UNITED NATIONS EDUCATIONAL, SCIENTIFIC AND CULTURAL ORGANIZATION

UNITED NATIONS ENVIRONMENT PROGRAMME

UNEP

HYDROGEOLOGICAL PRINCIPLES OF GROUNDWATER PROTECTION

Handbook of Scientific Methods

VOLUME I

USSR MOSCOW

The monograph "Hydrogeological Principles of Groundwater Protection" has been written by an international team of authors and prepared for publication by the Centre of International Projects of the USSR State Committee for Science and Technology in accordance with the program of the UNEP/UNESCO/USSR project "Protection of the Lithosphere as an Environmental Component".

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28 NOV. 1988

PREFACE

Activities with the aim of preparation of the two-volume monograph "Hydrogeological Principles of Groundwater Protection" have been carried out within the framework of Project FP/1107-79-04 "Protection of the Lithosphere as a Component of the Environment" sponsored by the United Nations Environment Programme (UNEP) and the United Nations Educational, Scientific and Cultural Organization (UNESCO). Some of the main tasks of Project FP/1107-79-04 were the generalization of the advanced international experience in the area of integrated management and rational use of groundwater with allowance for environmental protection aspects and passing on this experience to developing nations.

The present-day level of using natural resources for meeting demands of economic growth and raising standards of living has given impetus to the development of geology and related sciences. The solution of these complicated problems on the global scale is the main content of the international co-operation in the area of the lithosphere.

Groundwater, being one of the essential components of the geological environment, plays an important role in the life of modern society. Of particular importance is fresh groundwater as one of the principal drinking water supply sources. Therefore groundwater pollution and depletion control is an urgent hydrogeological and water-management problem of to-day whose solution should be considered within the framework of environmental protection as a whole.

Both for UNESCO and UNEP, the successful completion of the preparation of the monograph "Hydrogeological Principles of Groundwater Protection" is an important stage in the implementation of the broad programme of international co-operation in the area of the protection of the geological environment and rational use of its resources.

The elaboration of the main scientific and practical recommendations on the content of the present monograph has been performed within the International Scientific Council on Problems of the Lithosphere, which consisted of 18 well-known scientists from nine countries, UNESCO, and UNEP.

The editor of the Russian text of the monograph is V.M. Goldberg (USSR) and the editor of the English text is Ch. Young (Great Britain).

A large number of the sections of the monograph have been prepared by USSR scientists, mostly engaged at the All-Union Research Institute for Hydrogeology and Engineering Geology, USSR Ministry for Geology. Among them are such well-known experts as A.A. Konoplyantsev, V.M. Goldberg, L.S. Yazvin, V.A. Mironenko, K.I. Sychev, and A.E. Oradovskaya. Well-known scientists from France (J.J. Fried, L. Zilliox, G. Castany), USA (H. Meyer, Ph. Cohen, B. Foxworthy), Czechoslovakia (J. Vrba, V. Pelikan, J. Švoma, M. Vrana), Hungary (F. Saky), Greece (J. Ganulis), and the Netherlands (G.B. Engelen) have contributed much to the monograph. So, the contributors to "Hydrogeological Principles of Groundwater Protection" are competent and from countries where groundwater protection issues receive considerable emphasis.

The monograph has been successfully completed largely due to the attention and practical help on the part of the leadership of UNESCO and UNEP which offered aid and means that made it possible to describe the present state of the hydrogeological problems of groundwater protection on the international scale. I would like to acknowledge all the authors participated in the preparation of the monograph and express thanks to all the persons whose names are not mentioned in the monograph, but who contributed by their efforts to successful completion of the monograph, being an all-round scientific publication on the subject it deals with.

The participation of many leading scientists in the studies under discussion, broad representativeness and scientific importance of the materials enable one to express an opinion that the monograph will be timely and useful for scientists and experts from both developing and developed countries.

The task of further studies in the area of groundwater use and protection is the theoretical, technological and organizational substantiation of the system of prediction and optimal management of the groundwater regime and groundwater resources under conditions of the increasing human impact on the lithosphere.

Development of the research and integrated assessment of the influence on the geological environment of water-management and reclamative activities, mining, fuel and energy production, and major urbanized areas are of great scientific and practical importance. Elaboration of dynamic models of regions, being intensively developed, establishment of experimental environmental protection areas, and application of advanced international experience to efficient solution of the problems of rational use of natural resources and environmental protection in many countries are gaining importance.

The creation, at the regional and national levels, of lithomonitoring as a single system of control, evaluation

of the state and prediction of changes in hydrogeological and engineering-geological conditions under the human impact may be recommended as one of guidelines of further research.

Prof. E.A. Kozlovsky
Editor-in-Chief, Scientific leader of the project

INTRODUCTION.

Tasks and main contents of the monograph

The increasing role of groundwater in the economic development of some countries and regions and the need for provision of people with drinking water of good quality promoted the international co-operation in the area of rational groundwater management, use, and protection.

In many countries, groundwater is the principal drinking water source and is also extensively used in industry and agriculture. In the USA, aquifers provide about half of the nation's population with drinking water and meet about 40 % of irrigation demands. Groundwater accounts for 8.5 % of total water resources, however, it satisfies 25 % of water demands of industry, agriculture, and population.

In connection with the wide use of groundwater for drinking, the importance of its quality within the human health protection system increased substantially. Unfortunately, meeting of human demands for pure water is far from complete in all regions.

Despite the existence of unconsoling statistical data on groundwater quality in many regions, it is evident that the quantity of pollutants entering aquifers decreases slowly and in some cases even increases.

Though prevention of groundwater pollution is undoubtedly the most effective means of preservation of the good quality of groundwater, aquifer contamination continues and this requires solution of groundwater protection problems. The efforts of many scientists, national organizations, and institutions are aimed at solution of these problems.

A number of valuable scientific works on groundwater occurring under various climatic and hydrogeological conditions have been published recently. They include such UNESCO publications as "Groundwater Pollution and Protection" (1980), "Groundwater Studies" (1982), "Mathematical Modelling for Groundwater Resources Assessment" *1982), "Groundwater of Magmatic Rocks" (1982), "Problems of Salinization of Coastal Aquifers" (1983), and Proceedings of the International Symposium on Groundwater Quality (1981).

However, all the above publications discuss separate aspects of hydrogeology and are no all-round descriptions of processes related to groundwater pollution and depletion control with allowance for environmental protection. In this connection, UNESCO and UNEP made a decision to publish an extensive monograph "Hydrogeological Principles of Groundwater Protection", in which all aspects of studying processes of groundwater pollution and depletion, as well as activities for prevention of groundwater quality deterioration might be systematically and purposely discussed.

The monograph deals with the theoretical, methodological, and practical issues of the problem of groundwater pollution and depletion control.

The monograph comprises four chapters.

Chapter 1 discusses the state of the problem of groundwater use and protection and theoretical aspects of the migration of pollutants in groundwater.

Chapter 2 covers issues concerning groundwater pollution conditions, the effects of various man-induced factors, and seawater intrusion into coastal fresh-water aquifers. Considerable attention is given to laboratory and on-site methods for studying migration and for estimating migration parameters, as well as seepage properties of clayey deposits, which largely determine conditions of groundwater vulnerability. The zone of aeration performs the same protective role in relation to unconfined groundwater. Much importance should be paid to the zone of aeration when studying groundwater pollution processes. The chapter terminates with discussion of practical methods for predicting the migration of pollutants in groundwater, conditions of their approach to water-supply wells, and the time of entering first portions of polluted groundwater to water-supply wells.

Chapter 3 deals with issues of groundwater depletion and characteristics of processes caused by groundwater level lowering in the course of groundwater development. The sources of generation of safe groundwater yield, processes of groundwater depletion under different natural conditions and accompanying phenomena (surface runoff variations, secondary consolidation and compaction of rocks, suffosion and karst processes) are discussed in this chapter. Main principles of hydrogeological forecasts in connection with groundwater depletion control are presented.

Chapter 4 discusses problems of groundwater protection from pollution and depletion. Water-protective measures are subdivided into preventive and special protective. The study and search for groundwater pollution sites, location of observation well networks, estimation of groundwater vulnerability conditions, delineation of zones of sanitary protection of water-supply wells, principles of location of industrial plants and waste disposal dumps for diminishing the scale of groundwater pollution are discussed. Various aspects of artificial recharge of groundwater and methods of its prediction are analysed in detail. The problems of drain water utilization and integrated use of surface and groundwaters are discussed.

It is expected that the monograph, containing the advanced experience in hydrogeological investigations, integrated use and protection of groundwater as one of natural resources of the lithosphere, will be a valuable and useful book not only for hydrogeologists – scientists and experts, but also for technicians and the personnel of organizations and departments engaged in the management and rational use of groundwater and in environmental protection.

The monograph is one of the results of the activities in keeping with the project "Protection of the Lithosphere as a Component of the Environment" that has been performed under the sponsorship of UNEP in co-operation with UNESCO and with the support of the USSR State Committee for Science and Technology and USSR Ministry for Geology, the latter being the leading organization of the project activities.

In view of the interdisciplinary nature of the studies, the activities involved in the "Protection of the Lithosphere as a Component of the Environment" project were co-ordinated with the International Hydrological Programme (IHP), International Geological Correlation Programme (IGCP) and with such non-governmental organizations as International Association of Hydrogeologists (IAH), International Association of Engineering Geology (IAEG), International Association of Soil Mechanics (IASM), International Commission on Lithosphere of the International Council of Scientific Unions and some others.

The activities of the "Protection of the Lithosphere as a Component of the Environment" project were based on the UNESCO and UNEP programmes which, in turn, had been determined by the decisions of sessions of UNESCO General Assemblies, Intergovernmental Council of UNEP Managers, international conferences such as the UN Conference on Water Resources in Mar del Plata (Argentina, 1977), and the UN Conference on Science and Technology for Development of Vienna (Austria, 1977).

The study of water resources, including groundwater, is one of the main guidelines of the UNESCO general programme, oriented at the combined solution of problems, involved in the multipurpose use and protection of water resources with allowance for ecological, economic, and social factors.

In this connection, the monograph, prepared within the framework of the "Protection of the Lithosphere as a Component of the Environment" project as a scientific and methodological guide on rational use and protection of groundwater, will not only promote the implementation of the International Hydrological Programme, but also will contribute to the fulfillment of the tasks of the International Decade of Drinking Water Supply and Sanitation (1981–1990) that was officially initiated in the November of 1980 by the XXXV Session of the UN General Assembly.

Chapter 1.

STATUS OF THE PROBLEM OF GROUNDWATER USE AND PROTECTION

1.1. USE OF GROUNDWATER AT PRESENT

At the present time, the role of water resources is becoming increasingly more important. In many countries it is the absence of water resources of the appropriate quality which limits further economic growth. Population growth, urbanization, the improvement of living conditions, the development of industry and agriculture have led to a substantial increase in water consumption. According to available estimates [11] the total water used in the world at the beginning of the twentieth century amounted to about 400 km^3/year, by the middle of the century (1950) it had increased to 1100 km^3/year and by the end of the 1970's had exceeded 3000 km^3/year.

Abstraction for domestic and drinking water needs rose considerably, due to both absolute population growth and the increase in per capita consumption (in litres per capita). From early in the twentieth century per capita consumption increased fourfold in the USSR and the USA, and in Europe more than twice [11]. If the total water use for the needs of the population in 1900 is estimated at about 20 km^3/year, with a population of 1.6 billion people, in 1950 this growth had tended 60 km^3/year with a population of 2.5 billion, and in 1975 – 150 km^3/year with a population of 4 billion. Therefore, the average per capita consumption for all countries of the world rose from 30 l/day in 1900 to 60 l/day in 1950, and to 100 l/day in 1975. The per capita consumption does not exceed 100–120 l/day in large urban centres, reducing to 20–30 l/day in the underdeveloped regions of Africa and Latin America. The growth of average per capita consumption is expected to continue in the future. According to forecasts, by the year 2000 the population of the world is expected to exceed 6 billion, and the magnitude of water usage for the needs of the population will amount to about 440 km^3/year, i.e., specific water consumption will have increased to 200 l/day.

Water consumption in industry has risen from 30 km^3/year in 1900 to 650 km^3/year in the mid-70's. It is anticipated that by the year 2000 this quantity will have risen to 1900 km^3/year, although it is true that the consumption of non-renewable water resources is estimated at only 70 km^3/year. The largest consumer of water is irrigated agriculture, which is responsible for more than 70 % of total world water consumption and for more than 90 % of world consumption of non-renewable water resources. The area of irrigated land has increased from 40 million ha (at the beginning of the century) to 250 million ha, whilst water abstractions for these purposes rose from 350 to 2100 km^3/year. The proposed extension of irrigated areas to 420 million ha by the year 2000 will require an increase in water abstractions to 3400 km^3/year [11].

A comparison between water demands and total water resources shows that in many regions of the world the possibility of satisfying these requirements is becoming a serious problem. The situation is aggravated by the extremely uneven spatial and temporal distribution of fresh water resources and by the contamination of water supplies by industrial and agricultural wastes. Meeting the needs of a population for domestic and drinking water supplies with strict quality constraints paves a particularly complex problem. It was for this reason that the decade of 1981–1990 was proclaimed by the United Nations General Assembly as the "International Drinking Water Supply and Sanitation Decade".

The study and rational use of groundwater play a central role in the solution of the problem. Groundwater, as a source of domestic and drinking water supply, has a number of advantages over surface water. As a rule, it is characterized by higher quality and does not require expensive treatment. It is often protected against contamination and evaporation. Groundwater resources, due to the storage of capacity of aquifers, do not undergo any substantial seasonal or annual fluctuations, and in many cases groundwater can be obtained in immediate proximity to the consumer. Groundwater abstractions may be phased into operation, as needs grow, whereas the construction of facilities for the utilization of surface water usually requires large capital investments. All these circumstances pointed towards a substantial growth in the use of groundwater in many countries for domestic and drinking water supplies, and in some regions, mainly arid, for the irrigation of large land areas.

As an example, in the Soviet Union the share of groundwater in the total balance of domestic and drinking water supply has risen from 40 to 70 %. At the present time more than 60 % of the cities in the USSR meet needs for domestic and drinking water with groundwater, about 20 % have mixed sources of water supply (surface and ground) and less than 20 % of cities have municipal water supplies completely based on surface water. In recent years in the USSR there has been a substantial increase in the use of groundwater for the irrigation of land, mainly in the southern regions of the country (Central Asia, Southern Kazakhstan, Transcaucasia, Northern Caucasus). The possibility of using fresh groundwater of drinking quality for irrigation and other needs unrelated to the drinking and domestic water supply in the USSR is governed by "The Principles of Water Legislation of the USSR and the Union Republics". In accordance with this instrument, the use of groundwater of drinking quality for irrigation is allowed only in regions where the necessary sources of surface water are absent and where there are sufficient supplies of groundwater to satisfy the needs for domestic and drinking water

supply for a long time to come. At present in the USSR a total of about 30 km^3/year of groundwater is withdrawn annually, of which about 20 km^3/year is used for water supply. According to present forecasts, by the year 2000 the use of groundwater will increase to 65–70 km^3/year.

The use of groundwater plays an important role in the USA where it reaches 90–100 km^3/year and makes up about 20 % of the total water usage for all needs including industry. During the period from 1955 to 1975, the use of groundwater increased by almost 80 %. In the USA the main consumer of groundwater is irrigated agriculture, which is responsible for about 65 % of the groundwater used. About 15 % of the groundwater withdrawn is intended for municipal water supply, while the water supply of large cities is based mainly on surface water. Rural water supplies are based almost entirely on groundwaters.

Groundwater is the main source of domestic and drinking water supply of the urban and the rural population in many European countries. In Bulgaria, Hungary, the German Democratic Republic, Austria, Belgium, Denmark, Switzerland, the Federal Republic of Germany, and the Netherlands groundwater sources represent from 60–100 % of the total water consumption for these purposes, while in the United Kingdom, France, Poland, Czechoslovakia, Italy, Sweden, and Finland it represents from 30–50 %. An idea of the quantity of groundwater utilized for water supply in the countries of the European Community is given by Table No. 1.1, [31].

Table 1.1

GROUNDWATER ABSTRACTION IN DIFFERENT COUNTRIES

Countries	Use of groundwater for all purposes, km^3/year	Use of groundwater for drinking water supply (municipal water supply), km^3/year	Groundwater share in drinking water supply, %
FRG	6.24	2.08	67
Belgium	0.57	0.39	76
Denmark	0.70	0.32	98
France	5.00	2.00	50
Ireland	0.13	0.01	47
Italy	9.95	2.51	36
Luxemburg	0.03	0.02	64
Netherlands	1.13	0.43	63
United Kingdom	2.50	1.31	32
Total	26.25	9.07	

Groundwater is also extensively used in Australia, in a number of African (Tunisia, Egypt, Morocco, Libya) and Asian countries (Iran, India, People's Republic of China, Pakistan, Saudi Arabia, Japan). In some of these countries the withdrawal of groundwater, mainly for irrigation, amounts to 20–50 km^3/year (Iran, India).

A sharp increase in the withdrawal of groundwater for water supply and irrigation is, therefore, one of the main distinguishing features of the present use of groundwater.

The increase in groundwater use was accompanied by the concentration of water abstraction facilities. At the same time, water abstraction systems developed in two main directions. For centralized municipal and industrial water supplies, pumping stations were usually set up consisting of large numbers of wells, arranged either in rows or in the form of concentrated areal or annular systems. The pumping stations could, in this case, be located at a considerable distance from the consumer. Irrigation and decentralized agricultural water supply, normally consists of single wells or small groups of wells, located adjacent to the consumers. In some regions, both centralized installations and single wells scattered in area developed at the same time. The greatest volumes of abstractions are found at the large centralized abstraction facilities, the number of which has substantially increased in recent years. The yields of such grouped systems may amount to hundreds of litres, and sometimes m^3 per second. Increased drainage in regions of industrial development of solid mineral deposits anowated with measures taken to prevent rises in groundwater levels, have also contributed to the concentration of groundwater abstraction. The concentration of water abstruction into large-scale systems is the second distinctive feature in the use of groundwater at the present time.

The sharp increase in the use of groundwater and the concentration of abstraction points, in many parts of the world, have led to substantial changes in hydrogeological conditions and other components of the environment. Artesian discharges from wells have stopped, levels and heads in productive aquifers have fallen, and there have been changes in water quality caused either by the inflow of saline groundwater, or by sea water intrusion. In regions of intensive groundwater use such industrially-induced processes as changes in surface run-off, intensification of karotic processes, surface subsidence and changes in the features of the landscape have been observed. The situation is aggravated by the fact that in many cases the construction of water abstraction installations was carried out without proper hydrogeological studies, with no account being taken of possible interactions between individual water abstraction and water drawdown systems and the effect of groundwater abstraction on the environment. An important role was also played by uneconomical and irrational abstraction of groundwater which substantially exceeded the real requirements, particularly artesian wells not equipped with shut-off valves. All this has made it necessary for hydrogeologists to resolve the problem of the rational use of groundwater, integral parts of which are the protection of groundwater from depletion and the evaluation of the effect of groundwater abstractions on changes in other components of the environment with a view to determining the admissible limits of the use of groundwater. These questions are considered in chapter 3.

1.2. THE ROLE OF GROUNDWATER IN THE TOTAL WATER RESOURCES AND THE WATER BALANCE OF REGIONS

Groundwater, as one of the integral parts of the hydrosphere, is closely connected with other natural waters. Consequently, an alteration in the groundwater balance may lead to changes also in other components of the hydrosphere. In this connection, great practical importance is attached to evaluating the role of groundwater in the total water resources and the water balance of a region, of which the most important characteristic is the size of the natural groundwater resource.

Investigations for the study, evaluation and mapping of natural resources of groundwater have developed extensively in the last two decades. This is because the results of regional quantitative evaluations of natural resources of groundwater and subsurface flow are used for the solution of a number of important practical problems, including: 1) the long-term planning of groundwater use for water supply in the region under study or part of it; 2) determining the magnitude of groundwater recharge for a regional evaluation of the possibility of its use in drawing up water-management balances of economic regions; 3) determining amounts of base flow to rivers for a description of the subsurface component of streamflow and for a forecast of possible changes in streamflow under the influence of intensive groundwater development; 4) evaluating subsurface flows as an element of the water balance and the quantitative inventory of groundwater resources in drawing up plans for the comprehensive utilization and protection of total water resources. Investigations of subsurface flow also make it possible to obtain quantitative data on the migration of water in rocks, which have considerable importance for the study of general conditions of groundwater regimes, the evaluation of replenishment times, as well as of heat and mass transfer processes in the earth's crust.

Before proceeding to the consideration of the substance of the problem,the fundamental concepts and definitions used in investigations of natural resources of groundwater and subsurface flow should be defined. The natural resources characterize the amount of groundwater recharge under natural conditions due to the infiltration of atmospheric precipitation, the irecharge from influent streams and leakage between water tables, summarily expressed as the magnitude of discharge or the infiltration route.

The mean value over many years of precipitation minus evaporation is equal to the value of subsurface flow. Therefore, in the practice of hydrogeological investigations, natural resources of groundwater are usually expressed as mean annual and minimal values of modules of subsurface flow (in litres per second per km^2).

Subsurface flow is the process of movement of groundwater, under the effect of hydraulic head difference, from recharge zones to discharge zones. The quantitative characteristic of this process is the discharge value of the groundwater flow (the value of subsurface flow). One of the manifestations of subsurface flow is subsurface flow into rivers (base flow), which takes place from the saturated zone of rocks, located within the sphere of drainage of the river network. The groundwater in this zone, which is recharged mainly by direct infiltration, important quantitative characteristics are the coefficients of subsurface flow and the coefficients of subsurface recharge of rivers. The coefficient of subsurface flow is the ratio of subsurface flow to atmospheric precipitation. The coefficient of subsurface recharge of rivers (the ratio of the drained subsurface flow to the total streamflow) shows which part of the flow is due to the discharging of groundwater.

The evaluation of natural resources of groundwater should be carried out with respect to cutchments, defined in accordance with the objectives of the investigation.

First order groundwater cutchments include individual hydrogeological structures such as: artesian basins, mountain fold belts, crystalline shields. Second order cutchments would typically be a complete single aquifer, including its zones of recharge, flow and discharge. In more detailed investigations cutchment areas of the third and lower orders are singled out, for example, river basins or parts of them, development sites of groundwaters of various types (waters in karotic systems, waters in alluvial or fluvio-glacial forms, etc.)

The basic methods for regional evaluation of subsurface flow in zones of groundwater recharge are the water-balance method and calculations of changes in the base flow of rivers on sections between two gauging stations. In groundwater discharge zones, the methods employed include river hydrograph separation, calculation of changes in river base flow between gauging stations, estimation of the long term mean water balance, and the determination of mean long term values of spring flow. Given the necessary data, it is possible, in all cases, to make use of the hydrodynamic method of evaluation, including finite-difference calculations and modelling. A brief description of methods is given below.

Wide use has been made of the method of hydrograph separation [7, 10] in humid zone with well developed river networks. The essence of the method consists of taking into account the hydrogeological conditions in the river basins and the laws governing subsurface flow into the river from all the aquifers within the drainage zone. The regime and dynamics of subsurface flow into the rivers of the individual aquifers, are determined by the occurrence and recharge of unconfined and confined groundwaters in the river basin, and by the relationships between the point of discharge and the river.

Different methods of hydrograph separation may give rise to differences to interpretation [7, 9, 10].

The technique of river hydrograph separation, which isolates the subsurface flow, makes it possible to obtain the mean annual groundwater recharge value in area under study. The method relies on the fact that the subsurface flows in the active groundwater flow zone of areas with a permanent river network are controlled by the draining effect of the river systems. Accordingly, the natural resources of fresh groundwater from the saturated zone may be characterized by the size of the base flow component.

In some cases the value of the subsurface flow may be determined approximately by calculating the change in the low water flow of the river over the length between two gauging stations. The change in streamflow under prolonged low flow conditions in a stretch without any tributaries (or deducting the total flow of the tributaries), stimulate the base flow from the aquifers or the value of groundwater recharge in the case of influent stream. The selection of the gauging stations must be such that the difference in the streamflow between the gauges exceeds the possible total value of the errors in the measurement of streamflow.

A very important advantage of these methods of determining base flow to rivers lies in the potential for obtaining their mean long term values as a result of the use of already existing hydrometric data without the need to carry out special expensive exploratory and experimental hydrogeological surveys. These methods are fundamental in evaluating subsurface flow and natural resources of groundwater in regions with a well-developed river network, and readily available long term hydrometric data.

In a number of cases, the use of the methods under consideration is made difficult or impossible because of special features of the region, such as the development of artificial irrigation, which may distort the natural pattern

recharge; the state of regulation or control of the river, or a marked disparity between the surface and the groundwater catchments. In particular, artificial regulation of rivers makes it practically impossible to use the base-flow separation methods for the evaluation of groundwater resources.

If sufficient long-term, reliable groundwater level data are available, the value of recharge may be determined by analysis of the data. The value of groundwater recharge is calculated for individual wells from their mean annual range of water level fluctuations, and the specific yield of the aquifer. The specific yield values may be obtained from aquifer test data, or may have to be estimated from general values. If the latter is the case, the value of recharge which is calculated must be considered very approximate. The application of this method to the regional evaluation of recharge is complicated, and often indeed impossible, because of the need to extrapolate the value obtained at a single point to a large area.

The successful application of hydrodynamic methods, including mathematical simulation technique, depends on the extent to which hydrogeological parameters and boundary conditions of the groundwater system are known. Hydrodynamic techniques, given the necessary data, may provide estimations of the values of subsurface flow both in limited areas, and within complex, multilayered aquifer systems. There can be no doubt that in the future, as hydrogeological knowledge increases, the importance of hydrodynamic methods will grow steadily. A wider use of computerized methods of processing hydrogeological information may contribute to this end.

Evaluation of the flow in deep artesian aquifers may be made by estimating the mean long term water-balance for a river basin, or part of it. The limitations on this method are determined by how much larger the value of deep infiltration (or discharge) is than the value of the errors in the calculation of the remaining elements of the water-balance. However, the accuracy with which the main elements of the water-balance may be determined, especially evaporation, may not be sufficiently high for the method to be widely applicable.

The brief description given above outlines the advantages and shortcomings of each of the methods, and the choice of method of evaluation depends on the data available and the purposes of the investigation.

The methods outlined are not mutually exclusive. Given the necessary initial data, a comprehensive application of several methods should be attempted, which considerably enhances the reliability of the calculations.

The world's first complete nationwide evaluation of natural resources of fresh groundwater has been carried out in the Soviet Union.

The total value of natural resources of fresh groundwater (subsurface flow in the zone of action circulation) amounts to more than 30,000 m^3/s. As a result of the work carried out both for the territory of the USSR as a whole and for individual regions, the following principal qualitative parameters have been estimated:

i) the mean annual and minimal values of subsurface flow in $l/s \cdot km^2$;

ii) the coefficients of subsurface flow and of subsurface recharge of rivers;

iii) in addition, the influence of the main factors on the formation and distribution of groundwater resources under various natural conditions have been determined.

The distribution of the main quantitative parameters of subsurface flow over the territory, by regions, is distinguished by a marked heterogeneity, clearly reflecting the influences of the main geological and structural elements and topographic and climatic zones. More than 55 % of the total amount of subsurface flow occurs within fold-mountain belts, about 42 % corresponds to the extensive expanses of continental platforms (the Russian, the Western Siberian and the Turanian) and only 3–4 % of the total value of subsurface flow is associated with crystalline shields.

The distribution of subsurface flows by climatic zones shows that more than 80 % of the total amount is associated with humid zones, about 18 % of the flow is formed in the semi-arid zone and only 2 % in the arid zone.

Investigations carried out for the regional evaluation and mapping of subsurface flow in the territory of the USSR and the territory of Central and Eastern Europe (in cooperation with the socialist countries of Europe) have made it possible to estimate general values for subsurface flow under various natural conditions. The most general law is the difference in the distribution of the parameters of subsurface flow within platform-like regions and solid mountain belts, with a range of values varying from less than 0.1 to 6.0–8.0 l/s and from 0.1 to 30–50 $l/s \cdot km^2$ respectively.

On the continental platform it is normal to find that the parameters of subsurface flow follow a regular pattern imposed by climatic factors. Local changes, controlled by hydrogeological factors such as lithological variations in the active aquifer systems, stand out most sharply against the background of latitudinal distribution. Maximum values of subsurface flow parameters are typical of areas of strong karstic development and, to a lesser extent in regions where the upper part of the section is represented by coarsely fractured and sandy fluvioglacial and terminal-morainic deposits. High values are also characteristic of river valley aquifers composed of highly permeable alluvial deposits. Minimal values have been recorded from regions where the native circulation zone is represented by loamy and clayey rocks, and from low lying areas in which low permeability beds being close to the ground surface inhibit recharge from natural precipitation. In folded mountain chains the distribution of subsurface flow values is determined mainly by sudden lithological changes and by the orographic increase in precipitation with the height. Thus, the high values of subsurface flow parameters in the Caucasus, the Carpathians, and the Balkans are brought about by the wide distribution of permeable fractured rocks in what are, properly speaking, fold mountain belts and in highly permeable coarse grained deposits in intermontane basins. When combined with the deep erosional disintegration of the terrain and the considerable quantity of precipitation these features lead to favorable conditions for groundwater recharge.

The average coefficient of subsurface flow in the USSR has been estimated to be 9 %, with local variations between 1 % and more than 50 %. In flat lying regions, latitudinal zonality is characteristic in the distribution of values of subsurface flow coefficients.

Determining the values of the coefficients of subsurface river recharge is of important practical significance in evaluating the effect of the use of groundwater on streamflow. For the territory of the USSR as a whole the value of the coefficient of subsurface river recharge amounts to 24 %, varying from 5–10 % in areas with a relatively shallow zone of active groundwater circulation,

poorly defined relief and favorable conditions for the formation of surface run-off, to 40—50 % or more in areas composed of highly permeable strata with densily developed river drainage.

The importance of regional investigations of groundwater natural resources and of subsurface flow is far from being limited to questions of use alone. The results of investigations of subsurface flow make it possible to study on a quantitative basis the processes of interrelationship among different aquifers, the reliable idenfication of the boundaries of hydrogeological structures, zones of groundwater recharge and discharge, and to determine periods of groundwater replenishment. Further possibilities are opening up in the study of the geological role of groundwater — of the subsurface ionic flow, of the activity of groundwater in denudation, and of heat loss via groundwater.

In recent years, significant research has been carried out in studying the role of groundwater in the water and salt balance of seas and large lakes. This is important, in particular, in studying the intrusion of seawaters into groundwater due to overpumping, which is a form of induced contamination.

The discharge of groundwater into the sea takes place either in the form of submarine springs, usually associated with tectonic disturbances in areas of strong fractured and karstified rock development, or by upward seepage from confined, submarine aquifers. Large submarine springs are the most striking but not the main type of groundwater discharge into the seas. The predominent form is diffuse discharge by seepage through low permeability sea bed deposits.

The evaluation of subsurface flow into the world ocean was made by a comprehensive hydrological and hydrogeological method. Calculations were made for the separate regions of the coastal areas for individual seas and oceans. For each region, not only was the total value of the inflow of groundwater into the sea calculated, but also specific parameters — the modulus of subsurface flow from 1 km^2 of the land catchment area and the groundwater discharge for 1 km of coastline. This made it possible to compare the areas, to relate the values of subsurface flow to the various natural factors and to identify the general laws governing this process.

The evaluation of the entry of salts with subsurface flows into the seas and oceans was carried out taking account of the total value of the submarine discharge of groundwater and its average mineralization. The analysis made of the sources of subsurface flow into the inland seas of the USSR has shown that the main part is derived from the upper part of the saturated zone (zone of active circulation), which is characterised by groundwater with a mineralization of less than 1 g/l.

The total amount of salt transported by groundwater into the world ocean reaches 1230 million tons per year, or to more than 50 % of the salt inflow with rivers (2316 million tons per year). Accordingly, the submarine discharge of groundwater exerts a substantial effect on the salt and hydrobiological regimes of the seas and oceans, and also on the processes of biogenous sediment accumulation and the formation of economic mineral deposits.

In conclusion, the fundamental issues involved in the regional evaluation of groundwater resources and the determination of their role in the water balance and in total water resources have been discussed. To evaluate the potential use of groundwater supply of specific consumers and of preventing its depletion and contamination further work may be necessary (Ch. 2 & 3).

1.3. MAJOR ISSUES IN GROUNDWATER PROTECTION

One of the urgent problems of our times is the protection of tne natural environment.

The effect of human activities on the environment manifests itself in various ways. Large-scale changes in the geological and hydrogeological conditions are potentially of great impact and include the working of economic mineral deposits, extensive excavation and drainage works, mine drainage, the construction of hydraulic structures, the redistribution of the surface run-off of rivers, the use of the earth's interior for various purposes, (the underground disposal of industrial wastes, the artificial recharge of groundwater, the building of subterranean gas storage tanks, etc.), and the carrying out of reclamation projects.

The influence of man's activities on the earth's interior, brought about by industrial, agricultural and cultural development, is now becoming a powerful force for change in the hydrogeological conditions of water-bearing systems.

Of the technically-induced factors the greatest significance is assigned to the intensive abstraction of groundwater, the disposal of contaminants in the natural environment, as well as all kinds of underground workings, connected with the extraction of useful minerals, construction and the use of the earth's inner resources.

The effect of man's economic activities manifests itself in two main directions: 1) a change in the hydrochemical conditions and quality of groundwater and 2) a change in the hydrodynamic regime of aquifers, the lowering of groundwater levels and a depletion of supplies.

The change in hydrochemical conditions shows up first of all in the contamination of groundwater and the deterioration of its quality, while the change in hydrodynamic conditions takes the form of a lowering of groundwater levels, and a change in the conditions of its recharge and discharge. The hydrodynamic and hydrochemical conditions of water-bearing systems are closely interrelated: a change in hydrodynamic conditions may entail a change in hydrochemical conditions while a change in hydrochemical conditions, which affects not only the composition of groundwater but also the aquifer hydraulic properties, particularly effective porosity, may lead to secondary hydrodynamic changes.

Contamination reduces the quantity of fresh groundwater available for potable supply. Similarly, the intensive lowering of groundwater levels, may induce the invasion of the aquifer by contaminated or naturally substandard waters.

The protection of groundwater may be defined as a set of measures designed to preserve and improve such a qualitative and quantitative state of groundwater as will enable it to continue to be used in an economic manner.

Thus, as a result of the intensive extraction of groundwater in coastal areas, salt seawater is encroaching on aquifers and, as a consequence of this, salinization and the deterioriation of the quality of the groundwater.

At the present time groundwater contamination is mainly localised, but widespread, and therefore may be considered as a regional phenomenon. As many specialists point out, the threat of groundwater contamination represents a much greater danger than the threat of a physical shortage of water.

Groundwater contamination is intimately linked with contamination of the natural environment, and forms the basis on which conservation activities to protect groundwater must be based. It is impossible to prevent groundwater contamination because of continuing contamination of surface waters, the atmosphere and the soil, since the contaminants almost inevitably pass from them into the groundwater. Therefore, the protection of groundwater must be an integral part of a comprehensive programme for the protection of the environment as a whole. This shall be the main emphasis in any strategy for groundwater protection.

The extent to which groundwater contamination occurs depends on the extent of natural protection of the aquifer.

Confined groundwater is generally protected to a greater or lesser extent while unconfined groundwater is practically unprotected. The majority of contaminants from the surface enter the unconfined aquifer which bears the main brunt of man-made contamination. From the unconfined groundwater the contamination may enter deeper aquifers. Unconfined groundwater is therefore the first water-bearing layer contaminated from the surface. The unconfined groundwater reservoir plays a two-fold role with respect to the deeper aquifers: on the one hand, the contaminants may pass from the unconfined groundwater to lower-lying aquifers and, accordingly, the unconfined groundwater acts as a conveyor belt for contamination into these layers; on the other hand, the unconfined groundwater layer acts as a "buffer" and thus performs protective functions with respect to the deeper aquifers.

The study of contamination of unconfined groundwaters should be carried out in conjunction with a study of the contamination of rocks of the zone of aeration, the soil layer and atmospheric precipitation.

Groundwater contamination may first be recognized at abstraction points, and also at industrial and agricultural abstraction where a network of observation wells exists and where more or less systematic observations of the quality of groundwater are carried out.

It is necessary, therefore, to set up a groundwater monitoring system in order to provide base-line data. But contamination should be integrated into an overall programme for monitoring of the state of the environment.

The protection of groundwater includes:

– strict observance of legislative acts and legal instruments for the protection of the environment as a whole and of groundwater in particular;

– the implementation of measures designed to reduce contamination of the environment and of groundwater, particularly the reduction of wastes produced by industry, the development of waste-free process, the recycling and reuse of water in industrial processes, the construction of special waste treatment facilities, the prevention of losses of effluents from sewage networks, etc.;

– the strict observance of procedures to be followed in prospecting for groundwater and in the planning, construction and operation of groundwater sources;

– the application of conservation measures for safeguarding groundwater.

An adequate legislation framework and an available technology are essential to the profit protection of groundwater resources. Thus, the elimination of a source of groundwater contamination, pumping out the contaminated groundwater must be accompanied by measures for the elimination of the sources of pollution. Otherwise, the action taken will fail to eliminate the source of contamination in the aquifer.

Conservation measures for the protection of groundwater against contamination may be subdivided into preventive and special measures.

Preventive measures include: the choice site for the facility or project concerned (whether it be industrial or agricultural) where its negative effects on the environment and on fresh groundwater will be minimal; the establishment and maintainance of sanitary protection zones around groundwater sources; an impact assessment of the planned project or installation on groundwater and the environment; a study of the state of protection of groundwater; the installation at major industrial facilities of a network of groundwater quality observation wells; systematic monitoring of the level of groundwater contamination and forecasting its development; the identification and cataloguing of actual and potential sources of groundwater contamination.

The most inportant type of conservation measure is the establishment of a network of observation wells at major industrial facilities and around large groundwater sources. Periodic reviews of the forecasts of the development of groundwater contamination should be made using the information supplied by the observation methods. The same data form the basis on which special protective measures may be formulated.

The identification and study of the main sources of contamination are essential both in order to assess the level of contamination, and also to select priority facilities as a basis for planning water conservation measures and developing a stage-by-stage program for their implementation.

Special protective measures are planned on the basis of existing knowledge of the level of groundwater contamination, the danger of contamination established for potable and industrial needs, on the basis of forecasts of migration of contaminants in the aquifer, and potential changes in the quality of water at supply prints.

The improvement and development of methods for forecasting the migration of groundwater contaminants is therefore of special importance. Contaminated and fresh groundwaters represent a system of heterogeneous fluids, which is accompanied by complex processes of interaction in the water-rock system.

Theoretical analysis of the internations is very complex and may be practically impossible. It may be, therefore, necessary to carry out extensive field investigations of the migration of specific contaminants in order to gain an appreciation of the potential effects on groundwater quality.

Special protective measures are: the elimination of sources of groundwater contamination by pumping out the source of contamination; restricting further migration of the pollutants by pumping from specially sited wells; the construction of protective water-intakes for the interception of contaminated groundwater and the creation of hydraulic barriers; the installation of grout curtains around the source of contamination; simultaneous pumping, using wells penetrating to different depths, etc.

Such measures may be expensive and often represent considerable technical difficulties including the disposal of large quantities of pumped out contaminated groundwater. Therefore, in the protection of groundwater, special emphasis should be attached to preventive measures designed to avert and forestall (wherever pos-

sible) the contamination of groundwater and to reduce the scale of such contamination.

1.4. THE SCIENTIFIC AND TECHNICAL BASIS OF INVESTIGATIONS IN THE FIELD OF GROUNDWATER PROTECTION

The practice of groundwater hydrology requires, most importantly, knowledge of the sciences of geology, mathematics, physics, and chemistry (Freeze and Cherry, 1979, p. 10–13). Application of the science of geology provides understanding of the matrix, or framework, of groundwater flow systems. Volumes of storage and patterns and rates of flow are determined by the three-dimensional configuration of the rocks through which flow takes place, by the lithological boundaries of groundwater reservoirs, and by the porosity and permeability of rock materials. Investigation of the geological history of the region under study and present-day rock structure and lithology enables delineation of these basically important hydrogeologic features.

The science of physics provides the foundation for quantitative analysis of groundwater reservoirs. The physical laws that govern the movement of groundwater are part of the branch of physics devoted to fluid mechanics. The basic mechanical properties of fluids and solids (density, pressure, energy, work, stress, and hydraulic head) are the fundamental physical controls on flow through natural porous media [27, 37, 42, 14]. Evidence accumulated over the past several decades indicates that, in addition to hydraulic head, water may be induced to flow through porous media under the influence of other gradients, including temperature, electrical potential and chemical gradients [22, 14]. Although each of these gradients may play a role in producing flow under special circumstances, most practical problems of groundwater hydrology permit the assumption that flow gradients are solely hydraulic. Chemical gradients, however, may be significant forces in the movement of chemical constituents, including pollutants, in groundwater systems.

Mathematics has enabled groundwater hydrology to advance from a qualitative science to a quantitative one. It is the language of quantitative methods of groundwater description and analysis, simulation modeling, and prediction of the effects of physical and chemical stresses imposed on groundwater systems. Many of the recent advances in understanding and solution of both scientific and practical groundwater problems were attained through mathematical approaches known collectively as numerical methods.

Evaluation of the chemistry of groundwater, the behaviour of both natural constituents and pollutants, and the design of protective or reclamation measures require application of principles of organic, inorganic, and physical chemistry. Elements of biology also may be involved. Minerals dissolved in groundwater help to explain its probable source and paths of flow; mineral constituents also have a bearing on usefulness of the water for industrial, irrigation, or domestic uses. Fem (1970) and Stumm and Morgan (1970) describe the methods for study and interpretation of the chemical characteristics of natural water, and the geochemical principles that control the behaviour of dissolved minerals, including pollutants, in surfacewater and groundwater environments.

Groundwater pollution studies seek to understand the physical, chemical, and biological conditions that permit the introduction and migration of contaminants in groundwater systems. Inasmuch as it is flowing groundwater that transports pollutants, investigations invariably require joint consideration of the hydrodynamics of water flow and of solute transport in porous media. They include, also, the application of low-temperature aqueous geochemistry, groundwater systems analysis, and certain biological process.

The kinds of data required to analyze groundwater systems and to solve hydrologic problems depend on the hydrogeological characteristics of the groundwater reservoir under study and on the nature of the water problems confronted. Hydrologic analysis is most productive when the hydrogeologic data are collected for definable problems, and with at least a preliminary conceptualization of how the hydrologic system functions. Knowledge of the economic, legal, and other nonhydrologic constraints that must be taken into account may aid efficient design of the investigation.

Broad, general planning may require only description and interpretation of existing, readily obtainable hydrologic and geologic information. Groundwater information adequate for at least general regional planning is available for many countries; in others, large voids in areal knowledge may necessitate appraisals that provide an information base for general nationwide planning and management of groundwater resources. Complex regional and local problems necessitating greater detail and better understanding require carefully designed investigations, including geologic mapping, test drilling, aquifer tests, geochemical investigation, modeling, and other studies appropriate to the particular problem at hand.

The principal data requirements for groundwater analysis, problem solution, and application of modelling methods are listed in Table 1.2. Most groundwater reservoirs, however, have a uniqueness or individuality of hydrogeologic character that requires design of special investigations tailored to those features. In addition, the stresses imposed usually have unique features. For these reasons general lists of data needs, such as those given in Table 1.2, invariably must be adapted to reservoir and problem conditions peculiar to the case at hand. Though the general procedures listed in the Table may be used as guidelines, in most instances additional new or modified methods of problem solving must be devised to address each reservoir and its water problems.

The initial phase of groundwater study consists of an evaluation of geologic and hydrologic data readily available in order to define the broad hydrogeologic features of the reservoir. This first evaluation enables development of an early conceptual model of the system and how it functions, and the most promising approaches to problem solutions. A first-stage model, now most commonly a digital model, even though based only on the limited data initially available, may be helpful even at this early stage of investigation.

Table 1.2

PRINCIPAL DATA REQUIREMENTS FOR GROUNDWATER ANALYSIS, PROBLEM SOLUTION, AND MODELLING

Physical framework	Hydrogeologic maps showing extent and boundaries of all aquifers and non-water-bearing rocks. Topographic map showing surface-water bodies and land forms. Water-table, bedrock-configuration, and saturated-thickness maps. Transmissivity maps showing aquifers and boundaries. Maps showing variations in storage coefficient. Relation of saturated thickness to transmissivity. Hydraulic connection of stream to aquifers.
Hydrologic stresses	Type and extent of recharge areas (irrigated areas, recharge basins, recharge wells, natural recharge areas). Surface-water diversions. Groundwater pumpage (distribution in time and space). Precipitation. Areal distribution of water quality in aquifer. Streamflow quality (distribution in time and space). Geochemical and hydraulic relations of rocks, natural water, and artificially introduced water or waste liquids.
Model calibration	Water level change maps and hydrographs. Streamflow, including gain and loss measurements. History of pumping rates and distribution of pumpage.
Prediction and optimization analysis	Economic information on water supply and demand. Legal and administrative rules. Environmental factors. Other social considerations.

Preparation of a preliminary report at this stage may be advisable to organize the information on hand and to make judgments about additional work needed. These first efforts alone may be adequate to resolve some or all the problems of concern. The initial report will serve as a guide to the design of further investigation that may be warranted.

If the problem is one of chemical quality or pollution, substantial geochemical and water-quality information to supplement hydraulic data may be necessary. Areal variations in water quality, and its relations to flow, rock composition, and other factors controlling the chemistry of the groundwater will need to be incorporated into the study. Problems associated with the influx of pollutants or the transport of waste liquids in groundwater may require specialized data on:

1) the physical, chemical, and thermal nature of the foreign liquid introduced; 2) its sources and volumes; 3) physical parameters necessary for quantitative studies and prediction (dispersion coefficient, for example); 4) the mineralogical composition of the reservoir rocks and associated finer-grained beds.

Data collection should proceed hand-in-hand with continuing refinement of the hydrological problem, progress in analytical studies and modeling, and report preparation. In this interative manner each of the elements of work should be revised throughout the life of the investigation to accommodate progressive improvement in understanding. Thus, the final report may be quite different from the content and conclusions envisioned at the start of the study.

The following brief explanation of the modelling approach to groundwater solute-transport analysis illustrates more specifically the information required, and the methods of integration of geologic parameters, fluid parameters, chemical reactions, and mathematics for purposes of pollution assessment.

The most widely utilized techniques for prediction of quality changes during movement of groundwater through porous media employ mathematical equations that express the relations among the important physical and chemical processes at work. These equations are solved, either analytically or numerically, with properly designed initial boundary conditions that reflect the hydraulic and chemical stresses that may be imposed on the system. Various physical and chemical processes may take place during groundwater movement, and the mathematical description of these processes and controlling parameters need to be determined.

Groundwater solute-transport models normally consist of two parts: the groundwater flow model and the groundwater transport model. These two parts are connected through the use of Darcy's law, which relates the hydraulic gradient to the seepage velocity. It is necessary to understand the hydraulics of the system before evaluating the changes in water quality during flow. Konikow and Grove [30] express the groundwater flow equation as:

$$S_s \frac{\partial h}{\partial t} = \frac{\partial}{\partial x_i}\left(k_{ij} \frac{\partial h}{\partial x_j}\right) - W^* \qquad (1.1)$$

where S_s is the specific storage coefficient, L^{-1};
h is the hydraulic head, L;
t is the time variable, T;
x_i and x_j are the Cartesian coordinates, L;
k_{ij} is the hydraulic conductivity tensor, LT^{-1}, and
W^* is the volume flux per unit volume (positive sign for outflow), T^{-1}.

Equation (1.1) is solved for the hydraulic gradient in the system of interest with various boundary or initial conditions that may be specified. This gradient then is used in Darcy's equation,

$$V_i = \frac{k_{ij}}{\epsilon} \frac{\partial h}{\partial x_j} \qquad (1.2)$$

where V is the seepage velocity, LT^{-1}; and
ϵ is the effective interstitial porosity, L^0,

to calculate the seepage velocity. The solute transport equation has been derived by Konikow and Grove [30] and can be written as:

$$\epsilon \frac{\partial c}{\partial t} + \frac{\partial}{\partial x_i}(\epsilon c V_i) - \frac{\partial}{\partial x_i}\left(\epsilon D_{ij} \frac{\partial c}{\partial x_j}\right) + C'W^* = \text{CHEM} \qquad (1.3)$$

where c is the chemical composition, ML^{-3};
D_{ij} is the dispersion tensor, L^2T^{-1};

C'' is the chemical composition input to the system, ML^{-3}; and

CHEM represents any chemical reactions that may be taking place, $ML^{-3}T^{-1}$.

Examining these three equations shows how they relate to one another. Equation (1.1) is evaluated first to obtain the hydraulic gradient, and equation (1.2) is used to obtain the seepage velocity. Then the seepage velocity is used, with the other appropriate parameters in the solute transport equation (1.3), to calculate the solute concentration as a function of time and space. The physical processes of advection, dispersion, and sink-source relationships and the chemical processes expressed by the chemical-reaction term are the most significant influences on concentration when computed using equation (1.3).

1.5. MIGRATION AND EVOLUTION OF POLLUTANTS IN SATURATED ZONES

The migration and evolution of pollutants in groundwater depend on three sets of factors:

1) mechanical factors: groundwater velocities, density and viscosity of pollutants, geometry of sources of pollution, duration and zone of contact between sources of pollution and water, dilution of pollutants, mechanical percolation, pumping policies and, more generally, water resource management policies;
2) geological, structural and geographical factors: groundwater geology, stratification, types of aquifer, lithology, general hydrology and climatology in time and space;
3) physico-chemical, chemical and biochemical factors: physicochemical or chemical properties of the medium and of the pollutants; self-purifying processes.

These various factors act within the framework or a fairly general theory of groundwater pollution, the main lines of which are sketched out below.

In order for substances to act as pollutants, they must be either in solution or suspended in water. In the latter case, such substances appear as traces or as light particles carried by the water and behaving like tracers miscible with water. To give an example, the products of leaching by water of hydrocarbon lenses afford a good illustration of this kind of pattern. The evolution of pollutants is in line with a theory of fluid flows miscible with groundwater characterized by the following phases:

1) convection of the pollutant, i.e. movement of the pollutant with mean groundwater flow;
2) dispersion of the pollutant, i.e. scattering of the particles of the pollutant through mixing or spreading around the mean flow;
3) chemical or physico-chemical reactions of the pollutant and the solid matrix of the porous medium;
4) biochemical reactions of the pollutant and its environment, in the form, for instance, of bacterial biodegradation;
5) reactions in the pollutant, for instance radioactive decay.

These various phases do not necessarily all occur. However, convection or dispersion generally takes place, and together they form the mechanical basis of the theory of pollutant movement in groundwater. Convection and dispersion have been studied very fully both in the laboratory and in the field, in the context of the theory of hydrodynamic dispersion.

For this phenomenon to be clearly understood, a few very straightforward examples first need to be considered.

Example 1: a porous medium saturated with water, contained in a cylindrical tube. At time t = 0, a chemical product dissolved in water at concentration c_0 is injected into the tube. The concentration profile is a step function at t = 0, movement is unidirectional, at constant rate of inflow and without physicochemical interactions between liquid and solid. It is observed that the concentration of the injected product varies with time and the concentration profile has the aspect of an S-curve (Fig. 1.1) in a "transition zone" between the zones at concentration c_0 and zero.

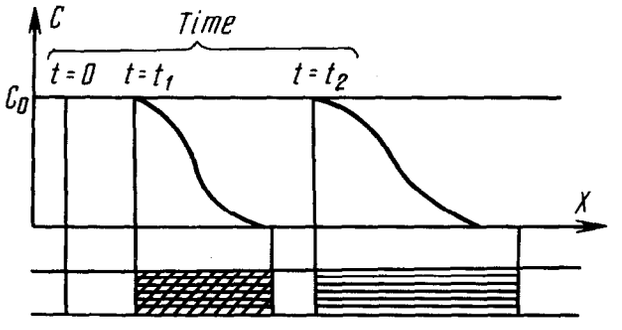


Fig. 1.1. Distribution of concentration and formation of transitional zone in a unidimensional flow.

Example 2: an aquifer in uniform flow. A volume of marked water (brine, for instance) is injected through a well for a relatively short period of time. A shift in the centre of gravity of the dispersed tracer is observed at the rate of flow of the aquifer and, simultaneously, the traced water is seen to spread laterally and in the direction of the flow (Fig. 1.2).

In some zones, the movement of the tracer is thus faster than the mean flow of the aquifer; in other zones, on the contrary, it is slower.

Example 3: the experiment described in Example 1, but in this case the injected liquid is not miscible with water (oil, for instance). The water-injected liquid interface moves at the rate of inflow and the concentration profile at any particular moment is a step function. This is a case of "pure convection" (Fig. 1.3).

Darcy's law, which governs mean groundwater flow, cannot account for the appearance of the transition zone or the planar spread, which is both longitudinal in the direction of the mean groundwater flow and lateral to that flow. It is necessary to descend to the level of the pore to find an explanation for these phenomena which are given the name of hydrodynamic dispersion and which are due to the superposition of:

1) mechanical action linked to variations in the distribution of local velocities in the porous medium: wall effects creating a velocity gradient in a pore, variations in pore dimensions leading, by Venturi effect, to variations in velocity, and fluctuations of streamlines in relation to the mean direction of flow (Fig. 1.4);
2) molecular diffusion, which tends to regularize variations in concentration through the migration of particles within the liquid (Fig. 1.5).

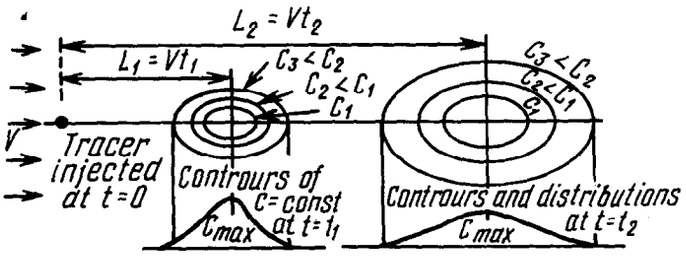


Fig. 1.2. Formation of a contamination halo in a groundwater flow.

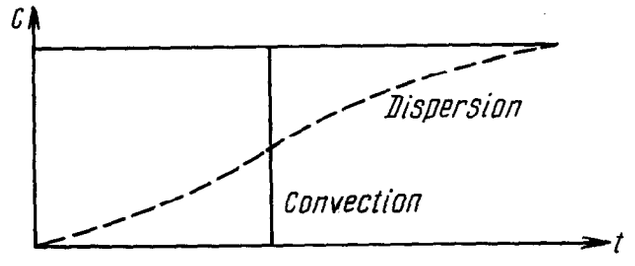


Fig. 1.3. Position of a front of convective transfer in a zone of dispersion.

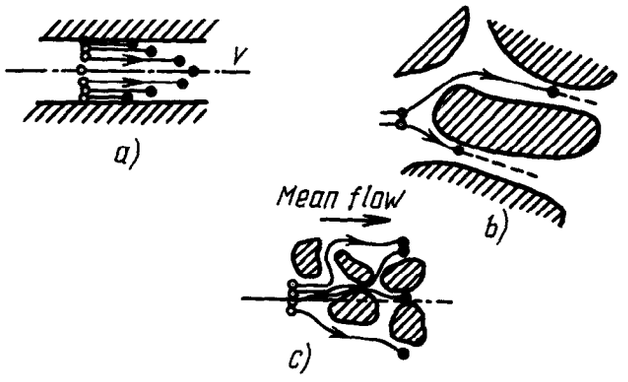


Fig. 1.4. Diagram of the deviation of stream lines from the direction (vector) of mean flow:
a — velocity distribution in a single pore of constant size;
b — branching of flow lines;
c — deviation of lines of flow in pores of different configuration from the direction of mean flow.

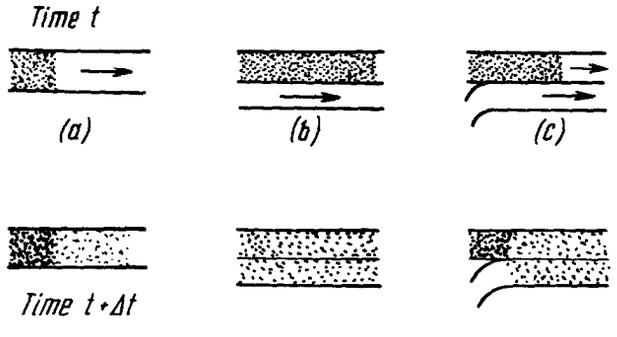


Fig. 1.5. Effect of molecular diffusion on changes in the concentration of a substance in the flow of a fluid.

These various elementary phenomena reveal a geometrical aspect of the theory of dispersion, suggested by Example 2, which will be turned to account in designing and applying mathematical models of groundwater pollution. Two basic dispersive effects exist, one in the mean direction of flow, or longitudinal dispersion, the other in a direction orthogonal to the mean direction of flow, or lateral dispersion.

