Naturalmente, a estas pérdidas de rendimiento, debemos añadir el costo adicional de los equipos.

Según hemos podido ver en los dos casos expuestos, las previsiones realizadas han sufrido un deterioro de considerable magnitud. Naturalmente, dependerá del tipo de yacimiento, el que tal o cual empeoramiento de las expectativas sea o no soportable.

Al estudiar la explotabilidad del yacimiento y una vez determinadas la evolución de contenidos y los tonelajes beneficiables, llegamos al punto de dimensionar la inversión y estimar el coste de explotación. Ambos parámetros serán los definidores del contenido mínimo en metal beneficiable o ley de corte, factor éste que se obtiene a partir de los anteriores, una vez sea conocido el valor del punto metal, que, a su vez, se determina con escaso margen de error, excepción hecha de las posibles variaciones de cotización. Así, la ley de corte viene dada por :

$$Lc = \frac{i + c}{Vp}$$

En la que i es la inversión necesaria, por tonelada a beneficiar, al precio de coste c. Siendo V_p el valor del punto metal, en función de :

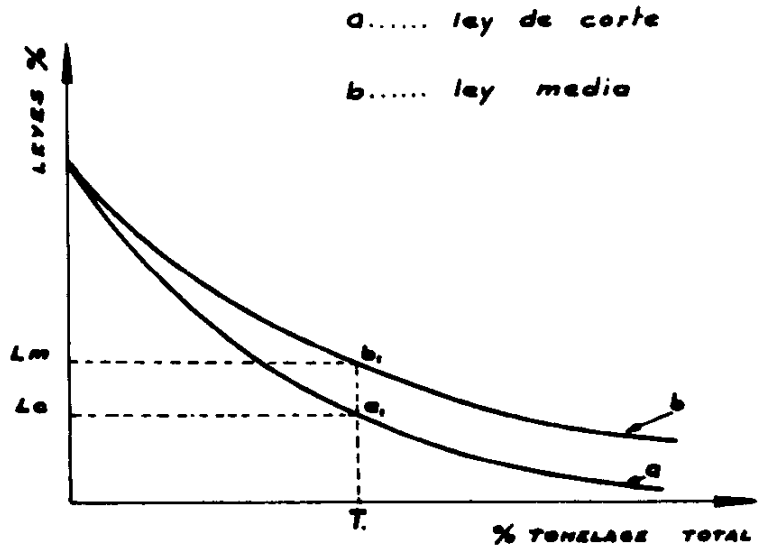
Rc, rendimiento de la concentración,
Lc, ley de los concentrados
Pc, valor de la tonelada de concentrados

viene dado por la expresión

$$V_p = \frac{Rc}{Lc} \times Pc$$

Del análisis de la expresión anterior, concluimos que la invariabilidad de Lc se produce cuando permanecen constantes (i + c) y V_p. En condiciones normales de mercado, las variaciones que pudieran producirse serían en el mismo sentido y en cuantías proporcionales, lo que constituye una garantía de constancia de Lc para determinados plazos.

La rentabilidad de la futura explotación aparece íntimamente relacionada a la diferencia de contenidos entre Lm, o ley media, y Lc, puesto que el beneficio esperado, por tonelada, será V_p (Lm-Lc). Por tanto, cualquier incremento incontrolado de i + c supone un incremento de Lc y, paralelamente, una reducción de Lm-Lc, que puede hacerse incluso negativo.



Actualmente, la evolución de los costos se ve agravada por la desfavorable evolución de los mercados de metales, situación que, efectivamente, pudiera ser transitoria, pero es evidente que esta situación, al menos para determinadas sustancias como, por ejemplo, Cu., Zn., no se manifiesta tanto como coyuntural, sino más bien derivada de la propia estructura del sector, lo que obliga a extremar la cautela a la hora de decidir el inicio de una nueva explotación.

Volvemos, pues, al punto de partida, concluyendo en la imperiosa necesidad de reducir al máximo la incertidumbre y, principalmente, en aquellos aspectos en que los conocimientos actuales lo permiten. Así, en lo referente a la hidrogeología, su aportación puede ser inestimable, en casos como los expuestos.

Si se tiene en consideración que, previamente a la implantación de labores mineras, se dispone de una densa malla de sondeos, resulta evidente la utilización de los mismos en la investigación de los parámetros hidrogeológicos. Los cuales, por otra parte, pueden ser perfectamente interpretados por los geólogos, al mismo tiempo que efectúan la evaluación mineralógica y estratigráfica.

Así, al confeccionar el log. correspondiente, deberían añadirse al mismo los resultados de observaciones realizadas con ese objeto y que van a permitir, tras su posterior análisis, determinar las características hidrogeológicas, para enfocar cuál será el mejor medio de acción, frente a las mismas. Y, lo que es fundamental, permitirán evaluar el costo de la incidencia de las aguas subterráneas en la futura inversión.

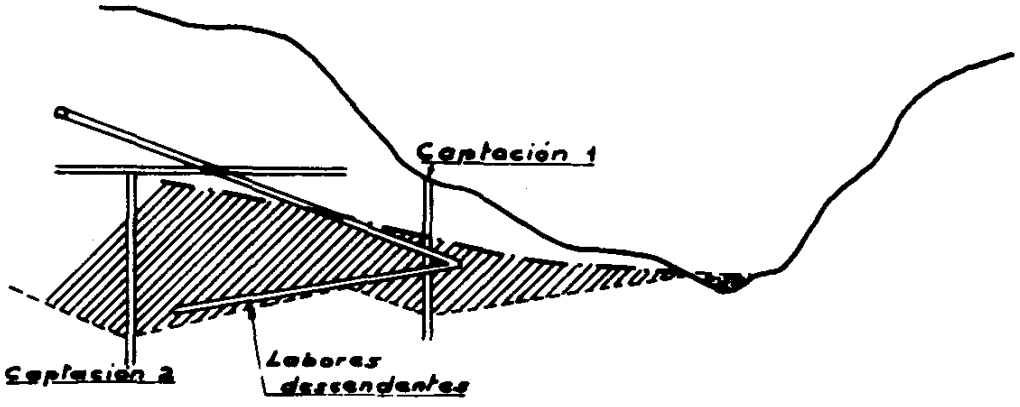
En el caso que hemos descrito anteriormente, se trata, en efecto, de una situación extrema, tanto en cuanto a los caudales cortados, como en cuanto a la geometría del yacimiento, que impone un extenso programa de labores de acceso, a consecuencia de la reducida potencia de la capa y la posición subvertical de la misma.

La geometría del yacimiento es, tal vez, el condicionante principal para decidir los accesos en rampa que, si bien no progresan en profundidad, al ritmo de un pozo vertical, sí proporcionan una mayor superficie de capa accesible.

La acción negativa del agua en los avances de rampa ascendente, se suma a las dificultades de ventilación que, sobrepasando los 750 m. en fondo de saco, hacen problemático conseguir los tres ciclos de trabajo en el día. Llegada la rampa descendente a ese desarrollo de 750 m., equivalentes, en general y para las pendientes usuales de, aproximadamente 17 %, a 125 m. de diferencia de la cota, resulta imperativo poder disponer de una canalización o enlace con el exterior, que elimine la ventilación forzada en fondo de saco.

Es en este punto, precisamente, cuando deben considerarse detenidamente las características hidrogeológicas y proceder a su más cuidada ponderación que nos llevará a determinar el punto más adecuado en que pueda establecerse una captación profunda, desde la cual, si no llegar al agotamiento, sí provocar importantes descensos de la superficie piezométrica, de suerte que ésta resulte en todo momento situada por bajo de las labores mineras.

Un sondeo de explotación de 150/200 m. de profundidad y diámetro de 600 mm. es una obra usual en el ámbito de las aguas subterráneas. Bien desarrollado sobre unas calizas kársticas, es también muy normal obtener del mismo caudales de 200 m. cúbicos por hora.



AUMENTO DE LA SUPERFICIE NO ANEGADA POR EFECTO DEL BOMBEO EN LAS CAPTACIONES 1 y 2

En cuanto al coste de establecimiento de una tal captación, oscila en torno de 2 M. de pesetas, cuya repercusión sobre los 750 m. de rampa a que nos estamos refiriendo, se cifra en, aproximadamente, 2.600 ptas./m.

Ahora bien: hemos de tener en cuenta que, llegados con las labores de rampa al punto de conectar con el sondeo, puede éste, en el tramo inferior a la intersección, seguir utilizándose como dren, en tanto que el tramo superior puede realzarse a mayor sección e integrarse en el circuito de ventilación como chimenea, bien para retorno de ventilación o para entrada de aire fresco.

Aquí es, precisamente, donde reside la principal ventaja del método propuesto, ya que nos brinda la posibilidad de efectuar el trabajo de preparación y acceso en seco, y disponer de chimeneas para ventilación de esas labores.

Naturalmente, el coste de evacuación al exterior de las aguas subterráneas no podemos eludirlo, ello es evidente, y continuará manifestando su acción sobre el precio de explotación. Lo que sí habremos conseguido es conocer, a priori, la incidencia de las aguas subterráneas sobre ese precio de coste y, lo que es fundamental, habremos reducido en mucho el índice de incertidumbre que existe en todo proyecto minero.

L'ABAISSEMENT PROFOND DU NIVEAU PIEZOMETRIQUE DES SOURCES THERMALES DE TEPLICE (NORD-OUEST DE LA BOHEME, TCHECOSLOVAQUIE) AU COURS DE L'EXPLOITATION DE LIGNITE A CIEL OUVERT

Homola, V.

ABSTRACT : In order to ensure the thermal water supply for the Teplice Spa and to cover the safety of brown coal mining in the vicinity of Duchcov, regarding the possible underground water burst, starting 1896 the ground water level is depressed 23-30 m below the natural piezometric surface. In future it is planned to extract the coal by open pit mining. It will be necessary to depress the piezometric surface of ground water by further 80 - 90 m. By this action the underground runoff of thermal water does not exceed by more than 100 %, and the temperature of rocks within the zone of thermal water accumulation, does not drop during the period of 15 years water pumping more than by 2°C.

RESUME : Pour assurer l'approvisionnement en eaux thermales des bains de Teplice, et pour protéger les mines de lignite de Duchcov des irruptions d'eau porphyrienne, on tient, depuis 1896, le niveau piézométrique à 23 - 30 m au-dessous du niveau originaire. Dans l'avenir, on a l'intention d'extraire la lignite à ciel ouvert. Il sera nécessaire d'abaisser encore le niveau piézométrique de 80 - 90 m. Pour cette intervention le débit des eaux thermales n'augmentera que de 100 %, et la température des roches, dans la zone d'accumulation, n'abaissera que de 2° C, au cours de 15 ans de pompage.

RESUMEN : Con el fin de asegurar el abastecimiento de aguas termales para el balneario de Teplice, y de mantener la seguridad en las minas de lignito de los alrededores de Duchcov, ante las irrupciones de agua, se mantienen las aguas subterráneas, desde 1896, a un nivel de 23 a 30 m bajo el nivel piezométrico natural. Se ha planeado, en el futuro, explotar el lignito a cielo abierto, lo que obligará a rebajar la superficie piezométrica en otros 80 - 90 m. Con esta intervención el aflujo subterráneo de aguas termales no aumentará más del 100 %, y la temperatura de las rocas, en el sector de las termas no disminuirá, en 15 años de bombeo, más de 2° C.

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La ville d'eaux de Teplice était toujours et reste jusqu'aujourd'hui avec ses plus de 5 500 lits la plus grande station balnéaire de Tchécoslovaquie. Avec son débit total des sources (cca 30 $l.s^{-1}$) et leur température (46 - 50° C) Teplice appartient parmi les plus importantes station balnéaires de l'Europe centrale. Les eaux thermales de Teplice se forment dans un corps de porphyre quartzeux (dit porphyre de Teplice, Carbonifère supérieur - Permien inférieur), qui remplie une crévasse dans les schistes cristallins de Krušné hory (Monts Métalliféres de la Bohême), orientée de N vers S jusqu'à de NNO vers SSE. Les veines de porphyre granitique, plus vieux que l'intrusion de granite acide dit de Krušné hory (Permien supérieur) suivent la même direction. La largeur du corps de porphyre quartzeux varie de 4 jusqu'à 7 km, sa longueur atteint 20 km (y compris l'extension de 11 km sur le territoire de la République démocratique d'Allemagne, déjà de l'autre côté de la ligne de partage des eaux de 1er ordre). Sa puissance - d'après le témoignage des forages profonds - dépasse 1 000 m. Au pied méridional de Krušné hory, le porphyre quartzeux tombe sous les sédiments du bassin houiller miocéne, dit bassin de Chomutov-Ústí. Les restes dénudationels du Crétacé supérieur (grès, 0-15 m, marnes gréseuses, 0-90 m) se trouvent quelque part au-dessous de Miocène. Les tufs et tuffits volcaniques oligocénes sont intercalés en puissance de 0 - 20 m entre le Crétacé et le Miocène. Les veines et les épanchements de basalte et de phonolite sont rares. Le Miocène se compose d'une série argileuse inférieure (2 - 25 m), d'une série lignitfère (5 - 35 m) et d'une série argileuse supérieure (10 - 180 m). La largeur du bassin houilleux ne dépasse pas 3 - 4 km. Son bord méridional est formé d'un horst constitué par le porphyre quartzeux et par les assises du Crétacé supérieur, appelé le horst de la zone de Teplice-Lahošť. Plus loin vers le Sud, le Crétacé est couvert par les roches volcaniques et pyroclastiques (Oligocène - Miocène inférieur?) de České Středohoří (Monts volcaniques de la Bohême cen-

trale). Le porphyre quartzeux y est encore répendu mais il n'apparait plus nulle part sur la surface.

Quant à la tectonique, les directions d'age variscien et saxonien sont connues. La direction principale de la tectonique variscienne est conforme aux bordures du corps de porphyre quartzeux et aux veines de porphyre granitique, c.-à-d. de N - NNW vers S - SSO. Les directions tectoniques de NW vers SO et de NO vers SW sont un peu plus jeunes. La direction tectonique de WSW - W vers ONO - O, dite "direction de Krušné hory", prédomine dans la tectonique saxonienne. Les dislocations saxoniennes transversales et obliques sont rares et peu importantes. La dislocation de la "Source gigantesque" (de SO vers NW) est une exception. Il s'agit d'une dislocation variscienne plusieurs fois rajeunie (à la fin du Crétacé, du Oligocène et du Miocène). La formation des eaux thermales de porphyre quartzeux commence par l'infiltration des précipitations atmosphériques et probablement aussi d'eaux des ruisseaux aux sommets de Krušné hory (l'altitude approxim. de 800 m s.m.). C'est de là, que l'eau infiltrée descend jusqu'à la profondeur de 2 500 - 3 000 m et coule - peut-être - déjà dans les schistes cristallins dans le soubassement du porphyre vers le Sud. Au cours de ce chemin, la circulation d'eau est dirigée par les dislocations et fissures de la direction variscienne, lesquelles étaient rajeunies pendant l'époque saxonienne. La fonction de ces dislocations et fissures est variée, une fois elles agissent comme des conducteurs, une autre fois comme des isolants hydrauliques. En fonction de conducteur, les dislocations et fissures de la direction de NW vers SO (dite "direction sudétique") et de N vers S (dite "direction de Jizera") sont plus importantes.

En arrivant au bord septentrionel du horst de la zone Teplice-Lahošť, l'eau - déjà échauffée à 60 - 65° Celsius - monte vers la surface le long d'un système des dislocations longitudinales saxoniennes

(de la direction de WSW - W vers ONO - O, c.-à-d. "direction de Monts Métallifères"). La distribution de l'eau thermale pour différentes sources ne s'effectue qu'à la proximité de la surface.

Dans la partie orientale de la zone Teplice-Lahošť, les eaux thermales apparaissent dans deux groupes des sources. Les sources situées plus loin vers l'Est (le groupe de Šanov) s'écoulaient jusqu'à 1879 avec le débit total de 3 - 5 $l.s^{-1}$ dans l'altitude 206 - 216 m s.m. Les sources du deuxième groupe (dit de Teplice s.s.) éloignées à 1000 m vers l'Ouest du groupe de Šanov, s'écoulaient jusqu'à 1879 avec le débit total de 15 - 18 $l.s^{-1}$ dans l'altitude 221 - 223 m s.m. Dans la partie occidentale, les eaux thermales apparaissaient jusqu'à 1887 en distance de 5 km de Teplice dans une source appelée "Source gigantesque". Les eaux thermales mixtes avec les eaux froides jaillissaient y dans l'altitude de 223 m s.m. avec le débit total de 25 - 40 $l.s^{-1}$. En 1879, une irruption de l'eau de porphyre dans la mine de lignite "Döllinger", non loin de la ville Duchcov et de la "Source gigantesque" (1,6 km vers l'Ouest) s'est produit. Le lieu de l'irruption était en 1881 - 1882 entouré par les dames, mais le niveau piézomètrique restait à 1 - 3 m au-dessous de la surface naturelle des sources thermales. En conséquence, il fallait puiser la therme avec une pompe.

Les irruptions de l'eau porphyrienne se répetaient encore en 1885 et 1892, cette fois dans la mine "Viktorin" éloignée de 0,8 km vers NNW de la mine "Döllinger". Ces irruptions étaient suprimées aussi par les dames. Pour assurer l'alimentation des bains par l'eau thermale, on a fait construire en 1880 - 1882 de nombreuses puits au lieu des sources thermales, dont la profondeur atteignait 54 m. La sécurité des mines a été assurée par une mesure dite "la protection réparative". Le principe de cette "protection" est fondé sur le pompage des eaux porphyriennes, avec lequel on réussit à maintenir - de 1896 jusqu'à aujourd'hui - le niveau piézomètrique

des eaux porphyriennes au-dessous du niveau piézomètrique naturel, à savoir dans les bains de 23 mètres, dans les mines de 30 mètres. En conséquence il s'est produit une inversion de la pente de la surface des eaux porphyriennes en direction vers les mines, c.-à-d. vers l'Ouest. La pente hydraulique ainsi que la quantité d'eau porphyrienne (y compris la therme), laquelle doit être pompée pour maintenir le niveau fixé par la "protection réparative", ont agrandi en comparaison avec l'état naturel, à savoir dans les bains à 23 - 27 $l.s.^{-1}$, dans les mines à 30 - 55 $l.s^{-1}$. On n'a aucune information sur le changement de la température de l'eau thermale dans la zone profonde (d'accumulation). Aujourd'hui, d'après les résultats d'un forage de Teplice (le forage no. Tp-28 à la Place de la Paix), la température de la therme atteint dans la profondeur de 850 - 950 m même 55° C. À présent, l'abaissement de 48 - 49° C en 1879 à 42 - 43° C a été produit par le mélange de la therme avec de l'eau froide de la circulation peu profonde.

À partir de 1960, on prépare une exploitation du lignite dans les environs de Duchcov (y compris les mines souterraines "Döllinger" et "Victorin") à ciel ouvert. Dans ce cas, il sera nécessaire d'abaisser successivement le niveau piézomètrique des eaux porphyriennes encore plus bas, en somme de 80 - 90 m en comparaison de l'état actuel et de conserver ce niveau d'abaissement pour 15 ans au maximum.

Pour pouvoir réaliser ce projet, il est nécessaire d'abord d'effectuer plusieurs travaux préparatoires et trouver la solution de quelques questions théoretiques:

1. Construire une nouvelle station de pompage de l'eau porphyrienne froide en voisinage de la mine planifiée à ciel ouvert, mais déjà en dehors des rayons de déblais.
2. Supprimer les lieux d'irruptions de l'eau porphyrienne froide (de la fin de XIX^{ème} siècle) dans les mines souterraines "Döllinger", "Viktorin" et Gizela".

3. Capter de nouveau les sources thermales de Teplice par les forages, dont la profondeur doit être supérieure à 300 m.
4. Réaliser plusieurs forages de drainage pour pouvoir abaisser le niveau piézomètrique des eaux porphyriennes jusqu'à la cote + 80 m s.m.
5. Calculer les quantités prévues d'eau, lesquelles devraient être pomper pour tenir le niveau de la surface abaissée, successivement jusqu'à la cote + 80 m s.m.
6. Déterminer l'extension et la forme attendues du cône d'appel et les changements dans le régime de la structure hydrogéologique du porphyre de Teplice.
7. Évaluer l'influence d'abaissement attendu de la surface des eaux porphyriennes sur le régime géothermal dans la zone de la genèse et de l'accumulation des eaux thermales, particulièrement sur la réduction de la température des eaux thermales au cours du pompage et sur sa restitution après avoir fini le pompage.

Jusqu'à la fin de l'année 1977, on a effectué ou on est en train de terminer les travaux suivants:

1. La "Source gigantesque" a été reconstruite pour une station stable de pompage sur la cote 192,0 - 193,0 m s.m. La puissance des pompes est 3 x 60 $l.s^{-1}$.

2. Le lieu d'irruption de l'eau porphyrienne dans la mine "Viktorin", situé à la cote + 165 m s.m., c.-à-d. de 75 m au-dessous de la surface, a été supprimé dans les années 1976 - 1977 par 78 forages verticals fondés sur la surface et 80 forages inclinés fondés dans la mine près des dames à eau. La profondeur des forages verticals a été 90 m, la longueur des forages inclinés de 40 à 50 m. On a employé divers mélanges sur la base du ciment et des résines synthétiques comme l'agent étauchéifiant. Pour remplir les cavités d'une grande dimension, on a utilisé le cendre et le machefer. Actuellement la liquidation du lieu d'irruption dans la mine "Döllinger"

est effectuée de la même façon. On prépare la liquidation d'un troisième lieu d'irruption - dans la mine "Gisela", 1 km vers l'Est de la mine "Viktorin".

3. La source thermale principale dite "Pravřídlo" (dans le groupe de Teplice s.s.) a été captée en 1967-1972 par un forage profond de 972 m, situé à la Place de la Paix (forage no. Tp-28). L'eau thermale avec température de 55,0° C afflue d'une zone tectonique dans l'intervalle de 884 à 972 m. Au cours du pompage avec l'abaissement de 65 - 70 m on a atteint le débit de 23 - 27 $l.s^{-1}$. Un deuxième forage de captage dans le groupe de Teplice est en état de comstruction. Dans le groupe Šanov, on prépare un troisième forage pour la source appelée "Horský pramen" ("la source sur la pente d'une colline"). Le niveau piézomètrique dans le premier forage est à 2,5 m au-dessous du niveau dans la source "Pravřídlo". L'essai de pompage, effectué de 30. juin à 30. septembre 1977, a montré une indépendance hydraulique de la circulation profonde entre le forage mentionné et la source "Pravřídlo", quoique leur distance ne dépasse pas 250 m.

4. Quant à la recherche des rapports hydrogéologiques, dans la partie du corps de porphyre quartzeux couverte par les sédiments miocènes et dans la zone de Teplice-Lahošť on a réalisé de 1879 jusqu'à 1977 plus de 100 forages, dont la profondeur était de 60 à 972 m. De ce nombre on dispose dans la partie de l'Ouest (dans la région des mines) de 14 forages d'observation et 4 de pompage, dans la partie d'Est (dans la région des bains) 36 forages d'observation et 3 de pompage. Le débit des forages de pompage varie dans les limites très vastes, de 1 - 2 $l.s^{-1}$ avec abaissement de 1 - 20 m jusqu'à plus que 20 $l.s.^{-1}$ avec abaissement seulement de 0,15 m. Cette variation du débit mentionné est propre aux toutes roches fissurées. On a constaté que le niveau des eaux porphyriennes abaissé, entretenu par le pompage sur la cote + 192 m à + 193 m s.m. s'étend à partir de la limite méridionale et occidentale du corps de porphyre

vers l'Est jusqu'à bord occidental de Teplice. Plus loin vers l'Est, on suive un bloc partiellement séparé au point de vue hydraulique souterraine, dans lequel le forage de captage à la "Place de la Paix" fut fondé. Encore plus loin vers l'Est, d'autres blocs hydrauliques partiellement séparés suivent: le bloc de la source thermale "Pravřídlo", le bloc avec les eaux porphyriennes froides de la rue "Mlýnská" ("Rue des moulins") et le bloc des sources thermales du groupe de Šanov. Une fissure remplie de la brèche volcanique qui suit la direction de NNW vers SSO, sépare ce bloc des gneiss qui apparaissent de l'autre côté. La dislocation longitudinale nommée "Barborská" qui poursuit le centre du bassin miocène, sépare la région avec le niveau des eaux porphyriennes abaissé au Sud de celle du Nord, où le niveau s'élève de la cote + 192 m s.m. jusqu'à la cote + 301 m s.m. Cela signifie, que la dislocation "Barborská" fonctionne comme une barrière hydraulique semiperméable (voir fig. 3). Pour vérifier les rélations hydrauliques on a réalisé deux essais de pompage. Le premier avait lieu de 13. à 23. juillet 1964 (Hálek et col., 1964). Il a été effectué par l'ouverture des vannes à eau dans les dames dans la mine "Döllinger". Le débit de l'eau était en moyenne de 150 $l.s^{-1}$, l'abaissement total dans le lieu de pompage atteignit 4,4 m. Le deuxième essai de pompage était réalisé dans la "Source gigantesque" de 22.5. jusqu'à 23.6.1969. Le débit moyen était de 50 $l.s^{-1}$, l'abaissement à la fin d'essai dans le lieu de pompage de 1,28 m, dans la mine "Döllinger" (éloignée à 1,6 km vers l'Ouest) de 1,20 m, dans le forage TH-31 (éloigné à 2,6 km vers Nord-Est) de 0,74 m. L'influence d'essai mentionné sur les sources thermales de Teplice n'est pas évaluable à cause des essais de pompage lesqueles avaient lieu en même temps dans le forage Tp-28 à la Place de la Paix. Les résultats de tous ces essais permetèrent de diviser le corps du porphyre à quelques blocs hydrauliquement moins ou plus isolés (voir fig. 3 et le texte plus haut).

5. La quantité de l'eau porphyrienne laquelle devra être pompée pour tenir le niveau piézomètrique sur la cote de + 80 m s.m. ne dépasse pas - d'après l'extrapolation des résultats des essais de pompage - 200 $l.s.^{-1}$. On ne peut pas encore dire quelle partie du débit de 200 $l.s^{-1}$ formeront les eaux porphyriennes froides et quelle partie les eaux thermales. La même quantité - de 200 $l.s^{-1}$ - de l'eau porphyrienne a été pompé en 1881-1882 au temps du bétonage des dames dans la mine "Döllinger". Cependant il faut dire que le niveau piézomètrique était abaissé au cours de ces travaux seulement sur la cote de + 172 m s.m. Cette coincidence montre que le débit environs 200 $l.s^{-1}$ représente la somme de l'écoulement souterrain total provenant de l'étendue entière du corps de porphyre (approximativement 80 km^2 découverts pour l'infiltration). Cela signifie qu'à partir de l'abaissement de 50 m contre l'état contemporain, le débit ne croîtra plus et restera en 200 $l.s^{-1}$. Il faudra conformer la vitesse de fonçage de la mine proposée à cette constatation. Cette hypothèse sera vérifiée par un essai de groupe de pompage en durée de 2 ans, que l'on prépare pour les années de 1982-1984.

6. Actuellement on n'a pas des données suffisantes pour pouvoir déterminer l'influence de l'abaissement profond du niveau des eaux porphyriennes ni sur la forme du cône d'appel, ni sur le changement du régime hydrogéologique de porphyre etc. Les données nécessaires pour ces évaluations n'offrira que l'essai de pompage à long terme mentionné plus haut.

L'agrandissement du débit des eaux thermales en conséquence d'abaissement du niveau piézomètrique des eaux porphyriennes peut être estimé si l'on prend pour base cette considération: Le niveau des eaux porphyriennes au bord septentrional du bassin miocène atteint + 360 m s.m. Près de la dislocation dite de Mstišov, laquelle représente une barrière hydraulique analogue à la dislocation "Barborská"

le niveau des eaux porphyriennes n'est que + 220 m s.m. La distance des points mentionnés étant de 1900 m, la pente hydraulique résulte de 0,074. Abaissant le niveau des eaux porphyriennes de la côté meridionale de la dislocation de Mstišov sur + 80 m s.m., la pente hydraulique augmente sur 0,147, c.-à-d. à peu près deux fois. Cela signifie aussi l'agrandissement du débit des eaux thermales deux fois, l'acroissement de 23 - 27 $l.s^{-1}$ à 46 - 54 $l.s^{-1}$.

7. Faute du bilan précis de la nappe de l'eau porphyrienne il est aussi impossible de faire le bilan géothermique. Mais on peut évaluer (à la manière très simplifiée) l'influence d'écoulement souterrain élevé sur l'abduction de chaleur des roches dans la zone d'accumulation des eaux thermales. On va considérer l'état actuel comme l'état d'équilibre géothermique. La densité de flux de chaleur terrestre étant de 84.10^{-3} $W.m^{-2}$ (21.10^{-3} $cal.s^{-1}.m^{-2}$), l'abduction de chaleur d'une superficie de 40 km^2 (qui tombe sur la substructure hydrogéologique des thermes de Teplice) fait approxim. 3 350 kW ($\triangleq$ 800 $kcal.s^{-1}$). Le débit originaire des sources de Teplice étant de 21 $l.s^{-1}$, la température superficielle de 8° C et dans la zone d'accumulation de la therme de 58° C, le débit calorifique des sources était 4 400 kW ($\triangleq$ 1 050 $kcal.s^{-1}$). Combien de fois augmentera la quantité de l'eau passée par la zone d'accumulation, tant de fois augmentera la quantité de chaleur terrestre portée par les eaux thermales. La différence entre le flux de chaleur enlevé par l'eau actuellement et dans le futur sera couvert par la chaleur, laquelle se libérera pendant la diminuation de la température des roches dans la zone d'accumulation de l'eau thermale. La quantité de chaleur, laquelle sera enlevée en plus en comparaison d'état actuel, atteindrait au cours de 15 ans de pompage

$$Q_{15an} \triangleq (54 - 21)\ l.s^{-1} \cdot (58 - 8)\ deg \cdot 4{,}19\ W.s.deg^{-1} \cdot$$
$$\cdot\ 31\ 560\ 000\ s \cdot 15\ an \triangleq 3{,}27 \cdot 10^{12}\ kW.s\ (\triangleq 7{,}83x \cdot 10^{11} kcal)$$

Cette quantité de chaleur produit un volume de porphyre 1,96 . 10^9 m pendant la diminuation de la température sur 1^0 C. Avec 40 km^2 de l'aire de la zone d'accumulation des eaux thermales représente ce volume une plaque de 39 m de puissance.

Le calcul estimatif mentionné ci-dessus prouve que l'abaissement du niveau piézomètrique des eaux porphyriennes sur la cote de + 80 m s.m. en durée de 15 ans n'amenera qu'un abaissement peu important de la température des roches dans la zone d'accumulation des eaux thermales.

À la fin de cette contribution je voudrais souligner que les irruptions d'eau porphyrienne dans les mines de Duchcov et leur influence sur les sources thermales de Teplice ainsi que la "protection réparative" comme l'extension des travaux suivants de recherches et d'exploration sont sans précédent dans le monde entier. Également le projet d'abaissement profond du niveau des eaux porphyriennes pour pouvoir exploiter la lignite dans les environs de Duchcov à ciel ouvert est unique en son genre. Mais dans le proche futur à defaut des combustibles et minerais il sera nécessaire de suivre le même chemin dans d'autres rayons miniers de la Tchécoslovaquie (p. ex. dans la Bohême de l'Ouest, le bassin de lignite de Sokolov et la ville d'eaux Karlovy Vary, le bassin de lignite de Cheb et la ville d'eaux Františkovy Lázně, le basin de lignite de Nováky dans la Slovaquie centrale et la station balnéaire Bojnice etc. (et peut-être plus tard dans d'autres pays du monde). Les expériences de Teplice pourront bien aider à résoudre les problèmes semblables dans le monde entier.

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Explication des tableaux

Tab. 1. Carte géologique schématique de la structure hydrogélogique de Teplice.

1 - gneiss (Protérozoicum sup.); 2 - phyllite (Paléozoicum inf. ?); 3 - porphyre quartzeux de Teplice (Permien inf.); 4 - granit acide de Cínovec (Permien sup.); 5 - marnes (Crétacé sup.); 6 - roches volcaniques et pyroclastiques (Oligocène - Miocène inf.); 7 - argiles avec une couche de lignite (Miocène inf.); 8 - dislocations; 9 - sources thermales: H - Horský, K - Kamenolázeňský, P - Pravřídlo, O - Obří (Source gigantesque), Tp-28 - forage sur la Place de la Paix; 10 - lieux des irruptions dans les mines en 1879 - 1897: D - Döllinger, V - Viktorin, G - Gizela; 11 - forages actuellement existants.

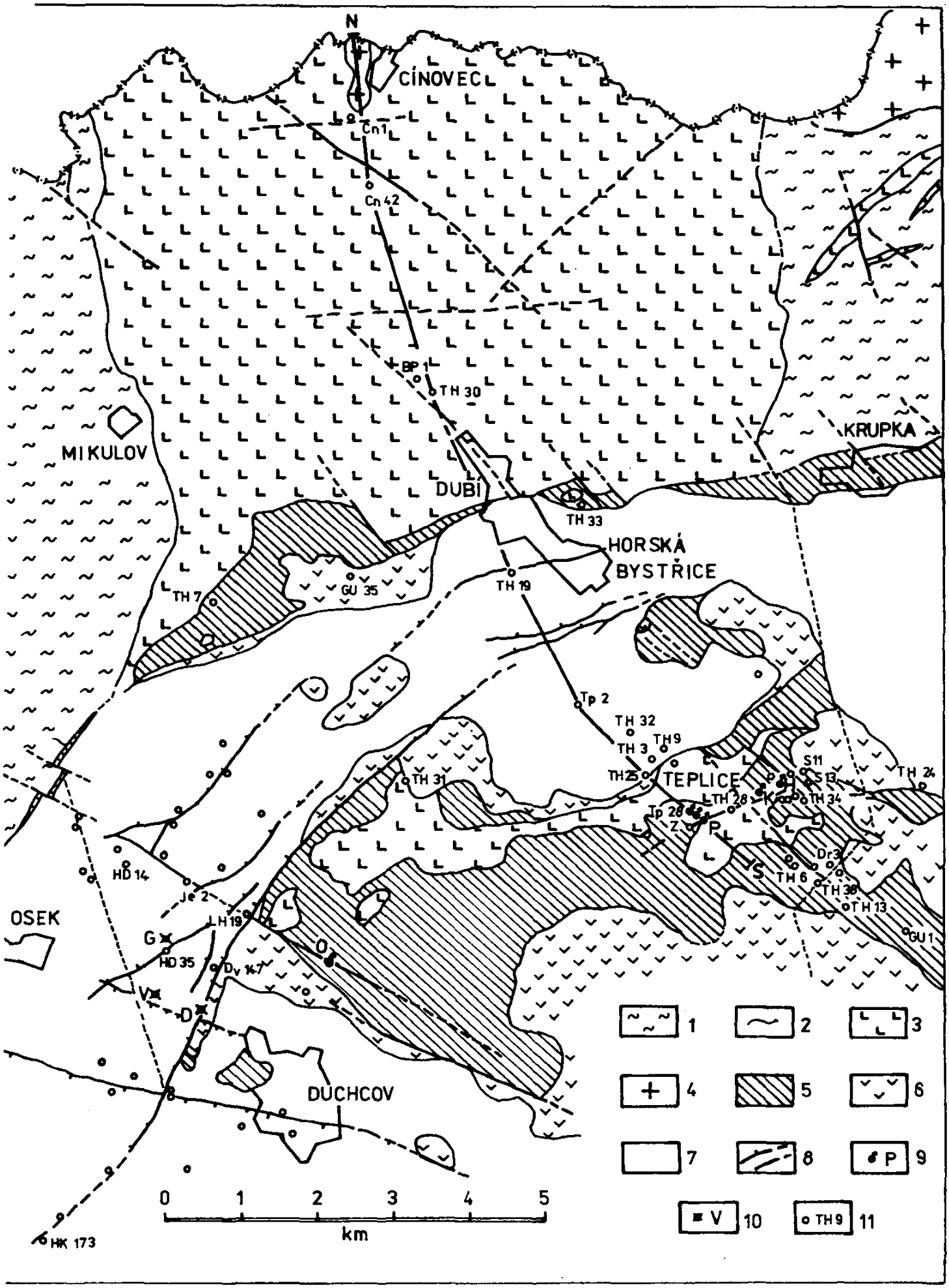
Tab. 2. Coupe géologique à travers de la structure hydrogéologique

de Teplice (du Nord vers Sud).

