Research on the Design of Vertical Drainage for Sodic Solonchaks of the Ararat Plain

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The Ararat Plain is a typical mountain valley. It is situated between Mounts Great and Minor Ararat, on Turkish territory, and Aragats and Gegam, on Soviet territory.

The plain extends over 120 kilometres from the mouth of River Akhuryan to the Black Gorge (Wolf Gates) and has a maximum width of 20—25 kilometres in the central portion.

In the south it is bounded by the River Araks. Its southeastern boundary coincides with the administrative border between the Armenian Soviet Socialist Republic and the Nakhichevan Autonomous Soviet Socialist Republic. The Armenian part of the Ararat Plain has an area of 100,000 hectares. The plain within the territory under consideration varies in elevation from 800 to 1000 m above sea level.

The mean annual air temperature equals 11.3°C. The annual minimum temperature of —32° falls in January while its maximum of 41° in July—August. The soil temperature at a depth from 0.1 m to 2 m varies from 41° to 21° in summer and from 5° to 6° in winter. Diurnal fluctuations in soil temperature cease at a depth of 0.8 m in summer and at a depth of 0.4 m in winter. The Ararat Plain has a low precipitation rate. The mean annual precipitation equals 230 mm in the Arazdayan Steppe. Precipitation is distributed over the year as follows: 20 per cent in winter, 43 per cent in spring, 17 per cent in summer and 20 per cent in autumn. The Ararat Plain has a negligible snow cover. The average monthly relative humidity in winter varies within 70—80 per cent. In the space of half a year, the average monthly relative humidity drops to 33 per cent with an absolute minimum of 10 per cent. This results in a decreased moisture content of the soil to a level not exceeding 8 per cent.

The evaporation rate for the Ararat Plain equals 1,100 mm per annum while for land surface only 450 mm per annum. The evaporation rate (less the transpiration) is equal to 272 mm per annum with a ground water table occurring at a depth of 0.6 m on the average. At a depth of 3 m or so practically no evaporation occurs.

The soils of the Ararat Plain are chiefly represented by serozem, solonchak and solonetz alternating with irrigated brown desert soils.

The bulk of the precipitation falling over the vast basin of the upper Araks is transported into the Ararat Plain by fluvial discharge. Within the bounds of the Ararat Plain, the Araks receives tributaries on the left bank.

They include the Akhuryan, Sevjur, Kassakh, Razdan, Azat and Vedi. The tributaries are fed by ground, rain and snow waters, with the underground feed amounting to 45 per cent.

The Sevjur is a river that differs from the others in the basin by being exclusively fed from springs and having an abundance of water (with an annual average run-off of some 20 m^3 per second) and a highly stable discharge from season to season. It is the only river to form within the Ararat Plain having its source in the Aygerlich—Kulibeklin springs. The ground waters of the Ararat Plain mostly form in the submountain and mountain zone of the upper and middle Araks basin.

Hydrogeological test wells throughout the Ararat Plain have supplied evidence indicating that it has a water reservoir under it. The waters from the reservoir, deprived of free discharge, are almost completely expended within the plain's limits. The incoming underground water is supported by lacustrine-fluvial deposits. Ascending currents of underground water cause the springs to break surface. In depressions, where the ground water table comes close to the surface, ground water evaporates more intensively.

A clayey-loamy layer 5 to 20 m thick covers the uppermost horizon of a complex of quaternary deposits. Underground waters between lake clays and the covering horizon represent the first (low-head) aquifer. The waters occurring in lavas (under lake clays) are artesian (second aquifer) with an effective head in places being equal to or even exceeding 20 m.

The ground waters in the covering layer are mainly fed from the first aquifer. The amplitude of fluctuations of the ground water level in the Ararat Plain differs from place to place being equal to 1.5 m on the average. The minimum occurrence depth of the ground-water level in places may reach 0.5 m from the surface.

To prepare the way for designing drainage networks, a water balance chart has been compiled for the Ararat Plain. The chart accounts for the area of formation and discharge of ground waters. The Ararat Plain has also been divided into test areas according to the type of drainage best suited in each case.

Brief description of test area

The hydrogeological and soil characteristics of the Ararat Plain are many and varied. Therefore, to make the right choice of the type and parametres of drainage, much research and testing was required.

The biggest test area to accommodate vertical drainage is situated in the Arazdayan Steppe spreading over some 2,000 hectares. Some 25 wells have been drilled on that area, 15 of them already functioning on a regular basis while the remainder are still to be put into operation. Almost all the wells have observation holes and are equipped with run-off metres and deep-well pumps.

A detailed account of research that has been done in the Arazdayan Steppe for the past three-four years follows. The research was aimed at establishing the feasibility of vertical drainage in conditions of ground waters under head and with washing irrigations. It was also aimed at developing improved procedures for the calculation of a vertical drainage network in the complex hydrogeological environment prevailing in the Ararat Plain.

Changes in the level of ground and head waters in natural conditions during washing irrigations were under regular observation on an area of 500 hectares. Also under watch were changes in the mineralization of ground waters and the chemical composition of soils before and during washing irrigations and during soil reclamation.

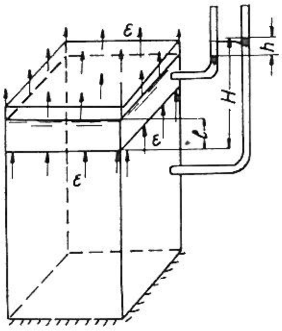

Fig. 1

Diagram of flow of water from low-head aquifer into covering layer

The desalinizing effect of vertical drainage before and during washing irrigations and during land development was subjected to detailed investigation on three plots (Fig. 1) each one hectare in area.