To these phenomena concerning local velocities at the level of the pore or at the microscopic level should be added variations in mean rates of flow, at the macroscopic level, due to heterogeneities (reflected in variations in permeability) in the aquifer, giving rise, in respect of mathematical description and the forecasting of movements, to macrodispersion.

To sum up, according to the theory of hydrodynamic dispersion, which serves as a basis for the description of the mechanisms of pollutant migration and evolution in groundwater, the distribution of the pollutant in water tends to become homogeneous. If the pollutant initially occupies a region A separated from the rest of the water by an abrupt interface, this interface disappears, and is replaced by a transition zone where the concentration of the pollutant varies from its value in A to zero. Dispersion influences the evolution of pollution in two ways: it dilutes the pollutant and reduces the concentration peaks; it accelerates the arrival of pollution in relation to the arrival of a front of the same pollution assumed to be immiscible.

It has been demonstrated in the laboratory that hydrodynamic dispersion depends on the mean velocity of the mixture, fluctuations in velocity, porosity, grain-size distribution, consolidation, the specific mass and miscibility of the fluids, the length of the displacement and the molecular diffusion. In the field, the mechanisms are complicated by heterogeneities and preferential paths. The validity of the description depends on the scale of the phenomena. Over large distances and for long periods, the orders of magnitude of the characteristic parameters of dispersion are very different from those in the laboratory (macrodispersion). Furthermore, on a large scale, it will sometimes be possible to disregard the spreading effect, i.e. the transition zone in relation to the mean displacement, and to think in terms of convection alone.

Dispersion in the subject of mathematical modelling based on a general diffusion equation:

$$\text{div}\left(k_\rho \,\text{grad}\frac{c}{\rho}\right) - \text{div}\,(vc) = \frac{\partial c}{\partial t} \qquad (1.4)$$

where c is the concentration of the pollutant in the mixture, k the dispersion tensor, v the pore velocity, t the time and ρ the specific mass of the mixture.

Of course, if dispersion is discounted, the foregoing equation becomes a pure convection model in the form:

$$-\text{div}\,(vc) = \frac{\partial c}{\partial t} \qquad (1.5)$$

The various mechanical or geological factors mentioned at the beginning of this section enter into the model either through the parameters or variables of the equation or under the initial and boundary conditions chosen in order to solve this equation.

Physicochemical, chemical and biochemical conditions, like adsorption, biodegradation or radioactive decay, are brought into the model by means of additional terms in the general equation. For instance, a dispersive flow at constant mean specific mass with adsorption of

the pollutant on the pore matrix will be represented by the following equation:

$$\text{div}\,(\mathbf{k}\ \text{grad}\ C) - \text{div}\,(vc) = \frac{\partial c}{\partial t} + \rho_s \frac{1-n}{n}\frac{\partial s}{\partial t} \qquad (1.6)$$

where n is the porosity, s the pollutant mass adsorbed per solid mass unit and ρ_s the specific mass of the solid.

The model adopted reveals the characteristic parameters of pollutant movement: the rate of flow, already referrred to several times, and the dispersion tensor K, which measures the extent of the dispersion. Mathematically, tensor K is a square matrix (3 x 3) whose nine components, known as dispersion coefficients, can be reduced to three by diagonalization (longitudinal and lateral dispersion coefficients). The dispersion coefficients are measured in the field and their numerical values vary according to the heterogeneity of the aquifer. They may attain very large values in comparison with similar measurements made in the laboratory, which is why dispersion is referred to in the field as macrodispersion.

Macrodispersion is at the present time a subject of research, particularly as regards the behaviour of dispersion coefficients according to the scale on which the groundwater is studied in time and space.

In conclusion, the migration and evolution of pollutants in saturated zones depend on a large number of factors which can be linked together in a general theory largely based on the theory of the flow of fluids miscible with water in a porous medium, or dispersion theory. This theory provides methodological guidance for the study of the behaviour of pollutants in groundwater, particularly through the building of mathematical models of dispersion-convection and other physicochemical phenomena. However, this is still an area of research and although the basic phenomena are beginning to be clearly understood, it is very difficult to predict the behaviour of pollutants; the use of mathematical models at the forecasting stage is still somewhat unreliable, owing particularly to the difficulty of measuring dispersion coefficients.

1.6. THEORETICAL MODELS OF MASS AND HEAT TRANSFER IN GROUNDWATER

In section 1.5 the various physical or chemical phenomena affecting mass transfer in groundwater were described. In order to gain a more systematic understanding of these phenomena, and describe and predict in quantitative terms their interactions, models of them have to be constructed or in other words they have to be represented in a more practical form for study. Modelling is envisaged here with the help of mathematical tools.

Mathematical models are generally classified in three groups according to the initial degree of knowledge possessed about the structure of the aquifer:

- Black-box models, when nothing at all is known about the structure of the aquifer. This is described as a system without particular structure, the pollution inputs and outputs alone being known experimentally and their linkages being studied with the help of a fairly general mathematical formulation based on simple hypotheses, as for instance the convolution relation or the mass conservation equation. These models are generally characterized by a transfer function which, combined with the input function, provides the output function, and whose form and numerical values are determined by adjusting the model to existing data. These are global models, usually serving the purposes of description rather than forecasting.
- Grey-box models describing the aquifer as a system possessing a small number of structural properties, for instance, a series of reservoirs with various patterns of behaviour. These models have a transfer function whose form depends on the structural hypotheses and whose parameters are determined by adjustment to existing data. These are also global models.
- Full structural models showing the various aspects of pollution transfer and capable of forecasting its spatio-temporal distribution. Traditional models are systems of partial differential equations describing dispersion and convection by means of a general diffusion equation and also able to describe other phenomena, whether chemical, biochemical or linked to radioactivity, by adding to the diffusion equation various functions and their derivatives. Of course, full structural model does not mean precise knowledge of the medium and its behaviour. Owing to a large number of incertainties connected with the actual structure of the medium and the experimental approach to it, probabilistic notions are introduced into the model which will, of course, influence the choice of mathematical methods of treatment.

In fact, if the spatial variations of the parameters or variables of the state of the system are known, whether they are continuous or limited to a number of points in the system, it is possible to envisage a distributed parameter model. If these spatial variations are not known and only mean values in space are computed, the result is a lumped parameter model.

The methods of computation depend on the types of simulation adopted. In general, they will be statistical methods or convolution or Fourier analysis for the black-box models, and functional analysis and numerical methods for solving parabolic equations for structural models.

1.6.1. METHODS OF SIMULATION

Black-box models

Let e(t) be the input and s(t) the output of the system; assuming the system to be linear and stationary (unvarying by translation in time), e(t) and s(t) can be linked together through a kernel convolution operator h(t) (h(t) and its Fourier transform are known respectively as impulse-response and transfer function:

$$s(t) = h(t) + e(t) = \int_0^\infty h(t-\tau)e(\tau)d\tau \qquad (1.7)$$

The object is to determine h on the basis of functions s and e measured experimentally. To do this, there are many numerical methods, which relate to problems of stability and sensitivity to experimental errors, such as the Fourier transformation or Walsh transformation [35, 32] developed essentially in order to identify hydrogeological parameters, transmissivities and storage coefficients. For mass transfers, mention should be made of an original approach developed by Migault [36] which leads to the best approximation of the kernel h on the basis of a family of specific generators by translating the input concentration into a suitable Hilbert space and by resolving the matrix system obtained through successive projections onto a series of subspaces.

Uncertainties concerning the experimental input or output data, due in particular to the spatial variability of

the hydraulic parameters or to the various heterogeneities, have led some authors to introduce probabilistic notions into the black boxes. For instance, h can be thought of as a random variable. Thus Jury [29], studying solute transfer from land surface to a variety of depths, has introduced a transit time probability density, which is a function of time or of the quantity of water at the entrance, making it possible to forecast concentrations at various depths after calibration on an experimental measurement of residence time.

Traditional statistical methods analysing input-output correlations can of course also be used. For instance, the spectral method introduces the crossed spectral density function for the two series of measurements in time (or processes) in input and output (Fourier transform of crossed covariance), of which the real part or cospectrum and the imaginary part or quadratic spectrum enable the displacement phase and coherence to be computed. The displacement phase is the time necessary for the first process to correspond to the second and the coherence, a criterion of the dependence between processes, reveals whether the series of data are linked, whether a noise exists and interfaces with the data, and whether the system is not linear or whether the output derives from other inputs.

Grey-box models

The basic methodology consists in representing the medium by a series of mixing cells or reservoirs, in other words, in linking together mathematically inputs and outputs by a linear differential continuity equation of the first order in the form

$$e(t) - s(t) = \frac{dR(t)}{dt} \text{ with } R(t) = ks(t) \qquad (1.8)$$

where R(t) is the reserve of the system and k the storage coefficient. The system constituted by this continuity equation and the transfer equation s = h x e provides a solution in the form

$$h(t) = \frac{1}{k} e^{-t/k} \qquad (1.9)$$

(with the condition s(o) = 0)

The next step is to generalize to N reservoirs. Adjusting the model consists in determining N and K while minimizing the rms deviation between real outputs and outputs computed according to certain hypotheses.

Collonques [17] has proposed a method combining spectral analysis and grey-box modelling, determining a coherence value, a mean transfer time and applying the cascade reservoir model.

Structural models

Structural models of transport phenomena in groundwater are built in two stages [20]:

1) by various conceptual approaches, continuous media equivalent to homogeneous porous media in the laboratory are defined. A diffusion equation is then constructed representing the dispersion process at laboratory scale, that is to say, the scattering of solute particles in relation to a mean direction of flow due to molecular diffusion and velocity variations in the pore space;
2) an attempt is then made to extend the laboratory findings to the field. In practical terms, this means considering the analogy between dispersion due to velocity variations in the pore space and dispersion due to large-scale heterogeneities in the continous medium.

Although the traditional approach provides models in the form of partial differential equations that can be solved analytically or numerically, it gives rise to many difficulties, which we shall be considering, that make it desirable to find other methods for representing macrodispersion.

The usual scheme of transport in groundwater consists of: 1) the transport equation (or dispersion equation), 2) Darcy's equation, 3) the continuity equation and 4) various state equations:

$$\text{div}\left(K\rho \,\text{grad}\frac{c}{\rho}\right) - \text{div}(vc) = \frac{\partial c}{\partial t} \qquad (1.10)$$

$$v = -\frac{k}{\mu\phi}(\text{grad } P + \rho g \,\text{grad } z) \qquad (1.11)$$

$$\text{div}(\rho u) + \frac{\partial \rho}{\partial t} = 0 \qquad (1.12)$$

$$\rho = \rho(c),\ \mu = \mu(c) \qquad (1.13)$$

where c is the solute (or pollutant) concentration, g the gravity, K the dispersion tensor, k the permeability coefficient, P the pressure, t the time, v the pore velocity, z the vertical co-ordinate, ϕ the porosity, μ the dynamic viscosity, and ρ the specific mass of the mixture.

Once specific initial and boundary conditions have been set for each problem, this system can be resolved case by case, and according to the possibilities for computation by analytical or numerical means, with the help of the methods described in the following paragraph. However, inherent in the use of this system are a large number of difficulties of conceptual and mathematical origin that have to be borne in mind when proceeding with the simulations.

The conceptual difficulties generally concern the conventional scheme's inadequacy in representing dispersion phenomena, particularly in the field. In the laboratory, systematic deviations can be revealed between the experimental results and the solutions computed on the basis of the corresponding model; in the field, one finds that the orders of magnitude of the dispersion coefficients determined experimentally are far greater than those in the laboratory and that those coefficients vary with the size of the domain studied. Lastly, many cases exist in which the pollutants are transported preferentially in layers that are relatively more permeable than others, by a process not of diffusion but of fingering without mixing. Confirming these experimental findings, theoretical studies, using either regularizing methods by spatial averaging in elements of volume representative of the porous medium, or stochastic methods with overall averaging over suitable probabilistic spaces, have shown that a dispersion process is not necessarily of a diffusive type and that the emergence of a diffusive regime depends on the possibilities for the solutes being well-mixed, and particularly on there being sufficient mixing time.

The mathematical difficulties are connected with the very nature of the dispersion scheme constituted by interdependent non-linear equations (for the dispersion tensor depends on velocities and hence on concentrations), and with the behaviour of these equations in the numerical schemes used, in particular the appearance of oscillations and parasitic diffusion.

Lastly, one should not forget the difficulties bound up with the uncertainties regarding the experimental data allowing the parameters of the dispersion schemes to be identified.

The difficulties just mentioned have considerably impeded the use of mathematical models of transport in porous media. However, the development of pollution problems has led to further research in this sphere and to an improvement, if not in the models proposed then at least in modelling methods, making it possible also to improve their relevance to actual field conditions. The type of difficulty encountered, connected mainly with the heterogeneous and not easily determined structure of the aquifer, has caused the research effort to be slanted towards stochastic simulations based on work in fluid mechanics on turbulent diffusion in pipes and ducts, of which a few characteristic examples follow, showing that the dispersion regime does not necessarily appear. If the conditions are right for it to appear, it emerges for very long periods of time and the object, in practice, is to evaluate the asymptotic time, that is, the time necessary for a diffusion regime to become established with sufficient approximation, and it is essential to know what this time is in order to determine the scale of the field experiments to measure the dispersion coefficients.

Matheron [34] simulates the movement of tracer particles in a bidimensional flow in a horizontally stratified medium; the velocity is horizontal and constant in each stratum. This movement is the sum of a convective movement at the velocity in the stratum and of a dispersive, Fickian and isotropic movement; the concentration expectation at one point at a given time is the probability density function of a particle at that point at that time. Taking the x-axis parallel to the direction of velocity, Matheron computes the variance of the co-ordinate $x(t)$ of a particle in the light of vertical variations in velocities. He defines the mathematical conditions governing variance in order to obtain a diffusive regime (it should be remembered that a Gaussian process is usually characterized by a variance proportional to time) and he reveals the existence of a large number of cases where, even over very long periods of time, a diffusive regime will not emerge: for instance, a stratified flow at horizontal velocities in an infinite bidimensional medium, where convection does not result in any mixing; in the field, these are flows that follow more or less parallel preferential paths. Pursuing this approach further, Matheron and Marsily [34] consider that the direction of flow is no longer parallel to the stratification and reveal the emergence of a diffusive regime on the basis of hypotheses which, although relatively weak, apply to infinite periods of time. Referring to examples of aquifers of a common type, the authors show that the usual methods for measuring dispersion coefficients for fairly short periods of time do not give good results. A possible approximation for a point source is to use a diffusion equation with dispersion coefficients dependent on time with the following significance: if $t \rightarrow K(t)$ is the dispersion coefficient, $K(t_1)$ will be the value to be used to integrate the diffusion equation between 0 and t_1 in order to obtain a good spatial description of pollutant concentration at time t_1. The need to start again for each moment in time makes this an unwieldy method.

Another interesting approach is proposed by Gelhar et al [23] for a bidimensional flow in a horizontally stratified medium, the velocities being horizontal and constant in each stratum. Using a conventional method of turbulence analysis, the authors construct a concentration perturbation equation, bringing to the fore the Reynolds term $\overline{u'c'}$, this being the expected product of the velocity and concentration perturbations. By spectral analysis they link the Reynolds flux to a mean concentration gradient through a "dispersion coefficient", which is the product of the mean velocity and of dispersivity dependent on time. With Gelhar's hypotheses, this dispersivity tends towards a limit for very long periods of time, leading to the emergence of a diffusive regime and the possibility, in theory at least, of making evaluations in asymptotic time. This approach has been generalized to heterogeneous media in three dimensions [24] formulating the dispersion coefficients in accordance with the statistical parameters of the field situation, including the variance of the logarithms of permeability, which in most cases can be reasonably estimated. Having applied their formulae to a few field situations and compared them with results obtained by other means, the authors feel that their method provides an interesting basis for the prediction of long-term, large-scale transport in field situations where the hydraulic parameters and their variations can be correctly assessed.

Another approach develops a systematic basis for taking into account spatial heterogeneities of permeability. Smith and Schwartz [47] propose a stochastic process consisting in a deterministic movement of tracer particles with flow velocities in a heterogeneous macroscopic porous medium with permeabilities, combined by addition with a random movement of such particles representing the anisotropic microscopic dispersion. A Monte Carlo simulation is used to produce a large number of realizations of a field of permeabilities regarded as random variables with a probability density in each point of the domain and such that near values of permeability are autocorrelated spatially. Each realization conserves the experimental data.

The particle movement described above occurs in each realization. The spatial variance of all the particles in it is computed and expressed as a function of time. As the diffusive regime shows a variance proportional to time, the ratio of the number of realizations in which variance depends linearly on time to the total number of realizations is computed. Thus, on the basis of 300 realizations, the authors note that less than 20 per cent behave in a diffusive manner, which means that in a large number of cases the velocities of the particles are not subject to a sufficient number of variations for correct application of the central limit theorem. From a practical point of view, it can be concluded that it is often wrong to build models of field situations with the help of a general diffusion equation incorporating large diffusion coefficients, and that an attempt should be made to use a representation of permeability heterogeneities and the resulting mixture. It should be pointed out, however, that the simulation of an aquifer with the help of Monte Carlo techniques, which, in theory, is quite possible on the basis of adequate knowledge of the hydraulic parameters of the medium, cannot be easily achieved in practice owing to the amount of computer time required for such simulation. For this reason, Tang, Smith and Schwartz [51] propose a more direct approach, considering velocity to be a random variable, without seeking to link it to the heterogeneities of the medium. By a method of perturbation and overall averaging they transform the diffusion-convection equation

$$\frac{\partial \phi c}{\partial t} + v \text{ grad } c = D \text{ div } (\phi \text{ grad } c) \qquad (1.14)$$

into the mean equation

$$\frac{\partial \phi \bar{c}}{\partial t} + (\phi \bar{v}_j - D \frac{\partial \phi}{\partial x_j}) \frac{\partial \bar{c}}{\partial x_j} = (D\phi\delta_{jk} + \phi\rho_{jk}) \frac{\partial^2 \bar{c}}{\partial x_j \partial x_k} \qquad (1.15)$$

where D is the coefficient of molecular diffusion, constant, v the convection velocity of components v_j, random variable of x and t, v the Darcy velocity, x the vector of the Cartesian co-ordinates x_i, the origin of the datum point being the source of pollution, $\bar{c}$ the overall mean concentration, δ_{jk} the Kronecker delta, ϕ the porosity, ρ_{ik} a coefficient of overall dispersion defined by the relation

$$\rho_{jk} = \frac{1}{2} \frac{\overline{v'_j v'_k}}{\bar{v} \cdot \bar{v}} x \cdot \bar{v} \qquad (1.16)$$

where $\overline{v'_j . v'_k}$ represents the covariance of two neighbouring velocities and v' the perturbation of the velocity v in relation to the mass velocity $\bar{v}$. It may be noted that when the neighbouring velocities are not correlated, the dispersion coefficients are increasing functions of the distance covered from the source. The mean equation can be solved either analytically in certain straightforward cases (for instance in one-dimensional flow, by Laplace transformation) or by the stochastic method described earlier, the movement of the particles being the combination of a deterministic displacement at the velocity of convection and a random displacement of dispersion with overall dispersion coefficients through node spacing of the domain, and concentration being computed on the basis of the number of particles at each of the nodes.

The models just described provide interesting possible solutions to the mathematical study of mass transfer in porous media, but they call for high performance computers. Following the same line of thinking, Dagan [18] explicitly computes the solution to a simple problem of transport in a uniform mean flow through unbounded formations, obtaining expressions of concentration expectation and variance, hence without having recourse to the excessively large numerical computation.

Mention is to be made, latsly, of a solution proposed by Pickens and Grisak [39], who use the deterministic dispersion scheme taking into account the asymptotic time needed for the establishment of the diffusive regime, and propose a dispersivity dependent on time, constant at all points of the system at all moments. On the basis of the conventional relations

$$K = \frac{1}{2} \frac{d\sigma^2}{dt}, \quad K = d_L v, \quad \bar{x} = vt \qquad (1.17)$$

where K is the dispersion coefficient, σ^2 the space variance of the tracer or pollutant distribution, t the time, d_L the longitudinal dispersivity, v the pore velocity, $\bar{x}$ the mean distance covered by the tracer or the pollutant during time t, introducing d_L as a function f of the distance covered, one obtains

$$\sigma^2 = 2 \int f(\bar{x}) \, d\bar{x} \qquad (1.18)$$

where f can be chosen on the basis of experimental results. Thus the authors propose a variety of formulae, like $d_L = d\bar{x}$ and $\sigma^2 = d\bar{x}^2$ or $d_L = a\bar{x}^b$ and $\sigma^2 = 2a\frac{\bar{x}^{b+1}}{b+1}$, where a and b are constants. The dispersion equation thus obtained is solved by a finite-element method.

1.6.2. METHODS FOR RESOLVING THE DISPERSION SCHEME

Very generally, owing to its complexity and particularly to its non-linearity, the dispersion scheme is resolved numerically on a computer with the help of finite-difference and finite-element methods, characteristics or stochastic simulation, just described. These are traditional methods of numerical analysis applied to parabolic diffusion equations, which are very extensively dealt with in a large number of works and articles and whose main features will be described. However, owing to their practical value, particularly for the use of micro- or mini-computers, analytical methods deserve to be described in some detail, despite their application being limited to fairly simple cases. They also have the advantage of avoiding the serious difficulties of numerical methods, such as overshoot or numerical diffusion.

Analytical methods

Uniform one-dimensional flow

Disregarding variations in specific mass and viscosity, the transport equation is reduced to

$$K_L \frac{\partial^2 c}{\partial x^2} - v \frac{\partial c}{\partial x} = \frac{\partial c}{\partial t} \qquad (1.19)$$

where K_L is the coefficient of longitudinal dispersion and v the uniform pore velocity.

For the following initial and boundary conditions, in a semi-infinite medium

$$c(x=0, t>0) = c_0 \quad c(x\infty, t\geqslant 0) = 0 \quad c(x\geqslant 0, t=0) = 0$$

by Laplace transformation, one obtains

$$\frac{c}{c_0} = \frac{1}{2} \text{erfc}\left[\frac{x - ut}{2(K_L t)^{1/2}}\right] + \exp\left(\frac{ux}{K_L}\right) \text{erfc}\left[\frac{x + ut}{2(K_L t)^{1/2}}\right] \qquad (1.20)$$

$$\text{with erfc } X = \frac{2}{\sqrt{\pi}} \int_X^{\infty} \exp(-y^2) \, dy$$

Given x, for sufficiently long periods of time, the second term becomes negligible in relation to the first and the concentration distribution verifies the relation

$$\frac{c}{c_0} = \frac{1}{2} \text{erfc}\left[\frac{x - vt}{2(K_L t)^{1/2}}\right] \qquad (1.21)$$

Adding the radioactive decay expressed by the coefficient linked to the period t' of the radioactive body by the relation $\lambda = \frac{\ln 2}{t'}$ (t' is the half-life) the transport equation becomes:

$$K \frac{\partial^2 c}{\partial x^2} - v \frac{\partial c}{\partial x} = \frac{\partial c}{\partial t} + \lambda c. \qquad (1.22)$$

the solution of which, for the previous conditions, is (de Marsily, 1981):

$$\frac{c}{c_0} = \frac{1}{2} \exp\left(\frac{Vx}{2K}\right) \left\{ \exp(-\beta x) \text{ erfc}\left[\frac{x - t(V^2 + 4\lambda K)^{1/2}}{2(Kt)^{1/2}}\right] + \right. \qquad (1.23)$$

$$\left. + \exp(\beta x) \text{ erfc}\left[\frac{x + t(V^2 + 4\lambda K)^{1/2}}{2(Kt)^{1/2}}\right] \right\}$$

$$\text{with } \beta = \left(\frac{V^2}{4K^2} + \frac{\lambda}{K}\right)^{1/2}$$

In an infinite medium, instantaneous point injection of a mass M of stable tracer. The system becomes

$$K\frac{\partial^2 c}{\partial x^2} - V\frac{\partial c}{\partial x} = \frac{\partial c}{\partial t} \qquad (1.24)$$

$$c(x,o) = \frac{M}{\phi}\delta(x)$$

with $\delta(x)$ the Dirac measurement and ϕ the porosity

$$c(\infty,t) = 0$$

$$\int_{-\infty}^{+\infty} c(x,t)\,dx = \frac{M}{\phi} \qquad (1.25)$$

The solution is written as follows [14]:

$$c = \frac{M/\phi}{2(\pi Kt)^{1/2}} \exp\left[-\frac{(x-vt)^2}{4Kt}\right] \qquad (1.26)$$

In an infinite medium, continuous, constant point injection of stable tracer: according to the foregoing formula, if dM = Co v dt is the mass of tracer injected during time dt, the concentration at time t will be:

$$\frac{c}{c_0} \equiv \frac{V}{2(\pi K)^{1/2}} \int_0^t \frac{1}{\sqrt{\theta}} \exp\left[-\frac{(x-v\theta)^2}{4K\theta}\right] d\theta \qquad (1.27)$$

Uniform bidimensional flow

The velocity is parallel to ox and constant. The dispersion equation is written as follows:

$$K_L\frac{\partial^2 c}{\partial x^2} + K_T\frac{\partial^2 c}{\partial y^2} - V\frac{\partial c}{\partial x} = \frac{\partial c}{\partial t} \qquad (1.28)$$

The Green function, i.e. the solution of this equation for the injection of an originally unit amount of pollution, instantaneously at t = 0, is:

$$G(x,y,t) = \frac{1}{4\pi t(K_L K_T)^{1/2}} \exp\left[-\frac{(x-Vt)^2}{4K_L t} - \frac{y^2}{4K_T t}\right] \qquad (1.29)$$

For an injection at concentration c_0, at rate Q during time dt, the solution becomes

$$\frac{c}{c_0} = GQdt \qquad (1.30)$$

For continuous injection, the solution at time t is then

$$\frac{c}{c_0} = \frac{Q}{4\pi(K_L K_T)^{1/2}} \int_0^t \exp\left[-\frac{(x-V\theta)^2}{4K_L\theta} - \frac{y^2}{4K_T\theta}\right]\frac{d\theta}{\theta} \qquad (1.31)$$

If t tends to infinity, an asymptotic regime is obtained characterized by the relation:

$$\frac{c}{c_0} = \frac{Q}{4\pi(K_L K_T)^{1/2}} \exp\left(\frac{xv}{2K_L}\right) K_0\left[\left(\frac{v^2x^2}{4K_L^2} + \frac{v^2y^2}{4K_L K_T}\right)^{1/2}\right] \qquad (1.32)$$

where K_0 is the modified Bessel function of the second kind and zero order.

Radial flow

Apart from a few approximate solutions, there exists no complete analytical solution to this type of problem other than that proposed by Tang and Babu (1979) [49].

The flow in question is due to a well of radius r_0 injecting at constant rate Q in a horizontalle infinite, homogeneous, isotropic aquifer of thickness b between two impermeable layers. The pore velocity at a point at a distance r from the well is

$$v = \frac{Q}{2\pi br\phi} = \frac{A}{r} \text{ with } A = \frac{Q}{2\pi b\phi} \qquad (1.33)$$

The transport equation is written

$$\frac{\alpha A}{r}\frac{\partial^2 c}{\partial r^2} - \frac{A}{r}\frac{\partial c}{\partial r} = \frac{\partial c}{\partial t} \qquad (1.34)$$

assuming $K_{radial} = \alpha v$. It is completed by the following initial and boundary conditions:

$$c(r \geqslant r_0,\ t=0) = 0$$

$$c(r=r_0,\ t>0) = c_0$$

$$c(r\infty,\ t>0) = 0$$

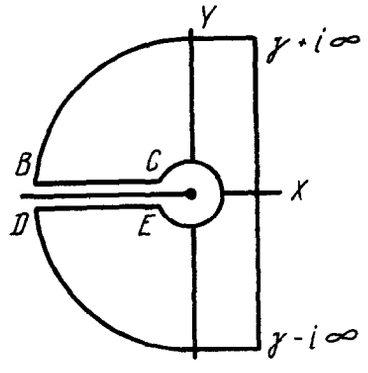
By Laplace transformation α, this equation becomes

$$\frac{d^2\bar{c}}{dr^2} - \frac{1}{\alpha}\frac{d\bar{c}}{dr} - \frac{Pr}{\alpha A}\bar{c} = 0 \qquad (1.35)$$

with $\bar{c} = \alpha(c)$

By inverse transformation on the following contour where

$$\gamma = \frac{2}{3}\left(\frac{r_0^3}{\alpha A}\right)^{1/2},\ \beta = \frac{2}{3}\left(\frac{r^3}{\alpha A}\right)^{1/2}$$



the following expression is obtained

$$\frac{c}{c_0} = \left(\frac{r}{r_0}\right)^{1/2} \exp\left(\frac{r-r_0}{2\alpha}\right) \cdot \frac{1}{2\pi i}\int_{\gamma-i\infty}^{\gamma+i\infty} \exp(pt) \cdot \frac{(P+\frac{\alpha}{r})^{1/2}}{(P+\frac{\alpha}{r_0})^{1/2}} \frac{K_{1/3}\left[\frac{\beta}{P}(P+\frac{\alpha}{r})^{3/2}\right]}{K_{1/3}\left[\frac{\gamma}{P}(P+\frac{\alpha}{r_0})^{3/2}\right]} \frac{dP}{P} \qquad (1.36)$$

where $K_{1/3}$ is the modified Bessel function of the second kind of the order 1/3, an expression which can be reduced to integrals on the real axis.

In particular, simpler approximate expressions can be deduced from this expression for fairly short or fairly long periods of time, or respectively

$$\frac{c}{c_0} \sim \left(\frac{r}{r_0}\right)^{1/4} \exp\left(\frac{r-r_0}{2\alpha}\right) \operatorname{erfc}\left(\frac{\beta-\gamma}{2t^{1/2}}\right) \qquad (1.37)$$

and

$$\frac{c}{c_0} \sim 1 - \exp\left(\frac{r-r_0}{2\alpha}\right) \cdot \frac{I_4+I_5}{\pi} \qquad (1.38)$$

with

$$I_4 = \int_{\frac{\alpha}{r}}^{\frac{\alpha}{r_0}} \frac{dx}{x}\left[1 + \frac{(r-r_0)x}{4\alpha}\right] \exp\left[-tx - \frac{\gamma}{x}\left(\frac{\alpha}{r_0} - x\right)^{3/2}\right] \cdot \sin\left[\frac{\beta}{x}\left(x - \frac{\alpha}{r}\right)^{3/2}\right] \qquad (1.39)$$

$$I_S = \int_{\frac{\alpha}{r_0}}^{\infty} \frac{dx}{x}\left[1 + \frac{(r-r_0)x}{4\alpha}\right] \exp(-tx) \cdot$$

$$\cdot \sin\left[\frac{\beta(x-\frac{\alpha}{r})^{3/2} - \gamma(x-\frac{\alpha}{r_0})^{3/2}}{x}\right] \qquad (1.40)$$

Flow in media of a particular geometrical structure

In stratified media. Al-Niami and Rushton [13] propose an analytical solution for the case of a flow parallel to the strata of length L and for the case of a flow perpendicular to the strata of lengths L_1, L_2, L_3. In the first case, the system of dispersion equations is written as follows:

$$K_x^i \frac{\partial^2 c^i}{\partial x^2} + K_z^i \frac{\partial^2 c^i}{\partial z^2} - v^i \frac{\partial c^i}{\partial x} = \frac{\partial c^i}{\partial t} \qquad (1.41)$$

where i is the index of the strata (i = 1,2 in the example with two strata dealt with). It is completed by the following initial and boundary conditions for two strata:

$$c^i(x,z,o) = 0 \quad c^i(o,z,t) = c_0 \quad \frac{\partial c^i}{\partial x}(L,z,t>0) = 0 \qquad (1.42)$$

$\frac{\partial c^i}{\partial z} = 0$ on the outer edges of the strata

$c^1 = c^2$ and $K_z^1 \frac{\partial c^1}{\partial z} = K_z^2 \frac{\partial c^2}{\partial z}$ at the interface between the strata.

In the second case, the system of dispersion equations and its initial and boundary conditions are written as follows:

$$K_x^i \frac{\partial^2 c^i}{\partial x^2} - v^i \frac{\partial c^i}{\partial x} = \frac{\partial c^i}{\partial t} \quad i = 1, 2, 3 \text{ with } \frac{\partial c^i}{\partial x}(L_i,t) = 0 \qquad (1.43)$$

$$c^i(x,0) = 0; c^1(0,t) = c_0; c^2(L_1,t) = c^1(L_1,t);$$
$$c^3(L_2,t) = c^2(L_2,t);$$

The solutions are obtained by Laplace transformation.

For fissured media, interesting analytical solutions have been computed, in particular by Rasmuson and Neretnieks [43, 44], to study the migration of radioactive elements in the fissures of rocks surrounding an underground deposit, taking account of particle diffusion in the microfissures of the rock masses (system with double porosity). The dispersion scheme becomes:

$$(i) \frac{\partial c_f}{\partial t} + v_f \frac{\partial c_f}{\partial z} - K_L \frac{\partial^2 c_f}{\partial z^2} = -\frac{1-\epsilon_f}{\epsilon_f} \frac{\partial \bar{q}}{\partial t} - \lambda_d c_f \qquad (1.44)$$

an equation describing the flow, adsorption and radioactive decay in the fissures, with c_f concentration in the liquid of the fissures, $\bar{q}$ the unit mean concentration in a rock mass, v_f the mean velocity in the fractures, ϵ_f the porosity of the fissures, λ_d the constant of radioactive decay.

$$(ii) \frac{\partial c_p}{\partial t} = \frac{\epsilon_p D_p}{K}\left(\frac{\partial^2 c_p}{\partial r^2} + \frac{2}{r}\frac{\partial c_p}{\partial r}\right) - \lambda_d c_p \qquad (1.45)$$

an equation describing the diffusion within the particles, with c_p concentration in the liquid of the microfissures, D_p the diffusivity in the water of the pores, K a constant of volume equilibrium ($q_i = KC_p$ where q_i is the internal concentration of the rock masses), r the radial distance calculated from the centre of a mass assimilated to a sphere, and ϵ_p the porosity of the rock matrix.

A solution of this system is proposed for the following initial and boundary conditions:

$$c_f(0,t) = c_0 e^{-\lambda_d t}; \; c_f(\infty,t) = 0; \; c_f(z,0) = 0;$$

$$c_p(0,z,t) < \infty; \; c_p(r_0,z,t) = c_{p/r=r_0} \text{ given by}$$

$$\frac{\partial \bar{q}}{\partial t} = \frac{3k_f}{r_0}(c_f - c_{p/r=r_0}); \; c_p(r,z,0) = c_s(r,z,0) = 0 \qquad (1.46)$$

with c_s the concentration in the solid, k_f the coefficient of mass transfer, and r_0 the effective radius of the sphere.

For a stable kind of pollutant ($\lambda_d = 0$), Laplace transformation gives:

$$\frac{c_f}{c_0} = \frac{1}{2} + \frac{2}{\pi}\int_0^{\infty} \exp\left\{\frac{v_f z}{2K_L} - z\left[\frac{(x'(\lambda)^2 + y'(\lambda)^2)^{1/2} + x'(\lambda)}{2}\right]^{1/2}\right\}$$

$$\cdot \sin\left\{\sigma\lambda^2 t - z\left[\frac{(x'(\lambda)^2 + y'(\lambda)^2)^{1/2} - x'(\lambda)}{2}\right]^{1/2}\right\} \frac{d\lambda}{\lambda} \qquad (1.47)$$

where x' and y' are highly complicated functions of λ and the various parameters of dispersion and the medium (dispersion coefficients, velocities, dimensions of the masses, etc.). From this is deduced the solution with λd different from zero:

$$\left(\frac{c_f}{c_0}\right)_{\lambda_d>0} = e^{-\lambda_d t}\left(\frac{c_f}{c_0}\right)_{\lambda_d=0} \qquad (1.48)$$

Mention is also to be made of another solution proposed by Tang et al [50] for transport in a single fracture with molecular diffusion in the rock matrix and adsorption, represented by the following equations (the z-axis being taken along the fracture):

$$K_z \frac{\partial^2 c}{\partial z^2} - V\frac{\partial c}{\partial z} = \frac{\partial c}{\partial t} + \frac{1}{b}\frac{ds}{dt} + \lambda c + \frac{\lambda}{b}s + \frac{q}{b} \quad 0 \leqslant z \qquad (1.49)$$

where s represents the mass of solute adsorbed per unit length of the surface of the fracture, 2b the width of the fracture, q the diffusive flux perpendicular to the axis of the fracture, λ the constant of radioactive decay. This equation describes the dispersive flow in the fracture.

$$D^1 \frac{\partial^2 c^1}{\partial x^2} = \frac{\partial c^1}{\partial t} + \frac{P_b}{\phi}\frac{\partial s^1}{\partial t} + \lambda c^1 + \frac{P_b}{\phi}\lambda s^1 \quad b \leqslant x \qquad (1.50)$$

where x is the co-ordinate perpendicular to the axis of the fracture, s^1 the mass of solute adsorbed per unit mass of solid in the pore matrix, P_b the mean density of the pore matrix, ϕ the porosity, D^1 the coefficient of molecular diffusion in porous media. This equation represents diffusive transport in the pore matrix.

These general equations can be simplified, particularly with the hypothesis of a linear isotherm ($S = K_f c$ and $S^1 = K_m c^1$, $R = 1 + K_f/b$ and $R^1 = 1 + \frac{P_b}{\phi}K_m$) resolved for the initial and boundary conditions

$$c(0,t) = c_0; \; c(\infty,t) = 0; \; c(z,0) = 0; \; c^1(b,z,t) = c(z,t);$$
$$c^1(\infty,z,t) = 0; \; c^1(x,z,o) = 0$$

by Laplace transformation, providing the following solutions:

$$\frac{c}{c_0} = \frac{\exp(\nu z)}{\pi^{1/2}}\int_e^{\infty} \exp\left(-\xi^2 - \frac{\nu^2 z^2}{4\xi^2}\right)\exp(-\eta z^2)\left\{\exp(-\lambda^{1/2} y)\cdot\right.$$

$$\left.\cdot \operatorname{erfc}\left(\frac{y}{2T} - \lambda^{1/2}T\right) + \exp(\lambda^{1/2} y)\operatorname{erfc}\left(\frac{y}{2T} + \lambda^{1/2}T\right)\right\} d\xi \qquad (1.51)$$

in the fracture

$$\text{with } I=\frac{3}{2}\left(\frac{R}{K_z t}\right)^{1/2};\ T=\left(t-\frac{Rz^2}{4K_L\xi^2}\right)^{1/2};\ y=\frac{\nu^2 K_L z^2 \phi (R^1 D^1)^{1/2}}{bu^2};$$

$$\nu=\frac{v}{2K_L};\ \eta=\frac{\lambda R}{4K_L\xi^2}.$$

$$\frac{c^1}{c_0}=\frac{\exp(\nu z)}{\pi^{1/2}}\int_I^{\infty}\exp\left(-\xi^2-\frac{\nu^2 z^2}{4\xi^2}\right)\exp(-\eta z^2)\left\{\exp(-\lambda^{1/2}y')\cdot\right.$$

$$\left.\cdot\operatorname{erfc}\left(\frac{y'}{2T}-\lambda^{1/2}T\right)+\exp(\lambda^{1/2}y')\operatorname{erfc}\left(\frac{y'}{2T}+\lambda^{1/2}T\right)\right\}d\xi \quad (1.52)$$

in the pore matrix, with

$$y' = y + \left(\frac{R^1}{D^1}\right)^{1/2}(x-b)$$

Of course, a variety of simplifying hypotheses are possible on the basis of these general results, including the steady-state solution which gives the maximum distance of penetration.

Numerical methods

The numerical methods used in practice are the finite-difference of finite-element techniques. Examined by Fried [19], finite differences rely on traditional interative methods by points of the Gauss-Seidel type or over-relaxation, or on block methods of the alternating direction and divided time-step type in explicit, implicit or Crank-Nicolson, or mixed explicit-implicit schemes; spatial discretization is effected on rectangular grids with variable grid intervals.

In order to introduce grids better suited to representing domains of a complex geometrical structure, a great deal of use has been made of finite-element methods (Pinder, Gureghian, Volker et al., Pickens et al., Varoglu et al., [40, 25, 55, 38, 54]. The most widely used techniques are based on weighted residuals, like Galerkin's method, which may be summed up with references to one example, as follows:

$$L(c) = K\frac{\partial^2 c}{\partial x^2} - V\frac{\partial c}{\partial x} - \frac{\partial c}{\partial t} = 0 \quad (1.53)$$

That is, a set of n basic functions $a_i(x)$ verifying the boundary conditions of the dispersion equation, forming the first n functions of a complete system in the domain. An approximated function of c is

$$\tilde{c}(x,t) = \sum_{i=1}^{n} c_i(t)\, a_i(x) \quad (1.54)$$

Carrying forward $\tilde{c}$ to the dispersion equation gives

$$L(\tilde{c}) = K\frac{\partial^2 \tilde{c}}{\partial x^2} - V\frac{\partial \tilde{c}}{\partial x} - \frac{\partial \tilde{c}}{\partial t} \neq 0 \quad (1.55)$$

The object, then, is to determine $c_i(t)$ in order to minimize $L(\tilde{c})$, which is done by projecting $L(\tilde{c})$ on N weighting functions and by cancelling those projections:

$$\int_{\Omega} L(\tilde{c})\,\omega_i(x)\,dx = 0 \qquad i = 1, \ldots N \quad (1.56)$$

Through a careful choice of weighting functions (for instance, the orthogonal base a_i or Dirac measurements at various points of the domain), algebraic equations are obtained that are solved by the traditional methods of matrix systems.

These finite-difference or finite-element methods have given rise to a great number of difficulties, in particular overshoot linked to the choice of time-step and numerical diffusion linked to the hyperbolic character of the convection-dominated dispersion equation. A number of methods have been proposed for overcoming these difficulties, such as introducing factitious dispersion coefficients in order to cancel the effect of numerical diffusion or, for Galerkin's method, polynominal basis functions of an order higher than one, for instance cubic functions. Mention may be made of recent techniques used by Pinder et al (1979) involving cubic Hermite polynomials.

1.6.3. HEAT TRANSFER

At each point of an elementary volume representative of a saturated porous medium, it is possible to define a mean temperature for the solid phase T_f and a mean temperature T_S for the fluid phase. Mathematical modelling of heat transfer depends on the magnitude of the difference between T_S and T_f (Combarnous, 1978).

If the difference between T_S and T_f is considered to be negligible, it can be assumed that there exists only one temperature for the porous medium, T equal to T_S and T_f. The heat transfer is represented by a single equation in an equivalent continuous medium:

$$A - \operatorname{div}(\lambda^* \operatorname{grad} T) - (\rho c)_f V \operatorname{grad} T = (\rho c)^* \frac{\partial T}{\partial t} \quad (1.57)$$

where $(\rho c)^*$ is the specific heat of the equivalent porous medium, $(\rho c)_f$ the specific heat of the fluid at constant pressure, V the filtration rate, $\bar{\bar{\lambda}}^*$ the tensor of equivalent thermal conductivity. The specific heat of the medium can be computed by the relation

$$(\rho c)^* = (1-\phi)(\rho c)_s + \phi(\rho c)_f \quad (1.58)$$

where $(\rho c)_s$ is the specific heat of the solid and ϕ the porosity. The components $\bar{\bar{\lambda}}^*$ can be determined experimentally and depend on the isotropic conductivity λ_0 of the porous medium in the absence of flow and on a macrodispersivity term linked to the heterogeneity of velocity. The following can be written, for instance, by analogy with the dispersion coefficients in a system of specific axes:

$$\lambda_L = \lambda_0 + \beta_L (\rho c)_f |v|$$
$$\lambda_T = \lambda_0 + \beta_T (\rho c)_f |v| \quad (1.59)$$

β_L and β_T being constants.

This approach is the one most commonly used and it seems satisfactory when the filtration rate is not too great and if the solid and fluid phases are dispersed. This is true of the usual type of aquifer.

If the difference between T_S and T_f cannot be disregarded, the transfer between phases needs to be made explicit and the medium has to be regarded as equivalent to two continuous media. Two equations are used:

$$\text{B}\quad \begin{aligned} &\operatorname{div}(\bar{\bar{\lambda}}_s^* \operatorname{grad} T_s) = (1-\phi)(\rho c)_s \frac{\partial T_s}{\partial t} + \alpha(T_s - T_f) \\ &\operatorname{div}(\bar{\bar{\lambda}}_f^* \operatorname{grad} T_f) - (\rho c)_f v \operatorname{grad} T_f = \phi(\rho c)_f \frac{\partial T_f}{\partial t} + \alpha(T_f - T_s) \end{aligned} \quad (1.60)$$

with α the volumetric coefficient of heat transfer, and the tensors of equivalent thermal conductivity $\bar{\bar{\lambda}}_s^*$ taking into account the dispersed structure of the solid matrix and $\bar{\bar{\lambda}}_f^*$ dependent on the hydrodynamic dispersion linked to

the microscopic heterogeneity of the velocity field of the saturating fluid. The coefficient α depends on the velocity of the fluid, the characteristics of the two phases and the texture of the porous medium.

The simulation of heat transfer in a saturated porous medium is, then, based on a system of equations comprising:

the continuity equation $$\operatorname{div}(\rho v) + \frac{\partial}{\partial t}(\rho\phi) = 0 \qquad (1.61)$$

the Darcy equation $$V = -\frac{R}{\mu}\left(\operatorname{grad} p + \rho g \operatorname{grad} z - \frac{P}{\phi}\frac{\partial v}{\partial t}\right) \qquad (1.62)$$

Either the equation A or the system B with μ the dynamic viscosity of the fluid, k the tensor of permeability of the medium.

Heat transfer thus lends itself to a mathematical representation formally identical to that of mass transport. The methods for solving the system of equations will be the same.

1.7. GROUNDWATER QUALITY MONITORING SYSTEM

The control of the effects of human activity on hydrogeological system consists in identification of these effects, measurement of their impact on quality and quantity, and the development of methods and technological processes for their minimalization. In those regions where the effects of man on the hydrogeological system have not yet become apparent, the control measures consist of developing and applying methods of preventive protection of ground water resources.

Effective control of groundwater quality from impact of human activities requires establishment of a monitoring system to observe changes in the hydrogeological system, in space and time. Long-term measurement and observation of quantitative aspects of a groundwater system is carried out in many countries as part of the normal monitoring of groundwater level fluctuations, spring flow discharges, etc. Unfortunately, a corresponding programme for monitoring qualitative characteristics of a hydrogeological system is quite exceptional.

So far only local or regional groundwater quality programmes operate, focused at existing cases of pollution. The monitoring period is usually limited to the time of pollution. On the other hand, there exists a global project of groundwater quality monitoring and other international networks. However, the number and density of monitoring points in these types of network are insufficient from the standpoint of national interests and needs. The optimum solution is a centrally directed and methodically unified national groundwater monitoring system covering the whole state or the economically important regions of a state, with a network of selected observation wells and springs, located with regard to the natural conditions of the territory, to water supply systems and to the main existing and potential pollution sources, which themselves should also be subject to regular monitoring.

If we do not define the groundwater protection criteria at an early stage, sooner or later pollution may occur. If no monitoring system exists, pollution is identified late. Rehabilitation of aquifers, treatment of water and other technological processes result in economic, ecological and social consequences. We cannot leave aside the impact on the physical and mental health of man. Reduced productivity of work or migration to another area are usually not a subject of economic evaluation of consequences of water and environment pollution, but they do exist.

Because of the many different possible types of pollution sources, differences in size and function of monitoring networks and variations in industrial and agricultural potential and density of population, it is generally not possible, on the international level, to unify monitoring networks.

National and regional groundwater monitoring systems enable timely identification of pollution and recognition of the mechanism of movement of the pollutant in the vertical profile of aquifer to be made, or isolation and removal of the pollutant still in the unsaturated zone to be carried out. An insufficient monitoring system means that our position is in most cases a passive one, in the sense that we are not usually confronted with groundwater pollution until it has grown to the point of an ecological hazard.

The design of a national groundwater quality monitoring system with regional subsystem in the economically most important regions of a country should be an integral part of the national water resource quality monitoring network. It represents an important, though financially demanding programme. The success of its implementation will depend on unification and optimization of the monitoring methods and data evaluation techniques and its administrative legislative and financial implementation.

Different methods for design and operation of water quality monitoring system have been developed and utilized. The monitoring system is described in a simplified form in Fig. 1.6. Some of mentioned activities are pointed especially.

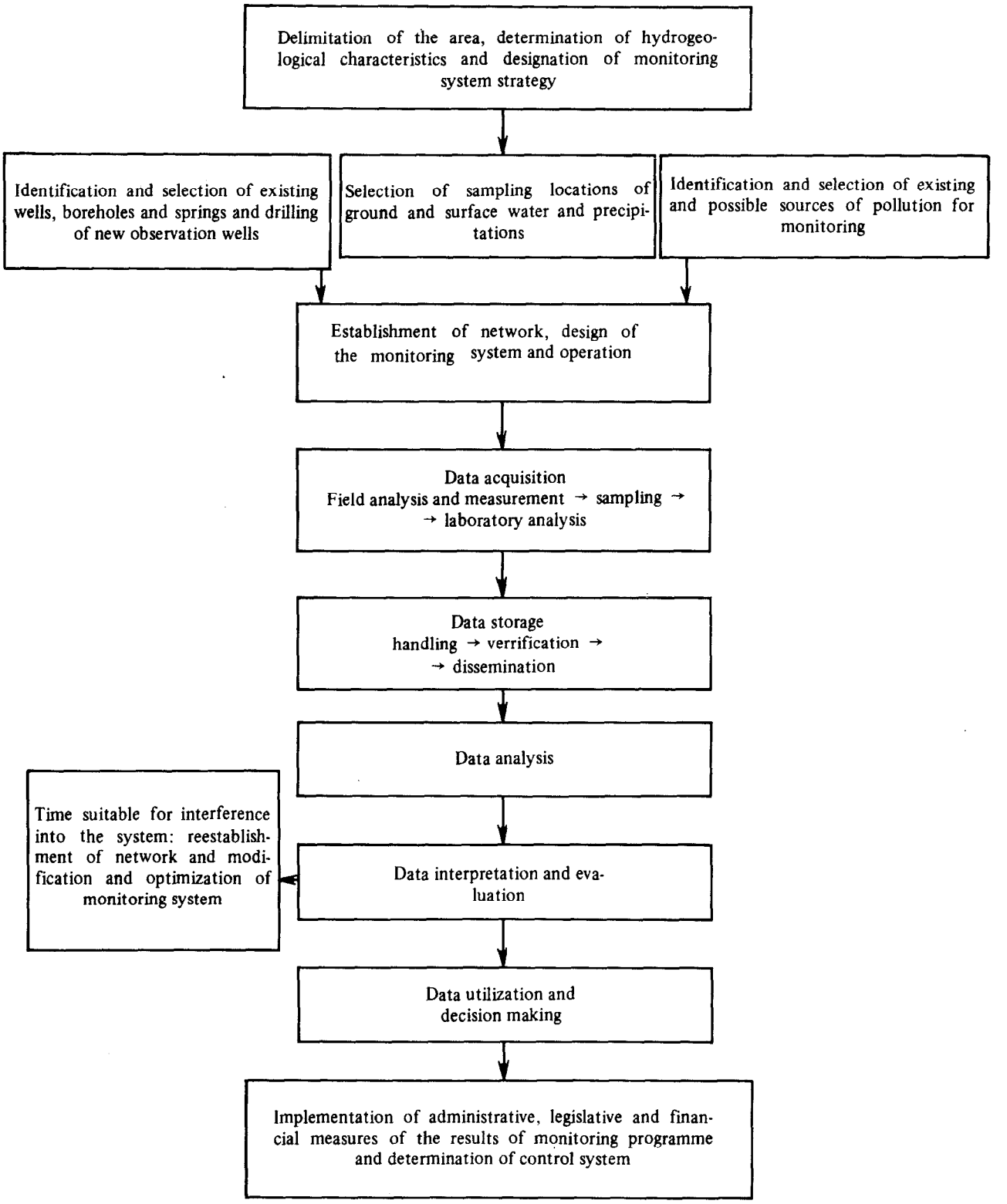


Fig. 1.6. Monitoring system

Design of Monitoring System

The design of a monitoring system depends on the objectives, the national governmental conditions and the area of the proposed monitoring network (local, regional, state, continental), and must take account of financial constraints. Determination of characteristics of a hydrogeological system, especially flow and migration parameters, are the basic requirements for the establishment of network and for the design of a monitoring system for the region.

Existing boreholes may be included into the monitoring network but their geological and hydrogeological data and construction must be known.

A spring is a natural discharge of an aquifer and is therefore an important part of the network.

Vertical stratification of contaminants in aquifers exists and the selected monitoring boreholes must be screened for sampling of vertical profile of aquifer. This is particularly important in local monitoring systems in areas with numerous sources of pollution.

All components and phases of the hydrological cycle must be monitored together with the hydrogeological system. Special attention must be paid to the properties and to the thickness of the unsaturated zone, including the soil layer.

In the case of identification and evaluation of pollution sources, the separation of natural effects from man's impact on hydrological systems must be very carefully considered.

The density of monitoring points in the study area depends strongly on the natural groundwater conditions, the intensity of man's activity, the size of the study area, the financial constraints and on the objectives, aims and goals of the monitoring system. Differences between local, regional and state scale monitoring networks make it impossible to set universal standards for the number and density of monitoring points.

Data Acquisition

The most important aspect of groundwater quality monitoring system is data acquisition which include the location of sampling points, and the definition of sampling procedures, sampling frequencies and the selection of water quality variables for analysis in the field and in the laboratory.

The location of a sampling point is strongly influenced by the objectives of the groundwater quality monitoring system, the natural conditions, especially to the hydrology, geography and geology of the territory, the soil characteristics, aquifer and unsaturated zone properties, the sources of pollution, the anthropogenic impacts in general and by financial constraints.

Sampling procedures depend on physical, chemical and biological properties of water and contaminants. In the case of a monitored borehole with stagnant water table conditions, the sampling is recommended after two or more hours of pumping. Samples from a spring or from a well with artesian flow conditions can be taken immediately but the origin of groundwater should be known. Special devices for the separated sampling of precipitation (rain-fall) should be installed. In the case of surface water, sampling is most commonly made from the zone of completely mixed flows.

Vertical hydrochemical profiles of contaminants in the unsaturated zone and in aquifer may be determined by means of sampling from multi-screening wells or from several wells of a different depth or sampling between packers. Special samplers, or multi-level samplers, should be employed. When vertical flow exists in the well, special methods of sampling must be used. Techniques and methods of sampling are very important for getting of a representative sample and incorrectly obtained mixed samples may prevent the correct detection of contaminants.

Sampling frequency is another important aspect for monitoring the changes in groundwater quality and the movement of contaminant in space and time. Sampling frequency often depends on financial constraints.

When setting a sampling frequency, account should be taken of the generally slow movement of contaminants in aquifer systems, and the effects of variations in thickness of the unsaturated and saturated zones on aquifer vulnerability. The objective of a monitoring system is decisive in determining the sampling frequency. Diurnal cycles, as functions of the rotation of the earth, yearly cycles as functions of the hydrological cycle, seasonally variable components, the impact of man's activity and other factors must be noted when sampling frequency is determined.

Sampling frequency cannot be unified for several reasons. Sometimes, a sampling interval of once per month can be too much, another time, it is not enough. In case of State groundwater quality monitoring system, sampling twice per year is generally sufficient. In the case of local monitoring networks, sampling should be more frequent. The monitoring of certain physical and chemical components of water quality continuously, for a limited period of time, may be useful.

For scientific, technical and financial reasons there is a trend to determine physical and biochemical parameters of water, soil and contaminants in situ, in the field. Continuous flow, quality sensors or continuous sampling apparatus which exclude the influence of air contact, changes in temperature, pH and pressure which may lead to physical and biochemical changes of water samples during transport to the laboratory are particularly suited to the purpose. Water quality data obtained by these sensors may be simultaneously recorded, and translated into computer compatible form for computer processing and analysis.

Different devices have been developed for the in situ identification and analysis of contaminants of anthropogenic origin (e.g. Horiba OCMA 200 – field analyzer of oil polluted groundwater). However oil products, volatile chlorinated hydrocarbons, heavy metals and pesticides are analysed on special precision devices in laboratories. Transport of a sample to the laboratory must be rapid and unstable components should be presented. The use of a unified laboratory analysis procedure for all monitoring is crucial to ensure the quality of the data, and the tabulation of field and laboratory analyses should be unified for computer evaluation.

Data storage

Data from newly installed monitoring points normally forms only a small part of the monitoring system. The principal inputs would be expected to be from existing state monitoring systems, data from water users, data from hydrogeological investigations, and data from pollution sources. These data may have been collected at different times, by different methods of measurement and analysis and they may not always have been directly comparable. An important aspect of designing a scheme in the selection of representative data for analysis, i.e. the removal of imbalances between data aquisition and storage. The importance of the verification and dissemination of old data or of data obtained from sources outside the monitoring system is cannot be overstressed. Data storage strategy is closely connected with monitoring system purpose.