1 - porphyre quartzeux de Teplice (Permien inf.); 2 - granit acide de Cínovec (Permien sup.); 3 - marnes, à la base avec grès et conglomerates (Crétacé sup.); 4 - argiles (Miocène inf.); 5 - lignite (Miocène inf.); 6 - dislocations constatées et supposées; 7 - puit et galérie inférieur de la mine Cínovec; 8 - galérie de transport planifiée; I+G - zone d'infiltration, de descente et (dans profondeur de 2000 - 3000 m) de genèse des eaux thermales; G+R - zone de genèse et d'accumulation des eaux thermales (dans profondeur de 2000 - 3000 m); A - zone d'ascension des eaux thermales.

Tab. 3. Zones hydrodynamiques de système thermal de Teplice

1 - 3: zones d'infiltration, de descente et (dans profondeur de 2000 - 3000 m) de genèse des eaux thermales: 1 - dans porphyre de Teplice, 2 - dans granit de Cínovec, 3 - dans gneiss; 4 - 5: zones avec nappe captive, avec niveau piézomètrique 240 - 380 m s.m., dans lesquelles la circulation profonde est empêchée par les dislocations longitudinales imperméables: 4 - dans porphyre; 5 - dans gneiss; 6 - 8: zones sous l'influence de drainage dans les mines de Duchcov, dans lesquelles la circulation profonde est freinée par les dislocations transversales semiperméables: 6 - dans porphyre, avec niveau piézomètrique 195 - 201 m s.m.; 7 - dans porphyre, avec niveau piézomètrique 204 - 215 m s.m.; 8 - dans gneiss, avec niveau piézomètrique 210 - 225 m s.m.; 9 - 10: zones sous influence immédiatement de drainage dans les mines de Duchcov, avec niveau piézomètrique 192 - 194 m s.m.: 9 - dans porphyre, 10 - dans gneiss; 11a: sources thermales: H - Horský, K - Kamenolázeňský, P - Pravřídlo, O - Obří (Source gigantesque), Tp-28 - forage sur la Place de la Paix; 11b - lieux des irruptions dans les mines en 1879 - 1897: D - Döllinger, V - Viktorin, G - Gizela; 12 - forages qui ont atteindu le socle paléozoique: a - encore existants (avec chiffre), b - déjà liquidés (sans chiffre).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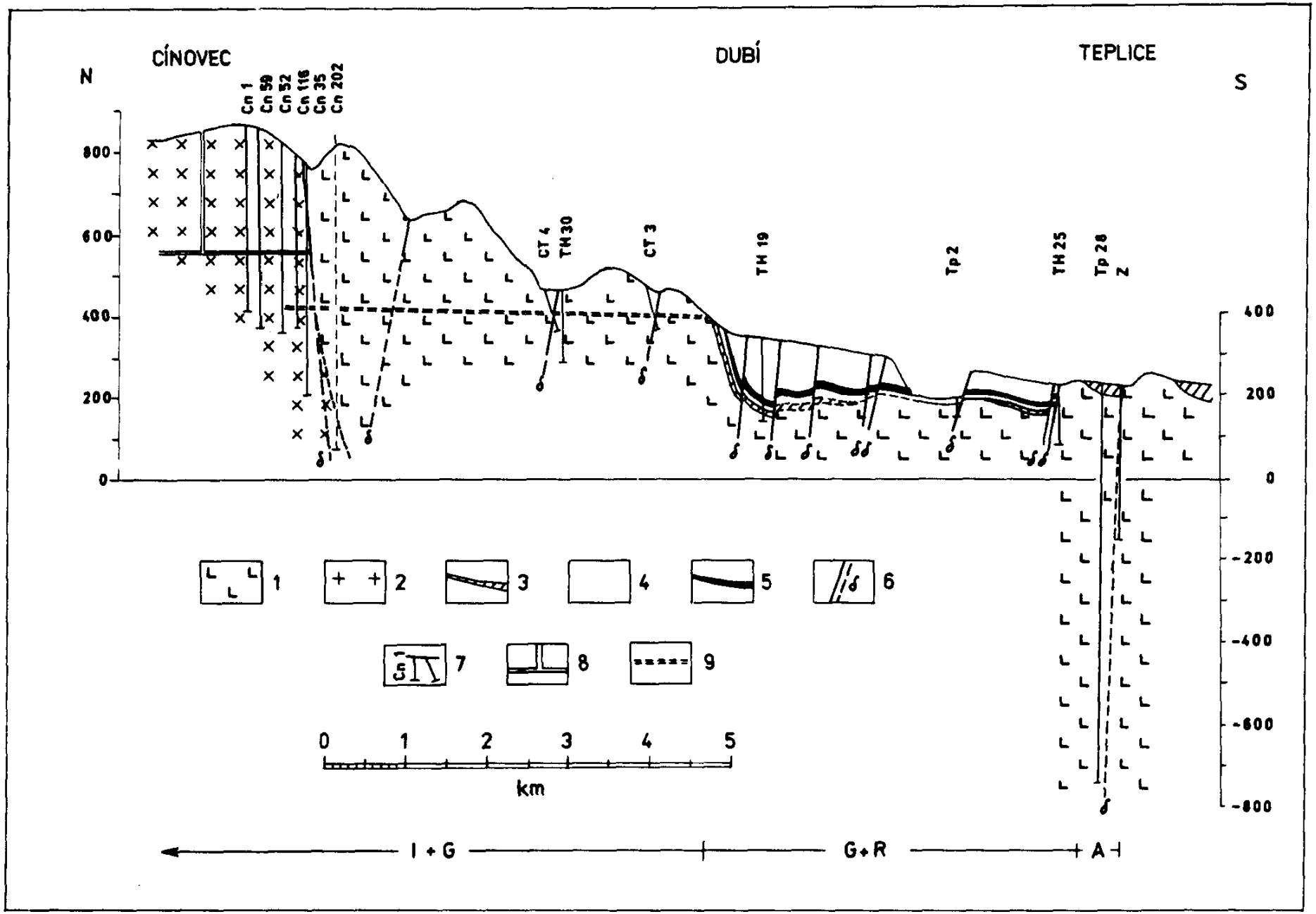
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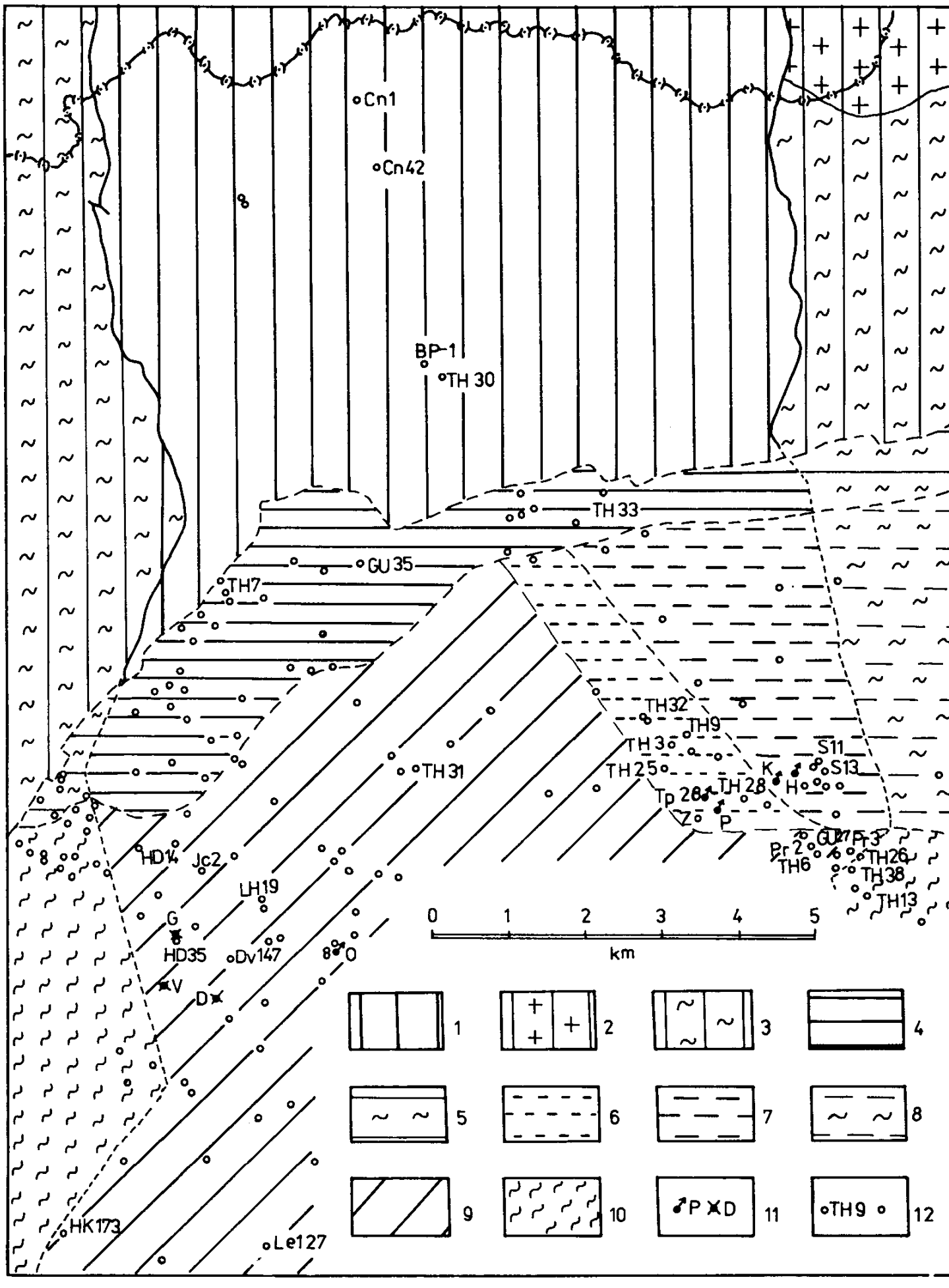
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THE IMPLICATIONS OF GRAVEL EXTRACTION ON GROUNDWATER CONDITIONS

Lloyd, J. W.

ABSTRACT : The mining of gravel for construction is a major industry in the United Kingdon. A large proportion of the gravel extracted is from alluvium in river valleys, where groundwater abstraction for water supply purposes is important and farming is extensive. Gravel operations affect the groundwater flow patterns on a short term basis when dry working is carried out and on a long term basis when poor permeability back-filling is used.
The paper outlines the principles of the effects of gravel operations causing derogation of wells, pollution and water loggind of farming land. A digital model is used to illustrate the degree of permanent effects under gravel back-filling in a valley.

RESUME : L'exploitation de gravier pour la construction constitue une industrie importante en Royaune-Uni. Une grande partie du gravier provient d'alluvions de vallées fluviales où existe une extraction intensive d'eaux souterraines pour la consommation humaine et l'usage agricole. L'extraction du gravier affecte la distribution de l'écoulement souterrain pendant peu de temps lorsque l'opération se réalise à sec et pendant longtemps lorsqu'on utilise un colmatage de faible perméabilité.
Ce travail présente les effets de l'extraction de graviers qui entraîne l'abandon des puits, la pollution et l'inondation de la terre du chantier. On utilise un modèle digital pour illustrer la permanence des effets sous un colmatage de gravier dans une vallée.

RESUMEN : La explotación de grava para la construcción constituye una importante industria en el Reino Unido. Gran parte de la grava procede de aluviones de valles fluviales en los que existe una extracción extensiva de aguas subterráneas para consumo agrícola y humano. La extracción de la grava afecta a la distribución del flujo subterráneo durante poco tiempo cuando la operación se realiza en seco, y durante largo tiempo cuando se usa un relleno de baja permeabilidad.
Este trabajo presenta los efectos de la extracción de gravas que traen consigo el abandono de pozos, la polución y el encharcamiento de la tierra de labor. Se utiliza un modelo digital para ilustrar la permanencia de los efectos bajo un relleno de grava en un valle.

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Introduction

With the reorganisation of the water industry in the United Kingdom increased attention is being paid to many of the minor aquifers in the country among which may be listed the widespread river-valley gravels. Parallel with the developing groundwater interest in these deposits is the continued extraction of gravel and sand for the construction industry and the growing environmental problems of pollution and land reclamation related to extraction sites. In certain areas, therefore, there can exist a conflict of interests in a gravel deposit: the gravel extractor may be interested in the deposits as a material, and the water-supply engineer in the water-bearing potential of the deposit. As the availability of construction materials decreases and the development of groundwater resources and environmental and pollution controls increases, a much clearer understanding of the implications of gravel extraction for water supply will be required, so that, where possible, compatible development for both industries can proceed.

The possible impact of gravel abstraction on the groundwater situation is outlined and discussed here. The gravels that are most likely to provide a dual resource potential are those in the valley floor, as these will probably be water-bearing. Higher-level terrace gravels may not contain water. If it is present, however, the situation is analogous to the valley-floor gravels. A digital model representation is used to illustrate some of the long-term effects of gravel extraction operation.

Short-Term Impact

The impact of gravel abstraction upon groundwater should be considered both in the short term and in the long term. The short-term problems can be listed under five broad categories: groundwater abstraction from workings, which may result in derogation of neighbouring water supplies (Figure 1); phenol pollution, which may enter the aquifer from site plant; bacterial pollution of surface water in working pits, which may be transferred into the aquifer; siltatio of secondary permeability aquifers and induced polluted recharge, drawn into the workings from neighbouring rivers, which may be introduced into the aquifer saline water can be a problem in estuary areas.

Groundwater abstraction during gravel extraction can sometimes reach large prop ortions; derogation is inevitable in the overall context, although locally it will depend upon the detailed groundwater flow distribution. Irrespective of the local derogation situation, however, a disposal of good-quality water to polluted rivers or canals frequently occurs - which is obviously poor management of resources. Planning authorities therefore increasingly include in

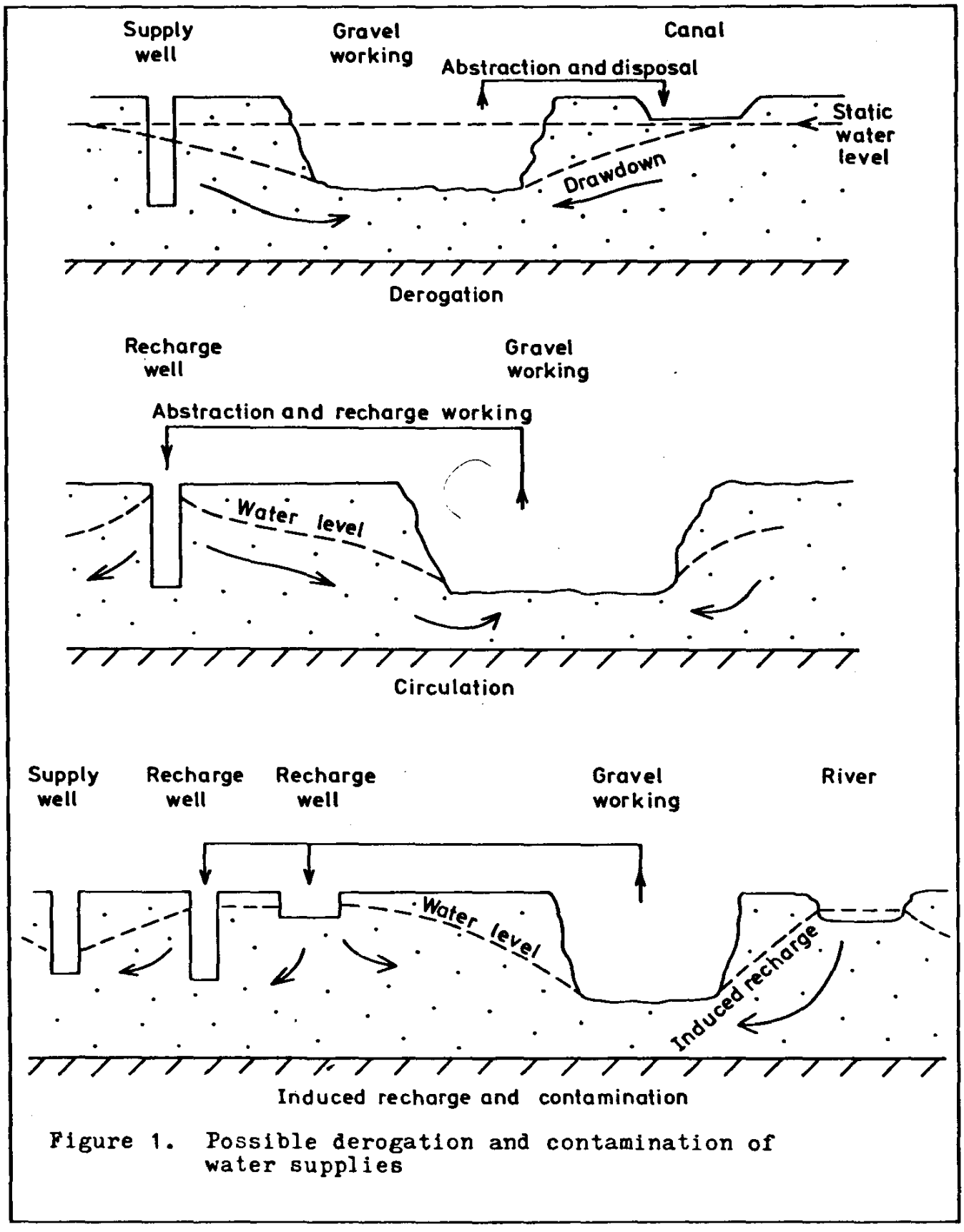


Figure 1. Possible derogation and contamination of water supplies

their planning requirements for extraction sites, clauses that demand that abstracted groundwater should be returned to the aquifer. Such requirements undoubtedly impose constraints on the gravel extractor, but in groundwater terms they may not be totally realistic; for, although the return to the aquifer of groundwater by artificial recharge techniques is in concept simple, in reality it can be difficult.

Artificial recharge can be carried out by spreading and basin techniques or recharge wells. Several factors can, however, mitigate against effective recharge:

(i) the volume of water to be returned to the aquifer is likely to require a large prepared recharge area where spreading is proposed;

(ii) recharge wells can prove expensive;

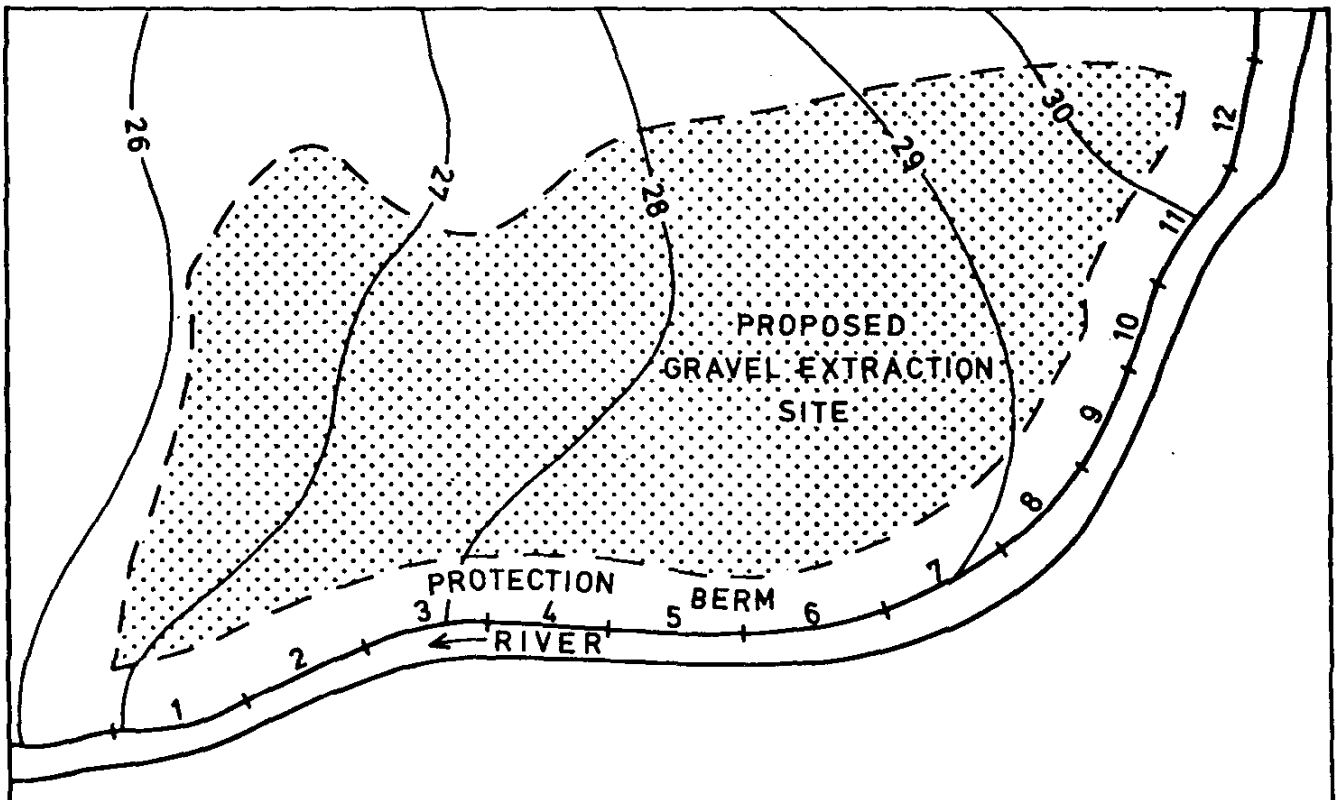
(iii) as a cone of depression will exist around the extraction site, a recharge mound must be located at an adequate distance to avoid circulation (Figure 1) - this is particularly critical with well input, as steep groundwater gradients may be created;

(iv) groundwater velocities may be high under recharge gradients, so that polluted water can be rapidly introduced into the aquifer (Figure 1);

and (v) gravel operators may well produce aerated and turbid water, which, if introduced directly into the aquifer, can reduce effective permeability, and algal growth in spreading areas can reduce effective permeability.

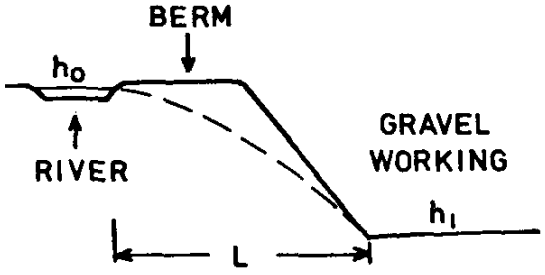
If gravel extractors wish to work valley gravels, they may therefore eventually be faced with only two alternatives: either to operate under water, or to artificially recharge, despite the inherent difficulties. As can be seen from the calculations given on Figure 2 the possible flow into a proposed gravel working can be large so that the ensuing derogation and artificial recharge can be considerable. Undoubtedly, most of the recharge problems can be overcome with experience, but the extractor must expect to embark upon a reasonably comprehensive hydrogeological study of the site and adjacent areas before extraction is commenced, and to formulate a definite recharge policy that will minimize the wastage of groundwater resources. It seems probable that a certain amount of field experimentation may be required by the extractor for a decision on the most cheap and efficient methods.

The short-term pollution problems vary in degree. Phenol pollution can be minimized by careful plant operation. If large spillages occur, these can usually be skimmed off and pumped to waste. A certain amount of low-level bacterial pollution of open water surfaces in gravel pits is inevitable, and its significance with respect to neighbouring supply wells will depend upon groundwater travel times within the aquifer. A residence of two weeks will usually reduce most bacterial content to acceptable levels. Where artificial recharge is being practised, the possibility of the direct introduction of phenols and bacteria to the aquifer will require attention, and the location of the recharge site will be important.



Possible Inflow Calculation

$$Q = \frac{Km \quad h_o^2 - h_1^2}{2L} \quad (m^3/day)$$



1	95	7	324
2	735	8	344
3	780	9	392
4	340	10	435
5	595	11	642
6	621	12	189
			5492
	Groundwater flow		950
			6442 m^3/day

K = coeff. of permeability

m = section width

= Final abstraction requirement for dry work.

Figure 2. Gravel working in flow calculation

The problems of polluted recharge from neighbouring rivers, etc., into gravel workings, as a function of abstraction, may be difficult to assess before operations. It can prove the most severe of the pollution hazards, particularly where artificial recharge is to be adopted, in that high levels of inorganic and organic pollutants may be introduced into the aquifer. The chief difficulties lie in the assessment of the permeability of the adjacent river bed, the distribution of this permeability and the possible inflow values. Although pumping tests at selected well-sites may provide information about induced recharge through yield-drawdown analysis, hydrochemical data, tracers, etc., the drawdown effects of a large pit operation will undoubtedly prove more extreme with time. Any indications from the investigation stage that induced polluted water can occur must, therefore, jeopardize the development of a deposit as a material resource, or at least restrict extraction to a safe distance from the polluted source.

Potential pollution hazards should also be considered where old backfilled gravel workings are present near a new site. In certain cases, untreated refuse may have been used as backfill material. Dewatering of the new site can disturb the groundwater and hydrochemical situation in the backfilled area and induce polluted water into the groundwater system.

Where gravel extraction occurs in deposits overlying fissured aquifers siltation can pose problems and effectively reduce permeability. It is unlikely that siltation will be extensive under normal groundwater velocities ; however, if active pumping wells are close to an extraction site then high velocities may occur and siltation may become a serious problem. Some assessment of the likely transport velocities and the particle sizes involved may be obtained from Figure 3.

Long-Term Impact

The long-term impact of gravel extraction upon groundwater concerns the reclamation of the extraction site after operations have terminated, or progressive reclamation during operations. Two basic situations can occur: either the pit is left open, or it is backfilled. The long-term decision in this respect is the responsibility of the local planning authorities. Preference varies throughout the country, and is dependent largely upon the requirements of farming, sporting amenities, waste disposal, etc. The impact on the groundwater resources is largely overlooked.

Where the planning permission imposes no backfill requirements on the extractor, the position is relatively straightforward. The presence of a large open body of water in a groundwater system can, however, pose problems. The hydraulic continuity between the open water area and the aquifer is the most important factor. If good continuity exists, then it may be asked whether pollutants such as coliform bacteria, which will undoubtedly be present in amenity areas, are likely to degrade before they enter a supply well. In general, continuity is reduced by the growth of algae along the sides and base of the pit, and a fairly impermeable flow-barrier situation may develop under steady-state conditions. This impermeability, although it reduces pollution of the aquifer, may, however, seriously affect the local groundwater flow distribution (see below) (Figure 4).

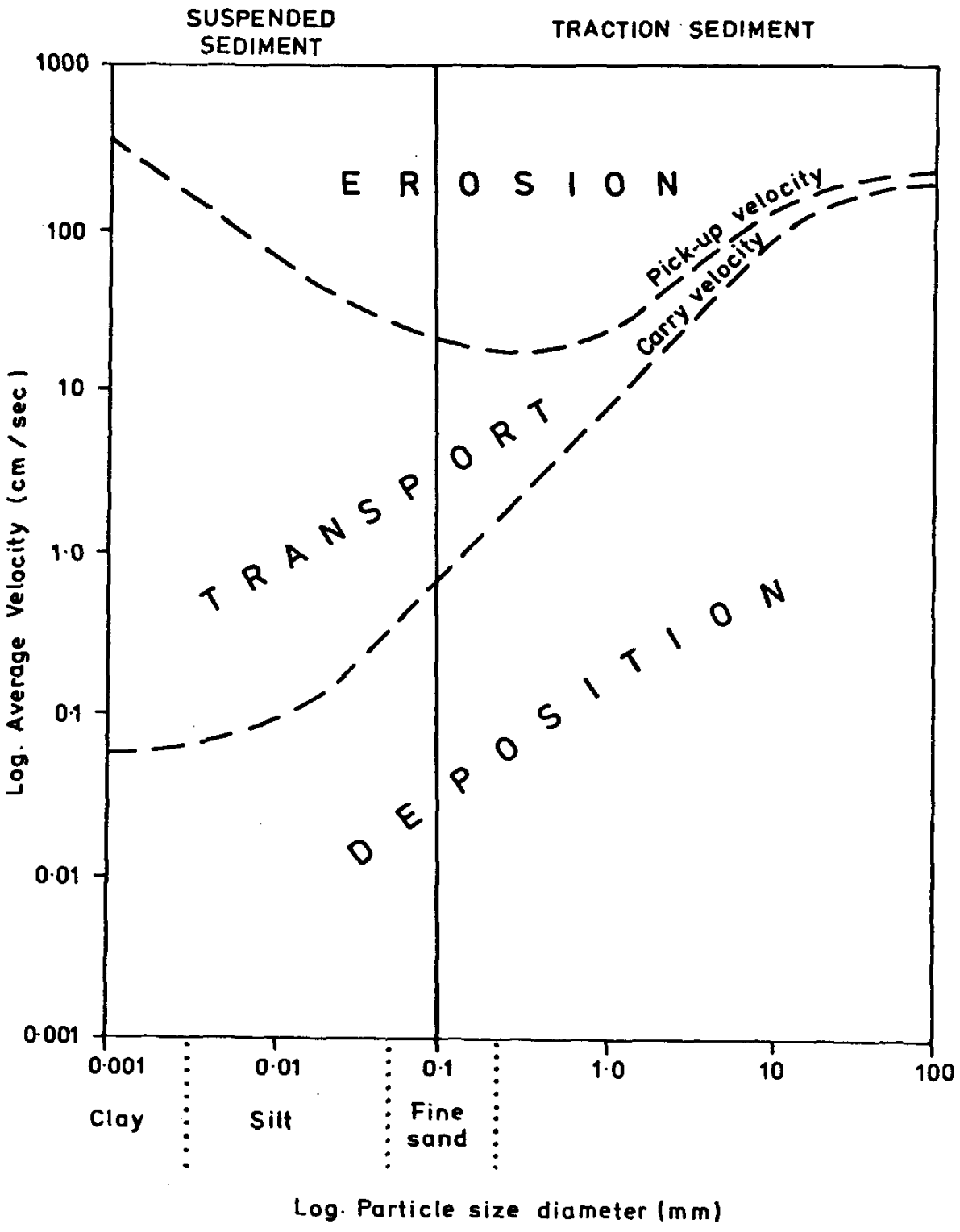


Figure 3. Particle transport

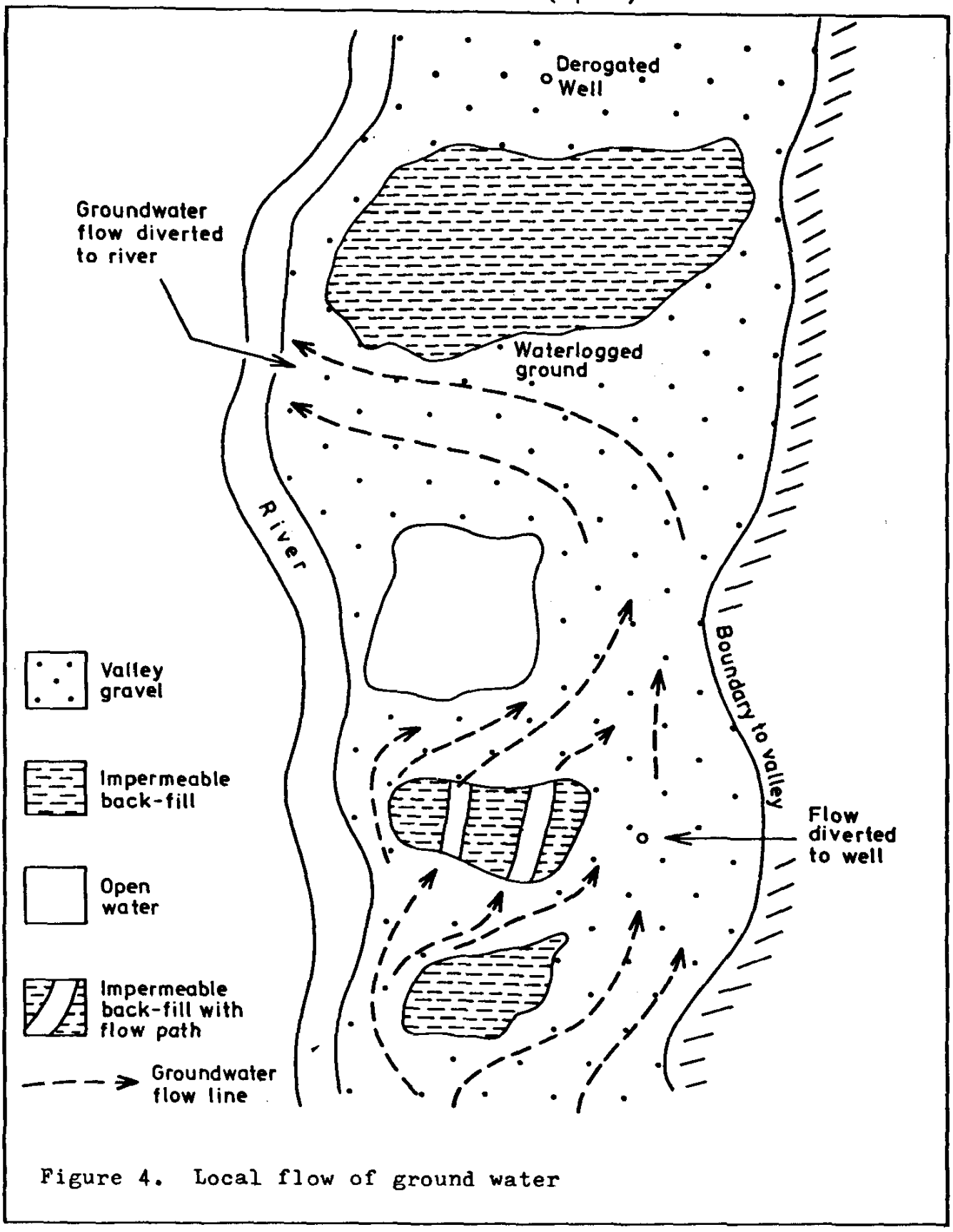


Figure 4. Local flow of ground water

The extent of the impermeable zone will be important: if it is too large, groundwater flow will be significantly curtailed and long-term derogation of downstream supplies may result. With an extensive impermeable backfill zone, groundwater can either be forced to the surface - which results in water logged land - or move into adjacent water courses, where possibly it may be lost into a polluted system (Figure 4). Where an impermeable zone of only minor extent is created, the flow distribution can, in fact, be disturbed to the advantage of the water abstractor in that flow may be diverted towards a well. Whatever the extent of an impermeable backfill, the possible effects on the groundwater flow pattern of its introduction should be analysed prior to implementation.

In general, from a water-supply viewpoint, the exclusive use of impermeable backfill should be avoided. Where possible, an attempt should be made to maintain an acceptable groundwater flow through backfilled sections. This could be achieved either by retaining permeable flow paths of in-situ gravel through the site, or by planned backfill operations with permeable waste materials used to create flow paths through impermeable sections. Problems obviously arise for the extractor, in that, in the first case, the extractable amount of gravel is reduced, and, in the second, backfill operations may become more expensive owing to the batching of materials. Whichever technique is adopted, the problems of grading and filtration between materials will have to be examined to avoid siltation of the flow paths. Despite the difficulties, however, the introduction of flow paths in backfill sections may well become a requirement of future planning permissions for extraction sites (Figure 4).