The evaporation of ground waters was determined by lysimetric tests, fluctuations in the level of ground waters and changes in the pressure of head waters. The data collected provided a basis for compiling a water balance chart for the 2000 hectare area now under drainage. The vertical drainage wells (imperfect) have a depth of from 35 to 60 metres and an average diametre of 0.4 m. All the wells are equipped with SP—12 deep-well pumps. The wells have a capacity varying from 40 to 60 litres per second. They have gravel filters. The porosity coefficient of their casing pipes equals 20 per cent. At their lower end the wells have sedimentation tanks up to 5 m long. The filter portions of the wells vary in length from 20 to 25 m. They are spaced 700 to 800 metres apart. On the average a well drains 70—80 hectares of land and lowers the ground water level by approximately 2.2 m.

Methods to determine hydrogeological parametres of aquifers for designing vertical drainage

As stated earlier, the ground waters in the covering horizon have a head feed. Therefore, the head (in the first low-head aquifer) and the water level being constant (in the covering layer), there will be an outflow of water which will in the long run completely evaporate (Fig. 1). Evaporation is determined by the formula:

$$\varepsilon = K_0 \frac{H - l}{l} = K_0 \frac{\Delta H}{l}, \tag{1}$$

where ε — is the evaporation of ground waters per unit of area,

K_0 — seepage coefficient of the covering layer in a vertical direction,

H — pressure head at the foot of the covering layer,

l — distance from the water level in the covering layer to the bottom of the covering layer.

When seepage is unstable, i.e. when H and l are temporal functions, the expression (1) will take the following form:

$$\varepsilon = K_0 \frac{\Delta H}{l} - \mu_0 \frac{\Delta l}{\Delta t} = \frac{K_0}{2} \left(\frac{H_{n-1} - l_{n-1}}{l_{n-1}} + \frac{H_n - l_n}{l_n} \right) - \mu_0 \frac{l_n - l_{n-1}}{t_n - t_{n-1}} \tag{2}$$

where the first term of the right part stands for the ascending current while the second one for the amount of water which is confined in the ground layer as distance changes with time Δt, μ_0 — water loss coefficient of the covering layer, H_{n-1} and l_{n-1} — readings of water level (counting from the foot of the covering layer) at moment of time t_{n-1}, H_n and l_n — the same at moment of time t_n.

Formulas (1) and (2) are also valid for pumping out (1). In this case, under certain conditions, the water level in the covering layer will be higher than the head in the first low-head aquifer.

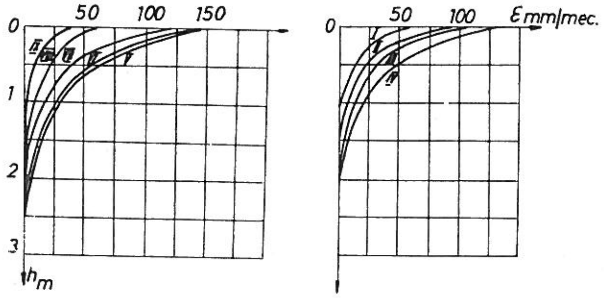

Fig. 2

Test curves for evaporation as related to ground water level

In the case of pumping out, in formula (2) the second term of the right part should be preceded by a minus. Then, the equation may be used for determining the value of the flow of water which feeds the underlying low-head aquifer from the covering layer.

With minor changes with time of values ΔH and Δl, the second term of equation (2) may be neglected. Then, the seepage coefficient for the covering layer may be determined by formula (1).

In cases of highly variable processes, the seepage and water loss coefficients of the covering layer should be calculated by formula (2). In order to do this, it is necessary to plot curves expressing the evaporation as related to the time and depth of the occurrence of the ground water level using lysimetric data (Fig. 2).

By substituting data collected in situ (method of least squares) into formula (2) it is easy to determine the numerical values of coefficients of seepage and water loss in the covering layer.

The coefficient of seepage, piezoconductivity, for the low-head aquifer, seepage being unstable, may roughly be determined by using Boussinesque's equation:

$$a\left(\frac{\partial^2 H}{\partial r^2}+\frac{1}{r}\,\frac{\partial H}{\partial r}\right)+\frac{\varepsilon}{\mu}=\frac{\partial H}{\partial t} \tag{3}$$

where ε is the flow of water which passes (per unit of area) from the covering layer into the low-head aquifer or in opposite direction. Value ε may be determined in the first approximation either by formula (1) or (2) and:

$$a=\frac{K\,m}{\mu} \tag{4}$$

Provided the initial and boundary conditions are $t = 0$, $H = H_e =$ const, when $t > 0$, $r \to r_0$, $2\,\pi r_0\,KH\,\dfrac{\partial H}{\partial r} = Q =$ const; when $t = 0$, $r \to \infty$, $H < M$, (where M is a constant value), equation (3) may be solved as follows [2]:

$$H = H_e + \frac{Q}{4\pi\,K\,m}\,R + \frac{\varepsilon\,t}{\mu} \tag{5}$$

where H is head in the first aquifer at any moment of time and any point of the layer, H_e being the same at the initial period of time, K, μ and m — respectively the coefficients of seepage, water loss and the thickness of the layer, t — time, while

$$R = E_i\left(-\frac{r^2}{4\,at}\right) \tag{6}$$

When

$$\frac{r^2}{4\,at} < 0{,}1; \qquad R \approx ln\frac{2\cdot 25\,\mathrm{at}}{r^2} \tag{7}$$