Data analysis

Water quality monitoring is a statistical process. Statistical analysis is, therefore considered as a basis for modelling and simulation of processes and interactions between atmosphere, biosphere, hydrosphere, lithosphere and contaminants. When modelling the movement of contaminants in groundwater system, selection of an available model is always an important task connected with the aim of the monitoring programme. Analytical solutions for determination of the hydrodynamic dispersion coefficient, numerical solutions for the calcula-

tion of contaminant movement, evaluation of hydrobiochemical interactions between groundwater system and other systems and specification of initial and boundary conditions are especially important for the dynamic modelling of pollutant movement in groundwater systems. Deterministic system – descriptive models are mostly applied at data analysis of monitoring systems.

Data interpretation and evaluation

When the process of data analysis is finished and data have been compared and evaluated with the "reality in the field" then some inconsistencies are usual, providing feedback on the effectiveness of the monitoring programme. Before the transmission of results for utilization and for decision makers, modification and optimization of monitoring system should be done (changes in design of monitoring network, changes in sampling frequency). The results of the monitoring system and recommendations for modifications to the system should be clearly described in a Final report on monitoring system.

Data utilization and decision making

In case of state monitoring system, the results of monitoring may be published in annual reports. However, they may merely be archival but not interpreted and used for the control of the quality of the groundwater system. The financial return in such a case is not sufficient and it is generally accepted that monitoring is useful only when it is applied. Data utilization as a part of the monitoring process helps society and governments to identify negative impacts on water quality and to determine strategies, control systems and special programmes for water protection. Decision makers have a very important task in this point at the State or governmental level. Their decisions should recognise that groundwater quality management is an integral part of a national or regional system of water planning and management.

Implementation of results of monitoring programme

As final step of process of monitoring system we could consider implementation of administrative, legislative and financial measures of the results of monitoring programme and determination of control system – based on the recommendations of decision makers. The State is responsible for the realization of these measures and has also the prime responsibility for the protection of environment.

Conclusions

We have mentioned only some aspects connected with the design, operation, evaluation and implementation of a monitoring system, and have highlighted the importance of monitoring human impacts on the environment, especially on groundwater systems, and the scientific, economic and social complexity of the problem.

Chapter 2.

THE PROTECTION OF GROUNDWATER AGAINST CONTAMINATION

2.1. THE CONDITIONS OF GROUNDWATER CONTAMINATION

2.1.1. THE CONCEPT OF GROUNDWATER CONTAMINATION AND EVALUATION OF THE SCALE OF CONTAMINATION

The contamination of groundwater is brought about mainly by human economic activities. It may occur also naturally, for example, the salinization of fresh groundwater in coastal regions by sea waters as a result of gale and wind phenomena. But the main factor causing the contamination of groundwater and the deterioration of its quality is the transport of contaminants of industrial origin from the earth's surface into aquifers.

Groundwater contamination originates primarily in fresh groundwaters, present in the upper part of the saturated zone and, primarily, in the zone of active groundwater circulation. It is into this upper, actively circulating layer that the main load of contaminants are transported from the earth's surface, and therefore recent groundwater is most subject to contamination.

Groundwater contamination may take the form of an increase of its mineralization, of the appearance in groundwater of specific, uncharacteristic substances (for example, pesticides, petroleum products, etc.), changes of temperature and of pH value, and the appearance of odour, colour and other indicators of a change in water quality.

In describing the concept of contamination one should proceed from concrete and numerically determined indicators. As such use should be made of indicators of water quality under natural conditions (or background indicators) and indicators of maximum permissible levels, the use to which the groundwater may be applied. Fresh groundwater is the most important source of the drinking water supply. Therefore, it is advisable to use the indicators of drinking water quality for the specification of maximum permissible levels for fresh groundwater.

The quality of drinking water is defined usually by three groups of indicators: bacteriological, organoleptic organisms and the chemical composition of the water.

In each country official standards and norms exist for the quality of drinking water. In the USSR norms for the quality of drinking water are laid down by the state standard "Drinking water" (GOST 2874–73). According to this standard, the bacteriological indicators are subdivided into: 1) the total quantity of bacteria in the water and 2) the quantity of the bacterium Escherichia coli. The first indicator is determined by the total quantity of bacterial in 1 ml. of raw water, which should not exceed 100. The second indicator is determined by two values: the coli-index (K_e) – this represents the quantity of Escherichia coli in one litre of water, the coli-index should not exceed 3; coli-titre (K_t) is the quantity of water in ml, in which 1 Escherichia coli is contained, the coli-titre should be not less than 300. There is an approximate dependence between K_e and K_t:

$$K_e \approx \frac{1000}{K_t} \qquad (2.1)$$

The indicators of the chemical composition of water include the norms of the substances included in it (the maximum permissible concentrations, MPC), encountered in natural water and occurring in them as a result of industrial, agricultural and municipal contamination, and also as a result of the purification of water. GOST 2874–73 limits the content in water of dry residues, chlorides, sulphates, iron, natural uranium and uranium 238, radium-226, strontium-90, selenium, berylium, lead, copper, zinc, hexavalent chromium, manganese, molybdenum, arsenic, nitrates, aluminium, hexametaphosphate, tripolyphosphate, polyacrylamide, values of total hardness and pH. When several of the substances mentioned are present in the water (excluding fluorine, nitrates and radioactive substances) their total concentrations should not exceed 1, i.e.

$$\frac{c_1'}{c_1} + \frac{c_2'}{c_2} + \frac{c_3'}{c_3} + \ldots + \frac{c_n'}{c_n} \leqslant 1, \qquad (2.2)$$

where c_1', c_2', c_3' ... c_n' – are measured concentrations, mg/l; c_1, c_2, c_3, c_n ... – are the maximum permissible concentrations in mg/l.

Groundwater may be contaminated by a great variety of substances. The determination of each separately represents a complex problem and it is not always practicable.

Therefore it is advisable to characterize zones of contamination both by general indicators, and by specific indicators which are unique to each source of contamination.

These general indicators of contamination include: mineralization, temperature, pH-value, total hardness, content of chlorides, nitrates, petroleum products, heavy metals, biochemical oxygen demand (BOD) and chemical oxygen demand (COD).

The use of general indication make possible compasiron between different zones.

Based on the above, the concept of contamination of fresh groundwater is defined in the following way.

Contamination of groundwater occurs when its quality is changed from that defined by the levels of indication instances in the mutually occuring water, and the appropriate water quality structures, so that it became partially or wholly unfit for use.

If the background indicators of groundwater quality in a natural state are designated by c_e and the indicators of the maximum permissible norms by c_{mpc}, then in the

majority of cases the background concentrations of substances in fresh groundwater are less than the maximum permissible concentrations of these substances, i.e., $c_e < c_{mpc}$. However, in individual cases, inverse correlations may occur. For example, groundwater with a low degree of mineralization (a dry residue content of less than 1000 mg/l), may contain individual substances in quantities exceeding their maximum permissible concentrations.

Two main degrees of contamination of groundwater should be distinguished:

I. The content of contaminants is higher than the background content and continues to increase in time, but, for the time being, remains lower than the maximum permissible concentrations:

$$c_e < c \leqslant c_{mpc} \text{ or } \bar{c}_e < \bar{c} \leqslant 1. \quad (2.3)$$

where $\bar{c} = \frac{c}{c_{mpc}}$.

II. The content of contaminants is higher than the maximum permissible concentrations:

$$c > c_{mpc} \text{ or } \bar{c} > 1; \quad (2.4)$$

the second degree of contamination is subdivided into sub-degrees:

$$\begin{array}{ll} \text{II a} & 1 < \bar{c} \leqslant 10 \\ \text{II b} & 10 < \bar{c} \leqslant 100 \\ \text{II c} & \bar{c} > 100 \end{array} \quad (2.5)$$

For the first degree of contamination the deterioration of water quality does not exceed the permissible norm. However, the excess in the concentration of contaminants over their background concentrations and the tendency expressed over time of a growth in their content makes it possible to classify this case as the initial stage in the contamination of groundwater. The most dangerous is the second degree of contamination, when the content of contaminants exceeds the permissible level. It is the condition which is generally referred to as groundwater contamination. However, it is no less important to establish the trend towards deterioration of groundwater quality at the initial stage. The identification of the initial stage of contamination is essential in connection with preventive protective measures to avert the complete contamination of groundwater. It is much simple at the initial stage of this process to localize the zone of contamination and to reduce its intensity than when the level of contamination has reached high values. Thus, the identification of the initial stage of contamination has great practical importance for the timely implementation of conservation methods.

The scale of groundwater contamination is estimated by the following indicators: the size (area) of the zone of contamination, the intensity or rate of contamination, and the rate of contaminant migration through the aquifer.

The area of the zone of contamination is delineated by the boundary of mineralization of groundwater of 1 g/l or by the boundary of maximal permissible concentrations of the appropriate compounds. If the contamination of groundwater is characterized by several contaminant components and the boundaries of the maximum permissible concentrations of each of them taken separately do not coincide, then the zone of contamination should be defined by the totality of the boundaries of the maximum permissible concentrations of these components. The ranking of the size of areas of contamination is presented in Table 2.1.

Table 2.1

RANKING OF AREAS (F) OF GROUNDWATER CONTAMINATION

Rank	I	II	III	IV	V	VI	VII	VIII
Area, km^2	<1	$1 \leqslant F < 5$	$5 \leqslant F < 10$	$10 \leqslant F < 20$	$20 \leqslant F < 50$	$50 \leqslant F < 100$	$100 \leqslant F < 200$	$\geqslant 200$

The rate of groundwater contamination is characterized:

1) by the value of relative mean mineralization ($\bar{M}$) within the zone of contamination, limited by the boundary line of mineralization of 1 g/l;

2) by the value of the relative mean concentration ($\bar{c}$) of the content of individual components within the boundary lines of the maximum permissible concentrations of these components. Mean mineralization of the water (M) or mean concentration of individual components (C) depending on the extent of knowledge about the zone of contamination may be determined in two ways: either as a weighted mean value according to area (M_n or C_n) or as an arithmetical mean (M_A or C_A) of the individual values. In the first case

$$M_n(C_n) = \frac{M_1(C_1)\cdot F_1 + M_2(C_2)\cdot F_2 + M_3(C_3)\cdot F_3 + \ldots + M_n(C_n)\cdot F_n}{F_1 + F_2 + F_3 + \ldots + F_n} \quad (2.6)$$

where F_1, F_2, F_3, ... F_n – are the areas within the zone of contamination, characterized respectively by mineralizations (or by concentrations of individual components) M_1 (C_1), M_2 (C_2), M_3 (C_3), ... M_n (C_n).

In the second case

$$M_A(C_A) = \frac{M_1(C_1) + M_2(C_2) + M_3(C_3) + \ldots + M_n(C_n)}{n}, \quad (2.7)$$

where $M_1(C_1)$, $M_2(C_2)$, $M_3(C_3)$, ... $M_n(C_n)$ – are the values of mineralization (concentrations) at individual points within the zone of contamination; n – is the number of points.

The relative mean mineralization of groundwater ($\bar{M}$) or the relative mean concentration of the individual components in groundwater ($\bar{C}$) is defined as the ratio of the absolute mean mineralization (M) or of the concentration (C) to their maximum permissible values.

Ranking of the intensity of contamination of groundwater, characterized by value $\bar{M}$ or $\bar{C}$ is represented in Table 2.2.

Table 2.2

RANKING OF INTENSITY OF CONTAMINATION

Rank	A	B	C	D	E
Value $\bar{M}$ or $\bar{C}$	1–5	5–10	10–50	50–100	>100

Example. The zone of contamination of groundwater within the boundary line of mineralization M = 1 g/l is characterized by an area of 17 km^2 and a mean relative mineralization of 27; the zone of contamination by chlorine (Cl) is characterized by an area of 15 km^2 and

a mean relative concentration of 8; the zone of contamination by nitrates (NO_3) is characterized by an area of 9 km^2 and a mean relative concentration of 6.5. The above zone of contamination of groundwater on the basis of the gradations given above may be expressed in the following way:

$$M - IV - C,\ Cl - IV - B,\ NO_3 - III - B$$

Along with the indication of the size of the entire area of contamination, characterized by mean value $\bar{M}$ or $\bar{C}$, it is advisable to indicate the size of the area with maximum value $\bar{\bar{M}}$ or $\bar{\bar{C}}$.

The velocity (V) of progress of the contaminated water through the aquifer is estimated by the actual displacement (x) during a fixed period of time (t) of the boundary line of mineralization of 1 g/l or of the boundary lines of the maximum permissible concentrations of the individual components:

$$V = \frac{x}{t}. \qquad (2.8)$$

Calculations of V should be made in the direction of the natural flow of the groundwater, in the direction of the nearest abstraction points and in the direction of the nearest natural drains (rivers, lakes), into which the discharge of groundwater takes place.

2.1.2. CONTAMINANTS, TYPES OF CONTAMINATION AND SOURCES OF GROUNDWATER

Contaminants of groundwater are derived from: 1) industrial wastes; 2) domestic wastes; 3) agricultural wastes and chemicals, and, 4) natural (surface and subsurface) sub-standard waters.

Industrial wastes. Of greatest importance are industrial waste waters, due to the large volumes produced by industry, and their high content of strongly contaminating and toxic components.

The largest quantity of waste water is produced by plants of the chemical, oil-refining, textile, metallurgical, ore-refining, pharmaceutical, pulp and paper industries. The waste waters of industrial plants are distinguished by large discharges and the variety of their composition. This is especially true of waste waters of the chemical industry.

Industrial waste waters may be subdivided into the following groups: 1) waste waters containing mainly inorganic compounds; 2) waste waters containing mainly organic compounds; 3) waste waters containing both inorganic and organic compounds; 4) waste waters containing radioactive substances; 5) heat-exchanging waste waters. The first group of effluents includes waste waters of plants of the soda and calcium industries, plants for the production of mineral fertilizers, galvanizing shops (electro-plating shops), etc.; the second group includes waste waters of plants of the aniline-dye industry, plants for the production of synthetic materials, organic fertilizers, pharmaceutical plants, textile and other plants; the third group includes waste waters of metallurgical plants, coke and chemical plants, ore-refining plants, plants of the pulp and paper and other industries; the fourth group includes waste waters of nuclear plants; the fifth group, waste waters of power plants and of many industrial plants among those enumerated above. For the majority of industrial waste waters high mineralization, intensive coloration and odor are typical, which disappear only after repeated dilution. The waste waters of many industries are highly toxic.

Industrial waste water frequently contain chlorides, sulphates, iron, ammonia, nitrates and nitrites, heavy metals, phenols and petroleum products.

Domestic waste water (sewage). Contamination by domestic sewage is characterized mainly by bacterial (microbial) contamination. Household waste waters also contain organic substances, the products of their decay, nitrogenous compounds, fats, surface-active substances (detergents) and other components.

Contaminants of agricultural origin. The contamination of groundwater is brought about by the following main types of contaminants: 1) pesticides; 2) fertilizers; 3) effluents of stock-raising units; 4) wastes of poultry factories, etc. Contamination by pesticides is particularly dangerous because of their high toxicity, persistence and capacity for migration in groundwater and the natural environment. Pesticides, by their chemical composition, are divided into the following main groups: the organochlorine, the organophosphorus, the organomercury, the arsenic-containing, carbonates, and di-thio-carbonates. Most widespread are pesticides of the organochlorine group, characteristic representatives of which are hexachlorocyclohexane, hexachloran, Lindane, DDT, heptachlor, dieldrin, aldrin, etc. Pesticides are biologically active and toxic substances; many of them are degraded into intermediate compounds (metabolites), often no less, and even more, toxic, than the parent substances.

Considerable contamination of groundwater is caused by the effluents which is accumulated in large quantities at major stock-raising units and farms. The main contaminants in animal effluents are nitrogen, phosphorous and potassium. The phosphorous and potassium are distinguished by low mobilities and are retained mainly in the soil layer. The main effect on groundwater quality of discharge animal effluents is performed by nitrogen in nitrate form, which may be accompanied by bacterial contamination.

Natural substandard waters. Surface and underground substandard waters are characterized in their natural state either by high mineralization (as compared with drinking water standards) or by high content of individual components with low mineralization. Among the types of contamination caused by surface substandard waters, the most common is chloride contamination, brought about, mainly by seawater in coastal areas, as well as by salt river and lake waters in arid zones. Surface waters in areas of impeded drinkage (marshes) may contain high quantities of iron. A large content of chlorides is most typical, in the first place, for substandard subsurface waters; in addition, quite widespread are waters with a high content of sulphates, fluorine, iron, hydrogen sulfide with a total increased hardness.

Chemical contamination arise from industrial and domestic wastes, chemical products and fertilizers, natural substandard waters and waters coming from mine pit and shaft drainage. Hydrocarbon contamination is brought about by petroleum and petroleum products, by the waste waters of oil and gas fields and refineries; bacterial contamination by domestic and agricultural wastes; thermal contamination by industrial and domestic wastes; and radioactive contamination by industrial wastes. The most persistent and toxic are chamical, hydrocarbon and radioactive contamination.

Chemical contamination shows itself in an increase of the mineralization of the groundwater above background values, and by an increase in concentrations of introduced mineral and organic compounds. Chemical contamination is persistent and may spread over large distances.

Hydrocarbon contamination is characterized by an intense odour, the presence in the water of an iridescent film, and a high content of various petroleum hydrocarbons. Some hydrocarbons (aromatic) form real solutions with the water, while other hydrocarbons, indissoluble in water, form an emultive.

The bacterial contamination of groundwater is expressed by the appearance in the water of pathogenic and other micro-organisms of potential public health significance; in particular, of bacteria of the Escherichia coli group. Because of the limited life-span of these bacteria in aquifers (30-50 to 200-300 days), this type of groundwater contamination is usually localized in a comparatively small area and may be temporary in nature.

Thermal contamination of groundwater becomes apparent by an increase in the temperature. Processes accompanying thermal contamination may be a drop in the oxygen content of the water, and a change in its chemical composition, an increase in triability due to exsolution of gases of the water and an increase in the content of micro-organisms. Thermal contamination is usually combined with chemical contamination, caused by industrial waste waters.

The most common contaminants in groundwaters are chlorides, sulphates, nitrates, petroleum products, phenols, heavy metals, and pesticides.

Zones of groundwater contamination are associated chiefly with major industrial and agricultural installations, producing large quantities of wastes. These include plants of the chemical, ore-refining, metallurgical, pharmaceutical, oil-refining, textile and pulp and paper industries.

A major source of groundwater contamination by agriculture arises from large scale stock-raising units and poultry factories. The most intensive contamination of groundwater is found at sites for the storage and disposal of wastes on the earth's surface. They include: 1) surface reservoirs (artificial and natural) of industrial, municipal and agricultural liquid wastes (ponds — cesspools (settling tanks), storage tanks, evaporator ponds, slurry storage tanks, etc.); 2) storage facilities for solid wastes (salt dumps, ash dumps, municipal garbage dumps, etc.); 3) infiltration areas, on which waste waters are discharged for natural purification by filtration; and 4) irrigation of field by waste waters.

Other sources of groundwater contamination may be storage sites for petroleum products, toxic chemicals, fertilizers, and chemical reagents, filling-stations and car washing stations, oil pipelines, burial grounds, oilfield sites, highways, intensive gas and smoke emissions to the atmosphere, and contaminated surface waters.

Contaminated waters may enter aquifers by a number of routes: 1) infiltration of industrial waste waters at industrial plants and especially on the sites of surface waste reservoirs; 2) infiltration from fields irrigated by waste waters and from filter beds; 3) infiltration from agricultural areas where pesticides and fertilizers are applied; 4) contamination by industrial and domestic wastes through abandoned pumping and observation wells; 5) lateral and vertical infiltration of contaminated river waters during floods; 6) leakage of substandard groundwaters from adjacent aquifers, and induced recharge from substandard surface waters; 7) infiltration of contaminated atmospheric precipitation.

2.1.3. THE INTERRELATION BETWEEN GROUNDWATER CONTAMINATION AND CONTAMINATION OF THE ENVIRONMENT

The contamination of groundwater to a considerable extent is brought about by the contamination of the environment as a whole — the surface of the earth (the soil), surface waters, the atmosphere, and atmospheric precipitation. Contaminants enter groundwater from the environment as part of the natural water cycle.

Contaminants from the surface of the earth (the soil), together with atmospheric precipitation, percolate into groundwater. Contaminants in surface waters, as a result of the interrelation of surface and groundwater may also enter aquifers. Atmospheric contaminants fall onto the surface of the earth and into surface waters as particulate and dissolved substances and hence enter groundwater.

The highest levels of environmental contamination are found in industrial areas, both directly to groundwaters and indirectly via the air, soil and surface waters.

Contamination of the soil occurs through the storage of solid and liquid industrial waste, by accidental spills or releases from industrial sewerage and pipeline systems, by losses and accidents during the transport of raw materials and finished products, from the effect of motor and railway transport, and by other factors. The contaminants contained in soil, together with atmospheric precipitation, percolate into groundwater and deteriorate its quality.

No less important a factor in the contamination of the soil, surface and groundwater, are the emissions of wastes of industrial plants into the atmosphere. Unlike the intensive local contamination of soil and groundwater directly on the site of an industrial plant and especially in the vicinity of wastes reservoirs, the contamination of the atmosphere may spread over large areas although at a lower rate of intensity. The appearance of a required increase in the contamination of an aquifer may be the result of atmospheric pollution.

A great deal of groundwater contamination may be explained by the contamination of surface waters. This is especially characteristic of river valleys containing alluvial aquifers. The movement of contaminated river water into the aquifers takes place through lateral infiltration from the rivers channel, and vertical infiltration from the surface of the earth during flood periods. Contaminated groundwaters may (in their turn), affect the quality of surface waters. Howerver, in general the effect of contaminated groundwater discharging into rivers on their quality is considerably less than the effect of effluent discharges to rivers.

Groundwater contamination by river water may be traced along the river valley and is elongated and linear in nature. In contrast, the groundwater contamination, caused by atmospheric pollution may appear, at a relatively low level, over a considerable area. Ever so, individual elongated zones with a greater degree of contamination, associated with the directions of the prevailing winds, may be isolated in the areal contamination of groundwater and soils. Concentrated, intensive groundwater contamination, although occurring on a comparatively small scale, is typical of industrial sites.

Thus, the contamination of groundwater is closely linked with the contamination of the environment as a whole. In practice, it is impossible to prevent contamina-

tion of groundwaters whilst contamination of the atmosphere, surface waters and the soil continues. The campaign against groundwater contamination is an integral part of the campaign against contamination of the environment, and must be carried out within the framework of this general problem. Monitoring the state of groundwater should be closely coordinated with the results of observations on the state of the environment.

The soil*

The soil plays a very important role when groundwater protection and impacts of pollution are studied. Special attention must be paid to the soil layer in cultivated agricultural land. Attempts to increase soil productivity and crops yields result in considerable impacts on soil properties by means of irrigation, drainage, fertilizers, pesticides, heavy equipment, etc. Soil degradation, especially soil erosion can be a negative effect of modern agriculture. Annual soil losses 10 tonnes per hectare are not exceptional in European countries and in USA.

The movement of contaminants in soil and in water (included vegetation root system) is influenced by processes of convection, molecular diffusion, mechanical dispersion and by physical, chemical and biological interactions.

In vertical soil profiles, the upper part is usually unsaturated. The movement of contaminants in soil is expressed as one-dimensional vertical process.

The movement of water and solutes in soil layers depends mainly on soil type, texture and structure of soil and its thickness, soil water content and potential degree of saturation, storage capacity, adsorption in the soil complex, etc. The type of vegetation cover is very important, especially the root system (quality of soil as biological filter increases with the density of root system and with the quantity of microorganisms in soil).

In order to determine the water movement in soil and its availability to plants, characteristics of soil water content are defined (hydroscopic coefficient, wilting point, moisture equivalent, field capacity – Bear 1979). Several methods of soil water content measuring exist (gravimetric method, electric resistance method, electric capacity method, gamma method, neutron method).

The soil water potential depends on the soil physical properties, i.e. grain surface and shape and soil water content. Soil temperature and the history of wetting and drying are usually not determined. The soil water potential has a wide range of values and therefore the semilogaritmic form is generally used for expressing relationship (pF curve = log/–h).

Hydraulic conductivity in the unsaturated zone depends on the same factors as the soil water potential. Various empirical relations of hydraulic conductivity as soil water content and soil water potential are given in the literature (Tanji 1981, Selim 1980, Rijtema 1965). The unsaturated hydraulic conductivity is measured in laboratory (on undisturbed samples of soil) and directly in the field (measurements of pressure head and water content).

For the calculation of solute adsorbtion in the soil complex several models are used (Selim 1980, Tillotson 1980, Wagenet 1981 and others). The quantity of adsorbed substance is directly proportional to its concentration in the solution.

Water uptake by the root system of the vegetation and its temporal and spatial distribution over the vertical soil profile is dependent on type of vegetation, climatic conditions, the season of the year, soil characteristics and soil water content. Various models describing this process have been given in literature (Feddes 1976, Hovgland 1981, Selim–Iskandar 1980, etc.). There exists an interaction between climate (temperature, humidity), soil (physical and chemical properties) and the biological activity (microbial life, vegetation, etc.) humidity and groundwater regime.

1) With shallow stagnant or upward seeping saturated groundwater (not saline), gleying takes place. The high groundwater level causes a reduced environment characterized by bluish ferrous iron compounds. Mostly in cool climates, bogs.

2) With continuous downward seepage (annual precipitation excess) and an oxygen-rich unsaturated zone, podzolization takes place with a leached upper (A) layer and an enriched lower (B) layer. In the A layer, the exchangeable cations ("bases" Na, K, Ca, Mg) adsorbed to the clay minerals are replaced by hydrogen, making the soil rather acid. In the B layer clay-humus colloids and oxides of iron and alluminium which connect humus and clay particles to each other, precipitate. In moist-warm climates, the clay minerals are further decomposed to Gibbsite and silica, the latter being leached out (laterization, giving neutral latosols).

3) With continuous capillary rise and evaporation in semi-arid climates calcification takes palce (caliche formation). Mg and Ca ions remain in the soil and colloids are flocculated. In desert climates salinization is created (alkaline soils).

Dissolution, hydration, oxidation and hydrolysis are main chemical processes which take place in soil layer.

2.2. INDUSTRIAL AND NATURAL FACTORS IN GROUNDWATER CONTAMINATION

2.2.1. GROUNDWATER CONTAMINATION IN INDUSTRIAL AREAS

Groundwater contamination at industrial premises may occur over the whole site of the industrial area, but is the most intensive at surface reservoirs storing wastes, and at infiltration beds, where waste waters are treated or disposed of.

Waste reservoirs may vary in size from a few hundred meters up to a kilometer. They may be lined or unlined. Infiltration of waste waters from surface reservoirs may lead to contamination of the surface layer of soils on the surrounding land. Unconfined groundwaters beneath the reservoir, may be subject to a constantly expanding zone of contamination. At large waste reservoirs the area of groundwater contamination may amount to tens of square kilometers.

Hydrodynamically, the main sources of contamination may be divided into two groups. The first group includes industrial waste lagoons; for which a more or less constant height of a column of waste waters is characteristic, i.e. the condition H = const. The second group

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includes infiltration beds and fields irrigated by waste waters; for them, the discharge of waste waters disposed of on the surface of the earth may be considered as more or less constant, i.e. the condition H = const.

The formation of a zone of groundwater contamination in areas of water reservoirs occurs in several stages. In the initial period, free infiltration from the reservoir takes place, which continues until the flow from the waste reservoir coalesces with the groundwater flow. From that point the condition changes to the supported infiltration regime. The length of the free infiltration period is comparatively small. As a result of the infiltration from the reservoir of a mound of contaminated water is formed at the water table, which with time may merge with the base of the reservoir. Simultaneously with the growth of this mound of lateral migration within the aquifer takes place. The spread of waste waters is brought about by the following factors: the effect of the reservoir itself, the effect of the natural movement of groundwater, and the effect of the closest water withdrawal installations, water lowering or draw-down installations, mine drainage, etc. The main influences on the lateral spread of pollution within the aquifer are the natural groundwater flow pattern and the effects of local pumping sources. Lateral movement due to the recharge mound beneath to waste reservoir is normally very restricted in extent.

An analysis of the conditions of groundwater contamination at sites of surface reservoirs of waste waters has shown the following: the contamination of the groundwater is of the several times the permissible standard; the zone of groundwater contamination is persistent even after the elimination of the reservoir (in this case residual contamination of the soil and in the unsaturated zone may act as secondary source of pollution); the pollutant water is often of artificially high density to penetrate deeply into the aquifer; heterogeneity in the distribution of intergranular and fracture porosity and permeability may lead to irregular movement of the contaminants through the aquifer; the rate of movement of the contaminants is often in the range of several tens to hundreds of meters per year; in strongly fractured and karstified rocks, the velocity of movement of contaminated waters may reach several kilometers per year.

Therefore, a forecast of groundwater contamination at an industrial site should include: a) an estimate of the time in which the waste waters may reach the water table; b) an estimate of the infiltration rate of the waste waters; c) an estimate of the size of the zone of propagation of pollutants within aquifer.

In order to simplify hydrogeological prediction, waste water reservoirs may be assumed to conform to one of two patterns, depending on (i) circular, if the reservoir has a closed outline and the longitudinal and transverse dimensions are comparable, and (ii) linear, where the length is many times the width. Examples of calculations based on those simplifications are given in references [2, 3 and 13].

An approximate estimate of the time it takes for waste waters to reach the water table under conditions of a constant level and distribution of hydraulic parameters in the unsaturated zone may be made from well-known formula:

$$t = \frac{n \cdot H_0}{K} \left[\frac{m}{H_0} - \ln\left(1 + \frac{m}{H_0}\right)\right], \qquad (2.9)$$

where: H_0 – is the hydraulic level of waste waters in the reservoir; K, m – are the coefficients of permeability and the depth of the unsaturated zone respectively; and n – is the porosity.

Calculations by this formula have shown that the time of penetration of waste waters to groundwater level depends principally on the permeability of the rocks in the unsaturated zone, but that, on the whole, the time is very short. Thus, when $K \geqslant 0.5$ m/day, the time of movement rarely exceeds several days even with relatively thick unsaturated zones ($m > 10$ m); when $K < 0.5$ m/day, the transit time increases to several tens of days, and when $K < 0.01$ m/day and $m > 20$ m, the time may extend to hundreds of days.

If the unsaturated zone is composed of two layers, with a low permeability of the upper layer the time of infiltration to the water table is made up of time t_1, movement in the upper layer, and of time t_2, movement in the lower layer. The time t_1 is determined according to formula (2.9) by substituting in it parameters (K_1, m_1) of the upper layer. The time t_2 of movement through the lower layer is determined on the basis of the solution by Verigin [2]. Analysis has shown that given $\frac{K_1}{K_2} < 0.1$, the time of movement on the 2-layer system is determined mainly by the time of movement through the upper low-permeability layer (K_1, m_1). Therefore, an approximate estimate of the time of movement through a 2-layer system may be made by applying the formula (2.9) to the low-permeability layer alone.

Given the disposal of the waste waters at a constant discharge Q over an area of F there may be two cases.

1) If the discharge cited $q = \frac{Q}{F}$ is less than or equal to the permeability k of the rocks, the whole discharge infiltrates without forming a free water surface on the ground level. In this case, the time to reach the water table may be estimated according to the formula:

$$t = \frac{n \cdot m'}{\sqrt[3]{q^2 \cdot k^1}} \qquad (2.10)$$

where: m' – is the depth of the unsaturated zone, and k and n – are the permeability and the porosity of the rocks, respectively.

2) if $q > k$, then the throughout of the underlying layer is less than the discharge and a variable column of water is formed on the surface. In this case, the transit time to water table is determined by the formula:

$$t = \frac{m'}{\frac{(1-n) \cdot k}{2n} + \sqrt{\frac{(1-n)^2 k^2}{4n^2} + \frac{q \cdot k}{n}}} \qquad (2.11)$$

The symbols are the same as in formula (2.10).

The coalescing of the groundwater with the base of the reservoir normally occurs comparatively quickly, over a period of not more than 1–2 years, which is considerably less than the normal operating life of a reservoir, which is often from 15 to 20 years. Therefore, the estimate of the discharge from the reservoir may be made on the basis of the conditions of infiltration with head. Because the estimates cover a long time period, the infiltration regime may be taken as inquasi-equilibrium.

Under these conditions, the discharge (q_Φ) from the reservoir, assumed to be linear per unit length, may be determined from [39]:

$$q_{\phi} = \frac{km \cdot H_0}{\Delta L}, \qquad (2.12)$$

where H_0 – is the depth of waters in the reservoir and k, m – are the permeability and the depth of the aquifer. The expression ΔL is:

a) for a homogeneous stratum

$$\Delta L = B - \frac{2m}{\pi} \ln \sin \frac{\pi B}{2m}. \qquad (2.13)$$

b) for a 2-layer stratum (k_1, m_1 – are the parameters of the upper layer; k, m – are the parameters of the lower layer);

$$\Delta L = \sqrt{\frac{km\, m_1}{k_1}} \operatorname{cth} \left(B \sqrt{\frac{k_1}{km\, m_1}}\right); \qquad (2.14)$$

or

$$\Delta L = \sqrt{\frac{km\, m_1}{k_1}} \quad \left(\text{with } \frac{B}{m} > 20\text{–}30\right), \qquad (2.15)$$

where B – is the half width of the reservoir.

For circular reservoirs of radius R_k the discharge on filtration is determined in the following way. For conditions of a homogeneous structure we have [2, 39]:

$$Q_{\phi} = \frac{km H_0}{0{,}366 \cdot \lg \frac{R}{R_k}}, \qquad (2.16)$$

where: H_0 – is the height of the column of liquid in the reservoir, R – is the distance from the centre of the reservoir to the boundary line of aquifer recharge.

For a 2-layer structure of an aquifer the discharge from the reservoir, [2, 39], is equal to:

$$Q_{\phi} = \frac{2\pi km H_0}{\ln \frac{R}{R_k} + \frac{I_0(\alpha R_k)}{R_k \alpha I_1(\alpha R_k)}}, \qquad (2.17)$$

where: $\alpha = \sqrt{\frac{k_1}{km\, m_1}}$, I_0, I_1 – are Bessel's functions of an imaginary argument of zero and first orders.

The expression (2.17) may be represented in the form:

$$Q_{\phi} = \frac{km H_0}{0{,}366 \lg \frac{R}{R_k} + \frac{I_0(\alpha R_k)}{2\pi R_k \alpha I_1(\alpha R_k)}} \qquad (2.18)$$

After the waste waters have entered the aquifer they move with the groundwater flows. The rate of lateral movement of the contaminated waters from the reservoir of elongated form is estimated approximately according to the formula:

$$v = \frac{q}{2mn_0} + \frac{v_e}{n_0}, \qquad (2.19)$$

where: q_0 – is the infiltration losses per unit of length of the reservoir, m^2/day; n_0 – is the porosity of the waterbearing rocks; m – is the mean width of the aquifer; $v_e = k \cdot i_e$ – is the rate of flows (Darly velocity) of the national groundwater flow.

The distance x, by which the contaminated waters will move during time t, is equal to

$$x = v \cdot t \qquad (2.20)$$

In the case of a round reservoir, which may be approximated by a round gallery with a discharge equal to the discharge of the waste waters from the reservoir, the distance x over which the flow will occur within the aquifer during time t, with losses from the reservoir of Q_{ϕ} and a natural groundwater flow of v_e may be determined approximately by:

$$x = \sqrt{R_k^2 + \frac{Q_{\phi} \cdot t}{\pi n_0 m} + \frac{v_e \cdot t}{n_0}} - R_k. \qquad (2.21)$$

The propagation of contamination in the upstream direction is limited.

Using formulae (2.19) – (2.21), it is possible to estimate the time during which contaminated groundwater will reach the closest reservoirs and waterways to which it discharges.

2.2.2. CHARACTERISTICS OF GROUNDWATER CONTAMINATION IN MINE WORKINGS

Large-scale mining operations often lead to deterioration of groundwater quality over large areas around quarries and mines. In order to dewater ore-deposits, large volumes of water (hundreds and thousands of m^3/hour) may be pumped, giving rise to cones of depression around mines with radii of tens of kilometers, with lowering of water levels, measured occasionally in hundreds of meters. This dewatering of aquifers may seriously disturb the established hydrochemical equilibrium. On the other hand, slag-heaps, tailing-dumps, etc., which accompany mine operations, may be sources of additional recharge and of intense contamination of groundwater.

The change in hydrodynamic conditions are often regional in extent: the position of major recharge zones may be changed, recharge rates may be altered, additional leakage between aquifers may occur and the whole water balance of the region may shift. The most important changes of such a nature occur in closed artesian basin where, under the influence of mine drainage, an irreversible reduction in groundwater resources is usually noted. This may involve a serious reduction in confined storage, and in arid regions with the continuous reduction of the saturated boundary. Against this background of regional changes, a marked redistribution of hydraulic funds between individual aquifer units occurs, brought about by different dewatering or mine drainage and difference in response of the intervening aquicludes. These changes may be greater where the mining involves allowing the roofs of worked out areas to collapse which leads to fracturing of the overlying aquicludes. Localised changes in flow pattern may appear: 1) natural watercourses which have suffered inversion – from zones of run-off they are transformed into zones of recharge; 2) recharge may be increased due to the topographic disturbance (subsidence), and to the partial stripping of covering deposits, and the lowering of levels in unconfined aquifers; 3) a water-balance may be re-established with time, due to additional drainage from slag heaps and tailing dumps compensating for the additional mine drainage. However, in most cases an unstable system permits with insufficient recharge to compensate for the mine drainage.

At the same time, a change in the natural hydrochemical conditions almost inevitably occurs. The existing dynamic equilibrium among the elements of the multicomponent rock-water system is disturbed, frequently leading to serious deterioration of the groundwater quality. In a number of mining regions, mine drainage has contributed to an intensification of the processes of leakage from adjacent aquifers containing poorer quality water.

Reductions in hydraulic head may also encourage the compaction of low permeability layer, with the expulsion of highly mineralised water. Generally, the hydrochemical situation in a region changes markedly for the worse, so that the maintenance of a normal water supply requires additional, and often costly, conservation measures. The principal objective must be to establish an optimal balance, based on an understanding of the actual hydrogeological regime, between the water removed by mine drainage and the natural and induced recharge to the system. In this context, optimisation is taken to make use of the drainage water, from an efficient system, for practical purposes, and measures to maintain the quality of water.

The objectives of hydrogeological investigations in mining regions, in the light of the managemental problems discussed above are: 1) the evaluation and forecasting of the groundwater regime, taking into account changes in the conditions of recharge and interactions between aquifer units. Assessment of recharge from artificial ponds and due to man-induced charge in the permeability of the strata (fracturing, etc.), are of primary importance; 2) the evaluation and forecasting of the hydrochemical groundwater regime taking account of both contamination from technical reservoirs and possible changes in the surface water – groundwater interrelationships; 3) to determine suitable monitoring programmes and preventive measures which will ensure the protection of groundwater against depletion and contamination, as well as providing for the rational use or safe disposal of waters pumped out by drainage systems from mines and quarries.

It is essential for efficient management of groundwater resources and quality in mining regions that close coordination exists between the mine drainage and the water supply plans of the region. A policy of maximum use of drainage water for water supply should be pursued. Experience has shown that opportunities may be created for the organization of the drinking water supply on the basis of drainage waters. The most serious limitations, in addition to the composition of the water pumped out, are imposed by the requirements for a reliable sanitary protection of the drainage water abstractions. However, the analysis carried out shows that the requirements may often be met, even if mining operations and surface installations accompanying mining production are present within the zone of sanitary protection. Under these circumstances, the attenuating properties of the unsaturated zone should be assessed, requiring a preliminary study of the processes of mass transfer in unsaturated media. At the present time the methodologies for such studies are poorly developed.

A critical link in the management of groundwater resources and quality is the groundwater observation network, both in the area of the mine in quarry and in the surrounding region.

Based on the discussions above, further research and study should be carried out: 1) the development of a method for determining leakage losses from industrial waste lagoons; 2) the development of methods for the evaluation and forecasting of changes in hydrodynamic conditions in regions where working practices allow the progressive collapse of completed mine workings; 3) the development of methods for defining sanitary protection zones around mine drainage installations; 4) the development of protocols for monitoring at sites of groundwater contamination; 5) the development of computer methods for forecasting groundwater contamination; 6) the development of principles and methods for the optimization of hydrogeological research in assessing mineral deposits, with particular attention paid to new requirements for the protection of groundwater.

The successful development of further scientific methods to deal with these problems are likely to stem from: 1) comprehensive research leading to unique solutions of the problem of drainage of mining workings and of groundwater protection, which will involve many areas of hydrogeological expertise; 2) an intrusive approach, by the gradual refinement of computational models on the basis of a stage-by-stage accumulation of information in the course of the exploration, construction and operation of mines; 3) the analysis of data from representative examples, where reliable migration and hydraulic parameters have been obtained and verified in the process of operation by solution of the corresponding inverse problem. The latter two aspects appear to be especially important because, in studying the problem under consideration, it is generally necessary to rely on ideas and methods which have been insufficiently confirmed by practice and which go substantially beyond the framework of standard hydrogeological practices. The current experience, derived from investigations in a number of major mining areas convincingly demonstrates the economic efficiency and promising long-term prospects of the comprehensive approach described above.

2.2.3. GROUNDWATER CONTAMINATION BY PETROLEUM AND ITS PRODUCTS

Crude oil constitutes a complex mixture of a large number of hydrocarbons and other compounds. The hydrocarbons may be subdivided into four main groups:

1. The alkanes (paraffins) are hydrocarbons with a linear arrangement of their carbon atoms. The series begins with gaseous methane and ends with paraffin waxes. The alkanes constitute a desirable component in petroleum.

2. The cycloalkanes (naphthenes) have a cyclic arrangement of carbon nuclei with a single bond, the so-called saturated compounds.

3. The aromatics constitute unsaturated cyclical compounds, which in part have double bonds between the carbon atoms. As a rule, these are liquids, having a characteristic odour as for example, benzene-toluene.

4. Naphtene aromatic hydrocarbons or hybrid hydrocarbons have collateral ramifications, or are polycyclic.

From the point of view of safeguarding water quality, importance attaches to those properties which determine the propagation of petroleum products in rocks and in a water medium, particularly specific gravity, surface tension, viscosity and solubility in water, volatility, colour, electrical conductivity and odour. The latter is caused more often by decomposition products than by the original hydrocarbons and may assist in the detection and delineation of the oil contamination.

Viscosity considerably affects the rate of propagation of petroleum products in the ground. As a rule, the viscosity of hydrocarbons increases with molecular weight and boiling point.

Various types of crude oil show considerable differences in viscosity in accordance with their place of origin. The light grades of oil from the Sahara or Kuwait have a viscosity of 10–20 mm^2/s. Oil from the Soviet Union and South America has a viscosity of several hundred mm^2/s (Table 2.3.).

Table 2.3

PROPERTIES OF VARIOUS GRADES OF CRUDE OIL
(Houzim, Pelikan, 1978)

Crude oil	Specific gravity, g/cm³		Kinematic viscosity, mm²/s		Solidification point, °C
	10 °C	20 °C	10 °C	20 °C	
Romashkino oil	0.862	0.855	31.0	18.7	–29
Light grades of oil from the USSR	–	–	–	3–70	–
Bashkir oil	–	–	8–185	6–90	–
Heavy grade of oil from the USSR	–	–	–	100–250	–
Iranian	0.869	0.862	26.0	14.1	–24
Kuwait oil	–	–	27.0	15.0	–

Specific gravity. All grades of crude oil and many petroleum products are lighter than water. The specific gravity of substances increases as the boiling point goes up, for example, from gasoline to the Diesel fuels right up to fuel oil. Only some grades of fuel oil and halogenated hydrocarbons have a specific gravity greater than the specific gravity of water.

The magnitude of surface tension acts on the displacement of these substances as a result of the effect of capillary forces and the formation of emulsions of petroleum substances in the water. The laminar or interlaminar tension is decreased by increasing temperature and also by the presence of some polar compounds, such as naphthene acids and oxidation products, which are naturally dissolved in crude oil. The best known artificial chemicals suppressing surface tension include emulsifiers, soaps and detergents. The surface tension also plays its part in emulsions produced by rapid motion, shocks and mixing of immiscible media, in that it influences the stability of mechanical emulsions. Pumping of water with its floating oil layer results in the first emulsification during the flow into the well through the well screen, followed by more extensive formation during passage through the inlet strains and in the interior of the pump. Before discharging the water, the emulsified hydrocarbons have to be thoroughly removed from the water, together with the free and dissolved oil substances.

The solubility of petroleum hydrocarbons in water is the most important property in the process of groundwater contamination.

The solubility of the main groups of hydrocarbons increases in the following sequence: paraffins–cyclanes––olefins–aromatic hydrocarbons.

Petroleum hydrocarbons may be present in, and migrate in, the interior of the earth: a) in the gaseous phase; b) in a separate liquid phase; c) in a liquid solution (emulsions).

Gaseous hydrocarbons penetrate the ground in large quantities as a result of the leakage of gas from gas pipelines and underground gas storage tanks. In all other cases they constitute a very limited layer (in space and by concentration) in the aerated zone around a petroleum body and diffuse in the direction of the earth's surface.

The need for investigation of the movement of petroleum products as a separate liquid phase was caused by the growth in the number of cases of oil spills in the 1950's. Great attention was paid to the study of this problem in the Federal Republic of Germany, where a group of experts (F. Schwille et al.) developed the principles of a completely new branch of science, namely, the hydrogeology of oil contamination. The movement of petroleum substances in the ground is determined by a series of different factors. Along with the physical properties of petroleum products, to which reference was made above, the effect of the surrounding rocks under the earth may be noted: distribution according to granularity, density, porosity, permeability, mineralogy, moisture content in the zone of aeration, direction and flow rate of underground water and change in its water table.

Table 2.4

SOLUBILITY AND THRESHOLD VALUES OF OIL ODORS

Petroleum products	Concentration of oil, mg/l	Threshold odor values, mg/l
Benzene	1700	1–10
Gasoline	50–500	0.001–0.01
Diesel fuel	10–50	0.001–0.01
Kerosene	0.1–5	0.01–0.1

The laws and formulae mentioned below have been derived on the basis of experimental data obtained on sand models under laboratory conditions, on the basis of field investigations and as a result of actual oil spills. In the aerated zone, oil percolates vertically downwards. The column of oil works like a piston, inasmuch as it is hydraulically joined with the spilled product which has collected on the surface or the active source of contamination. As the hydraulic pressure falls, the further progress of the oil contaminant will be slowed down. In the majority of cases, the descending flow encounters an obstacle which may be constituted by a less permeable layer, or by a groundwater table, or by its capillary border. The body of oil formed in this way is called a nucleus or a "pancake". The form of the body of oil depends on the quantity and type of the product spilled and on the lithology of the locality.

To evaluate the vertical propagation of the free petroleum product the following empirical equation was obtained (Arbeitskreis (1970)):

$$S = \frac{V_i \cdot 1000}{A_i \cdot u_0} \qquad (2.22)$$

where S – is the range of depth (m); V_i – is the quantity of infiltrated petroleum product (m^3); A_i – is the area of surface infiltration (m^2); u_0 – is the retention capacity of the rocks (dry soils) for oil (liter/m^3).

Approximate values of the indicator u_0 for the main types of soils: gravel, sandy gravel – 5 l/m^3, large-grained (coarse) sand – 8 l/m^3, medium-grained sand – 15–25 l/m^3, fine-grained sand, clayey sand – 40 l/m^3.

The maximum propagation along a plane of a free petroleum product along a groundwater table (or along a capillary border) may be calculated according to the following formulae:

$$A_{gr} = \frac{(V_i - V_s) \cdot 1000}{D_0} \qquad (2.23)$$

where

$$V_s = \frac{A_i \cdot H \cdot u_0}{1000}. \qquad (2.24)$$

In formulae (2.23) and (2.24) the following designate: A_{gr} – the maximum propagation along the aquifers of the petroleum product in its liquid phase along the groundwater table (m^2); V_s – the quantity of the product retained in the zone of passage of the contaminating agent (m^3); H – the depth of the groundwater table from the earth's surface (m); D_0 – the thickness of the layer of petroleum product (mm).

Some indicators D_0 (valid for Diesel fuel): sandy gravel – 200 mm, finegrained sand – 40 mm.

The permeability of soils for petroleum products differs considerably from the indicator of the same medium for water.

Under natural conditions, contamination by oil spreads in the form of a multi-phase movement, involving soil-gas-water-petroleum product systems.

The movement of one or more phases is determined by the degree of saturation. In the water-oil system, oil may flow only when its residual saturation reaches 18–20 %. Exactly similarly, water stops flowing in the system when water saturation falls below 20–22 %.

The relative and, possibly, also absolute permeability of both media varies exponentially with intermediate contents of the phases of oil and water. The relative permeability for oil with oil saturation of 80, 60, 45, and 20 % in this case reaches approximately the corresponding values of 75, 45, 23 and 0 %.

The complexity of determining the rate of propagation of an oil product in phase under natural conditions is thus due not only to viscosity and specific gravity, but also to relative permeability and relative saturation of sediments resulting from multiphase flow. The hydraulic conductivity and the actual rate of flow of a product is thus generally lower than the rate of flow of water. Only petrol can theoretically propagate at a higher rate. However, this may occur only when the product is spreading through a coarsely granular material with a low capillary fringe above the groundwater table. In the case of a larger capillary fringe, the relative permeability for water will decrease due to a combination of water and oil saturations.

Sand models and field experiments have proved that the rate of propagation for fuel oil will not exceed a few metres per day even in well permeable sands.

The movement of water to a considerable extent is determined and caused by the system of fissures and tectonic fractures in diagenetically compressed deposits, volcanic rocks and metamorphites. Sandstones and aleurites possess lower porosity and permeability, which is explained by the presence of a cementing component as compared with loess soils with corresponding grain sizes. Fracture zones may be important groundwater flow paths and thus simultaneously serve the propagation of the free petroleum product. As a result of this process, as well as of a very low or zero residual saturation, the petroleum product may travel distances of hundreds of meters in a relatively short period of time. The leakage of aviation kerosene fuel in the Prague airport manifested itself in fissured springs, in the Cenomanian sandstones, at a distance of 700 m from the point of the kerosene leakage, five months after the incident.

In a medium of fractured permeable rocks the source and propagation of petroleum substances is markedly different from their movement in permeable unconsolidated deposits. These differences were systematized in the following five points:

1) The propagation of petroleum substances is considerably less uniform than in a porous medium. It passes chiefly through the fractures, often rapidly and at high concentrations. Thus, the determination of the degree of contamination of the groundwater surface by means of soundings is made much more complicated. Moreover, the uneven movement considerably complicates their extraction during the pumping of water for the purpose of decontaminating the aquifers.

2) The system of fractures, oriented obliquely to the direction of flow, often leads to raising the water level. As a result, such fractures raise the oil contaminants or, at least, divert them away from the original direction of propagation. In the final analysis, this leads to increasing the quantity of petroleum products and to a divergence between the direction of groundwater flow and the direction of propagation of the petroleum products.

3) The fluctuation in the groundwater level in porous soils also leads to a fluctuation in the level and thickness of the layer of oil due to the effect of capillary forces. On the other hand, in rocks whose permeability is determined fissuring, any fluctuation in the groundwater level may open up new paths for the displacement of contaminants, which leads to a new mobilization of petroleum products. Therefore, the final stage of propagation of petroleum products in fractured rocks is reached after relatively long periods of time.

4) Considering the factors enumerated above, it may be said that petroleum hydrocarbons under these circumstances display considerably greater fluctuations and variety than in permeable porous soils. The pumping out of water for decontamination must be continued for very long periods of time even when the petroleum hydrocarbon content has been temporarily reduced.

5) In this case any information on residual saturation and on the thickness of the oil layer, strictly speaking, is not quantitative in nature and can in no way be compared with similar data obtained in porous permeable soils.

The dissolved hydrocarbon components represent the greatest threat to groundwater quality. However, the natural petroleum product content in groundwater is very small, of the order of some 100 parts of a milligram per liter. The palatability of the water may be spoiled as a result of the presence in it of concentrations of hydrocarbons in any of the concentrations mentioned above. It follows from this that the horizontal propagation of contamination is considerably more extensive than the propagation of the immiscible phase component. Dissolved hydrocarbons often lead to the contamination of large areas, of up to several km^2.

The dissolution of hydrocarbons may continue so long as a body of oil remains in contact with the saturated zone. In addition, the petroleum products retained in the zone of aeration are leached out by percolation and, possibly, as a result of the fluctuation in the level of the aquifers. Ideally, the contaminated medium in a porous permeable rock form an oval, elongated in the direction

of the groundwater flow. However, the determination of the boundaries of such a zone is carried out only conditionally by determining the so-called line of critical concentration. The critical concentration (mg/l^{-1}) depends on the background of the natural hydrocarbon content, on organoleptic properties, on the sensitivity of the analytical method and, possibly, on the degree of danger to the environment.

The boundaries of contamination are not constant, since they characterize the temporary balance between the contamination and the processes of self-purification. The latter include the dilution, and the chemical and biological degradation of hydrocarbons. When the source of contamination ceases to act, the boundary of contamination will shrink to the point of the original petroleum product spillage. When the process of contamination is renewed, the boundary of critical contamination will move to the periphery of the existing contamination. Oxidation constitutes an important chemical process, in the process of purification: the cyclanes oxidize to naphthanic acids, which dissolve easily in water; the aromates are most easily oxidized to alcohols, phenols, ketones, carboxylic acids and resins. The most active decomposition of hydrocarbons takes place under the action of microorganisms, principally bacteria. Petroleum products in the zone of aeration close to the groundwater surface, are subjected to processes of aerobic degradation by the following bacteria: Pseudomonac, Achromobacter, Flavobacterium, Nocardia, Mycobacteria.

The presence of dissolved hydrocarbons in groundwater encourages deoxygenation, which is characterized by a high concentrations of ferrous iron and ammonia, as well as by the elimination of the sulphate and nitrate content. The end product of degradation is carbon dioxide. Intermediate degradation products may possess more troublesome organoleptic properties than the penetrating petroleum products. In carrying out investigations, this fact complicates the replacement of analytical methods by odour tests. However, the latter method is nevertheless very sensitive, efficient and inexpensive.

Hydrocarbons, in the form of fuel and other widely used petroleum products, acutely toxic at concentrations greatly exceeding those detectable by smell. These substances have allergic and narcotic properties, as a rule in the form of emulsions or in the free phase. Odour thresholds of dissolved hydrocarbons are shown in Table 2.4.

In estimating the toxicity of petroleum products, it is also necessary to take into account those substances which are added to petroleum products for the purpose of improving their physical properties (anti-knock and anti-freeze additives, emulsive additives), as well as the changes which they undergo in natural and technical processes. It has been proven that some products of chemical and biological decomposition, as for example, phenols and cresols are chronically toxic in nature.

The combustion of gasolines and aviation kerosenes (jet fuel) leads to the formation of polycyclical aromatic benzpyrenes. Some of them possess carcinogenic properties.

The maximum admissible content of petroleum products in drinking water is indicated in the majority of national standards in the form of a mandatory standard. Although threshold concentrations have different numerical expressions, their content varies from hundredths to tenths of parts of a milligram per liter. These concentrations are close to the threshold of detection by standard analytical methods, i.e., by means of infrared and ultraviolet spectrophotometry. It has been established that in the industrially developed countries groundwater has a slightly increased petroleum content, which often exceeds 0.02 mg/l. This indicator may be considered as the threshold content of natural hydrocarbons.

Knowledge of the background hydrocarbon content in groundwater is necessary in order to determine the propagation of contamination, as well as to determine the time necessary for decontamination.

2.2.4. POLLUTION RESULTING FROM AGRICULTURAL ACTIVITIES

Groundwater pollution due to agricultural activities has four sources: fertilizers, pesticides, domestic and livestock effluents and the food industry (Fig. 2.1). Such pollution is introduced by the application or ploughing in of chemical products and by the discharge of various substances and of solid and liquid wastes.