To illustrate the groundwater disturbance problems involved in gravel workings a digital model example from northern England may be used. The model has been prepared to study further gravel extraction and back-fill operations in an already disturbed valley. The valley is typical with gravel thickness ranging up to 12 m and is underlain by marls with thin limestones.

Where the planning authorities require that the gravel extractor should back-fill a site, stipulations vary. Frequently, reclamation for agricultural purposes is envisaged, and the chief requirements are related to suitable top soils. Backfill is stipulated down to a certain level, but below the 'agricultural zone' fill materials are often left to the discretion of the gravel extractor. Backfill materials that can be used are basically fine-grained tailings and overburden from the gravel pit area, industrial and domestic waste.

As the gravel extractor will be working the highest-grade gravels, the removal of these materials will, in itself, radically disturb groundwater conditions. The degree of permanent disturbance, however, will depend upon the nature of the backfill material. Undoubtedly, the tailings will be incorporated in the backfill, reducing the permeability of the ground. Additional fine-grained waste material such as pulverized fly ash will frequently be used, and will effectively create flow-barrier conditions within the aquifer (Figure 4).

A classical hydrogeological study was carried out in the valley and simulation of the aquifer was obtained using the equation adopted by Walton (1970)which describes the time - variant drawdown throughout an aquifer when the vertical components of flow are considered sufficiently small to be neglected.

$$\frac{\partial}{\partial x}\left(T_x \frac{\partial s}{\partial x}\right) + \frac{\partial}{\partial y}\left(T_y \frac{\partial s}{\partial y}\right) = S \frac{\partial s}{\partial t} + Q \qquad \text{(i)}$$

where Tx and Ty are the transmissivities in the x and y directions, s is the drawdown, S is the specific yield of the aquifer in this case and Q is the quantity of water entering the aquifer per unit area per unit time. For a practical solution of the equation throughout the aquifer system time and space dimensions were divided into discrete intervals (Pinder and Bredehoeft, 1968) and for the finite difference approximation a backward difference approach was adopted (Rushton, 1973) in which

$$T_x \frac{\partial^2 s_{n+1}}{\partial x^2} + T_y \frac{\partial^2 s_{n+1}}{\partial y^2} = S \frac{s_{n+1} - s_n}{t} + Q_{n+\frac{1}{2}} \qquad \text{(ii)}$$

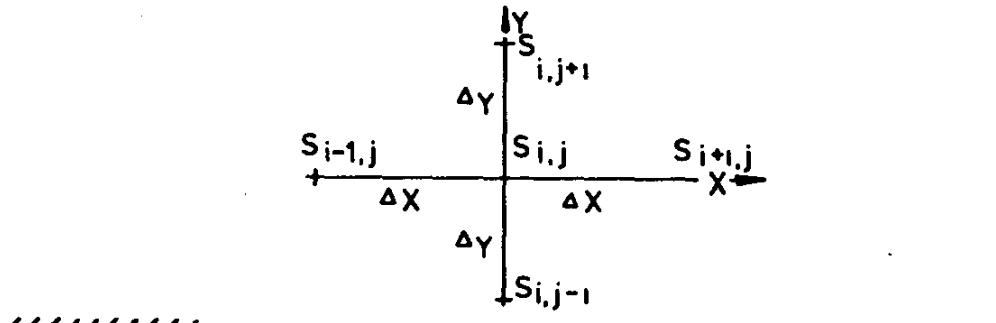
With this approximation the space derivative is centred at a time $(n+1)\,\Delta t$ and the time derivatives at time $(n+\frac{1}{2})\,\Delta t$. For computational purposes the aquifer is divided into a grid with a nodal distribution (Si, j) as shown on Figure 5 and equation (ii) is given as

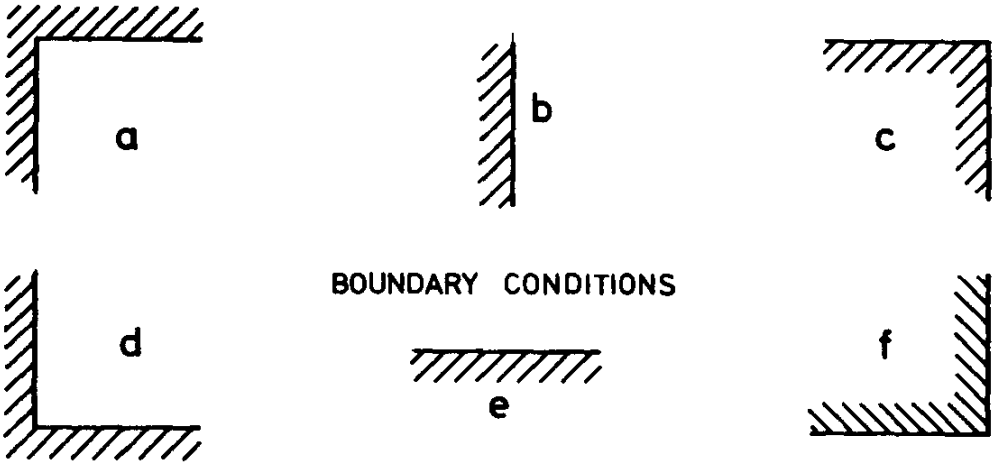
$$T_{x,i,j} \frac{s_{i+1,j} - 2s_{i,j} + s_{i-1,j}}{\Delta x^2}_{n+1} + T_{y,i,j} \frac{s_{i,j+1} - 2s_{i,j} + s_{i,j-1}}{\Delta x^2}_{n+1}$$

$$= S \frac{s_{i,j,n+1} - s_{i,j,n}}{\Delta t} + Q_{i,j,n+\frac{1}{2}} \qquad \text{(iii)}$$

The boundary condition of a 'fixed head' is applied directly by retaining the appropriate head at the specified value. For an impermeable boundary the condition of 'no flow' crossing the boundary s/y = 0 is applied through a fictitious node i,j+1 by setting $S_{i,j+1} = S_{i,j-1}$. The finite difference approximations lead to a large number of simultaneous equations which were solved using iterative over-relaxation techniques (Rushton, 1974).

To construct the model, the area was subdivided on a grid as shown in Figure 6 with grid intersections representing data input nodes equally spaced at 0.5 km. intervals. The area of influence for each node is a square 0.5 x 0.5 km. based on the node as a centre point. Using the basic data obtained from field studies namely transmissivity, specific yield, recharge and groundwater level, values were selected for each adjacent nodal pair. In the non-boundary affected nodal areas these values were balanced with surrounding nodal areas using the groundwater flow equation discussed above. Where





BOUNDARY CONDITIONS

TRANSMISSIVITY FACTOR

	X	Y	-X	-Y
a	0·5	0·0	0·0	0·5
b	1·0	0·5	0·0	0·5
c	1·0	1·0	0·5	0·5
d	0·5	0·5	0·0	0·0
e	0·5	1·0	0·5	0·0
f	1·0	0·5	0·5	1·0

SPECIFIC YIELD FACTOR

a	b	c	d	e	f
0·25	0·50	0·75	0·25	0·50	0·75

Figure 5. Boundary modification factors

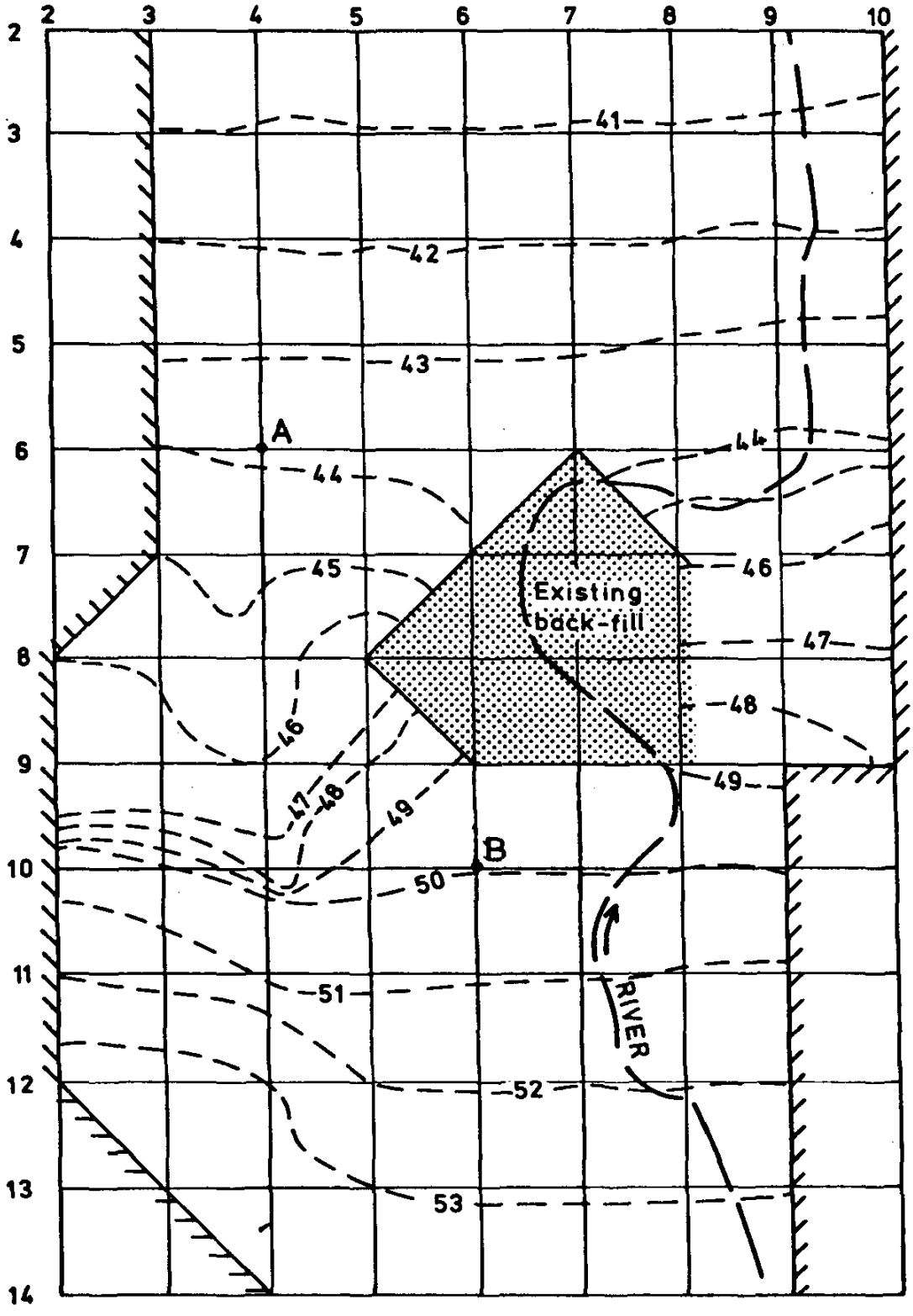


Figure 6. Geometry of model area

boundaries are present different approaches were used. The eastern boundary of the aquifer was determined as a boundary of 'no flow' and along the western boundary a minor recharge was applied due to the presence of some lateral leakage from thin limestones. Otherwise the boundary was treated as a 'no flow' boundary. The northern and southern boundaries were varied seasonally in relation to well hydrograph data.

In attempting to obtain a simulation considerable difficulty was experienced in matching well hydrographs in the area west of the existing back-filled area. In this narrow section and in such a thin aquifer it was found that the simulation was most sensitive to transmissivity; specific yield though also important, proved less sensitive in the model as shown in Figure 7. Eventually a close simulation was obtained and the piezometic surface shown in Figure 6 was portrayed.

The object of the study was to evaluate the effects of proposed further abstraction of gravel west of the existing back-filled area with subsequent impermeable back-filling. Two major problems were envisaged, firstly the derogation effects on well complex A downstream of the proposed workings and secondly the possibility of water-logging of ground in the vicinity of B. In the model the long-term back-fill was increased in three stages as shown on Figure 6 and the water level responses obtained at the two investigation points varied as shown on Figure 8.

From Figure 8 it will be seen that although the water level at A drops slightly the ground permeability is adequate to transmit the diverted groundwater and derogation in theory is not serious. However, the levels at B are seen to rise extremely close to the ground surface in stage 2 and it is anticipated would cause local unacceptable water-logging. Under stage 3 conditions, surface discharge would occur and in fact the water level at A would no doubt drop considerably as the model has only allowed for groundwater flow and discounted surface flow.

The model has therefore demonstrated that although minor extensions to the gravel operations may be contemplated (i.e. stage 1) no further operations should be allowed under the proposed impermeable fill reclamation programme. It is feasible however, that should adequate permeable fill be incorporated in the back-fill that more gravel may be extractable over and above stage 1 without seriously disturbing groundwater conditions.

Conclusions

The potential groundwater development of gravel deposits can, in many situations, be irrevocably reduced by inadequate planning of gravel extraction. It may be argued that in some areas planning authorities should place a complete restriction on gravel extraction to safeguard groundwater flow and long-term water supplies. Elsewhere, provided that adequate hydrogeological data have been obtained and the effects of extraction and backfill methods have been tested, it is reasonable to expect that gravel extraction can progress in harmony with water supply.

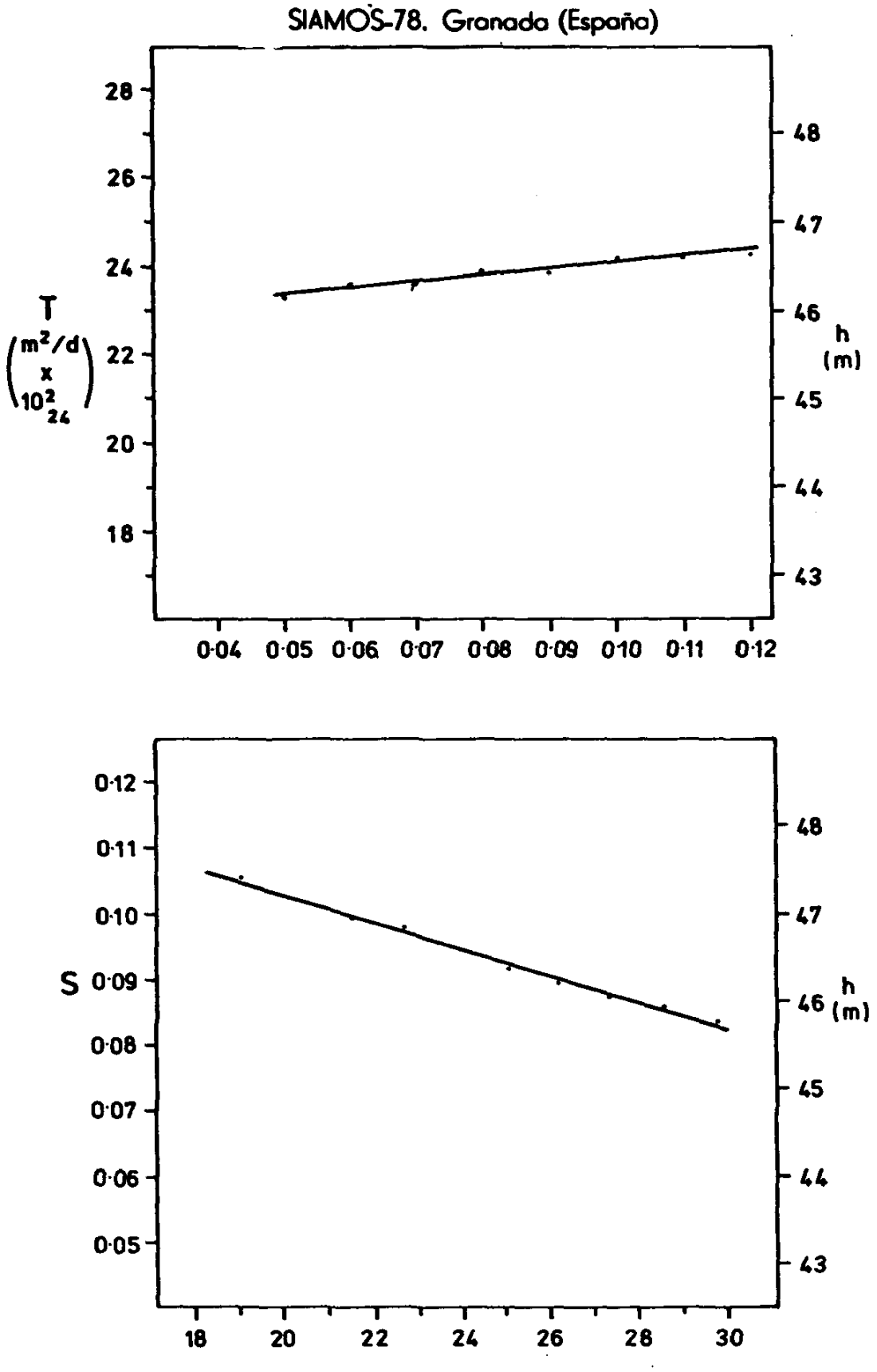


Figure 7. Model sensitivity to T and S

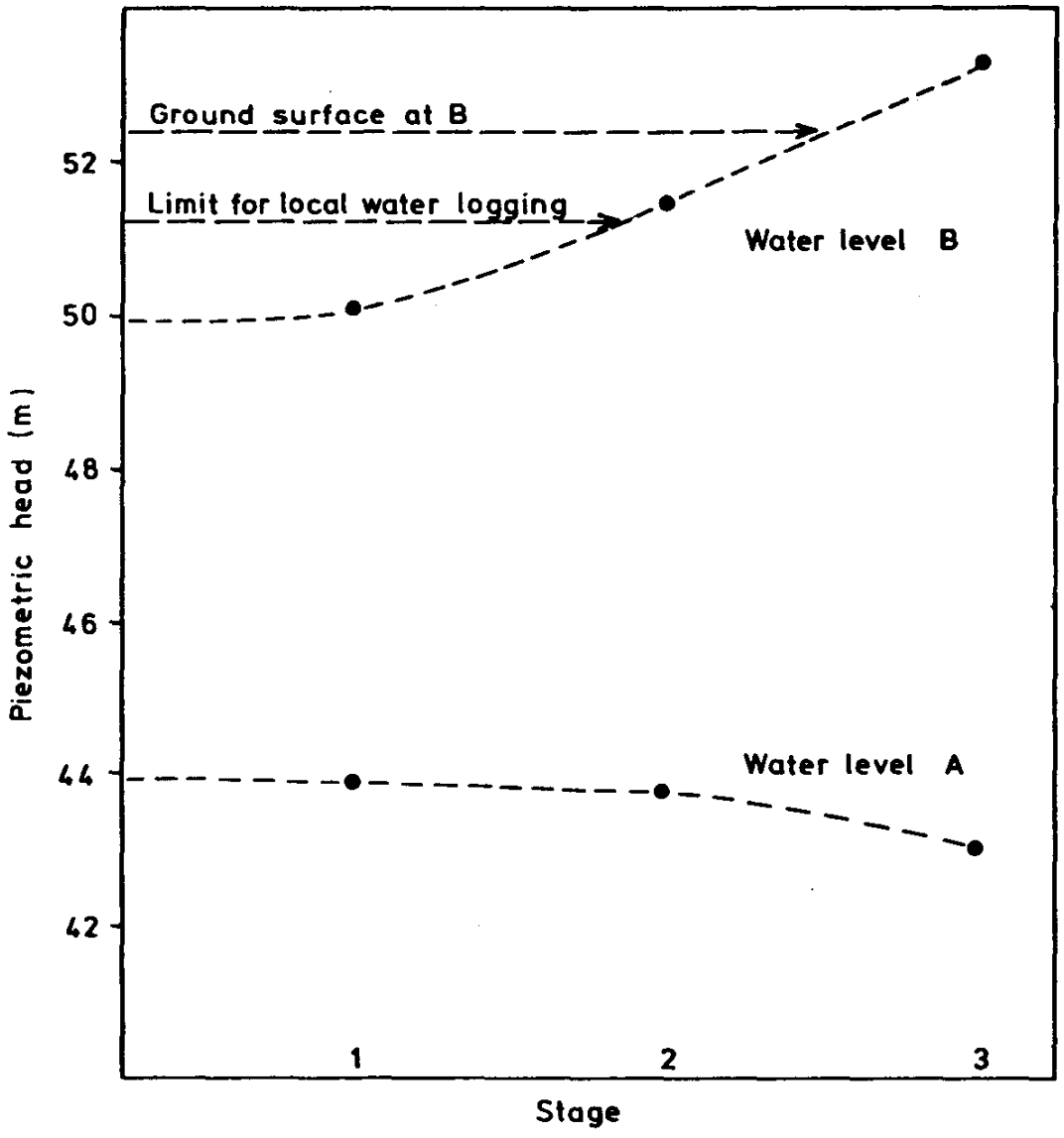


Figure 8. Effect of proposed back-fill operation

The overall responsibility for compatible development ultimately rests with the planning and water authorities. Because of the potential danger to water supply, there is a possibility that planning requirements for extraction sites could become unrealistic; there is a need for direction on both the short- and long-term techniques that can be adopted to solve the dual development of gravel deposits. Undoubtedly, much of the technical onus will fall on the gravel operators, with the increased costs which they imply. As the availability of construction materials continually decreases and the environmental and water-supply constraints multiply, increased costs are inevitable.

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COMPUTATION OF FLOOD FLOWS IN OPEN PITS

Marjanović, N.* ; Prohaska, S.*

ABSTRACT : The paper presents the method of computation of flood flows of different recurrence intervals, under open pit working conditions. The method applied is based on the wellknown relationship for maximum overland flow rate per unit drainage area versus the rainfall depth for the relevant time of duration of the storm event and the runoff coefficient. The paper will be illustrated with an application of the suggested method for the given open pit working conditions.

RESUME : Ce travail présente la méthode de calcul du débit de crue pour différents intervalles de récurrence dans des chantiers à ciel ouvert. La méthode qui est appliquée ici est basée sur la relation connue entre le ruissellement maximal et la hauteur de précipitation pour le temps de durée de l'averse et le coefficient d'ecoulement. Le travail sera illustré par une application de la méthode pour les exploitations à ciel ouvert.

RESUMEN : Este trabajo presenta el método de cálculo del caudal de crecida, para diferentes intervalos de recurrencia en explotaciones a cielo abierto. El método que se aplica aquí se basa en la conocida relación entre el arroyamiento máximo, la altura de precipitación para el tiempo de duración de la tormenta, y el coeficiente de escorrentía. El trabajo estará ilustrado por una aplicación del método para las explotaciones a cielo abierto ya citadas.

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The computation of floods of different return periods in open channels no longer represents a practical difficulty. However, the problem is more complicated in the case of overland flow from natural slopes. The present paper proposes the application of the method of Soviet hydrologist Alekseev, based on the well-known expression for maximum overland runoff

$$q_{max_p} = 16,67 \cdot \varphi \cdot \frac{H_{\tau}}{\tau} \qquad (1)$$

where

q_{max_p} - maximum overland flow rate per unit drainage area ($m^3/s/km^2$)

φ - dimensionless summarized runoff coefficient

H - rainfall depth for relevant time of storm duration τ

τ - relevant time of storm duration

The dimensionless summarized runoff coefficient is given by the product

$$\varphi = \varphi_1 \cdot \varphi_2 \cdot \varphi_{\tau}$$

where

$\varphi_1 \leq 1$ coefficient of nonuniformity of rainfall spatial distribution

$\varphi_2 \leq 1$ coefficient which describes the natural regulation ability of the drainage area

φ_{τ} - coefficient of runoff from maximum rainfall depth $H = a_{\tau}\tau$ for relevant time of storm duration τ.

$a_{\tau,p}$ - maximum average rainfall rate during the relevant time of storm duration τ with the given return period (mm/min).

To render expression (1) practically applicable it is necessary to determine the relevant time of storm duration. This can be done by means of an expression which takes into account the physical and morphological characteristics of the drainage area, in the form

$$\phi = \frac{(100L)^{1/2}}{m_1 I^{1/4} \cdot (\varphi H)_p^{1/2}} \qquad (2)$$

where

L - mean length of the slope; (km)

I - mean gradient of slope;

m_1 - coefficient of resistance of the slope

$(\varphi H)_p$ - daily rainfall total multiplied by the summarized runoff coefficient

One of the key quantities to be determined is the product of the daily rainfall total and the summarized runoff coefficient $(\varphi H)_p = \varphi . H_p$. It is found from the expression

$$(\varphi H)_p = \frac{100 \cdot q_i}{S(\tau_i)} \qquad (3)$$

using actual extreme discharge records and the corresponding values from the regional curve for reducing the average rainfall rate for the time

$$\tau_i = \frac{16,67\ L}{a \cdot I^{1/3} Q_i^{1/4}} \qquad (4)$$

where a is a coefficient.

This method implies the availability of the following storm reduction curves for various time intervals:

1 regional rainfall intensity reduction curve

$$\Psi_p(\tau) = \frac{H_{\tau,p}}{H_p} = f(\tau); \quad (5)$$

2 regional average rainfall intensity reduction curve

$$S(\tau) = 1667\ \frac{\Psi_p(\tau)}{\tau} = 1667\,\overline{\Psi}_p(\tau)\ ;\ \text{and} \quad (6)$$

3 auxiliary regional curve

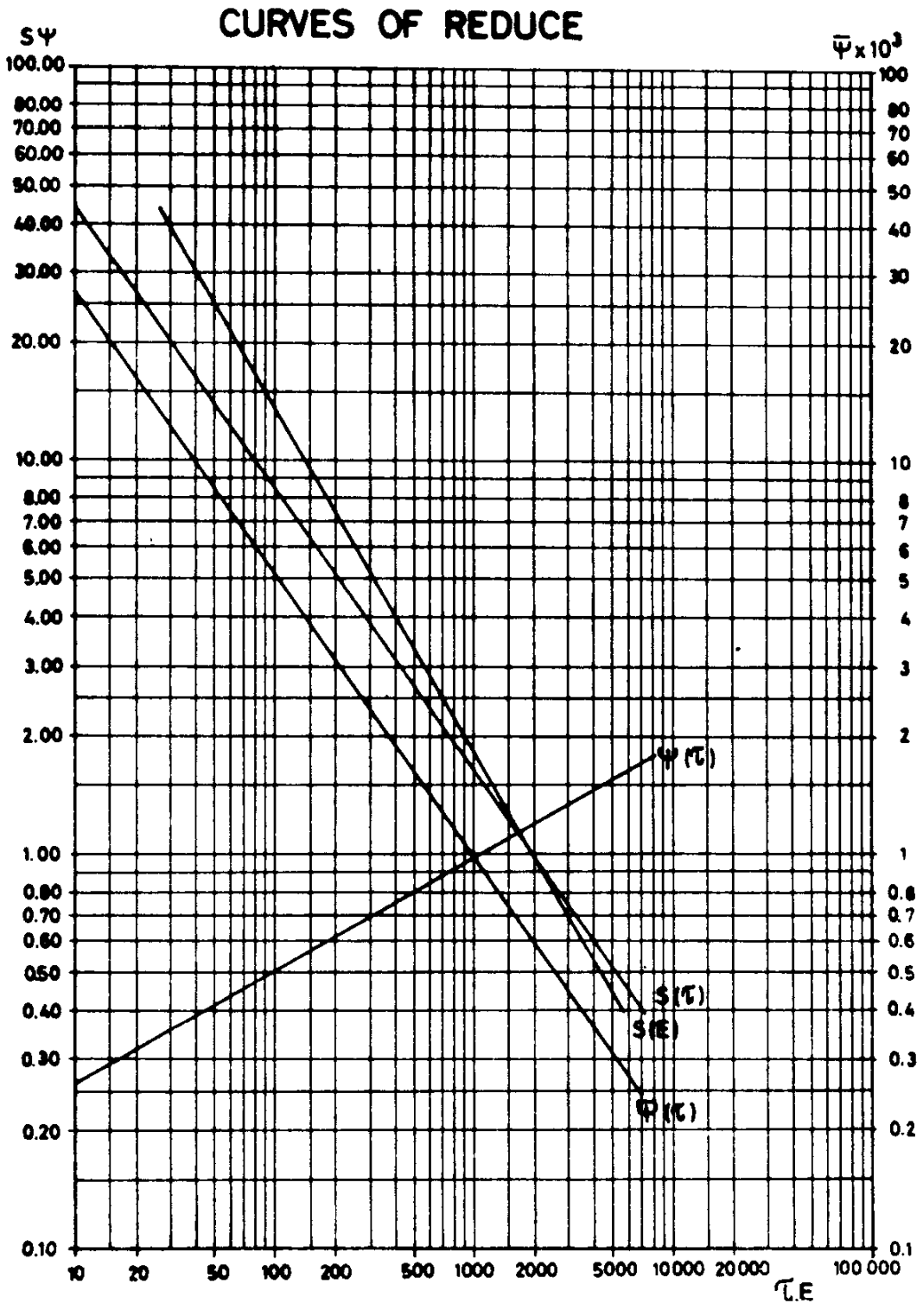
$$E(\tau) = \sqrt[4]{S(\tau)} \quad (7)$$

The procedure for obtaining these curves will be demonstrated on a practical example.

For various values of ϕ determined from expression (2), reference [1] gives corresponding values of τ. For each τ values of $\Psi(\tau)$ are found from curve 1, and then H is found by expression $H=\Psi_\tau H_{1\%}$; $H_{1\%}$ is the rainfall depth with a 100-year return period. Thus all the quantities appearing in (1), and hence the overland flow rate per unit drainage area for different return periods can be determined. Multiplying these values by the drainage area F yields the corresponding total overland flows Q.

In connection with the protection of one of the largest iron ore mines in Yugoslavia from surface water, it was the task of the present authors to determine the total volume of water falling directly into the open pit. The method described here was used. To render its application possible the following initial data were required: drainage area F, slope and average length of the slopes, storm daily rainfall totals in the region, maximum annual discharge records of a nearby gauging station on the same or on a

FIG. 1


CURVES OF REDUCE
SΨ
Ψ̄x10³
100.00
80.00
70.00
60.00
50.00
40.00
30.00
20.00
10.00
8.00
7.00
6.00
5.00
4.00
3.00
2.00
1.00
0.80
0.70
0.60
0.50
0.40
0.30
0.20
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100
80
70
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10
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7
6
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4
3
2
1
0.8
0.7
0.6
0.5
0.4
0.3
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10
20
50
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200
500
1000
2000
5000
10000
20000
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τ.E
Ψ(τ)
S(τ)
S(E)
Ψ̄(τ)

neighboring watercourse of similar physical characteristics. In this particular case the drainage area was divided up into several characteristic parts; the numerical example is given for one of them.

The calculation procedure is as follows:

Using records of storm daily rainfall totals from a rain gauging station in the immediate vicinity, equations (5), (6) and (7) are used to calculate values of the three reduction curves for the relevant durations τ. This calculation is shown in Table 1. Values of $(\varphi H)_p$ are then determined for al the return periods of interest from discharge records of a gauging station on the same or a similar watercourse in the same basin and certain physical and morphological characteristics of the drainage area, using reduction curve 2 and equations (3) and (4), as shown in Table 2. The probability distribution function for (φH) is shown in Fig. 2. Values of ϕ are determined by means of equation (2), and using the tables in reference and the physical and morphological characteristics of the drainage area, values of τ for each ϕ. Using reduction curve 1 and equation (8) H is determined, and thus al the elements needed for the application of equation (1) are found. Then the overland flow rates per unit drainage area are calculated for all the return periods of interest, and multiplied by the drainage area to obtain the floods with the given return periods, as shown in Table 3.

The parameter m_1 appearing in equation (1), and a in equation (4) were found by trial-and-error fitting with annual maximum discharge records of the guaging station, yielding in this case

$$m_1 = 0,20$$

$$a = 0,15$$

T = 100 years

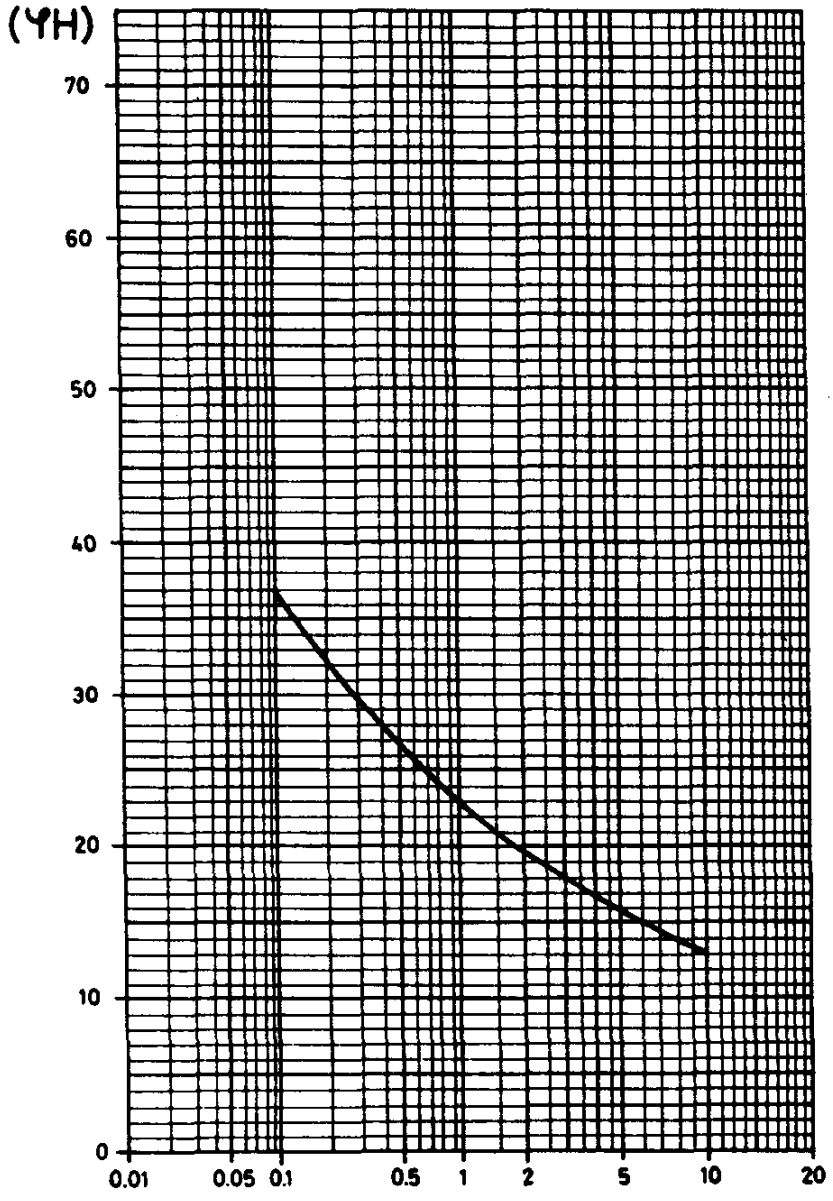
H_p = 128 mm

Tabel 1.