Formula (5) may be presented as follows:

$$S_t = \frac{Q}{4\pi\,K\,m}\ln\frac{2\cdot 25\,\mathrm{at}}{r^2}, \tag{8}$$

where

$$S_t = (H_e - H) - \frac{\varepsilon\,t^*}{\mu}, \tag{9}$$

or

$$S_t = A_0 + A\,lnt, \tag{10}$$

$$A_0 = \frac{Q}{4\pi\,K\,m}\ln\frac{2\cdot 25\,a}{r^2} \tag{11}$$

$$A = \frac{Q}{4\pi\,K\,m} \tag{12}$$

* to be solved by the method of successive approximation.

Marking experimentally obtained values t (decreases in water level S_{t_1}, S_{t_2}, S_{t_3} in the well at moments of time t_1, t_2, t_3 after pumping out began) on the X-axis and S-values on the Y-axis we shall obtain a straight line where value A_0 is determined from the graph, while values A are determined by the formula:

$$A = \frac{S_{t_2} - S_{t_1}}{ln\,t_2 - ln\,t_1} \tag{13}$$

where S_{t_2} and S_{t_1} stand for decreases in water level in the well at moments of time t_2 and t_1 after pumping out began.

Upon determining coefficients A_0 and A, it is rather easy to calculate coefficients of seepage, piezoconductivity and water losses by the formulas:

$$a = 0{,}44\,r^2\,e^{\,A_0/A} \tag{14}$$

$$K = \frac{Q}{4\pi\,\mathrm{mA}} \tag{15}$$

$$\mu = \frac{K\,m}{a} \tag{16}$$

where

$$\varepsilon \approx K_0 \frac{l - H}{l} - \varepsilon_{\text{evap.}} \tag{17}$$

$$\varepsilon_{\text{evap.}} = \varepsilon_0 \left(1 - \frac{m_0 - l}{m_0 - l_k}\right)^3 \tag{18}$$

In processing the data on decreases in water level obtained in the test well, r is taken to equal r_0(r_0 being the radius of the well); in processing the data on decreases in water level obtained in the observation hole, r is taken to equal r_1, where r_1 is the distance from the test well to the observation hole.

The above methods were used for determining hydrogeological parametres of the covering layer and the first aquifer.

The following data were obtained in these calculations. Seepage coefficient K_0 of the covering layer (6 to 10 m thick) changes from 0.3 to 0.8 m per 24 hours. The coefficient of water losses of this layer grows from 0.08 to 0.1. The seepage coefficient of the first aquifer grows from 5 to 10 m per 24 hours.

Methods to calculate vertical drainage to be installed in a two-layer seepage medium

Next we determined changes in head in the low-head aquifer and the ground water level in the covering layer. Let us suppose that the main current is passing from the covering layer into the low-head aquifer in a vertical direction.

For a problem with symmetrical axes (2), the influx of ground waters to the well may be determined from the system of equations:

$$\varepsilon - K_0 \left(\frac{h - H}{h - m}\right) = \mu_0 \frac{\partial h}{\partial t} \tag{19}$$

$$a \left(\frac{\partial^2 H}{\partial r^2} + \frac{1}{r} \frac{\partial H}{\partial r}\right) + \frac{K_0}{\mu} \left(\frac{h - H}{h - m}\right) = \frac{\partial H}{\partial t}$$

$$a = \frac{K m}{\mu} \tag{21}$$

Equation (19) accounts for seepage in the covering layer being a ratio between the rate of seepage and the true rate of the vertical movement of free surface. The second equation accounts for the elementary water balance in the lower low-head aquifer. Decoded below are the symbols used in equations (19) and (20):

h — depth of water in the covering layer counting from its foot,

H — head in the lower layer,

μ_0 — coefficient of water losses of covering layer,

μ — coefficient of water losses of head layer,

K_0 — coefficient of seepage of covering layer,

K — coefficient of seepage of head layer,

ε — the modulus to account for the feeding of covering layer as related to seepage and evaporation,

m — thickness of low-head aquifer.

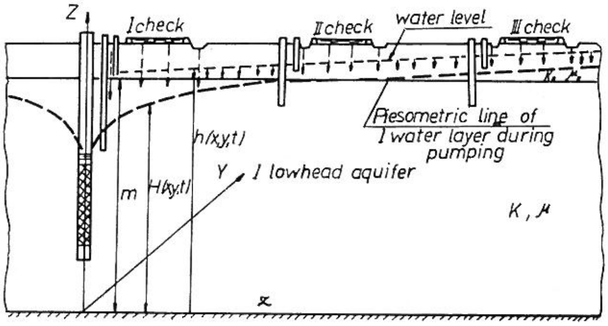

Fig. 3
Diagram to calculate vertical drainage systems

The planar seepage problem (Fig. 3) is solved by equations (19) and (20) which may be presented in the system of Decart's coordinates as follows:

$$\varepsilon + K_0 h_{av} \left(\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial y^2}\right) - K_0 \frac{h - H}{n - m} = \mu_0 \frac{\partial h}{\partial t} \tag{22}$$

$$K m \left(\frac{\partial^2 H}{\partial x^2} + \frac{\partial^2 H}{\partial y^2}\right) + K_0 \frac{h - H}{h - m} = \mu \frac{\partial H}{\partial t} \tag{23}$$

It should be pointed out that equation (22) which is intended for the overall solution of the problem, also accounts for seepage in a horizontal direction, whose value is however negligible as compared with that of seepage in a vertical plane.