Pollution resulting from agricultural activities is serious because it is widespread, extends deeply into the subsoil over vast areas and is gradually reaching all the regions of the world. A significant example is the increasing nitrogen content of groundwater which has occurred during the last decade (Fig. 2.2). It has been detected over large areas and at depths of up to a hundred metres

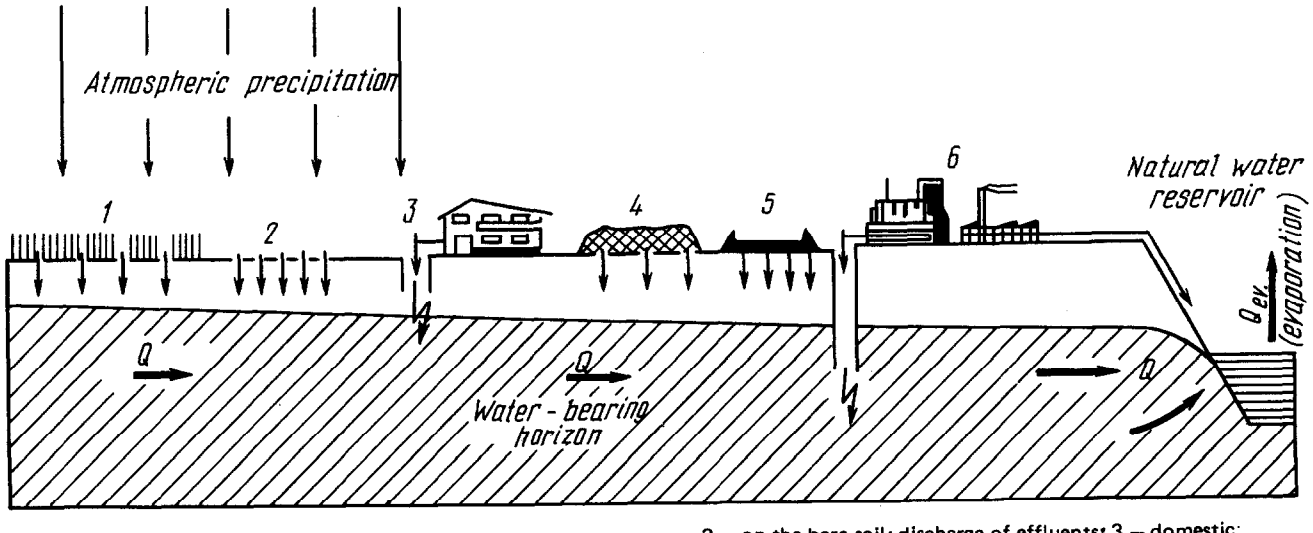


Fig. 2.1. Sources of groundwater pollution due to agriculture. Application of fertilizers and pesticides: 1 — on crops; 2 — on the bare soil; discharge of effluents: 3 — domestic; 6 — industrial; deposit of substances: 4 — solid; 5 — liquid

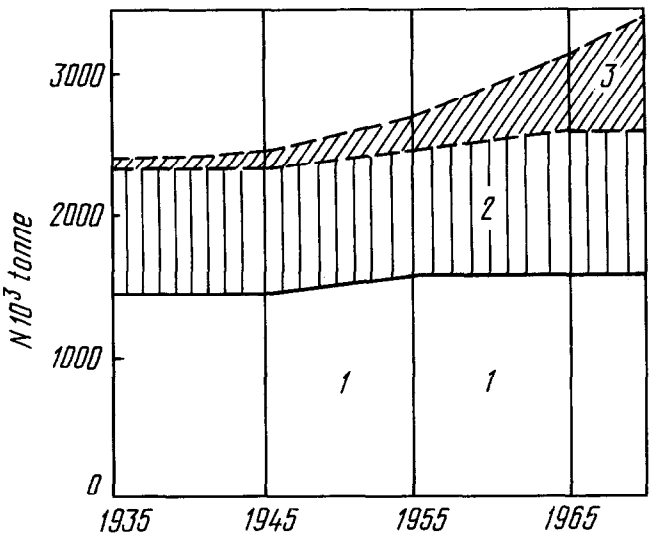


Fig. 2.2. Extent and increase in the sources of nitrogen in the soil from 1935–1965 in the United Kingdom. Source: Central Water Planning Unit, 1977:
1 – natural atmospheric nitrogen fixed by biological activity; 2 – discharge of domestic and livestock effluents; 3 – artificial fertilizers.

in many countries. In addition, through the introduction of pollutants into the global water-cycle, pollution spreads, via groundwater to surface waters, the oceans and the atmosphere. A typical example is provided by DDT. In 1966, 1,500,000 tonnes of this product were discharged on to the soil all over the world. Due to its water-solubility, it moved rapidly into the water-cycle, traces being detected even in the Antarctic ice.

The main pollutants are of mineral, organic, synthetic organic and bacteriological origin. Fertilizers and pesticides sometimes contain quite high quantities of heavy metals-arsenic, cadmium, mercury, copper, etc. Pesticides and heavy metals are micropollutants. The three main pollutants are, (1) the nitrogen compounds, (2) pesticides and (3) phosphorus. These evolve in the soil through complex chemical, physico-chemical and biological processes.

Mineral pollution by fertilizers

Four types of contaminants are introduced into the soil through the application or ploughing in of fertilizers and manure: nitrogen compounds, sulphates, chlorides and phosphates. The fertilizers also introduce various heavy metals. The quantities involved vary, according to the climate and the farming methods employed, from 10 to 10^3 kg/hectare per year.

Pollution by nitrogen compounds

Nitrogen can be present in four different forms: nitrates, nitrites, ammonium and organic nitrogen. The main pollutant is nitrate whose anion NO_3^- is extremely mobile and remains for lengthy periods in the water cycle.

Pollution by nitrates

The nitrate content of groundwater, in the form of the anion NO_3^-, has increased steadily in all agricultural regions over the last decade. It often exceeds 50 mg/l, the norm established for drinking water. Studies published recently refer to the presence of nitrates more especially in the chalk and the triassic sandstone of England, the aquifers of the Paris basin, the aquifers of the major Hungarian basin, the south-east area of the United States, various regions in the USSR, the south-west part of India and central Nigeria.

The treatment of nitrate-rid water for distribution, though technically possible, is too costly for small- and medium-sized communities. For this reason, actions to combat this form of pollution must be undertaken at its source in the farming methods.

The toxicity of nitrates

Nitrate, after being reduced to nitrite both in nature generally and also in the digestive tract, breaks down the haemoglobin in the blood (methaemoglobinaemia in infants), which is fatal to young mammals. It has also been suggested that they are precursors of carcinogenic nitrosamines which are dangerous at all stages of the development of living creatures. In addition, their transformation into toxic substances is favoured by the presence of chlorine (organo-chlorines) and metals such as aluminium, the main material used in kitchen utensils.

Origins of pollution

Nitrates are primarily agricultural in origin, resulting from the application of organic fertilizers and the excessive use of chemical fertilizers and the discharge of domestic and livestock effluents. These additional nitrates increase the natural production of nitrogen in the atmosphere which is absorbed by plant life. However, the principal cause is the development of modern intensive farming methods: deep ploughing, the absence of crop rotation, irrigation, etc. Nevertheless the presence of nitrates in old groundwater (in the absence of tritium) shows that this contamination is not always of recent origin. Their migration into the aquifers is related to the turnover rate of the groundwater reserve and to the velocity of flow and consequently those aquifers with a rapid replenishment rate are less vulnerable because of the effects of dilution. Hydrochemical methods, supplemented by the quantitative analysis of the nitrogen 15 content, are used to detect the source of nitrate pollution. The degree of accuracy is such that they can identify the brand of fertilizers used. Natural purification is achieved by denitrification, a process which is as yet little understood.

Nitrites and ammonium

The presence of nitrites (NO_2^-) or ammonium (NH_4^+) is evidence of pollution resulting either from organic fertilizers (dung and liquid manure) or from the discharge or spreading of household and industrial effluents (sugar refineries, distilleries, food canning factories, etc.). Their presence, which may be the sign of bacteriological contamination, inevitably leads to the detection of the sources of pollution.

The nitrites are unstable and are usually rapidly oxidized into nitrates in the unsaturated zone. They are therefore virtually absent in groundwater. The ammonium cation is eliminated by absorption into the soil particles. It is easily eliminated, in the form of nitrogen gas, in the treatment of tap water through the action of specific bacteria.

Pollution caused by sulphates

Sulphates, which occur geologically through the dissolving of sulphate compounds in the water-bearing rocks, are stable elements in the groundwater. However, an increase in their content in the aquifers of agricultural regions is indicative of pollution. While excessive quantities of these sulphates can result from rainfall, they are also introduced, in the same way as the nitrates, by artificial fertilizers, which contain considerable proportions. Modern farming methods are partly to blame for such

pollution. They can be detected by isotopic chemical hydrology based on the sulphur 34 content. However, this identification technique is difficult to apply on account of the fact that the range of content of this isotope in fertilizers is almost identical with that of natural sulphates.

Pollution caused by phosphates

Phosphates are one of the compounds used in many chemical fertilizers. They are characterized by mobility, in solution, of phosphorus in the form P_2O_5. They may occur naturally (through the leaching of phosphatic water-bearing rocks or as the product of the decomposition of living matter), or through the excessive use of fertilizers. The phosphorus content of groundwater has not yet reached levels which give rise to concern, but the discharge of phosphates into surface waters can encourage eutrophication.

Pollution by chlorides

Chlorides are persistent elements, of geological origin, which are naturally present in groundwater. However, higher than average amounts have been found downstream from land where fertilizers have been applied. In addition, some chemical fertilizers contain high quantities of chlorides. The Cl anion, an excellent tracer of the water cycle, is not eliminated when water runs off into the soil and the aquifers. The content is only lowered through dilution. However, the chlorides are not, apart from rare and localized exceptions, a worrying source of groundwater pollution. Their content is reduced either naturally through their dilution in the aquifers (depending on the turnover rate) or artificially in the water treatment stations.

Mineral pollution by micropollutants: pesticides and heavy metals

Micropollutants are toxic substances present in very small quantities in water, of the order of one millionth of a gram (a microgram), or even one nanogram per litre. They are dangerous, however, even in very small amounts, because of their cumulative effect in the food chain. Regular consumption by animals and by humans is often fatal. They include numerous substances from among the pesticides and heavy metals.

Pollution caused by pesticides

The pesticides or biocides include all the products used to combat the parasites which occur in agriculture and livestock raising: insecticides, fungicides, rodenticides, herbicides, etc. they are organic compounds, in general synthetic, and are very complex and varied. Their pollutant effect is generally accentuated due to their content of heavy metals—arsenic, cadmium, mercury, copper, lead, etc. — of which small quantities are toxic.

Their evolution in the soil and subsoil, which can lead to the formation of highly toxic derivatives, is still little understood. Their behaviour is characterized by their miscibility and their solubility, their polar properties (exchange of ions, polar interactions, etc.), which encourage their absorption and complexing action with the organo-humic complex of the soil. Their elimination is therefore aided by the presence of humus, but which is may be insufficient in quantity following the use of chemical fertilizers. The pollution they cause is increased when they are dissolved due to the action of reducing substances (Table 2.5).

Many of them certainly have harmful cellular, genetic, metabolic and systematic effects from accumulation. In addition, the fat-solubility of certain substances

Table 2.5

WATER-SOLUBILITY AND PERSISTENCE OF CERTAIN PESTICIDES

Type of pesticide	Amount applied, kg/ha	Water-solubility, ppm	Persistence (Time necessary for 95 % disappearance)
Insecticides			
Organo-chlorine:			
DDT	1–2.5	0.01	4–30 years
Dieldrine	1–3	1	5–25 years
Lindane	1–2.5	10	3–10 years
Organo-phosphorate:			
Parathion	—	24	—
Diazinon	—	40	—
Chlorfenvinphos	—	145	—
Carbamates:			
Carbofuran	0.5	700	120 days
Aldicarbe	1	6,000	90 days
Oxamyl	10	280,000	90 days
Pesticides			
Triazines:			
Atrazine	2–4	70	4 to 7 months
Simazine	2–5	5	12 months
Phenoxyacetic acids:			
2,4 D	5	620	1 month
2,4,5 T	5	278	3 months
Substituted ureas:			
Monuron	2–4	230	5 to 6 months

results in their accumulation and concentration in the food chain. This leads to toxicity and its dissemination throughout the natural environment, even in those cases when pesticides have been lightly applied, at authorized levels, of the order of about ten micrograms per litre. The example of DDT, which was mentioned above, is particularly significant.

In most cases, when the soil conditions are favourable (aeration, the presence of humus, etc.), these substances are rapidly fixed or destroyed through the natural self-purifying capacity of the unsaturated zone, which operates as an extremely effective filter and totally eliminates biodegradable substances.

Pollution caused by heavy metals

Heavy metals are contained in chemical fertilizers, pesticides and household and livestock waste, the principal examples being arsenic, mercury, cadmium, lead and copper. Like the pesticides, they are extremely toxic in small quantities and accumulate in the food chain. Arsenic, which is toxic in very small amounts, accumulates in the human body. It may have carcinogenic effects on the skin. Mercury is the most toxic of all the heavy metals because of its soluble salts. A dose of one to two grams of mercury chloride is fatal. Its consumption produces digestive and nervous troubles and kidney damage. Through microbial action it becomes transformed into methyl mercury which is highly toxic, causing nervous disorders leading to intellectual decline, to insanity and often to death. Even in very low doses it is responsible for chromosomal disturbances. Cadmium, which has a

very high toxicity (2 g will kill a man) inhibits cell respiration and causes genetic problems. It accumulates in the kidneys and the liver. Lead can also act as a poison (saturnism). It is stored in the bones which it then destroys. Copper may produce organic disturbances.

Pollution from domestic and livestock waste

Liquid and solid effluents from both domestic and livestock sources, whether applied on or below the surface of the soil, contain mineral, organic and bacteriological pollutants (Table 2.6).

Table 2.6

AMOUNTS OF POLLUTANTS IN LIQUID LIVESTOCK EFFLUENTS [56]

Kinds of polluting agents	Cattle	Pigs	Sheep	Poultry
Suspended solids, g/l	90	100	90	36
BOD*, g/l	10	30	10	36
Total nitrogen, g/l	4	5	4	12
(in kg per 100 kg of weight)	150	150	130	85
Phosphorous, kg/tonne	20	45	30	30
Potassium, kg/tonne	100	100	100	30
Faecal coliforms per g	2.10^5	3.10^6	2.10^7	1.10^7
Faecal streptococci per g	–	8.10^7	4.10^7	1.10^7

*BOD – biochemical oxygen demand

Domestic effluents contain high amounts of solid substances in suspension, organic matter, faecal bacteria and minerals of the order of 10 m^3/year per average-sized housing unit. Subsurface disposal of vegetable matter produces high amounts of suspended solids, particularly in organic compounds of the phenols. The phenols, small amounts of which are extremely toxic, are banned under regulations governing standards of quality. Even traces of them produce, in the presence of chlorine, chlorophenols tasting of iodoform, which is very unpleasant in drinking water, and chloromethanes which are reputed to be carcinogenic. However, they are generally rapidly broken down and their presence in the soil is of short duration.

Solid livestock effluents amount to 15 m^3/year/per animal for cattle, 3 m^3 for pigs, 2 m^3 for sheep and 0.05 m^3 for poultry.

In France, the nitrogen originating from livestock amounts to 2,000,000 tonnes, whereas the corresponding figure for the human population is 200,000 tonnes.

The prevention of pollution resulting from agricultural activities

The attempt to combat groundwater pollution by agricultural activities is mainly of a preventive nature. Prevention is based on three specific forms of action: the improvement of the farming methods employed, the treatment of domestic effluents and the use of biodegradable pesticides.

Two main types of action need to be taken in order to reduce as far as possible the migration of polluting agents towards the aquifers and their transference into the surface waters:

- the assimilation of such products by plants should be encouraged;
- the should be made of the natural self-purifying capacity of the unsaturated zone (absorption and biodegradation).

Farming methods

Growing nitrate pollution results partly from the excessive use of artificial fertilizers, which are often applied at unsuitable periods, but its main cause is the development of modern farming methods, which encourage the rapid migration of pollutants into the aquifers.

The following measures are recommended and will, in addition, result in appreciable economies, having regard to the growing cost of fertilizers:

- fertilizers appropriate for the type of crops being cultivated should be used in the correct amounts, thereby avoiding any excessive application. This measure represents a means of combating waste;
- their application during periods of heavy rainfall should be avoided;
- the destruction of the soil texture should be avoided (deep ploughing);
- the level of humic substances should be maintained through the addition of organic fertilizers and humus;
- appropriate crop rotation methods should be used, ensuring the shortest possible periods of soil denudation and avoiding soil exhaustion or the excessive use of fertilizers, etc.

Treatment and disposal of domestic and livestock effluents

All domestic and livestock effluents must be treated and purified before discharge. Prior lagooning has a considerable purifying effect. Subsurface disposal or deep well injection should be avoided. The use of broad irrigation is recommended since it makes maximum use of the selft-purifying capacity of the unsaturated zone.

Conclusions

Groundwater pollution caused by agricultural activities is, together with that of the atmosphere, the most harmful and the most serious form of contamination. It produces gradual and widespread damage to the subsurface environment. Through its introduction into the water cycle it brings about the contamination of all the environments: inland surface waters, the oceans and the atmosphere. The pollutants which cause the most concern are the nitrates and, to a lesser extent, the pesticides. The answer to this problem lies essentially through preventative action in moden farming methods and through the introduction of proper sewerage systems in rural areas.

2.2.5. INTRUSION OF SEAWATER IN AQUIFERS OF FRESH GROUNDWATER

The intrusion (or encroachment) of seawater in aquifers of fresh groundwater in coastal regions is an important hydrogeological problem for many countries of the world possessing an extended shore line.

The intrusion of seawater in aquifers of fresh groundwater is caused mainly by the economic activities of man and, first of all, by the use of groundwater. These same processes take place under natural conditions, with less strength and on a smaller scale.

According to the published literature, cases of encroachment of seawater take place in the USA, Japan, Canada, Italy, the USSR (the Baltic region, the Far East, the Crimea), the Netherlands, Mexico, Venezuela, India, Yugoslavia, Israel, Belgium, the GDR, France, United Kingdom, Morocco, Tunisia, Algeria, Australia, etc.

The intrusion of seawater into fresh groundwater takes place both under natural conditions and, in particular, when the groundwater regime is upset. Under natural conditions, the intrusion of seawater in a stratum

takes place under the effect of the difference in the densities of fresh and salt water, of tidal and gale phenomena.

The main factor causing the encroachment of seawater on aquifers is the intensive removal of groundwater, leading to a lowering of the groundwater level and the capture of seawater by the recharge zone of the groundwater source.

Along with water removal, the intrusion of seawater is encouraged by stripping and excavation work, the working of deposits of solid useful minerals, drainage, the working of gas and oil fields, etc., i.e., all types of mining works leading to the disruption of the hydrodynamic and mechanical equilibrium of the interrelated system "aquifers – sea".

The main paths of intrusion of seawater are: 1) the encroachment of seawater directly into the aquifer at the place of its discharge into the sea and 2) the tidal incursion of seawater into rivers and canals and the subsequent filtration from rivers and canals into the aquifers. In addition, the entry of seawater into the aquifer may be caused by the leakage of seawater through slightly permeable deposits, lithological "windows" and by penetration along fissures and fractures.

Intrusions of seawater affect principally unconfined groundwater and the uppermost confined aquifers.

Deep-seated aquifers, as a rule, are not subject to the encroachment of seawater.

The intrusion of seawater is favored by the existence of rivers and canals communicating with the sea which constitute one of the main avenues of penetration of seawater far inland; an indented coastline, and the presence of bays and fjords; a direct connection of seawater with the aquifers; a small discharge of fresh groundwater into the sea and the degree of heterogeneity of the aquifer and its stratification.

The surge of seawater into rivers and canals occurs under the influence of winds, gales and high tides. The distance of the seawater surge depends on the discharge of the river waters, the hydraulic gradient of the river, the width and the depth of the river bed, the duration and intensity of the winds and high tides. The surge of seawater into rivers and canals is a periodic phenomenon throughout the year.

The rate of progress through the aquifer of the encroaching seawater, varies from several meters to hundreds of meters per year; the mean values of the rate of intrusion amount to 30–60 m/year. The progress of salt water through the aquifer is not a uniform process, but a cyclical one, in which relatively short intervals of rapid progress of the front of salt water alternate with the slowing-down of the rate of displacement, temporary stabilization and even partial retreat of the salt water front. The rate of displacement of the salt water is affected to a considerable extent by such climatic factors as drought and rainfall; an increase in the duration and frequency of droughts speeds up the process of intrusion, while an increase in rainfall slows it down. The boundary between the fresh groundwater and seawater is not a sharp one, it is blurred and constitutes an area (a transitional zone) within which a gradual transition occurs from the mineralization of salt seawater to the mineralization of fresh groundwater.

As a result of the difference in the densities of fresh groundwater and seawater, the latter encroaches on the aquifer in the form of a wedge, elongated along the base of the stratum.

Its length depends on the correlation of densities of the fresh and the seawater, the width of the aquifer and the slope of flow of the groundwater. In a number of coastal areas of the United States (California, Florida) the length of the wedge, which has formed over 15–20 years, amounts to from 1–2 km to 7–8 km.

A review of the literature has shown that investigations of the interrelationships of groundwater with seawater are being carried out in various directions.

The main issues considered in these works are the following:

the methodology of calculation and identification of the "tongue" of salt water; the use of geophysical methods to study salt water intrusion; analog and physical modeling of seawater intrusion processes; groundwater regime in the coastal zone; submarine discharge of groundwater; the encroachment of seawater in stratified aquifers; optimal use of groundwater in the presence of seawater intrusion.

For the study of seawater intrusion into fresh groundwater aquifers, a set of measures is being used extensively abroad, including geophysics, tracer techniques, modeling and observations of the groundwater regime.

Great importance is attached to the setting up of a network of observation wells. Examples of the use of this method are considered in detail in the following work [15].

A prominent place is occupied by theoretical investigations of an extensive range of issues connected with this problem.

Such questions include the determination of the position of the boundary line dividing fresh and salt water, the progress of the front of salt seawater within the aquifer deep inland the modeling of the process of intrusion, the dispersion of the boundary of salt water, the intermixing of fresh and salt water, the influence of ebb and flood tides on the groundwater level, the movement of fresh and salt water in a stratified bed, the formation of a wedge of salt water under the influence of partially penetrating wells, the justification of simultaneous and separate withdrawal of salt and fresh groundwater to prevent the salinization of the latter, etc. In these studies, the works of the following authors are of particular interest: N.K. Girinskii, P. Ya. Polubarinova-Kochina, S.F. Averianov, Glover, Hubert, Lushchinskii, De Wiest, Todd, Vir, Cooper, Rumer, Karner, Harleman, et al.

The encroachment of seawater and the salinization of fresh groundwater makes it necessary to develop special measures to prevent these phenomena [15].

Measures for monitoring and preventing the intrusion of salt water deep inland have been proposed on the basis of the results of these investigations. These include: 1) decreasing the withdrawal of groundwater for the purpose of increasing its level; 2) artificial recharge of groundwater for the purpose of raising its level; 3) the setting up along the shore of a hydraulic barrier of fresh water by means of injections of such water into the aquifer; 4) the establishment of mechanical barriers (dikes, dams, partitions made of piling); 5) withdrawal of salt ground water by pumping from wells located along the seashore.

The intrusions of seawater in California, Florida and Long Island (USA) are those most studied and described in the literature.

These examples are given in work [15].

The encroachment of seawater in an aquifer, as has already been pointed out, is primarily caused by the withdrawal of groundwater. The formation of a wedge of seawater in the stratum also depends on the contrast in the densities between the fresh and seawater.

Under natural conditions, a wedge of salt water, elongated along the base of the stratum (Fig. 2.3), forms in the aquifer at the place of its contact with the sea under the influence of the difference in densities. The length of this wedge, as was shown by the works of N.K. Girinskii, D. Todd, I.A. Charney, V.P. Pilatovskii, K. Henry, A. Scheller, A. Scheidegger et al. depends on the discharge (gradient) of the natural flow of groundwater emptying into the sea, the contrast of densities between fresh ground- and seawater, and the width of the aquifer.

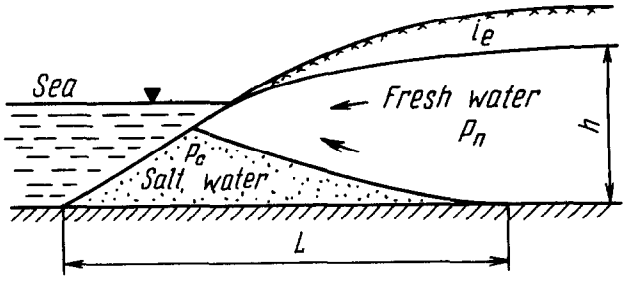


Fig. 2.3. Formation of a wedge of seawater in an aquifer of fresh groundwater (ρ_n and ρ_c – are the specific gravities of fresh ground and salt seawater).

An approximate expression for the evaluation of the final length of the wedge of salt water in the aquifer in conditions of hydrodynamic equilibrium of the fresh and salt water according to [4, 31] may be written in the form:

$$L = \frac{(\gamma_0 - 1)\,Th}{q} \qquad (2.25)$$

or

$$L = \frac{(\gamma_0 - 1)\,h}{i_e}, \qquad (2.26)$$

where i_e – is the gradient of groundwater flow; $\gamma_0 = \frac{\gamma_c}{\gamma_n}$ – is the ratio of the densities of salt and fresh water; h – is the width of the aquifer, m; T – is the conductivity of the stratum, in m^2/day; q – is the specific discharge of flow per unit of length of the coast line, m^2/day.

With a change in the discharge of groundwater, caused by the withdrawal of water, the length of the wedge changes: it increases with a decrease in the discharge of groundwater into the sea or decreases with an increase in that discharge.

The boundary between the fresh and salt water in the stratum as a result of the difference in their densities has a complex form. This question has been studied in the works of V.P. Pilatovskii, F.M. Bochever, N.N. Verigin, V.S. Sarkisyan, G. Henry et al.

Let us consider in an approximate statement the conditions of formation of a wedge of seawater in a semi-limited stratum with an outline of constant pressure (the sea shore) under the influence of a water withdrawal installation (isolated or an areal concentrated installation, or a linear row of wells) both taking into account and neglecting the difference in densities of fresh and seawater. The yield of the water withdrawal installation is constant, the wells are perfect. The connection of the aquifer with the sea is assumed to be direct, the natural flow of the groundwater discharges into the sea at the shore line.

A single water withdrawal installation

The installation is located at distance d from the shore line, which is taken as the boundary of constant pressure (Fig. 2.4).

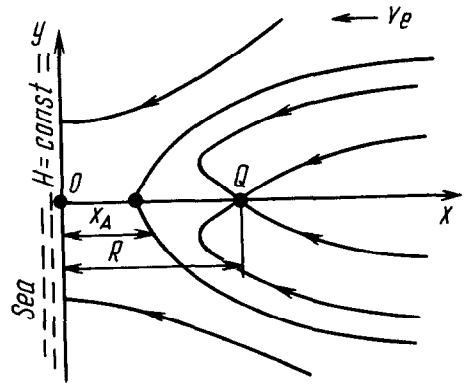


Fig. 2.4. A single water-intake near the seashore.

Axis y coincides with the coast line. The natural flow is directed along axis x towards the sea. Firstly assume that the difference in the densities of the fresh ground- and the seawater is insignificant, and that this difference may be neglected. With this assumption, the wedge of salt water in the stratum is not formed under natural conditions and the encroachment of seawater in the aquifer takes place only under the influence of the removal of groundwater.

As was established earlier [10, 22], with the well working under conditions of natural flow of groundwater, a recharge zone is formed limited by the neutral line of flow with the water-dividing point A on axis X downstream along the flow (Fig. 2.4). The position of the water-dividing point (x_A) is approximated by to the formula:

$$x_A = \sqrt{d^2 \frac{qd}{\pi V_e}}, \qquad (2.27)$$

where q – is the yield of the water withdrawal installation, related to the width of the stratum; V_e – is the velocity of the natural flow of the groundwater; K – is the coefficient of transmissivity of the water-bearing rocks.

The distance x is counted not from the water withdrawal installation, but from the shore line (of axis y).

For example, with q = 100 m^2/day, d = 2000 m, K = 20 m/day and i_e = 0.002 value x_A = 1550 m, i.e., the water-dividing point is removed from the water withdrawal installation downstream along the flow by 450 m. With the increase in water withdrawal or the decrease in the discharge of groundwater the zone of recharge of the water withdrawal installation increases and the water-dividing point is displaced towards the

shore line. Thus, with q = 200 m^2/day and the same values of the other parameters, x_A = 900 m.

The drawing up of salt water towards the well can take place only if the salt water enters its zone of recharge, and for the conditions under consideration this will happen when the water-dividing point A reaches the sea shore. Until the water-dividing point crosses the line of the sea shore, the drawing up of seawater will not occur.

The condition for this is:

$$\frac{q}{\pi V_e d} \leqslant 1 \qquad (2.28)$$

given the equality sign in formula (2.28) point A is located on the line of the sea shore. The water-dividing point crosses the sea shore and the latter is captured by the zone of recharge of the water withdrawal installation when

$$\frac{q}{\pi V_e d} > 1 \qquad (2.29)$$

From (2.28) it is possible to evaluate the maximum output from the water withdrawal installation, at which the drawing in of seawater just fails to occur:

$$q_{пр} = \pi V_e \cdot d \qquad (2.30)$$

Thus, with d = 2000 m, K = 20 m/day, i_e = 0.002 value, $q_{пр} \approx 250$ m^2/day. Assuming h = 30 m, we will obtain $Q_{пр} = q_{пр} \cdot h = 7500$ m^3/day.

If we now consider the formation of a wedge of seawater, directed towards the water withdrawal installation, taking into account the difference in densities of fresh and salt water.

Under natural conditions, the length (L) of the wedge of salt water in the stratum is determined according to formulae (2.25) and (2.26).

The withdrawal of groundwater will change the natural hydrodynamic situation and the value of the gradient of the flow discharging into the sea will decrease. As a result of this a change in the equilibrium of the fresh-saltwater system will occur. Under the conditions of the changed gradient of flow, the "tongue" of salt water will progress deeper into the aquifer and will take up a new position, corresponding to the new state of equilibrium between the fresh and the salt water. In order to determine the new length of the "tongue" of salt water, it is necessary to know the gradient of flow in the changed conditions.

The value of the gradient of flow along axis x is equal to:

$$i_x = \frac{qd}{2\pi K} \cdot \frac{d^2+y^2-x^2}{(d^2+y^2-x^2)+4x^2y^2} - i_e. \qquad (2.31)$$

The component i_x of the gradient of resultant flow is made up of two terms; the first term characterizes the gradient of flow to the water withdrawal installation (i_B), the second term is the natural flow of groundwater (i_e) discharging into the sea.

Directly along the axis x the value of gradient i_{x0} is equal to:

$$i_{x0} = \frac{qd}{2\pi K} \cdot \frac{1}{d^2-x^2} - i_e \qquad (2.32)$$

Let us consider how i_{x0} changes along axis x. In the zone x > d, the first term has a negative value and both terms of the gradient are directed the same way — against the positive direction of axis x.

In zone $x_A < x < d$ the first term has a positive value, the second a negative, the terms of gradient i_B and i_e are directed in opposite directions, but gradient i_B, determined by the rate of pumping at the water withdrawal installation is greater than the gradient of natural flow and the resulting gradient i_0 is directed towards the water withdrawal installation.

In the water-dividing point A the gradients i_B and i_e have opposite directions and are equal in absolute value and the resulting gradient $i_0 = 0$.

In zone $x < x_A$ gradients i_B and i_e have opposite directions, gradient i_e is greater than gradient i_B and the resulting gradient is directed towards the sea.

On the edge of the sea shore (x = 0, y = 0) the value of the gradient is equal to:

$$i_0 = \frac{q}{2\pi Kd} - i_e. \qquad (2.33)$$

On fulfilling condition (2.28) the first term in formula (2.33) is less than or equal to i_e. The value of gradient i_x in other points on the edge of the shore line (x = 0, y ≠ 0) is equal to:

$$i_{oy} = \frac{qd}{2\pi K} \cdot \frac{1}{d^2+y^2} - i_e \qquad (2.34)$$

or

$$i_{oy} = \frac{q}{2\pi Kd} \cdot \frac{1}{1+\bar{y}^2} - i_e, \text{ where } \bar{y} = \frac{y}{d}. \qquad (2.35)$$

As is seen from (2.34), with an increase in the values of y, i.e., with the removal from the main line of flow (of axis x), the value of the first term (i_B) rapidly diminishes and value i_{oy} strives towards value i_e.

The main zone of decrease of the natural gradient i_e on the shore line is associated with the zone of width d on both sides of the main line of flow (axis x). The maximum decrease of this gradient takes place on axis x.

Accordingly, the most important changes in the length of the wedge of seawater will be found in this zone; the maximum change along axis x.

Substituting in formula (2.26) instead of i_e the value i_{oy} from formula (2.35), we obtain an equation for the estimation of the length of the wedge of seawater under conditions disturbed by water withdrawal:

$$L = \frac{(\gamma_0-1)\,h}{i_e - \frac{q}{2\pi Kd} \cdot \frac{1}{1+\bar{y}^2}} \qquad (2.36)$$

(so that value L may be positive, the signs of the terms in the denominator have been changed). With $\bar{y}$ = 0 we find the length of the wedge along axis x.

Before pumping from the water withdrawal installation began and under conditions of an identical value of the gradient of natural groundwater flow along the whole length of the shore line, the zone of encroaching seawater had the form of a strip, the width of which characterized the length of intrusion of salt water into the stratum and was determined according to formula (2.26). Under conditions of work of the well, the change of the gradient, as shown above, will be uneven along the front of flow. Therefore, the new position of the zone of seawater, corresponding to the new state of equilibrium, will have the form of a wedge, elongated along axis x. Substituting in formula (2.36) the fixed values of $\bar{y}$, we find the position of the wedge of seawater in the stratum.

Thus, given the width of the aquifer h = 30 m, γ_0 = 1.03 and the values of the parameters indicated above, the length of the salt water zone in the stratum under natural conditions amounts to ≈ 450 m. A change in the length of this zone given water removal calculated according to formula (2.36) is characterized by the data given in Table 2.7.

Table 2.7

LENGTH OF SALT WATER WEDGE IN DIFFERENT POINTS OF SHORE LINE

$\bar{y}$	0	0.5	1.0	1.5	2.0
L, m	590	530	500	480	470

It should be pointed out that gradient i_x or i_0 (formulae (2.31) and (2.32)) does not remain constant with a change in abscissa x. At the same time, in formula (2.36) a constant value of the gradient was taken, equal to its value at the shore line, according to formula (2.33). Let us ascertain that is the error of this assumption in the example of gradient i_0. The mean value of gradient i_0 on the section (x_A, 0) is equal to:

$$i_{0cp}=\frac{1}{x_A}\int_{x_A}^{0}\left[\frac{qd}{2\pi(d^2-x^2)}-i_e\right]dx=i_e-\frac{q}{4\pi Kx_A}\ln\frac{d+x_A}{d-x_A} \quad (2.37)$$

The value of the gradient for the values of the parameters given above, calculated according to formula (2.37), yielded a value of 0.0015, and according to formula (2.33) — a value of 0.0016, i.e., very close values.

With the increase in water removal, the water-dividing point of the recharge zone of the water withdrawal installation is displaced in the direction of the sea, and at the same time the wedge of seawater stretches deeper into the aquifer.

Obviously, the drawing up of seawater to the water withdrawal installation is possible when the salt water wedge is captured by the recharge zone of the water withdrawal installation.

Accordingly, the drawing up of salt water to the water withdrawal installation is possible in the event that

$$x_A < L \quad (2.38)$$

or

$$\sqrt{d^2-\frac{qd}{\pi v_e}} < \frac{(\gamma_0-1)\,h}{i_e-\frac{q}{2\pi Kd(1+\bar{y}^2)}} \quad (2.39)$$

And, on the contrary, given

$$x_A \geqslant L \quad (2.40)$$

the drawing up of seawater to the water withdrawal installation will not occur.

From condition (2.40) the yield of the water withdrawal installation may be determined, given which the drawing up of seawater by the water withdrawal installation may be ruled out:

$$q \leqslant \pi v_e d-\frac{\pi(\gamma_0-1)^2h^2K}{i_e d}. \quad (2.41)$$

Comparing expression (2.41) with expression (2.30) for maximum yield without taking into account the difference in densities, it may be seen that the first yield is less than the second yield by the value of the correction which takes into account the difference in densities. When the densities are equal, expression (2.41) passes over into (2.30). The magnitude of this correction depends not only on the difference in densities, but also on the width of the aquifer (h) and the value of the gradient of natural flow (i_e). Analysis has shown, that given h ⩽ 30–40 m, γ_0 ~1.03 (the density of seawater usually does not exceed 1.03 g/cm^3) and $i_e \geqslant 0.002$ the value of this correction is small (not more than 5–7 % — of the value of the first term) and under these circumstances it may be neglected, and the value of the maximum yield may be estimated from condition (2.30). Thus, given γ_0 = 1.03, h = 30 m, k = 20 m/day, i_e = 0.002 and d = 2000 m, the value of the maximum yield amounts to $q_{пр}$ = 235 m^2/day, i.e., is not very different from the value of the maximum yield calculated without taking into account the difference in the densities of fresh and salt water.

With the same values of γ_0 and h, but with a decrease in i_e the value of the correction increases and the yields, calculated according to (2.41) and (2.30) already differ substantially. For example, when i_e = 0.001, the maximum yields, without taking account of, and with taking into account, the differences in the densities respectively are equal to 125 m^2/day and 100 m^2/day, whereas given i_e = 0.0005, are equal to 60 m^2/day and 20 m^2/day; with an increase in the width, the difference in the yields becomes even more pronounced. In these cases, the maximum yields should be calculated according to formula (2.41).

Thus, the drawing up of seawater to the water withdrawal installation can begin only when the recharge zone of the water withdrawal installation reaches "the tongue" of salt water which has entered the aquifer, or the sea shore along the line of which the discharge of groundwater is assumed. This is the basic condition for the drawing up of seawater. By itself, the lowering of the groundwater level at the water withdrawal installation below sea level without the observance of the basic condition in the general case is not the cause of the drawing up of salt water into the water withdrawal installation.

A linear row of wells

Let us consider a linear row of wells of great length, arranged parallel to the shore line and at a distance d from it (Fig. 2.5). The yields of the wells of the row are the same, the yield of an individual well is Q. The distance between wells is l. The axis x coincides with the shore line, axis y goes through one of the wells of the row. The

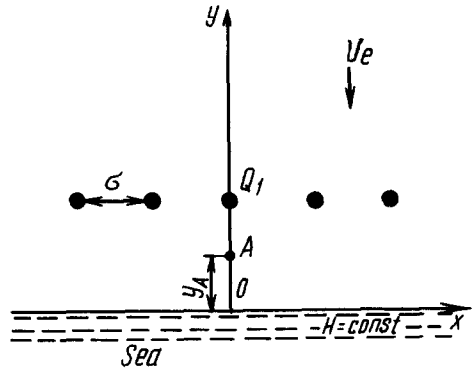


Fig. 2.5. A linear row of wells near the seashore

natural groundwater flow is in the direction of the sea and is characterized by rate of filtration v_e. The filtration regime is steady.

Just as in the case of the single water withdrawal installation, let us first consider the conditions for the drawing up of salt water from the sea boundary without taking into account the difference in the densities of fresh and salt water.

Since the filtration flow is basically rectilinear towards an unlimited series, let us confine ourselves to the consideration of the rate of filtration along the main line of flow, axis y. The expression for velocity was obtained in work [10] and has the following form:

$$v_y = \frac{q}{l}\,\frac{\operatorname{sh}\frac{2\pi d}{l}}{\operatorname{ch}\frac{2\pi d}{l} - \operatorname{ch}\frac{2\pi y}{l}} - v_e, \qquad (2.42)$$

where

$$q = \frac{Q}{h}.$$

The first term is velocity (v_s), caused by the pumping of the water withdrawal installation, the second term is the velocity of filtration of the natural flow (v_e). Under certain conditions, between the row and the boundary of recharge, a water-dividing point A is formed in which velocity $v_y = 0$.

Given $y > d$ both velocities have the same direction and add up; in the interval $y_A < y < d$ the velocities v_B and v_e have opposite directions, but $v_B > v_e$; in the water-dividing point A we have $v_B = v_e$; in the interval $0 < y < y_A$ the velocities v_B and v_e have opposite directions, $v_e > v_B$ and the discharge of groundwater into the sea takes place, but with a gradient less than the natural one. The value v_y on the boundary of the sea ($y = 0$) is equal to:

$$v_0 \approx \frac{q}{l} v_e \qquad (2.43)$$

The position of the water-dividing point A is found from the condition $v_y = 0$ and is determined by the equation:

$$y_A = \frac{l}{2\pi} \operatorname{Arch}\left(\operatorname{ch}\frac{2\pi d}{l} - \frac{q}{v_e l}\operatorname{sh}\frac{2\pi d}{l}\right) \qquad (2.44)$$

The water-dividing point between the row and the recharge boundary may exist only given a certain value of q. This value q is determined by the equation:

$$q \leqslant \frac{v_e l\left(\operatorname{ch}\frac{2\pi d}{l} - 1\right)}{\operatorname{sh}\frac{2\pi d}{l}} \qquad (2.45)$$

or approximately

$$q \leqslant v_e l \qquad (2.46)$$

The maximum value of yield ($q_{пр.}$), corresponding to the position of the water-dividing point A on the shore line, is equal to:

$$q_{пр.} \approx v_e l \qquad (2.47)$$

With the increase in the yield of the wells of the row to their maximum value, the water-dividing point shifts to the shore line and, when the yield of the wells becomes more than the maximum, the recharge zone of the water withdrawal installation captures the shore line and under these conditions the drawing up of seawater to the water withdrawal installation may occur.

Let us evaluate the values of $q_{пр.}$. With $v_e = 0.05$–0.1 and $l = 200$–400 m the value of $q_{пр.}$ varies from 10 to 40 m^2/day, i.e., the water-dividing point between the row and the sea shore is formed only given comparatively low yields of the wells of the row; with an increase in yield it rapidly moves in the direction of the shore line.

Thus, if the yield of a well of the row corresponds to the expression (2.46), the drawing up of seawater to the water withdrawal installation will not occur. If the yield of the wells is more than the maximum value, the movement of seawater towards the water withdrawal installation will begin. These criteria occur when the differences in the densities of fresh and seawater are insignificant and may be neglected.

Under circumstances of a substantial difference in the densities in the stratum in the natural situation, a zone (wedge) of seawater is formed, whose length is determined according to formula (2.26). Under the influence of water removal, the seawaters will progress deep into the stratum and will take up a new position of equilibrium. The length of this new zone of seawater may be approximately expressed by the formula:

$$L = \frac{(\gamma_0 - 1)h}{i_e - \frac{q}{lk}}, \qquad (2.48)$$

where k – is the coefficient of transmissivity.

Thus, if given $\gamma_0 = 1.03$, $h = 30$ m, $i_e = 0.002$ and $k = 20$ m/day, the length of the wedge under natural conditions amounts to 450 m, that is given $q = 10$ m^2/day and $l = 400$ m its new length will be equal to 600 m.

The drawing up of seawater to the water withdrawal installation is possible only if such water enters the recharge zone of the water withdrawal installation. The condition for this is

$$y_A < L \text{ or}$$

$$\frac{l}{2\pi}\operatorname{Arch}\left(\operatorname{ch}\frac{2\pi d}{l} - \frac{q}{v_e l}\cdot\operatorname{sh}\frac{2\pi d}{l}\right) < \frac{(\gamma_0 - 1)h}{i_e - \frac{q}{lk}} \qquad (2.49)$$

If $y_A \leqslant L$, seawater will not enter the recharge zone and will not be drawn towards the water withdrawal installation.

From condition $y_A = L$ we obtain the equation for the approximate evaluation of the maximum yield taking into account the difference in the densities of fresh and seawater, at which the drawing up of seawater to the water withdrawal installation just fails to occur:

$$q_{пр.} = lv_e - \frac{2\pi kh\,(\gamma_0 - 1)}{\operatorname{Arch}\left(\operatorname{ch}\frac{2\pi d}{l} - \frac{q_{пр}}{lv_e}\operatorname{sh}\frac{2\pi d}{l}\right)}. \qquad (2.50)$$

The condition $y_A = L$ corresponds to the case where the water-dividing point A is found on the salt water boundary line, but does not cross it.

Calculation $q_{пр.}$ is done by the iteration method. As a first approximation of $q_{пр.}$, the value is taken $q'_{пр.} < lv_e$. Substituting this value $q'_{пр.}$ in the right-hand part of the formula (2.50), the value $q''_{пр.}$ is calculated. The value found $q''_{пр.}$ is used as a second approximation for the calculation of $q''_{пр.}$ etc. Two-three approximations are enough to obtain an acceptable value of $q_{пр.}$.

Formula (2.50) is not always convenient for calculations, and it is advisable to simplify it.

Representing the inverse hyperbolic cosine through the natural logarithm and considering that given $d \geqslant l$ (which practically always occurs in real conditions) an

equality $ch\frac{2\pi d}{l} \approx sh\frac{2\pi d}{l}$ takes place and the expression (2.50) may be reduced to the form:

$$q_{пр.} = lv_e - \frac{2\pi kh(\gamma_0 - 1)}{\frac{2\pi d}{l} \ln\left(1 - \frac{q_{пр.}}{lv_e}\right)} \qquad (2.51)$$

A sub-logarithmic expression is always positive, since $q_{пр.} < lv_e$. The difference in the densities of fresh and salt water decreases the value of the maximum yield, as compared with its value under conditions where the densities of fresh and salt water are assumed to be identical.

Thus, the maximum yield of a well in the row, calculated according to formula (2.47), without taking into account the difference in densities, gives, for the initial parameters indicated above, the value $q_{пр.} = 16\ m^2/day$. The maximum yield calculated according to formula (2.51), taking into account the difference in densities, is equal to $12.3\ m^2/day$.

To study the intrusion of seawater into fresh groundwater aquifers it is necessary to organize an observation network of wells for the systematic monitoring of the hydrochemical conditions in the aquifer.

Observations of the groundwater regime, in particular of the hydrochemical regime, in coastal sites make it possible to estimate the discharge of groundwater (q) emptying into the sea and to forecast changes in the length of the wedge of salt water (L) depending on the value of the groundwater discharge.

2.3. FEATURES OF EXPERIMENTAL, LABORATORY AND TEST STATION INVESTIGATIONS IN STUDYING HYDROGEOLOGICAL PARAMETERS AND PROCESSES IN CONNECTION WITH THE PROTECTION OF GROUNDWATER AGAINST CONTAMINATION

2.3.1. INVESTIGATION OF THE PERMEABILITY OF DEPOSITS LINING INDUSTRIAL WASTE LAGOONS

Data on the permeability of deposits screening the bottom of industrial reservoirs are usually of great interest to hydrogeologists from the point of view of estimating the effect of filtration from the reservoir on the hydrodynamic and hydrochemical regimes of the groundwater of the adjacent area. It is the extent of infiltration losses which determine the potential intensity of contamination of the aquifers by industrial wastes.

There are two ways of solving the problem. If the industrial reservoir is in the planning stage, the infiltration characteristics of the lining deposits may be estimated by tests on specially drilled boreholes or by laboratory data. For reservoirs in operation, however, direct determination of the infiltration discharge is possible either from the data of a water-balance estimation carried out for the reservoir as a whole or by direct instrumental measurements at particular sites, performed by means of infiltrometers or finally, on the basis of indirect indicators, for example, of the velocities and directions of infiltration flow near the reservoir, determined under field conditions by measurements of the speed of propagation of thermal anomalies caused by the reservoir, etc. Naturally, the best results are achieved by a comprehensive approach using both methods of approach at the different stages of the investigations.

Injections into boreholes over a long period of time remain the basic method of estimating the permeability of underlying strata in the zone of aeration; in particular, of the lining deposits underlying planned hydraulic installations. Injection techniques and their interpretation are described in the literature [7, 28]. At the same time, it has been noted repeatedly that for a number of reasons, mainly because of the extremely complex field conditions of filtration near the borehole, the results of the injections often prove unreliable (especially when testing low permeability deposits). Moreover, the injection results are affected substantially by the following factors which are either completely neglected or only partly taken into account in standard experiments:

1) lateral spread of the water injected into the borehole;
2) incomplete water saturation of the pore space in the process of injection;
3) change in the estimated gradient because of failure to take account of stratification and capillary effects or, on the contrary, of head near the water percolation front;
4) diffusion effects;
5) rock stress at variance with conditions forecast.

Therefore, reliability of the experiment results may be expected only in those cases where the following conditions are met:

- direct tracing of the moisture front displacement or attaining a stationary regime during the experiment;
- monitoring changes in rock moisture content at different depths or making the experiment last long enough for a stationary moisture distribution enough for a stationary moisture distribution within the moisture zone to be attained;
- obtaining in the course of the experiment a stress state of the rocks which corresponds to subsequent operating conditions.

On the whole, the quality of the experiment improves with its scale and duration. Thus, enlarging the area of injection reduces the importance of lateral spread and smooths out the effect of the planar heterogeneity of the deposits tested. It has now been proved that the use of two or even three protective casings during the injection does not rule out the effect of lateral spread on the results of the experiment. Even in relatively homogeneous soils it is possible to obtain a practically one-dimensional filtration pattern beneath the centre of the borehole only where the diameter of the injection hole is of the order of several meters. A comparison of the results of large-scale and standard injections has shown substantial discrepancies (of several orders of magnitude) between the values of permeability obtained.

An increase in the duration of the injection reduces the influence which capillary and diffusion effects near the moisture front have on the results of the experiment and helps to limit the role of moisture dynamics within the moisture zone. In addition, the organization of protracted large-scale injections usually implies various methodological improvements designed to enhance the information derived from the experiment, including the setting-up of infiltrometers, point piezometers and pore pressure sensors.

It is true that one must bear in mind that in experiments with soils of natural composition it is considerably more difficult to set up equipment (for monitoring and measuring) without noticeably disrupting the soil being tested, than to study artificially created screens, and, moreover, the hydrogeologist here is extremely limited

in the choice of means and methods of investigation. The method of testing the cross-section of heterogeneous rocks of the zone of aeration which is simplest in its concept, but extremely laborious, is the layer-by-layer method of testing. It consists of the successive study by injections of the entire thickness of the rocks of the aeration zone which is made accessible for examination by gradually deepening the boreholes.

Laboratory experiments constitute an important supplement to experimental injections. In particular, it is advisable to carry out laboratory determinations of infiltration properties on specimens of comparatively homogeneous soils of different grain sizes (such for example as tailings) or of clayey soils of disturbed structure. In addition, under laboratory conditions some specific experiments can be conducted much more simply and smoothly; for example, the investigation of changes in soil permeability under the influence of external stresses or the study of the influence of various dissolved components on the infiltration properties of soils [14]. The importance of such investigations is borne out by the results of infiltration tests of loess-like loams from the deposits of the Kursk magnetic anomaly, whose infiltration coefficient was reduced 2–3 times under a load of 1–1.5 kg/cm^2. Another example may be found in experiments on the infiltration of diluted (to 20 mg/l) solutions of polyacrylamide through specimens of sands, fine-grained tailing and loams from the same deposits. The above polymer was interesting because it was used as a flocculating additive during the concentration of tailings of iron-quartzites, and therefore was found in large quantities in the tailing dump of the Lebedinsk ore-concentration combine. The experiments mentioned demonstrated that when solutions of polyacrylamide were passed through finely dispersive soils the coefficient of transmissivity of the latter can decrease by several times [24].

With a view to direct tracking of the intensity of both lateral and vertical spread, it is possible to observe the displacement of a chemical tracer which is added to the water injected. In experiments of this nature [24], use has been made of a common salt solution and the changes in the electrical resistance of the rocks observed after injection, making it possible to determine the position of the boundaries of lateral spread in a fairly detailed way (but only once – at the final stage of the experiment). However, the use of water with high mineralization may lead to a serious increase in the permeability of rocks containing a clayey fraction.

In order to estimate the vertical velocity of moisture flux and to map the boundaries of lateral spread, use is sometimes made of neutron logging by means of a moisture tracer NMT [28], which makes it possible to observe the dynamics of moisture redistribution in the soil without disrupting its natural composition. However, the use of NMT may be expected to have a positive effect only in those cases where the initial volumetric moisture content of the rocks being tested does not exceed 15–20 %. If, however, the rocks possess a higher moisture content, the NMT can be used to track the displacement of the front of the tagged liquid. In this case, a 3 to 5 % solution of boric acid may be used as a tracer. The additional advantages of such a method are due to the fact that NMT establishes the averaged tracer content in a spherical soil volume with a diameter of 20–30 cm.

Observations of the ground and surface water regime at construction sites of hydraulic installations, which are, comparatively, uninformative during a period of undisturbed conditions may aquire greater significance after the starting up of operations of artificial reservoirs.

They make it possible to evaluate the nature of the hydrodynamic relationship between reservoir and groundwater, and also to determine both the total losses from the reservoir and the intensity of infiltration at particular sites. In addition if the value of the hydraulic gradient is known the coefficients of transmissivity of the screening deposits may be easily determined.

The balance method of determining infiltration losses is the most accessible and widely used in practice. The planned period of time for which the balance is drawn up should comprise not less than several 24-hour periods or, even better, several weeks.

In cases where the operation of a circulating water supply system of an enterprise does not make it possible, during such a period, to determine accurately enough the specific aspects of the balance (as a result of low daily filtration losses compared with the daily water circulation) the most favorable periods for estimating the balance are down-times in the operation which are known in advance.

Direct methods of determining filtration losses include infiltrometer observations at particular sites of the reservoir with subsequent integration of the results over its entire area. In this connection, infiltrometers of various types [24] can be used, which provide a reliable record of filtration velocities exceeding 0.1 m/day.

In conducting infiltrometer observations, the permeability of deposits near the bottom may be determined at the same time. For this purpose, piezometers must be set up in the lower part of the lining layer or pore pressure sensors must be installed at previously marked experimental sites.

Comparing two methods under consideration, it should be noted that both the balance calculation of filtration losses and infiltrometer measurements are each important in their own right and can mutually supplement each other. If, under favorable conditions, the first method makes it possible to obtain sufficiently reliable values of the total infiltration from the reservoir, the second enables us to identify sites with different permeability of the bottom deposits and specific filtration losses.

For the determination of filtration velocity in slightly permeable bottom deposits, hydrogeothermal methods may be used successfully. The thermal sounding method consists of determining the temperature gradient in the upper (approximately 1 m) layer of deposits near the bottom by means of a special probe equipped with a temperature-sensitive element. It may be assumed that a change in temperature in the reservoir in the course of 24 hours corresponds approximately to the sinusoidal law. In this case, the change in temperature with depth will depend to a substantial extent on the velocity of filtration and will be determined by the solution obtained by Stallman [73].

The main cause of possible inaccuracies in this method turns out to be error in evaluating the coefficient of heat conductivity of the deposits near the bottom. However, substituting in Stallman's solution the extreme possible values of the coefficient of heat conductivity of the water-bearing rocks – 25 and 75 kcal/m in ^{o}C – it is easy to see that even in these cases the error in determining the velocity of filtration will not exceed 0.02 m/day.

Moreover, this value may be considerably reduced by carrying out standard thermal soundings at sites where

infiltration is known to be absent and after having determined the required thermophysical parameters.

The most convenient time for conducting thermal soundings is the period when temperature gradients in the bottom deposits reach their maximum values.

2.3.2. FIELD EXPERIMENTAL MIGRATION INVESTIGATIONS IN WATER-BEARING STRUCTURES

Experimental migration work (EMW) is the name we give to field experiments in the course of which the tracing of groundwater (usually through boreholes) is conducted for the purpose of determining migration parameters. The duration of these experiments, depending on the feasibility of conducting them during the prospecting of a site, is comparable with the time required for experimental filtration tests.

The works [2, 6, 23, 24, 33, 39, 40, 46, 50, 51, 56, 57, 65, 66, 82] are devoted to the development of theoretical models of testing using tracers.

Type curves for tracer testing of strata

Tracer experiments may be classified, primarily, on the basis of the method of tracer infection into the aquifer and the filtration regime of the experiment; the relevant type curves are classified accordingly.

In carrying out experimental migration work (EMW), tracer concentration at the point of injection into the water-bearing stratum may vary as follows: 1) instantaneous rise of tracer concentration and maintenance at this constant level throughout the entire experiment; 2) "package" injection — maintenance of a constant level of concentration only during a pre-determined time; 3) "pulsed" injection — production of high concentrations of tracer during an extremely short interval of time, during which the entire tracer enters the stratum.

The type of tracer input signal determines to a considerable extent both the information derived from the experiment, with respect to the migration parameters being measured, and the simplicity and ease of processing the "response" curves.

In particular, in applying the "package" and, especially, the "pulsed" types of injection under conditions of large tracer dilution, as well as in media with high dispersive or absorbent properties (for example, in fractured porous rocks), the reliability of the experiment falls sharply and is substantially limited by the sensitivity of the tracer methods used.

Experimented migration testing of water tables may be subdivided into experiments in natural and artificially created (disturbed) filtration flows. In all cases however, the filtration regime in the zone of tracer progress should be, as far as possible, stationary or close to such — conditions, as a rule, realistic enough in view of the low transfer velocities as compared with the propagation velocities of filtration disturbances.

In natural flows, experiments with two boreholes injection (charging) and monitoring (observation), are the most common.

Both these experiments and their variations suffer from substantial shortcomings due to the need for an independent estimate of the hydrodynamic parameters and the difficulties of determining the exact direction of flow.

In addition, the small velocities of natural filtration (and, as a consequence, the need of considerable expenditures of time) do not make it possible to cover areas of representative size, which leads to scale effects difficult to take into account.