Parameter	TIME PERIOD τ [MIN]											
	10	30	60	120	240	360	720	1440	2880	4320	5760	7200
H	23.5	32.5	39.5	48.5	59.5	66.0	82.0	100.0	123.0	138.0	150.0	160.0
Ψ	0.252	0.349	0.425	0.525	0.640	0.710	0.882	1.075	1.323	1.484	1.613	1.720
$\bar{\Psi}$	0.025	0.012	0.007	0.004	0.003	0.002	0.001	0.001	0.0005	0.0003	0.0003	0.0002
S	42.12	19.42	11.80	7.24	4.44	3.29	2.04	1.25	0.77	0.57	0.47	0.40
E	25.48	62.98	111.20	196.84	348.38	484.84	860.48	1520	2694	3757	4761	6720

FIG. 2

PROBABILITY FUNCTION FOR (ΨH)



DRAINAGE AREA'S PARAMETERS

F = 29,23 km^2

I = 23.9‰

L = 8.14 km

Table 2

N°	Year	Q_{max}	q	τ	S(τ)	φH	φH	$p=\frac{m-0,3}{n+0,4}$
1	1967	12.28	0.420	503.2	3.35	12.53	15.93	6.73
2	1968	9.24	0.316	540.6	3.30	12.15	15.27	16.35
3	1969	5.80	0.198	607.6	3.10	8.25	13.17	25.96
4	1970	9.24	0.316	540.6	3.30	12.15	12.15	35.96
5	1971	4.72	0.161	639.0	3.05	7.00	12.15	45.19
6	1972	5.44	0.186	616.8	3.08	7.75	9.22	54.80
7	1973	10.20	0.349	527.2	3.28	13.17	8.25	64.42
8	1974	5.80	0.198	608.1	3.10	8.25	8.25	74.04
9	1975	12.80	0.438	498.0	3.35	15.93	7.75	83.65
10	1976	6.60	0.226	588.3	3.13	9.22	6.09	93.27

OPEN PIT No. 1

F = 0,244 km^2

I = 171‰

L = 0,75 km

l = 0,29

Table 3

Return Periods T	(φH)	ϕ	τ	ψ_{τ}	H	q	Q
1000	36.52	8.15	185	0.61	78.08	4.93	1.20
100	23.21	9.11	210	0.63	58.59	3.26	0.80
50	19.90	9,46	220	0.63	52.92	2.81	0.69
20	15.96	10.02	230	0.64	46.72	2.37	0.58
10	13.33	10.51	250	0.66	42.24	1.97	0.48

In the author's opinion this method of calculating flood flows under open pit conditions offers certain advantages over standard synthetic procedures, since its parameters are calibrated with respect to actually recorded data for the basin. Its basic shortcoming is that it identifies the summarized parameter (φH) under streamflow and overland flow conditions, which was necessary in this case because no observation data for surface runoff from the slopes were available.

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GROUNDWATER PROBLEMS CAUSED BY EXCAVATION OF BUILDING BASEMENT FROM THE VIEWPOINT OF NATURE CONSERVATION

Okutsu, H.

ABSTRACT : Groundwater problems discussed here are as follows : 1) Influence of excavation of building basement (-9 to -17 m) recorded by automatic table register, especially bad influence on Zelkova trees planted along the road side as symbol tree of Sendai City. 2) Payment of damage for wells influenced by lowering of groundwater table originated by excavation. 3) Future groundwater problems and protection standing on nature conservation.

RESUME : Les problèmes discutés ici, concernant l'eau phréatique, sont les suivants: 1) Impact de l'excavation de fondement des bâtiments (-9 à -17 m) enregistré par "enregistreur automatique de niveau" et en particulier impact néfaste sur les Transcaucasias (Zelkova) plantées le long des avenues comme symbole de la ville de Sendai. 2) Dédommagement pour dégâts causés aux puits par abaissement du niveau d'eau phréatique. 3) Problèmes de l'avenir concernant l'eau phréatique, et mesures protectrices destinées à préserver la nature.

RESUMEN : Se discuten los siguientes problemas, relativos al agua subterránea : 1) Influencia nefasta de las excavaciones para cimientos de construcciones (-9 a -17 m), controladas por registrador automático de nivel, especialmente sobre las Transcaucasia (Zelkova) plantadas a lo largo de las avenidas, como árbol símbolo de la ciudad de Sendai. 2) Indemnizaciones por los daños ocasionados a los pozos, por descenso del nivel del agua subterránea a consecuencia de las excavaciones. 3) Problemas futuros, referentes al agua subterránea, y medidas destinadas a la protección de la naturaleza.

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Introduction

The topic in this paper deals with the groundwater problems associated with the excavation in the course of advanced urban development.

Urban groundwater is influenced not only by the pumping during the excavation work, but also by the pumping from the seepage water pit even after the completion of the underground structure. Representative examples for such problems are well drought and land subsidence in the vicinity of the construction site.

Among such problems, what is often neglected is the effect on the trees planted along boulevards in the process of heavy urban development. The author would like to introduce this problem based upon the results recorded by automatic water table register. Furthermore, it is also discussed here as to what kind of counter-measures be taken in order to prevent such adverse effect due to lowering of the groundwater table.

Cause of the problem and some background

Automatic water table register which will be frequently mentioned here was placed in the center of a busy downtown area of Sendai which is the largest city in the Tohoku Region with population of 620 000 (Figs. 1 and 2). This central area of the city has the concentration of buildings with more than 10 stories above and 2 - 3 stories below the ground level.

Although Sendai City had completely lost its long renowned beauty and changed from what was called a tree-clad town after the destruction during the war, thanks to the enthusiasm of the citizens of Sendai, grand view of Zelkova trees along the boulevards was reconstructed in downtown area where buildings are heavily concentrated making green columns of giant trees of 50 years of age.

Since Zelkova tree is named Tree of the City, citizens have strong affection toward these trees and it is one of the main background reasons for these people to seriously tackle with the problem of groundwater. Many of them have begun concerned about the fact that, along with the modernization of the city, ground surface is covered more and more by concrete or asphalt, almost totally cutting the percolation of rain water off, making groundwater necessary for Zelkova more scarce.

On the other hand, there has been much frequent construction of underground structure and galleries for the municipal water supply

works and for the installation and control of telecommunication cables. Not only during the construction, but even after its completion, pumping is constantly conducted from the seepage water pit in order to maintain the underground structure.

Besides these adverse condition of modernization, it has become of recent trend that there are some years with precipitation of less than 1 000 mm (annual average of 1 245 mm). Furthermore, between April and July when plants should grow most vigorously, there are sometimes nearly two months consecutively with monthly precipitation of less than 30 mm (Figs. 6 and 7). If groundwater is pumped during this period with poor seasonal distribution, it is quite clear that normal growth of Zelkova tree becomes inhibited. Especially, in the future when trees eventually reach near the age of 100 years with a meter in diameter and 20 m in height, the amount of evapo-transpiration draft will considerably increase.

In view of these facts, it is evident that the groundwater balance for Zelkova would only turn out to be negative. If not corrected, it seems that it would become no longer a hypothesis to have a situation where the green would eventually disappear and the area becoming complete desert region.

Plant conservation department of Sendai municipal office, carefully paying attention to such future probability, decided to conduct an artificial groundwater recharge for the purpose of conservation, when a giant Zelkova tree of 150 years of age was left (1.5 m in diameter and 18 m in height) in the green belt of National Road Route 4 after recent extension works. Based upon such future perspective, the plant conservation department undertook the measures in order to improve the infiltration of groundwater in downtown area which will be discussed more in detail later in this paper.

Plant called Zelkova tree

Zelkova tree (Zelkova serrata Makino = Planera japonica Miq., Ulmus Keaki Sieb.) is a tall tree (broad leaved deciduous arborescence) which belongs to the genus of Ulmaceae and widely distributed in the main island of Japan.

Since the bright green of its sprout in early spring and sober yellow of its colored leaves in autumn are in good harmony with the aesthetic taste of Japanese people, this tree is the subject of popular affection and is found in the gardens and estates of Japanese people. Lumber of huge Zelkova trees of nature growth has sometimes very finely figured grains of annual rings. An expensive chest of drawers is made of such trees and varnished with Japanese

lacquer (Urushi). Antique furniture lovers treasure it.

Since Zelkova is a tall tree which favors furtile land and running groundwater, it is planted to symbolyze water god by the well in ar old estate of traditional families, serving as a simple and naive indicator of the presence of groundwater. Therefore, where an old well is, one usually finds a giant Zelkova tree nearby standing loftily with its height of more than 20 m and diameter of 1 m, and a small shrine for water god at its foot. In some places, Japanese cedars (Cryptomeria japonica D. Don) are planted in stead of Zelkova, which likewise favor running groundwater.

Environment indicated by fossil Zelkova

Fig. 3 shows three leaves of Zelkova, amongst which "A" indicates a leaf from a Zelkova tree growing at a site where automatic table register is placed. This particular one was taken from leaves defoliated at the end of August which is about two months ahead of normal defoliation.

This abnormal defoliated leaf is an evidence of weakened growth of the tree, and it may be attributable to the water supply deficiency. As seen in the figure, there are blackened parts and dots which can be interpreted as signs of desease due to weakened tree growth and air pollution.

"B" is taken from leaves defoliated in a normal defoliation period in November which means that they are the ones from better environment with sufficient water. Leaf surface shows clear sign of autumnal tints.

"C" is a fossil leaf of Zelkova taken from Miocene Nenoshiroishi plant bed near Sendai area (Okutsu, 1955). When this fossil leaf is compared with recent leaves of "A" and "B", they are almost the same in terms of the characteristics of serration and the number of secondary, suggesting that they have survived through 10 million years without changing the leaf shape at all.

On the other hand, in view of the environment of sedimentation, it is reserved in the varved tuffaceous shale which shows lake deposit. Therefore it can be considered that Zelkova trees from which these fossil leaves were taken grew on a slope along a valley with abundant groundwater near large lakes.

Thus, Zelkova trees have come through 10 million years maintaining the nature of favoring furtile area with abundant groundwater. This very fact is considered to be the source of its strong viability.

Site of automatic water table register and its geological condition

As above described, an automatic water table register has been installed at the site as shown in Figs. 1 and 2 to find out the cause of the weakening of Zelkova trees.

Since this site is close to the Sendai railway station, around where many modern buildings are being constructed, it is not favorable surroundings for Zelkova trees to grow, although the need of conservation is strongly felt.

Since 1970, the author has had the opinion that the cause of the weakening of Zelkova trees must depend on the water deficiency which comes from the lowering of groundwater table. To prove this assumption, the writer has emphasized the need of installation of the register.

Fortunately, in 1972, Sendai South Rotary Club adopted this idea into their social welfare project, and the register was installed at the site shown in Figs. 1 and 2. This register recorded the abnormal lowering of groundwater table by the excavation of building basement from 1972 to 1974. And it is still working to record the fluctuation of groundwater table.

The flat plane, where the above referred business center is located, correlates the middle terrace plane, and the subsurface geology of the area consists of the following 5 layers (Figs. 4 and 5).

Age:	Order of soil layers:	Thickness:
Alluvium	(1) Surface soil (organic clay)	0.5
	(2) Sandy loam	1.5
Diluvium	(3) River terrace gravel	4.8
Tertiary (Pliocene)	(4) Weathered bed (clay)	1.4
	(5) Bed rock (tuffaceous sandstone)	10.0

(1) and (2) are distributed within 2 m under the surface, which form the foundation bed of one or two story wooden houses. Surface soil (1) composed of organic clay or peat (Okutsu, 1972), sometimes makes soft ground of 1 - 2 m thickness. When a wooden house which is based directly on such soft ground often leans owing to the land subsidence caused by the excavation of building basement or pumping up of groundwater for air conditioning. This fact often leads to the law dispute of indemnity.

(2) is the root swelling zone of laterals or rootlets of tall trees such as Zelkova (Fig. 4).

(3) is the river terrace deposits left by the Hirose River running through Sendai area. The deposits are characterized by terrace gravel (Okutsu, 1967). The upper part of these deposits are mainly composed of loam with gravel, lower part consists of sand and gravel, and forms the storage of shallow groundwater (un-confined), which is used as aquifer for dug wells. The basements of three - five storied apartment houses are based on this gravel bed. The rootlets and hair roots of Zelkova trees reach the upper part of terrace gravel. So, the root swelling zone is deeply fastened into loam with gravel to get supply of fringing water.

This root extension shows the mechanism of resist the drought (Fig. 4). But when abnormal lowering continues for several months during vigorous growth season, it is clear that the growth of Zelkova trees faces the crisis.

(4) is the clay derived from weathering of bed rock (tuffaceous sandstone), which distributes on the contact plane between terrace gravel and bedrock. This clay has usually 0.5 - 1 m thickness, and shows weak bearing power (S.P.T., N value = 10). Accordingly, where clay is thicker than 1.5 m, it is necessary to change the design of excavation depth of building basement.

(5) is the bedrock of this area. Geological age is Pliocene (Neogene Tertiary), and mainly consists of tuffaceous sandstone ∿ tuffaceous mudstone (Tatsunokuchi formation of Sendai area). As this formation is characterized by fossil shells and fossil plants such as Zelkova tree, at the time of excavation, a tremendous amount of fossil shells are being found. Sometimes shark teeth and fossil skeletons of whales or dorphines are also discovered. Under the Tatsunokuchi formation, Kameoka formation is laid which is characterized by lignite seam.

As the coarse sandstone of both formations forms good aquifer of deep groundwater (confined groundwater), a lot of deep wells (depth 150 - 250 m) penetrate to both formations (Okutsu, 1968, 1971), and these wells pump up 500 - 700 cubic meter of water per day (Fig. 5). The natural level in this area is less than -60 m under the surface, and pumping level less than -120 m, because of the high density of deep wells. There are some examples of submersible pump set in -200 m.

The lowering of the groundwater level as above cited decreases the confined goundwater reservoir. Furthermore, it squeezes the shallow groundwater from the un-confined groundwater reservoir in the gravel bed. The reader will understand how serious influence the underground works give to the groundwater situation.

The land subsidence (Okutsu, 1972) has already occurred owing to the excess withdrawal of groundwater in the Sendai coastal industrial area. Therefore Sendai municipal office has inacted groundwater control law for industrial use.

In addition to industrial supplies the utilization of groundwater for air conditioning in urban area on the terrace gravel has increased. So Sendai municipal office is going to conduct the analytical survey of groundwater to prevent the land subsidence.

The influence of excavation of building basement recorded by automatic water table register

In July 1972, automatic water table register was installed in the site shown in Fig. 2. The groundwater table at the time was about -4 m. In June 1973, the excavation for construction of the building with 9 stories above and 3 stories below the ground level started ("Y" building in Fig. 2). Since the register was set at 15 m from the building it clearly recorded the fluctuation of groundwater table (Fig. 6).

By this graph the drawdown of groundwater table was recognized at each stage: the excavation of gravel bed (-7 m), and of bedrock (-17 m). Comparing the stages before and after excavation, it is noticed that the amount of maximum drawdown was 1.3 m at the deepest excavation. The drawdown of 1 m kept continuing until the backfilling started. When the constructor started backfilling, groundwater table gradually rose up and it finally recovered the original level at the completion of backfilling.

Ten months after the completion of "Y" building, excavation of another building basement ("S" building in Fig. 2) began at the opposite side of the register location. The shortest distance from the register was 70 m. The register recorded the fluctuation of the same tendency (Fig. 7).

Underground environmental change which accompanies the urban development

Comparing the two graphs of the register which recorded the underground excavations of the two buildings it is clear that the influence of underground development is becoming serious. In the case of excavation of "Y" building, the groundwater of 400 - 500 litre per minute sprang out from gravel bed and bedrock. However, in case of "S" building, built 10 months after the other, in spite of the same hydrogeological condition the groundwater scarcely sprang out. This may be because of the difficulty of infiltration of rain water through the pavement surrounding "Y" building area,

and also by pumping up of the seepage water pit constructed under "Y" building basement. The groundwater amount pumped up from the seepage water pit is about 2 cu. m per day (gross volume of "Y" building besement 68 250 cu. m).

The influence to shallow dug wells in the vicinity of excavation site

In the overcrowded area where the register is located, there are no old style shallow dug wells because this area has changed to business center. However, in other less crowded areas there remain a number of shallow dug wells which are still on use. Excavation of building basements surrounding these wells are also giving unfavorable influence on them.

Figs. 9 and 10 show the example of the influences. The depth of excavation of this example was -9 m from the surface, interference radius reached 70 m, and caused trouble to the inhabitants and the well drought (well depth 4 - 7 m).

Since such trouble occurs frequently, the necessity of prospecting of interference radius by the reasonable boring survey and simulation on electronic computer is strongly felt. Several examples show the interference radius is about 70 m by simulation. The data during excavation showed almost the same result. Shallow dug wells (depth 4 - 7 m) within radius 70 m - 100 m were affected by pumping of excavation. The dispute for compensation of this case was settled by laying the city water pipe.

Countermeasure for the protection

The abnormal going down of groundwater table which reaches radius 70 - 100 m gave bad influence to Zelkova trees. In order to prevent such bad influence and to increase infiltration capacity of rain water, following installations were designed:

(a) Plant conservation department of Sendai municipal office buried infiltration pipe (diameter 150 mm, length 2 m, depth 50 cm) connecting catch basin of drainage system and infiltration basin which was newly designed (Fig. 11). Accordingly rain water flows into the catch basin and infiltrates freely into sandy loam of root swelling zone. The distribution of these basins is shown in Fig. 13. It is designed to supply water to every Zelkova tree planted in a row.

(b) Another countermeasure: Installation of the infiltration pits on the paved sidewalk where the row of Zelkova trees stands (Fig. 12). These pits are most effective to increase infiltration

capacity compared with the ordinary paving plate which covers the ground completely. As shown in Fig. 13, porous concrete pipe (diameter 150 mm , vertical length 600 mm) is buried which easily infiltrates rain water into the sandy loam after filtration by filled gravel. Besides, paving plates surrounding the infiltration pits were replaced by porous concrete plate for the rain water to infiltrate into surface soil. Porous concrete plate, however, has the following weak points:

a) To decrease its porous capacity by filling of sand, mud and dust.

b) Its weak registivity to heavy load.

These artificial recharge methods were just completed last August (1977). Remarkable effect is being expected for the sake of the trees. As a matter of fact, the groundwater table has a tendency to rise gradually as proved by the record of the register (Fig. 8). Today it is urgently needed to prevent the lowering of groundwater table by artificial recharge. From the nature conservation point of view recharging system above described is not only useful for Zelkova trees but also effective for the rising of shallow groundwater table.

On this point of view, Zelkova tree may be called an indicator of groundwater conservation. On the other hand, at the excavation, it is necessary to find better methods to prevent lowering of groundwater. For example: (a) cement covering and grouting to prevent the gushing out of groundwater from gravel bed. (b) cut shallow aquifer by sheet pile, and recharging pump-upped groundwater around the excavation site observing the quality of water. (c) practice of underwater excavation, etc.

Conclusion

The protection of trees on boulevard is becoming important accompanied by highly developing density of the city. One of the reasons is the lowering of groundwater table during the excavation of building basement (Figs. 6 and 7). Irrigation to the root is somewhat useful, but construction of infiltration pipe and infiltration basin is more effective (Fig. 8).

Acknowledgements

The writer would like to express thanks to the staff of plant conservation department of Sendai municipal office. Last but not least would the writer like to acknowledge Mr. Shigeki Suganami, graduate student of Tohoku University, for filing references and recording data.

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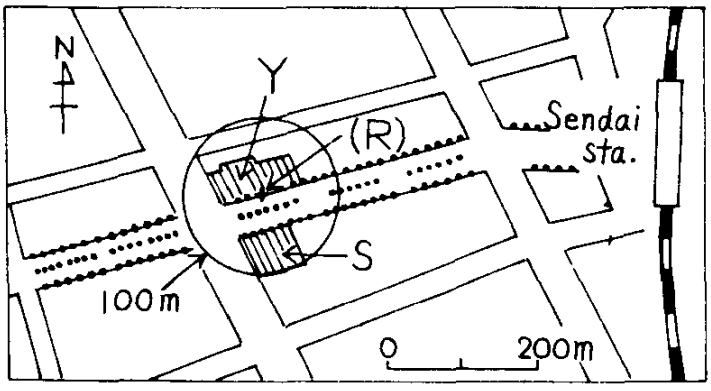


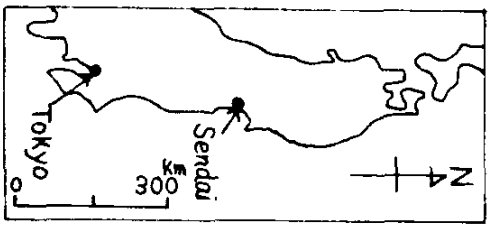


Fig. 1 shows: Excavation sites ("Y" and "S"), boulevard of Zelkova trees (dotted line), and site of automatic water table register (R).

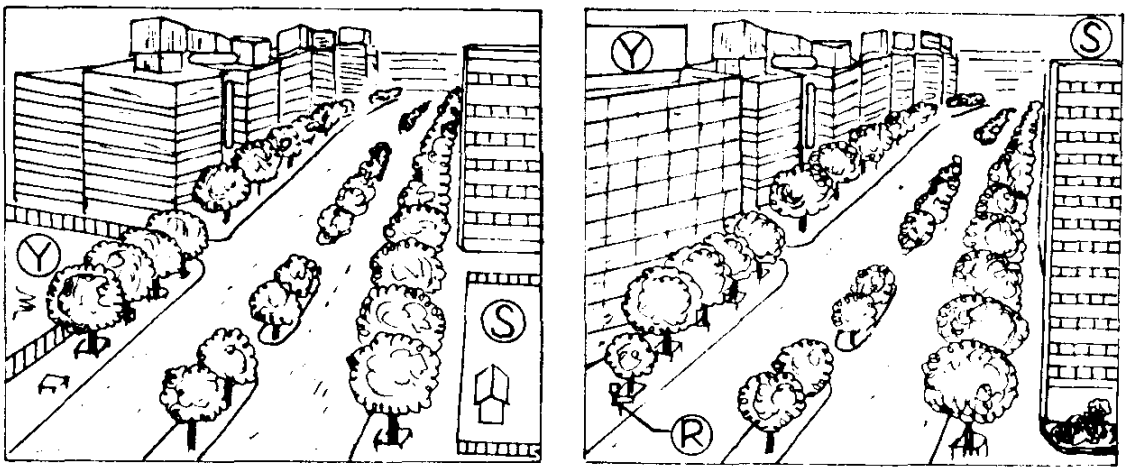
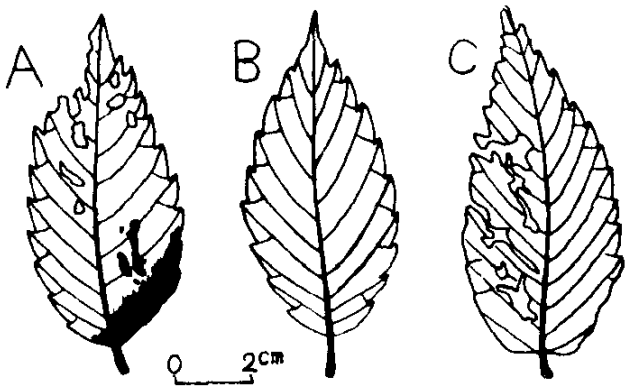


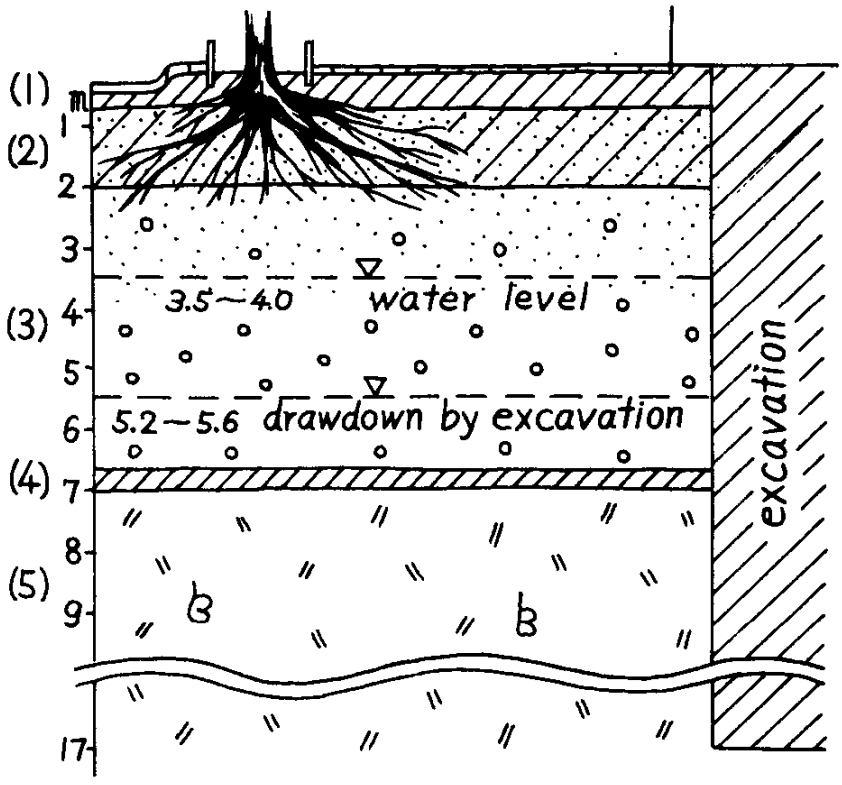
Fig. 2. Change of environments by urban renewal.

Left = 1972
Right = 1977



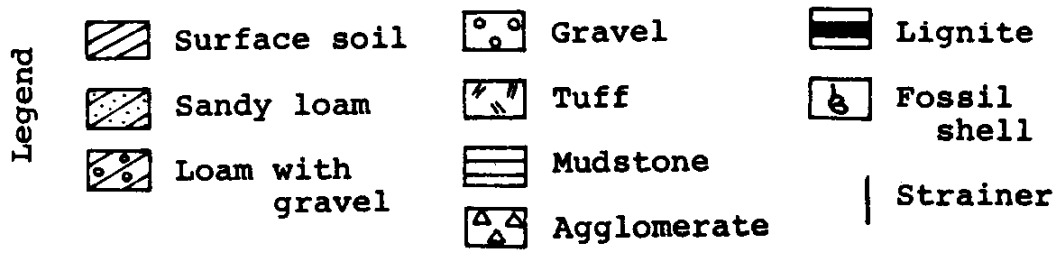
A = Abnormal defoliated leaf.
B = Normal defoliated leaf.
C = Fossil leaf.

Fig. 3. Recent and fossil leaves of Zelkova trees.



(1)(2) = Surface soil and sandy loam.
(3) = Terrace gravel.
(4) = Weathered zone.
(5) = Bed rock (tuffaceous sandstone)

Fig. 4. Ideal diagramic section of root swelling zone and change of groundwater table by excavation by building basement ("Y" building).



A = Non-confined groundwater.
B = Confined groundwater.
(1) - (5) = The same as Fig. 4.

Fig. 5. Columnar section of hydrogeology in the environs of the register.

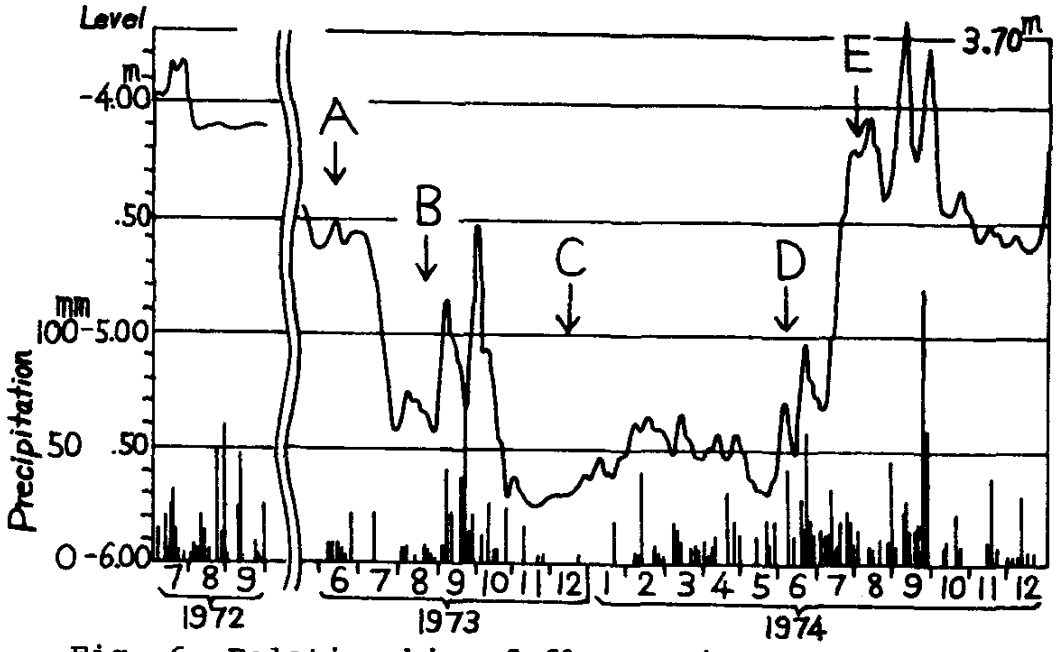


Fig. 6. Relationship of fluctuation of groundwater table and excavation of building basement ("Y" bldg).

A = Starting of excavation of gravel.
B = Starting of excavation of bedrock.
C = Completion of excavation of bedrock.
D = Starting of backfilling.
E = Completion of backfilling.

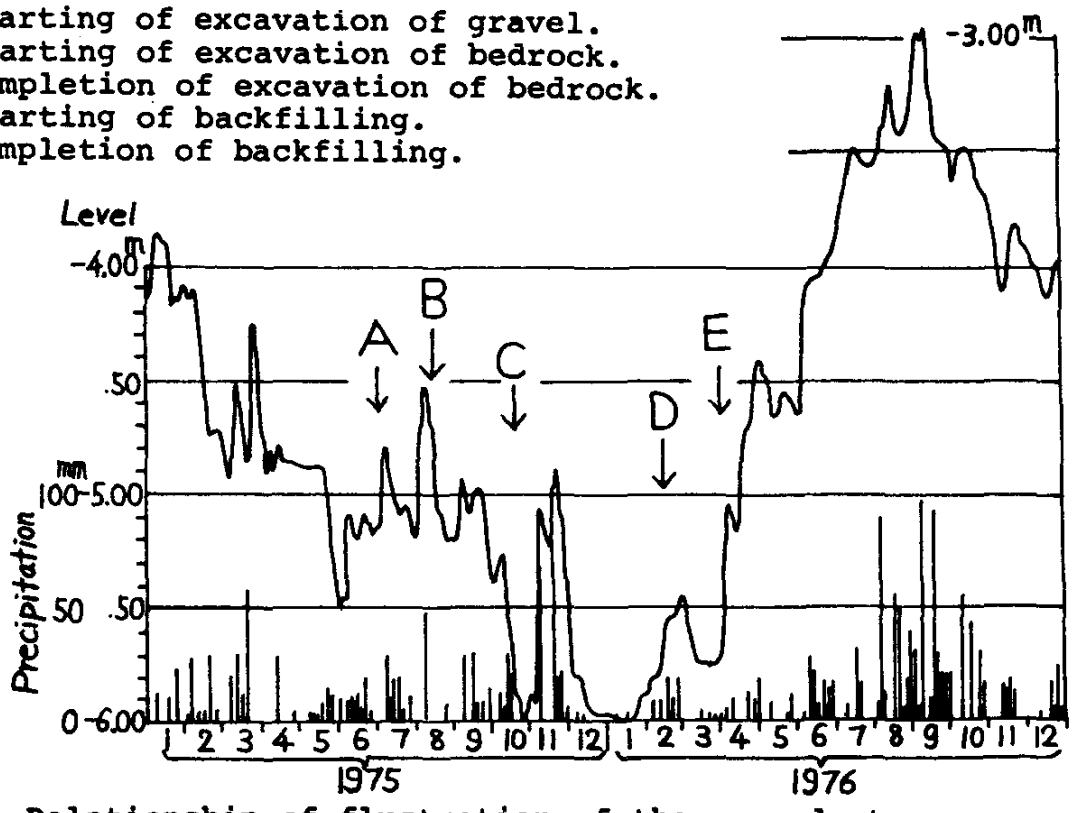
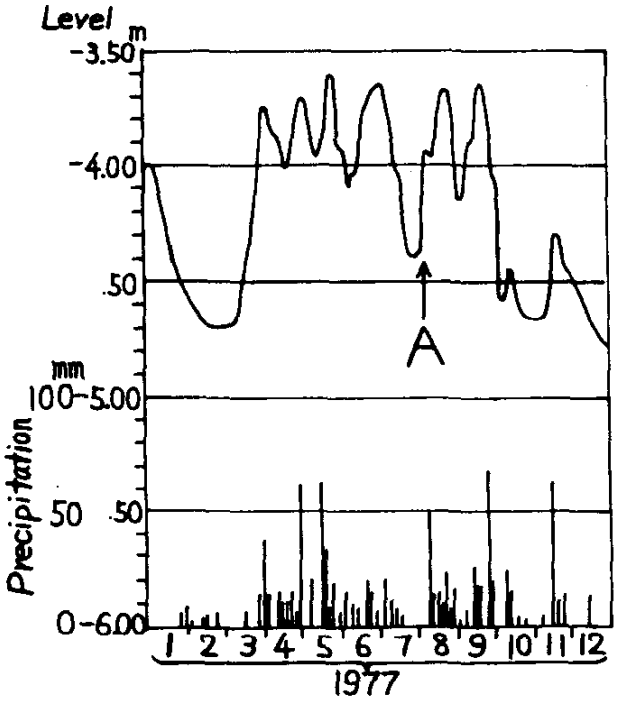
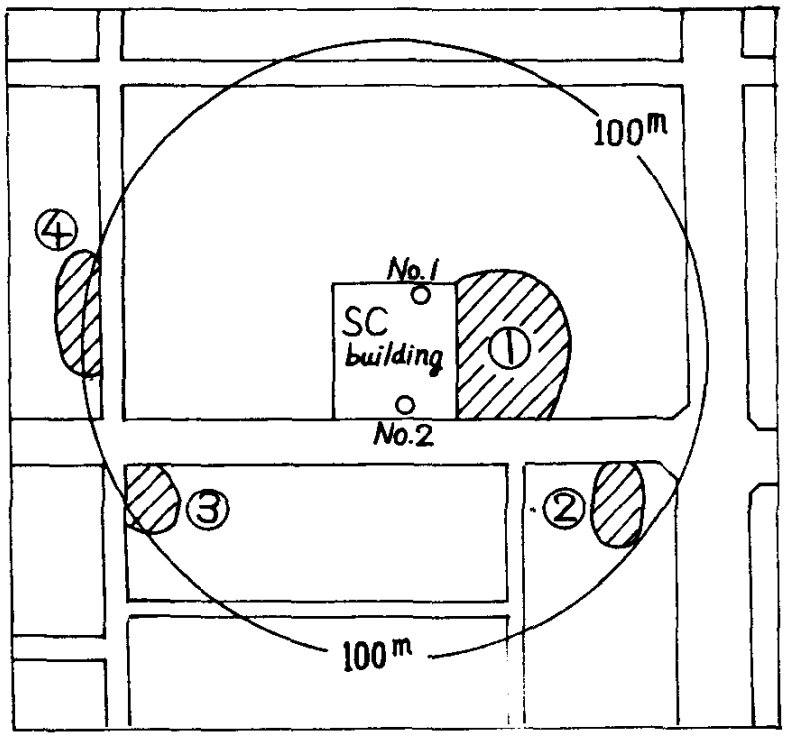


Fig. 7. Relationship of fluctuation of the groundwater table and excavation of building basement ("S" bldg).



A = Start of recharge (August 1977)

Fig. 8. The fluctuation of groundwater table during no excavation after recharge of rain water by infiltration pipe and basin.



SC = Excavation site of SC building basement. Nos. 1 & 2 = Boring point.
(1) - (4) = Order of influence.

Fig. 9. Distribution of droughty well by influence of excavation of building basement.