Equations (19), (20) as well as (22) and (23) are solved at the following initial and boundary conditions:

when $$t = 0, \quad h = \varphi_1 (x_1 y), \quad H = \varphi_2 (x_1 y) \tag{24}$$

when $t > 0$, at the zone of feeding the aquifer

when $$h = \text{const}, \quad H = \text{const} \tag{25}$$

when $$t > 0, \; r \to r_0, \; Q = 2 \pi K r_0 H \frac{\partial H}{\partial r} = \text{const} \tag{26}$$

Differential equations (19) and (20) are precise expressions of seepage. The errors arising while calculating by the existing formulas mainly come from averaging or discarding some of the terms of equations (19) and (20), which is usually done to obtain a solution to the problem in the analytical form.

At present, the precision of solutions to equations (19) and (20) may be improved by using a computer. The application of computers becomes all the more necessary in calculating vertical drainage to be installed in very complex hydrogeological conditions. Besides, computers are essential in calculating optimum parametres for drainage systems.

Here is a brief account of methods for the solution of equations (22—24) on analogy computer USM—1 (general-purpose grid-type).

According to LIEBMANN [3], equations (22) and (23) may be presented in an ultimate differential form after the discretisation in time and space of a continuous seepage process as follows:

$$\varepsilon \Delta X \Delta Y + K_0 (h_{tn-1} - m) \left[\frac{\Delta Y}{\Delta X} (h_{1tn} - 2h_{tn} + h_{2tn}) + \right.$$

$$\left. + \frac{\Delta X}{\Delta Y} (h_{3tn} - 2h_{tn} + h_{4tn}) \right] - \frac{K_0 \Delta X \Delta Y}{h_{tn-1} - m} (h_{tn} - H_{tn}) =$$

$$= \mu_0 \frac{(h_{tn} - h_{tn-1})}{\Delta t} \Delta X \Delta Y \tag{27}$$

$$Km\left[\frac{\Delta Y}{\Delta X}(H_{1tn}-2H_{tn}+H_{2tn})+\frac{\Delta X}{\Delta Y}(H_{3tn}-2H_{tn}+H_{4tn})\right]+ \quad (28)$$

$$+K_0\frac{\Delta X\Delta Y}{h_{tn-1}-m}(h_{tn}-H_{tn})=\mu\frac{H_{tn}-H_{tn-1}}{\Delta t}\Delta X\Delta Y,$$

where H_{t_n} and $H_{t_{n-1}}$ are heads in the centres of block in the low-head aquifer at moments of time t_n and t_{n-1}, H_{1t_n}, H_{2t_n}, $H_{3\,t}$ — heads in the centres of adjacent blocks, h_{tn} and h_{tn-1} ground water levels in the centres of blocks of the covering layer at moments of time t_n and $h_{tn-1}, h_{1t_n}, h_{2t_n}, h_{3t_n}, h_{4t_n}$ — ground water levels in adjacent blocks, Δx, Δy — steps on coordinates, x, y — planar dimensions of blocks, t — time step.

According to the electrical diagram presented in Fig. 4 the equation of electric current balance at the nodal point of a grid of resistors may be written as follows:

$$\frac{V_3-V_2}{R_x}+\frac{V_1-V_2}{R_x}+\frac{V_4-V_2}{R_y}+\frac{V_5-V_2}{R_y}=\frac{V_2-V_t-\Delta t}{R_t} \quad (29)$$

where V is an electrical potential, R — a resistor. Equations (27), (28) and (29) are analogous.

Values $Km\,\frac{\Delta y}{\Delta x}$ and $\mu\,\frac{\Delta x\,\Delta y}{\Delta t}$ in equations (27—29) correspond to values $\frac{1}{R_x}$ and $\frac{1}{R_t}$ in equation (29), i.e. the process of variations in voltage on the electrical diagram is analogous to the process of variations in the level and head of ground water for unstabilized planar seepage. By comparing respective values in equations (27—29), it is possible to obtain a similar scale for resistors (4) as a ratio of their values in situ to their simulated values on the model, namely:

$$\alpha_R=Km\frac{\Delta y}{\Delta x}R_x=Km\frac{\Delta y}{\Delta x}Ry=\mu\frac{\Delta x\,\Delta y}{\Delta t}R_t \quad (30)$$

Hence the simulated performance of the resistors may be presented as follows:

$$R_x=\alpha_R\frac{\Delta x}{K\,m\,\Delta y} \quad (31)$$

$$R_y=\alpha_R\frac{\Delta y}{K\,m\Delta x} \quad (32)$$

$$R_t' = \alpha_R \frac{\Delta t}{\mu_0 \Delta x \Delta t} \tag{33}$$

$$R_t = \alpha_R \frac{\Delta t}{\mu \Delta x \Delta y} \tag{34}$$

$$R_z = \alpha_R \frac{h_{tn} - m}{K \Delta x \Delta y} \tag{35}$$

where R_x, R_y and R_z are components of resistances on coordinate axes; R_t' and R_t — resistances of the respective layers which are proportional 'to the pitch of change in the process in time Δt.