Creating a disturbed filtration regime and, especially, combining migration and experimental filtration work eliminates the main shortcomings characteristic of experiments in natural flows. Thus, in some cases it is possible to estimate the migration parameters in a filtration flow produced by a discharge borehole when the tracer is introduced into a nearby observation borehole and recording is carried out in the discharged water. Also, it may be possible to carry out experiments on drainage boreholes of mines, water withdrawal wells for industrial purposes, etc. However, such an experimental plan has a number of shortcomings, of which the most important are the great dilution of the tracer in the discharged water and the extremely short time available for recording it.

The most widely used experiments follow the method of injecting the tracer solution into a central borehole with subsequent tracking of its propagation in the stratum by means of observation boreholes.

A serious shortcoming of this method is the marked irregularity of the velocity field, with velocities falling rapidly as the distance from the central borehole increases.

In coarsely fractured rocks this conditions often conflicts with the need to eliminate the influence of scale effects.

In addition, the existence of a natural flow seriously distorts the transport of the tracer in peripheral zones. From this point of view, the situation may be improved by conducting the experiment by the "doublet" method, where, prior to the injection of the tracer, a steady filtration flow is created between the injection (charging) and the discharge (tracer detecting) boreholes; which also reduces substantially the amount of drilling required for the experiment.

Additional advantages of the "doublet" method consist of more even velocity distributions at the test site, more rational utilization of tracer (which, moreover, partly returns to the surface), the simplicity of ensuring continuous monitoring of the concentration at the outlet and the possibility of an additional check estimate of the experimental results on the basis of the total volume of "returned" tracer. Conversely, experiments of this type suffer from great technical complexity and difficulties in the interpretation of the relevant type curves.

Analysis of requirements for conducting experimental migration work

At the present time experimental migration work (EMW) is carried out mainly in the form of single experiments of an investigative nature and can in no way be considered as a standard type of hydrogeological work.

However, we observe a substantial increase in the number of hydrogeological investigations which require a sound solution of the problems of groundwater protection and, hence, a sufficiently reliable determination of initial migration parameters. In this connection, it is relevant to raise the question of the advisability of including EMW in the general range of hydrogeological investigations.

To give a valid answer to this question, due account must be taken of the following: 1) the nature of the proposed forecasting problem and the significance of the parameters to be determined for calculating forecasts; 2) the real possibilities of determining particular parameters by field experimental migration work; 3) the factors which, even with optimal organization of the

EMW (within the limits of reasonable expenditure) can considerably impair the reliability of the experiment; 4) possible alternatives to EMW, i.e., the competitiveness of other methods (in the first place, laboratory methods) of determining parameters; 5) the methods adopted in practice for filtration testing of water-bearing systems, on the basis of which the geofiltration parameters, necessary for the interpretation of EMW, are obtained.

With respect to the nature of the forecasting problem, we shall further assume that we are concerned with estimating the danger of contamination of particular water withdrawal installations (using groundwater) by fixed existing or proposed surface sources of contamination. As an example, it is assumed that the time of transfer is measured in months, and more often in years; in other words, it is assumed that in the presence of real engineering investigations the scale and the duration of the EMW will turn out to be, as a minimum, one or two orders less than in the subsequent forecasting problems in the solution of which the results of the EMW are used.

The other factors listed above are reflected – for typical lithologic water-bearing structures – in Table 2.8 and its subsequent explanatory notes.

From Table 2.8, a sharp distinction is drawn between the conditions for conducting experiments in porous rock formations, on the one hand, and substantially fractured formations, on the other, particularly, differences in capacity and in real filtration velocities, as well as in the possibility of preserving the structure of water-bearing rocks under experimental laboratory conditions. Taking account in particular of the real transfer velocities, let us further assume that in rocks of the first type, the scale characteristic of the area covered by the experiment is measured in meters, while in fractured rocks it is measured in tens of meters (of course, these are somewhat approximate, averaged figures).

Chart I-1 corresponds to conditions where tracer testing covers only part of a stratum with a quasi-homogeneous structure. However, field experiments give, as a rule, noticeably under-estimate values of active porosity in sandy rocks, due to the heterogeneity of the real strata. At the same time, laboratory experiments on homogeneous specimens of sandy rocks (if properly organized) usually given substantially higher and more realistic values of active porosity, close to the total porosity values of sands.

Chart I-2 deals with consolidated stratified systems and is subdivided into two standard versions.

Subchart I-2a is valied for close permeability values (within the limits of one order) of the rocks in separate layers in the tracer testing zone, i.e., when the requirements for applying the planar filtration model are met in all layers. As a rule, filtration tests of such layers are carried out jointly, as a result of which an averaged parameter (total conductivity) of the stratum is obtained. However, filtration testing of this nature, perfectly valid per se, rules out the possibility of interpreting the results of EMW. The testing of such a stratum by small intervals in width gives an idea of the parameters of individual layers penetrated by the tracer, but only if there is a detailed interpretation of the heterogeneity of the stratum, which is often impossible. However, total testing of the stratum is usually inadvisable both for technical reasons and from the point of view of the quality of the experiment: the observation boreholes will record successively concentration waves corresponding to the inflow of tracer into the separate layers, so that the requirement of layer-by-layer differentiation of flow discharge must be met for the interpretation of the discharge curves. Among the migration parameters, active porosity (the weighted mean over the widths of layers) is of major importance for predictions, and this parameter is determined reliably enouth under laboratory conditions. Moreover, the accuracy of subsequent predictions will depend not on the migration parameters but rather on the reliability of the structural, geological and filtration analysis of the water-bearing stratum. In general, field experimental migration work in these cases adds little to what can be done in the laboratory.

Subchart I-2b is valid for markedly different permeability contracts (1–2 orders and more) between separate layers of rock in the tracer testing zone, i.e., when the required conditions for overflow exist to meet the corresponding filtration problems. At the same time, heterogeneity shows itself in different ways depending on the nature of the experiment and the parameters of the medium. In the case of thinly interbedded high and low permeability strata a layer by layer measurements of hydraulic conductivity are required. With separate testing, however, the experimental conditions are usually equivalent, to chart I-1.

Chart I-3 applies to water-bearing systems of unconsolidated materials, containing statistically distributed inclusions of rocks of relatively low permeability. In order to define the parameters of macrodispersion, necessary in predictions of contamination in such systems, the tracer testing zone must encompass the entire thickness of the stratum and exceed by approximately one order of magnitudes the extent of the low permeability inclusions. These conditions are obtained only in experiments comparable in scale with test projects in operation. On the other hand, it is important to recall that the porosity of the rocks of the structure under consideration varies to a relatively small extent and its estimated value may, in a first approximation, bet set a priori.

Thus, in determining migration parameters in water-bearing formations composed of porous (sandy-clayey) rocks, EMW is either not effective or does not yield tangible advantages in comparison with laboratory determinations (with the exception of those particular cases where EMW is implemented in the form of experimental operating projects). At the same time, the importance of laboratory experiments is also enhanced because of the need to study the sorption properties of rocks.

These conclusions, however, do not concern tracer testing carried out incidentally with other experiments (i.e., without substantial additional expenditure). Here questions of combinations of tracer injections with experimental filtration work deserve special attention – chiefly in the concluding stages of conducting group test pumpings. New possibilities are thereby also opened up for the analysis of pumpings, and for the detailed examination of filtration properties in cross-section.

Chart II-1 refers to experiments in fractured rocks, with negligibly low porosity of the rock masses. The stepped-up role of dispersion effects in this case requires a reliable determination of two estimated parameters – those of active fracturing and of the coefficient of microdispersion. Under the conditions represented by this chart there is no other alternative to EMW. The same may be said also about chart II-2, dealing with fractured-porous rocks, but here a combination of EMW with laboratory work is necessary to determine the porosity

and the coefficient of molecular diffusion of the rock masses.

Both theory and practice indicate that the groundwater contamination of potential areas is in substantially fractured rock formations, and following conclusions may be drawn: EMW must be included in the rage of hydrogeological investigations at all sites where it is essential to protect groundwater associated with fractured rock formations against contamination. The following considerations of methods are mainly concerned with these rocks.

Planning field migration experiments

It is advisable to tie in tracer testings with group test pumping sites. The use of observation wells of the filtration cluster usually does not eliminate the need to drill additional piezometers, since the disposition of observation wells in experimental pump tests is guided by essentially different criteria. The experiments must be carried out in such a way as to make it possible to bring an area of representative scale within the action of the tracer, particularly in the case of fractured rocks, because of the peculiarities of the macro-block structure of the medium and the anisotropy of its filtration properties. In the case of low storage capacity of the porous rocks (chart II-1), the propagation of the tracer is characterized by comparatively high velocities, so that, during a time of the order of the first 24-hours, the testing may actually cover a zone with a radius of tens of meters, and sometimes, of the first hundreds of meters. Where the blocks have substantiated porosity (chart II-2) a considerable part of the tracer is assimilated rapidly into the pores of the blocks, and the speed of its travel along the fractures is determined essentially by the rate of the process of molecular diffusion, which depends, in turn, on the degree of fracturing.

Two methods of conducting EMW are to be considered most effective in testing fractured media: 1) injection of the tracer into the stratum and tracking its propagation by means of observation wells; 2) injection of the tracer in the operating area of the discharge well with recording of it in this well — the "doublet" method. Both methods may be applied with either with continuous or lengthy "package" injection.

For water tables which do not lie very deep (up to several tens of meters), the first method of clustered testing proves preferable, since at shallow depths effective tracking of tracer transfer may be carried out at the simultaneously in a number of observation wells. In planar homogeneous strata the experimental cluster must include not less than four observation wells situated along the same path, but in testing a formation of "purely" fractured rocks (chart II-1) there usually must be a minimum of two such paths, with not less than three wells in each of them. For preliminary testing preceding the final planning of the main experiment, a brief "package" or "pulsed" injection of tracer in the observation well is advisable, coinciding with group pumping.

In testing water tables located at relatively large depths, the method under consideration becomes inpracticable for two reasons: a) the amount of drilling increases considerably, which makes the work more expensive; b) the effective monitoring of the movement of the tracer tag becomes much more complicated. Therefore, the migration testing of strata using the second — "doublet" — method is more acceptable under such conditions.

The maximum distance between the points of injection and withdrawal is limited also by the velocity and direction of the natural flow: this distance may be approximately evaluated according to the formula [10]:

$$r = (0.01 - 0.02)\frac{q}{v_0}, \qquad (2.52)$$

where v_0 — is the velocity of natural filtration flow.

The choice of the length of the test interval in the section plays an important role for subsequent qualitative interpretation of the experimental data. It should not encompass zones of noticeably different permeabilities (depending on the thickness of the stratum); otherwise the necessary averaging of filtration velocity will entail significant errors.

The quality of testing in EMW can depend substantially on the extent to which the water composition (tracer concentration) varies vertically. Particular attention should be paid to the design of experimental wells so as to minimise the chances of mixing or bypassing of flows from different levels (in particular, in the space behind the well lining). Test conditions may be considerably improved by installing special sectional piezometers which do away with the mixing of water in the well shaft and make it possible to evaluate the tracer concentration in each section separately. The use of such piezometers not only increased the accuracy of the tests to be achieved but also permits much more detailed information on the velocity distribution of the tracer movement within the stratum to be obtained. In this way, the scope of EMW as a whole will be significantly expanded, particularly with respect to the interpretation of the results of tracer experiments in stratified deposits and formations of fractured rocks.

Hydrogeophysical investigations in experimental wells are an important and useful supplement in the successful planning and conduct of experiments, in particular flow, and resistance measurements (using a thermal tracer — thermometry), which make it possible: 1) to determine the permeability (fracturedness) of the formation under test in the vertical plane; 2) to monitor filtration velocity determinations at observation well sites; 3) to track the tracer wave continuously by means of an observation well; and 4) to evaluate the hydrochemical inertia of the piezometers caused by the dilution of the tracer in the observation well and by the clogging of its filtration zone. During the course of the experiment the concentration of tracer in the stratum immediately at the outer wall of the well will differ from the concentration in the well. The period of time during which such a disparity is observed may be comparable with the duration of the experiment itself. Therefore, it makes sense to speak of the parameter of hydrogeochemical inertia of observation wells as of the final speed of response of the measuring system to the dynamics of tracer flow. In order to estimate this parameter before the beginning of the main experiment, salinization, and resistance measurements of the observation well are carried out. Otherwise, it is necessary to prime the observation well with a small discharge before withdrawing a test sample.

Let us proceed to the choice of a tracer. The first requirement is low cost. From this point of view, among the most useful are readily soluble electrolytic salts such as NaCl, $CaCl_2$, NH_4Cl, as well as dyestuffs (for example, fluorescein, methylene blue). When using common electrolytes, field measurements, accurate in the range

5–7 % are commonly made using a portable specific ion electrodes.

The use of radioactive tracers has great advantages: 1) reliable recording of the tracer tag is guaranteed even under conditions of high dilution (for example, in the water of a discharge well), and therefore there is no need to prepare concentrated solutions; 2) there is a sufficiently wide choice of various isotopes, including some which are extremely inert; 3) it is possible to use highly sensitive methods of monitoring the radioactive tag in the well shaft without having to withdraw samples.

At the same time, the use of radioactive tracers is often connected with considerable difficulties of an organizational nature caused by special requirements for the transportation and storage of isotopes, as well as by conditions of health protection; all of which, at times, makes radioactive tracer methods inaccessible to the majority of hydrogeologists.

The initial concentration of tracer is determined by a number of factors: the sensitivity of the detection methods, the degree of dilution expected in the stratum and in the measuring system (the well), and the effect of mineralization on changes in the filtration properties of the rocks.

A factor common to all experiments, which limits the use of high initial concentrations, is the density convection which occurs because of differences in the volumetric weights of the carrier flow and the migrating tracer liquid: under the effect of gravitational forces the "tracer" pulse may be displaced from the horizontal plane.

Accordingly, in testing a water table containing fresh water, the mineralization of the tracer solution should not exceed 1–2 g/l – for experiments in natural flows and 2–4 g/l for experiments in conditions of forced filtration.

The analogy between the processes of mass and heat transfer makes it possible to use thermal tracers also for the evaluation of migration parameters of fractured – porous rocks. Their main merit consists of the possibility of deriving from the experiment substantially different correlations of the diffusion (conductive) and convective components of the transfer process; moreover, the combination of thermal and salt tracers in migration testing enables us to highlight some processes and to "tone down" others – within the framework of the method adopted. In addition, thermal tracers – as compared with salt – have a number of other advantages: sorption effects are left out of account, continuous tracking of output data is possible by thermal sensors, and it is unnecessary to prepare large amounts of salt solutions.

Interpretation of type curves

The interpretation of the experiments begins with an analysis of the experimental tracer curves of change in relative concentration ($\bar{c}$) in time (t): for purely fractured rocks (chart II-1), these are smooth symmetrical curves, whereas for fractured-porous rocks (chart II-2) an acute asymmetry is typical (rapid growth of tracer concentration with values of $\bar{c}$ of less than 0.5–0.7 and, on the contrary, a comparatively slow change in it in the final parts). These characteristics are expressed most clearly in experiments with the injection method using a continuous or lengthy tracer shot; in other alternatives the direct interpretation of the curves by their shape may prove difficult.

The quantitative interpretation of experiments with injection into purely fractured rocks may be carried out according to the estimated method of microdispersion, in particular, following the formula:

$$\bar{c} = 0{,}5 \operatorname{erfc} \xi, \left(\xi = \frac{1 - \frac{ql}{\pi n r^2}}{2\sqrt{\frac{4}{3}\frac{\delta_1}{r} + \frac{2\pi}{q} D_M}}\right), \qquad (2.53)$$

where δ_1 is the indicator of the geometry of the fractured rock taking account of the linear relationship between filtration velocity v and the coefficient of microdispersion: $D = D_M + \delta_1 v$ (the second term in the denominator may usually be neglected).

The experimental curve, according to this formula, is rectified in the coordinates $\xi \div t$.

For fractured-porous rocks, unlimited capacity of the blocks is assumed for subsequent calculations. Neglecting hydrodispersion in the fractures and convective transfer in the blocks, the solution may be written as follows:

$$\bar{c} = \operatorname{erfc} \xi, \left(\xi = \frac{\pi r^2 S\delta}{2q} \sqrt{\frac{n_0 D_M}{t - \frac{\pi r^2}{q} n}}\right) \qquad (2.54)$$

For processing, the experimental curve is rectified in the coordinates $\frac{1}{\xi^2} \div t$. For purposes of interpretation it is recommended that those parts of the experimental graph where the relative concentration exceeds 10–20 % should be used. The processing of experiments with a two-well (doublet) system makes it necessary to use special calculation tables. For the purposes of approximate evaluation, the value of the extent of fracturedness (n) for purely fractured rocks may be found from the formula:

$$n = \frac{q t_{cp}}{\pi r^2}, \qquad (2.55)$$

where t_{cp} is the time of recording of the relative concentration $\bar{c} = 0.5$. In experiments with fractured-porous rocks the data on time t_{cp} make it possible to approximately estimate the complex parameter of mass exchange:

$$S\sigma^2 D_M n_0 = 0.04 \left(\frac{q}{r^2}\right)^2 t_{cp}. \qquad (2.56)$$

In using a thermal tracer the equations given above may lead to large errors since the calculation method of unlimited capacity may prove unacceptable; therefore, for thermal tracers solutions are used which are derived from the method of concentrated capacity.

In conclusion, it should be emphasised that special attention must be paid to tracer testing of the zone of aeration, which is only beginning to be studied.

2.3.3. LABORATORY METHODS OF DETERMINING MIGRATION PARAMETERS

Two models, homogeneous and heterogeneous, are used for the description of mass transfer in the process of filtration through porous media. The homogeneous (dispersion) model is described by the equation of convective diffusion, which, for a uni-dimensional flow, has the form

$$n\frac{\partial c}{\partial t} + v\frac{\partial c}{\partial x} = \tilde{D}\frac{\partial^2 c}{\partial x^2}, \quad \tilde{D} = D + \delta v, \qquad (2.57)$$

where n – is the active porosity of the rock, $\tilde{D}$; δ – is

the effective coefficient and parameter of microdispersion; D – is the coefficient of molecular diffusion; v – is the velocity of filtration; c – is the concentration of the solution; x – is the linear coordinate; t – is the current time. This model is the most widespread in the hydrogeological literature. However, experimental data exist, which show that the homogeneous model does not always satisfactorily describe the process of migration, even in macroscopically homogeneous porous media. In particular, the model [4] proves unsuitable in practice for the description of processes in media of limited extent, with variable boundary conditions and heterogeneous initial conditions [21, 33].

In a number of works [1, 4, 21, 33], the study of the laws of migration a porous medium is presented in the form of a system of fully mixing cells successively connected by narrow flow-through canals. The assumption that the time of entry and exit of a particle from the cell is a random value leads to the heterogeneous model of migration

$$n\frac{\partial c}{\partial t}+v\frac{\partial c}{\partial x}=D\frac{\partial^2 c}{\partial x^2}-\frac{n\delta}{\omega}\frac{\partial^2 c}{\partial x\partial t}-\frac{n^2\delta(1-\omega)}{\omega v}\frac{\partial^2 c}{\partial t^2}, \quad (2.58)$$

where ω – is the relative volume of the cells in a volume of pores. Equation (2.58) also follows from the stochastic theory of random walks, in which the effect of autocorrelation is taken into accout [71].

The comparison of (2.57) and (2.58) shows that the homogeneous model is a two-parameter one (n and $\widetilde{D}$), while the heterogeneous model is a four-parameter one (n, D, δ and ω). It is known that already at comparatively low velocities of filtration molecular diffusion may be neglected. Therefore, in the majority of cases of practical interest, one may assume in the homogeneous model that $\widetilde{D}\approx\delta v$ (i.e., parameter δ appears as the unknown quantity), while in the heterogeneous model D = 0 (i.e., the heterogeneous model becomes a three-parameter one).

Another approach to the construction of a heterogeneous model, set forth in a number of works, [8, 17, 43], is based on the premise of a two-component structure of the medium: a) a system of open (flow-through) pores; b) a system of dead-end (blocked) pores. The laws of migration in such a medium may be described by the following system of equations:

$$n(1-\omega_1)\frac{\partial c}{\partial t}+n\omega_1\frac{\partial c'}{\partial t}+v\frac{\partial c}{\partial x}=\widetilde{D}\frac{\partial^2 c}{\partial x^2},\ n\omega_1\frac{\partial c'}{\partial t}=\alpha(c-c'), \quad (2.59)$$

where c and c′ – are the concentrations of substance in the open and blocked pores respectively; ω_1 – is the relative volume of the blocked pores; α – is the coefficient of mass exchange between the open and the blocked pores. Analysis shows that the system (2.59) is practically identical to the model (2.58) with the following substitution of parameters $D\to\widetilde{D}=D+\delta v$ and $\delta\to\omega_1 v/\alpha$. This means that the parameter of microdispersion δ depends on the velocity of flow. However, the nature of this dependence requires experimental study, inasmuch as the parameters α and ω_1 in turn depend on the velocity of filtration.

For the description of the processes of sorption in model (2.57) or (2.58) instead of term $n\partial c/\partial t$ there should be substituted $n\partial c/\partial t+\partial a/\partial t$ (where a – is the concentration of the substance in the absorbed phase). In addition, the corresponding model is supplemented by the equations of the kinetics of sorption and the isotherms of adsorption.

With comparatively low concentrations of substance in solution c (i.e., provided that the concentration a is small in comparison with the maximum sorption capacity [33]) the linear isotherm of adsorption of Henry is valid ($a=Kc^*$), where c^* – is the concentration of the dissolved substance at the boundary of phase separation, K – is Henry's constant.

In this case the equation of kinetics of sorption may be written in one of the following forms:

$$\frac{\partial a}{\partial t}=\alpha^*\left(c-\frac{a}{K}\right),\ K\frac{\partial c^*}{\partial t}=\alpha^*(c-c^*), \quad (2.60)$$

where α^* – is the coefficient of mass exchange between the liquid and solid phases.

It follows from this that the homogeneous model of migration (2.57), given the existence of sorption, turns into the model with the blocked pores (2.59), so that in the final analysis it is identical with the heterogeneous model (2.58) given the following substitutions $n(1-\omega_1)\to n, n\omega_1\to K$, $\alpha\to\alpha^*$, $D\to D^*$ and $n\to n^*$ (where $n^*=n+K$ – is the effective porosity). The heterogeneous model of migration, given the presence of processes of sorption, remains valid, if the corresponding parameters are replaced ($n\to n^*$ and $\delta\to\delta+K^2v/\alpha^*(n^*)^2$).

It is customary also to consider that with comparatively low velocities of filtration the processes of sorption take place practically instantaneously [39]. Given such conditions both models (2.57) and (2.58) maintain their form with the substitution of active porosity n by effective n^*.

From what has been said above it follows that the scope of application of the heterogeneous model is much broader. It satisfactorily describes the process of migration in media with blocked pores and in the presence of processes of sorption.

At the same time, this model is multi-parametric, which certainly makes the organization of laboratory experiments and the methodology of interpreting experimental data more difficult.

Laboratory experiments for the study of migration in terrigenous rocks are carried out in a special column [2, 39], in which a specimen of undisturbed structure is placed. The experiments are carried out at a constant velocity of filtration for which purpose Mariotte's vessel is used.

Before the experiment is conducted, a specimen is saturated with a solution of a concentration of a particular component, equal to c_0. (In the process of saturation, so far as possible, the trapping of air in the specimen should be prevented). Then, a solution of the same concentration c_0 is filtered through the specimen. After attaining a stable discharge, set by the conditions of the conduct of the experiment, the solution under investigation is introduced into the column according to the particular pattern followed at concentration c_0. At the exit from the column, samples of filtrate are removed, which are subsequently subjected to chemical analyses. In order to rule out noticeable distortion of "instantaneous" values of the concentration at the exit from the column, the volume of test samples must be at least one order less than the actual volume of the specimen of the rock nSL (S and L – are the area of the cross-section and length of the specimen).

The tracer solution may be introduced into the column in a number of ways: 1) the pulsed method, with which, in the process of filtration of the initial solution

(without change in the velocity of filtration), the concentration for an insignificant time Δt varies from value c_0, and then once again is restored to its initial value; 2) the continuous method, with which, at a given moment of time, the concentration of the solution under investigation jumps to value c_0 and further remains unchanged to the end of the experiment; 3) the periodic method, under which, at the entry to the column, by means of special devices, a change in the concentration of the solution is provided according to the sinusoidal law (a modification of this method is the periodic introduction of pulses of solution of rectangular form).

With the pulsed method of introduction of the solution, the output curve may be described by the following expression:

$$\bar{c}=\frac{c-c_0}{c^0-c_0}=\frac{L\Delta t}{t}\sqrt{\frac{n}{\pi\tilde{D}t}}\exp\left[-\frac{(nL-vt)^2}{4\tilde{D}nt}\right]. \qquad (2.61)$$

The processing of the experimental data in this case may be conveniently carried out according to the value of time t_m, corresponding to the maximum concentration of the solution c_m. The value of this time depends substantially on the diffusion criterion of Peclet ($Pe = vL/\tilde{D}$).

If it is considered that with comparatively large velocities of filtration $\tilde{D} \approx \delta v$, then $Pe \approx L/\delta$. Inasmuch as the value of parameter δ is of the order of the linear size of the pores, then $L \gg \delta$ and $Pe \gg 1$. It follows from this that the time for the passage of the maximum amount of concentration in practice does not depend on the value of the coefficient of dispersion and is equal to $t_m \approx nL/v$. Then the maximum concentration is equal to

$$c_m=\frac{c_m-c_0}{c^0-c_0}\approx 0{,}5v\Delta t\,\frac{1}{\sqrt{\pi\tilde{D}nt_m}}. \qquad (2.62)$$

accordingly, the evaluation of the migration parameters n and δ (or $\tilde{D}$) may be made according to the formulae

$$n=\frac{vt_m}{L},\ \delta=0{,}25\,\frac{\Delta t^2 L}{c_m^2\pi t_m^2},\ \tilde{D}=0{,}25\,\frac{v\Delta t^2 L}{c_m^2\pi t_m^2} \qquad (2.63)$$

The solution of a similar problem for a heterogeneous model (2.58) (with $D = 0$) may be represented in the form

$$\bar{c}=\Delta t\sqrt{\frac{\lambda R}{t'}}\exp(-\lambda t'-R)\,I_1(2\sqrt{\lambda t' R}). \qquad (2.64)$$

$$\lambda=\frac{v\omega}{n\delta},\ t'=t-(1-\omega)\frac{nL}{v},\ R=\frac{\omega^2 L}{\delta},$$

where $I_1(z)$ – is the modified Bessel's function of the first kind of the first order, which, with large values of the argument, is approximated by the exponential function. Therefore, the expression (2.64) may be written for large values of L and t in the form

$$\bar{c}\approx 0{,}5\,\frac{\Delta t}{\sqrt{\pi t'}}\sqrt{\lambda t' R}\exp[-(\sqrt{\lambda t'}-\sqrt{R})^2]. \qquad (2.65)$$

With the condition $L \gg \delta/\omega^2$ the maximum concentration c_m, as in the case of the homogeneous model, takes place given $t_m \approx nL/v$. Inasmuch as the expression for the maximum concentration coincides at the same time with (2.62), for the determination of parameters n and δ the relations (2.63) are also used.

Therefore, the available experimental techniques and analytical methods do not make it possible to identify a single model which satisfactorily describes the process as a whole. In a number of cases, the identification may be made on a qualitative level by means of an interpretation of the nature of the output curves. For a homogeneous model relatively symmetrical curves are typical, while for the heterogeneous model, markedly asymmetrical curves are characteristic. The most typical characteristic of the heterogeneous model is a long and stable "tail" at the end of the output curve. The asymmetry of the output curves depends substantially on the value of parameter ω, increasing when $\omega \to 0$. With an increase of the length of the column, the differences in the nature of the output curves gradually disappear (this, however, does not indicate that the models are becoming identical). Therefore, the parameter of the heterogeneous model ω may be determined only in relatively short columns.

For a quantitative interpretative analysis after the determination of the parameters δ and n a calculation of the theoretical curve (formula 2.61) should be carried out and it should be compared with the experimental one. In case there is a noticeable difference between the curves, it is necessary to set oneself a series of values ω and carry out a calculation of the theoretical curves according to formula (2.64). These standard curves are plotted on tracing paper in the coordinates $\bar{c}$–t and are laid on the experimental output curve. The parameter ω is determined by best fit of the experimental data to the standard curve.

For the processing of the experimental data with continuous injection of the solution, the result of equation (2.57) is used, which had been obtained for a column of limited length [42]. This solution, at high values of the Peclet number assumes the following form [2, 29, 33, 39]:

$$\bar{c}\approx 0{,}5\,\mathrm{erfc}(\xi),\ \mathrm{erfc}(\xi)=1-\mathrm{erf}(\xi),\ \xi=\sqrt{\frac{Pe}{n\bar{t}}}(n-\bar{t}),\ \bar{t}=\frac{vt}{L}, \qquad (2.66)$$

where $\mathrm{erf}(\xi)$ – is the integral of probability, $\bar{t}$ – is dimensionless time, corresponding to the ratio of the volume of solution which has filtered through the column by the moment of time t, to the complete volume of the specimen in the column.

The processing of the experimental data according to (2.66) is carried out by the analytical – tabular method. For a relative excessive concentration $\bar{c}$ the value of the argument $\xi = \mathrm{inf\ erf}\,(1-2\bar{c})$ is derived from tables. Subsequently, a graph is constructed of the dependence $\sqrt{\bar{t}}\xi$ on t. If the process fits within the frame of the homogeneous model, this graph will have the appearance of a straight line (this is valid with large Pe), intersecting axis $\bar{t}$ at point $\bar{t}_0$. From the tangent of the angle of inclination of the graph towards the value of the segment $\bar{t}_0$ it is possible to evaluate the migration parameters

$$n=\bar{t}_0,\ \delta=\frac{0{,}25}{\bar{t}_0\,\mathrm{tg}^2\alpha},\ D=\frac{0{,}25VL}{\bar{t}_0\,\mathrm{tg}^2\alpha}. \qquad (2.67)$$

A more general method [55, 70] is based on the fact that an exact solution of the problem obtained in [42] is well approximated by the expression

$$\bar{c}\approx 0{,}5\,\mathrm{erfc}(\xi_1),\ \xi_1=-A\left[\ln\left(\frac{\bar{t}}{n}\right)+\frac{B}{A^2}\right], \qquad (2.68)$$

$$A\approx\frac{\sqrt{Pe}}{2}\exp\left(\frac{1}{4Pe}\right)\approx\frac{\sqrt{Pe}}{2},\ B\approx 0{,}25,$$

where A and B — are some constants, dependent on the Peclet number. The approximate expressions for A and B are valid with an error of not more than 3 % when $P_e > 20$ (the second expression for A is valid when $P_e > 50$). In works [33, 70] data are presented on A and B in a broader range of values of P_e.

The processing of experimental data according to (2.68) is carried out by constructing a graph $\xi_1(\lg \bar{t}) = \text{inf erf}\,(1-2\bar{c})$ on semi-logarithmic graph paper. From the tangent of the angle of inclination of the straight line there is obtained $A = 0.434 \operatorname{tg} \alpha$, the value of the Peclet criterion corresponding to it and parameter $\delta = L/P_e$. Furhter, segment $\xi_1^0 = -B/A \approx -1/4A$ is laid out on axis ξ_1, a straight line is drawn, parallel to axis $\lg \bar{t}$ until it intersects the graph. The second migration parameter $n = \bar{t}_0$ is determined from the value of $\bar{t}_0$ which corresponds to this point.

For the heterogeneous model (when D = 0) the solution for the continuous injection has the form [19, 34, 72]

$$\bar{c} = J(\lambda t', R) = 1 - \lambda \int_0^{t'} \exp(-R - \lambda t')\, I_0(2\sqrt{R\lambda t'})\, dt', \quad (2.69)$$

where $I_0(z)$ — is the modified Bessel's function of the first kind of zero order; $J(z_1, z_2)$ — is the special function, the tables of which are to be found, for example in [19, 29].

The processing of experimental data according to (2.69) is carried out by the method of standard curves. With large values of L ($R > 50$) the following asymptote of the solution is valid

$$\bar{c} \approx 0{,}5\,\text{erfc}(-\xi_2),\ \xi_2 = \sqrt{\lambda t'} - \sqrt{R} = \sqrt{\frac{\omega L}{\delta}}\left[\sqrt{\frac{\bar{t}}{n} + \omega - 1} - \sqrt{\omega}\right]. \quad (2.70)$$

Therefore, it is possible to use the analytical — tabular method consisting of constructing a graph $\xi_2(\sqrt{\bar{t}}) = \text{inf erf}\,(2\bar{c}-1)$. This graph intersects axis $\sqrt{\bar{t}}$ at a point which corresponds to value $\bar{t} = \bar{t}_0$. In the general case, this graph is not rectilinear, and therefore, having set a series of values $\bar{t}_1$, it should be reconstructed in the coordinates ξ_2 from $\sqrt{\bar{t} - \bar{t}_1}$ and its maximum correction should be obtained. After this, from the tangent of the angle of inclination and the values $\bar{t}_0$ and $\bar{t}_1$ there is carried out the calculation of the three migration parameters which characterize the heterogeneous model

$$n = \bar{t}_0,\ \omega = \frac{\bar{t}_0 - \bar{t}_1}{\bar{t}_0},\ \delta = \frac{(\bar{t}_0 - \bar{t}_1)L}{\bar{t}_0 \operatorname{tg}^2 \alpha}. \quad (2.71)$$

For the processing of the experimental data according to dependence (2.69) a more general method may be used which makes it possible to carry out simultaneously an analysis of the applicability of the models of migration [33]. Indeed, for a large range of the criterion R the following approximation is valid

$$\bar{c} \approx 0{,}5\,\text{erfc}(\xi_3),\ \xi_3 = -A_1\left\{\ln\left[0{,}5\left(\frac{\bar{t}'}{\omega n} + 1\right)\right] + \frac{B_1}{A^2}\right\}. \quad (2.72)$$

$$A_1 \approx \sqrt{R},\ B_1 \approx 0{,}25,\ \bar{t}' = \bar{t} - (1-\omega)n,$$

where A_1 and B_1 — are some constants, dependent only on the dimensionless criterion R. In work [33] graphs of dependences are given for $A_1(R)$ and $B_1(R)$ in the range $R = 1 \div 200$. The asymptotes given for A_1 and B_1 are valid with an error of not more than 3 % when $R > 50$.

The processing of the experimental data according to (2.72) is carried out by constructing graph $\xi_3(\lg \bar{t}) = \text{inf erf}\,(1-2\bar{c})$ on semi-logarithmic graph paper. This graph has the form of a straight line only when $\omega = 0.5$.

When $\omega > 0.5$, the initial and final points of the graph deviate to the left from the straight line, and when $\omega < 0.5$ — to the right. If the deviations of the graph from the straight line are considerable, this testifies to the inapplicability of the homogeneous model.

In such a case it is necessary to set a series of values $\bar{t}_1$, to reconstruct the graph in the coordinates ξ_3 from $\lg(\bar{t} + \bar{t}_1)$ and to obtain its maximum correction. The further sequence of actions coincides with that described above for the homogeneous model. From values R_1, $\bar{t}_1$ and $\bar{t}_0$ the three migration parameters are determined

$$n = \bar{t}_0;\ \omega = \frac{\bar{t}_0 + \bar{t}_1}{2\bar{t}_0},\ \delta = \frac{(\bar{t}_0 + \bar{t}_1)^2 L}{L\bar{t}_0^2 R}. \quad (2.73)$$

Using the periodic method of injection, the concentration of the solution varies according to the sinusoidal law with the circular frequency θ. The amplitude of the signal at input, at the same time, is equal to 0.5 $(c^0 - c_0)$. If the periodic injection is carried out by means of the successive introduction of pulses of a duration of Δt_1 with intervals equal to Δt_2, then these characteristics are respectively equal to $\theta = 1/\pi(\Delta t_1 + \Delta t_2)$ and $(c^0 - c_0)\Delta t_1/\Delta t_2$.

For the case when the process may be described by the homogeneous model (2.75), the concentration at the output from the column will vary according to the following law [21]

$$\bar{c} = 0{,}5 + E\sin(\theta t - \varphi),\ E = \frac{c_m - c_0}{c^0 - c_0} - 0{,}5 = \exp(-\varphi_0^2/P_e), \quad (2.74)$$

$$\varphi = \varphi_0[1 - 2(\varphi_0/P_e)^2],\ P_e = \frac{vL}{D} \approx \frac{L}{\delta},\ \varphi_0 = \frac{nL\theta}{v},$$

where E, φ, c_m — is the relative amplitude of variation of the concentration, the phase shift and the maximum concentration at the output from the column.

For the processing of the experimental data according to expression (2.74), the relative amplitude of the signal E and the phase shift φ are determined. From these values the unknown migration parameters are calculated

$$n = \frac{v\varphi_0}{\theta L},\ \delta = \sqrt{0{,}5(\varphi_0 - \varphi)}\,\frac{L}{\varphi_0},\ \tilde{D} = \sqrt{0{,}5(\varphi_0 - \varphi)}\,\frac{Lv}{\varphi_0}, \quad (2.75)$$

$$\varphi_0 = 0{,}5\left[\varphi + \sqrt{\varphi^2 + 8\ln^2(1/E)}\right].$$

The solution of the problem for the heterogeneous model also looks like (2.74); however, the relative amplitude and phase shift in this case (with D = 0) are equal to [21]

$$E_1 + \exp\left[-\frac{R\varphi_0^2}{RP_e + \varphi_0^2}\right],\ \varphi_1 = \varphi_0\left[1 - \sqrt{\frac{R}{P_e}}\,\frac{\varphi_0^2}{RP_e + \varphi_0^2}\right]. \quad (2.76)$$

It is natural that from the two values E_1 and φ_1 determined with one frequency θ, it is not possible to calculate the three migration parameters characterizing the heterogeneous model. In this context, experiments with a periodic introduction of the solution must be carried out with at least two values of the frequency θ_1 and θ_2. At the same time, the range of frequencies used must be carefully chosen.

Thus, for example, at small frequencies ($\theta \ll \omega v/n\delta$) the expression for phase shift acquires the form

$$\varphi_1 \approx \varphi_0[1 - \frac{\varphi_0^2}{\sqrt{P_e^3 R}}] = \varphi_0 [1 - \frac{\delta^2 \varphi_0^2}{L^2 \omega}], \qquad (2.77)$$

while the relative amplitude will be expressed by a second formula (2.74).

The expressions given for the amplitude and phase characteristics make it possible to carry out a comparison of the conditions of applicability of the homogeneous and heterogeneous models. Indeed, the estimated values of the coefficient of dispersion, determined from the characteristics of the heterogeneous model $\tilde{D}_a$ and $\tilde{D}_\phi$ prove different

$$\tilde{D}_a = \delta v, \tilde{D}_\phi = \frac{\delta v}{\sqrt{2\omega}} \qquad (2.78)$$

It follows from (2.78) that when $\omega < 0{,}5$, the "phase" coefficient of dispersion proves greater than the "amplitude" coefficient, and when $\omega > 0.5$ it is the contrary. Only when $\omega = 0.5$ do these values coincide (see methodology of processing experimental data in continuous injection). The experimental data given in [61], bear witness to the difference of the estimated values of "phase" and "amplitude" coefficients of dispersion.

For the evaluation of the three migration parameters of the heterogeneous model, it is necessary, with two values of frequencies θ_1 and θ_2 to determine the values of the relative amplitudes E_{11} and E_{12} and of phase shifts φ_{11} and φ_{12}. According to these data,

$$n = \frac{v}{L}\Psi, \delta = \frac{L}{2\Psi^2}\eta; \omega = \frac{\eta^2 \theta_1^3}{4\Psi(\Psi\theta_1 - \varphi_{11})}; \qquad (2.79)$$

$$\Psi = \frac{\varphi_{11}\theta_2^3 - \varphi_{12}\theta_1^3}{\theta_1\theta_2(\theta_1^2 - \theta_2^2)}, \eta = \frac{1}{\theta_1^2}\ln(\frac{1}{E_{11}}) + \frac{1}{\theta_2^2}\ln(\frac{1}{E_{12}}).$$

At large frequencies ($\theta \gg \omega v/n\delta$) the expressions (2.74) acquire the form

$$E_1 \approx \exp(-R), \varphi_1 \approx \varphi_0(1 - \sqrt{\frac{R}{P_e}}) = \varphi_0(1-\omega). \qquad (2.80)$$

Hence the expressions for "amplitude" and "phase" coefficients of dispersion have the form

$$\tilde{D}_a = \frac{\omega^2 v^3}{\delta n^2 \theta^2}, \tilde{D}_\phi = \sqrt{\frac{\omega}{2}}\frac{v^2}{n\theta}. \qquad (2.81)$$

The characteristics obtained show not only that the estimated coefficients of dispersion may substantially differ, but also that they depend on the frequency of the periodic signal. It follows from this that at large frequencies (i.e., given marked non-stationary nature of the process of migration) the homogeneous model proves inapplicable.

At large frequencies, according to (2.80), the three parameters of the heterogeneous model cannot be determined. In this case only the comprehensive, or integrated, migration parameters are evaluated

$$\frac{\delta}{\omega^2} = \frac{L}{\ln(1/E_1)}; n(1-\omega) = \frac{\varphi_1 v}{L\theta}. \qquad (2.82)$$

For the most complete study of the process it is necessary to carry out investigations in a broad range of frequencies. Having determined for all values of the frequencies the amplitude and phase characteristics $E_1(\theta)$ and $\varphi_1(\theta)$, it is possible to construct the graph of the dependence $\xi_4 = 1/\lg(1/E_1)$ on $1/\theta^2$. This graph has the form of a straight line intersecting axis ξ_4 at point ξ_4^0. According to the tangent of the angle of inclination of the straight line and the size of the segment ξ_4^0 the computation of the parameters is carried out

$$n = \frac{v}{L\sqrt{\xi_4^0 0{,}434 \operatorname{tg}\alpha}}, \frac{\delta}{\omega^2} = 0{,}434\,\xi_4^0 L. \qquad (2.83)$$

Further, according to the known value of parameter n the variable $\xi_5 = nL\theta/(nL\theta - v\varphi_1)$ is calculated, and the graph of dependence of ξ_5 on $1/\theta^2$ is constructed, which has the form of a straight line intersecting axis at point $\xi_5^0 = \omega$. To check the applicability of the heterogeneous model and the correctness of the determination of the migration parameters the estimated value of the tangent of the angle of the inclined straight line is used $\xi_5(1/\theta^2)$, which, as follows from (2.74), is equal to $\omega v^2/\delta^2 n^2$.

Thus, of the laboratory methods considered for determining migration parameters, the simplest is the pulsed method. However, this method proves to be the least informative from a point of view of analyzing the applicability of migration models. The investigation of migration using sinusoidal variation of the concentration at the input into the column, although giving rise to certain technical difficulties, makes it possible to study the laws of migration in the most detailed way, and to substantiate the applicability of migration models and to evaluate all migration parameters which characterize these models.

2.3.4. DETERMINATIONS OF MIGRATION PARAMETERS ACCORDING TO DATA OF WATER-REGIME OBSERVATIONS

The inevitable limitations of the study of migration parameters by experimental tests and laboratory experiments compel us to seek additional sources of information for the solution of this problem. It is obvious that most effective here is the use of the data of water-regime observations of the processes of migration during the construction and operation of an engineering installation.

The obvious advantage of such evaluations, as compared with EMW, is determined by a number of circumstances:

1) by the natural elimination or smoothing out of the influence of spatial-temporal scale effects, caused by the heterogeneity of the medium;

2) by the maximum approximation of the structure of the flow field to the forecasting conditions;

3) by the appearance in the data of the observations of man-induced factors which inevitably affect migration processes, but which are extremely difficult to take into account on the basis of results from standard hydrogeological tests (variability of permeable man-made lining deposits, density convection, various effects of the self-purification of groundwater, man-induced changes in the filtration properties of water-bearing strata, etc.).

The advantages noted, generally speaking, not only result in enhanced accuracy in the study of migration parameters, but also in the possibility of evaluating those of them which are not determined by experimental tests, and in particular the parameter of transverse dispersion. On the other hand, observations of migration processes often make it possible to obtain independent evaluations of the rate of flow of different points, which in this context also may be considered as an initial indicator of

migration. Finally, the long term migration of industrial wastes is often accompanied by the manifestation of processes of gravitational differentiation, as well as sorption, ionic exchange, etc., usually excluded from consideration and study in experimental tests.

We shall first carry out a further analysis for the case of a neutral addition of tracer and then for the case of real wastes.

Observations of tracer migration

To evaluate migration parameters from the data of water-regime observations we may use, in principle, the same calculating models which were analyzed above for EMW, while, for heterogeneous media, it is often admissible to consider the asymptotic approximations of these models, which are valid for long periods of time. The bi-dimensionality of the filtration field is also taken into account by the construction of flow lines and by the fragmentation of the flow into separate uni-dimensional elements. The development of the processes of transverse dispersion may noticeably complicate the calculation and interpretation of migration, due to the increase of its dimensionality. Thus, the profile of transverse dispersion often invalidates the premise of the layer-by-layer transfer for stratified thicknesses. Another important factor is planar transverse dispersion, leading to the propagation of contamination beyond the limits of the extreme trajectories of the movement of the liquid from the source of contamination. Its role in the system of interpretation is particularly important, if the inflow of water from the source of contamination is small in comparison with the discharge of the transit flow. To evaluate the parameter of transverse dispersion, as a first approximation, it is possible to use the known analytical solutions for a point or linear salt source. In more complex cases, mathematical modeling may be used.

Experience, however, shows that the interpretation of the idea of water-regime observations often proves incorrect. The cause of this is the complex bi-dimensional, and sometimes tri-dimensional nature of the flow field, which often requires an interpretation much more detailed than is customary. It is clear, for example, that the gradual expansion of the "flow lines", emerging from the industrial waste lagoon, finds its reflection in the increase of the transverse dimensions of the aureole of contamination, caused by convection. Similarly, due to an underestimation of the planar bi-dimensionality of the flow, the results of observations of the propagation of contamination may be erroneously linked with transverse planar dispersion, which, naturally, leads to an overstatement of the corresponding estimated parameter.

The effect of the bi-dimensionality of the flow may lead to even more serious complications which are difficult to interpret, even when it is possible to establish an averaged profile of the estimated rate of filtration in accordance with the known principles of the theory of planar flows. The problems of migration require a much more detailed profile differentiation of the velocity field of velocities, together with the consideration of the interstrata interchanges of substance as a result of transverse dispersion.

On the whole, the reliability of the determination of migration parameters on the basis of the solution of inverse problems of migration will prove extremely low, unless a sufficiently well-founded filtration pattern is constructed in advance. Meanwhile, data for such a detailed justification usually prove to be insufficient, because the traditional organization of geofiltration observations is not designed to provide them. Therefore, the need for an independent evaluation of the rate of filtration often arises and is carried out by special observations or experimental works. Resistance and thermal measurements in observation wells may prove most effective for this purpose.

Up to now it has been tacitly assumed that the observations themselves of the transfer of tracer in the stratum did not introduce any additional errors and, in particular, evaluated unerringly the actual concentration of tracer at a particular point in the stratum. However, the observation wells used in practice for these purposes are far from being an ideal measuring system. If we leave out of account the effect of hydrochemical inertia and additional changes in the chemical composition of the water in the well the most important point is the question of the correspondence of the calculation model adopted on the one hand, and the profile distribution of the filtering interval of the observation well, on the other. If such a correspondence is not observed, serious errors are occasionally possible.

Observations of the migration of wastes

The computation model, describing the migration of a waste component, must, generally speaking, take into account explicitly each type of mass exchange, attenuation and other processes determining additional changes in the concentration of the given component in the composition of groundwater. With such an approach, however, the number of parameters to be defined may increase markedly, which will make a reliable solution of the inverse problem unrealistic. Therefore, it is advisable to construct the model of migration according to the principle of rank, aiming it each time, as a rule, towards the determination of one unknown parameter. Ranking according to the significance of the process is carried out on the basis of data provided by laboratory experiments or preceding observations. At the same time, it is important that other migration parameters should be considered in such a model, which are known because of earlier independent determinations obtained by laboratory experiments, EMW, and observations of tracer or waste migration.

It must be expected that, with the approach described, each time only some fictitious ("generalized") parameters will be determined, formally reflected by the coefficients of differential equations (corresponding to the given rank of the model). In some cases this circumstance will have to be treated as an unavoidable shortcoming, while trying to eliminate it by drawing on additional material. In other cases, however, such generalized evaluations may be considered as a fully satisfactory end result, byt only if we can take as proved the absence of significant scale factors between the conditions of migration observed and those predictions based on the values of parameters derived from the evaluations.

The processes of gravitational differentiation caused by differences in the densities of groundwater and the wastes entering it deserve special mention. In the first place, they markedly complicate the profile distribution of the wastes, causing the formation of "tongues" invaded by wastes in the upper and lower parts of the water-bearing stratum; the length of such "tongues" often is commensurate with the length of the zone of transfer.

In addition, particularly in the case of heavy wastes, the density convection causes comparatively rapid vertical displacements of the wastes, which can intensify the transverse (crosswise to the stratification) convective dispersion transfer or, on the contrary, may weaken it, and sometimes fully "clog" it up. The intensity of these processes depends to a large extent on the anisotropy of the filtration profile.

Furthermore, unlike the traditional problem of the intrusion of ocean waters, such wastes enter groundwater systems from lagoons of limited surface extent, and this confers on the process of their propagation a strongly expressed tri-dimensional nature.

Added to the complexities of the analysis of the process above are the difficulties of eliminating errors of measurements in observation wells. In addition to the obvious errors connected with the location of the intake sector, serious errors of observation are possible due to the displacement of the pressure head and the processes of density convection in the well itself.

An important role in the determination of the spatial (especially areal) position of saline waters may be played by surface geophysical methods. However, the interpretation of the results of such observations requires refinement of the theoretical bases of these methods for the purpose of taking into account the presence of a transitional zone with varying electrical resistances, caused by hydrodispersion or non-uniform convection.

Everything said above forces us to direct the interpretation of the observations on the movement of liquids with different densities towards the simplest, extremely crude models: usually they must be bi-dimensional in nature, and the location of the "tongue" invaded by wastes is approximated for each lapse of time by elementary considerations of hydrostatics.

Thus, the approach usually practiced now, based on approximate estimates of parameters may, under a broad range of conditions, lead to results which are harmful rather than useful. Therefore, it is very important to make comprehensive use in the calculation model of all the information obtained by laboratory experiments, field experimental studies, and observations.

2.3.5. LABORATORY DETERMINATIONS OF THE PERMEABILITY OF CLAYEY ROCKS

The determination of the filtration properties of clayey rocks from the data of experimental hydrogeological investigations represents considerable difficulties and is not always possible.

A medium with a large specific surface area such as clay, is characterized by the frequent occurrence of intermolecular forces and surface phenomena. This is the reason for its extremely low permeability and the special state of water retained in the pores of clays, causing the occurrence of various flows anomalies (deviation from d'Arcy's linear law, the initial gradient of filtration), as well as the dependence of the permeability of clays on the physical-chemical and temperature conditions of the medium.

The results of numerous experimental investigations, carried out in the All-Union Scientific Research Institute of Hydrology and Engineering Geology, show that the permeability of clays increases with a growth in the mineralization of the filtering liquid and temperature, and decreases with a geowth in geostatic pressure. The scale of these changes depends on the minerological type of the clays and the ionic composition of the filtering solution. Thus, for example, in the filtration of chloride solutions the permeability of kaolin and montmorillonite clays may increase (as compared with fresh water) by 1.5–3 times tight up to one order, whereas with a growth in temperature from 20 to 80–90 °C, the coefficient of permeability increases from 3–5 times (kaolin clays) to 10–15 times and more (montmorillonite clays). Otherwise, investigations show that the permeability of clay rocks is not an absolute parameter – it merely reflects the situation in which the rock finds itself and the conditions for determining a given parameter must correspond to the conditions (natural or artificial) of this medium.

There can be no doubt that the highest quality information on permeability may be obtained only in full-scale field experiments, carried out on a rock mass while preserving natural conditions of occurrence. Laboratory experiments, however, are carried out on specimens of limited size and characterize permeability directly at the point of testing, ruling out the effect on the parameters of filtration of the factors of fracture distribution, lithological heterogeneity, etc. Laboratory parameters of permeability in individual cases are 1-2 orders lower when compared with full-scale field conditions, and this discrepancy forces us to assume that in field conditions filtration through separating clay strata is selective in nature and, takes place chiefly not through pores but through fractures and through lithological windows. The greatest convergence of the results of field and laboratory determinations occurs in the case of a lithologically homogeneous, monolithic clay stratum, constant in plan and section, excluding any effects on the results of the determination of micro- and macroheterogeneities. At the same time, the reliability of the laboratory determinations of permeability and the extent of their correspondence to full-scale conditions may be enhanced by means of a large number of analyses.

In practical hydrogeological work it is often necessary to determine the parameters of a dividing clay stratum because it is not always possible to carry out pumping or injection tests of the groundwater regime in adjacent aquifers and to set up the necessary special observation networks. In particular, this applies to deep aquifers, the exploration of which is done with a comparatively small number of wells. The determination of the permeability of the dividing layers often proves difficult also for the reason that because of the low permeability of the clays, significant changes in water levels may not be measurable even after test lasting tens or hundreds of days. Under these circumstances, information about filtration properties of the dividing layer may be obtained from the results of direct investigations of the permeability of core material.

Permeability, determined from cores, reflects the lower limit of the filtration properties of the clay stratum, and these data are useful, and in a whole series of cases even sufficient for the solution of practical problems. Thus, for example, if, in investigations concerning the justification of underground disposal of wastes, it turns out that the properties of the isolating layer, determined from the core, are insufficient for reliable isolation of the contaminating wastes, it is no longer necessary to determine the generalized filtration parameters of the water-resistant layer taking into account the presence of strong fracturedness, lithological windows, etc. Moreover, as has already been pointed out, the permeability of clays changes depending on the conditions of filtration (temperature, mineralization, etc.). Full-scale field investi-

gations in the given case do not always make it possible to recreate those hydrochemical and temperature conditions which may occur in the process of lengthy action on the stratum. With more or less approximation the general hydrodynamic picture may be reproduced on the basis of laboratory investigations and their integration with other known methods. Hence, determinations of permeability on the cores are valuable both because of the possibility of their direct use for the solution of applied problems, and for the solution of questions connected with the study of overflow and the forecasting of the behaviour of dividing layers during the filtration of a liquid in natural and disturbed conditions. The determination of permeability in laboratory conditions must be brought as close as possible to natural conditions and must be carried out on a specific type of liquid (a porous solution).

At the present time, a number of instruments exist intended for laboratory determinations of the permeability of cohesive soils. However, they suffer from a series of substantial defects, among which the absence of the possibility of simulating the conditions of natural occurrence of the rock should be considered as fundamental. This defect is not present in the UIPK-IM apparatus which makes it possible to study the permeability of clayey rocks in conditions which correspond to the conditions in the ground, by controlling the hydrostatic head, changes in head and by regulating and to maintaining the necessary temperature regime.

The UIPK-IM device is intended to determine absolute and phase permeability of rocks at a geostatic pressure of from 1–600 km/cm^2 and at temperatures from 20–80 °C. The operating regime of the UIPK-IM is determined taking into account the conditions of natural occurrence of the rock under investigation, as well as of the purposes and problems of the investigations. Developed by M.K. Kalinko [14], the methodology ofinvestigations using the UIPK-IM apparatus is applicable for the study of the filtration properties of rocks which act as collecting agents of oil and gas, characterized by relatively high permeability and considerable properties of toughness, by which they are markedly different from clays. This in turn places certain demands on the methodology of conducting experiments on clays. Let us consider the characteristics of the preparation and conduct of experiments on clay specimens.

The determination of permeability is carried out on specimens of a cylindrical form, 3 cm in diameter and 2–5 cm in height. High demands are placed on the specimens under investigation with respect to the cleanliness of the finish of the surface. The specimens are prepared from monoliths by means of a cutting ring or a special crown drill.

Conduct of the experiment

In designing the experiment, it is necessary in the first place to assign an appropriate pressure to the hydrocompression of the specimen. The determination of the value of this pressure is carried out taking into account the following premise.

In its natural state, the rock experiences the action of geostatic pressure, created by the weight of the overlying rocks. The pressure at depth h is determined by the product of the volumetric weight of the rocks by the thickness of the overlying strata. If the mean volumetric weight is taken as equal to 2.3 g/cm^3, the geostatic pressure P_{rc} will amount to: P_{rc} = 0.23 (kg/cm^2) (here the value of h in m). At the same time, the hydrostatic pressure also acts on the structure, preventing the compression of the rock under the effect of the geostatic load. The value of this pressure is determined as: P_r = = 0.1 h (kg/cm^2). Thus, the resulting compacting pressure may be presented in the form of the difference between these loads. At the same time, only part is taken as geostatic pressure (0.85 of this pressure; P = 0.23 h – 0.85 (0.1) h = 0.145 h (kg/cm^2). With the use of this equation the required pressure in the hydrocompression system of the specimen is determined. The methodology of conducting experiments to determine the permeability of clay rocks amounts to the following.