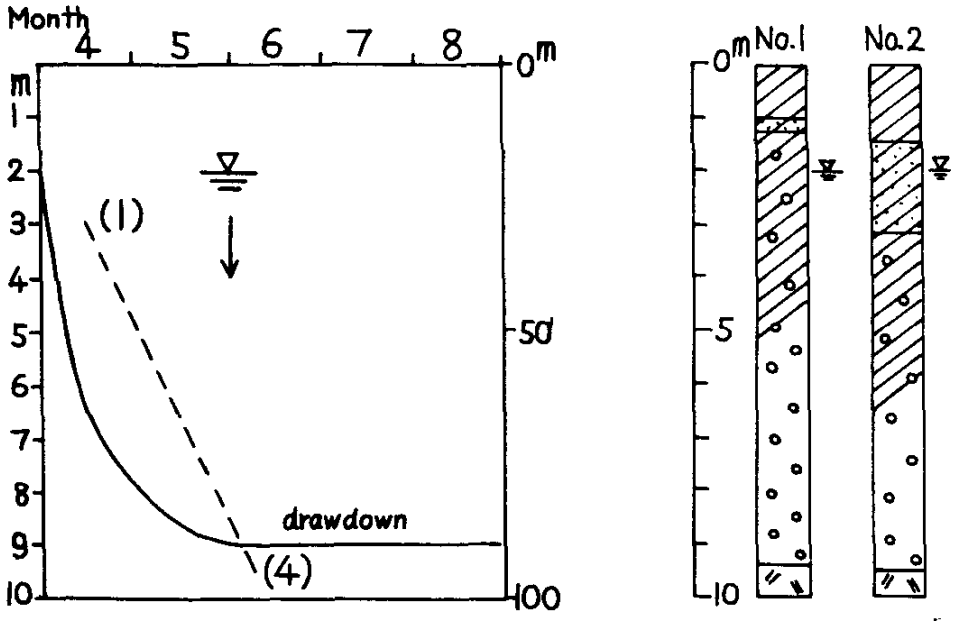


Fig. 10. Subsurface geology and relationship of progress of excavation and drawdown of groundwater table of well (symbol of soil and rock same as Fig. 5). (1)-(4)= Order of influence (Fig. 9).

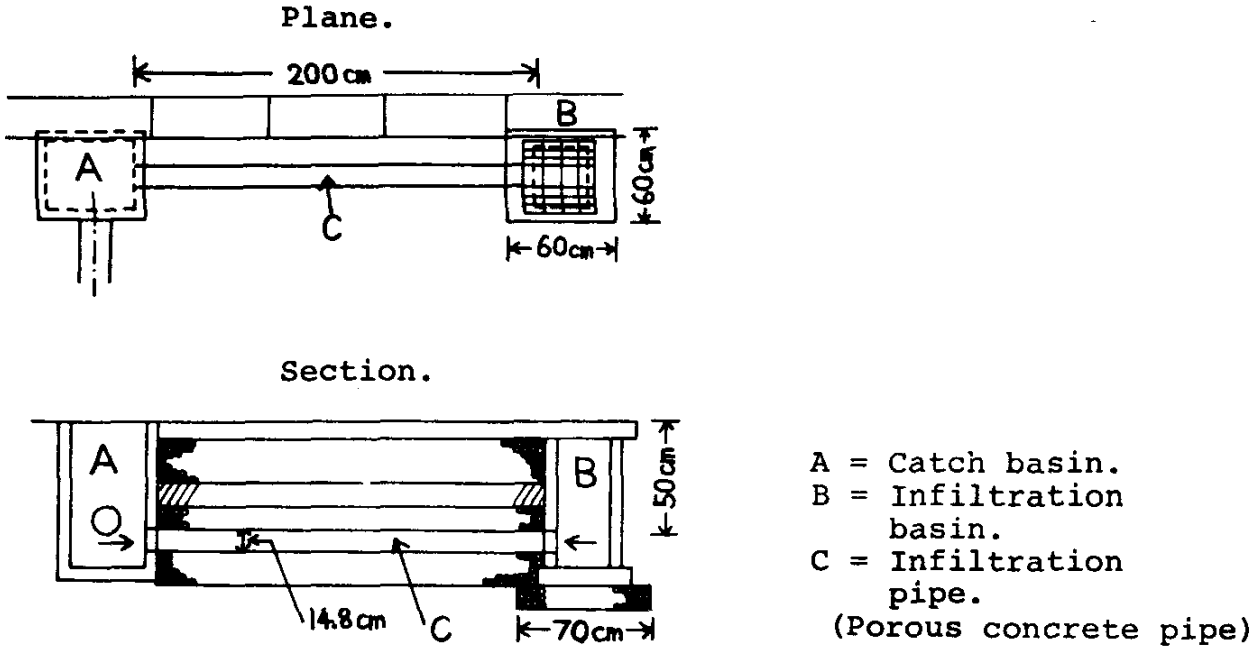
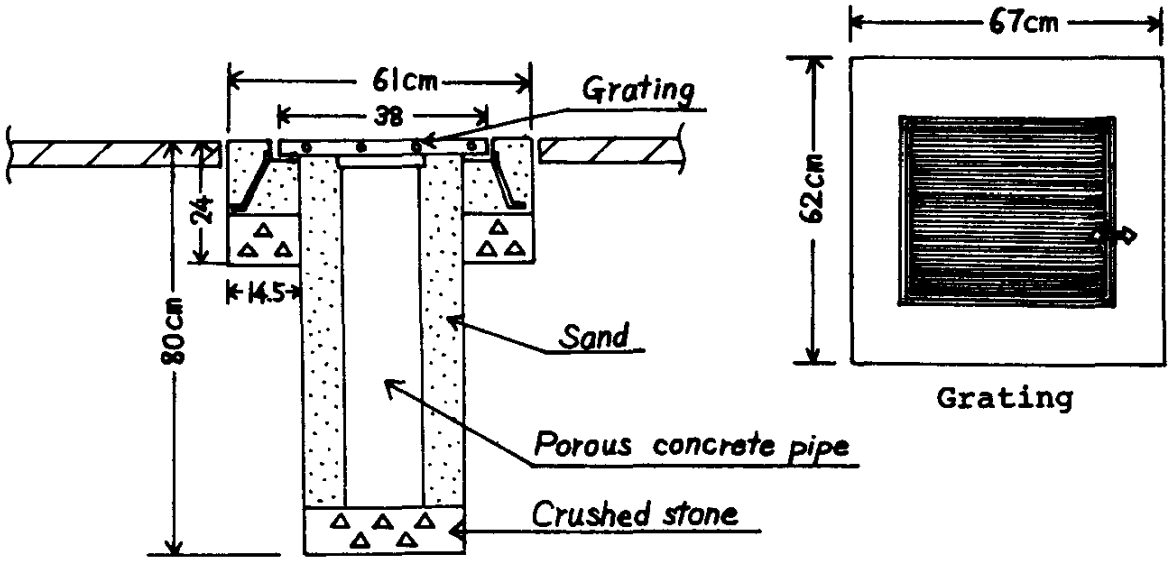
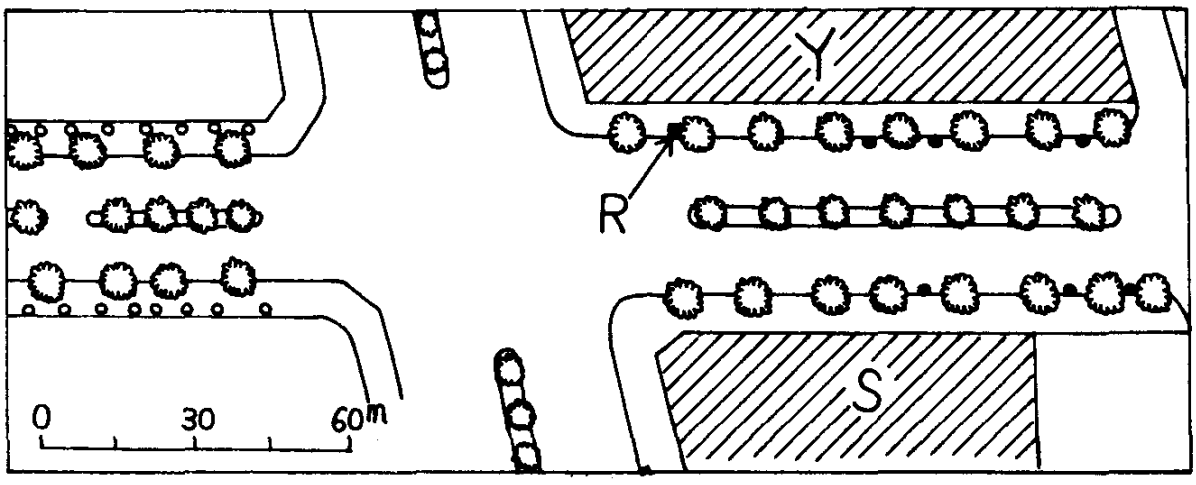


Fig. 11. Structure of infiltration pipe for Zelkova trees.



Left = Section.
Right = Plane.

Fig. 12. Structure of infiltration basin for Zelkova trees.



● Infiltration pipe.
o Infiltration basin.
O Zelkova trees.
R Register.

Fig. 13. Distribution of infiltration pipe and infiltration basin for Zelkova trees (Plain).

WATER CAPACITY OF ABANDONED WORKINGS IN UNDERGROUND COAL MINES

Rogoz, M.

ABSTRACT : The quantity of water being held in old workings may be calculated by multiplying the volume of mined out coal bed by the water capacity coefficient. On the basis of investigations carried out in mines and in laboratories equations were established, that enabled to determine the mean values of water capacity coefficient depending on the mining method used, depth at which old workings are situated, or rock pressure and on the quality of material used for stowing the old workings.

RESUME : La quantité d'eau contenue dans les vieux travaux miniers peut être calculée comme le volume de la couche déhouillée multiplié par le coefficient de la capacité en eau dans les vieux travaux. A la base des recherches effectuées in situ et en laboratoire, on présente les équations permettant de déterminer les valeurs moyennes du coefficient de la capacité en eau dans les vieux travaux en fonction de la méthode d'exploitation, profondeur de la couche ou pression des terrains et qualité du matériau de remblayage.

RESUMEN : La cantidad de agua almacenada en las antiguas labores mineras, se puede calcular multiplicando el volumen de la capa de carbón por un coeficiente de capacidad de agua en los trabajos. Basándose en las investigaciones ejecutadas in situ y en laboratorio, se han establecido ecuaciones, que permiten determinar valores medios del coeficiente de capacidad de agua en los trabajos, en función del sistema de explotación empleado, profundidad a la que se localizan los viejos trabajos, presión de la roca y calidad del relleno.

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1. Introduction

In underground coal mines there is a lot of water reservoirs in parts of old workings having no gravitational outlets, whose capacity reach hundreds of thousends of cubic meters. They represent the serious hazard of mining work carried out in their proximity, because, in some circumstances, they cause the risk of water inrush into active workings. The only reliable way to eliminate this water hazard is the dewatering of old workings by letting off the water by gravity or by pumping it out.

In order to designe and to carry into effect dewatering of old workings, it is necessary to know, besides the general mining conditions, the quantity of water being held in the reservoir. This quantity is equal to the water capacity of flooded workings, it means to the total volume of all voids in workings that may be or are filled with free water. The water capacity of workings is always lesser than the volume of coal bed mined out in consequence of their convergence under the weight of the subsiding rock massive. The relation of water capacity of old workings V_z to the volume of extracted coal bed V_p is called water capacity coefficient c

$$c = \frac{V_z}{V_p} \qquad /1/$$

The water capacity of old workings may be then expressed as

$$V_z = c\, V_p = c\, \frac{A\, M}{\cos \alpha} \qquad /2/$$

where: A - area of the flooded workings in horizontal projection,
M - the thickness of extracted coal bed,
α - inclination angle of coal bed.

The variables appearing in the formula /2/, except the water capacity coefficient, may be read from the mine map or be calculated with known methods, therefore they do not require any closer discussion. The problem is to establish the water capacity coefficient c, that can not be determined from the mine map. Its value depends on several factors contributing to the local geological and mining conditions, from which the most important seem to be:

- method of filling old workings,
- degree of convergence of old workings.

The method of filling old workings depends on mining method and roof control used, whereas the degree of their convergence depends on the depth at which old workings are situated and on the time. There is no doubt that the process of roof breaking as well as the increase in rock pressure in workings depend, less or more, on geological conditions and particularly on the lithology of roof rocks and inclination angle of the coal bed. In this paper the approximated values of water capacity coefficient of old workings are determined depending on the mining method and roof control used and on the depth of the coal bed or on rock pressure and the quality of stowing material when the back-filling is used. Too small a number of direct measurements of workings capacity during their dewatering in mines did not permit to take into consideration the influence of local geological conditions. In order to eliminate the time factor, only the water reservoirs in old workings were taken into account, where the process of roof fall or the roaf subsidence is practically finished.

2. Geological conditions of investigated workings

The investigations of water capacity of old workings were carried out in coal mines of the Upper Silesian Coalfield. The investigations covered the workings in coal beds belonging to Namurian B, C and Westphalian A, B, C, D.

The Namurian B and C deposits are developed in alternating layers of sandstones and mudstones with coal seams. The mudstones are gray, firm, less or more sandy. They contain sphaerosiderites and coalified plant debris. Sandstones form thick layers. In general, they are compact, medium-grained and coarse-grained, in some places conglomerates can be met. The part of sandstones in the vertical profile is of about 40 %.

The Westphalian A and B deposits are developed similary as those of Namurian B and C. The part of sandstones being here smaller does not exceed 25 %. The thickness of particular layers is highly variable ranging from some tens of centimeters to about 20 meters. The mudstones are similar to those of Namurian B and C. The mudstones accompanying the coal beds have sometimes ability to plastyfying in contact with water. In such cases floor heaving can be observed. The sandstones have compressive strength ranging from 20 to 30 MN/m^2. Their porosity amounts to about 15 % and Darcy's coefficient to about 10^{-8} m/s, on the average.

Westphalian C and D are developed as very thick series of sandstones, chiefly arkosic, with poor clay binder. They have high porosity, exceeding often 20 % and low compressive strength, in general lower than 10 MN/m^2. The Darcy's coefficient amounts to about 5.10^{-8} m/s, on the average. The percentage of sandstones in the profile of Westphalian C and D exceeds 80 % on

the average. Particular series of sandstones are interbedded by mudstones usually accompanying the coal beds. Mudstones are similar as in Westphalian A and B.

3. Water capacity of caved gobs

In order to determine the values of water capacity coefficient, investigations of 37 water reservoirs in caved gobs were carried out in 25 coal mines. The investigations consisted in possibly exact measurement of water quantity drained from old workings during their dewatering with simultaneous control of the lowering of water table in the workings, quantity of water affluing into the workings and leakage from the waste to other workings. The purpose of the investigations was to determine the water quantity contained in a precisely delimited part of old workings and to compare it with the precisely calculated primary volume of this part of the extracted coal bed.

During the investigations some technical difficulties were encountered along with those associated with interpretation of results. The chief technical difficulty was the measurement of water quantity drained out from old workings through the drain bore-holes. In many cases, some uncontrolled quantities of water inflowing to the old workings were obstacles in the proper realisation of measurement. Operational reasons frequently required stopping the dewatering for a certain time or decreasing the discharge of draining bore-holes. Such disturbances of dewatering old workings rendered the interpretation of measurements more difficult and, sometimes, even impossible. In consequence, in many cases, the results obtained were incomplete or not fully reliable. After a detailed analysis, 12 water reservoirs from the total number of 37 covered by the investigation were selected. Their water capacity was measured with a satisfactory accuracy.

The difficulty of interpretation of the results of measurements encountered in all investigations which reduced the accuracy of obtained results, consisted in the unknown water capacity of roadways situated among inundated gobs and connected with them. Usually, neither the dimensions of roadways, nor the degree of their caving or self-stowing were known. In general, it was assumed that the water capacity coefficient determined for caved gobs could not be extended for roadways. In particular mines the water capacity of roadways was estimated by assuming that the dimensions mostly encountered in a given mine and the degree of squeezing the roadways depend on the behaviour of roadways in a given coal bed of the mine. Using these estimations and the data read from the mine map, the water capacity of roadways in the dewatered part of the reservoir was calculated and subtracted from the volume of water drained from the workings. It was assumed

that the remaining quantity of water derives exclusively from the production workings and that its relation to the primary volume of these workings is the searched water capacity coefficient.

The capacity of galleries usually low with respect to that of whole dewatered reservoir, brought about that the error of determination of the water capacity coefficient, resulting from unexact estimation of roadway capacities, was very small and, in practice, it did not exceed $\pm$ 2 %.

In the case of getting coal by caved longwall face system the values of water capacity coefficient show a good correlation with the depth at which old workings were situated. Such a correlation was not found in the case of caved shortwall workings. This may be explained, on one hand, by a small number of dewatered reservoirs /4 reservoirs/ and on the other hand by a moderate depth at which they were situated not exceed 200 m. Thus, the water capacity coefficient of caved shortwall workings was calculated as an arithmetic mean of values obtained from measurements

$$\bar{c} = 0{,}438$$

The standard deviation of the mean value amounts to $\pm$ 0,0128, which is an accuracy for mining purposes.

For caved longwall face workings the results of measurements were used to establish the regression equation of the water capacity coefficient c in relation to the depth H at which the old workings were situated. This equation has the form:

$$c = 0{,}485\ e^{-0{,}00205\ H} \qquad /3/$$

where e is the base of natural logarithms.

The graph of equation 3 is shown in fig. 1.

Although the number of experimental data of the water capacity coefficient for caved longwall face workings is small /8 reservoirs dewatered/, nevertheless the relation between the values of coefficient c and the depth of workings is quite distinct. It justifies the use of regression calculus.

The standard deviation of real values of the coefficient c from those estimated with the use of the equation /3/ amounts to $\pm$ 0,016, which seems to be an accuracy sufficient for mining practice.

4. Water capacity of sand filled gobs

The water capacity coefficient of sand filling c is equal to the specific yield μ of the sand compressed in workings under the effect of overlying rocks pressure, multiplied by the ratio of the sand layer thickness M to the primary thickness of the extracted coal bed M_o

$$c = \mu \frac{M}{M_o} \qquad /4/$$

Contrarily to the water capacity of caved workings, the water capacity coefficient of workings filled with sand stowing may be tested in laboratory. The tests consist in determination of specific yield of sands having various granular compositions after their compaction in the hydraulic press.

130 samples of sand used for hydraulic filling, taken from various pits situated on the territory of Upper Silesian Coalfield were submitted to tests. In order to establish the relation between the specific yield of sand and the degree of its compression in workings, each sample was placed in the special cylindric container and its specific yield was determined 5 times, namely: in loose state and after compaction of sand in the press. Pressures of 5,2; 10,4; 15,6 and 20,8 MN/m^2, were applied which corresponded to depths of 200, 400, 600 and 800 m respectively, with taking simplificated assumption that after a sufficiently long time the rock pressure in gobs is equal to the product of the depth and the specific weight of overburden, which amounts to 26 kN/m^3, on the average.

Besides the tests on the specific yield, every sample was submitted to size analysis and its characteristic parameters were determinated, i.e. the effective size diameter d_{10} and coefficient of uniformity

$$U = d_{60} : d_{10}$$

The tests allowed to establish for each sand sample the specific yield μ in loose state and after the compaction of the sample in the press at various loads. The measurements of thickness of sand layer in the cylinder before compression M_o and after every compression M, allowed to determine the water capacity coefficient c with the use of the formula /4/.

Consequently, for each sand sample were five values of coefficient c, corresponding to five values of pressure p caused by

the press load. In addition for each sample parameters of grain sizes d_{10} and U were determined. This allowed to set up 650 fours of numbers for c, d_{10}, U and p. Next, coefficients of the linear regression of c with respect to d_{10}, U and p were calculated in various combinations of variables and of their logarithms. These calculations aimed at finding such form of regression equation, at which the correlation coefficient could be the highest and the remaining variance the smallest. The equation finally obtained has the form:

$$c = 1{,}673\ d_{10}^{0{,}716}\ e^{-0{,}329\ U}\ e^{-0{,}0147\ p} \qquad /5/$$

where p is the rock pressure expressed in MN/m^2.

The coefficient of multiple correlation R = 0,682 and the standard deviation s = 0,0572 proves that the correlation is significant on a very high confidence level.

In the case of no data on the quality of sand used to back-filling, it is suggested to take average values of d_{10} and U for an approximate assessement of coefficient c. On the basis of tests on 130 samples of sand taken at random from 9 active and abandoned sand pits, the mean values of size composition parameters are as follows:

$$d_{10} = 0{,}21 \text{ mm}$$

$$U = 2{,}09$$

with standard deviations from the average ones being of 0,0517 and 0,5011 respectively. The values of coefficient c for sand with such parameters may be estimated by means of the following formula:

$$c = 0{,}275\ e^{-0{,}0147\ p} \qquad /6/$$

Its graph is shown in fig. 1.

Adoption of mean values of size composition parameters introduces an aditional error in assessment of the water capacity of old workings. Its mean value m_c may be determined by the formula:

$$m_c = \sqrt{\left(\frac{\partial c}{\partial d_{10}}\right)^2 s_d^2 + \left(\frac{\partial c}{\partial U}\right)^2 s_U^2} \qquad /7/$$

where s_d - standard deviation of d_{10}
s_U - standard deviation of U.

By substituting in the formula /7/ $s_d = 0,0517$ and $s_U = 0,5011$, we get after differentiation:

$$m_c = \pm 0,066\ e^{-0,0147\ p} \qquad /8/$$

Consequently, the relative error of the coefficient c resulting from adoption of the mean values d_{10} and U amounts to:

$$\frac{m_c}{c} = \pm 0,24$$

6. Water capacity of gobs supported by packing

The water capacity coefficient of gobs supported by packing may vary within large ranges depending on the quality of material used for packing and the way of its setting. It is convenient to calculate these values, starting from the porosity of bulk density of the material used and from its compressibility. With such a calculation the results obtained will be a little overestimated with respect to the real water capacity of gobs. In order to avoid this error, it would be necessary to adopt the thickness of coal bed M reduced by the average hight of the void left between the packing and the roof of the packed gob. This height can not be established generally. It may be determined in relation to definite gobs if the way of their packing is known.

The compressibility of packing material composed of coarse lumps of carboniferous rocks of the Upper Silesian Coalfield was investigated by Hurysz and Adamek /1/. They established curves of relations between the compressibility and rock pressure for packing with clayey shales /siltstones/, mudstones and sandstones. The curves compressibility of siltstones and mudstones are close to each other on the diagram, while the sandstones show a lower compressibility. Taking into account the prevalence of siltstones and mudstones over the sandstones in the material used for packing in mines and the similar compressibility of siltstones and mudstones the mean values of compressibility of these rocks were calculated and next the resultant curve of compressibility was approximated by the regression equation:

$$S = 0,19\ p^{0,2} \qquad /9/$$

where: S - compressibility of packing material,
p - overlying rock pressure, MN/m^2.

The water capacity coefficient of gobs supported by packing may be determined by the formula:

$$c = P - S \qquad /10/$$

where: P - the porosity of bulk density of packing material.

The tests on the porosity of bulk density of rocks were carried out on 278 samples of barren rock material used for packing taken from various coal mines. The tests allowed to determine the mean value P = 0,403 with standard deviation of $\pm$ 0,12.

Using the equation /9/, an approximate equation determining the relations between the water capacity coefficient of pack-supported gobs c and the overlying rocks pressure p was derived

$$c = 0,4 - 0,19\, p^{0,2} \qquad /11/$$

The graph of equation /11/ is shown in fig. 1.

6. Summing up of results

The water capacity of old workings is the product of primary volume of extracted coal bed and of the number lesser than unity, called water capacity coefficient c.

The values of water capacity coefficient depend on the quality of material filling the old workings and on the degree of its compression by the rock pressure. For caved gobs the empirical relation /3/ was established between the water capacity coefficient and the depth at which the coal bed was worked, on the basis of measurements of quantity of water drained off from dewatered reservoirs. For gobs filled with sand and supported by packing relations /5/, /6/ and /11/ between the water capacity coefficient and rock pressure were determined.

Table 1 shows the approximate values of water capacity coefficient under different conditions. The values of coefficient c for hydraulic filling refer to the stowing sand with an average size composition parameters, i.e. d_{10} = 0,21 mm and U = 2,09.

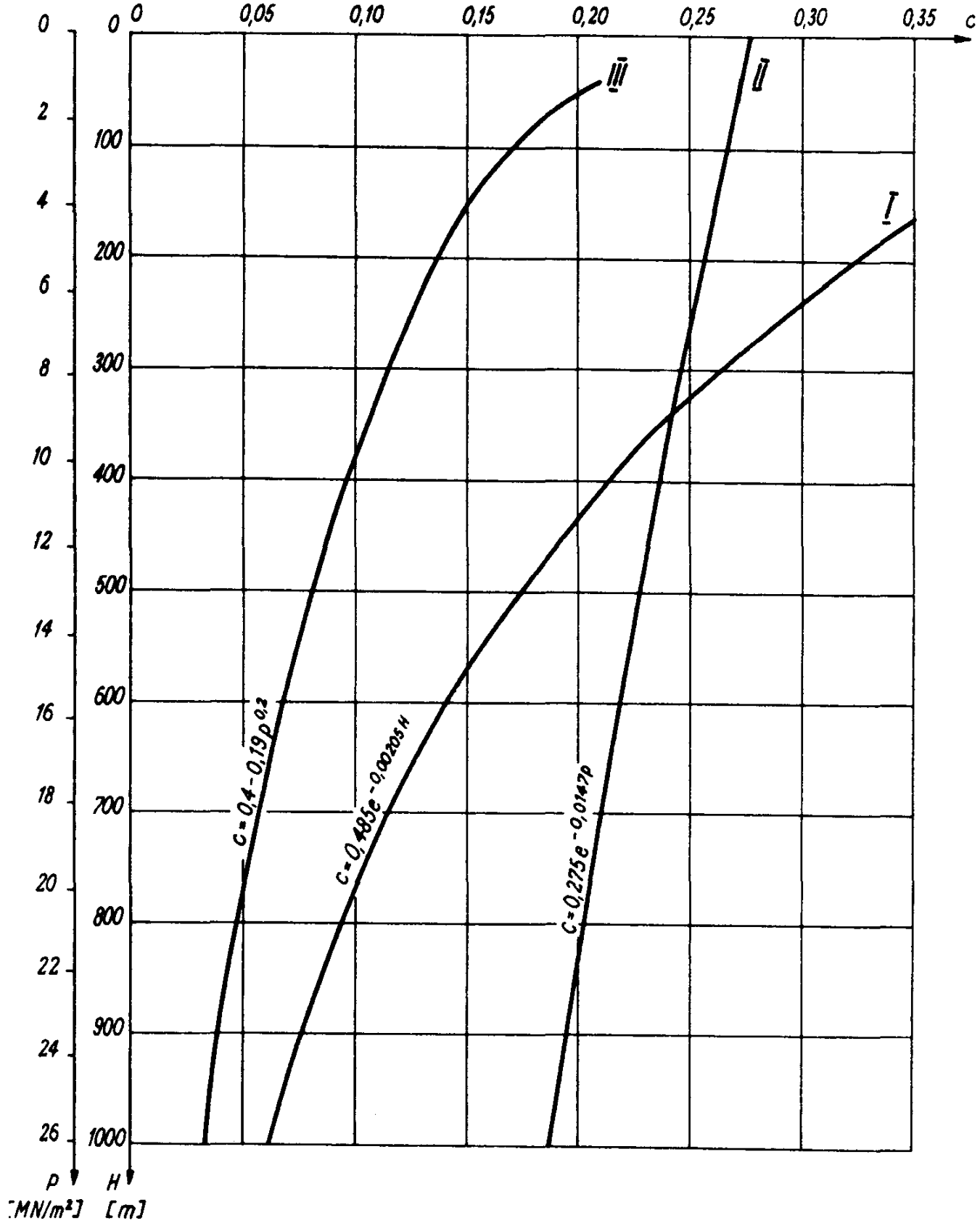

Fig. 1. Water capacity coefficient curves for caved gobs (I), hydraulic-filling (II) and packing (III)

Table 1

Mean values of water capacity coefficient c

Caving			Filling		
Depth of workings m	Shortwall system	Longwall system	Rock pressure MN/m^2	Hydraulic filling	Packing
200	0,438	0,322	5,2	0,255	0,137
400	-	0,213	10,4	0,237	0,097
600	-	0,141	15,6	0,218	0,068
800	-	0,095	20,8	0,200	0,047
1000	-	0,062	26,0	0,187	0,033

The curves of water capacity coefficient values depending on depth or rock pressure, for caving, hydraulic-filling and packing are given in figure 1.

References

Hurysz J., Adamek R., 1960. Skruszone skały płonne, jako materiał do podsadzki płynnej. /Crushed barren rocks as a material for hydraulic filling/. Prace GIG, Komunikat nr 265, Katowice.

THE DETERMINATION OF THE TRANSMISSIVITY OF COASTAL AQUIFERS BASED ON THE OBSERVATION OF SINUSOIDAL, ONDULATORY, TRANSITORY REGIMES INDUCED BY TIDAL OSCILLATIONS

Ucero Aoiz, L.*; Uriel Romero, S.**

ABSTRACT : It is not infrequent to find that work to be carried out in harbours and shipyards demands large-scale excavation in dry conditions below sea level. The pumping operations required in the course of the execution or development of the work often dictate the nature of the constructional process and its typology. In this paper, an attempt is made to develop a method for ascertaining the transmissivity of the terrain affected by the excavation, on the basis of the damping effects or time-lag with which the oscillation of the tide is transmitted through the filtering media. This method, which was initially developped by FERRIS for one-way propagation in homogeneous terrain, is here extended to cover more frequent, irregular situations where such propagation is bi-directional.

RESUME : Les chantiers qui sont réalisés dans les ports et les arsenaux, exigent fréquemment l'exécution de grandes excavations à sec, au-dessous du niveau de l'eau. Les épuisements nécessaires en phase d'exécution ou d'exploitation du chantier conditionnent fréquemment le processus constructif ou même la typologie de celui-ci. Dans cette communication on développe une méthode pour estimer la transmissivité des terrains, affectés par l'excavation, à partir de l'adoucissement et du retard avec lesquels l'oscillation de la marée se transmet à travers un milieu filtrant. Cette méthode, développée par FERRIS pour la propagation à une direction en terrain homogène, s'étend aux cas plus fréquents d'irrégularités, et de propagation à double direction.

RESUMEN : Las obras que se realizan en puertos y astilleros precisan, con frecuencia, la ejecución de grandes excavaciones en seco, bajo el nivel del mar. Los agotamientos precisos, en fase de ejecución o explotación de la obra, condicionan frecuentemente el proceso constructivo o incluso la tipología de la misma. En la comunicación se desarrolla un método para estimar la transmisividad de los terrenos, afectados por la excavación, en base a la amortiguación y al desfase con que la oscilación de la marea se transmite a través de un medio filtrante. Este método, desarrollado por FERRIS para propagación unidireccional en terreno homogéneo, se extiende a los casos más frecuentes de irregularidades, y a propagación bidireccional.

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1. Introduction

In relation with harbour works, ship building and repair facilities, it is frequent to be confronted with the problems arising from the need to dewater excavations at depths of up to 15 m. below sea-level.

This occurs in the case of large-sized dry docks. Here, the dewatering operations during the execution of the work and remaining water flows during actual exploitation, if the dry dock is of a "drained" type, may constitute one of the fundamental conditioning factors when selecting the structural typology of the work and the methods of construction. Before such problems can be solved, the transmissivity of the terrain affected by the excavation work must be known with the greatest possible degree of accuracy.

We were faced with a particularly difficult case of this type when making and analysis of the problems which might arise during the construction and maintenance of the dry dock for ship repairs which Diatlansa intends establishing on the island of Tenerife. This involved the performance of large-scale excavation works in volcanic terrain lying along the Atlantic coast, where the soil consisted of a layer of highly pervious slag lying on top of a relatively impervious basaltic "coulée".

In pervious soils, the oscillations produced by the ocean waves are transmitted inland to a considerable distance, with the resultant damping effects and time-lag which are a function of the hydrogeological characteristics of the terrain involved.

Consequently, it was felt that a quick, economical way of estimating these parameters might be to analyse the oscillations of the water table level in a grid of borings which had been carried out in the area for the geological investigations. For interpreting the results obtained, we resorted initially to the method developed by FERRIS in 1941. But the limitations of this method, soon became quite clear, since it refers exclusively to the unidirectional propagation of a pure sinusoidal wave in homogeneous, isotropic, indefinite terrain.

This paper is a summary of the principal results of the studies which, taking the original idea as their point of departure, were carried out in an attempt to find solutions for the greatest possible number of practical cases. It should be pointed out that other methods must even

tually be employed in order to estimate the effective porosity of the terrain over a period equal to the half-life of the tidal wave.

2. Working hypotheses and the range of application of the study

Apart from those mentioned specifically in each section, the following are some of the more general hypotheses adopted for this work:

- Newtonian fluid.
- The absence of incoming flows of water from the terrain.
- The absence of surface recharge.
- An aquifer of homogeneous thickness and a horizontal bottom.
- No account is taken of phenomena such as evapo-transpiration, capillarity or the effects of air dillution in the water.

Furthermore, in order to be able to apply the Dupuit approach, which is indispensable for the validation of the results, it is vital that the extension of the aquifer be at least twice its thickness. Moreover, if the flow is to be laminar, the thickness of the aquifer must be at least four times the amplitude of the tidal oscillation (if the transmissivity is less than 2,000 m2/hour, the effective porosity is 30% and the average diameter of the grains is 1 cm.)Lesser thicknesses may be tolerated for smaller values of these characteristics.

3. Unidirectional

3.1. General equation

Bearing in mind the Dupuit approximation and the fact that the free - surface is sufficiently flat, if the thickness of the aquifer is relatively large with respect to the oscillation, then the equation representing movement may be written as follows:

$$\frac{\partial^2 h}{\partial x^2} = \frac{n}{c} \frac{\partial h}{\partial t}$$

where:

c is the transmissivity of the aquifer.
h is the height of the free surface or the piezometric head, as the case may be.
n is the effective porosity.

This equation is really exact in the case of an artesian aquifer, and is sufficiently close in the case of a water table whose only variations are the result of tidal oscillations.

For x = O (with, pure sine oscillation on the coast), the general - boundary condition will be:

$$H = H_o \sin \frac{2\pi t}{T}$$

where H_o is the semi-amplitude of the tidal oscillation and T is its duration. Further on, an analysis will be made of the influence of - the fact that such oscillations may not be purely sinoidal.

3.2. Homogeneous, indefinite aquifer

In the case investigated by FERRIS, the following is the equation - which here defines the oscillation of the water at a point at distance x from the coast:

$$h = H_o \, e^{-x\sqrt{\frac{\pi n}{TC}}} \sin \left(\frac{2\pi t}{T} - x\sqrt{\frac{\pi n}{TC}}\right)$$

The damping coefficient (that is, the ratio between the semi-amplitude of the oscillation of the water at that point, h_o, and that of - the oscillation of the sea) would be:

$$\frac{h_o}{H_o} = e^{-x\sqrt{\frac{\pi n}{TC}}}$$

and the value of the time-lag in hours, t_α, between the oscillation at that point and that of the sea, would be:

$$t_\alpha = \frac{x}{2}\sqrt{\frac{T}{\pi C}}$$

Figure 1 shows the values of the damping coefficiente (more easily - observed that those of the time-lag) for n = O.1, T = 12.67 hours - and different values of C. It should be stressed that the curves are valid as long as the value $\frac{n}{TC}$ remains unchanged.