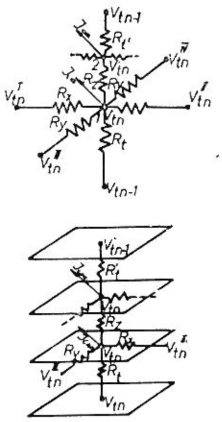

Fig. 4

Electrical diagram for a two-layer seepage medium

The simulated potential is analogous to head h (or H) in situ. Therefore scale α_h (or α_H) is defined as the ratio of the difference of simulated potentials to variations in head in situ Δh (or ΔH), i.e.

$$\alpha_V = \frac{\Delta V}{\Delta h} \tag{36}$$

The simulated power of current is analogous to the Q-value in situ, therefore the scale of run-off is defined as a ratio of the power current to the run-off:

$$\alpha_i = \frac{I}{Q} \tag{37}$$

$$\alpha_i = \frac{I}{\varepsilon_{\text{evap}} \Delta x \Delta y} \tag{38}$$

The simulated process on an USM – 1 computer, i.e. the redistribution of potentials at definite points on the simulated model, occurs almost instantaneously. Therefore no time data are fed into the computer while simulating coefficients α_v and α_i which are conveniently expressed in relative units.

The location of the test well is simulated in accordance with the division of the seepage area into separate blocks (Fig. 5).

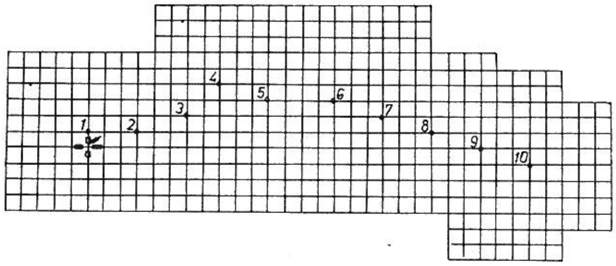

Fig. 5

Planar distribution of vertical drainage and division of seepage area into separate blocks

Resistors (31—35) are installed in the grids of the computer to obtain a discrete model of the given seepage area.

Definite potentials which are analogous to the corresponding heads in situ are installed at the boundary points in the model.

The simulated model is calculated on the following principle.

The simulated seepage area (upper covering layer and lower low-head aquifer) is divided into a network of blocks identical in size in the plane.

At the given value of $R_x = R_y$ (the USM—1 computer being a grid of active resistors: $R = 1000$ ohms) and at the given hydrogeological parametres of the layers, the scale of resistance α_R is determined by formula (30). For the given time step, according to the α_R value obtained by formulas (33) and (35), resistances R_z, R_t', R_t are determined (where R_z is the hydraulic relationship between layers 1 and 2 in a vertical direction, while R_t' and R_t stand for time resistances whose values are determined after each step by formulas (33) and (35).

The initial chart of hydroisohypses is drawn up according to the initial value for the evaporation of ground waters. The initial values of ground water levels and heads are fed into the computer through the upper terminals of the resistors.

The potential for the next moment of time $t + \Delta t$ (wells being in operation) is obtained by measuring voltages at the nodal points of the grid of resistors. After the first step, the evaporation and the corresponding power of electric current are determined by formulas (18) and (35), while resistances are R_t' and $R_t + R_z$ by formulas (33) and (35).

The obtained potentials are again fed into the computer (as initial conditions for the next time step $t + 2\,\Delta t$) at the terminals of the resistors, while the power of electric current in the well and the potential at the boundary of the seepage area remain the same. The calculation is thus continued throughout the given period of time.