In setting the geostatic pressure, in the first place a small (up to 5–6 atm) axial compression is produced. The major share of this pressure is expended on the raising and weak thrust of the plunger against the upper box of the core clamp. Further, under conditions of absence of lateral compression at the input into the specimen, pressure is created (tenth parts of atmosphere) until the fluid appears at the outlet in the measuring capillary. After the rinsing out, the necessary all-round compression, drop in pressures and temperature are produced. At the same time, special attention is paid to the uniformity of the lateral and axial loads. As soon as the minimal pressure (5–6 atm is applied) the system of lateral compression is linked to the system of axial pressure. This ensures the mechanical integrity of the specimen. For the creation of pressures in the range from 0 to 15 atm compressed nitrogen is used, delivered directly from a gas cylinder, while at large values of P a system of a manual presses is switched on.

In conducting experiments on clays and clayey rocks it is necessary to bear in mind the effect of such factors as the rheology of the filtration properties of the clays and the partial compacting of the specimens, caused by the decompacting of the rock at the time of recovery and extraction of the core from the well. The experience of these investigations shows that the most marked change in the permeability of clays is observed in the course of the first 2–3 days, when the absolute values of the coefficient of permeability in individual cases decreased as compared with the original values by 30–50 %. Subsequently, these changes are less important and, as a rule, the permeability of clays, determined at 5 and 10 days, differs by not more than 5 %. Thus, sufficiently accurate information on permeability may be obtained only after a preliminary ageing of the specimen during a period of not less than 5 days.

To cleate the temperature regime on the UIPK-IM apparatus provision is made for thermostatically controlled heating. Practice in operating the instrallation shows a number of shortcomings in using this method, including: considerable inertia of the contact thermometer on the thermostat, which causes fluctuations in temperature at the set value within the limits of 3–5 °C, the long time required to stabilize the temperature and its restricted upper limit (to 80 °C). In order to eliminate these shortcomings, a method of electrical heating of the specimen has been proposed, where an electrical heating element is installed in the core clamp. Its heating temperature is set by an auto-transformer with stabilized power supply from the mains.

As was pointed out, the UIPK-IM apparatus makes it possible to determine permeability and to carry out experiments under conditions of variable temperatures,

pressures, changing mineralizations, as well as under conditions of joint action of all these factors. Such investigations are necessary for the solution of a broad range of questions involving the study of the laws of change of the filtration properties of clays under the influence of natural and man-made factors. The conduct of these experiments is characterized by a number of features, on some of which we shall dwell briefly.

In carrying out investigation of the effect of mineralization and of the composition of the fluid on the permeability of clays the main difficulties involve the long time required to replace one type of solution by another, which is caused by the low permeability of clayey rocks. Thus, an extremely rough calculation shows the following. Let us use for the experiment a specimen characterized by a permeability of $K_п = 10^{-3}$ m D'Arcy (the order of values $K_п$ for kaolin clays), the height of the specimen is 4 cm, the area of the cross-section – 7.07 cm^2. Let us take porosity (effective) as conditionally equal to 0.05. Taking this into account, the total volume of effective porosity with the given geometric dimensions of the specimen will amount to ~1.5 cm^3, which may be taken as the volume of fluid directly participating in filtration. This volume, even at gradients of 1000 and more, is replaced after about 10 days.

To speed up the rinsing of the core-holder and the specimen, the head of the fluid at the inlet is temporarily increased at constant pressure of the hydrocompression, or the hydrocompression is decreased at constant head. At the time of establishment of the hydrocompression pressure, equal to the pressure of the fluid, filtration occurs at the walls and in a short time all the water-supplying channels and cavities of the core-holder are filled with new fluid. After rinsing, the experiment returns to the initial regime of a drop in head and pressure and changes in filtration discharges are observed. Under these conditions, as practice shows, the discharge is stabilized 4–5 days after the injection of the new solution. At the same time, the permeability of clays may increase as compared with the initial permeability if a more mineralized solution is injected or may diminish if a less mineralized solution is injected.

Investigation of the effect of pressure heads on the permeability of clays. The conduct of the present experimental investigations involves the study of the phenomenon of the initial filtration gradient and the disruption of the filtration regime in the zone of low pressure heads. The technical implementation of these experiments, the conditions and methodology of conducting then are distinguished by a considerable degree of complexity and require special "care" in the experiment. In the first place, this applies to the study of the initial filtration gradient – a gradient at which no visible filtration of the fluid in the clays is observed.

Inasmuch as it is a question of visible filtration, the technical facilities must provide maximum resolving power for the recording of filtration discharges. In conducting these experiments, it is necessary to exclude any occurrence of wall filtration and the effect of the process of recompacting of the specimen, i.e., the specimen must provisionally be kept until full stabilization. Fluctuations of temperature are permitted within the limits of not more than ±0.5 °C. The experimental methodology is as follows:

The specimen prepared for testing is placed in the core-holder and the necessary hydrocompression pressure and head is created. In order to speed up the saturation of the specimen at the initial moment in time, maximum pressure head is established. The process of saturation is accompanied by the stabilization (recompacting) of the specimen. The completion of the recompacting is recorded, either by the stabilization of the filtration discharge at constant gradient of head or by the complete absence of movement of the fluid (squeezing out) at a gradient equal to 0, and by the termination of pressure drop in the hydrocompression system of the specimen, which is recorded by the manometer of the specimen. The time of stabilization generally amounts to not less than 15–20 days and depends on the type of clay under investigation, the depth of its recovery and the pressure of the hydrocompression of the specimen. In the course of the experiment, a phased reduction of head is carried out from maximum values to gradients at which filtration is not observed. In the field of large pressure heads, the time of recording of the discharge at each phase is limited to 1–2 days until 5–7 values of the coefficient of permeability are obtained, which differ by not more than 10 %. At comparatively low gradients (~50–100), this time increases considerably. Thus, for a clay characterized by a permeability of $K_п = 10^{-4}$ m D'Arcy, with a gradient of 1 % and a resolving power of the measuring capillary of 0.0001 cm^3, from the data of daily observations only one value (only one measurement) of the coefficient of permeability may be obtained. As investigations show, the permeability of kaolin and montmorillonite clays at gradients of less than 100–200 falls markedly, and this in turn considerably increases the time required for conducting the experiment. For clays with a permeability of 10^{-4}–10^{-5} m D'Arcy, taking account of the accuracy indicated above for carrying out the measurements (0.0001 cm^3), such values of gradients may be taken initially at which filtration is not observed in the course of 20–30 days. In this case, without excluding the possibility of filtration, permeability amounts to not more than 10^{-6}–10^{-7} m D'Arcy, which is 2 orders lower than the initial value of $K_п$.

Investigation of the effect of temperatures and pressures on the permeability of clays. The methodology of conducting experiments for the study of the effect of temperature on the permeability of clays requires no special explanation: in the course of the experiments, a phased increase of temperature from room temperature to the established upper limit is carried out. It is important only to pay attention to the fact that at each temperature phase a full stabilization of the filtration discharge should occur. As practice shows, constant discharges at 15° phases of temperature change are established after 2.5–3 hours. From the time of stabilization of the temperature, fluctuations are admitted within the limits of ±0.5–1 °C.

The temperature is established and regulated by means of a system of electric heating, and monitoring of temperature changes is carried out by means of a thermometer which is placed in the upper plunger of the core clamp. The real values of the temperatures, however, taken into account for the determination of the viscosity of the fluid, are taken from a calibrated equation according to the known value of the capacity required.

In experiments for the study of the effect of pressure on the permeability of clays, phased changes in the pressure of the hydrocompression of the specimen are carried out. At the same time, at each phase the specimen is kept during not less than 2–3 days, until constant filtration discharge has been achieved.

Processing of the results of the experiment

For the calculation of the coefficient of permeability the following equation is used

$$K_п = \frac{1000 \cdot Q \cdot h \cdot \mu}{F \cdot P}, \quad (2.84)$$

where $K_п$ — is the coefficient of permeability, m D'Arcy; h — is the height of the specimen, cm; Q — is the discharge of the fluid, cm^3/s; F — is the area of the cross-section of the specimen, cm^2; P — is the pressure of the fluid at the entry to the specimen, kg/cm^2; μ — is the dynamic viscosity, centipoise.

In the course of the conduct of the experiments the discharge is recorded. The height of the specimen and the area of the cross-section are constant, the pressure of the fluid at the entry is set, on the basis of the conditions of the experiment. The computation equation (2.84) includes the parameter of dynamic viscosity, the value of which depends mainly on temperature, mineralization and composition of the fluid, while the effect of pressure may be neglected. According to the data by M.P. Vukalovich and other investigators, the value of μ shows very little change over the pressure range 0 to 5000 atm and in the temperature range 0–60 °C.

The values of dynamic viscosity of pure water for various temperatures are presented in many reference works. The difficulty of determining the parameter μ occurs when, in the course of the experiment, not only the temperature varies, but also the mineralization. The determination of viscosity under these circumstances may be carried out according to the empirical equation of L.L. Ezrokhi [40a]

$$\lg \Psi_{\Sigma} = (A_i c_i + D_i c_i c^{n}). \quad (2.85)$$

where Ψ_{Σ} — is the relative viscosity of a complex solution; c_i — is the concentration of the given salt in the solution, g — equivalent/l; A and D — are the empirical coefficients, dependent on the nature of the electrolyte and the temperature; c — is the total concentration of all salts in the solution, g — equivalent/l, with n = 2 for $MgCl_2$ and $CaCl_2$ and n = 1 for K_2SO_4 KCl, Na_2SO_4, NaCl, $MgSO_4$.

The values of coefficients A and D for the basic binary systems, determined experimentally, are presented in work [24a].

The dynamic viscosity is calculated according to the relative viscosity:

$$\mu = \Psi \cdot \mu_B, \quad (2.86)$$

where μ_B — is the viscosity of pure water at the same temperature and determined by reference data [1].

Control investigations show that the error in the calculation of dynamic viscosity by the above method usually does not exceed 2 % as compared with the determination of viscosity by a capillary viscosimeter [24a]. Estimates calculated by V.P. Morozov also show that, with sufficient accuracy for practical purposes, determinations of the value of viscosity of a multicomponent solution may be reduced to a computation according to prevailing ions. Thus, for stratal waters with a total mineralization of c = 2.233 g-equivalent/l with a content of NaCl c = 1.770 g-equivalent/l, the value calculated taking into account all components of the saline composition, differs from the value μ, calculated only according to NaCl (the concentration of which is conventionally reduced to c = 2.233 g-equivalent/l) by not more than 3 %. This fact should be borne in mind in carrying out and processing experimental investigations. It makes it unnecessary to engage in the complex process of preparing a solution, the composition of which would correspond to the saline composition of stratal waters, and makes it possible to use solutions of simple ionic composition in experimental investigations. Formula (2.85) in this case is reduced to a simpler equation:

$$\lg \Psi = A \cdot c + D \cdot c^{n+1}, \quad (2.87)$$

where c — is the concentration of the solution of the salt selected, g-equivalent/l.

2.4. INVESTIGATIONS OF THE ZONE OF AERATION OR UNSATURATED ZONE

The interchange between groundwater and the atmosphere (and, partly, with the hydrosphere) through the zone of aeration largely determines the conditions of the protection of groundwater against depletion, contamination and salinization. Waters are formed and maintained at a particular quantity and quality (fresh, brackish, salt, etc.), depending on the depth and lithology of the zone of aeration and the local topographical and climatic conditions. In evaluating the conditions of contamination of groundwater from the earth's surface, the transport of contaminants by the moisture flux (convective transfer) also plays, a role of prime importance.

The study of the water regime and of moisture transfer in the zone of aeration makes it possible to describe the way in which groundwater interacts with the environment, taking account of the effects of geotechnical and hydrogeological engineering works.

At the present time, the laws of equilibrium and combined movement of pellicular and capillary (meniscus) moisture have been investigated under a broad range of thermodynamic conditions, which makes it possible to evaluate and predict a scientifically the evaluation and the water flux through the zone of aeration under any hydrogeological condition. The evaluations made show that the real recharge of moisture through the zone of aeration amounts to a layer of water of from 0–25 mm annually in arid and semi-arid conditions, to 100–200 mm/year in humid conditions, and up to several thou. mm annually at the sites of artificial recharge of groundwater. In some cases recharge through the zone of aeration is practically non-existent or the rising flow of moisture occurs through the zone of aeration (the latter two cases are typical of arid regions and, to a certain extent, of regions of seasonal freezing of the upper layers of the zone of aeration). It is obvious that the conditions of protecting groundwater against depletion, contamination and salinization, in the cases mentioned, will be substantially different.

In order to evaluate the flows and velocities of moisture transfer in the zone of aeration, on the basis of the joint consideration of the equilibrium and movement of pellicular and capillary moisture, a special version of the thermodynamic method has been developed and widely tested in the USSR: the method of moisture graphs.

On the basis of the theory developed and the data bank obtained about the hydraulic parameters of the soil, the following problems are solved:

1) classification and zoning of an area according to the conditions of replenishment, consumption and contamination of groundwater through the zone of aeration;

2) estimation of the natural and modified recharge of groundwater through the zone of aeration at individual sites (points) of the locality (including those involved in artificial replenishment of groundwater by means of infiltration lagoons);

3) evaluation of the consumption (depletion) of groundwater given physical evaporation and the water-absorbing capacity of plants (i.e., evapotranspiration as a whole);

4) extrapolation of areal data and the construction of maps of the types and values of moisture flux through the zone of aeration;

5) forecasts of the changes in the hydrogeological, geotechnical, hydrological and ecological situation given a change in the level of groundwater;

6) evaluation of the protective properties of the zone of aeration at individual points of the locality, in rock masses and at the regional level.

At the present time it is customary to distinguish between three aspects of the protection of groundwater 1) against depletion, 2) against contamination and 3) against salinization. At the same time, all these processes may take place either naturally (i.e., their regime is not modified), or under the action of man (modified by human activity). On the other hand, approaching the evaluation of the natural processes from a geological view point, we distinguish, between endogenous and exogenous processes. From the point of view of groundwater protection, the exogenous processes play a role of prime importance. It is also obvious that the majority of man-made and exogenous processes exert their influence on unconfined and confined groundwaters via the zone of aeration. This zone acts as a kind of buffer, weakening and modifying the actions of the atmosphere, the hydrosphere, the biosphere and man-made factors on the groundwater.

In other words, all abiotic processes taking place in the zone of aeration may be considered as mass energy exchange processes, for example, to the transfer and exchange of water (moisture), dissolved salts, heat, electrical charges, vapour, transfer, etc. The most important of these processes, hydrogeologically, is moisture transfer. The direction and strength of this process is principally determined by the processes of accumulation and consumption of groundwater. In most cases, the protection of groundwater against contamination from the surface, as well as its protection against salinization, depends on the process of moisture transfer. It is apparent that, in solving problems of a hydrogeological nature (unlike, for example, soil science and reclamation problems), a decisive role in the transfer of salts is played by convective transfer (hence the importance of the study of moisture transfer).

The period of operation of a groundwater resource is usually taken as between 20 and 30 years or more. Under these conditions, the sorptive capacity of the rocks may not be efficient to prevent contamination, and the process of penetration of a contaminant through the zone of aeration is determined by the velocity of convective transfer. In the case of non-sorbed components, convective transfer will obviously play a role of prime importance.

In the present section we consider mainly the question of the moisture flux in the zone of aeration, assigning it an exclusive role in the problem of groundwater protection.

In conclusion, it should be pointed out that the reverse effect of groundwater on the environment may also occur through the zone of aeration. Changes in groundwater levels induced as a result of geotechnical work may modify both the quantity and quality of groundwater discharges in surrounding areas.

Thus, the zone of aeration is an important regulating link, interconnecting the lithosphere, hydrosphere, atmosphere and biosphere, both in its natural and anthropogenically modified states.

In studying the process of water flux through the zone of aeration, two basic cases are distinguished:

– water flux in the upper layers (from 3 to 5 m deep);

– water exchange in the middle and deep layers of the zone of aeration from 5 to 30 m and more).

In the first case the moisture transfer regime is generally unsteady and because of this certain difficulties arise when studying this layer. The mechanism and characteristics of moisture flux are relatively well studied, and this zone has been successfully investigated for a long time by soil scientists, hydrologists, climatologists, reclamationists and hydrogeologists [27, 32].

The second case is apparently simpler, inasmuch as the moisture flux regime is close to being steady-state. However in studying the thick zones of aeration new difficulties appear, arising from the fact that the transfer of moisture occurs extremely slowly and mainly as pellicular water [30, 37]. It is not surprising that up to now hydrologists have not been in a position to evaluate directly this important component of the water balance, as outflow into groundwater. For a long time the laws of equilibrium and of displacement of pellicular moisture have remained unclear, as well as the interrelationship between pellicular moisture and capillary (meniscus) moisture [27, 37]. Many unresolved problems remain in estimating the gravitational movement of moisture in clayey and loamy layers of the zone of aeration, with similar difficulties relating to estimating evapotranspirative losses from the zone.

For the solution of all these issues a special modification of the thermodynamic method – the method of moisture graphs has been developed. Using this method, it is possible to determine unambiguously the direction and strength of moisture flux through the zone of aeration, based on information about the moisture field of the site of the locality under investigation, and having available a set of moisture graphs for the various types of soils. At the same time, pellicular and capillary moisture are considered in a single hydrodynamic system.

The essence of the moisture graph method derives from the fact that from natural curves of moisture content the value and direction of the flow of moisture passing through the zone of aeration may be determined unambiguously. For this purpose, the part of the natural moisture curve is laid out as a graph. Then the number placed over the combined graph curve and the natural moisture curve will indicate the direction and quantity of groundwater recharge. The curve with the number "0" (Fig. 2.6) corresponds to the case when the movement of liquid moisture through the zone of aeration is excluded; all curves to the right of this curve (equilibrium) will indicate positive groundwater recharge, to the left, the expenditure of groundwater on physical evaporation of water absorption by plants. It is obvious that, for the

use of the graph method, it makes no difference whether the moisture flow is brought about by natural causes (precipitation) or whether it is created artificially (for example, by irrigation).

Fig. 2.6 shows, as an example, the case of an estimate of the atmospheric recharge of one of the large freshwater lenses of the Karakum desert. It is easy to see that the quantity of this recharge on the average amounts to a water layer of 7 mm annually. Incomplete coincidence of the curves is due to spatial variations of the granulometric composition of the sandy soils. With a steady regime estimation of the quantity of water flux through the ground, requires only information on the amount of moisture of one particular aquifer of the zone of aeration and the height of this aquifer above the water table. From the appearance of the moisture graph (Figs. 2.6, 2.7) it follows that for the case of positive recharge of groundwater through a thick unsaturated zone, the quantity of recharge does not depend on the thickness of the zone of aeration and may be computed, without using the graphs, from a single curve according to the equation of the coefficient of moisture conductivity as a function of moisture K = f/w/.

In estimating quantities of unsteady moisture flows through the zone of aeration, two methods are used. The point is that, on any curve of changes in the moisture of the ground in time or with depth, corresponding to one cycle of accumulation and drainage of moisture (for example, annually), there are extreme points (MAX. and MIN.). These points may be plotted on a graph, and, according to which graph curve they coincide with, for a specified height above the water table, it is possible to judge the minimal, maximal and the mean quantities of groundwater recharge during the period under review. In the first case, the annual fluctuations in moisture content in the zone of aeration of particular aquifer is analyzed, providing an estimate of the maximum quantity of groundwater recharge. In the second case, moisture distribution may be analyzed by depth for one particular period of time and from the maximum quantity of moisture in the curve, the maximum possible atmospheric recharge may be judged. It is obvious, that in both cases, the deeper the layers of soil analyzed, the smaller the difference between the extreme values of recharge and the more the estimate of its mean quantity will approach the true value. Some of the implications stemming from the moisture graph theory may be considered:

1) As may be seen from the shape of the curves, the traditional idea of the capillary border is conditional, inasmuch as, (1) a number of curves go upwards theoretically into infinity and, (2), the appearance of the curve depends, for the given type of soil, on the direction and quantity of water flux through the zone of aeration, so that an infinite number of curves corresponding to the concept of a capillary border exist.

2) The potential for gravitational (descending) movement is determined mainly not by the mobility of moisture, but by the height of the aquifer (cross-section) above the water table and by the volumetric moisture content of the ground in relation to the moisture on the equilibrium curve for the same cross-section. Accordingly, pellicular water may move gravitationally even at extremely low moisture contents, which was previously considered impossible.

3) In the case of vertical drainage, moisture curves at sufficient heights above the water table have the appearance of vertical segments of straight lines, and therefore,

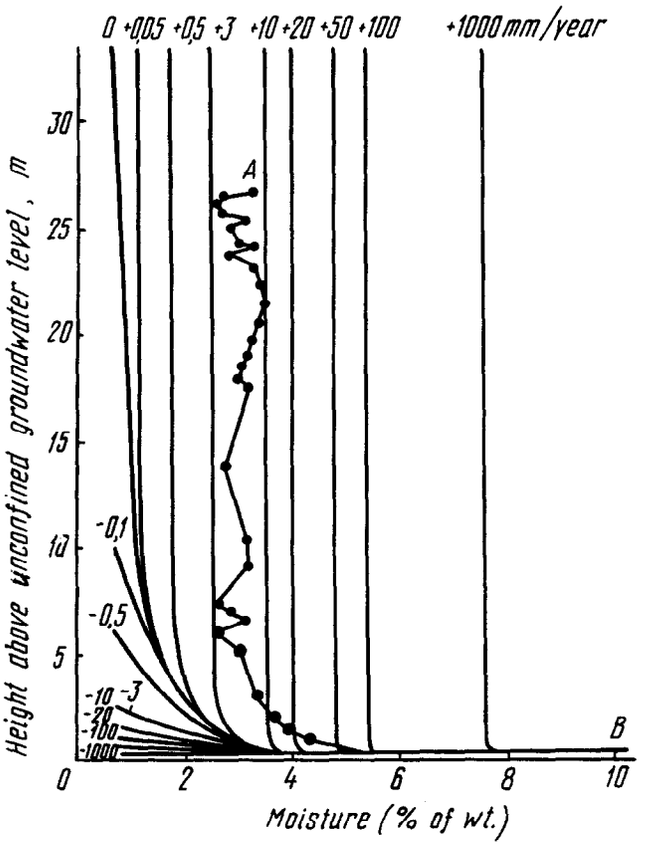


Fig. 2.6. Moisture graph, sandy soil, coefficient of filtration 3 m/day.
AB — The natural moisture curve for the sand dune sector of the Karakum Desert.

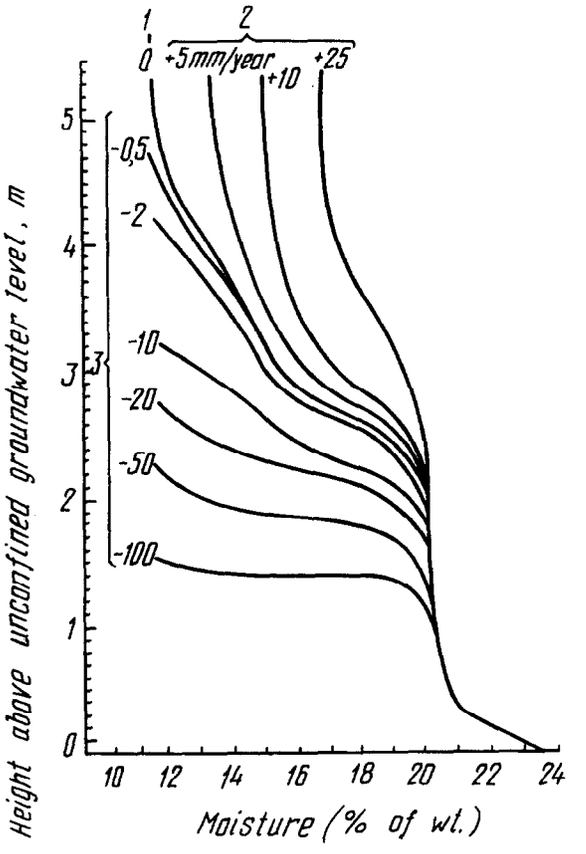


Fig. 2.7. Moisture graph, medium loam, coefficient of filtration 0.1 m/day.

for the evaluation of the descending flows, it is possible to simplify considerably the estimation by plotting on the graph only one value of moisture, for a particular level in the zone of aeration. In order to be able to do this, there must be certainty that a steady state, or quasi-steady state, regime exists and that the level of groundwater is located sufficiently deeply below the test horizon. For an estimate of the descending movement it is also enough to know the height of the point and the moisture value, but for this it is necessary to have the moisture graph.

4) With an increase in the thickness of the zone of aeration, the quantity of the maximum possible ascending moisture flux falls markedly. Thus, a sufficiently thick zone of aeration safeguards groundwater against possible depletion through physical evaporation or through the water supply of the root system of plants. Given a steady descending flow, the thickness of the zone of aeration is obviously not important.

5) To evaluate the direction of the movement of a steady moisture flow, it is clear that it is sufficient to know only the shape of the equilibrium curve, the height of the test horizon and the quantity of moisture. Points to the right of the equilibrium curve indicate the descending movement and those to the left ascending movement.

Fig. 2.8 shows the coefficient of moisture conductivity – moisture content relations for various types of soils (including clayey soils). Using these curves, it is possible to evaluate the infiltration under varied hydrogeological conditions. Special attention should be paid to the low values of infiltration recharge (the coefficient of moisture conductivity), corresponding to pellicular moisture transfer. For example, in order to obtain one of the curves given (for sands), the organization of a uniquely prolonged experiment was necessary, in which the physical modelling of moisture flow under the action of the forces of gravity was continued for 15 years, carried out on an extremely high column (10 m).

To obtain other curves, experiments were organized for the physical modelling of the process of moisture transfer on a centrifuge. Centrifuge modelling makes it most completely possible to simulate the action of gravitational volumetric forces, operating in the real zone of aeration in the process of atmospheric recharge of groundwater. The graphs show that gravitational drainage transfer of moisture is also possible in loamy and clayey rocks. The modelling processes employed allowed the same pressures on the rock as occur at testing depths in the zone of aeration of between 10 and 30 m to be simulated, making it possible to observe in the fullest extent all the thermodynamic conditions expected in the real zone of aeration.

To evaluate the discharge of moisture through the zone of aeration as a whole, or through its individual layers, we need either a set of moisture graphs or a set of the curves of the coefficient of moisture conductivity – moisture content relationships. The data necessary for these calculations may be obtained in various ways: (1) by the long experimental columns, (2) by the centrifuging of soil specimens, (3) by the analysis of the natural and modified moisture field, and (4) by other methods.

As has been pointed out, the moisture transfer regime is close to steady state in thick zones of aeration. However, a slow process of unsteady moisture discharge under gravitational forces may occur over many years. It is also convenient to model this process, on extremely long columns, and by centrifuging soil specimens. In order to predict these effects, it is advisable to use generalized parameters, without calculation the coefficient of water-conductivity and the capillary-pellicular potentials. In this case, the results of modelling are represented in the form of graphs of the function of discharge of draining moisture in time in a double logarithmic scale (Fig. 2.9). The graphs have the appearance of straight lines, with different gradients for various soil types. With a set of graphs for various soil types it is possible to determine the discharge of run-off moisture at any point in time, i.e., to forecast the process of groundwater recharge through the zone of aeration (even at very slow processes of pellicular moisture flow).

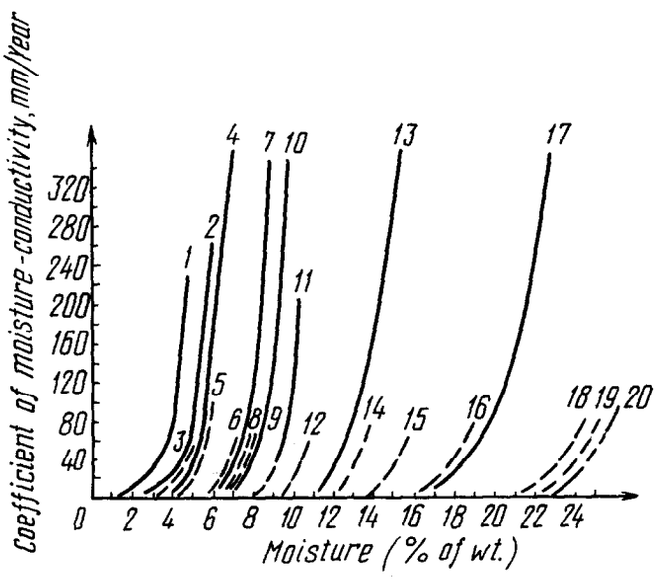


Fig. 2.8. Dependence of the coefficient of moisture conductivity on moisture content for various types of soils:
1–9 – sands; 10–14 – sandy loams; 15–20 – loams;
the dotted curves are drawn on the basis of an extrapolation of the data.

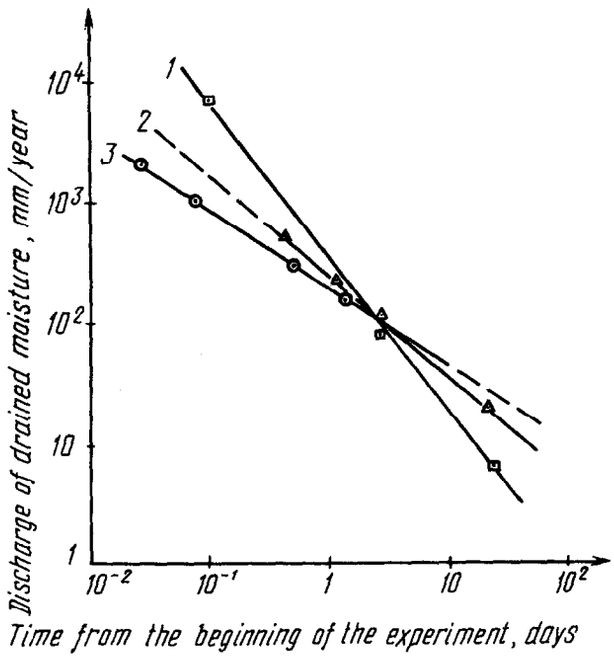


Fig. 2.9. The dependence of the discharge of moisture drained during centrifuging in time (the data are given).
1 – medium grained sand; 2 – sandy loam; 3 – loam.
(height of the specimen 6 cm)

When the thickness of the zone of aeration is small and a markedly unsteady regime of moisture transfer takes place, it is necessary to use the thermodynamic method in its most complete form. For this purpose, special moisture flow meters are used. The results of such a remote-sensing apparatus, set up beneath the bottom of an infiltration basin with artificial replenishment of groundwater, are shown on Fig. 2.10. The graph illustrates how quickly the clogging of the bottom of the experimental basin took place, working in this present case as a settling tank. It may also be seen from the graph how rapidly the infiltration increased with a partial removal of the clogging layer.

Thus, for this zones of aeration, it is recommended that observations of the process of moisture transfer be made by means of remote-sensing instruments.

Calculations made using the thermodynamic method have shown that in the desert regions of the USSR infiltration varies from 0 to 7 mm of water per year, in semi-desert regions it amounts to 10–25 mm and in humid zones from 5 to 200 mm per year.

Quantitative determination of the natural or modified recharge through the zone of aeration, also estimates the protection of groundwater resource against depletion.

To evaluate the protective properties of the zone of aeration (against contamination) it is also necessary, to evaluate the infiltration recharge, on the basis of the methods set forth above.

However, in this case it is advisable to calculate the maximum possible rates of moisture flux in the largest pores of the soil, using the equations shown in Fig. 2.8. In this case the maximum velocities may be determined as the tangents to the curves shown on Fig. 2.8. In this case, both moisture and the discharge of moisture through the zone of aeration may be considered as independent variables. The types of curves shown indicate that maximum velocities are only slightly influenced by the nature of the soil. Thus, from the point of view of convective transfer, a zone of aeration of sandy-clayey or clayey deposits does not possess any special protective properties. This conclusion, of course, is dinstict from the case when processes of sorption are predominant.

Questions of the protection of groundwater should be considered at various hierarchical levels. For example, the process of contamination of groundwater may be studied at the molecular level, in terms of the mineralogical and petrographic properties of a single rock layer, at individual sites, within irrigation land tracts, within major aquifer units, and at regional and global levels.

For the extrapolation of the data obtained from individual points of measurements using moisture graphs, a special genetic classification of types of water flux through the zone of aeration has been developed (Table 2.9). For the use of the moisture graph method, the most convenient are zones II (middle) and less often III (lower), where the moisture transfer regime is

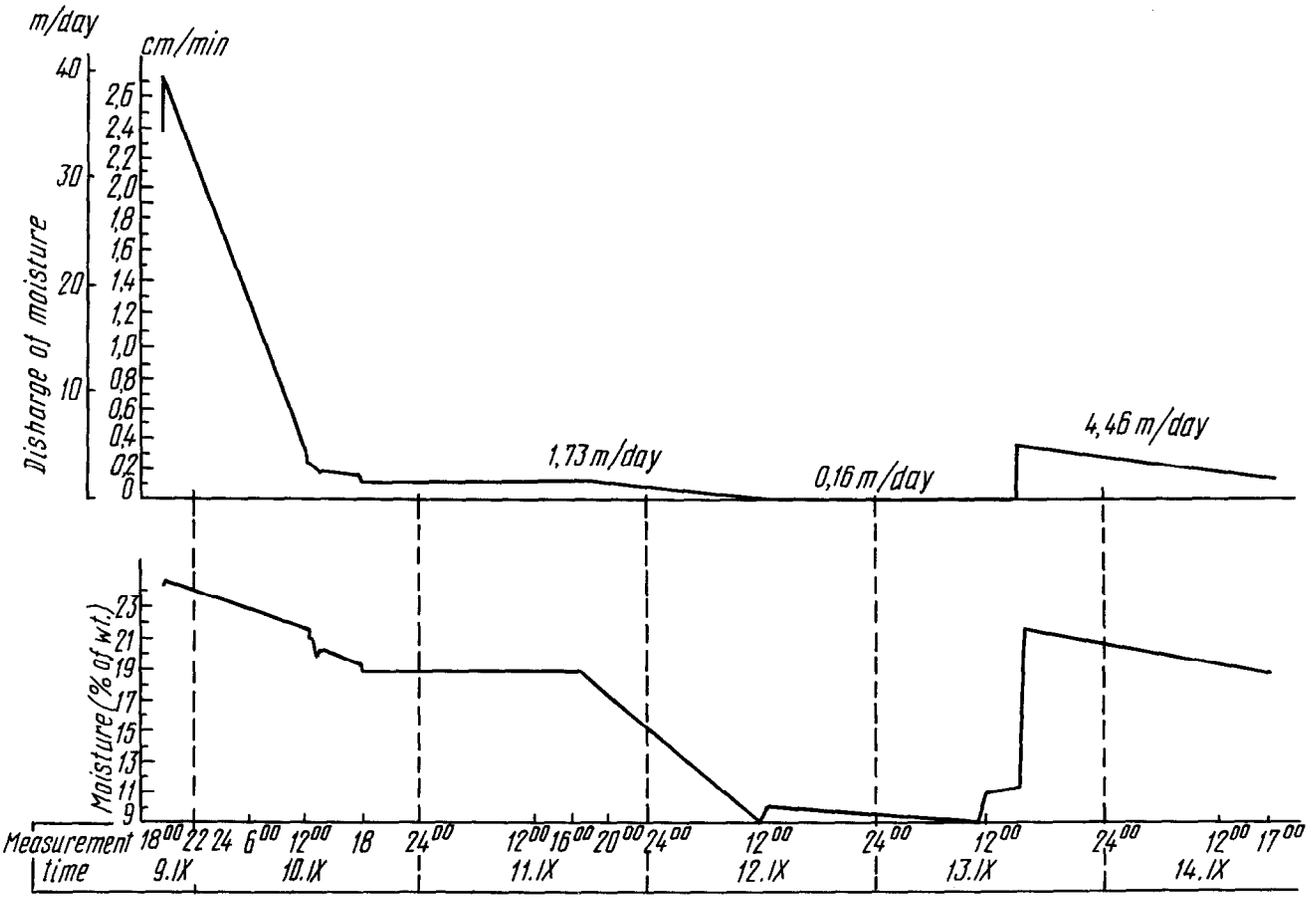


Fig. 2.10. Graphs of the change in time of moisture content and the discharge of moisture in the zone of aeration beneath an artificial groundwater recharge basin.

Table 2.8

TYPIFICATION OF WATER-BEARING FORMATIONS ACCORDING TO THE CONDITIONS FOR CONDUCTING EXPERIMENTAL MIGRATION STUDIES

Rock formation	Diagram designation	Structural and lithological description of the water-bearing formation within the testing zone	Correlations between parameters determining the specifics of mass-transfer	Migration parameters to be determined	Quality of parameter evaluations under laboratory conditions
	I-1	Homogeneous rock formation	–	n	High
Formations of porous rocks	I-2a	Consolidated-stratified formation with limited differences in the permeabilities of rocks in the individual strata	Ratio K_1/K_{i+1} within the limits of one order	n_n	Corresponds to the accuracy of the differentiation of the structural elements of the stratum
	I-2b	Stratified formation with marked differences in the permeabilities of rocks in the individual strata	$K_0 \ll K$	D^* n n_0, D_M D^*	Ditto High High High
	I-3	Formation of unconsolidated structure (contains layer and lenses of slightly permeable rocks, not constant in area and size)	$K_0 \ll K$	n_n D^*	Corresponds to the accuracy of the differentiation of the structural elements of the stratum Ditto
	II-1	Formation of fractured rocks with negligibly small porosity of the rock masses	$K_0 \ll K$, $n_0 \approx 0$	$n = n_n$ $D = D^*$	Not determined in practice Ditto
Formations of fractured and porous – fractured rocks	II-2	Formation of fractured-porous rocks	$n \ll n_0$ $K_0 \ll K$	n $n_0 = n_n$ D_M S_δ (or m_δ) D^*	Ditto High High Not determined Low

Note. The following signs and symbols are used in the Table: n, D – active porosity (fracturing) and the coefficient of hydrodispersion in relatively easily permeable rocks; n_0, D_M – porosity and the coefficient of molecular diffusion of the rocks of the dividing layers or porous rock masses; n_n – estimated active porosity of the stratum; D^* – the estimated coefficient of macrodispersion of the stratum; S_δ – specific surface of the filtering fractures; m_δ – characteristic size of the porous rock masses; K, K_0 – the coefficients of filtration of relatively easily permeable rocks and of rocks of the dividing layers (porous rock masses).

Table 2.9

CLASSIFICATION OF THE MAIN TYPES OF RECHARGE AND EXPENDITURE OF GROUNDWATER THROUGH THE ZONE OF AERATION

No.	Types of water recharge	Description (definition) of the types selected			Conditions under which the type of recharge is encountered or is assumed in sandy desert	
		Infiltration I	Evaporation E	Plant suction PS	Thickness of the zone of aeration, m	Nature of the landscape
1.	INFILTRATION (rinsing, periodically rinsing)*	>0	0	0	More than 3	Sand dunes
2.	EVAPORATION (exudation)	0	>0	0	0–3**	Saline soils and sand dunes
3.	SUCTION (plant suction, not rinsing)	0	0	>0	3–17	Overgrown sands
4.	EQUILIBRIUM (not rinsing)	0	0	0	More than 17	Ditto
5.	MIXED TYPES, for example: suction-infiltration-evaporation					

Table 2.9, continued

No.	Types of water recharge	Description (definition) of the types selected			Conditions under which the type of recharge is encountered or is assumed in sandy desert	
		Infiltration I	Evaporation E	Plant suction PS	Thickness of the zone of aeration, m	Nature of the landscape
	(periodically rinsing, periodically exudation, periodically suction)	$E > 0$	$I > 0$	$PS > 0$	0–3	Ditto

* In parentheses the types of water regime of the soil under A.A. Rode's classification are given for information.

** Only for arid years.

For regions subject to seasonal freezing it is possible to single out, in addition, a frozen type of groundwater expenditure. The critical thickness of the zone of aeration (h_{Kp}) in excess of which it will be impossible to draw up liquid moisture to the frozen layer, is calculated for sandy soils according to the formula: $h_{Kp} = 2.5 + m$, where m – is the maximum thickness of the frozen layer in m. Thus, the frozen type, as a rule, may be encountered in thicknesses of the zone of aeration of less than 3–4 m. In this case, moisture is drawn up in winter to the base of the frozen layer, and in spring once again infiltrates right down to the groundwater level. Given the frozen type of water-exchange, favorable conditions are created in winter time for the protection of groundwater against depletion and contamination from the surface. From the point of view of the effect of groundwater on the zone of aeration given a gradual lowering or raising of its level, i.e., given excitement of the zone of aeration from the lower boundary, we may in turn single out respectively two types of water-exchange but not types of groundwater recharge): the drainage and underflooding (rise in the level of groundwater) types. At the same time, the recharge regime (expenditure regime) through the zone of aeration as a whole may be any type (equilibrium, infiltration, etc.).

at steady state or quasi-steady state. For infiltration of the recharge type, only the process of replenishment of groundwater due to atmospheric moisture is possible, whilst for the plant section type only the consumption of groundwater on the supply of plants occurs. It is obvious that the recognition of the type of water flux represents an extremely important task, inasmuch as the natural conditions of replenishment, depletion, salinization and contamination of groundwater are determined by this. For example, under the conditions in the Karakum desert, the location of all large sub-sand fresh-water lenses coincides with the areas of water recharge through the zone of aeration (G.N. Kunkin [20]), where infiltration recharge is possible or, at least, simultaneous loss of groundwater to the atmosphere is impossible. This case represents one where the conditions for the protection of groundwater and against depletion and salinization are at a maximum. To classify the types of water flux, the idea has been introduced of critical thicknesses of the zone of aeration under evaporation and plant suction, beyond which processes of physical evaporation and expenditure of groundwater for the supply of plants become impossible [37]. The critical thicknesses depend mainly on the lithology of the zone of aeration.

For the conditions of the Karakum, for example, the critical thicknesses with evaporation and plant suction respectively are equal to 3 and 17 m (Table 2.9). Thus, in sand dunes with a thickness of the zone of aeration of more than 3 m, and where vegetation is absent, only an infiltration type of water regime is possible and fresh-water lenses may be formed and sustained.

Under conditions of overgrown sands with a thickness of the zone of aeration of more than 17 m, recharge and expenditure of groundwater are absent and again fresh-water lenses formed in the past will be preserved for a long time unchanged, and the conditions of their protection against contamination are the most favorable. With a thickness of the zone of aeration of between 3 and 17 m in overgrown sands there will be a slow process of the loss of groundwater on the supply of plants and the salinization of this water. With thicknesses of less than 3 m the water will be salinized. At the same time, each type of water flux has its own curve on the moisture graph. Therefore, from the nature of the moisture field, and with information only on the critical thicknesses and conditions of vegetation it is possible to judge the conditions of groundwater protection. It should be pointed out that, for the conditions of a humid climate, the infiltration type of water exchange will predominate even with small thicknesses of the zone of aeration. Therefore, in those cases, the critical thicknesses should be used cautiously. Within the regions classified into a particular type of water flux through the zone of aeration, it is possible to refine the groundwater losses (for arid zones) by means of extrapolation curves. Fig. 2.11 shows such curves for the conditions in the Karakum desert (sandy soil). For clayey soils and other conditions of vegetation these curves will have a somewhat different appearance, but the general law remains unchanged.

From analysis of the extrapolation curves two important conclusions follow: a) with a decrease in the thickness of the zone of aeration the expenditure of groundwater on evaporation and plant suction increases markedly; b) with thicknesses of the zone of aeration of more than 4 to 5 m (for sands) and in the absence of vegetation, the quantity of atmospheric recharge becomes independent of the thickness of this zone. Under conditions of clayey composition of the zone of aeration, and at the sites covered with vegetation, the independence of the quantity of infiltration recharge from the thickness of the zone of aeration is observed with thicknesses of more than 8 to 12 m.

Therefore, with infiltration type systems, the best conditions for the conservation (replenishment) of

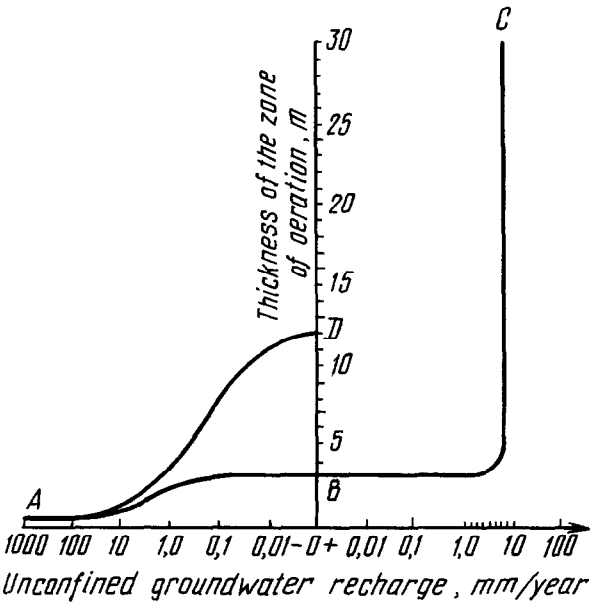


Fig. 2.11. Relation of the quantity of atmospheric recharge of the sandy Karakum Desert to the thickness of the zone of aeration.
AD – for sandy areas overgrown with vegetation;
ABC – for areas of sand dunes.

groundwater occur, but the protective properties of the zone of aeration may include unfavorable conditions. With the equilibrium type system, the replenishment of groundwater does not occur, but the protective properties of the zone of aeration are at a maximum. With the plant suction and evaporation type system, the prospects for groundwater loss are worst, and the protective properties of the zone of aeration are respectively favorable and unfavorable.

The classification of water flux systems makes it possible to evaluate sites and regions according to their groundwater protection states, when this depends on water flux through the zone of aeration. For protection at local sites, it may be necessary to carry out detailed work using thermodynamic, water-balance, tracer and other methods.

Forecasting the conditions for the protection of groundwater and the environment in situations which change with time is a complex task, which requires the construction of a dynamic hydrogeological model of the zone of aeration. An example is given of such a partial model, showing how the quantity of operating resources of groundwater will change with fluctuations in the position of the water table (Fig. 2.12). Under the humid climate, in a pine forest with a sandy zone of aeration and lowering of the level of groundwater from 0.9 to 2 m, the operating resources drawn upon will increase by 80 mm on the basis of an annual calculation. However, at the same time, the conditions for the water supply of the forest will deteriorate. The further lowering of the level during pumping will not lead to any noticeable gain in operating supplies. On the contrary, the increase in the level and the lowering of the thickness of the zone of aeration to 0.9 m and less will have a favorable effect on the water resources, but this is inadmissable from the ecological point of view, since serious effects on tree growth would ensue.

In drawing up hydrological, geotechnical and ecological predictions, a role of prime importance is played by the prediction of possible variations in the moisture field, of the capillary and pellicular potential, and the quantities of water flux through the zone of aeration, occurring during the raising or lowering of the groundwater level. In many practical cases it is often enough to use the moisture graph method to provide answers. Knowing the position of the groundwater level, from the graph it is possible to predict, for a given type of soil, the moisture distribution in the zone of aeration, which corresponds to the specific quantity of water flux through that zone. Using this method it is possible to forecast the lowering of the moisture content in the process of drainage of a marsh or a quarry or, conversely, the moistening of a soil during the rise in level and the subsequent flooding of industrial sites, premises and structures.

The zone of aeration may also be considered as a kind of self-regulating cybernetic system with feedback.

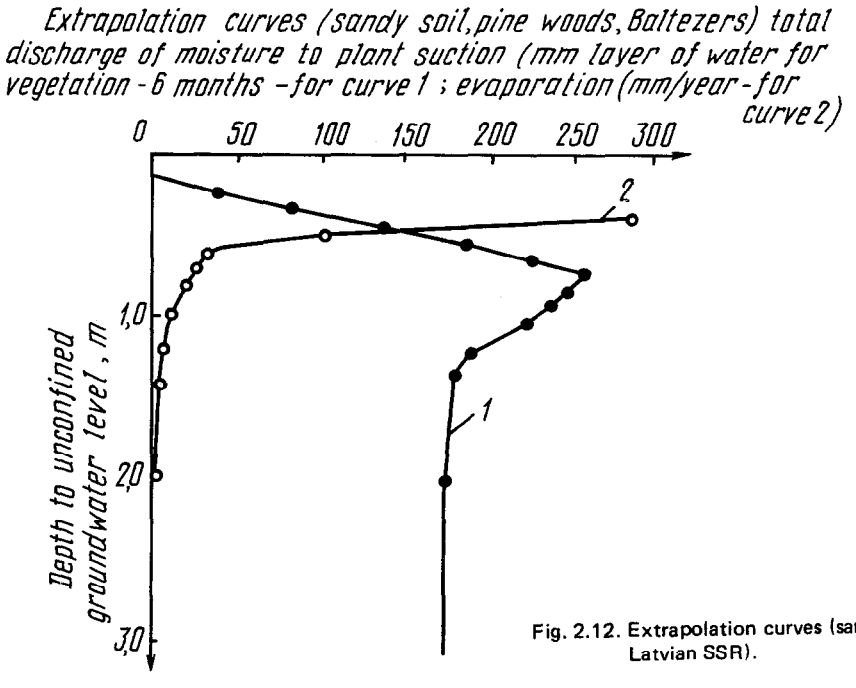


Fig. 2.12. Extrapolation curves (sandy soil, pine woods, Latvian SSR).

With an increase of filtration due to meteorological variations or man-induced action, a rise in groundwater level occurs. However, at the same time, the processes of evapotranspiration and lateral outflow of water in the aquifer increase, which reduces the rate of the rise in the level and leads in the last analysis to a stabilization of the water regime (according to Le Chatelier's principle). On the contrary, with intensified evapotranspiration at first a lowering of the groundwater level occurs. Subsequently, this process stabilizes, because evapotranspiration decreases, and the lateral outlow of groundwater is reduced. This same mechanism of self-regulation acts during the drainage or flooding of sites.

Thus, in a real-life hydrogeological situation, the levels, discharges of moisture and quantities of moisture in the zone of aeration fluctuate within strictly defined limits, indicating the stability and equilibrium of the zone of aeration as a thermodynamic system. With substantial man-induced effects and in cases of natural disasters, the system may go out of equilibrium, which will lead to irreversible and often undesirable consequences for the protection of the geological environment.

2.5. HYDROGEOLOGICAL FORECASTS OF THE MIGRATION OF CONTAMINANTS IN GROUNDWATER AND CHANGES IN GROUNDWATER QUALITY AT WATER INTAKE INSTALLATIONS

2.5.1. CONDITIONS FOR GROUNDWATER CONTAMINATION ON THE WATER INTAKE SITE

The movement of contaminated groundwater to a water intake depends on different factors: the type of water intake installation (single well, linear drain), the boundary conditions of the aquifer, the presence or absence of natural movement of the groundwater, flow pattern heterogeneity of the rocks (planar and stratified).

The natural movement of groundwater is a most important factor, on which the size of the recharge zone (sometimes called the capture zone) of the water intake depends. The recharge zone of the water intake is defined as the area of the aquifer, within the limits of which the lines of groundwater flow are directed immediately to the water intake installation.

The conditions of groundwater contamination at water-intakes differ in principle depending on the presence or absence of natural movement of the groundwater, characterized by a rate of filtration $v_e = K \cdot i_e$, where K — is the coefficient of filtration of the water-bearing rocks, i_e — is the gradient of natural flow. In the absence of natural movement of groundwater ("basin" conditions, $v_e = 0$) the water-intake recharge zone extends throughout the entire stratum. Because of this, under "basin" conditions the movement of groundwater to the water-intake proceeds from any point in the stratum. Therefore, if contaminated water is contained in such a stratum, sooner or later it will reach the water-intake. In this case the conditions for the contamination of groundwater at the water-intake works will be determined by the time of travel to it of the contaminated water and by the subsequent change in the quality of water.

With the presence of natural movement of groundwater ("flow" conditions, $v_e \neq 0$) a localised zone of recharge of the water-intake is formed in the stratum, which contributes to the water-intake. This zone is separated, from the remaining part of the stratum by a neutral flow line. The zone of recharge of the water-intake works is limited by the water-dividing point A downstream and extends an unlimited distance upstream. The length of the recharge zone is defined as the distance from the water-intake to the water-dividing point A. Laterally, the zone of recharge is limited by the neutral flow lines, with the width of the recharge zone increasing in the upstream direction. The length and width of the zone of recharge determine its size. The size of the zone of recharge and, the location of the water-dividing point A depend on the relationship between the yield of the water-intake and the discharge of the natural flow, the type of water-intake and the boundary conditions.

The vulnerability of the groundwater to contamination depends on whether or not a zone of contaminated water is captured by the recharge zone of the water-intake. Inflow of contaminated water to a water-intake occurs only when contaminated water is present within the recharge zone. Otherwise, the contaminated water will not be drawn into the water-intake at all.

Therefore, forecasting of the quality of groundwater at a water-intake, given the presence in the stratum of contaminated water, includes:

1) an evaluation of the potential for capture of the contaminated waters by the recharge zone of the water-intake; 2) an estimation of the time for the contaminated waters to be drawn to the water-intake works; 3) an evaluation of the change in quality of the water at the water-intake after the contaminated water has arrived.

The main types of centralized water-intakes are artesian wells in extensive platform-like regions and the wells in fluvial valleys. Accordingly, the most common typical aquifer systems are those of effectively unlimited area and those which are semi-limited in area with constant head boundaries. The main computing equations for these systems are given below.

The methodology of calculating the movement of contaminated waters to the water-intake site and for forecasting the quality of water at the site is given in detail in the works [2, 10, 13, 22]. Let us note that analytical solutions have been obtained for the most simple cases. Under complex hydrogeological conditions analytical-tabular methods and computer modelling are used.

2.5.2. ESTIMATING THE POTENTIAL FOR CAPTURE OF CONTAMINATED WATERS BY THE WATER-INTAKE PRECHARGE ZONE

The estimate is made on the basis of a determination of the position of the neutral (water-dividing) line of flow, limiting the water-intake recharge zone in relation to the boundary of the contaminated waters. The position of the neutral line may be obtained by the analytical-tabular method or from the analytical equations given below.

An aquifer unlimited in area

In the case of a single water-intake (well) or a concentrated water-intake on a given site (a group of wells), working with yield q, the distance from the water-intake works to the water-dividing point A is determined by the formula:

$$x_A = \frac{Q}{2\pi mki_e}, \qquad (2.88)$$

and the width of the recharge zone according to the formulae is

$$y_0 = \frac{Q}{4mki_e},\ y_\infty = \frac{Q}{2mki_e}, \qquad (2.89)$$

where y_0 and y_∞ – are half the width of the recharge zone respectively on the line of the water-intake works and at a considerable distance from it upstream; m – is the thickness of the aquifer; the origin of the coordinates coincides with the point of location of the single water-intake or the center of the concentrated group water-intake, axis x is directed along the flow of groundwater.

For a linear row of wells of great lengts the position of the water-dividing point is equal to [10]:

$$x_A = \frac{l}{\pi} \operatorname{Arth} \frac{Q_0}{2mlki_e}, \qquad (2.90)$$

where Q_0 – is the yield of an individual well in the row; l – is the distance between wells in the row; axis y coincides with the line of the row, axis x is directed along the flow, the origin of the coordinates coincides with one of the wells of the row.

An aquifer semi-limited in area with a constant head boundary (river)

For a single or a concentrated areal water-intake, located on axis x at distance d from the river (axis y coincides with the river boundary), the position of the water-dividing point on axis x is equal to:

the natural flow of groundwater directed away from the river

$$x_A = \sqrt{d^2 + \frac{Q \cdot d}{\pi mki_e}}; \qquad (2.91)$$

the natural flow directed towards the river

$$x_A = \sqrt{d^2 - \frac{Q \cdot d}{\pi mki_e}}. \qquad (2.92)$$

For a linear row of great length, located at distance d from the river (axis y coincides with the river) the position of the water-dividing point on axis x is equal to:

the natural flow directed away from the river

$$x_A = d + \frac{l}{2\pi} \ln \left(\frac{Q_0}{mlki_e} + 1\right); \qquad (2.93)$$

the natural flow directed to the river

$$x_A = \frac{l}{2\pi} \operatorname{Arch} \left(\operatorname{ch} \frac{2\pi d}{l} - \frac{Q_0}{mlki_e} \operatorname{sh} \frac{2\pi d}{l}\right) \qquad (2.94)$$

Where the natural flow of groundwater is directed to the river, the water-dividing point is formed between the water-intake works and the recharge boundary. With a growth in the yield of the water-intake (or a reduction in the discharge of the natural flow), the water-dividing point will approach the boundary of constant head.

For a single well or a group of wells concentrated on given sites, the criteria for estimating when the water-dividing point is located between the boundary of constant head and the water-intake, lies on the boundary or goes beyond the boundary, are respectively the following correlations:

$$\frac{Q}{\pi mdki_e} < 1,\ \frac{Q}{\pi mdki_e} = 1,\ \frac{Q}{\pi mdki_e} > 1. \qquad (2.95)$$

Similar criteria for a linear row of wells are

$$\frac{Q_0 \cdot \operatorname{sh} \frac{2\pi d}{l}}{mki_e \cdot l \left(\operatorname{ch} \frac{2\pi d}{l} - 1\right)} \gtrless 1; \qquad (2.96)$$

if $d \geqslant l$, then instead of (2.96) we may write:

$$\frac{q}{mki_e} \gtrless 1, \qquad (2.97)$$

where q – is the total yield of the linear row, related to its length.