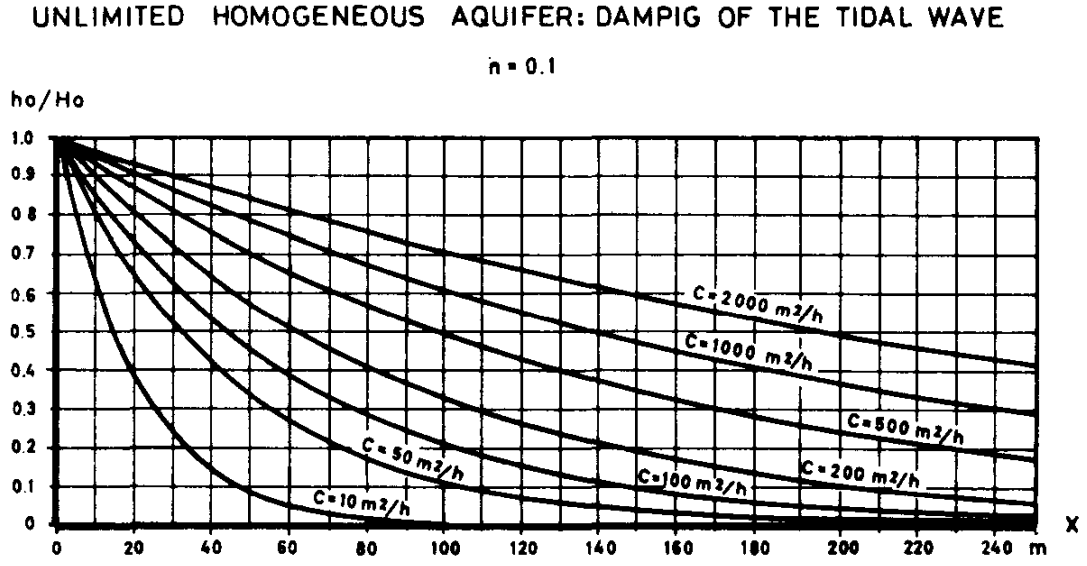

Fig. 1

3.3. An aquifer formed by two consecutive homogeneous media

This is the case when the terrain reveals certain n_1 and c_1 characteristics up to a distance L from the coast, and other different n_2 and c_2 characteristcs theareafter. Two different approaches may be used to analyse such a system:

a) The approximate method. Il is assumed that the oscillations in the first medium are not affected by the existence of the second, and that, in this latter medium, the movements are as if this imaginary boundary were actually a coastline, where the damping effects and time-lag of the tide were the result of their transmission through the first. In these conditions, the damping coefficiente would be:

In the first medium : $$\frac{h_{1O}}{H_o} = e^{-x\sqrt{\frac{\pi n_1}{TC_1}}}$$

In the second medium : $$\frac{h_{2O}}{H_o} = e^{-L\left(\sqrt{\frac{\pi n_1}{TC_1}} - \sqrt{\frac{\pi n_2}{TC_2}}\right) - x\sqrt{\frac{\pi n_2}{TC_2}}}$$

where the origin of the coordinates is the coast.

b) The exact method. The differential equations for both media are integrated, with their respective boundary conditions and flow continuity condition at the boundary. Calling $u_1 = x\sqrt{\frac{\pi n_1}{TC_1}}$, $u_2 = x\sqrt{\frac{\pi n_2}{TC_2}}$,

$p = L\sqrt{\frac{\pi n_1}{TC_1}}$ and $\nu = \sqrt{\frac{c_1\ n_1}{c_2\ n_2}}$, and adopting the boundary line as the origin of the coordinates for both media, the damping coefficient has the following values:

In the first medium: $$\frac{h_{10}}{H_o} = \frac{\sqrt{\sin^2 u_1(\nu \operatorname{Sh} u_1 + \operatorname{Ch} u_1)^2 + \cos^2 u_1(\operatorname{Sh} u_1 + \nu \operatorname{Ch} u_1)^2}}{\sqrt{\sin^2 p(\nu \operatorname{Sh} p + \operatorname{Ch} p)^2 + \cos^2 p(\operatorname{Sh} p + \nu \operatorname{Ch} p)^2}}$$

In the second medium: $$\frac{h_{20}}{H_o} = \frac{\nu}{\sqrt{\sin^2 p(\nu \operatorname{Sh} p + \operatorname{Ch} p)^2 + \cos^2 p(\operatorname{Sh} p + \nu \operatorname{Ch} p)^2}}\, e^{-u_2}$$

Figure 2 is a graphical representation of these coefficients, where a comparison is drawn between the two ways of solving the problem for $\mu_1 = \sqrt{\frac{\pi n_1}{TC_1}} = 0.0035\ m^{-1}$ and $\mu_2 = \sqrt{\frac{\pi n_2}{TC_2}} = 0.0100\ m^{-1}$ (and $\mu_1 = 0.0100\ m^{-1}$ and $\mu_2 = 0.0035\ m^{-1}$, in the inverse case). The distances from the boundary to the coast are 20; 40 and 100 m.

AQUIFER CONSISTING OF TWO HOMOGENEOUS ZONES: WAVE DAMPING

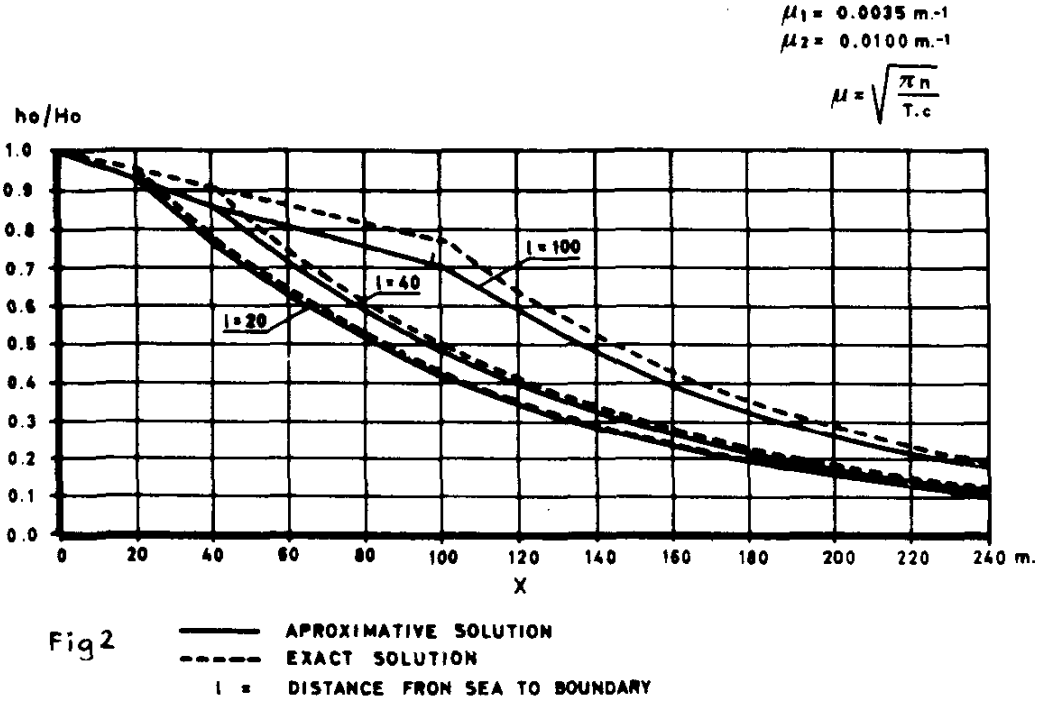

Fig 2

——— APROXIMATIVE SOLUTION
- - - - - EXACT SOLUTION
l = DISTANCE FROM SEA TO BOUNDARY

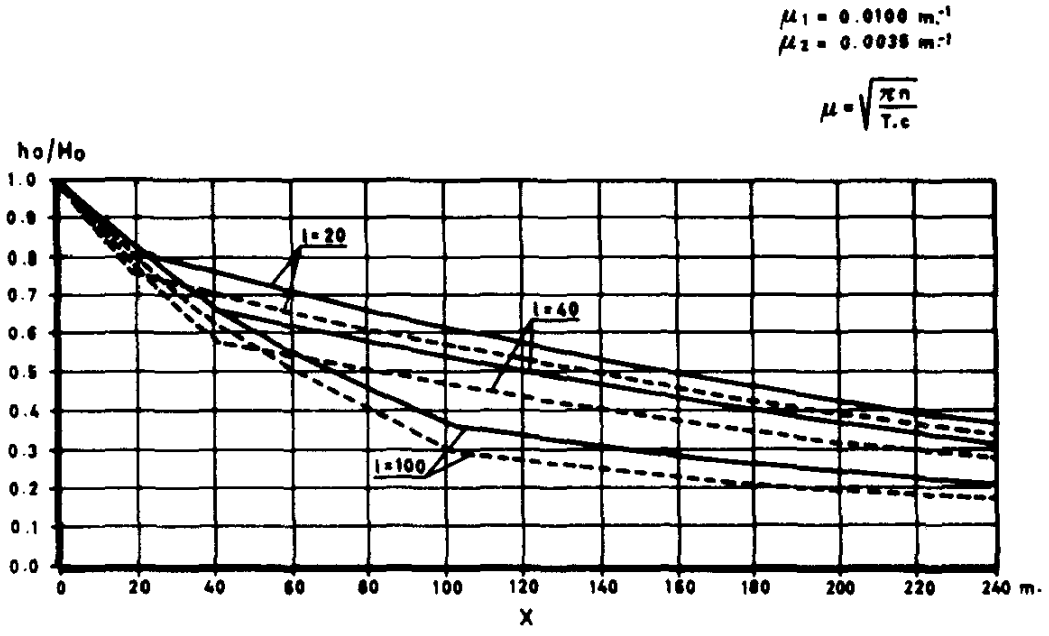

Fig. 2'

The approximate method, which is simpler and quicker, may be used in a first analysis if the following is the case:

- The observation point is not near the boundary.
- The diffusivity $\frac{C1}{n1}$ of the first medium is greater than that of the second.
- The value of $P < 0.2$

3.4. The case of preferential ways

This is the case when, in a homogeneous medium, there is a path whose conditions are much more favourable for the passage of water. It may be analysed in an approximate manner, by assimilating it to the case mentioned above (see Figure 3), provided that:

- The distance, d, from the analysed point to the preferential path is much shorter than the distance D from this point to the coast.

- The dimensions of this path are such that they do not impose any important restriction on the expected flow.

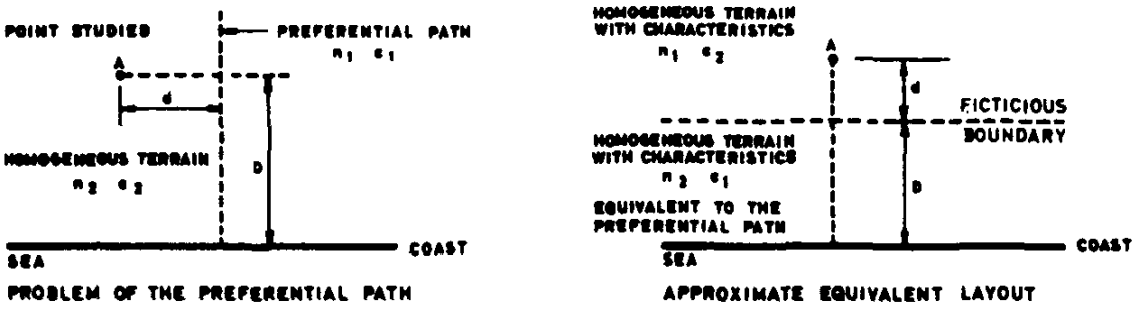

Fig. 3

Consequently, if the characteristics of the media are known, it is - possible to deduce the distance d from this point to any preferential path, as is shown in Figure 4 for two cases of D, μ_2.

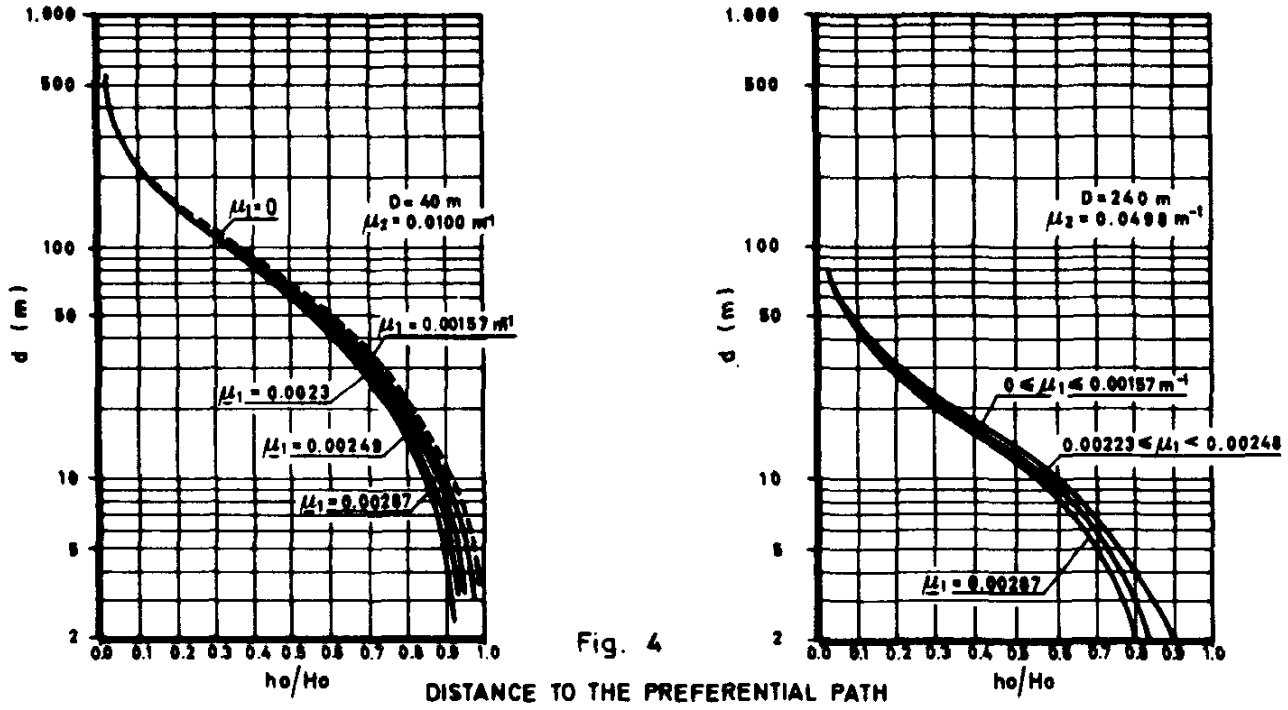

Fig. 4

As will be observed, the characteristics of a preferential way - provided that the diffusivity is very high - are of little importance - and the damping effects at any one point tend to be similar to those which would be present if the preferential path were a coastline.

3.5. An aquifer limited by an impervious barrier

This is the case of an aquifer which is limited by an impervious barrier at a distance L from the coast. The linear nature of the differential equation makes it possible to use the image method to satisfy the boundary cobdition which the existence of the barrier repre - sents, since the movement of the water will be the same as if, at a distance of 2L, there were another imaginary coast with synchronous oscillation. Consequently, this solution may also be applied in the case of an isthmus.

So, taking this barrier as the origin, it appears that the damping coefficient will be:

$$\frac{h_o}{H_o} = \sqrt{\frac{\cos 2u + \mathrm{Ch}\ 2u}{\cos 2p + \mathrm{Ch}\ 2p}}$$

and the angular displacement will be:

$$\alpha = \arctan \frac{\tan p\ \mathrm{th}\ p - \tan u\ \mathrm{th}\ u}{1 + \tan p\ \mathrm{th}\ u\ \tan p\ \mathrm{th}\ p}$$

Figure 5 represents both functions for different values of $\mu = \sqrt{\frac{\omega n}{TC}}$ and L = 50 m.

HOMOGENEOUS AQUIFER LIMITED BY AN IMPERVIOUS BARRIER
BARRIER AT 50 m.

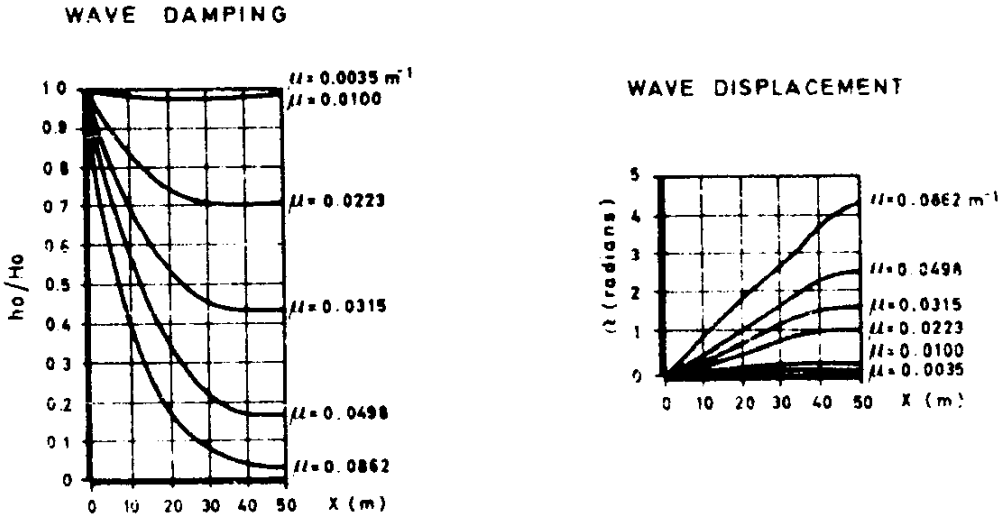

Fig 5

In order to know the extent to which the existence of a barrier affects the results, Figure 6 shows cases without any barrier, or with one at a distance of 250 m.

COMPARISON BETWEEN THE DAMPING COEFFICIENTS OBTAINED WITH OR WITHOUT AN IMPERVIOUS BARRIER

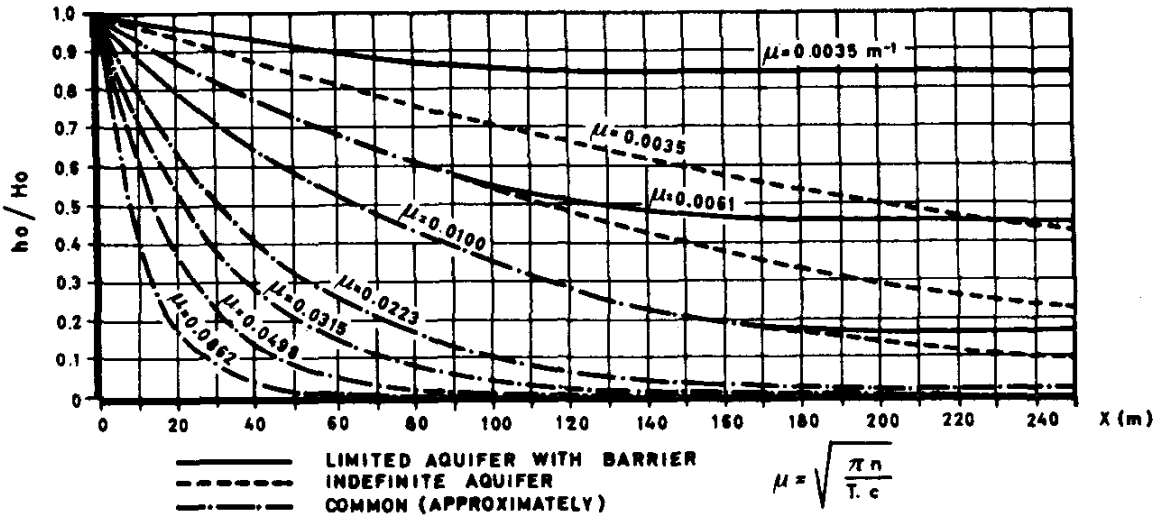

Fig. 6

Consequently, it may be observed that, if the diffusivity is not excessively high and the barrier lies at a considerable distance, then the analysis may be carried out without having to take these effects into consideration.

3.6. The influence of variations in the amplitude of the oscillations

All the foregoing analyses were based on the assumption that the tidal wave is a continuously pure sinoid. The amplitude of the tide is not constant, however, and actually undergoes a series of daily, fortnightly, monthly, biannual and annual variations. Here, returning once again to the basic case of a semi-indefinite, homogeneous medium, let us investigate the effects which each of these variations (considered separately) may have. In other words, let us analyse what actually happens if the amplitude follows a sinoidal law; that is, if the wave, in the sea, are of the following type:

$$H = H_1 \left(1 + \xi \cos \frac{2\pi t}{T_2}\right) \sin \frac{2\pi t}{T_1}$$

where T_1 is the basic oscillation and T_2 is the superimposed variation in the amplitud.

If we make $\frac{1}{T'_1} = \frac{1}{T_1} + \frac{1}{T_2}$ and $\frac{1}{T'_2} = \frac{1}{T_1} - \frac{1}{T_2}$ then, by integrating - the differential equation of the movement, the oscillation at any one point is griven by:

$$h = H_1 \left\{ e^{-x\sqrt{\frac{\pi n}{T_1 c}}} \operatorname{sin}\left(\frac{2\pi t}{T_1} - x\sqrt{\frac{\pi n}{T_1 c}}\right) + \frac{\xi}{2}\left[e^{-x\sqrt{\frac{\pi n}{T'_1 c}}} \operatorname{sin}\left(\frac{2\pi t}{T'_1} - x\sqrt{\frac{\pi n}{T'_1 c}}\right) + e^{-x\sqrt{\frac{\pi n}{T'_2 c}}} \operatorname{sin}\left(\frac{2\pi t}{T'_2} - x\sqrt{\frac{\pi n}{T'_2 c}}\right)\right]\right\}$$

It is demonstrated that, when T_2 3OT$_1$, (that is, fortnightly variations or greater) the assumption that, when analysing a period, the amplitude is constant throughout and equal to that measured during - the period, such an assumption does not give rise to errors greater than O.16 . In actual practice, therefore, the effects of such an - assumption are negligible.

Nor will there be any excessive degree of error, if no allowance is made for daily variations, provided that the point is near the coast (displacement of less than one period) and variations in the amplitude with respect to the mean are no greater than 10%. Figure 7 gives us an idea of the order of magnitude of the errors committed, through excess, when ξ = O,1. O.2 and O.4. This is done by means of curves - obtained from the logarithmic and exponential adjustment of a series of values adopted as maximum limits of error.

VARIATION IN THE ERROR AS A FUNCTION OF THE u COEFFICIENT OF A POINT WHEN CONSIDERING THE SEA LEVEL TO BE CONSTANT AND EQUAL TO THE MEAN

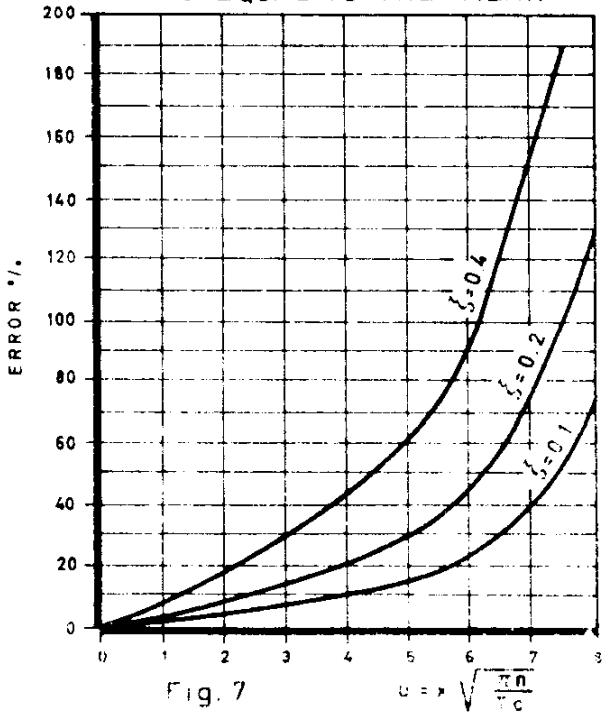

Fig. 7

Although, at a first glance, this graph might give rise to some alarm, in actual practice it is most unusual to find ξ or u values greater than 0.1 or 5 respectively. In other words, errors will be less than 15%.

4. Biaxial current

4.1. General equation

Under the same conditions and using the same notation as in section 3.1., the following is the equation which governs the movement of a two-dimensional wave through a homogeneous, isotropic medium:

$$\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial y^2} = \frac{n}{c} \frac{\partial h}{\partial t}$$

where the boundary condition, as in the above-mentioned section, is the sinoidal oscillation of the tide all along the coast.

4.2. Solution by numerical calculation

The above equation may be solved by means of finite differences and subsequently calculated numerically on a computer. PRICKETT and LONNQUIST have developed a solution to the equation together with the corresponding programme, on the basis of an iterative system using files and columns. The programme operates with any coherent system of units and, in principle, is valid for confined aquifers although, as already indicated, in our case this solution is sufficiently approximate to be applied to unconfined aquifers. The programme boundary conditions may be either impervious barriers, constant-head recharges or constant-flow recharges. The expected time increments increase in a uniform way at a rate of 1:2. What this means is that the programme cannot be directly applied to the case in question unless these elements are subject to some modification.

4.3. Application to the case of an indefinite, homogeneous aquifer, limited by two orthogonal coasts

The programme was adjusted by means of the following modifications:

- The time delays were taken as being constant and equal to 0.1.T.
- For each time delay, we assumed a constant-head recharge along the edges of the coasts (using values given by the sinoidal function of the tide) and along another two edges which are parallel to the coasts and lie sufficiently far in to nullify the effects of other edges, according to the movement equation defined in section 3.2.

For calculation purposes, we adopted a mean sea level of 1.35 m. a tidal semi-amplitude of H = 1.20 m. and T = 12.67 h.

At the moment when t = O, it was assumed that, at all points on the 20 x 20 m. grid, h = 1.35. Operations then continued until the observed heads at one point, at an equivalent instant in two consecutive periods, were markedly similar (differences of less than 10%). This was generally achieved for the third period.

On the basis of the numerical data obtained, it was possible:

a) To draw the variation of the damping coefficient in lines orthogonal to the coast, as may be seen in Figure 8 where n = 0.2 and C = 500 m^2/hour.

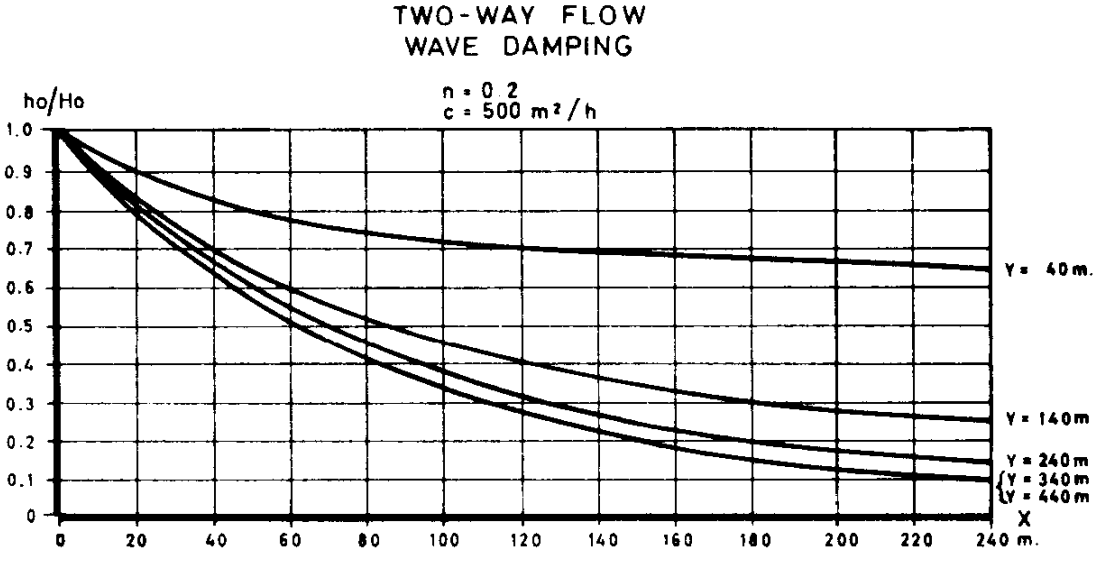

Fig. 8

b) To draw a ground plan of the lines of equal damping for the above case, as will be seen in Figure 9.

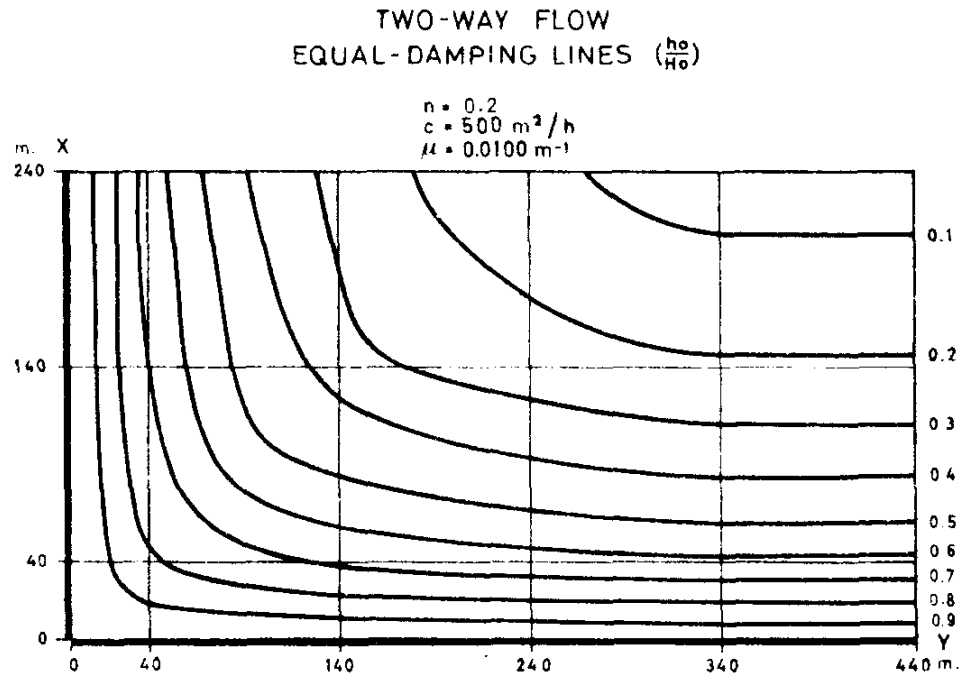

Fig. 9

c) Taking the data obtained from the different calculations as a point of departure, to try and find (for each point on the main grid in Figure 9) some correlation between the damping coefficient and the value of $\mu = \sqrt{\frac{\pi n}{TC}}$, which is characteristic of the medium and the oscillation. The results appear in Figure 10 which may be inmediately and directly applied to cases which often arise in actual practice, if the damping is measured at any point on the grid.

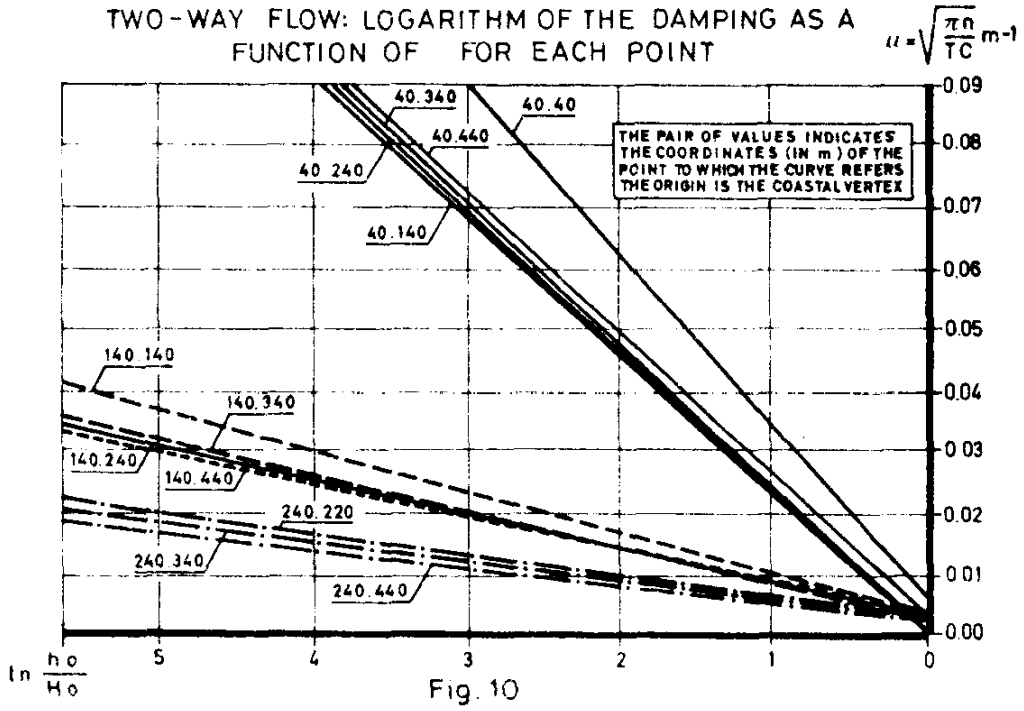

Fig. 10

Similarly, displacement could be treated in the same way, although, in view of the fact that it is rather more difficult to calculate in actual practice, it is felt that it need not be included in this context.

d) It is demonstrated that, at a specified distance from one of the coasts (a variable as a function of μ), the behaviour of the water table is practically unaffected by its existence. Therefore, it is unnecessaty to make a two-way analysis. These values are:

μ	D
< 0.0045 m^{-1}	> 500 m
0.0065	400
0.0090	300
0.0200	200
0.0500	100

4.4. The simultaneous equations method

This is a simple, approximate method for determining the characteristics of the terrain, making allowance for a lack of homogeneity and isotropy and with no particularly defined contour lines. Here, we rely on the characteristics of the oscillations observed in a network of points.

For this purpose, we assume a discretizated medium such that the water which enters or leaves a point O must do so through a system of orthogonal tubes; in other words, the flow is broken down vectorially in accordance with directions x and y.

Consequently, we take point O in the network, together with another four adjacent points. Thus, the oscillation at O will take the following form:

$$h = M \sin \frac{2\pi t}{T} + N \cos \frac{2\pi t}{T}$$

and at one of the four adjacent points:

$$h_i = M_i \sin \frac{2\pi t}{T} + N_i \cos \frac{2\pi t}{T}$$

Since all along the flow line i, O the one-way current differential equation must be satisfied, this equation may be integrated throughout the length of the flow tube, under the previously mentioned conditions taking into account the continuity at point O. If we assume that, along each path, the transmissivity, c, and the effective porosity, n, are constant, then we obtain:

$$M = U(p)\Sigma M_i + V(p)\Sigma N_i$$
$$N = -V(p)\Sigma^1 M_i + U(p)\Sigma^1 N_i$$

where U (p) and V (p) are highly complex functions of $p = L\mu$. L is the side of the network of observation points and μ retains the same significance as before, $\sqrt{\frac{\pi n}{TC}}$. Figure 11 summarises the variations in these expressions as a function of p.