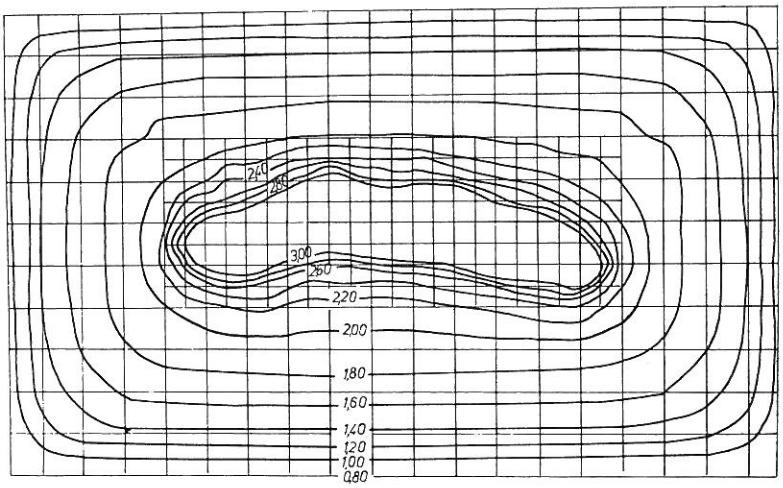

Fig. 6

Hydroisohypses for ground waters after 60 days of pumping from 10 vertical wells

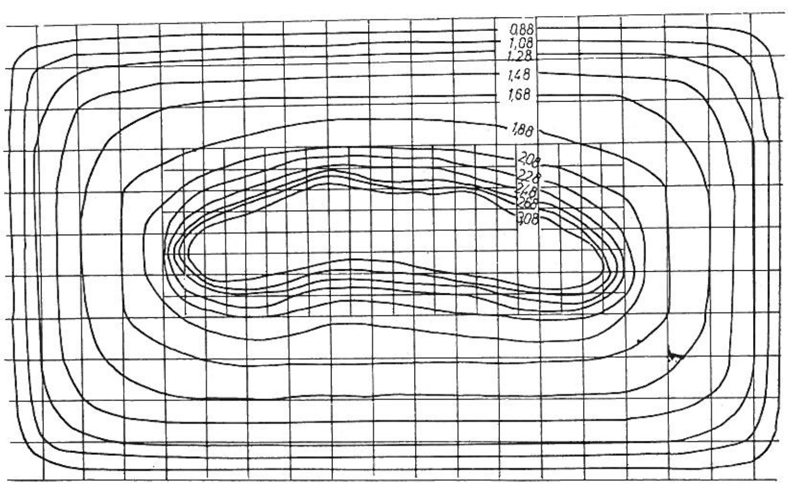

Fig. 7

Piezoisohypses for head waters after 60 days of pumping from vertical wells

The conversion of the data collected on the simulated model to the parametres in situ is carried out by means of the relative coefficients mentioned above.

The results of calculations are presented in the form of hydroisohypses for ground and head waters for different moments of time, regimes of pumping and the distribution of wells.

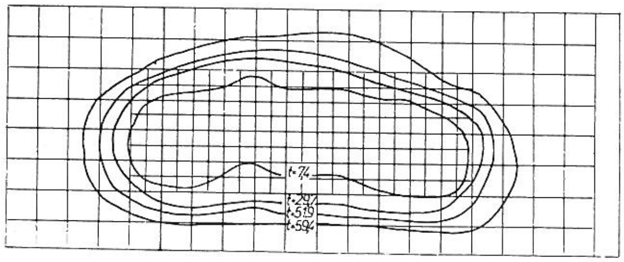

Fig. 8

Diagram of zero difference in pressure head at different periods of time

Shown in Fig. 6 and 7 are the results (obtained by Post-graduate G. MHITARIAN) of calculations of a system of wells whose planar distribution is presented in Fig. 8.

According to data presented in Fig. 6, the ground water level in the covering layer is determined while the water is continuously pumped from 10 wells for 60 days running. During calculation, the ground water level was taken to occur at a depth of 0.8 m from the earth's surface at the initial moment of time.

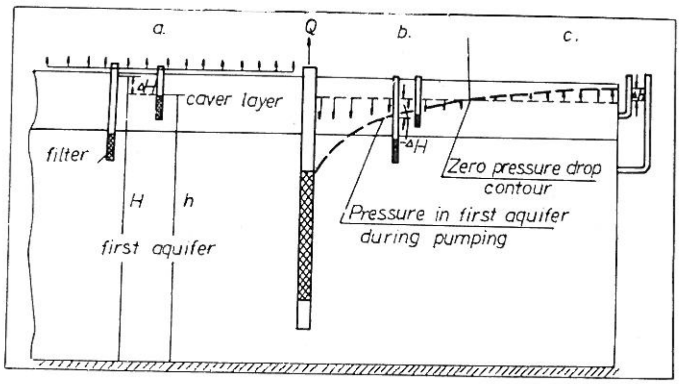

Fig. 9

Zone of descending and ascending currents while pumping out. a) Ascending flow (evaporation) before pumping. b) Descending flow zone during pumping. c) Ascending flow zone during pumping

According to data presented in Figs. 6 and 7, it is easy to obtain the boundary dividing the zones of the descending and ascending currents. This boundary is expressed as a locus of points at which the head in the first aquifer is equal to the height of the ground water level, i.e. the difference in pressure-head is equal to zero. The distribution of these points is easily determined by superimposing Fig. 6 on Fig. 7. To be sure, in conditions of more prolonged pumping the isoline with a zero difference will move towards the outer normal. As a result the zone of descending current will increase (Figs. 8. and 9).

The process of movement of the isoline with a zero difference in pressure head is of asymptotic nature, therefore, the drainage of an area by one well has to be founded on technical and economic calculations when it comes to practical work.

Many other problems can be solved on the basis of the obtained hydro-isohypses for ground waters and piezoisohypses for head waters. Thus, it is possible to determine the vertical velocity of decreasing ground water level in the covering layer, the effect of the intensity of washings and irrigation on changes in the ground water level in the covering layer, the height of the ground water level in the covering layer as related to the pumping out regime. And lastly, it is possible to determine the optimum number and distribution of wells by the method of variant calculations.

The method for calculating vertical drainage in a twolayer seepage medium with resort to a computer should be regarded as the principal one and is recommended for designing large scale systems of vertical drainage.

In the absence of suitable computers for preliminary calculations of smaller vertical drainage systems, one has to make the best of Bochever's solution. According to the diagram presented in Fig. 3, the calculating formula may be expressed as follows (2):

$$H = h\left[1 - \frac{\varepsilon}{K_0}\left(1 - \frac{\mu}{\mu_1}\right) - \frac{Q_3\,\mu_0\, e^{-\frac{r^2}{4at}}}{4\pi\, Km\,(K_0 - \varepsilon)\, t}\right] \tag{39}$$

$$a_1 = \frac{Km}{\mu_1}\left(1 - \frac{\varepsilon}{K_0}\right) \tag{40}$$

$$\mu_1 = \left(1 - \frac{\varepsilon}{K_0}\right)\mu + \mu_0 , \tag{41}$$

where H — head in the first aquifer,

h — ground water level in covering layer,

K_0, K — seepage coefficients for covering layer and first aquifer,

μ_0, μ — coefficients of water losses from covering layer and first aquifer,

ε — evaporation intensity, which remains constant whatever the depth of ground water level,

t — time,

r — radius at any point of aquifer,

m — thickness of first aquifer,

Q — capacity of the well.