The conditions where the water-dividing point will proceed to the boundary line of constant head and where the movement of the water to the water-intake works will begin from this boundary, are:

for a single well

$$\frac{Q}{\pi mdki_e} > 1, \qquad (2.98)$$

for a linear row of wells

$$\frac{Q_0 \cdot \operatorname{sh} \frac{2\pi d}{l}}{mki_e \cdot l \left(\operatorname{ch} \frac{2\pi d}{l} - 1\right)} > 1 \text{ or } \frac{q}{mki_e} > 1 \qquad (2.99)$$

The potentials for the drawing up of seawater to a water-intake site in a coastal zone are determined, in particular, by formulae (2.95) – (2.99).

2.5.3. MAIN ISSUES IN FORECASTING THE MIGRATION OF CONTAMINANTS IN GROUNDWATER

There can be two approaches in forecasting the displacement of contaminated water within an aquifer unit. The first, simpler one, assumes that the replacement of one fluid by another is "piston-like" in nature, and takes place uniformly without the formation of "tongues" and with a clearly marked surface of separation between the two fluids. In many cases such an assumption makes it possible to obtain relatively simple computing equations and ensures an accuracy of forecasting satisfactory for practical purposes. On the basis of the assumption of the "piston-like" replacement, analytical investigations have been made of the structure of filtration flows for various typical hydrogeological patterns taking into account the boundary conditions of the aquifer and the natural flow of groundwater, and computing equations have been obtained which make it possible to forecast the time and distance of travel of the boundary of contaminated water. In these constructions, contaminated water is usually presumed to be neutral in relation to rocks and groundwater and as not entering into interaction with them. The methodology for forecasting the movement of contaminated water in the stratum under the "piston" pattern of replacement is given by [2, 10, 22].

The other approach takes into account the random nature of the microstructure of the porous medium, the non-identical sizes of porous channels, the processes of molecular diffusion and sorption, as a result of which the "piston" replacement is disturbed, the fluids on the boundary of separation mix, and "tongues" are formed, so that scattering or dispersion of the boundary of separation of the two fluids occurs. According to [2, 6, 29, 36, 39, 66], the length of the zone of dispersion (L) may be determined from:

$$L_1 = \alpha' \sqrt{\frac{D}{n_0} \cdot t}, \qquad (2.100)$$

where D – is the coefficient of filtration diffusion (dispersion); t – is time; $\alpha' = 6.6$ for plane-parallel flow and $\alpha' = 4.7$ for plane-radial filtration.

Fresh and contaminated water usually differ in density. In the movement of fluids with different densities

a deformation of the boundary of separation occurs, which is expressed in the formation of a "tongue" of the heavier fluid along the base of the stratum. If we take it that the original boundary of fresh and contaminated (salt) waters was vertical and that the contaminated water is characterized by greater density, this boundary of separation subsequently becomes inclined because the heavier (contaminated) water supports the lighter (fresh) water, and at the base of the stratum a "tongue" of contaminated water is formed.

If, on the contrary, the contaminated water possesses a lower density than the fresh water, the "tongue" of contaminated water will be formed along the top of the layer. The length of the zone of deformation of the boundary of separation of fresh and contaminated water as a result of the difference in their densities is equal to [2, 6, 34, 39]:

$$L_2 = \alpha\sqrt{\frac{km\,(\gamma_1-\gamma_0)\cdot t}{n_0\gamma_0}}, \qquad (2.101)$$

where k and n_0 – are respectively the coefficient of filtration and the porosity of the water-bearing rocks; m – is the thickness of the aquifer; $\Delta\gamma = (\gamma_1-\gamma_0)$, γ_0 and γ_1 – are the densities of fresh and contaminated water; t – is time; $\alpha = 4$ – for the plane-parallel flow and $\alpha = 2.8$ – for the radial flow.

The total length of travel of the salt water is composed of path (L_0), determined from the pattern of "piston" replacement, the length of the zone of dispersion (L_1) and the length of the zone of deformation (L_2), i.e.:

$$L = L_0 + 0{,}5\,(L_1 + L_2). \qquad (2.102)$$

Usually the length of the zone of dispersion is small as compared with the length of the path traveled under the "piston" replacement pattern and amounts only to some tens of meters. Therefore, in most cases, this length may be neglected. It is only in markedly heterogeneous fractured media that the size of the zone of dispersion (or of macro-dispersion) increases and reaches hundreds of meters and even kilometers.

The density contrasts between fresh and salt or contaminated water is most important with mineralization of salt water greater than 10–15 g/l. With mineralizations of salt water of 5–7 g/l, the difference in the densities of fresh and salt water is small, especially if there is a transitional zone between fresh and salt water, within the limits of which a gradual change in the mineralization of groundwater occurs. Therefore, in the range of mineralizations of salt water up to 5–7 g/l, the difference in the densities of fresh and salt water may be neglected for an approximate evaluation of the displacement of salt water. Under these conditions, the path (or time) of movement of salt waters are determined by their displacement under the "piston" pattern of displacement.

The paths, or time of movement of contaminated water are evaluated by analytical equations, analytical-tabular methods or by the mathematical modelling techniques.

The analytical-tabular method consists of dividing the path along the line of flow into several segments, determining the rate of flow in the middle of these segments, and the time of passage of each segment; the times for the individual segments are integrated and the time of movement of salt water along the entire path is found.

Below are given the analytical equations for the evaluation of the time and path of movement of contaminated water under the pattern of "piston" replacement, for the systems of an aquifer unlimited in area and a semi-limited aquifer with a constant head boundary. In each system, "basin" conditions, where the natural movement of groundwater is absent or is so small that it may be neglected and "flow" conditions, where the natural movement of groundwater exists and is taken into account in the calculations are assumed. These equations may be applied to a broad range of contaminants, which, together with groundwater, form a system of mutually soluble and miscible fluids. The dispersion of the boundary of separation is taken into account according to formula (2.100); in the event of a substantial difference in densities, an adjustment is introduced to compensate the deformation of the boundary of separation using formula (2.101). At the same time, it is assumed that the contaminants are not sorbed or are poorly sorbed by the rocks. Sorption slows down the propagation of the contamination front and lowers the concentration of contaminants in the groundwater. Therefore, a hydrogeological forecast of contamination movement in the stratum without taking into account sorption processes gives an upper limit of the distance of propagation of the contamination in the stratum. In many cases such a "safety margin" in forecasting calculations of the distance of migration of contamination is considered fully justified.

The equations below are used for forecasts of contaminant migration through porous, homogeneous sediments or through rocks with a uniformly developed fracture pattern. In the case of karstified rocks and rocks with markedly heterogeneous fracturing, the use of the analytical equations is admissible only to obtain the roughest kind of estimates.

2.5.4. FORECASTING THE DRAWING-UP OF CONTAMINATED WATER TO A WATER-INTAKE INSTALLATION

A water-intake in an aquifer unlimited in area

A single well

A well or a group of wells is located at distance x_1 from the rectilinear boundary of contaminated water.

The origin of the coordinates coincides with the centre of the water-intake works. Under "basin" conditions the flow is plane-radial. The time of movement of the contaminated water along the shortest distance (of axis x) from the initial position x_1 to the marker x_2 is determined from:

$$t = \frac{\pi m n_0}{Q}(x_1^2 - x_2^2), \qquad (2.103)$$

where Q – is the yield of the water-intake works; m – is the thickness of the aquifer; n_0 – is the porosity of the water-bearing rocks.

If it is necessary to find the time of inflow to the water-intake works of the first arrival of contaminated water, then $x_2 = r_c \approx 0$ and then

$$t = \frac{\pi m n_0 x_1^2}{Q}, \qquad (2.104)$$

where r_c – is the radius of the well or the radius of "the big well".

The time of movement of the contaminated water from any other point of the boundary of separation is equal to:

$$t = \frac{\pi m n_0}{Q}(r_1^2 - r_2^2). \qquad (2.105)$$

where r_1 and r_2 – are respectively the initial and final positions of the point.

$$r_1 = \sqrt{x_1^2 + y_1^2}, \qquad r_2 = \sqrt{x_2^2 + y_2^2}.$$

Under "flow" conditions the time of movement of the contaminated water within the recharge zone along the main line of flow (of axis x) from the boundary of the contaminated water to the water-intake is estimated by:

$$t = \pm\frac{n_0}{v_e}[x_1 - x_A \cdot \ln(\frac{x_1}{x_A} + 1)], \qquad (2.106)$$

where $x_A = \frac{Q}{2\pi m v_e}$, x_1 – is the shortest distance from the boundary of contaminated water to the water-intake works.

The sign "+" or "–" is taken respectively in the movement along the flow or against the flow; in the latter case abcissa x_1 has a negative value. The time of inflow of the first arrival of contaminated water at the water-intake is determined from (2.106).

The time of movement of contaminated water along any line of flow within the recharge zone from the initial position of the boundary of contaminated water to the water-intake site is determined by the equation:

$$t = \pm\frac{n_0}{v_e}(x_1 - x_A \cdot \ln|\cos\frac{y_1}{x_A} + \frac{x_1}{y_1}\sin\frac{y_1}{x_A}|), \qquad (2.107)$$

where x_1 and y_1 are the abcissa and the ordinate of the initial position.

Circular group of wells

A circular group of wells with radius R, consisting of n wells; the capacity of an individual well is Q_0. The particles of fluid are displaced most rapidly along the main line of flow. The time of movement of contaminated water along the main line of flow from the initial marker r_1 to the boundary of the group (R) is determined according to the formula:

$$t = \frac{\pi m n_0 R_k}{n \cdot Q_0}(\bar{r}_1^2 - 1 - \frac{2}{n-2} + \frac{2}{(n-2)\bar{r}_1^{n-2}}), \quad \bar{r}_1 = \frac{r_1}{R}; \qquad (2.108)$$

if $\bar{r}_1 > 3$ and $n \geqslant 4$, then:

$$t = \frac{\pi m n_0 R^2}{n \cdot Q_0}(\bar{r}_1^2 - 1 - \frac{2}{n-2}). \qquad (2.109)$$

When $\bar{r}_1 > 5$ and $n \geqslant 10$, the results of calculations according to formula (2.109) do not practically differ from calculations using formula (2.108); for a single well or a group of wells with a discharge equal to the discharge of a circular cluster of wells, the error amounts to less than 5 %.

An unlimited row of wells

If the length of the row is 5–6 times more than the distance to the boundary of contaminated water parallel to it, such a row may be considered as infinite, in calculations of the time for drawing up contaminated water.

For "basin" conditions, the time of movement of contaminated water along the main line of flow from the initial position x_1 to the marker x_2 is determined according to the formula:

$$t = \frac{2n_0 m\sigma^2}{\pi Q_0}\ln\frac{\operatorname{ch}\frac{\pi x_1}{\sigma}}{\operatorname{ch}\frac{\pi x_2}{\sigma}}, \qquad (2.110)$$

where σ – is the distance between wells in the row, Q_0 – is the yield of an individual well.

If the time for the drawing-up of the contaminated water to the linear row exists, then $x_2 = r_c \approx 0$, where r_c – is the radius of a well in the row and

$$t = \frac{2n_0 m\sigma^2}{\pi Q_0}\ln\operatorname{ch}\frac{\pi x_1}{\sigma}. \qquad (2.111)$$

When $\frac{x_1}{\sigma} > 2.5$, we have approximately:

$$t = \frac{2n_0 m\sigma x_1}{Q_0} \text{ or } t = \frac{2n_0 m x_1}{q_0}, \qquad (2.112)$$

where $q_0 = \frac{Q_0}{\sigma}$ – is the discharge of the row reduced to a unit of its length.

For "flow" conditions, when the boundary of contaminated water is located higher upstream, the time for the advance of the contaminated water to the linear row of wells may be calculated approximately according to the formula:

$$t = \frac{n_0 \Delta x_1}{\frac{q_0}{2m} + v_e} \text{ where } q_0 = \frac{Q_0}{\sigma}. \qquad (2.113)$$

A limited linear row of wells

Consider a linear row with the length 2l; axis y follows the line of the row, axis x through its midpoint.

If the linear row is placed perpendicular to the natural groundwater flow, the rate of movement along the main flow line of axis x is described by the equation [22]:

$$v_x = \frac{Q}{2\pi n_0 l m}\operatorname{arctg}\frac{l}{x} \pm \frac{v_e}{n_0}, \qquad (2.114)$$

where Q – is the total yield of the linear row. The sign "+" of "–" before the second term indicates whether the boundary of contaminated water is upstream or downstream from the row. If the natural flow is insignificant and may be neglected, then in the formula (2.114) the second term is taken as equal to 0.

The flowrate in the points of the axis beyond the limits of the row is equal, for "basin" conditions, to [22]:

$$v_y = \frac{Q}{4\pi n_0 l m}\ln\frac{y+l}{y-l}. \qquad (2.115)$$

Given the existence of natural groundwater flow, directed along the row (along axis y), the rate of natural flow is added to, or subtracted from rate v_y, depending on whether the boundary of contaminated water is located upstream or downstream from the water-intake works.

A water-intake in an aquifer semi-limited in area with a boundary of constant head (river)

Single well

Let a single well, working with a constant yield, be located on axis x at a distance d from the boundary of constant head (river), axis coincides with the boundary. The time of movement of the groundwater along the main

line of flow (axis x) from the initial position x_1 to the well is equal to:

$$t = \frac{\pi n_0 m d^2}{3Q}\left[2 + \left(\frac{x_1}{d}\right)^3 - 3\frac{x_1}{d}\right]. \quad (2.116)$$

If the boundary of contaminated water is located between the water-intake and the constant head contour, then $x_1 < d$, if from the opposite side, then $x_1 > d$. If the boundary of contaminated water coincides with the outline of constant pressure, then $x_1 = 0$ and formula (2.116) assumes the appearance:

$$t = \frac{2\pi m n_0 d^2}{3Q}. \quad (2.117)$$

Under the conditions of natural groundwater flow, directed away from the outline of recharge (of axis x) to the water-intake works, the time of movement of the contaminated water to the water-intake site is determined by the equation:

$$t = \frac{n_0}{v_e}\left[(d - x_1) - \frac{x_A^2 - d^2}{2x_A}\ln\frac{(x_A + d)(x_A - x_1)}{(x_A - d)(x_A + x_1)}\right], \quad (2.118)$$

where x_1 – is the initial position of the boundary of contaminated water. If the movement of contaminated water takes place from the direction of the constant head contour, then $0 < x_1 < d$ but, if it is from opposite side, then $d < x_1 < x_A$, where x_A – is the position of the water-dividing point, determined from formula (2.91). If the boundary of the contaminated water coincides with the constant head contour ($x_1 = 0$) then

$$t = \frac{n_0}{v_e}\left(d - \frac{x_A^2 - d^2}{2x_A}\ln\frac{x_A + d}{x_A - d}\right). \quad (2.119)$$

When the natural groundwater flow is directed towards the constant head contour, then the time for the drawing-up to the water-intake of contaminated water from the initial position of x_1 is found according to the formula:

$$t = \frac{n_0}{v_e}\left[\frac{d^2 - x_A^2}{x_A}\left(\operatorname{arctg}\frac{d}{x_A} - \operatorname{arctg}\frac{x_1}{x_A}\right) + x_1 - d\right], \quad (2.120)$$

where x_A – is determined by equation (2.92).

Linear row of wells

In the case of an unlimited linear row of wells, situated parallel to the boundary of constant head and at a distance d from it (axis y coincides with the boundary, axis x is perpendicular to the row), under "basin" conditions the time of progress of the contaminated water from initial position x_1 to the water-intake is equal to:

$$|t| = \frac{n_0 \sigma m}{Q_0}\left[(d - x_1)\operatorname{cth}\frac{2\pi d}{\sigma} - \frac{\sigma}{2\pi}\left(1 - \operatorname{th}\frac{2\pi d}{\sigma}\right)\right]. \quad (2.121)$$

If the movement of the contaminated water occurs directly from the boundary of recharge, i.e., $x_1 = 0$, then

$$t = \frac{n_0 \sigma m}{Q_0}\left(d \cdot \operatorname{cth}\frac{2\pi d}{\sigma} - \frac{\sigma}{2\pi}\right); \quad (2.122)$$

when $\frac{2\pi d}{\sigma} > 2$ value $\operatorname{cth}\frac{2\pi d}{\sigma} \approx 1$ and then

$$t = \frac{n_0 \sigma m}{Q_0}\left(d - \frac{\sigma}{2\pi}\right). \quad (2.123)$$

Question of forecasting changes in the quality of water at the water-intake works after the first arrival of contaminated water are considered in detail in works [2, 10, 13, 22] and are omitted here.

Chapter 3.

PROTECTION OF GROUNDWATER AGAINST DEPLETION

3.1. GENERAL DESCRIPTION OF VARIOUS TYPES OF GROUNDWATER STORAGE AND STOCKS. SOURCES OF USABLE WATER SUPPLY FORMATION

The term "groundwater depletion" is generally understood to mean decreases in the amount of groundwater available. It is necessary, therefore to provide clear definitions of certain concepts referring to the amounts of groundwater accumulated both under natural conditions and in settings disturbed by economic activity, as well as examining factors relevant to the practical use of such waters.

3.1.1. TYPES OF GROUNDWATER SUPPLIES AND RESOURCES

Natural accumulations of organic and inorganic substances in the earth's crust which can be effectively tapped and used for material production are generally referred to as "useful minerals". By virtue of this definition, groundwater resources tapped as water supplies, and for other practical purposes, are also considered to be "useful minerals".

The advisability of using useful minerals, and more especially groundwater, can be determined only on the basis of the "size" of the available supply. The "reserves" of a useful mineral are usually taken to mean the weight of the mineral present in the ground. In contrast to many other useful minerals (solids, petroleum and gas), groundwater has a number of specific characteristics which preclude assessing prospects for its possible use on the basis of "supply size". The chief distinguishing characteristic of groundwater which makes it different from other useful minerals is the fact that it is renewable. Groundwater constitutes one of the links of the water cycle in nature. Its mobility allows its continuous replenishment and depletion. In undisturbed natural conditions water enters aquifers through infiltration of atmospheric precipitation, surface water absorption, leakage from other strata, etc. Water which has previously entered aquifers is discharged into surface watercourses and water bodies, and is lost by evaporation, spring flow and transfer to other strata. Variations in hydrometeorological conditions can affect significantly the rates of groundwater accumulation and discharge on both seasonal and long time scales. When considering groundwater use problems it is necessary to make allowance not only for the total amount of water available, but also for groundwater replenishment.

In contrast to all other useful minerals, groundwater use may be a process which entails not only water expenditure, but also additional water supply formation due to concomitant changes in water exchange conditions in the aquifers. The lowering of groundwater levels may lead to such effects as the onset or intensification of influent seepage from rivers, the reduction or complete cessation of spring outflows, loss by evaporation, and the start or intensification of water transfer from adjacent aquifers.

Additional groundwater accumulation may result from various water resource management measures (hydraulic engineering, irrigation), as well as by action directed at artificial groundwater recharge. In such circumstances it is possible to produce new aquifers (through decreases in the thickness of the aeration zone) and also to increase the annual volume of groundwater replenishment.

An important property of groundwater as a mineral resource is the continuous character of its use, which implies a steady removal of specified amounts of water from the ground. Such removal can be effected through water intakes (of which wells and boreholes are the most common examples) whose output is determined by the seepage properties of the water-bearing rocks, water table depth, and the engineering characteristics of the water intake, operations. Even in favorable circumstances, where large amounts of groundwater are present and where annual recharge of aquifers is satisfactory, groundwater supply may be impracticable due to very low discharge rates of the wells and to other technical difficulties of groundwater use (e.g. sand and salt deposition in wells and the lack of necessary pumping equipment).

The fundamental characteristics of groundwater, which make the resource different from other useful minerals necessitate the definition of method for describing:

a) the amount of water present in an aquifer;

b) the amount of water entering an aquifer under i) natural conditions and ii) under the influence of various water management measures, including those involved in groundwater abstraction;

c) the amount of water which can be withdrawn from aquifers by technically feasible and economically viable water intakes.

Whereas the assessment of prospects for the use of solid minerals, petroleum and gas is linked with the essential concept of "useful mineral reserves", in the special case of groundwater this concept does not afford a general yardstick for evaluating the feasibility of the rational use of this resource. It was this consideration which led F.P. Savarenskii to suggest the additional term "groundwater stocks" to reflect the replenishment of aquifers and thus to complement the concept of "groundwater storage".

At present a large number of systems for classifying groundwater storage and stocks are available. The differences between these classifications are essentially ter-

minological. The system applied most widely in hydrogeological studies in the USSR is that of N.N. Bindeman [1], whereby groundwater storage and stocks are subdivided into the following categories: 1) natural; 2) artificial; 3) attracted; 4) usable.

The term "natural (cavity) storage" is applied to the volume of gravity water contained in the pores and cracks of aquifer. As far as strata containing free water are concerned, a distinction is made between the volume of gravity water contained in the aquifer below the zone of natural water table fluctuations and the water volume within the zone of fluctuation on the other. The latter volume is referred to as the "regulated storage". Natural storage in artesian aquifers includes the "elastic storage". The latter is defined as the volume of water which can be withdrawn from an aquifer by lowering of the water table, when the elastic properties of the water and of the host rock come into play.

Natural groundwater storage has the dimensions of volume.

"Natural stocks" is the term applied to the amount of groundwater entering an aquifer under natural conditions due to the infiltration of atmospheric precipitation, influent seepage from rivers, leakage from overlying and underlying aquifers, and inflow from adjacent zones. The natural stocks constitute the sum total of all of the contributory elements of the balance for the aquifer. They are expressed in flow rate units and may also be determined by summing the uptake elements of the balance (evaporation, transpiration by vegetation, spring flow, effluent seepage into surface watercourses and water bodies, seepage into adjacent strata etc.).

"Artificial storage" is defined as the volume of groundwater which accumulates in an aquifer as a result of irrigation, leakage from reservoirs, and artificial replenishment of the groundwater supply (restocking).

"Artificial stocks" constitute the sum of inflows into the aquifer through seepage from canals, reservoirs and irrigated areas, as well as those due to measures to augment groundwater accumulation.

"Attracted storage" is the water inflow into an aquifer which is associated with the increased groundwater accumulation due to the operation of water intakes, resulting in the onset or increase of seepage from rivers and lakes, leakage from adjacent (usually overlying) aquifers and decreases in surface evaporation of groundwater due to the lowering of the water table.

The terms "usable storage" and "usable stocks" are virtual synonyms as far as groundwater is concerned. They stand for "that amount of groundwater which can be obtained by means of technically and economically practicable water intake facilities which are appropriate to a given use and which ensure adequate water quality throughout the entire water consumption period for which they are designed". This quantity represents the output of a given water intake facility and is expressed in flow rate units (usually in thousands of cubic meters per day). The term "usable storage" is currently used in assessing prospects for the application of groundwater to satisfy particular user needs. In those cases where the potential use of groundwater in a particular region is being considered as part of the overall available supply, the term "usable stocks" is preferred.

In tackling groundwater use problems it is vital to evaluate the usable groundwater storage (or stocks), since the results of such an assessment are the only possible basis for deciding whether or not to construct water intakes. At the same time the assessment of the natural storage and natural stocks may serve as a valuable aid in dealing with any number of general geological problems (determination of water exchange rates, groundwater age assessment, etc.) as well as problems of a more specialized kind (estimation of subsurface runoff into rivers, seas, oceans, etc.).

3.1.2. SOURCES OF USABLE GROUNDWATER STORAGE FORMATION AND THE FUNDAMENTAL BALANCE EQUATION

In the general case usable groundwater storage is related to other forms of reserves and stocks by the following balance relation:

$$Q_u = L_1 Q_n + L_2 \frac{V_n}{t} + L_3 Q_a + L_4 \frac{V_a}{t} + Q_{att}, \quad (3.1)$$

where Q_u is the usable storage (or stocks), in m^3/day;
Q_n and Q_a are the natural and artificial stocks, in m^3/day;
V_n and V_a are the natural and artificial storage, in m^3;
Q_{att} is the attracted stocks, in m^3/day;
t is the period of time for which the usable groundwater storage is being calculated, in days;
L_1, L_2, L_3, L_4 are, respectively, the coefficients of utilization of the natural stocks, natural storage, artificial stocks and artificial storage.

The above balance equation is simply a schematic representation of the formation of usable groundwater storage over the entire period "t". Depending on hydrogeological conditions, the dominant role in the formation of usable groundwater storage is played by various sources, which may vary with time.

As may be inferred from Equation (3.1), usable groundwater storage can be furnished by sources either for a specific period or over an indefinite time. In the latter case the sources contributing to usable storage formation comprise the natural and artificial stocks as well as attracted stocks (if these are in turn available over an indefinite use period), since as $t \to \infty$ the second and fourth terms in the right side of Equation (3.1) become zero.

The processes of variation of the amount of groundwater in aquifers during use vary in character, depending on the degree of involvement of various forms of groundwater storage and stocks in the formation of the usable groundwater storage.

The detailed description of usable groundwater storage formation, under various hydrogeological conditions, may be made by examining closely the specific role of the various sources which contribute to such a storage.

The feasibility of using natural groundwater stocks and the degree of their involvement in the formation of usable storage depend not only on the amount of natural stocks available, but also on a number of geological and technical factors. The most important of these is the permeability of aquifers, the permissible lowering of the water table, and the areal distribution deployment of water intakes within the groundwater catchment and discharge areas. The higher the permeability, the greater the permissible lowering of the water table, and the closer the water intakes are situated to the groundwater discharge contours, the greater the portion of natural stocks which can be tapped by means of water intakes.

The most favorable conditions for the fullest use of subsurface runoff are to be found in the groundwaters of artesian basins in intermontane depressions and pediment fans. Due to the small size of these deposits and their high permeability, such locations are characterized by the presence of groundwater discharges within the zone of influence of water intakes. This in turn ensures the most complete use of natural stocks. Deposits of this type are found in abundance in such areas as Transcaucasia and the Central Asian republics of the USSR. The usable groundwater stock moduli* formed by the attraction of natural stocks in such conditions may exceed 5–10 liters/sec·km^2, attaining values of 20–30 liters/sec·km^2 and even higher in certain individual cases (40 liters/sec·km^2 in the Ararat basin of Armenia).

A significant role in the formation of usable groundwater storage is played by natural groundwater stocks confined to smaller structures and massifs formed of fissured and karstic carbonate rocks. Such deposits, examples of which include certain structures found in central Kazakhstan, are characterized by the total utilization of natural groundwater stocks. Under the natural conditions which prevailed in the past these were discharged around the periphery of the structure via evaporation, plant transpiration and spring flow.

Conditions least conductive to the involvement of natural stocks prevail in mountainous regions. Here the usable storage is usually quite small despite considerable subsurface runoff. The conditions for groundwater accumulation through the infiltration of atmospheric precipitation are also more favorable in mountainous regions. Groundwater use in such areas is, however, largely confined to the capping of larger springs which emerge locally from the aquifers for example the fissured and karstic Upper Jurassic limestones and recent lava flows in the Caucasus.

Considerable natural stocks are also found in other areas where opportunities for constructing large number of wells are limited. In humid regions such locations include crack fissured rock basins, where natural stocks run as high as hundreds of cubic meters per day. However, such basins are generally characterized by low permeability of the fissured aquifers (10–20 m^2/day) and low effective thickness of the fissured zone. This combination of factors precludes the provision of dense well networks. The latter are located in zone of high fissure density associated with river valleys and tectonic fault zones. Consequently, the usable groundwater storage in such areas amounts to only a few per cent of the natural stocks.

The conditions in deep artesian aquifers may also be unfavorable to the development of natural groundwater stocks. An exception to this rule are the peripheral zones of artesian basins, where usable groundwater storage may be significantly enhanced by natural stocks present in both the tapped aquifer and, to a greater extent, overlying minor aquifers.

The size of natural groundwater storage depends on the lithology of the aquifer, the types of cavities which are present, and the thickness and extent (area) of the aquifer. The largest natural storage moduli** are usually exhibited by thick gravel deposits found in intermontane depressions and piedmont valleys. The thickness of such aquifers may be up to several hundred meters, and the water yield factors as great as 20–30 %. The modules of natural (gravity) storage under such conditions reaches tens of millions of cubic meters per square kilometer (for example, for a thickness of 100 m and a water yield factor of 0.2, the modulus is equal to 20 million m^3/km^2). Such large gravity storage moduli are typical, for example, of Quaternary aquifers situated in the depressions of Central Asia and Transcaucasia.

Large moduli likewise characterize the aquifers of platformtype artesian basins, where the water-bearing rocks consist of sandy formations with water yield factors of 0.1–0.2. For example, the natural (gravity) storage moduli of the Buchak aquifer which occur widely in the region of the Dnepr-Don artesian basin, are on the order of 5 million m^3/km^2.

Somewhat lower moduli (1–2 million m^3/km^2) are exhibited by river valley aquifers, where the thickness of the water-bearing sand-gravel deposits does not usually exceed 10–20 m, as well as by aquifers associated with fissured and karstic deposits whose water yields are low (1–4 %) despite their considerable thickness (up to 150–200 m). The lowest gravity groundwater storage moduli are exhibited by fissured igneous and metamorphic rocks where the thickness of the active fissuring zone does not exceed a few dozen meters, with the water yield ranging from tenths of one per cent to 1–2 %. Under such conditions the moduli do not exceed 0.5–1.0 million m^3/km^2.

An elastic groundwater storage is characterized by elastic water yield factors whose values are one to two orders of magnitude smaller than the gravity water yield factors. Hence, the size of an elastic storage usually constitutes only a few per cent of the total natural storage.

It should be noted that the possible involvement of natural groundwater storage in the formation of usable storage is determined by both size, and by the recharge conditions including the depth of the cover layer overlying the aquifer. For deeply buried aquifers (cover depths exceeding 300–500 m) the use of gravity reserves is not feasible, because operations under such conditions rely exclusively on the elastic groundwater storage without actually withdrawing water from natural storage. Because of this, the largest usable storage moduli are exhibited by intermontane depressions and piedmont valleys, with modulus values as high as 3–5 l/sec·km^2 over a use period of 50 years. At the same time the elastic storage depletion modulus (for example the Moscow, Baltic, Dnepr-Don and Azov-Kuban artesian basins) often does not exceed a few hundredths of a liter per sec·km^2. However, despite their small elastic storage moduli, the usable groundwater storage of deeply buried aquifers found in platform-type artesian basins is constituted largely by elastic storage and may be significant owing to the very large area of such aquifers.

The natural groundwater storage of a small valleys with periodic river flows plays an important role in usable storage formation because the natural storage is drawn

*The term "usable groundwater stock modulus" means the average water flow rate (in litres per second) obtainable from a single square kilometer of aquifer area.

**The term "natural reserves modules" means the amount of water (in millions of cubic meters) obtainable by voiding one square kilometer of an aquifer.

upon only during periods when there is no river flow. Under such conditions flooding is accompanied by the formation of new natural groundwater reserves, which are tapped during the next dry period.

Attracted groundwater stocks play a major role in usable storage formation, especially in river valleys and platform-type artesian basins.

The principal source of attracted resources in river valleys is surface runoff. The degree of surface water involvement in usable storage formation depends on hydrogeological and hydrological factors, as well on engineering use considerations. The principal factors and considerations are the amount and regime of surface runoff, the conditions of interaction between the subsurface and surface waters (composition and permeability of stream bed deposits), the seepage properties and thickness of the aquifer, the permissible water table drawdown, and the distance of the water intakes from surface watercourses or lakes. The combined influence of hydrological and hydrogeological factors determines the relative role of attracted stocks in the formation of usable groundwater storage in specific natural settings. Thus, in major river valleys with continuous water flow, a surface runoff rate in excess of possible water uptake by water intake facilities and active interaction between surface – and groundwater, attracted stocks form the main source of usable storage throughout the use period. Under such conditions the usable storage formed by surface runoff attraction attains quite high values. Practical experience shows that the linear moduli of attracted resources may be as high as 300–400 liters/sec, and reach 1–2 m^3/sec per linear kilometer in instances.

In the valleys of intermittently dry or frozen rivers and low-flow watercourses, as well as where the connection between subsurface and surface waters is impeded, the attraction of river runoff occurs only during high-water periods and is particularly intense during spates, i.e. when the water level in the river rises to flood additional catchment areas. Under these conditions groundwater use during drought periods entails depletion of the natural storage of the aquifer system, and is accompanied by a continuous lowering of the water table. Spates are accompanied by a complete or partial reestablishment of the water table, at its pre-drought level.

In groundwater resources found in platform-type artesian basins, the principal source of attracted stocks formation are the natural groundwater storage and the stocks available in overlying aquifers. Under conditions of this type, attracted stocks are formed by abstraction from the main aquifer, which initiates or intensifies seepage through interbedded low-permeability deposits. The intensity of such water transfer is determined mainly by geological and hydrogeological factors.

Factual information on the operation of large numbers of water intakes indicates that leakage through low-permeability layers occurs both through "hydrogeological windows" and through the actual low-permeability horizons when the latter is fissured, because the intergrannular permeability of such deposits is often insufficient to account for the amount of leakage observable under natural conditions. Experience gained in the operation of groundwater intakes shows that leakage occurs both in regions where sufficiently large "hydrogeological windows" are present and in places where discontinuities in low-permeability layers are not apparent. A classical example of leakage through "hydrogeological windows" is the formation of usable storage in the Middle Carboniferous strata found in the central portion of the Moscow artesian basin. The latter area region includes locations where the Upper Jurassic clays, overlying limestones aquifer of Middle Carboniferous age, have been entirely washed away. This in turn results in the leakage of groundwater from the overlying Mesozoic deposits, the attracted stocks being the principal factor contributing to usable groundwater storage. Analysis of data on the operation of water intakes in platform-type artesian basins carried out at the VSEGINGEO has shown that leakage processes are of fundamental importance in a large number of cases, especially when the aquifer is not too deeply buried (for example, in the Baltic, Moscow, Azov-Kuban and other artesian basins).

Artificial groundwater storage and stocks play an essential role in usable storage formation only in those areas which have been involved in water resources management measures (irrigation, hydraulic engineering construction), and in areas where groundwater supplies are replenished artificially (principally in operational catchments). Artificial stocks are especially important in irrigated regions where the assessment of usable storage necessarily entails allowance for the return water which contributes to the formation of subsurface runoff in the aquifers under assessment. The isolation of artificial stocks and storage is particularly important in such instances, since rebuilding of an irrigation system may alter the conditions of usable groundwater storage formation.

In areas where hydraulic engineering structures are present and operating, an important role in usable storage formation may be played by artificial storage formed in reservoir shore zones.

Artificial stocks formed during artificial replenishment of groundwater at operating intakes are the principal source of groundwater supplies at such intakes.

Changes in the size of individual usable storage formation sources (natural storage and stocks, attracted stocks, etc.) and their differing relative significance under varying natural conditions renders their distribution over a given area nonuniform. This must be borne in mind in assessing prospects for groundwater use and in the planning of prospecting operations.

3.2. THE CONCEPT OF "GROUNDWATER DEPLETION". DESCRIPTION OF GROUNDWATER DEPLETION PROCESSES UNDER VARIOUS NATURAL AND MAN-INFLUENCED CONDITIONS

3.2.1. "GROUNDWATER DEPLETION"

Having considered the principal forms of groundwater storage and stocks and described their role in usable storage formation we now turn to a more detailed discussion of the concept of "groundwater depletion". It should be noted that hydrogeologists have not yet agreed on a common definition of this concept.

Most researchers hold that groundwater depletion is what occurs in all instances where groundwater abstraction is accompanied by a continuous decline of the water table with concomitant decreases in replenished groundwater stocks. These authors identify groundwater depletion with the use of natural (cavity) groundwater storage, which leads them to develop recommendations whereby groundwater use should not exceed groundwater replenishment.

On the other hand, some researchers [1, 13] hold that groundwater depletion, being a direct result of unreason-

able groundwater use, occurs when groundwater abstractions exceed usable groundwater storage. These hydrogeologists include under the general heading of "groundwater depletion" the numerous instances of senseless dumping into rivers of groundwater taken up by the various drainage and water level lowering installations employed in mining, flood prevention and land reclamation. By this definition groundwater depletion includes other instances of uneconomical expenditure of abstracted groundwater, such as the use of drinking-quality groundwater in industry where strict requirements as to quality do not apply.

In practice, it is important in dealing with processes entailing decreases in groundwater storage to draw a distinction between processes requiring special groundwater protection measures and processes where such measures are not necessary. It may be noted in this connection that where usable groundwater storage is formed largely through the takeup of natural storage, groundwater use often remains a reasonable proposition even with an overall decrease in storage, and does not require special measures to protect the groundwater from depletion.

The problem of using groundwater should be approached just as one approaches the extraction of other solid, liquid and gaseous minerals, but it should be borne in mind, that groundwater use may be accompanied by the formation of additional supplies. Thus, as was correctly noted by N.N. Bindeman [1], "... Whereas at some far-distant point in the future the supplies of petroleum or iron will have been exhausted, there is no similar threat in regard to groundwater, and man will always be able to draw upon the water entering aquifers at the point of supply". The imposition of limits on groundwater abstraction on the basis of the size of available renewable storage would sharply reduce the potential for groundwater use, which in turn would certainly affect prospects for economic development of regions and areas. It is certain that the utilization of groundwater supplies can be confined to a specified period of time. However, groundwater use during a restricted period can make possible to postpone expensive water management measures. Very convincing arguments in favour of natural groundwater use have been advanced by US hydrogeologists Davis and De Weist [19]. Reasoning from experience in the use of groundwater in the arid areas of the United States, they point out that: "Few people in these areas appreciate the fact that the constant intensive groundwater use results in aquifer depletion. On the other hand, if groundwater use is confined to natural replenishment, then supplies will not be depleted, even though this leaves room only for small-scale and inefficient mining activities, scratch farming and minor industry. On the contrary, intensive groundwater use makes possible efficient use of roads, equipment, human and animal power etc. Groundwater use must therefore be planned along the same lines as the extraction of non-renewable supplies of coal, iron ore and other useful minerals". At the same time, in planning the use of natural stores of groundwater and making ample use of their excellence as drinking water one must see to the preservation of a certain portion of the natural storage as a backup for future use. This is particularly important in the vicinity of large cities, where groundwater is the sole source of water.

In this connection it important to isolate two concepts, 1) "groundwater depletion" and 2) "depletion of usable groundwater supplies".

The term "groundwater depletion" should be thought of as covering all cases of reduction of natural and/or artificial groundwater supplies which involve rates of groundwater expenditure exceeding the rate of groundwater replacement. Possible reasons for groundwater depletion include changes in replenishment conditions (deforestation, river channel alignment and diversion, etc.) and groundwater abstraction.

Cases of "usable groundwater storage depletion" in catchment areas include those where groundwater abstraction exceeds the established usable storage* as well as the reduction of usable storage due to changes in storage formation conditions. Another form of usable storage depletion is the use of drinking-quality groundwater for industrial purposes.

The distinction between these two notions consists in the fact that whereas the depletion of usable storage requires taking special protective measures (reduction of abstraction rate to usable storage levels, artificial groundwater replenishment), groundwater depletion necessitates such measures in particular cases only, for example when groundwater released during mining or flood control operations is needlessly dumped without ever having been put to use.

3.2.2. GROUNDWATER DEPLETION UNDER VARIOUS NATURAL AND MAN-INFLUENCED CONDITIONS

Groundwater depletion occurs whenever the natural and/or artificial removal of groundwater from an aquifer exceeds groundwater replenishment. One sure indicator of groundwater depletion is the continuous decline of the water table with a resulting diminution of natural storage (gravity storage in the case of unconfined aquifers and elastic storage in the case of confined formations). What occurs with depletion in the case of unconfined aquifers is drainage of the pores and fissures with a consequent decreases in their effective thickness; depletion in confined aquifers is accompanied by decreases in groundwater pressure heads.

A distinction should be made between temporary groundwater depletion and permanent depletion. In the former case depletion proceeds only within a given time interval and is generally associated with seasonal variations in groundwater supply, and no depletion may occur over a span of several years. In the latter case, which is generally associated with economic activities, the decline of the water table occurs over a very long period of time.

It should be noted that a continually declining water table over an operational area in an aquifer does not in itself constitute an unmistakable indicator of groundwater depletion. It is fairly common to note a continuous decline table water, despite the fact that the total recharge to the aquifer exceeds groundwater abstraction. As was pointed out in Section 3.1.2, the potential for attracting groundwater to intakes depends on a number of geological and engineering factors. This means that there is continuing abstraction of groundwater from the aquifer, with the intakes attracting only that part of the water received

*In France the notion of "usable groundwater storage depletion" is denoted by the term "overuse".

by the aquifer recharge which does not compensate groundwater in the area influenced by the intake. Under such circumstances the notion of "groundwater depletion" should be applied only to closed-balanced hydrogeological systems in which the total abstraction from the aquifers equals or exceeds the recharge rate.

Groundwater depletion can occur under the influence of either natural or artificial factors. Natural factors include changes in climatic conditions which determine the amount of groundwater recharge as well as changes in the position of the groundwater abstractions due to neotectonic processes. Artificial factors include various forms of economic activity, such as deforestation, hydraulic engineering and reclamation projects, and direct groundwater abstractions for water supply, irrigation, mining and drainage operations.

Groundwater depletion as a result of natural factors is generally very limited and does not play a principal role in water resources use. However, where the principal aquifers are quite thin (not more than 5–10 m in thickness) and of unconfined (phreatic) type, variations in effective aquifer thickness due to prolonged below average recharge (drought) may become quite important.

Deforestation has considerable bearing on groundwater depletion. In the case of slope forests, the forest growth holds back surface water, impeding its flow and thus enhancing groundwater replenishment by influent seepage. Logging activities tend both to increase surface runoff and to reduce subsurface flow due to the drop in groundwater level which occurs when the supply of water is reduced. For example, studies carried out in the Urals have shown that a reduction of forest cover density from 90 % to 55 % results in a three- to five-fold increase in flood runoff and a reduction of low-water winter runoff by a factor of five to fifteen.

Groundwater depletion may also be enhanced by water resources management and reclamation works (river channel diversion and alingment, draining of ponds, provision of too much drainage etc.) which have the effect of drying out aquifers.

All things considered, the principal role in groundwater depletion is played by the operation of water abstraction and dewatering facilities. The operation of such installations has the effect of altering locally the groundwater balance which is apparent by the formation of the cones of depression in the aquifer(s), in the alteration of the direction of groundwater flow, and in the transformation of recharge areas into discharge areas. The specific forms of ground water depletion (the rate and intensity of depression funnel formation) differ depending on the uptake rate and on the hydrogeological conditions which determine the principle sources of usable groundwater storage formation and the water inputs into mining excavations. On this basis water abstraction and water table lowering installations may be subdivided into the following five groups [16]:

Group I water intakes tap deep aquifers underlying aquicludes of several tens of meters in thickness and which in platform-type artesian basins. The principal source of active storage supplying mining excavations for this group is elastic storage; the operating condition is nonsteadystate; the radius of the cone of depression may be as large as 50–100 km and the maximum depression of the prezometric surface be in the range 50–100 m.

Group II water intakes tap mostly shallow aquifers connected through overlying strata by "hydrogeological windows" or moderately permeable layers. The principal sources of active storage under such conditions are lateral flows from adjacent aquifers. The flow regime in these cases generally reaches a steady state with time. The dimensions of the cone of depression are usually smaller (on the order of 10–30 km in radius, with water table drops in the order of 20–30 m). There may be noticeable, but very slight, drop in the water tables in overlying aquifers.

Group III intakes are those which are situated near permanent rivers whose flow rates and channel bed and channel throughput significantly exceed the water abstraction rates. In such cases the only source of active groundwater storage is surface run-off. The cones of depression are quite small (radius of 2–10 km and a depth of 20–30 m), and a steady state seepage regime is established. There is essentially no groundwater depletion, even though surface runoff decreases – a factor which must be allowed for in assessing overall water supplies.

Group IV intakes are located in the vicinity of intermittently dry or frozen rivers, or rivers with a limited flow. Active replenishment for this group of intakes is the natural storages of the tapped and adjacent aquifers, during low-water periods, and surface runoff during flows. The cone of depression is usually of limited size.

Group V intakes are those situated within structures and rock massifs of limited size (for example limited massifs of fissured and fissured karst rocks, intermontane depressions, and the lower portions of debris cones) within which groundwater discharge is virtually complete. When abstraction is less than the natural storage available the groundwater regime becomes stable with time, but in the case where abstraction exceeds recharge it is of a nonsteadystate character. Cones of depression are defined by the structural boundaries. Abstraction in excess of recharge entails continuous reduction of the natural groundwater storage.

Groundwater abstraction for water supply or irrigation purposes is a reasonable proposition, even though it may in certain circumstances, entail groundwater depletion because it constitutes a direct use of a useful mineral.

At the same time, solid ore mining as well as reclamation activities entail quite considerable groundwater depletion. In these instances an enormous quantity of water may be pumped from the depths of the earth, and then simply discharged as surface runoff, at times having undergone significant pollution on the way. The scale of water abstraction associated with various drainage operations may be considerable, as a result of which the sound use of pumped water is a matter of some importance. This issue is dealt with in greater detail in Section 4.6.2.

3.2.3. DEPLETION OF USABLE GROUNDWATER RESERVES

The depletion of usable groundwater reserves by operating wells and, in certain instances, those still in the planning stage, may occur: a) when groundwater abstraction exceeds the established usable resource; b) by alteration of usable groundwater storage in areas where water resource use and reclamation projects are underway; and c) when pumped water is not use rationally and economically.

Unquestionably the greatest hazard is presented by groundwater abstractions in excess of usable storage. This may result in water table drops more rapid than anticipated, which in turn reduces the useful life of the abstractions and necessitates the modification of wells, entailing as it does a decrease in their output.

In other cases water abstraction exceeding the usable storage may result in a considerable deterioration of groundwater quality due to the attraction to the abstraction of groundwater with high total especial dissolved solids hard. This is particularly the case when developing freshwater lenses and groundwater-bearing structures in the vicinity of seashore.

This makes it very important to establish the size of usable groundwater resource with some precision.

The depletion of usable groundwater storage may also be due to changes of water resources management and reclamation measures. In many parts of the world, populations rely on river valley aquifers as their source of water supply, in settings where the bedrock aquifer is separated from the river by an aquifer composed of alluvial deposits. If the river bed deposits become highly silted during low-water periods, groundwater is obtained at the expense of natural groundwater storage with subsequent replenishment of the storage during the high-water period. Under these circumstances the degree of replenishment is determined by the frequency and intensity of flooding. Replenishment of depleted storage is most complete when the catchment area is flooded. The building of reservoirs upstream of a catchment area can produce a radical change in the condition of available groundwater replenishment, due to decreases in the duration, number and intensity of floods. As an example, the construction of a reservoir upstream of a certain catchment area resulted in a continuous lowering of the water table because of continuation of groundwater abstraction at their previous rate coupled with the elimination of flooding. This in turn led to a reduction in the productivity of the water intakes and a deterioration of groundwater quality due to the lateral flow of extrahard water from the valley slopes.

Depletion of groundwater storage in irrigated areas can be attributed to the reduced groundwater recharge associated with irrigation network reconstruction. As is shown in [15], an increase in irrigation network efficiency from 0.52 to 0.60 can result in a reduction of almost 30 % in the artificial recharge due to seepage losses from the irrigation system.

Similarly, a negative role in the depletion of usable groundwater storage is played by uneconomical and improper groundwater use, with intakes taking up much more water than the user requires. This is especially so when someone has resorted to the use of free flowing wells, not equipped with regulating devices.

Another important problem requiring solution is the restriction of drinking-quality groundwater to appropriate uses only. Such restrictions must be legally enforceable. To take the USSR as an example, its "Principles of Water Legislation" permit the use of drinking-quality groundwater for nondrinking and household purposes only in those exceptional cases where available groundwater reserves exceed considerably the highest conceivable demand for such water.

On the basis of the foregoing considerations we can formulate the following basic aspects of usable groundwater storage protection (and under certain circumstances of groundwater protection in general) against depletion:

1) determination, on the basis of special hydrogeological investigations, of the size of usable groundwater storage in particular regions and subregions with a view to setting a top limit for rational use;

2) regulation of the groundwater use regime with due allowance for the interaction of groundwater with surface water and for the need to ensure the integrated use of all water resources based on data concerning the size of usable groundwater storage, and the institution of measures to monitor uptake and use;

3) artificial recharge of groundwater resources at existing intakes and the creation of artificial storage (groundwater stockpiling);

4) the integrated use of groundwater associated with the dewatering of mining excavations, flood protection schemes and land reclamation;

5) tightening up of groundwater use legislation and enforcement.

Therefore, in order to detect examples of potential groundwater storage depletion it is firstly necessary to establish the size of the reserve, which represents the possible output of water intakes as assessed and imposed in the light of hydrogeological, technical, economic and ecological constraints. The issue in question is discussed in Section 3.4.

3.3. A BRIEF DESCRIPTION OF THE BASIC PROCESSES INVOLVED IN GROUNDWATER USE

As was shown in the previous sections, the processes of groundwater resource depletion are most strongly affected by the intensive groundwater abstraction and drainage facilities. On the other hand, groundwater use not only alters the hydrogeological setting in which depletion processes occur it also influences other components of the human environment. Important amongst those are the concomitant changes in surface water/groundwater interactions, and changes in the surface water runoff regime; the drying out of ground and soil due to lowering of the water table and the stunting or death of plants; surface subsidence due to secondary consolidation and compaction of sand and clay layers associated with decreases in aquifer pressure; and finally, the intensification of collapse structure and karst formation when using karst fissure water.

Since the planning of measures to combat and contain these processes must go hand in hand with the development of recommendations on the protection of groundwater against depletion, we shall discuss the aforementioned technogenic processes briefly in the present section.

3.3.1. THE INFLUENCE OF GROUNDWATER UPTAKE ON SURFACE RUNOFF

As one of the constituent elements of the hydrosphere, groundwater is closely related to other forms of natural water and disturbing the equilibrium of groundwater through abstractions may alter other components of the hydrosphere. For example, a lowering of the groundwater table implies, among other things, reduced evaporation at the groundwater surface. Groundwater abstruction may have major effects on river flow. Its influence in this respect is not simple. In the majority of cases groundwater use is accompanied by a reduction (or cessation) of base flow discharge to rivers and by the onset of river seepage into the aquifer, therebye reducing surface water resources. In other circumstances, involving the abstraction from aquifers without an active hydraulic connection with surface watercourses, it is possible to observe a fairly protracted increase in surface runoff due to the dumping into rivers of waste waters derived from the groundwater.

Assessing the effect of groundwater uptake on surface water discharge rates is a very important matter when determining the overall (surface plus subsurface) water resources of a given territory and when compiling water resources management balances. It is occasionally suggested that the input side of a water resources balance equation must allow only for those usable groundwater storages which are hydraulically unconnected with surface waters and which can be drawn upon without reducing surface runoff.

Such a statement is not valid because the presence of a hydraulic connection between groundwater and surface water does not prevent very broad variations of surface runoff during groundwater use. Moreover, the abstraction of groundwater, even though it does tend to curtail surface runoff, nevertheless has the effect of increasing overall water resources. The amount by which river flow diminishes under the influence of groundwater usage and the way in which it varies with time depend on a large number of factors. It should be borne in mind that in many cases the period of flow reduction tends to be prolonged. Under certain conditions groundwater abstraction may have no effect on surface runoff for some considerable time. The length of this period depends on the distance between the wells and the river, the hydrogeological properties of the aquifers (their permeability and storativity), and the presence or absence of other pathways for water loss in addition to direct flow into rivers (e.g. evaporation and transpiration). There are cases on record where surface runoff underwent no change whatever. This can happen, for example, when groundwater uptake does not exceed the reduction of evaporation losses. In other instances the use of groundwater may have an impact on high-water runoff only, the low-water flow remaining unchanged. Such conditions are characteristic of aquifers with a large regulating capacity whose usable resource is provided by natural groundwater supplies during low-water periods with subsequent replenishment during flows. All of these peculiarities of surface runoff variation as influenced by groundwater intake operation must be taken into account in assessing usable storage. Analytical solutions are now available for determining the variation of surface runoff during groundwater use in relatively simple hydrogeological settings. For complex hydrogeological conditions estimations may be made using mathematical models [14, 22, 23]. In view of the fact that groundwater and surface water both form parts of a single hydrosphere, it is often not possible to consider their potential use in isolation. In other words, assessments should include both the possibilities for their conjunctive or concurrent use.

Studies carried out in the USSR have shown that groundwater abstractions have a significant bearing on the flow of smaller rivers only. Moreover, there have been cases where groundwater use has resulted in the complete drying up of certain segments of such streams.

Studies by researchers of the State Hydrogeological Institute as the Meshchera Depression (southeast of Moscow) have indicated [8] that the proportion of base flow to rivers situated within the cone of depression had dropped some 50 % when compared with basins with uninterrupted runoff as a result of groundwater intake operation. The normal value of the total flow of these same streams had decreased by 20 %. Similar results have been noted in studies of the effect of mine water discharge on the surface runoff of depressions. Thus, water losses within the cone of depression from the river Oskolets due to discharge from the Lebedyan' Quarry within the Kursk Magnetic Anomaly amounted to 0.3 m^3/s [12], whereas the decreases in the flow of streams estimated at different times by various researchers [4, 11] situated in the influence zone of the depression around the Mirgalimsai Ore Deposit in the Karatau region turned out to have virtually the same value, ~ 1 m^3/s. Section 4.6.3. contains a reference to a study by Belorussian hydrogeologists who showed by on-site experiments that groundwater abstraction on a scale commensurate with river flow could result in the reduction of an already low river flow by about 70—80 %, and even by 100 % in some cases.

3.3.2. THE INFLUENCE OF INTENSIVE GROUNDWATER UPTAKE ON SURROUNDING LANDSCAPE CONDITIONS

Groundwater uptake exercises its greatest influence on landscape conditions when water is drawn from unconfined aquifers or from artesian aquifers sufficiently closely linked with unconfined aquifers. What occurs under such conditions is that quite large dewatered zones from in the near-surface strata, which can result in marked, and most often adverse, changes in the landscape. It should be noted that landscape deterioration due to groundwater uptake is today a fairly rare occurrence and does not constitute such a major threat to the environment as does, for example, the contamination of surface and ground water or atmospheric pollution. However, instances of such calamities are not unknown, the abstraction of groundwater by water intake and drainage systems is on the rise, and the whole issue certainly merits continued study. Here we shall consider the damage to the landscape due to the killing off of vegetation because of lowering of the water table by groundwater abstractions.

When the lowering of the water table results in the dewatering of aquifers, in the increase in depth of the aeration zone, and in lowering of the capillary fringe surface, the consequent changes in the moisture contact of the soil and near-surface layers result in the dewatering and destruction of vegetation. For example, overpumping of groundwater and the periodic drying of the alluvium of the river Severskii Donets has resulted in the disappearance of flood plain and meander (oxbow) lakes and the dehydration of forests. The extensive cones of depression which formed as a result of the dewatering of mines in the Kursk Magnetic Anomaly area [5] (e.g. at the Stoilenskii Ore Deposit) have caused problems with rural water supplies, even though a sufficiently humid climate has prevented the complete desiccation and disappearance of vegetation. The assessment of usable groundwater reserve in the Samur-Kusarchai interfluve area on the borderland between Azerbaidzhan and Dagestan involved selecting a groundwater abstraction regime and groundwater level control strategy such as to prevent damage to valuable relic forests situated within the cone of depression. The close interconnection between vegetation, mainly forest, and groundwater has been noted by Kittredge [10] who gives examples both of changes in groundwater regimes as a consequence of logging operations in Cowita (North Carolina) and of forest desiccation following lowering of the water table.

One example of the effect of groundwater abstractions on an arid zone environment is that of the Aidos Water Intake in the vicinity of Dzhezkazgan. Under natural conditions at the Aidos Intake, situated in the

valley of an intermittent river where groundwater occurs in a fissured karstic aquifer and is closely linked with surface water during the flood season, the portion of the valley which lies nearest the stream bed is covered with moisture-loving vegetation, i.e. trichohydrophytes, phreatophytes and hydrophytes. The fluvial terraces above flood-plain are overgrown with ombrophytes with a projective cover of 50–70 % and take little part in transpiration, except at the foot of the bedrock shore, where dense *Lasiagrostis splendens* and *Sophora japonica* stands testify to the closeness of groundwater. The principal role in transpirational discharge is played by the upper and lower flood-plains and by the river bed, whose vegetation includes willow, moss, reeds, reed-mace and other species. The transpiration was determined in the field by White's method [32] and by the rapid weighing method devised by L.A. Ivanov [9], as well as by the *Blaney-Kridl* indirect method [18]. According to the findings, the transpiration loss in an average year with constituted 500–550 mm, with only the *Blaney-Kridl* estimates yielding a somewhat higher value of 600–620 mm.