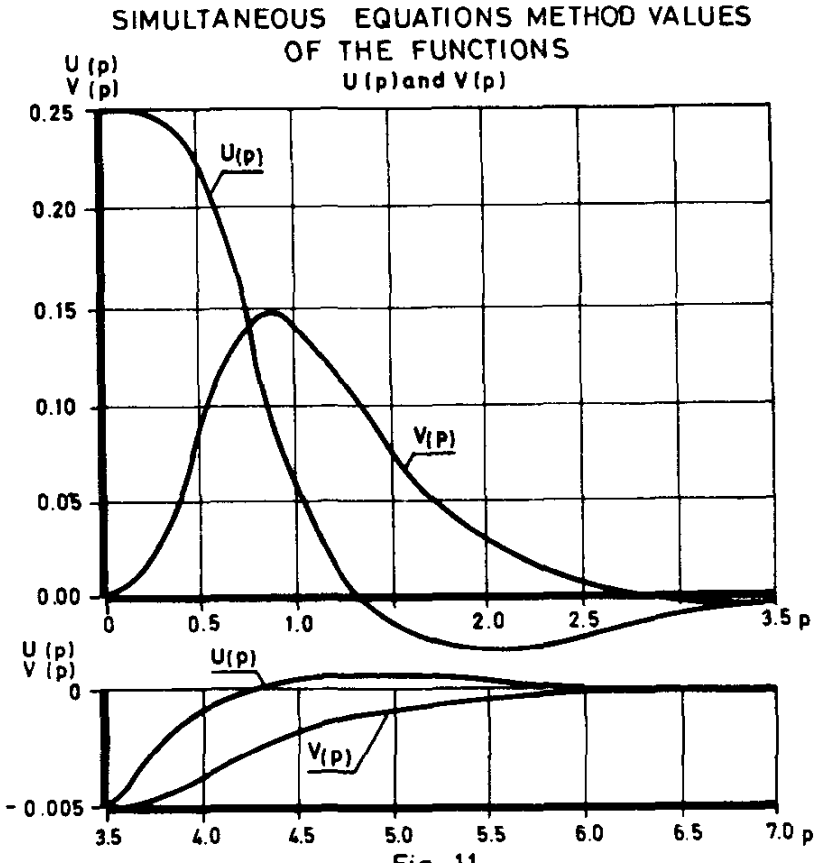

Fig. 11

Therefore, if we know the characteristics of the terrain and the oscillations at these four extreme points, we can use the above expressions to arrive at those of the centre point. Moreover, since

$$U(p) = \frac{M\Sigma M_i + N\Sigma N_i}{(\Sigma M_i)^2 + (\Sigma N_i)^2}$$

$$V(p) = \frac{M\Sigma N_i - N\Sigma M_i}{(\Sigma M_i)^2 + (\Sigma N_i)^2}$$

If it is true that c and n are constant throughout the space under consideration, then we can obtain the transmissivity of the terrain using the oscillations measured at a central point and another four adjacent points.

On the basis of this method, the operative process for finding the transmissivity of any terrain whatsoever, would be as follows:

a) The terrain is divided up into a square grid whose sides are made shorter as the heterogeneity increases. Using the piezometers at each of the vertices, we can find the values of M and N which define tidal oscillation at these points.

b) The points are taken in groups of five: one in the centre and four immediately adjacent points following the lines of the grid. Using the M and N values observed at these points, we find U(p) and V(p) which, when inserted in Figure 11, give us the values of p. These values should not differ from one another by more than the permitted degree of error (in the region of 20%, bearing in mind the characteristics of the method used). The mean of p will provide a transmissivity value. If, however, the difference between the p values is excessive, the grid must then be sub-divided into smaller stretches, since this reveals the characteristics of the terrain change too brusquely with respect to the initial dimensions of the grid. If $U(p) > 0.25$, this means that the centre point is not fed from the sides. In consequence, the grid must be rotated until the preferential paths are located. A swing of 45º, which allows the same square grid to be used, is normally sufficient.

c) In this way, we finally obtain for each stretch of grid two transmissivity values, depending on how either one end or the other was considered as being the centre point. The mean of the two may be taken as being the definitive value of the zone.

5. Conclusion

We feel that, using the procedures indicated, we have been able to generalize FERRIS, method in such a way that it may be applied to a large variety of situations. It must be remembered that the methods for measuring the elevation of the water in a piezometer may, of themselves, give rise to some margin of error. Consequently, the simultaneous equation method may, in practice, be applied as a method for obtaining, with some degree of accuracy, the characteristics of zones where there is some heterogeneity. Once the combined variation of the transmissivity is known, that will be the moment to apply some of the other, more exact systems which are normally used in cases of well-defined morphology to arrive at a more precise adjustment of this characteristic --

which is so decisive when planning not only the means of execution of the works but also the definitive morphology.

6. Analysis applied in the Granadilla dry dock, in Tenerife

To obtain data concerning the permeability of the ground at a large number of points in the area, intention was made to use the geological borings grid in order to measure the tidal oscillations at these points and compare them with oscillations at the sea. To this purpose all the borings were used as piezometers in order to observe variations in the level of the water in their inside. For illustrative purposes, figure 12 shows the measurements taken one day along a line of borings.

It was found that, in all borings, water level oscillation was very - important, even at the furthest points to the shoreline

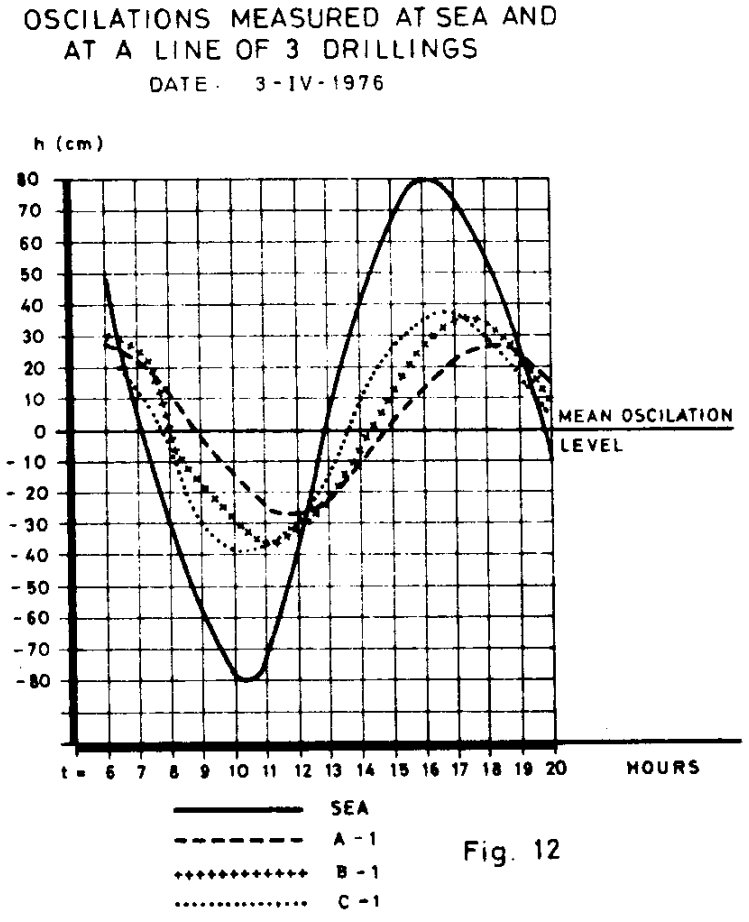

Fig. 12

and that the damping coefficient (oscillation amplitude at the boring divided by oscillation amplitude at the sea) ranged between 0,22 and 0,64.

The interpretation of the data provided by these observations tend to be very different, depending on the hydrogeological morphology assigned to the sub-soil and the estimates made for the free parameters. As far as morphological aspects are concerned, it should be pointed out that the Ferris method assumes a totally homogeneous aquifer and monodirectional propagation of the perturbance, and with this assumption there exists no satisfactory adjustment for any pair of n, C values. In consequence it was necessary to analyze a certain number of additional hypothesis. In this way the following additional interpretations were made:

a) The transmissibility along coast is less than in the rest of the area.

   In this way it turns out that the hypothesis which best adjusts to the observed variations in the water levels is that the width of the less pervious zone lies between only a few meters and 25 m. with an average transmissibility from 25 to 250 m2/hour; for the remainder of the aquifer the transmissibility would be in the region of 4000 to 6000 m2/hour (n=0,1).

b) An analysis was also made of the influence which might result from the form of the coast (an angle of practically 90º) since this implies that water may be enterring on two sides and that the problem is therefore, two-directional.

   The result was too precarious and did not give a satisfactory explanation for the observed oscillations.

c) The last hypothesis taken into consideration was the existence of preferential ways in the ground where resistance to the water circulation was either nil or very slight. In this case, if these preferential ways are sufficiently near to each other, even if the transmissibility of the ground is low, the damping effect can be adjusted to the observed values. On making these calculations the results show that, if such a preferential way does exist at 25 m. from the point of observation (n = 0.1, as always) the transmissibility wich corresponds to the observed damping effect would be between 19 and 30 m2/hour, whereas if such a preferential way lays 50 m. away, the transmissibility must be supposed higher, from 28 to 45 m2/hour.

The analysis just described, give a picture of the hydrogeological problem raised by the future construction of the dry dock which, in principle, may be considered as uncertain. The interpretation of these tests and their results may very greatly depending on the values assigned to the parameters involved, concerning which there exists no real certainty (for exemple, concerning the value of n).

Needless to say, when evaluating the results several different interpretations may be made. Amongst these here is this one which, in our opinion, may lead to the maximum degree of compatibility between the observations made:

i) There exists, between the sea and the land, a moderate transmissibility coastal barrier due to a smaller thickness of surface slag in this zone, or to a lesser degree of fissuration in the volcanic strata or even to minor openings of the fissures as resul of sediments.

ii) The inner zone has a medium-to-high transmissibility.

iii) The entire zone described above may be crossed by high transmissibility preferential ways represented by much more transmissible zones within the slag, or by fissuration in the basalt, or by both.

As a result, we think now, that any risks caused by hydrogeological problems during construction may be technically overcome. The same applies also to the risks which arise concerning the exploitation, which may need the adaptation of the design to the real conditions of the ground.

With regard to the real development of events and dewatering problems in the course of construction and exploitation (the accepted solution includes a drained slab), we must wait to the results of the works.

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CORRECCION DE LADERAS ARCILLOSAS MEDIANTE LA EJECUCION DE PANTALLAS DRENANTES SUBTERRANEAS

ERASO, A.

ABSTRACT: The paper deals with the problems of slump in clayish slopes which frecuently affect civil works in the south of Spain. A special technique is developed, which resolves the problem by influencing the reasons and modifying them accordingly. This special technique merely consists in the coordinated use of other specific and well know techniques, and its success lies in its simplicity and common sense. In all cases the obtained result has been a complete success, remaining as only question the life of these draining curtains, of which the oldest one is working already for four years without problem.

RESUME: L'auteur décrit les problèmes de glissement des pentes argileuses qui se produisent souvent dans les travaux de génie civil au sud de l'Espagne. Une technique spéciale est developpée, qui résolve le problème en influençant les causes et les moidifiant adequatement. Cette technique spéciale consiste simplement dans l'utilisation coordonnée d'autres techniques spécifiques et bien connues, et son succès réside dans sa simplicité et sens commun. Dans tous les cas le résultat obtenue était un succés complet, et la seule question est encore la durée de ces écrans de drainage, dont la plus ancienne fonctionne depuis déjà quatre ans sans aucun problème.

RESUMEN: Se describen los problemas de deslizamiento de laderas arcillosas que afectan frecuentemente a las obras civiles en el sur de España. Se desarrolla una técnica especial, que resuelve el problema por incidir sobre las causas modificándolas adecuadamente. Dicha técnica especial consiste simplemente en la utilización coordinada de otras técnicas específicas bien conocidas, residiendo su brillantez en su sencillez y sentido común. El éxito conseguido en todos los casos es total, planteándose únicamente la duración en el tiempo de las referidas pantallas drenantes, de las cuales la más antigua funciona ya desde hace cuatro años, sin problemas de ningún tipo.

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PLANTEAMIENTO DEL PROBLEMA

Entre los puntos kilométricos 157 y 158 de la linea de ferrocarril de Córdoba a Málaga (tramo situado en la margen derecha del río Guadalhorce), y entre las localidades de Alora y Pizarra, las arcillas gris pardas del Paleógeno, que allí aparecen y sobre las que se instala la vía ferrea, deslizan lenta pero inexorablemente hacia el río, a pesar de que su pendiente natural es del orden de 20°, arrastrando consigo los railes, circunstancia que se ve favorecida por las cargas dinámicas debidas al paso de los trenes.

La situación descrita viene reflejada en la figura 1, donde se aprecia que los niveles piezométricos del terreno se hallan muy próximos a la superficie natural, es decir, que el terreno se encuentra saturado por la existencia de lentejones de arena, más permeables, intercalados entre las arcillas, que funcionan a la manera de zonas de alimentación de las aportaciones pluviométricas, a favor de las pequeñas ramblas existentes, que las ponen al descubierto.

Los reconocimientos efectuados, mostraron un panorama bastante problemático.

Los límites de ATTERBERG dieron los siguientes resultados en 30 probetas experimentadas:

	Mínimo	Máximo	Media
Límite líquido	26,-	51,5	41,-
Límite plástico	12,-	21,5	19,-
Indice de plasticidad	14,-	30,-	24.-
Humedad natural	30,-	42,-	38,-

Los ensayos triaxiales efectuados indicaron para el ángulo de rozamiento interno φ y para la cohesión c valores comprendidos entre los siguientes valores extremos:

$$\varphi = 15^\circ\ 30' - 16^\circ$$
$$c = \text{nula} - 0{,}1\ \text{kgr/cm}^2$$

que suponen un elevado coeficiente de empuje activo para dichas arcillas:

$$\lambda_a = \text{tg}^2\ (45 - \varphi/2) = 0{,}578 - 0{,}568$$

Y los ensayos de resistencia al corte S_u para valores de humedad natural, es decir, del orden de la correspondiente al límite líquido, se encuentran entre los siguientes valores extremos:

$$S_u = 0{,}01 - 0{,}05\ \text{kg/cm}^2$$

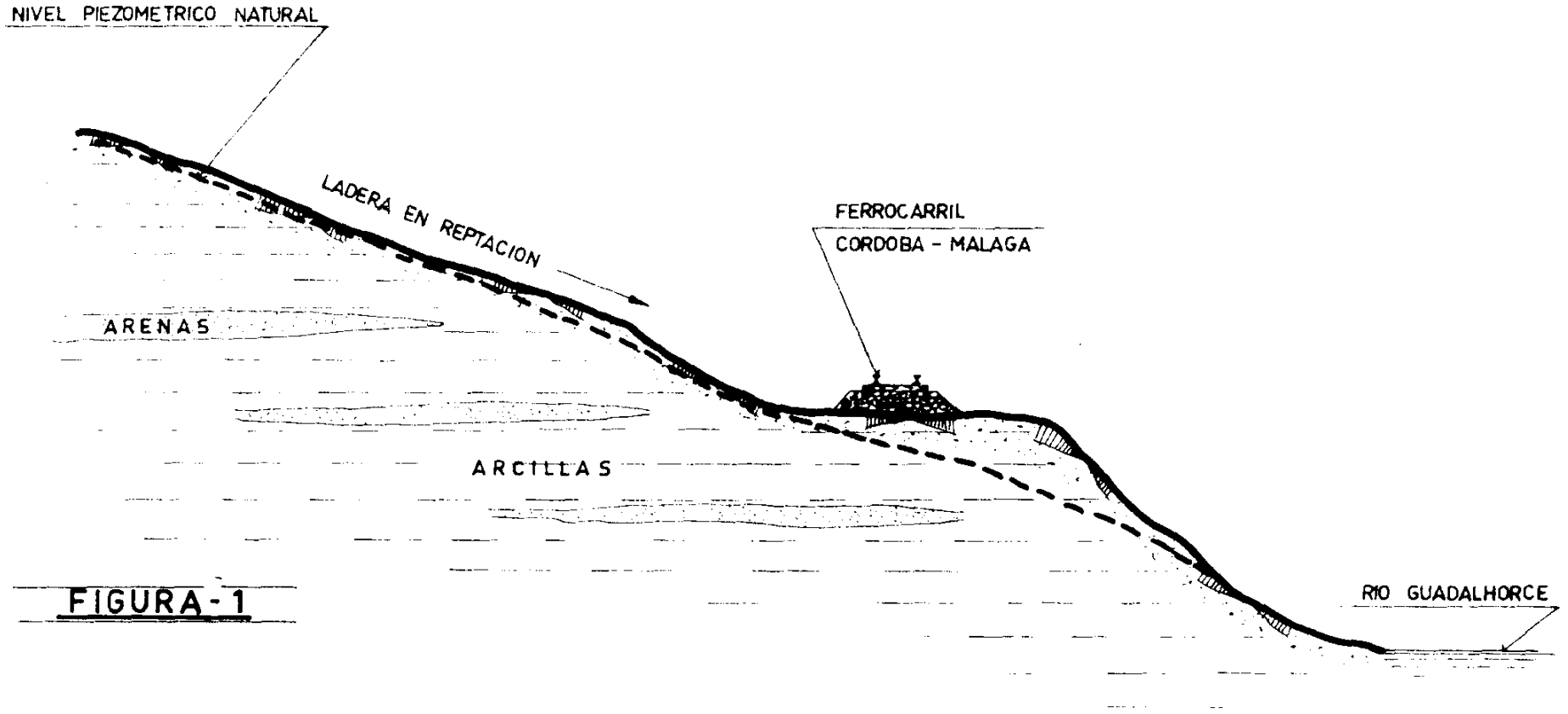
NIVEL PIEZOMETRICO NATURAL
LADERA EN REPTACION
FERROCARRIL
CORDOBA - MALAGA
ARENAS
ARCILLAS
RIO GUADALHORCE

FIGURA-1

En resumen:

- que la pendiente natural es mayor que el ángulo de rozamiento interno del terreno
- que la humedad natural es del orden del valor del límite líquido
- que la resistencia al corte, para dicha humedad natural, es muy baja.

definiendose un terreno muy inestable, saturado en agua, propenso a la reptación y al deslizamiento, lo que motiva la necesidad de ripar la vía cada pocas semanas, para situarla de nuevo en el trazado original.

SOLUCION ADOPTADA

A la vista del cuadro tan negativo de características geomecánicas del terreno, la solución intuitiva parece centrarse en proceder al drenaje del terreno.

Efectivamente, ensayando la resistencia al corte a humedades próximas al límite plástico, aquélla aumenta fuertemente, siendo sus valores extremos:

$$S_u = 0{,}98 - 1{,}23 \ \text{kg/cm}^2$$

evidenciando que el terreno drenado deja de estar en condiciones de fluencia.

Efectivamente, si el índice de fluidez viene dado por:

$$I_L = \frac{W - W_p}{W_L - W_p}$$

siendo:

W = humedad natural en %
W_L = límite líquido en %
W_p = límite plástico en %

de donde:

si $W = W_L$ ..... $I_L = 1$

y si $W = W_p$ ..... $I_L = 0$

siendo correlacionable el referido índice de fluidez con la resistencia a cortante S_u, observándose que cuando I_L pasa de 1 a 0, S_u aumenta hasta casi 100 veces su valor.

Para proceder al drenaje del frente de ladera, que debe de realizarse sin interrumpir el tráfico ferroviario, la solución que mejor ha resultado, consiste en la ejecución de una pantalla subterránea contínua drenante, que recoja las aguas de la ladera, y que las vierta por gravedad mediante una tajea o corto túnel bajo la vía, a cotas más bajas, modificando las condiciones iniciales de la figura 1, por las expresadas en la figura 2.

La realización de la referida pantalla, precisa de la combinación coordinada de numerosas técnicas, cada una de ellas conocida y utilizada aisladamente, sin la cual los problemas presentados no podrían olvidarse.

La primera técnica a emplear es la de *excavación subterránea de zanja contínua* por bataches, mediante cuchara bivalva guiada, con la que pueden alcanzarse profundidades de hasta 35 m.

Sin embargo, al estar el nivel piezométrico tan próximo a la superficie, y al liberarse en las arcillas tan lentamente las presiones intersticiales, las paredes verticales de la pantalla excavada en cuestión no se mantendrán, siendo necesario estabilizarlas.

La solución nos la aporta el empleo de la técnica de *lodos de perforación*, capaces de mantener estables paredes verticales en las condiciones de nuestro problema. Debemos pues excavar la pantaya por bataches, bajo lodo bentonítico.

Sin embargo, los lodos, en su cometido estabilizante, entarquinan y sellan las paredes de la zanja excavada, de la cual, no olvidemos, solicitamos una función drenante.

Hay pues que eliminar el lodo, pero sin perturbar la estabilidad de las paredes de la pantalla ya excavada.

La estabilidad se consigue mediante el vertido de *filtros*, a la manera de la técnica utilizada en los pozos de captación de agua.

La eliminación del lodo se consigue reemplazándolo con agua, mediante circulación en circuito cerrado, gracias a la aplicación de la técnica de *circulación inversa*, por insuflado de aire, empleada en la perforación de pozos de gran diámetro para captación de aguas.

Una vez logrado esto, queda todavía la costra (cake) o entarquinado, que sella las paredes y que es preciso eliminar.

Para ello, sin dejar de circular el agua, como en la operación anterior, se procede a flocular el referido entarquinado parietal, que es necesario desprender, para lo que se utiliza la técnica de *estimulación de pozos mediante acidificación.*

Cuando el agua salga ya limpia, sin residuos de costra, se da por terminada la operación de ejecución de la pantalla drenante propiamente dicha.

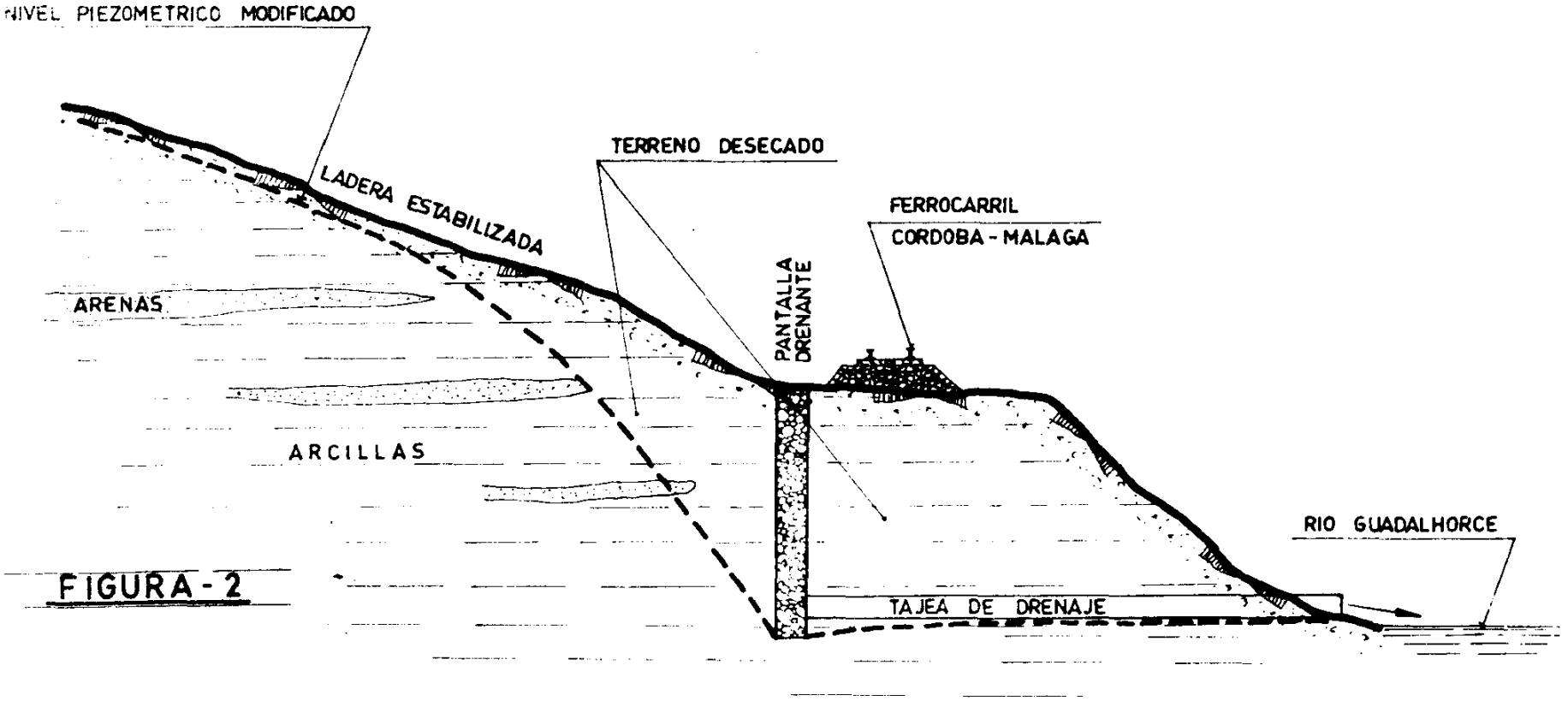
NIVEL PIEZOMETRICO MODIFICADO
LADERA ESTABILIZADA
TERRENO DESECADO
FERROCARRIL
CORDOBA - MALAGA
PANTALLA DRENANTE
ARENAS
ARCILLAS
RIO GUADALHORCE
TAJEA DE DRENAJE

FIGURA - 2

La última operación, consiste en la perforación del túnel de desagüe; se desarrolla mediante la aplicación de las técnicas de *excavación de túneles*, quedando la pantalla drenante apta para su servicio una vez que el referido túnel cala hasta la pantalla.

Resumiendo, las técnicas que se combinan en la ejecución de una pantalla drenante son:

- Excavación subterránea de zanja vertical en bataches con cuchara bivalva
- Empleo de lodos tixotrópicos como estabilizante
- Introducción de filtros para captación de aguas
- Circulación inversa por insuflado de aire
- Estimulación de pozos mediante acidificación
- Excavación de túnel (y sustentación) de drenaje.

RESULTADOS

El sentido físico de los resultados se interpreta a la vista de la figura 2, donde se recogen los niveles piezométricos modificados por la existencia de la pantalla drenante, de manera que las zonas desecadas por drenaje presentan elevada resistencia a esfuerzo cortante, actuando de cuña estabilizante frente a los empujes de la ladera no desecada más arriba.

A efectos de cálculo habremos de considerar el empuje pasivo de la cuña estabilizante drenada, que en nuestro caso presenta valores de coeficiente de empuje pasivo, comprendidos entre los siguientes extremos:

$$\lambda_p = tg^2 (45 + \varphi/2) = 1{,}73 - 1{,}76$$

En el ejemplo descrito, los resultados han sido espectaculares, ya que desde hace cuatro años que dicha pantalla drenante, primera ejecutada en España, fué realizada, el tramo de vía ferrea afectado, no ha vuelto a moverse, a pesar de que los caudales drenados son muy pequeños, del órden de:

$$Q \leq 1 \text{ litro /seg. por cada } 1000 \text{ m}^2 \text{ de pantalla}$$

Posteriormente se han realizado en diferentes arcillas de Andalucía, varios km lineales de pantalla drenante, siempre con éxito, logrando en todos los casos detener completamente la reptación de las respectivas laderas.

Ahora se cuestiona el problema de saber la vida útil de las referidas pantallas drenantes. Nada podemos decir al respecto, salvo que el referido problema no pasa de ser todavía una mera disquisición académica, ya que en el práctica el problema real aún no se ha planteado.

De todas maneras, su reactivación y estimulación, con los mismos procedimientos descritos, se presenta técnicamente muy viable y discreta en costos y plazos.

TEMPERATURE REGIME IN THE SAVA R. ALUVION NEAR ZAGREB

Sarnavka, R.

ABSTRACT: To preserve the ground water quality used for Zagreb water supply, the temperature regime of territorial waters has been investigated in order to prevent eventual thermic contamination, because of highgrade pollution of Sava r. that replenishes the alluvial reservoir. The hitherto results prove the immediate relation between Sava r. and ground water and explain certain characteristics relevant to it.

RESUME: Pour la qualité de l'eau souterraine dans la région de Zagreb, on a examiné le régime de ses eaux territoriales, pour prévenir une contamination thermique éventuelle, à cause d'une pollution, à haut degré, de la rivière Sava qui alimente le réservoir alluvial. Les resultats jusqu'ici ont prouvé une relation directe entre la rivière et l'eau souterraine, et ont aussi expliqué quelques détails caracteristiques.

RESUMEN: Para preservar la calidad del agua subterranea, utilizada en la región de Zagreb, se ha investigado el régimen de temperaturas de las aguas territoriales, al objeto de prevenir cualquier eventual contaminación, motivada por el alto grado de polución del río Sava, que alimenta al acuífero aluvial. Los resultados adquiridos, hasta el momento, ponen de manifiesto una relación directa entre el río y las aguas subterráneas, y permiten explicar ciertas características relevantes.

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INTRODUCTION

The town of Zagreb with its surroundings and cca one million in habitants is an obvious example of urban agglomeration where closely connected, nearly all possible water resources management and hydrotechnical problems are present, often in a very compound form as a consequence of morphological, hydrogeological and hydrological activities in this part of Sava r. alluvial plain. The regime of Sava river, its bed being built in the alluvial strata and partly cut in the water bearing horizon, enabling straight river-underground contact, has direct consequences on the underground flow regime too. 8,13.

The defence of the broader town area from long since repeated floods requires Sava high water regulation influencing in this way directly the underground water level on both river banks. Manifold motivated hydropower utilization in the town area, changing the waterlevel regime by its rising and stabilization, may act corrective on the underground water regime proved of greatest importance in town water supply. 8

It is of prarticular significance that rather extensive town area exploits for drinking purposes, by means of drilled and dug wells, its territorial water resources, still of good quality, from the alluvial reservoir on both river banks.

The progressive population growth and industrial expansion in town result with permanent water consumption and demand increase, but with the intensified pollution of waters simultaneously.

In such a way the necessity to protect waters from pollution, arises as a priority task in water resources development in Zagreb, especially with regard the influences of contaminated waters may have on the underground reservoir and its safe yield.

The above mentioned results with a fact that solving the town water resources management problems, so projects and designs need a complex consideration without partial solutions. That means that complicated definite solutions concernig the water supply of Zagreb area should be looked for in a complex city planning, power

utilization and water resources management regional solutions.

Further studies and search of final and lasting solutions of drinking, industrial and public needs water supply for greater Zagreb, ought to be oriented toward:

1) accomplishing the protective technical, economical and legal measures extending over the entire Sava r. alluvium as a pumping site, and

2) working out preparatory sudies enabling to determinate the capacity of this precious good guality water reservoir and its safe yield.

This work excerpts from the 1st targets group of problems only one, comprising the narrow scope of thermic contamination of waters, which but obligatory incorporates into the integral problem of conservation and protection of waters in the water management projects in Zagreb area.

HYDROGEOLOGICAL FEATURES

Numerous works, from 1948 (fragmentary) upto seventies, treat more or less concrete hydrogeological problems in this area. They have been founded on soil mechanical and hydrogeological interpretation of more than 2000 geomechanical, pedological, structural, piezometric & o.mostly temporary boreholes and wells. The basic deficiency of those attempts have been the insufficient depth of holes (rarely over 10 m), the density their at random selected locations, investigations oriented towards tasks and lack of more detailed systematization and analyses of the existing and newly obtained data, as well the lack of joint interpretation of the results 6,11.

Last ten years show more systematism in investigating of underground water regime by means of selecting observation wells, drilling supplementary deeper, structural and piesometric holes, constructing test wells on which long-term pumping test have been accomplished. Geophysical explorations in 1970 proved very usefull. Several prophiles with over 300 electrical resistivity sounds with Schlumberger electrode arrangements AB/2 = 140 m, covered the depths over 100 m (not yet proved by drilling) and were used in construction of isopach map of the water bearing horizon 6,11,13.

In this point of view in 1970 approved water management plan of Sava r. catchment basin, prover as advantageous contribution.

From g e o m o r p h o l o g i c a l standpoint the entire Zagreb area could be devided: - immediate Sava plain, and
- highlands encompassing it.

The highland close from N (the drainage basis of S slopes of Marija Gorica hills and Zagrebačka Gora) and from S (drainage basins of N slopes of Samoborsko gorje nad Vukomeričke gorice) the alluvial Sava r. plain that again could be divided in 3 separate unities: the area extending from Bregata to Podsused, from Podsused to Rugvica and from Rugvica upto Sisak. The width of the valley increases from W to E, being narrowest at Podsused; then the stream enters into relatively expanded plain of 15 km average width, flowing nearer to the left foot hill. From Rugvica to Sisak the right river bank has approximatly 10 km breadth (left bank not of interest to town targets and included in investigations sporadically). Scarcity of data available from defective investigations results with only informative conclusions, relative to this portion of plain.

L i t h o l o g i c a l l y highland consist of pretertiary (slates, slaty limestones, sandstones, limestones, marly limestones and dolomites) and tertiary layers (conglomerates, breccias, sandstones, sands, sandy clays, clayey and sandy marls multiple alternated).

Only limestones and dolomite are waterbaring in pretertiary formations, characterized by relatively productive springs, even thermal, while in the alternation of tertiary sediments prevail aquicludes.

H y d r o g e o l o g i c a l features of the hilly girdle rocks result with depression or contact mostly intermittend springs of low discharge rate developing subsequently in tipical torrents.

The actual results of geological and geophysical investigations display that the alluvial deposits consists of:

1) water bearing horizon

consisting of well graded gravels and sands its thickness varying from 5 to 100 m. In Podsused area varies from 6-16 m and consists of coarse gravels with intermediate strata and lenses of fine sized materials of lesser importance. From Podsused to Sloboda bridge the thickness increases from 5 to 10 m in average and farther eastwards saltatory upto 73,70 m in Čička Poljana (over 100 m measured geoelectrically, yet mot proved by drilling) These abrupt thickerring of the water bearing horizon have been explained by faulting activities during sedimentation periods, faults being detected seismically below recent sediments. Eastwards with thickness increase of the water bearing horizon, decreases the grain coarsiness, so that by Rugvica sands and fine gravel are dominant and clay lenses are more frequent.

That geomorphologic factors have been dominant and that changes in grain size are the logical consequence of abrupt gradient reduction of Sava r. by Rugvica is undoubted.

The information concerning the waterbearing horizon from Rugvica towards Sisak is partial only, defined by geophysical methods.

The continuity of the horizon,10-40 m in thickness,is noticeable on the NW, whilst towards SE, electrical resistiviti s decrease, that one explains with larger clay and sand particles share.

2) adjacent hanging wall

built up of sand, silty sand, silt and clays in various ratios, its thickness varying frow 0-5 m the Bregana-Podsused area, not exceeding 7 m in Podsused-Rugvica area, being mostly about 6 m farther east.

3) bottom horizon

made up of grey-blue and grey-green clays with occasional occurrences of eat and limestone concretio s. Below the clays,near Podsused drilled bore hole entered marl banatica at 10,6 m already. East of Rugvica, the fottom horizon has not been defined 6,11, 13, Rep 4 .

The results of numerous size analyses and of pumping tests on several locations 6,7,11,13 enabled to obtain the coefficients of permeability writhin sufficient accuracy range:

$1{,}0 \times 10^{-9}$ to $1{,}0 \times 10^{-4}$ m/s for top horizon

$4{,}5 \times 10^{-6}$ to $8{,}5 \times 10^{-1}$ m/s for waterbearing horizon

$4{,}5 \times 10^{-10}$ to $6{,}0 \times 10^{-8}$ m/s for bottom horizon

from grain size analyses, while from pumping test, for steady flow, computations based on Darcy´s law, gave:

$2{,}0 \times 10^{-3}$ to $1{,}2 \times 10^{-1}$ m/s

Coefficient of transmissibility computed after Ferris on 20 observation points was:

$1{,}0 \times 10^{-1}$ to $1{,}0 \times 10^{0}$ m/s

that corresponds to the permeability coefficient values 11 :

$1{,}8 \times 10^{-2}$ to $1{,}1 \times 10^{-1}$ m/s

From all previously mentioned one should come to the conclusion that, in geomorphological and hydrogeological point of view, the 50 km long and 2-15 km wide waterbearing alluvial

horizon along loth Sava r. banks, is one c o n t i n u o u s e n t i r e t y. Also, wilh respect to the rccasionally direct contact between the river bed and the waterbearing stratum, one should expect the river regime be directly connected w th the regime in the underground, as well the correspondent immediate influence of hydrometeorological factors of the adjacent catchment basins.