The formula is applicable only on condition that:

$$t \geqslant \frac{(3 \div 5)\, \mu_0\, h_{av}}{K_0\, \varepsilon}\,, \tag{42}$$

If $\varepsilon = 0$, $t \geqslant (5 \div 10)\, \frac{\mu\, h_{av}}{K_0}$, the solution to the problem may be written as follows:

$$h = h_e = \frac{Q}{4\pi\, Km}\, E_i \left(- \frac{r^2}{4\, a_1\, t} \right),$$

where h_e is the ground water level in the covering layer at the initial moment of time, $E_i \left(- \frac{r^2}{4\, a_1\, t} \right)$ is an integral exponential function.

Performance of vertical drainage as related to desalinizing effect

It has already been stated that the parametres of drainage laid out in the first low-head aquifer with a view to land improvement should not be determined by draining effect alone. When use is made of combination drainage and when the salts dissolved by washing irrigations are not removed with sufficient intensity by temporary horizontal drainage, the parametres of vertical drainage should be determined by the salinity regime of the ground water and the soil.

To evaluate the desalinizing effect of vertical drainage, special tests were made on three plots by Postgraduate S. Mkrtchian in the Arazdayan Steppe.

The test plots each 0.65 – 0.8 hectares in area were located across from the initial, middle and terminal portions of a depression. The depression was forming as water was being pumped continuously from a deep well. The effective range of the well reached 500 m owing to the topography of the depression where the ground water level had dropped to a depth of 2 m from the surface.

Horizontal drains up to 1.5 – 2 m deep were laid along the boundary of all the plots at 100 m spacage.

All the plots had observation holes to evaluate chemistry and water head in the covering layer and low-head aquifer before, during and after washing irrigations as well as during land development for the purpose of farming.

The soda solonchaks were washed with a one per cent solution of sulphuric acid in order to obtain good results in a comparatively short period.

The total mineralization of water pumped from the test plot was equal to 1.4 grams per litre while the content of CO_3, HCO_3, Cl and other components was not unduly high.

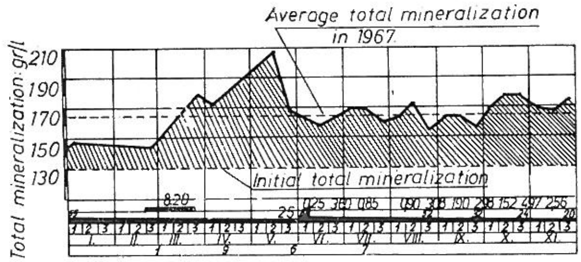

Fig. 10

Changes in the duration of mineralisation in pumped out water while washing a test plot

The total content of salts in a meter deep soil layer before acidification was equal to 0.5 %, or 60 – 70 tons per hectare, with a high content of soda, chlorine and other toxic ions.

Salts were mainly concentrated in the upper 20 – 40 cm layer of the soil on the test plot.

The amount of sulphuric acid necessary for washing irrigations was determined in accordance with the procedure developed by the Research Centre of Soil Science and Agrochemistry of the Armenian Ministry of Agriculture.

The first and second washing irrigations of the soil on the test plots were done using a one per cent solution of sulphuric acid while simultaneously pumping water from the well. Subsequent washing irrigations were performed using irrigation water free from sulphuric acid. On each washing irrigation, the test plots were flooded to a depth of 20 – 30 cm. The intervals between washing irrigations ranged from 3 to 7 days depending on the seepage capacity of the covering layer.

Whenever the time gap between the acidification of soil and the use of irrigation water lasted several months due to technical reasons, the formation of salts was observed on the soil surface. This suggests advantages of incessant washing irrigation.

The duration and number of washing irrigations were decided upon according to the composition of the meter deep layer of the soil. According to the procedure developed by the Research Centre of Reclamation and

Agrotechnics, a soil may be regarded as adequately washed when the residual content of toxic components (CO_3, HCO_3, Cl, etc.) does not exceed 0.01, 0.06 and 0.02 per cent respectively. On the other hand, the total content of salts in alkaline (soda) soils should not surpass 0.15–0.20 per cent.

Methods for staging experiments aimed at the evaluation of the efficiency of vertical drainage are described by LIEBMANN [5]. This paper quotes only some of the data elucidating the desalinizing effect of vertical drainage.

Illustrated in Fig. 10 is the total mineralization of pumped water before, during and after the washing irrigation of a test plot.

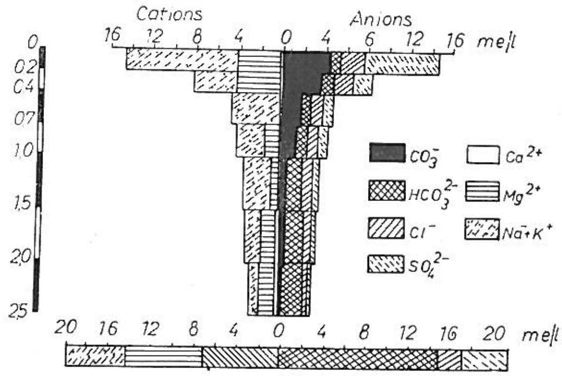

Fig. 11
Chemical composition of soil before washing

Plot 2 is situated at 300–400 m from well 1 (vertical drainage). By consulting the legend in Fig. 10 it is fairly easy to determine the regime of washing irrigations and the performance of pumps. According to the data presented in Fig. 10, in 2–3 days after the washing (which was started on February 20), the mineralization of pumped water increased reaching a maximum in 20–40 days after the washing was completed. Further on, the mineralization of pumped water either decreased or increased depending on the regime of pumping and the intensity of washing. Washing irrigations on nearer (100–200 m) and farther (500–600 m) test plots gave similar results. Changes in the content of such components as CO_3, HCO_3, and Cl in pumped water were also examined.