HYDROLOGICAL INVESTIGATIONS

Within the limits of this work one is not permitted but to a sketchy description of hydrographical features, although they are inseparable part of factors, equally valorized, which define this area from a water resources management point of view, especially with respect to the water quality which is the subject of this paper. The drainage basin, with exception of Sutla and Krapina r., NW Sava tributaries, ammounts to 1500 km2 in total, Sava plain -850 km2, highlands -650 km2. It is distinguished by violent shower intensities and by innumerable small water courses of rather diffuse pattern and uncontrolled regime. Floods are frequent, with noticeable sediment discharge 8 . Sava r. flowing through neighbouring Slovenia and through Zagreb area is highly polluted with industrial waste waters -without adequate proeessing. As all water courses in the N and W parts of the area mostly flow through dense populated districts, partly industrial zones, they are collectors of all possible dirt, and together with Sava, whose tributaries they are, risky endanger the quality of the ground water along both river banks. The lateral channels Lomnica and Odra r. receive the waters from N slopes of Vukomeričke gorice 8 therefore it might be expected they will not much affect the quality of underground water.

Within 10 years frame of systematical approacle to h y d r o l o g i c a l investigations, the greatest care, has been given to water level observations on numerous existing and newly erected observation wells. Todlay, the Republican Hydrometeorological Instetute observes the groundwater levels, twice a weeki on 23 wells and 70 piesometric holes, ground water temperatures on 52 localities as well. Sava r. is under observation since many years (temperatures since 1952) twice a day,other hydrometeorological terms after conventional standards. The municipal water works carry oat orderly water level, recordings on 106 observation wells in their pumping site areas, the temperature being recorded only sporadically 1 .

From water level recording data have been constructed groundwater contour maps relative to Sava r. maxima and minima, and therefrom the interrelations between Sava and ground water have been analized 1,2,4,11, Rep. 1,2 & 3 . Taking into consideration

that it is a question of momentaneous water level positions-changes are dayly, remarkable moreover during a one day only-the derived conclusions have merely relative significance, while the competent directions of grounduater flow, so as whether Sava r. and how many days in a year recharges the underground, or drains it, could be defined from duration curves.

Generally one should accept the direction of flow is, more or less, congruous to the well known manifestations of river bed meandering in flat pervious strata, thereafter almost parallel to the river, W-E on the left and WNW-ESE on the right river bank.
Using mathematical-statistics methods, one year period of daily records in 1966/67 has been elaborated 2, Rep 1, furthemore the two years period in 1966/68 3, Rep 2. Under the assumption of linear correlation, using the habitual methods, the correlation coeficients r 0,6 has been found on 55% of wells, out of 95 and interpreted as strong correlative relation, 0,6 r 0,4 on 26%, interpreted as good relation, while on the remaining, weak relation or none. The correlative relation decreases with the distance from the stream, increases but from W to E. The most distant wells with strong correlation are 1 km far from river, between Podsused and Sava bridge, 2 km from Sava b. to Jakuševac bridge and 3 km farther east. With respect to derived conclusion upon a strong interrelation between Sava r. and underground waters using the regression equations has been constructed the prognostic high ground water level contours map, relative to area of good correlation, and based on presumed Sava high water levels of 2 days duration, as long as it was necessary to record the reaction on rised Sava levels in the most part of hinterland.

On the basis of records on 60 selected observation wells in 1972, the multiple linear regression has been estimated and analized the mutual correlative relation between 1-3 neighbouring vells. The obtained coefficients of correlation are mostly beyond 0,9 and standard deviation within a range $\pm$ 25 cm, insinuating the very strong relation, thus the uniform groundwater regime 1, Rep 3.

WATER QUALITY

The role of water quality in Zagreb area has been already emphasised in the Intorduction:

- water is consumed for drinking purposes (inter alia)
- available water resources are subject to increasing pollution.

Resulting from hydrogeological relations and hydrological studies, that the alluvial groundwater reservoir is an integral

continuity, that direct, even very good connection exists between the regime of Sava r. and by it immediatly influenced regime of ground waters, the above accentuated role and significance of water quality becomes more valourous, so the introductory statement that final, asting solutions of Zagreb water supply, should be purposively oriented to implementation of adequate protective measures in Sava r. alluvium.

Already have been pointed out the drainage area factors, likewise the evident influence of surface water courses: torrents running down the slopes of highlands encircling the plain and Sava r. itself polluted to the utmost degree.

In order to support both the importance of water quality and the danger of contamination to which this area has been intensively exposed, as well as in order to get a clear idea of pollution grade and of conditions that stipulate it, it should have proved satisfactory in continuation, merely to recapitulate the polluters and their influence on ground water reservoir, exploited as drinking water by over one million people.

In addition to previously said the water resources management plan predicts on Sava r. construction of 3 low head power plants, in Podsused, Jarun and Rugvica, and yet we expect from those solutions positive effects only in the water management and hydrotechnical points of view, there should be given particular consideration regarding the water quality, having in mind that Jarun lake is envisaged as huge sport and recreation center, with over 20000 people capacity, heving in mind the former sanitary-technical faillure of artificial lake in this area.

Latest risk in sight are the thermo-nuclear plants located in Sava valley what will be further discussed.

S a v a r. remains further the principal collector of waste waters let out by growing chemical, pharmaceutical and textile industries to the west, the industry in the town itself, as well the chemical, oil, paper, and o. on the east, without any treatment, even regardless of enough precise legal regulations.

It was said enough about t o r r e n t s on urban territory. We may add that in lowlands portion, sereral have been turned into city waste water mains that often drain straight to Sava r., sure enough without treatment.

Taking into consideration that on over 800 km2 large area, the impervious cover is, here and there, considerable thin, or is missing at all, in natural or artificial way, particulary dangerous, especially for municipal pumping sites, including the

future ones, south of Sava r. are numerous inhabited areas. They are still of rural character with developed agricultural and live stock production, with insanitary garbage dumps, without convenient sewers.

The danger of the same character is the irresistible penetration of the town to the south, with indispensible accompanying projects, traffic arteries, depots, repositories, workshops, industrial plants, that along witto all negative consequences creates particular difficulties in defining and regular inspection of protective zones of long ago planned pumping sites.

The outright possible catastrophical peril are numerous gravel excavations on the west, but also in the narrower city territories. Pits in some places are up to 30 m deep uncovering the whole water-bearing stratum, thus opening the way to omniferious immediate pollution, dangerous to human health.

The thermic contamination of water is recently ever more rising the interst to reslarch, because the temperature factor is a very significant indicator of water quality. The temperature is one of most conservative characteristics of ground water. With respect to the insulating qualities of earth's crust, damping out the extreme temperature oscillations at surface, so that under ordinary conditions the groundwater temperature at a depth of 10-20 m are negligible, and according to the analises in USA they don't exceed the mean annual air temperature by 1-2°C 12.

It is well known that mean temperatures in open water courses and waters increase when pollution increses. The actual state of pollution of Sava r. in Zagreb area already causes the thermic contamination (there are imminent investigations on that). Taking into consideration that the temperature of Sava r. will be soon substantially raised, being utilized as cooling water in thermonuclear plants upstream of Zagreb, taking into consideration previously accentuated deductions concerning the steady and direct connection between Sava r. and underground, the immediate influence of Sava r. regime on the regime of ground water, finally, taking into consideration so far technical and practice unknowns about movement and eventual accumulation of heat in underground - in distinction from surface water courses, where, subject to large contact surface and considerable velocities,hot water will quickly evaporate and cool off, underground water moves through pore spaces in the soil with imperceptible velocity-on its way to pumping sites of Zagreb water works, then the temperature contamination of good quality drinking water resources in Sava r.alluvium becomes one of most essential water quality factor in Zagreb area.

This work is a pioneering attempt to analize the relation between the temperatures of ground water, Sava r. and air, to initiate systematical ground water temperature recording and to clear the way in setting up mathematical models concerning those mutual relations with an applicable aim to prevent the thermic contamination consequences in the underground reservoir.

METHODOLOGY AND INTERPRETATION

As was said a while ago, efforts have been made to find a suitable methodology to study the temperature regime of one region - Sava plain near Zagreb, using the existing data, to the one hand, and directing further efforts to the optimal definition of those relations, to the other hand, realizing their enormous significance to the quality of water, before all for drinking purposes. One should understand that this task was not a simple one, not an casy one. As distinguished from systematical,over 100 years lasting recording of air temperature and 25 years recording of Sava r., the groundwater temperature recording, twice a week, started in 1973 on 52 observation wells scattered over this area.

The mode of processing was following:

1. On the basis of logical supposition that temperature influence of Sava r. decreases with the distance from river, all observation wells have been grouped in 4 zones. First, including those upto 500 m far from river, the second, 500-1500 m, third, 1500-3500 m, and fourth, 3500-5500, supposing they appropriatly represent the temperature course in underground as a function of distance from the river (Fig.1).

2. The course of temperature in each observation well has been graphically represented during the total recording period, 1.7.1973.-30.6.1977., by each zone separately.

At first sight upon the graphically represented results of temperature records has displayed the necessity of s e l e c t i o n of the representative observation wells, regarding the accuracy and reliability of recordings, its frequency, site etc. The observing reliability is estimated, inter alia, pursuant to the departures from the expected course,in the choice of site the main criterion was to stick to the right river bank only, because of the city development plans, while the noticed blanks, not exceeding 4 months were filled out by means of correlation analysis and medium regression line:

$$y - \bar{y} = a(x - \bar{x})$$

$$a = tg\varphi$$

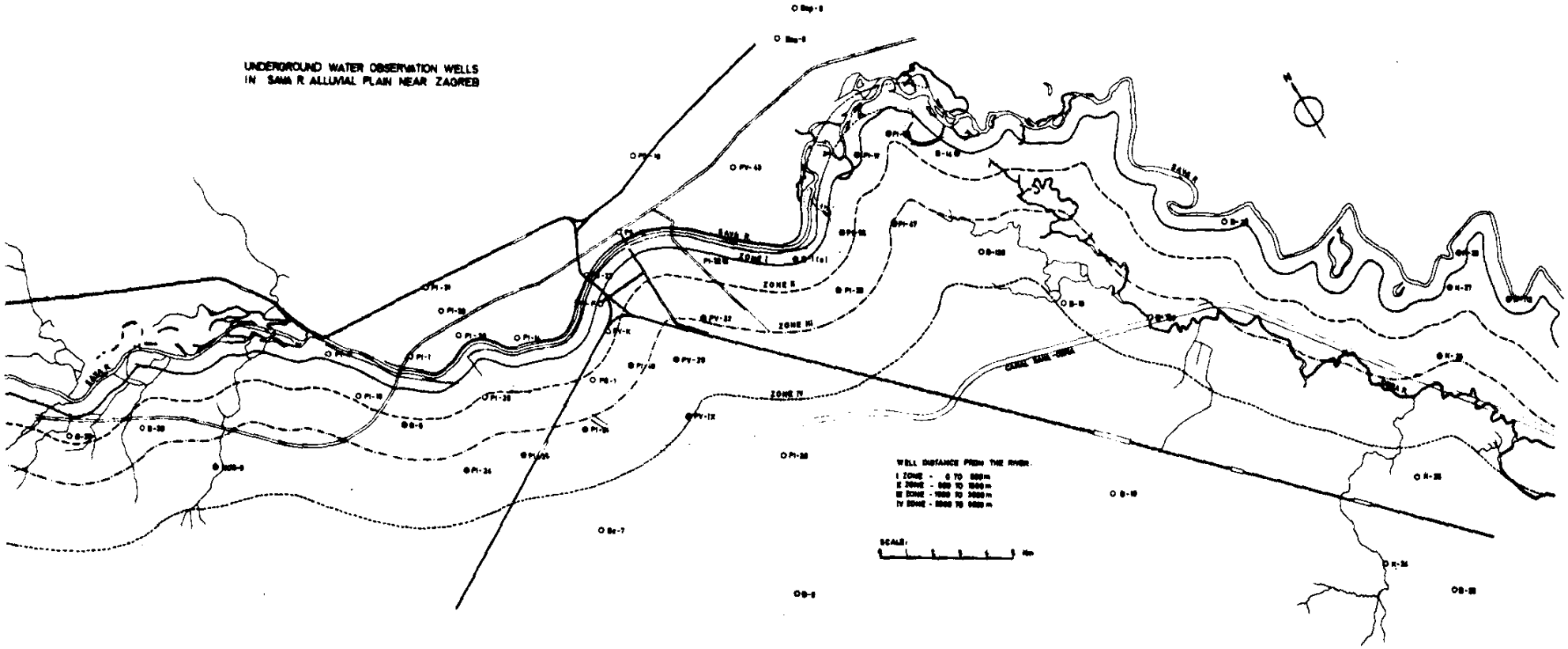
UNDERGROUND WATER OBSERVATION WELLS
IN SAVA R ALLUVIAL PLAIN NEAR ZAGREB
SAVA R.
ZONE I
ZONE II
ZONE III
ZONE IV
ZONE V
WELL DISTANCE FROM THE RIVER:
SCALE:

FIG.1

$$tg2\varphi = \frac{\frac{\sigma_y}{\sigma_x}\left(r + \frac{1}{r}\right)}{1-\left(\frac{\sigma_y}{\sigma_x}\right)^2}$$

where $\sigma_x = \sqrt{\frac{1}{N}\sum_{i=1}^{N}(x_i - \bar{x})^2}$ $\sigma_y = \sqrt{\frac{1}{N}\sum_{i=1}^{N}(y_i - \bar{y})^2}$ are standard

deviations for series x and y, and

$$r = \frac{\sum_{i=1}^{N}(x_i - \bar{x})(y_i - \bar{y})}{N\sigma_x \sigma_y}$$

the coefficient of correlation between a pair of observation wells chosen to show the strongest correlative relation,and if this was not a case,the well site has been eliminated. Finally,in this way have been adopted 21 observation representative wells, 4 in first, 5 in second, and 6 each in third and fourth zone (black circles in Fig. 1).

Taking into consideration that the statistical dimensions of ground water temperature dispersion are very small, I am of the oppinion that, at this stage of investigations, with such a selected statistical sample: too little number of recording years, the restricted number of observation sites and their disposition in long but narrow area, we could be at least partially satisfied, and that the way has been cut to continue investigation in order to suggest final, competent decisions about relevant temperature relations, that would serve as an indicator of water quality changes.

3. From the recorded temperatures on the selected wells of each zone there have been estimated the zonal mean temperatures for the 4 years period of observation (Fig. 2).

Regular and nearly identical course of temperatures, well expressed periodicity in each zone, almost simultaneous reaction to smaller and bigger changes in temperature, maxima and minima coincidence within a month time, are easy recognizable characteristics of those graphs. Amplitudes range from 8,5°C to 13,0°C in the first,

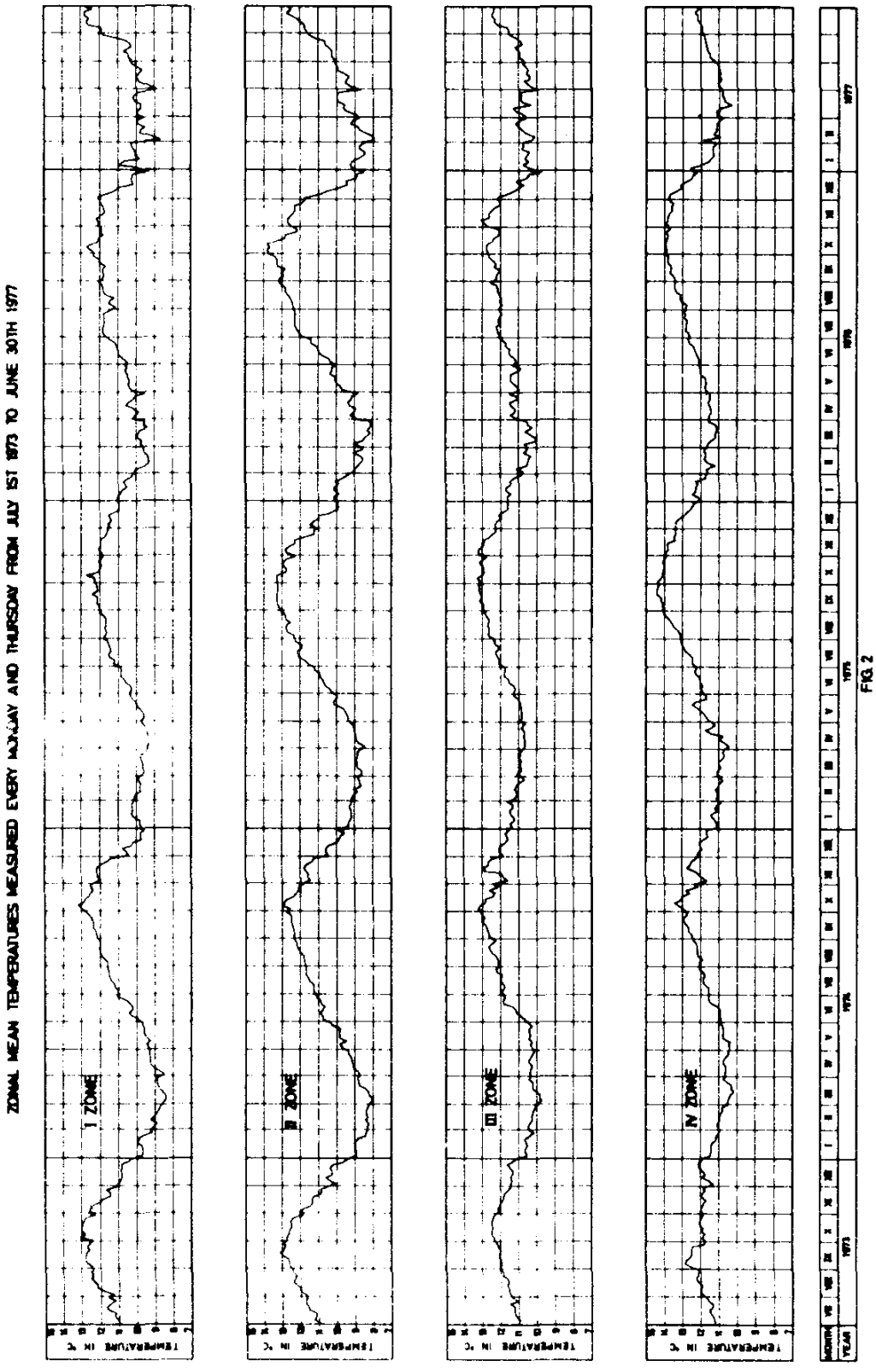
ZONAL MEAN TEMPERATURES MEASURED EVERY MONDAY AND THURSDAY FROM JULY 1ST 1973 TO JUNE 30TH 1977
I ZONE
II ZONE
III ZONE
IV ZONE
TEMPERATURE IN °C
MONTH
YEAR
1973
1974
1975
1976
1977

FIG. 2

fom 8°C to 14°C in the second, from 10°C to 13°C in the third and from 10,5°C to 14,5°C in the fourth zone. It should be expected that temperatures decrease with the distance from the river, thus apparently unexpected outcome in the second zone should rather be attributed to the sample deficiency, inadequate measurements or choice of representative observation sites, but to the alteration of local circumstances, in the fourth zone.

4. Out of zonal mean temperatures have been estimated the mean decade temperatures for the mean year of four years observation period (Fig. 3).

On the same figure are also shown the estimated four-year mean decade Sava r. and air temperatures. Beside the periodicity, well noticeable are: the interrelation between the temperatures of the 3 environment, the anomalousness of the temperatures in the second zone, and extremes retardation with regard to the Sava r. temperatures, too.

5. With regard to the obvious periodicity of the time series of the ground water, Sava r. and air temperatures, the harmonic functions have been defined by Fourrier series in a form:

$$X_t = \bar{X} + \sum_{i=1}^{3} \left(A_i \cos \frac{2\pi i}{p} t + B_i \sin \frac{2\pi i}{p} t \right)$$

$$A_i = \frac{2}{p} \sum_{t=1}^{p} \Delta x_t \cos \frac{2\pi i}{p} t \qquad B_i = \frac{2}{p} \sum_{t=1}^{p} \Delta x_t \sin \frac{2\pi i}{p} t \qquad \bar{x} = \frac{1}{p} \sum_{1}^{p} x_t$$

and With respect to the period length p = 36 (decades) and i = 1, 2, 3, all the equations have this common form:

$$X_t = \bar{X} + A_1 \cos \frac{\pi}{18} t + \ldots\ldots A_3 \cos \frac{\pi}{6} t + B_1 \sin \frac{\pi}{18} t + \ldots\ldots B_3 \sin \frac{\pi}{6} t$$

with Fourrier coefficients:

$$A_1 = \frac{1}{18} \sum_{1}^{36} \Delta X_t \cos \frac{\pi}{18} t \ldots\ldots\ldots\ldots \qquad B_1 = \frac{1}{18} \sum_{1}^{36} \Delta X_t \sin \frac{\pi}{18} t \ldots\ldots\ldots$$

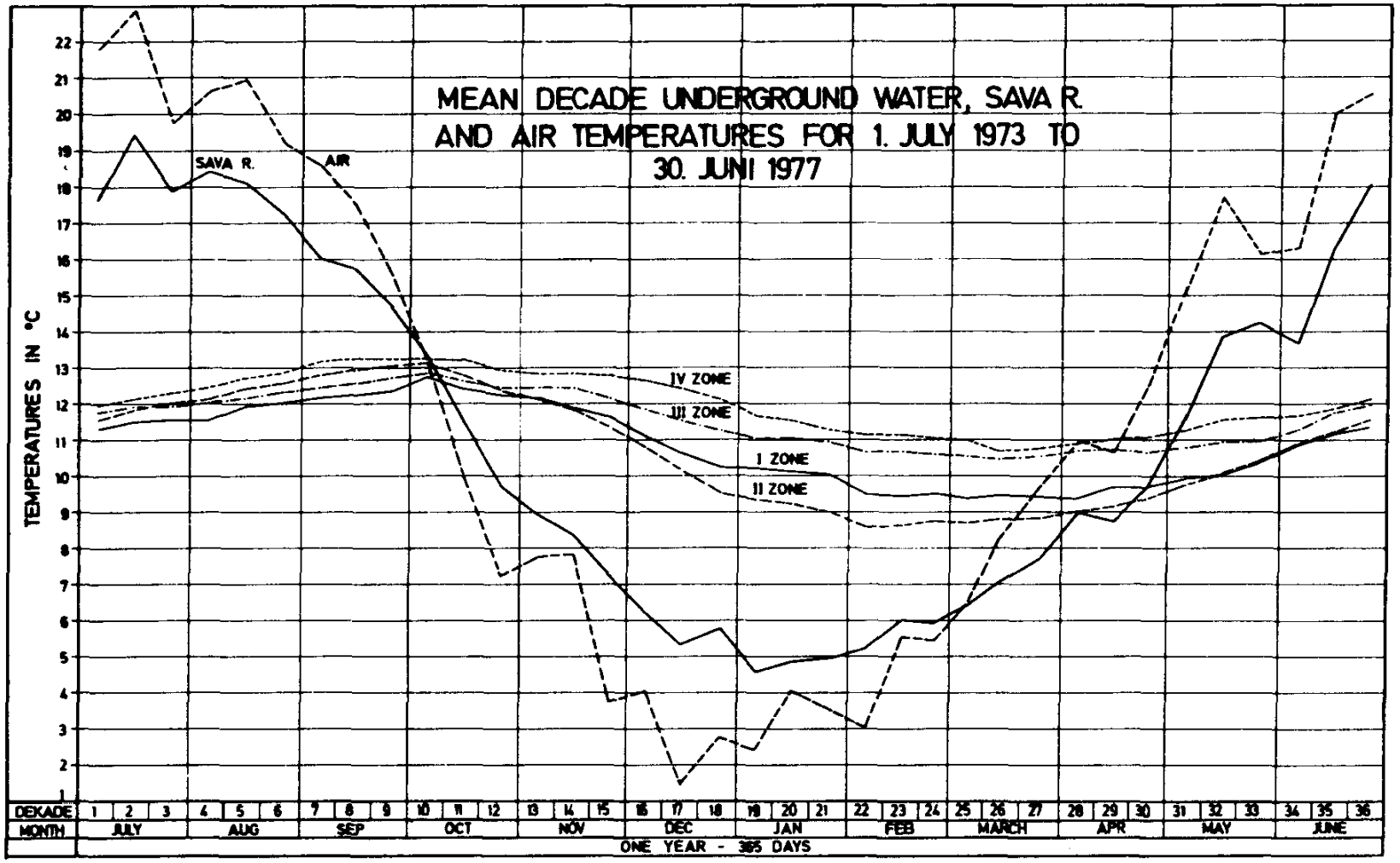
MEAN DECADE UNDERGROUND WATER, SAVA R.
AND AIR TEMPERATURES FOR 1. JULY 1973 TO
30. JUNI 1977
SAVA R.
AIR
IV ZONE
III ZONE
I ZONE
II ZONE
TEMPERATURES IN °C
DEKADE
MONTH
JULY
AUG
SEP
OCT
NOV
DEC
JAN
FEB
MARCH
APR
MAY
JUNE
ONE YEAR - 365 DAYS

FIG. 3

and have been shown of Fig. 4.

CONCLUSION

Although the investigations up to the present and the results obtained should be considered as preliminary, as a first step in studying the temperature relations in the Sava r. plain near Zagreb, not but without ambitions for generalization in similar conditions, one could consider that already have been proved:

- the presumed relation between the temperatures of the ground water, Sava r. and the air: underground water temperatures go along with temperatures of Sava r.

- the expected time shift is defined now more accurately. The maximal temperatures of ground water fall behind the maximal temperature of Sava r., 71 days in the first zone, 61 in the recond, 81 in the third and 71 days in the fourth zone, thus showing in the 1 st decade of September and first decades of october respectively. Similarly the minimal temperatures fall behind 65 days in the first, 40 days in the second, 65 days in the third and 59 days in the fourth zone, thus showing in the third february decade (by all means exceptionally, because of already mentioned anomalities in the second zone) and last decade of march respectively.

- the influence of the well distance (zone distance respectively) is evident. Generally it decreases with the distance from the river. Eventual departures are due to errors in the investigation, partly schould be but more analised.

- indirectly, this temperature interrelation veryfied the hydrogeological assumptions of the waterbearing stratum continuity, as well the immediate influence of Sava r. regime on groundwater, already being proved by hydrological analysis.

Such circumstances causing permanent increase of Sava water temperatures, suggest potential thermic contamination of underground water, raising thus the quastion of its exploitation after generally accepted drinking water criteria, and in addition to that, proving the aim and significance of this investigation for the water supply of Zagreb.

Finally, the results of hitherto investigations permit an optimism that resuming those researches we might come to the generalised solutions.

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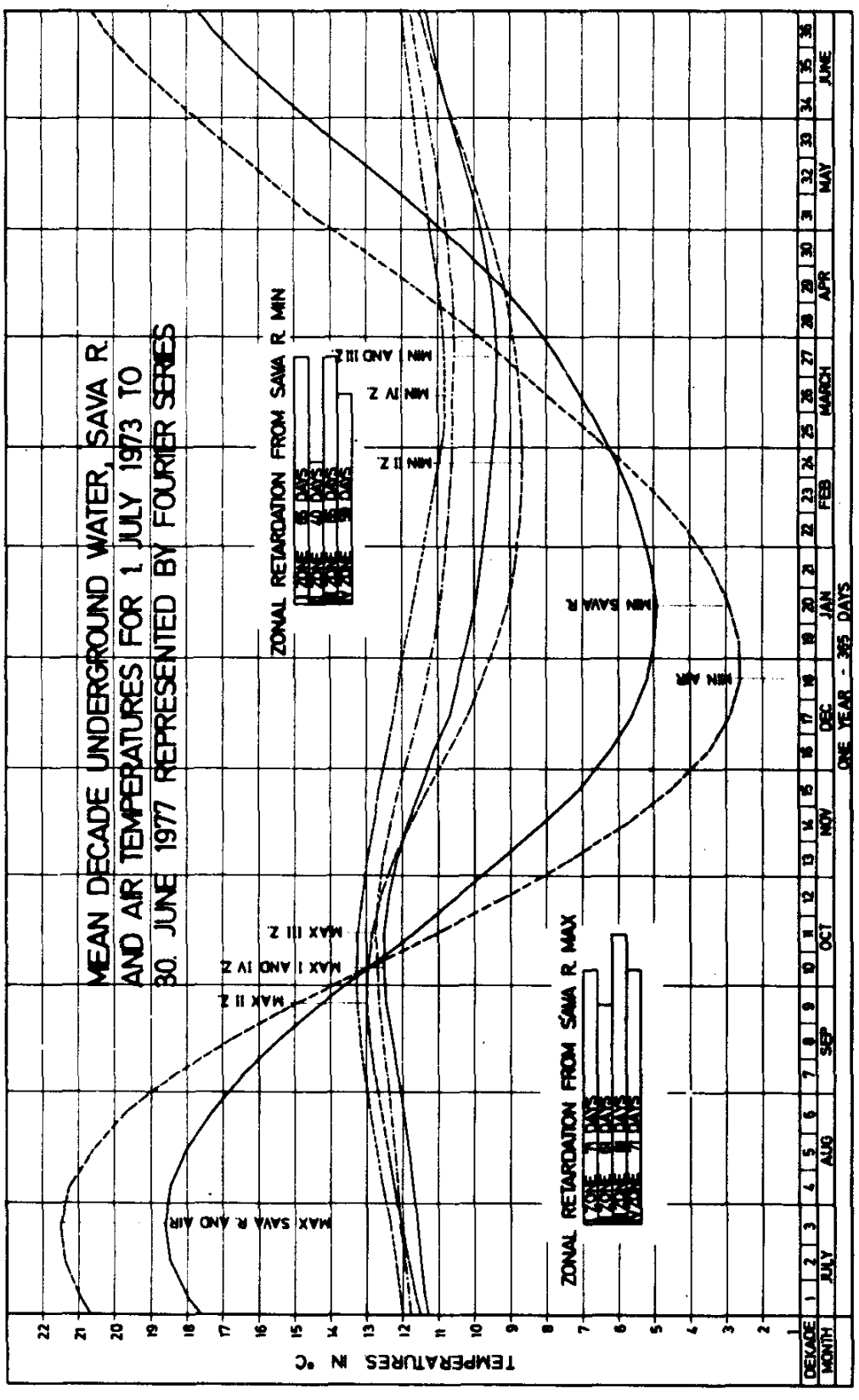
MEAN DECADE UNDERGROUND WATER, SAVA R.
AND AIR TEMPERATURES FOR 1 JULY 1973 TO
30 JUNE 1977 REPRESENTED BY FOURIER SERIES
ZONAL RETARDATION FROM SAVA R MIN
ZONAL RETARDATION FROM SAVA R MAX
MAX SAVA R AND AIR
MAX II Z
MAX I AND IV Z
MAX III Z
MIN II Z
MIN IV Z
MIN III AND I Z
MIN SAVA R
MIN AIR
TEMPERATURES IN °C
DECADE
MONTH
JULY
AUG
SEP
OCT
NOV
DEC
JAN
FEB
MARCH
APR
MAY
JUNE
ONE YEAR - 365 DAYS

FIG. 4

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INDICE DE AUTORES
INDEX TO AUTHORS
INDEX D'AUTEURS

	Página Page
ABRAO, P.C.	573
ADYALKAR, P. C.	001
AGRON, N. A.	173
AHCAN, R.	585
ANDERSON, D. L.	1111
ANDERSSON, A. C.	347
BAECKSTROM, L.	017
BAGDY, I.	359
BALCERZAC, A.	429
BANERJEE, S. P.	027
BARBERO SEVILLANO, J. P.	219
BAUER, G. E.	1125
BERGMANN, G.	347
BIANCHI, A.	825
BLACK, R. C.	045
BOBOK, E.	693, 883
BOGARDI, I.	789, 951
BONNIER, A.	707
BOON, H. F.	1095
BOUYAT, C.	719
CARLSSON, A.	373
CARLSSON, L.	017, 347
CARLSTEDT, A.	017
CARTWRIGHT, K.	061
CHANG, C.-K.	111
CHANG, C.-L.	111
CHATURVEDI, A. C.	1195
CIVITA, M.	1139
COCOZZA, T.	1139
CUADRA E IRIZAR, L.	1157
CUSTODIO, E.	085, 989,1169
DIAZ, E.	989
DU BOIS, H. L. E.	391
ECONOMOPOULOS, J. N.	463
ELLISON, R. D.	273
ERASO ROMERO, A.	1323
FAGES, R.	719
FERNANDEZ ALLER, R.	1013,1027
FERNANDEZ-RUBIO, R.	125, 741, 971,1207
FEUGA, B.	603
FILIPOVIC, B.	133
FILIPPI, L.	1139
FORTIER, D. H.	847
FOSTER, S. S. D.	409
FREITAS, M. H.	619
FRITZ, F.	227

GALOFRE, A. 989
GARCIA-CARO, P. 1225
GHIANI, M. 1049
GIZYNSKI, W. 429
GNATOWSKI, M. 429
GROWITZ, D. G. 153
GUISE-BROWN, F. H. 329
GUTIERREZ JAIMEZ, R. 751
HARR, M. E. 775
HEINEMANN, Z. 951
HEMMERICH, M. 863
HERVE, D. 1063
HOMOLA, V. 1233
HORNSTEN, A. 017
HUNT, C. S. 061
JACENKOW, B. 429
KAFRI, U. 173
KESSERU, Zs. 359, 637, 789
KOCSAYI, L. 359
KORGANOFF, A. 707
KOZA, J. 477
LAKSHMINARAYANA, V. 899
LLAMAS MADURGA, M. R. 183
LLOYD, J. W. 1249
LOPEZ VERA, F. 183, 293
LOUPANEC, M. 805
MARES, S. 199
MARINOS, G. P. 463
MARINOS, P. G. 441
MARJANOVIC, N. 1265
MARTINS, J. B. 825
MATOS, A. C. 825
MATURA, J. 477
MELIORIS, L. 207
MORELL EVANGELISTA, I. 219
MORENO BAS, E. 751
MORILLA, A. G. 847
MOSNA, J. 477
MUSSO, L. 1139
MUZAS, F. 501
NICOLAU, N. G. 463
OKUTSU, H. 1275
OLSSON, T. 373
PASQUET, R. 603
PAVLIN, B. 227
PEON PELAEZ, A. 183, 293
PERNA, G. 1139
PICOD, G. 719
PIETAK, A. 429
PRADO CALZADO, J. 1157

PRICE, M. 409
PROHASKA, S. 1265
PRUNA, V. 1139
PULIDO-BOSCH, A. 125
RAATS, C. M. S. 1095
RAJAGOPALAN, S. P. 899
RALSTON, D. R. 1081
RICCA, V. 863
RICCI, G. 933
RICHTER, R. 883
ROGOZ, M. 1291
SABHERWAL. R. K. 899
SANCHIS MOLL, E. 219
SARNAVKA, R. 1331
SASTRE, J. L. 1207
SCHIANNINI, P. C. 251
SCHMIEDER, A. 535, 553
SCHUBERT. J. PI. 911
SHYLIENGER, R. S. 027
SILAR, J. 199
SINGH, B. 657
SINGH, R. N. 675
SINGH, T. N. 657
SIPHER, D. J. 775
SOBOTHA, E. 553
SPILIOTIS, G. 237
SRINIVASAN, K. R. 001
STALKER, T. W. 251
STRASKRABA, V. 273
STREIFF, C. 933
SZIDAROVSKY, F. 798
SZILAGYI, G. 951
TAVODA, O. 207
THURMAN, A. G. 273
TONETTI, E. 1049
TREXLER, B. D. Jr. 1081
TRILLA ARRUFAT, J. 293
UCERO AOIZ, L. 1303
URIEL ROMERO, S. 1303
VAN DER HEIDEN, G. 1095
VELILLA, J. L. 501
WHITTAKER, B. N. 675
WIGHTMAN, D. 559
WILLIAMS, R. E. 1081
WILLIAMSON, W. H. 309
WOLMARANS, S. F. 329, 619
WU, HO 962
YAGUE BALLESTER, A. 741, 971

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