Shown in Figs. 11 and 12 is the content of soluble (1 : 5 soil to water) cations and anions in the soil before and after washing irrigations.

Judging by the data presented in Fig. 12, after washing irrigation, the content of toxic components in the soil still exceeded the permissible limit, while the total content of salts, vice-versa, tended to increase as against initial conditions. After washing, exchangeable anions and cations (SO_4, Ca, Na) come into solution causing sodium and calcium salts to appear in abundance. Yet, this high salt soil is suitable for farming on condition that the process of washing is maintained continuously throughout the draining period. Experiments show that in order to completely wash a meter layer of soil it is necessary to have 90–120 tons of sulphuric acid and 40–50 thousand m³ of water per hectare. In order to remove one ton of salts, 360—400 m^3 of water are required.

During washing, roughly 15—20 per cent of salts are removed through temporary horizontal drains. The remainder of salts are removed by vertical drainage. In this series of experiments it was observed that well No. 1 removed 4—5 tons of salts per 24 hours, which greatly depends on the technique and the scale of washing and also the capacity of the well.

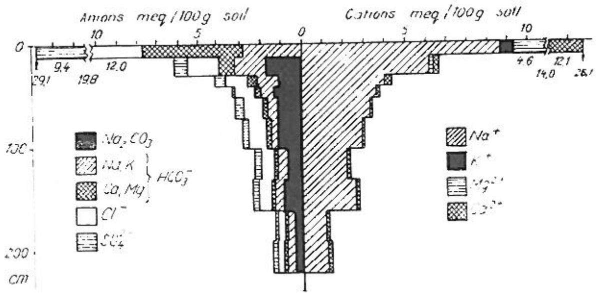

Fig. 12
Chemical composition of soil after washing

It was also shown that well No. 1 drained about 70 hectares in 3—4 months. The well was operated continuously and had a capacity of 60 litres per second.

Given these conditions, it should be possible to desalinize a meter-deep layer of soil and decrease the mineralization of ground waters to the permissible concentration in 6 years. These data are however still to be verified by further research.

In conditions of head feeding of ground waters, it is feasible to apply vertical drainage as the principal method for removing salts by washing. Indeed, vertical drainage tends to decrease the head of the low-head aquifer still further and thus makes it possible to use the head of irrigation water to obtain descending currents.

Both vertical and horizontal drainage create nonstandard conditions for washing irrigations, since the depression curve in the effective range of a well has different elevations. Therefore at different distances from the well, different conditions for the seepage of washing waters are likely to arise (Fig. 9). At the terminal area of the depression where the ground water level is comparatively high and the surface almost horizontal, the mineralization of water after washing irrigation takes longer to decrease. That is why, after the washing period is over, it is necessary to keep the drainage in continuous operation so as to maintain a descending current over the depression surface.

Vertical drainage, as a desalinizer of soil, is pretty complex to design since it is expected to ensure movement of water through the aeration zone to the complete saturation zone and at such velocities it will cause the dissolved salts to be removed in the running water thus enabling their removal by drainage.

In designing vertical drainage, it is necessary to take complete stock of local soil and hydrogeological environment in order to correctly determine the optimum depth of drainage and length of its seepage portion. If the ground water level drops too low, too much water may be required to maintain a continuous descending current, and on the contrary, if the ground water level is not too low, there is a danger of secondary salinization. Therefore it is necessary to maintain such a washing regime as to always have a descending current throughout the depression surface. Then upon irrigation, the inflow of salts into that current will be lower than their outflow through vertical drainage.

The methods for evaluating drainage as related to its desalinizing effect are still far from perfect and more profound research both in situ and the laboratory is needed to improve it radically.

The methods for evaluating vertical drainage for drainage purposes only, regardless of the salinity regime, have already been thoroughly studied in so far as hydrodynamics are concerned. Now that up-to-date digital and analog computers are available, the precision of the existing evaluating methods can be improved.

References

[1] ALEXANDRYAN, V. V.: Designing and calculation of vertical drainage. Yerevan. 1957.

[2] ANANYAN, A. K. & MKRTCHYAN, S. M.: The tests on washing out of Arazdayan steppe saline soils. Sbornyk trudov nauchnoi konferentsii Zakavkazskikh sel'skokhozyastvennikh Institutov. Yerevan. 1966.

[3] ARAVIN, V. J. & STREL'TSOVA, G. D.: About investigation of planned unsteady filtration, by means of integrators. Izvestia Vsesousnogo nauchno-issledovatel'skogo Instituta Guidrotekhniki, Moscow—Leningrad. **77**. 1964.

[4] BOCHEVER, F. M.: On the calculations of wells layed in deep head and aquifer strata of artesian basins. Trudiy Instituta VODGEO, **6.** Hydrogeology. Moscow. 1964.

[5] LIEBMANN, N.: A new method of electrical analogy to compute the problems of the unsteady heat conductivity. Sbornyk perevodov „Mekhanika", **3.** Moscow. 1957.