The abstraction of groundwater by the intake at the rate of 800 liters/s and the associated drop in the water table led to the desiccation and withering away of vegetation in the river valley and a sharp drop in the transpiration rate. The pure hydrophytes such as willow, reed-mace and reeds died off fairly quickly (see Figure 3.1). The phreatophytes (wormwood, Tatar honeysuckle, dogrose) withered away partly or exhibited a much inferior habit.



Fig. 3.1. Hydrophytes – willow thickets (Salix australior), reed mace (Typha angustifolia) and others which died out as a result of the lowering of the groundwater by 15–20 meters below the natural level.



Fig. 3.2. Reach of the Kara-Kengir River, which dried up as a result of the use of groundwater at the Aidosk-water-intake.

The lowering of the water table had almost no effect on the trichohydrophytes. However, as a result of the death of the hydrophytes and the worsened condition of the phreatophytes, as well as the dewatering of the alluvium and the resulting alteration of the soil moisture regime with consequent withering away of meadow herbs, the valley landscape deteriorated over a stretch 4 km long. The deeps were drained (Figure 3.2), free flow ceased, and much of the vegetation disappeared. In our view, however, this particular case: the damage to the natural setting, occasioned by water intake operation was to some extent justified. There are over 300 km of such river valleys with periodic flow in the area, so that the desiccation of a 4-km segment cannot be considered too serious a blow, the deleterious effects of groundwater abstraction not being manifested either to the sides of the watersheds, or, for that matter, upstream or downstream along the valley. At the same time the supply of 800 liters/s of highquality fresh water in an arid zone (the rainfall being 200 mm with an evaporation rate of 1 000 mm) was a considerable achievement, the advantages for the local economy in this instance certainly outweighing the damage done to its environment. One measure which might mitigate the deleterious effects described would be the irrigation of part of the valley with groundwater during the May-September period. Calculations show that this could be accomplished by providing a discharge of 20 liters/s (approximately 2 % of the pumped output).

The facts set out in the present section show that intensive groundwater uptake, especially in an arid region, is sometimes associated with undesirable environmental consequences (drying out of vegetation, desiccation of forests, reduction of the surface flow of smaller rivers). Although such incidents are still a rare occurrence and are not as a rule of disastrous scale, nevertheless the increasing rates of groundwater uptake should lead hydrogeologists, ecologists and other specialists to maintain a close watch on the problem under consideration.

3.3.3. THE PROCESSES OF SECONDARY CONSOLIDATION OF DEWATERED ROCKS AND DEPRESSIVE COMPACTION

The phenomenon surface settlement as a result of artificial lowering of the water table or the reduction of piezometric pressure has been spreading rapidly over the past few decades, and is now occurs in extensive zones in many regions of our planet.

Fifteen to twenty years ago surface subsidence incidents were being noted in Japan, the United States, the USSR, Mexico, Great Britain and Italy, but they are now occurring also in Thailand, the Republic of South Africa and other places.

Land subsidence is not simply the result of intensive groundwater use. The same effect may also occompany by-product of petroleum and gas extraction, and the discharge of mine water during solid ore extraction, land reclamation and other activities.

The following discussion considers mainly land subsidence occasioned by groundwater use.

As already noted, protracted, large scale withdrawal of groundwater radically alters virtually all of the elements of the natural hydrogeological setting, i.e. the conditions of inflow, runoff and discharge, water tables, pressure heads, flow rates, chemical composition and water temperature.

Recharge areas become the sites of depressed groundwater table or pressure-head surfaces which are capable of

attaining enormous size. Major depressions exist in a number of cities (London, Paris, Moscow, Leningrad, Kiev, Berlin, Tokyo, Osaka et al.). The depth of the cones of depression in urban locations can be 50–100 m and even larger. In Moscow, for example, the depth of the depression in Carboniferous aquifers reaches 40–50 m. The corresponding depth in the case of the Gdov aquifer in Leningrad is 65 m; that of the cone in the Cretaceous aquifer in London is greater than 100 m, and so on. The depressions in mineral extraction areas (petroleum, coal etc.) are large still.

In irrigation areas where groundwater is used the depth of depression cones in aquifers can exceed 150 m (as in the San Joaquin Valley in California, USA).

The degree of land subsidence depends not only on the magnitude and intensity of water uptake, but also on local rock compressibility. Solid coherent rocks are virtually noncompressible.

Unconsolidated pebble beds, gritty and sandy rocks undergo little compression, except for their loose components. The greatest susceptibility to consolidation is shown by clayey soils, silts, sapropels and peats. The degree to which these rocks can be compacted, with consequent subsidence, depends on their age, genesis and composition.

Recent, fine grained, poorly compacted and hydrophilous water-saturated soils show the highest defromability. The interbedding of clayey rocks, which can readily undergo considerable compression, with sand-gravel and other permeable formations (including fissured consolidated rocks) intensifies and accelerates the process of soil draining and compaction, when subject to substantial groundwater abstractions. Land subsidence in these geological conditions can attain large values (as in Mexico City, Tokyo, Osaka and elsewhere).

The dynamics of subsidence, resulting from a declining water table, depend on several factors; the intensity of water abstractions, the scale of water table drawdowns and pressure head changes, the geological structure and morphology of the region, rock and soil compressibilities, and the thickness of compressible strata.

Secondary consolidation of unconsolidated rocks is possible when significant water table declines such as to diminish the suspending influence of water with a concomitant increase in the net gravitational forces acting on the soil, augmentation of the effective load and compaction of the compressible rocks of the aquifer.

The increase ΔP in the pressure acting on the rock skeleton due to a lowered water table can be computed by means of the following formula:

$$\Delta P = (\delta - \delta_u) \cdot h \cdot 0.1 \qquad (3.2)$$

where ΔP is the increment in the load acting on the rock skeleton, in kg/cm^2;
δ is the density of the rock prior to the water table drop;
δ_u is the density of water-suspended rock;
h is the water table drop, in m.

Taking the average density of the rock prior to the water table drop to be 1.8 and the density of the rock suspended in water equal to unity, we find that the pressure increase ΔP for various water table drops is as follows:

Linear meters	10	20	30	40	50	60	70	80	90	100	200
ΔP, kg/cm^2	0.8	1.6	2.4	3.2	4.0	4.8	5.6	6.4	7.2	8.0	16.0

Artificially induced water table declines of several hundred meters have been reported and drops of several dozen meters are quite common.

Lowering of the piezometric level in artesian aquifers has the effect of increasing the pressure exerted by overlying rocks on such aquifers by 1 kg/cm^2 for every 10 m decline. The appearance of pressure head gradients is accompanied by water outflow from semipermeable clayey soils, in the high-pressure to low-pressure direction, and results in subsidence or lowering of the land surface.

Subsidence gives rise to a number of undesirable associated processes and effects which are at times exceedingly costly. The phenomenon in question entails rises in the groundwater table, which understandably means swamping of certain land areas as well as the flooding of basements in both private and industrial buildings. Impact on shores and banks includes the inundation of ocean, sea and lake beaches, river banks and reservoir shores, which may adversely affect the value of properties in towns and/or industrial centres. Examples of such situations are given below.

Nonuniform subsidence related to the special characteristics of local geology can wreck buildings, railways and roads, canals, petroleum, steam, water and sewerage pipelines and other subsurface conduits, as well as demaging bridges, port amenities, embankment walls and other structures.

Land subsidence in Japan was first noticed at the beginning of the present century in the cities of Tokyo and Osaka. Today such phenomena are a common occurrence in that country.

Yamamoto [34] notes that the number of subsidence-prone areas is continuing to increase, having reached 40 locations by 1975 (Figure 3.3).

For quite a number of years, groundwater use in Japan has been much more intensive than in other countries. This is due to the fact that being an archipelago, it does not have major rivers and lakes, while its water demand continues to increase from year to year due to industrial growth and the ever increasing acreage under irrigation.

The principal aquifers employed in Japan as sources of drinking, household and industrial water lie in Quaternary deposits. Those richest in water are the alluvial-flavial deposits of mountain river and padiment fans. It is these aquifers which provide the greater part of the water used in Tokyo, Osaka and other cities.

Tokyo is a metropolis situated in the southeastern part of the island of Honshu on the Kwanto Plain. It occupies an area of 573 km^2. The area of Greater Tokyo is 2000 km^2.

A high-precision levelling of the land within the Tokyo city limits was carried out after a strong earthquake had struck the Tokyo area in 1932. According to N. Miyabe [28], comparison of the results of the most recent levelling with those obtained in the same area one year previously showed that certain bench marks had sunk by as much as 15 cm. This prompted a careful check of several sets of levelling data obtained previously for the area in question.

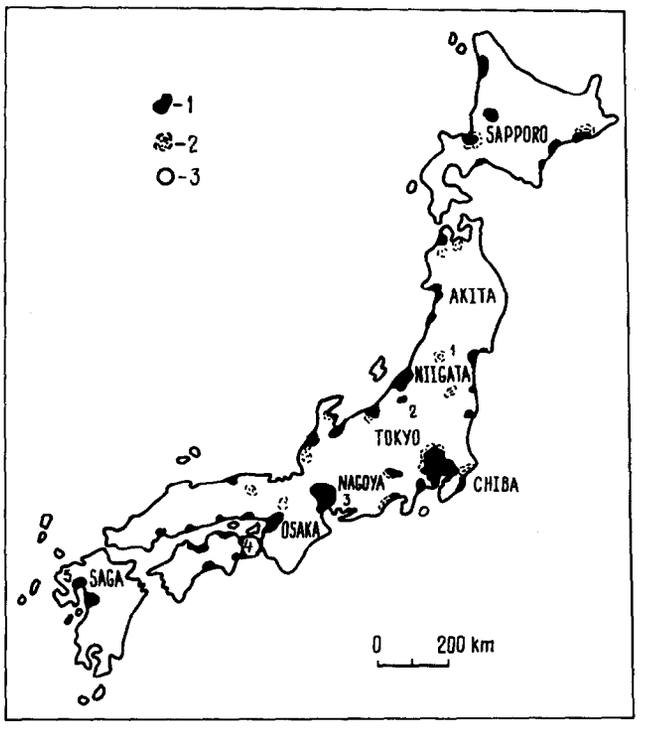


Fig. 3.3. Areas of subsidence of the earth's surface in Japan (Yamamoto, c. 1977)
1 — areas of subsidence of the earth's surface which had existed in 1965 and in 1975.
2 — new areas of subsidence of the earth's surface since 1975.
3 — presumed areas of subsidence of the earth's surface.

The repeated levellings showed that land subsidence in the city of Tokyo was gradually increasing and that the area within which such an acceleration could be ascertained amounted to some 100 km^2 between the rivers *Shumida* and *Ara-Kawa*.

It was established at the same time that the area exhibited an apparent uplift of well head and buildings with deep-set foundations relative to the land surface.

Land subsidence in coastal cities where the land lies only a few metres above sea level is undersirable and may cause enormous damage to private and public property as well as endanger the lives of millions of human beings. For this reason studies of land subsidence in Tokyo began to be conducted on a systematic basis.

Precise levelling operations to establish the magnitude of annual land subsidence in Tokyo are carried out once every two years, with annual measurements in particular areas where subsidence rates are especially high.

Artesian waters in Tokyo occur in low-density Quaternary deposits of dry floods from the rivers Ara-Kawa, Shumida and Edogawa. Studies have shown that subsidence is strongest in locations where the Quaternary deposits have their greatest thickness. In addition, the magnitude of subsidence depends on lithological structure and on the physical and mechanical properties of the local rocks. It has been shown, for example, that areas where sand and gravel soils predominate tend to subside only very slightly, and that maximum lowering of the land surface occurs in places whose geological section reveals mostly clays and silts.

Land subsidence ceased towards the end of the Second World War due to the reduced water uptake. Certain locations even exhibited the onset of land rise. After the War, when water abstraction began to increase the subsidence rate began to increase rapidly once again.

Tokyo municipal authorities organized a campaign to combat the disastrous lowering of the land surface in the city. The principal antisubsidence measure taken is the restriction of water consumption. Antiflooding measures include surrounding of subsidence-prone areas with dikes, earth fill, etc.

Groundwater in the city of Osaka and its suburb of Amagasaki lies in the Old Quaternary deposits of the river *Yodo-Gawa*, which are composed of interbedded gravel, gravelly sand and clay with a thickness ranging from several meters to 30 m. The Quaternary deposits have been found to contain artesian waters with a common piezometric level. Studies of subsidence in various parts of the city of Osaka were begun in 1935 with the installation of 200 special benchmarks. These studies have provided indications of subsidence rates during various periods.

During the years immediately preceding the War (1937–1940), increasing water uptake was accompanied by progressive subsidence at rates as high as 20 cm per annum. During the Second World War and somewhat later, when industrial production in Osaka had diminished and when water uptake fell accordingly, subsidence virtually ceased, with lifting of the land surface in certain areas. The resumption of industrial activity which began in 1950 brought with it a rapid increase in subsidence rates which eventually reached 5–11 cm annually. A similar phenomenon occurred in the Nobi Plain (Figure 3.4).

Land subsidence has also been monitored in a number of other cities in Japan (Nagoya, Yokkaichi and others: see Figure 3.3). Over the past few years, measures to forestall land subsidence have been instituted in various places in the world, Yamamoto notes [34], for example, that recent years have been marked by the reestablishment of previously depressed pressure heads to the extent of several meters per annum in most of the subsidence-prone locations in Japan (Tokyo, Chiba, Osaka et al.). This has produced a considerable change in subsidence intensity. Moreover, certain areas have actually experienced reestablishment of former land levels, just as happened during the period of the Second World War.

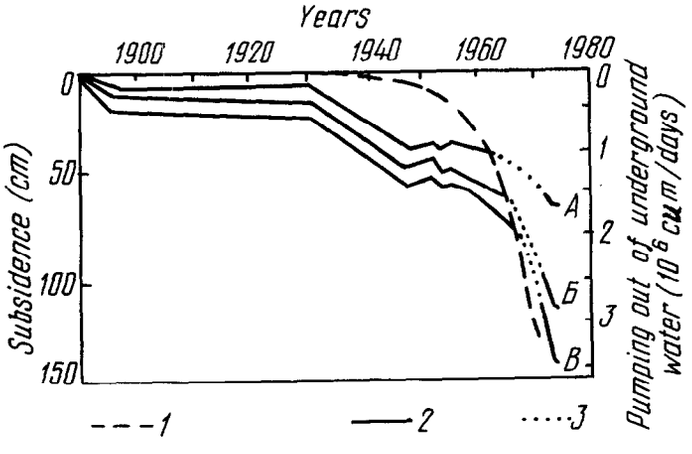


Fig. 3.4. Subsidence of the earth's surface in individual points and total pumping-out of groundwater in the Nobi Valley
1 — total value of water pumped out (units 10^6 m^3/day).
2 — subsidence of the earth's surface in cm.
3 — periods with an absence of observations.
А, Б, В — reference marks.

The maximum uplift rate of 30 mm per annum was recorded in the vicinity of the city of Kawasaki in 1974–75. The reestablishment of land surface levels is a direct result of legislative controls on groundwater usage, which may be so severe as to impose a total ban on groundwater abstraction.

Land subsidence phenomena have also been noted at a number of locations in the United States.

In the state of Texas subsidence is being monitored in a number of places along the Gulf of Mexico seacoast. The subsidence areas in that state are composed of unconsolidated water-bearing sand, clay and loam of Tertiary and Quaternary age. Several levelling surveys have been undertaken here since 1905, but it was not until 1943 that any lowering of the land surface was noted. Subsequent levellings revealed that considerable subsidence had occurred at various locations.

Some of the effects of land subsidence are the uplifting of well casings, rupture of pipelines and damage to building foundations. Land subsidence in coastal areas below sea level, such as that occurring at Texas City, is a very serious matter indeed.

Land subsidence in California is likewise scattered over several geographical locations, having first been noted in the 1940s.

The phenomenon has affected considerable tracts of land as a result of petroleum extraction (at Long Beach) and the use of groundwater for irrigation (in the San Joaquin Valley). As in Japan, the geology of these areas includes sand and clay deposits capable of undergoing compaction upon the relief of pressure heads or lowering of the groundwater table.

Land irrigation in the San Joaquin Valley is largely a matter of groundwater use. The number of operating wells here runs to several thousand. Land subsidence can be as large as several meters, and the total land area with a subsidence of 0.3 m and more exceeds 1 000 km^2.

Another country where land subsidence causes considerable problems is Mexico.

Mexico City is situated in a depression surrounded by mountains. Its geological cross section is characterized by a heterogenous lithological composition of the local soil and rock, with dense interbedding of sand, gravel beds, clay, silt, limestone and other rocks of varying origin (alluvial, lacustrine, volcanic). The lacustrine clays which interstratify with water-bearing sand and gravel beds are both plastic and hydrophilic. The pumping out of water and lowering of the piezometric level entails the dewatering of the sand and clay rocks of the alluvium. The clayey interbeds undergo compaction, which in turn produces subsidence.

The heterogeneous character of the Quaternary and Miocene deposits cross sections in the Mexico City area poses the problem of nonuniform subsidence of the entire city area, which causes considerable damage to private and public buildings and properties. Macau Vilar [26] reported that the average subsidence over the 75-year period from 1880 to 1956 was 5.6 m, reaching 7 m in certain areas by 1960.

Land subsidence is also taking place in the Po River delta in Italy due to gas extraction and groundwater pumping. As is the case in other subsidence-prone locations, the geological cross section in the Po River delta constitutes a thick wedge of interbadded sand, gravel, shinge beds and clays with occasional thin interlayers of peat, reaching depths of 600 m in some places. Over the past few decades land subsidence has also been threatening Venice.

Subsidence has been a recognized phenomenon in London for many years, although not on the scale of that in Mexico City, being limited within city limits to a level drop of just 0.3 m throughout the period since 1865. Monitoring of subsidence here has taken the form of repeated high-precision levelling surveys. The smaller subsidence values noted in London can be attributed to the geological structure of the area. The upper layers of the geological section beneath London comprises London Clay and Basal Sands of Tertiary age whose thickness ranges from zero to 100 m. Lying below the clay is Chalk aquifer which supplies water to the city. The maximum drop in the piezometric level had already reached 75–90 m by 1940.

In contrast to the soft and loose clays of Mexico City, the London Clay is more highly consolidated due to considerable gravitational compression under the weight of the glaciers which covered the country in Quaternary times as well as of the subsequently eroded overlying rocks. This explains the considerable more moderate subsidence within the city limits of London when compared with the cities of Tokyo and Mexico.

Without pausing to consider examples of land subsidence in other parts of the world, these being of a character similar to those described, we shall now draw some general conclusions.

Land subsidence occurs in areas of intensive groundwater abstraction whose geological structure consists of recent and, therefore poorly compacted, sands and clays.

All researchers who have studied the phenomenon in question have concluded that land subsidence is due primarily to the dewatering and consolidation of clays.

Observations have shown that when water abstraction is reduced, with a concomitant rise of the water table, subsidence comes to a halt and the land surface begins to rise.

A review of the literature on this problem shows that subsidence processes were initially detected in the first half of this century, but have accelerated over the last twenty or thirty years. That they will continue to develop in the years to come cannot be doubted.

All of this calls for special studies when dealing with the feasibility of water supply from groundwater resources. The most detailed observations of land subsidence and salling water tables anywhere in the world are those organized in Japan [28]. Monitoring has been via specially drilled wells equipped with special-purpose instruments and recorders mounted at the well head. The latter are usually affixed to casigns which pass through the clayey soil into the underlying sandy and clayey aquifer so that the well bottom is placed in higher-density ground.

The well head is equipped with a system of levers which makes it possible to register by means of an automatic recorder the amount of lowering or rise of the casing relative to a platform mounted on a concrete foundation. A separate automatic recorder keeps track of the fluctuations of the water table in the well. The platform is periodically levelled.

The magnitude of subsidence in most countries is monitored by means of numerous repeated high-precision levellings. Forecasting of land subsidence development is a matter of considerable practical interest. The theoretical

principles and methods of such forecasting are set out in [25, 26, 31, 33, 34].

3.3.4. COLLAPSE STRUCTURES IN KARSTIC REGIONS

Groundwater use entails other undesirable side-effects, including land collapse due to karstic processes.

Sink-hole (collapse-funnel) formation in the land surface is a common occurrence in areas underlain by calcareous rocks. However, it has become a particularly serious problem over the past few years, probably due to human activities involving groundwater use.

Newton [29] noted that since 1900 over 4 000 artificially triggered sinks, subsidences and similar incidents have occurred in the state of Alabama in the US. However, most of the happened since 1950, which marked the start of the current period of intensive groundwater use. It should be noted that less than 50 sinks have arisen under natural conditions over the same period.

The size of artificially produced sinks varies. Newton [29], for example, has noted that 40 % of the sinks in Alabama have a diameter of about 5 m, with sinks whose diameter exceeds 15 m constituting about 3 %. Sinks are a particular hazard in densely populated urban areas, as well as along major railways and roads. Artificially induced sinks are of two basic types: those associated with groundwater use accompanied by a lowering of the water table, and those associated with various major building construction projects. What follows is a discussion of groundwater-use-related sinks.

Land surface deformation is especially likely to occur in areas where calcareous strata underlying a layer of loose sand and clay soil. As a rule, calcareous rocks bear traces of paleokarst, where cavities formed by previous karsting are filled with loose noncoherent deposits.

The formation of sinks due to the development of cones of depression by intensive water abstractions has been observed in the US (Alabama, Pennsulvania and other states), the USSR, the Republic of South Africa and elsewhere.

Studies of sinking phenomena have shown [17, 20, 21] that the activation of karstic processes proceeds in two ways: a) the restoration of paleokarst cavities of their contens, and b) the creation of new cavities.

The onset or activation of karstic processes is enhanced by intensive groundwater use, accompanied by an alteration of hydrogeological conditions. Water abstraction has the effect of disturbing previously established natural hydrodynamic conditions. The resulting depression cones alter the direction of groundwater flow with changes of seepage gradients, acceleration of groundwater flows, and the appearance of new groundwater feed areas.

Intensive and protracted groundwater use gives rise to new recharge sources (influent seepage of industrial waste water, as well as, in certain instances, river water at locations which had previously constituted aquifer discharge zones), lending to important changes in hydrogeochemical conditions, which often constitute hydrogeochemical anomalies. Groundwater in such circumstances exhibits above-normal concentrations of sulphates, chlorides, bicarbonate, calcium and magnesium, as well as higher oxidation potentials.

New industrial waste water recharge and inflow sites sometimes exhibit geothermal anomalies. The alteration of the hydrodynamic, hydrogeochemical and geothermal conditions, and the alteration of hydrogeological conditions in general, is precisely the cause or series of causes which are behind the activation of karstic processes, and sink formation.

Therefore, intensive groundwater abstraction may put areas composed of karstic calcareous rocks overlies by loose noncoherent rock at the mercy of karst formation and karstic processes. Early detection of an actual sink or of sink formation may mitigate eventual losses.

Planning for new water intakes should pay due attention to the possible activation karstic processes. This in turn means carrying out appropriate surveys and predicting the future course of process development.

The sinks occurring in urban areas usually appear without giving any signs of their impending appearance. However, clues in the form of karstic development are available in certain cases. Possible clues include the following:

1. Formation of cracks or patches of crumbling on the ground and in asphalt or concrete pavements.
2. Appearance of shallow hollows and puddles in previously unaffected areas, swamp formation, sinking of pillars, posts, roadway borders, fences and other structures.
3. Appearance of cracks in walls and foundations; distortion of doorways and windows.
4. Sudden turbidity of water in wells.
5. Inhibition of vegetation over smaller patches of land; uprooting or topling of trees.
6. Sudden disappearance of small surface water bodies.
7. Sound of running water under the ground or asphalt pavement, and any number of other similar signs.

The detection of such signs means that an urgent detailed inspection of the affected area must be undertaken by hydrogeologists and civil engineers with a view to forestalling possible disastrous consequences.

3.4. HYDROGEOLOGICAL FORECASTS AND THE PROTECTION OF GROUNDWATER AGAINST DEPLETION

As was shown in Section 3.2, the principal form of hydrogeological forecasting in the context of the protection of groundwater resources against depletion involves determining the reliable, long term yield of the aquifer in question. In addition it is necessary to forecast changes occurring in the water table of the aquifer as a result of the operation of wells, adits and boreholes, as well as the possible changes in surface runoff, the lowering of the piezometric surface in the case of artesian aquifers, together with any possible land deformations and the activations of exogenic process. All of this is necessary in order to make proper allowance for groundwater in the overall water resources balances, as well as to develop measures for the control and compensation of adverse consequences of groundwater abstraction. It should be noted that methods for assessing the environmental impact of groundwater abstractions are still in their infancy.

In the present study we confine our attention to the fundamental issues and methods of available groundwater resource assessment as the basic form of hydrogeological forecasting involved in protection against depletion.

As the term "available groundwater resource" implies, its assessment consists in the determination of the potential output of a well for a given drawdown, or in the prediction of drawdown values for given outputs.

It is necessary to demonstrate the optimisation of groundwater use by a suitable system of wells for a given abstraction rate over a specified design period, or else over an indefinite time. Such determinations are dependent on the maintenance of a specified groundwater quality suitable to use needs throughout the entire period, whether limited or indefinite.

It may be seen from this that available groundwater resource assessment involves dealing with the following basic problems:

1. Determining the possible abstraction rate for a given drawdown and a specified use regime.

2. Choosing the most technically and economically viable type of intake installation (i.e. wells, adits, etc.; number of intakes, intake system configuration, spacing between intakes, individual intake discharge rates, maximum depth of pumping water levels).

3. Demonstrating that groundwater quality will correspond to specified requirements throughout the use period (when sources or factors capable of altering groundwater quality are present).

4. Determining drawdowns in the cone of depression.

Available groundwater resource assessment in the USSR and in a number of other countries involves two principal phases:

— local assessment of water supply for particular user needs and for irrigating particular land areas;

— assessment covering major hydrogeological regions as part of the planning in connection with prospecting operations and water resources use and protection feasibility studies.

Assessments of available groundwater resources are conducted with reference to the "most reasonable water intake installation" concept. In cases where recharge is built into groundwater use plans and projects, available resource assessment involves allowance for such measures.

In instances where the principal source of available resources takes the form of natural (including elastic) aquifer storage and where water intake operation will not be uniform, the design use period must be determined. This period is usually stipulated by the user (project) authority. If the appropriate bodies do not specify such a period and if the project is intended for use over an undefined period, that period is conventionally set at 25 years for calculation purposes. Such a use period exceeds the wear-and-tear period of any intake and is sufficiently long to make possible the use of long term operation data to reassess the storage, or else the securing of alternative and/or additional water supplies to satisfy the needs of the prospective user or users.

Assessing available groundwater resources involves setting an acceptable value for the drawndown at intakes. This value is one which depends on hydrogeological, technical, economic and ecological factors. The permissible decline must be established in such a way that the residual water column in the well ensures both attainment of the design output and normal pump operation. In the case of deep aquifers it is also necessary to allow for the maximum head against which the available pumping equipment can operate.

The permissible drawndown must make allowance for possible changes in the natural conditions (dewatering of overlying aquifers leading to vegetation death, etc.) and the need to protect the environment.

There now exists a fairly complete array of scientific methods for assessing available groundwater resources both locally and regionally. Relevant studies have been carried out in many countries. Their results have yielded numerous published articles and monographs concerned both with methodology and with findings for particular sites and regions.

Available groundwater resource assessment is performed mainly by the hydrodynamic method with the aid of mathematical models which describe the groundwater flow towards water intakes under various natural conditions. Mathematical simulation on analog, digital and hybrid computers is an important part of such work. Where hydrogeological conditions are complicated, the assessment is made by hydraulic, balance, and hydrogeological analog methods.

However, that due to the complexity of hydrogeological conditions and insufficient understanding of certain processes controlling available groundwater resources (such as the processes of leekage through low-permeability strata, the release of water from low permeability layers upon a decrease in pressure, groundwater flow in "dual porosity" systems, changes in groundwater recharge patterns over a scale of several years, etc.), the degree of reliability of hydrogeological forecasts in groundwater resource assessment sometimes turns out to be quite low, a fact which may lead to incorrect estimates of the practicality of developing the resources. This is why there is a pressing need for more theoretical and experimental research into available groundwater resources assessment.

Hydrogeological findings built up by analyzing experience in groundwater use and by supplementary hydrogeological investigations makes it possible to refine available resource estimates. In this connection it is useful to employ continuously operational, regional mathematical models, which make it possible to regulate the groundwater use regime with allowance for the extent of groundwater/surface water connection.

Some of the methods used for assessing available groundwater storage are considered in greater detail below.

3.4.1. THE HYDRODYNAMIC METHOD

The assessment of available groundwater resources by the hydrodynamic method comprises the design of water intakes under specified initial and boundary conditions and aquifer parameters. Hydrodynamic methods may be subdivided into analytic and computer types. The latter provide simulations using analog, digital and hybrid computers.

Assessment by the hydrodynamic method is carried out in the following sequence:

1. The results of previously completed surveys are used to establish and identify the principal geological structures of the area being assessed, the bedding conditions and extension of aquifers, low-permeability and effectively impermeable strata, the chief sources of available groundwater storage, the hydrogeological parameters and their variation in plan and in cross section, and regional groundwater quality variations. The reliability of estimates of the available groundwater resources and hydrological parameters is established at the same time.

2. A hydrogeological map is constructed to show the groundwater flow systems and boundaries, the conditions at the boundaries of the area under assessment, and the variation in hydraulic parameter between the aquifers and low-permeability strata.

The hydrogeology map shows the vertical cross-sectional structure of the groundwater flow systems defined by the presence of one or more aquifers and by the character of the interbedded aquicludes. It also gives in plan the outlines of the flow field. These are related to changes in lithological composition, aquifer thickness, faulting, contacts between aquifers and surface watercourses and water bodies, etc. Zones characterized by different permeability and storativity are also identified. The conditions at the aquifer boundary (impermeable boundaries, boundaries with constant or variable hydraulic heads) are also indicated. It must be borne in mind that conditions may vary along a single boundary. An example of this is the frequent occurrence of springs at the feather edge of an aquifer.

3. Analysis of the hydrology map yields another map which takes into account only the principal factors defining the available groundwater resource taking into account the extent to which the deductions are backed up by actual observations. Assessment of the available resource in a signle deposit may involve compiling several theoretical maps, each of which shows the reliability of the various sources of data on which the assessment is based.

Passage from natural to design conditions involves mapping the following elements:

a) the geometry of the aquifer systems in plan and in cross section with curvilinear boundary segments in plan view being replaced by straight lines and the multilayered structure of the aquifer being replaced by a one-, two-, or three-layer representation derived by combining aquifers.

b) the groundwater flow pattern with three-dimensional flows being reduced to flows of two dimensions or even one dimension by neglecting the vertical components of flow;

c) the aquifer boundaries conditions;

d) the transmissivity and storativity of the aquifers. In this case the variations of parameters are used to establish the possibility of representing an hetereogenous layer by one which is homogeneous, through the application of effective design parameters. Another alternative is zonation of the territory on the basis of aquifer parameters.

4. The design chart adopted is then used as a basis for choosing the method of computation (i.e. for selecting an analytical method in preference to a computer method or vice versa) and design equations (if an analytical method in fact is used).

Analytical methods for evaluating available resources are best employed when dealing with relatively simple hydrogeological conditions (relatively uniform transmissivity and storativity, straight aquifer boundaries, invariant boundary conditions) in which case they provide solutions to practical problems with sufficient accuracy. Such conditions are typical of thick, unconsolidated aquifers in platform-type artesian basins, outwash fans, rows, segments of river valleys characterized by good interconnection between surface and groundwater and where the groundwater is drawn from loose alluvial deposits, groundwater deposits situated in sand massifs, etc.

When it comes to complex hydrogeological conditions with marked nonuniformity of hydrogeological parameters, complicated aquifer boundaries, substandard groundwater quality, recharge sources which vary with time, and the presence of several interconnected aquifers, it is best to rely on mathematical simulation methods.

The use of mathematical simulation techniques is a highly effective tool for the regional-level assessment of available resources in areas with large numbers of operational and/or planned water intakes, and also when circumstances call for the consideration of a large number of options in the location and rate of pumping of wells.

In regions where wells are already in operation, mathematical simulation makes it possible to obtain a better understanding of boundary conditions and hydrological parameters of aquifers through the solution of inverse problems. In addition, mathematical simulation methods afford a means for the qualitative assessment of individual available resources with greater precision than that attainable by analytical methods.

5. The required water intake output and possible well discharge rates are used to determine the total number of wells and their configuration and spacing, making allowance for the boundaries of the area investigated. The above recommendations are then used to compute the design use lifetime and the permissible drawdown. The drawdowns for each intake are computed for the chosen system, and the resulting values are then compared with those considered permissible. If the design drawdown does not exceed the permissible value, then the presence of an available resource can be considered as having been established. It is advisable in this instance to carry out an additional computation in order to determine the maximum possible discharge rate for the given permissible drawdown. Such computations are performed either with allowance for the actual positions and numbers of wells, or when a large number of well are to be accounted for, by combining these into so-called generalized systems "large diameter wells".

6. The presence of groundwater of substandard composition in the area under assessment necessitates predicting the movement of the substandard water both in plan and in cross section. Another requirement is the determination of the resultant concentration, in the water, of elements whose concentrations must not exceed established standards.

Should the content of one or another component or components exceeds permissible normal values, then the available resource computed without allowance for possible quality changes must be cut down to values ensuring the preservation of a specified water quality.

As may be concluded from the foregoing discussion, hydrodynamic methods (both analytical and computer-based) are simply a matter of applying appropriate solutions to mathematical equations. Their most important and delicate phase is the schematization of natural conditions. It must be borne in mind that the computation scheme used to assess usable groundwater storage will entail certain assumptions based mainly on an imperfect understanding of the true natural setting. This is why the high precision afforded by hydrodynamic computations does not in itself guarantee the reliability of usable groundwater storage assessments. The obvious implication is that establishing the validity of identification of principal groundwater storage recharge sources and of hydrogeological parameter variations is a very important step. Differing degrees of validity must be reflected in terms of differing computational values, which must then be considered when selecting water intake facility design options.

In those instances where the initial design scheme in hydrodynamic computations makes allowance for additional recharge (leakage from overlying aquifers, lateral

groundwater flows, reduction of natural discharge, surface flow influent conditions, influent seepage of irrigation water), it is necessary to link water intake computations with estimates of the probability of usable storage being maintained with time, given the overall water resources available. Such probability estimates make it possible to justify the retention (or alteration) of the assumed boundary conditions for the design period of the abstraction.

Since groundwater intake systems usually consist of a large number of wells, the formulas presented below apply only to drawndowns in the "large diameter wells" which may be used to represent real well systems. Furthermore, because usable storage must be computed for long periods of time, the formulas apply to lengthy periods following pump startup.

All of the formulas given below apply to artesian aquifers. When dealing with water intakes operating in unconfined and/or confined/unconfined aquifers, the computational relations must be adapted by making the following substitutions:

$$2mS = H^2 - h^2 \text{ for phreatic (unconfined) aquifers, and} \quad (3.3)$$

$$2mS = h(2H_p - m) - h^2 \text{ for artesian-phreatic (confined/unconfined) aquifers,}$$

where m is the thickness of the artesian aquifer, S the drawdown in the artesian aquifer, H the thickness of the phreatic aquifer, h the height of the residual water column in the phreatic or artesian-phreatic aquifer, and H_p the initial pressure measured from the foot of the artesian aquifer.

The recommended conversion for artesian-phreatic aquifers is a strict one in the case of steadystate conditions. The same conversion is approximate, even though entirely acceptable for practical computations, in the case of nonsteadystate motion. It can be used when $\frac{at}{R_0^2} \geqslant 3$, where

a is the coefficient of pressure conductance, in m^2/day

t is the design use period, in days;

R_0 is the radius of the "large diameter well", in m.

The "large diameter well" radius R_0 for various well system configurations is given by the following relations:

$$R_0 = 0.2l \text{ for a row of wells arranged in a straight line;} \quad (3.4)$$

$$R_0 = 0.1\rho \text{ for a square or rectangular system;} \quad (3.5)$$

$$R_0 = R \text{ for a ring-shaped system,} \quad (3.6)$$

where l is the row length for a straight-line system, in m;

ρ is the perimeter of the square or rectangle in the case of such a system, in m;

R is the radius of the ring along whose circumference the wells are placed in the case of a ring-shaped system, in m.

In computing water intakes schematized in the form of generalized systems ("large diameter wells"), the total drawdown can be determined from the formula

$$S = S_{on} + S_{ск} \text{ or } S = S_{ad} + S_w \quad (3.7)$$

where S_{ad} is the drawdown due to the operation of the well system and depends both on the system type and on the aquifer boundary condition; (ad — aquifer drop)

S_w is the supplementary decline in the well which depends on the well configuration within the system, degree of penetration of wells, and the discharge rate of each well.

The relations for determining S_{ad} given below apply to typical conditions for single-layer aquifers and complexes in an infinite stratum and by a stratum delimited by a single contour with a constant pressure head at its boundary (the case most frequently encountered in practice).

Relations for other boundary conditions are given in [1, 2, 3].

For an infinite aquifer we have

$$S_{ad} = \frac{Q_{sum}}{2\pi km} \cdot \ln \frac{R_{red}}{R_0} \quad (3.8)$$

where Q_{sum} is the total well discharge rate, in m^3/day;

km is the transmissivity of the aquifer, in m^2/day;

R_{red} is the reduced radius of influence.

The value of the reduced radius of influence can be determined from the formula

$$R_{red} = 1.5\sqrt{at} \quad (3.9)$$

Calculations of S_d using formula (3.6) can be carried out for

$\frac{at}{R_0^2} \geqslant 25$ for a straight-line system,

$\frac{at}{R_0^2} \geqslant 4$ for a square or rectangular system, and

$\frac{at}{R_0^2} \geqslant 3.5$ for a ring-shaped system.

The case of a semifinite aquifer with a constant pressure head at its boundary can be dealt with as follows.

Water intakes in such instances are most frequently arranged in straight-line rows parallel to the river edge. Depending on the ratio of the water intake length to the distance to the constant-head contour (with allowance for additional resistance), the computations can be carried out on the basis of the following recommended formulas:

a) for $\frac{Z + \Delta Z}{l} \geqslant 0.5$

$$S_{ad} = \frac{Q_{sum}}{2\pi km} \cdot \ln \frac{10\,(Z+\Delta Z)}{l} \quad (3.10)$$

b) for $0{,}5 > \frac{Z+\Delta Z}{l} > 0.1$:

$$S_{ad} = \frac{Q_{sum}}{2\pi km}\left[\ln \sqrt{\left[\frac{4(Z+\Delta Z)}{l}\right]^2 + 1} + \frac{4(Z+\Delta Z)}{l} \operatorname{arctg} \frac{l}{4(Z+\Delta Z)}\right] \quad (3.11)$$

c) for $\frac{Z + \Delta Z}{l} \leqslant 0.1$

$$S_{ad} = \frac{Q_{sum}}{km} \cdot \frac{Z + \Delta Z}{l} \quad (3.12)$$

Formula (3.10) is used for calculating the drop in a "large diameter well", formula (3.11) for calculating the same quantity for a straight-line row of limited length, and formula (3.12) for a straight-line row of infinite lenght.

The symbols used in the above formulas have the following meanings:

Z is the distance from the well row to the constant-pressure-head contour;

ΔZ is the additional resistance exerted by bed deposits, with allowance for the incompleteness of the river edge and the presence of silt deposition in the channel, in m.

Formulas (3.8)–(3.10) are valied for times which are given by the experssions

$$\text{for 3.10 } t \geqslant \frac{10(Z+\Delta Z)^2}{a} \quad (3.13)$$

$$\text{for 3.11 } t \geqslant \frac{5(Z+\Delta Z)^2}{a} \quad (3.14)$$

$$\text{for 3.12 } t \geqslant \frac{2-2{,}5(Z+\Delta Z)^2}{a} \quad (3.15)$$

Formulas (3.10)–(3.12) are also suitable for approximate calculations for water intakes situated along the feather adges of aquifer marked by spring flows, provided the total discharge rate of the water intake facility does not exceed the total discharge rate of the springs.

In those instances where the water intakes are arranged in a ring-shaped or rectangular (square) configuration, the appropriate calculation formula is also (3.10), with R_0 as defined in (3.5) and (3.6). In using formula (3.10) it is necessary to ensure fulfillment of the following conditions:

$Z > 2.5R_0$ for a straight-line row system; (3.16)

$Z > 1.6R_0$ for a square or rectangular system; (3.17)

$Z > R_0$ for a ring-shaped system. (3.18)

In formulas (3.16)–(3.18) Z is the distance between the boundary and the water intake well laying closest to it.

Formulas (3.10)–(3.12) can be used only in the case of a backwater seepage condition for which the value of the drawdown at the river edge does not exceed the permissible drop (S_{per}) as determined from the relation

$$S_{per} \leqslant H_0 + m_0 \quad (3.19)$$

where H_0 is the depth of the water in the river and m_0 the silt deposit thickness.

Calculating the drawdown in a well

The additional drop S_w in a well which enters into formula (3.7) is given by the following which applies to all configurations:

$$S_w = \frac{Q}{2\pi km}\left[\ln \frac{r_{red}}{r_0} + 0.5\xi\right] \quad (3.20)$$

where Q is the discharge rate of the well, in m^3/day;

r_{red} is the reduced radius of the conventional domain of influence of the given well, in m;

r_0 is the radius, in m;

ξ is the magnitude of the seepage resistance with allowance for incomplete penetration of the well.

The reduced radius of the national zone of influence of a given well is given by the relations

$$r_{red} = \frac{\sigma}{2\pi} \text{ for straight-line and ring-shaped systems,} \quad (3.21)$$

where σ is the spacing between wells, and

$$r_{red} = 0.47\sqrt{F_0/\pi} \text{ for a rectangular or square system,} \quad (3.22)$$

where F_0 is the area of the domain bounded by the lines passing through the midpoints of straight lines running between neighboring wells.

3.4.2. THE HYDRAULIC METHOD

Usable groundwater storage assessment by the hydraulic method consists of determining the theoretical discharge rate of a water intake or the predicted levels in wells, using empirical data obtained by direct experiment and by taking into account the influence of various factors which determine the operating regime of the water intake.

Hydraulic methods are employed in usable storage assessment in three different ways:

a) for estimating the groundwater drawndown in a well for a given discharge rate using curves of discharge rate versus drawdown for steadystate conditions;

b) for determining water table cutoffs in the calculation of interacting wells (similarly for steadystate conditions);

c) for determining the drawdown by the end of the design period useful life of an operating well given a constant discharge rate based on empirical relationships between discharge rates and drawdowns.

In contract to hydrodynamic methods, whose application involves choosing the design relations to be used for determining the basic hydrogeological parameters on the basis of schematized boundary conditions, in hydraulic methods both the form of a function and its principal parameters are estimated on the basis of experimental data. This permits generalized allowance for factors such as aquifer inhomogeneity, the presence of various boundaries, resistances in the zone surrounding the well, departures from a linear seepage law. It is advisable to make wide use of hydraulic methods when dealing with complex hydrogeological conditions characterized by a highly inhomogeneous aquifer and when available storage formation sources are not easy to identify. Given steadystate seepage conditions, hydraulic methods of assessment have been used extensively in surveying deposits situated in river valleys where the aquifer is associated with nonuniformly fissured or karstic rocks. With nonsteadystate seepage this method can be used to assess usable groundwater storage in smaller deposits associated with tectonic faulting. In such circumstances the hydraulic method comprises the experimental determination of the rate of drawdown as a function of the discharge rate, for a given rate of water uptake.

Hydraulic methods are not suitable for establishing the probability of usable groundwater storage recharge, inasmuch as their empirical relationships do not include quantities characterising the flow balance.

Therefore, these methods can be applied only in combination with hydrodynamic or balance methods. One exception to this general rule is the use of the hydraulic method for nonsteadystate conditions, since the use of either hydrodynamic or balance methods under such complicated conditions is practically impossible. Determining the optimal duration of experiments becomes especially important under these circumstances. The experiment must run for a sufficient time that the resulting experimental functions reflect the influence of all identified aquifer boundaries. However, even with a proper choice of experimental time one cannot be completely certain that factors which remained undetected in the course of the prospecting period will not come into play, at some future time thus altering the form of the drawdown versus time. This means that forecasts developed by the method in question are approximate.

Dealing with the first of these problems (that of forecasting the table drop in a well for a given discharge rate) consists in constructing and then extrapolating curves of well discharge plotted as functions of drawdown in a given well. The procedure in question is described in detail in the literature.

The second problem (that of determining table cutoffs in wells operating in tandem) is handled by increasing or reducing the cutoffs obtained during pumping by a factor equalling that by which the design discharge rates of the wells producing cutoff is greater or smaller than the discharge rates of the same wells obtained with pumpout. The cutoff is calculated by means of the relationship

$$\Delta S_i^{pr} = \frac{Q_{pr}}{Q_{ex}} \Delta S_i^{ex} \qquad (3.23)$$

where ΔS_i^{ex} and ΔS_i^{pr} are, respectively the table cutoffs obtained in the course of experimental pumpout and those predicted for the design discharge rate;

Q_{ex} and Q_{pr} are, respectively, the experimental pumpout and predicted discharge rates of the cutoff-producing well.

In the case of water intake systems consisting of several wells, the drawdown in each well is defined as the sum of the drop in the well under consideration due to its operation as an isolated well (as determined from the discharge rate curve or by direct experiment) and the table cutoffs due to each interacting well as determined by formula (3.23). If additional wells are introduced into the water intake system, the table cutoffs occasioned by their operation should be computed with allowance for their distance from the well for which computations are made. For this the data obtained in the course of test pumping during the prospecting stage are used to plot the graph of $\Delta S_i = f(\log r)$ for a given discharge rate, or else a composite graph showing $\frac{\Delta S_i}{Q_i} = f(\log r)$. These graphs can then be used to determine the cutoff values at various distances from the perturbing wells for given discharge rates.

Dealing with the third problem (the prediction of the drawdown by the end of the design period for an operating well with a constant discharge rate) consists in analysing the data obtained during test or operational pumping. The analysis consists of determining the empirical time variation law and establishing the extent to which the law reflects actual conditions. In ascertaining the law of water level variation with time it is advisable to plot graphs in the coordinates $S - \log t$; $S - \sqrt{t}$; $S - t$ which correspond to the conditions of an infinite (semifinite) strip-like or closed stratum, carrying out calculations on the basis of the relationship which is closest to being rectilinear. In those cases where all three relationships depart significantly from linearity it is good practice to select a more general power function of the type $S - t^C$, where $C < 1$. In order to determine C one must construct several graphs using several possible values of C.

One version of the hydraulic method which is used in the case of nonsteadystate conditions is known as the "method of generalized parameters". In using this method real, inhomogeneous strata with a complicated boundary configuration are replaced for predictive purposes by some conventional infinite homogeneous stratum whose parameters have been determined on the basis of the extreme segments of the $S - \log t$ graphs (provided that the latter are rectilinear). This method affords a means of predicting drawdowns even for discharges in excees of those studied in the course of test or operational pumping with the proviso that the water uptake system remains unchanged.

3.4.3. THE BALANCE METHOD

Usable groundwater storage assessment by the balance method consists of determining the groundwater discharge rate which is attainable with water intakes within a given region over a given period of operations as a result of the contributions of particular sources. Each of the potential formation sources should be assessed separately, after which the individual results are summed.

With this method, the balance for a given area (region) is regarded as an entity with respect to water inflow and outflow across its boundaries. This permits determination only of the average lowering of water tables in the aquifer, and not of the drawdowns in water intake wells. Nor can balance methods be used to determine the possible output of a given well. All of this means that the balance method must be used chiefly as an adjunct to the hydrodynamic and hydraulic methods.

On the other hand, the balance method is the sole means available for establishing the role of individual sources in available groundwater formation and for ascertaining the probable availability of storages which have already been computed by other methods. This is why the use of the balance method in conjunction with other techniques of usable storage assessment is desirable in the vast majority of cases.

Balance methods have their greatest independent value in the assessment of usable groundwater storage within smaller finite structures, with high-permeability aquifers. In the course of operations the cone of depression in such locations rapidly spreads over the entire area of the structure without much difference in the drawdowns drops at the center of the given area and along its perimeter.

In addition, the balance method can be used to determine usable groundwater storage in those cases where groundwater use is to be effected by direct abstraction from springs. In such situations usable storage assessment consists of determining the variation of spring discharge rates with time. This can be done by correlation between reference-site spring flow measurements. The usable storage computed on the basis of spring flow must correspond to the discharge rates of the springs at a specified level of probability, which entails determining the spring flow which corresponds to a given level of probability. In those instances where reference-site observations have yielded only short time series, forecasts can be made on the basis of an analogous spring for which longer time series are available, or else using atmospheric precipitation data, provided the necessary correlations between the discharge rates of the springs and those of analogous springs or atmospheric precipitation have been established in advance. It is recommended that such work be done using double and multiple correlation methods of the type used widely in the forecasting of natural groundwater regimes.

3.4.4. THE HYDROGEOLOGICAL ANALOGY METHOD

The complexity of hydrogeological conditions and the practical impossibility of assessing usable groundwater storage formation sources quantitatively render it necessary, in many instances, to make extensive use of the method of hydrogeological analogies based on the extrapolation of groundwater use regime data from areas where water intakes are already in operation to prospective areas where conditions are similar. This method is

helpful in usable groundwater storage assessment inasmuch as it permits.

1. Direct assessment of usable groundwater storage by an integral parameter which characterizes storage formation processes taken as a whole. One such possible integral parameter can be the usable storage modulus as determined from data obtained from operating analogous water intakes.

2. Determination of the individual components of usable groundwater storage by analogy with the subsequent use of balance-type computation methods.

3. Determination by analogy of individual parameters which cannot be reliably computed on the basis of prospecting data (the gravity drainage of fissured rocks, the seepage factors of low permeability deposits, etc.).

4. Correction and choice of an appropriate computational scheme.

The correct assessment of usable storage by the analogy method requires that the hydrogeological conditions and useful groundwater storage formation sources within the area under investigation be identical with those in the reference area.

Applying the analogy method depends on the presence of an reference site. In justifying the possibility of drawing an analogy between the reference and assessment sites it is necessary to compare the principal factors influencing the formation of usable groundwater storage, as well as their values (aquifer conditions, boundary conditions, aquifer lithologies, recharge conditions, potential for induced leakage, character of overburden and the composition of overburden deposits, etc.).

An analogy may be either complete or partial. With a complete analogy the identity of the hydrogeological conditions within the reference site and the assessment site must have been established with respect to all of the principal factors which determine the size of usable groundwater storage under the given set of natural conditions. In the case of a partial analogy the identity of only some of these facts need be ascertained.

Where a complete analogy is possible, usable storage assessment can be completed on the basis of the usable groundwater storage modulus. The latter is defined as the discharge rate obtainable per unit area (per unit length in the case of seepage-type water intakes operating in river valleys) of the aquifer.

The usable storage modulus is computed on the basis of data from water intakes which are already in operation. Usable groundwater storage in the area under assessment is then determined by multiplying the computed modulus by the area (length) of the zone in question. In those cases where the permissible drawdown in the area under assessment differs from that for the analogue site under fixed head conditions (e.g. in river valleys, where the principal source is surface flow), the computed value of the modulus must be adjusted by applying the correction.

$$M''_u = M_u \frac{S_{per}}{S} \qquad (3.24)$$

where M''_u is the corrected value of the usable storage modulus;

M_u is the computed value of the usable storage at the analogue water intake site for the drawdown S;

S_{per} is the permissible drawdown for the area under assessment.

A similar correction must also be applied to the value of the modulus in those cases where the permeabilities at the analogue site and the site under assessment differ.

Usable groundwater storage assessment in the case of a partial analogy is carried out by the methods already described (i.e. the hydrodynamic, balance and hydraulic methods). Their use involves the determination by analogy either of individual formation sources or of individual parameters. A third possible use of the analogy method is the correction of the computational scheme.

An important plase of usable storage assessment in such cases is the identification of the factors which can be extrapolated onto the area under assessment. The remainder of the data needed to calculate storage are established by means of the generally accepted procedures.

For example, if the lithological composition of the aquifer happens to be the same while the groundwater feed conditions differ, the water yield value can be chosen by analogy, while the natural storage and other data necessary for computations are determined in the standard way.

Thus, the use of the analogy method requires preliminary surveying with the aim of establishing the identity of available groundwater resource recharge conditions. Usable storage assessment relying on the hydrogeological analogy method is performed in the following sequence:

a) selection of a site which by virtue of its hydrogeological characteristics might serve as an analogue to the area under assessment;

b) analysis of the use regime at selected the water intake system with a view to identifying the principal sources contributing to available storage and ascertaining the factors influencing the groundwater flow towards the water intakes;

c) surveying of the territory under assessment in order to obtain evidence of either complete or partial identity of factors contributing to groundwater resources in the reference area and in the surveyed location;

d) using the findings obtained in the earlier stages to establish the form (complete or partial) of the analogy, followed by selection of a suitable method for assessing usable groundwater storage.

A detailed description of the various methods of usable groundwater storage assessment will be found in [1, 2, 3].

Chapter 4.

MEASURES FOR THE PROTECTION OF GROUNDWATER AGAINST CONTAMINATION AND DEPLETION

GENERAL

Measures for the protection of groundwater against contamination are designed to prevent contamination, to eliminate its consequences and to preserve and improve the quality of groundwater for its effective use. Because the contamination of groundwater is basically related to the contamination of surface water, the atmosphere, atmospheric precipitation and soils, the problems of protecting groundwater against contamination must be solved at the same time as that of the protection of the whole environment against pollution.

The prevention of groundwater contamination is ensured, primarily by measures of a general technical nature, among which are the creation of closed systems of industrial water supply and waste water disposal systems; the introduction of waste-free technology or with a minimum quantity of waste water and other wastes; the improvement of the purification of waste waters; the isolation of waste waters from other waters; the elimination or purification of gas and smoke emissions; limiting and monitoring the use of toxic agricultual chemicals and fertilizers; deep underground disposal of particularly harmful wastes, for which there are no economically justified methods of detoxification or elimination; the creation of special water conservation zones, with the establishment of strict rules for economic and building activities, in areas of groundwater development. Experience shows that limiting the spread of contaminants which have already entered an aquifer, as well as their extraction and elimination, constitutes a complex problem, the solution of which is both expensive and often technically difficult to achieve. With a large accumulation of contaminants in the ground and their persistence, as well as with low flow rates in many aquifers, the time necessary for the complete removal of contaminants may be measured in tens and even hundreds of years. In those cases, however, when the source of contamination is spread over a considerable area, and the aquifer thickness is great, the elimination of the contamination may be practically impossible, and this part of the aquifer, sometimes with large reserves of groundwater, may prove to be irreversibly lost to use. Therefore, measures of a preventive nature should be considered as fundamental, in order to forestall the possibility of groundwater contamination.

The legislation and state standards adopted and developed for the regulation of the normas of maximum permissible concentrations and effluent discharge contents play an important role in preventing groundwater contamination. Other important factors include the rational use of groundwater and the earth's interior; planning rules governing the location and operation of new enterprises, groundwater intakes and other installations and rules governing prospecting and exploratory work for water, oil, gas and other useful minerals.

The first stage in a system of hydrogeological preventive measures is the organization of monitoring of groundwater quality and levels, as well as monitoring of the sources of potential pollutants on groundwater.

The need for systematic monitoring of groundwater quality is due to the existence of numerous and various sources of contamination and, to the fact that, when compared with the contamination of surface water and the atmosphere, the contamination of groundwater develops and manifests itself more slowly. Over the course of many years it may pass unnoticed, and when it is discovered, by changes in the composition of water in water-intakes, springs and drainage sites of groundwater into rivers, the extent of the contamination proves to be so great that substantial efforts are required for its elimination. With early detection of groundwater contamination, the elimination of the pollution sources and their consequences requires considerably less expenditure of time and money.

The preventive measures also include the establishment of sanitary protection zones for groundwater around water-intake works, and a geological and hydrogeological supervision of the location of new works which may be potential sources of contamination, the identification and study of sources of groundwater contamination in order to prevent further spread, measures for enhancing the efficiency of irrigation systems, etc. At sites where the contamination of groundwater has already occurred, it is necessary to prevent its further spread. In order to acheive this, measures and ways of localizing centres of contamination should be developed, and, if possible, measures for their elimination and the restoration of groundwater quality.

Taking into account the high cost of some types of water analyses, the long duration and the large numbers of observations necessary to monitor the state and degree of contamination of groundwater over a large area, it is important to optimise the lists of determinants and number and frequency of observations for individual aquifers, areas and basins. Properly organized systematic

monitoring of groundwater quality will make it possible to detect the initial signs of unfavorable effects of man-made contaminating factors. On the basis of these data, an approximate forecast is also possible of the rate and distance of the spread of contaminants in an aquifer. If it is necessary to take protective measures, these same data may be taken as a basis for more detailed investigations. More complete and detailed hydrogeological investigations are carried out to substantiate measures for the protection of groundwater against contamination at planning sites for installations which may be sources of contamination, as well as on sites where measures are necessary for the protection of water-intakes and the elimination of the consequences of contamination.

4.1. ESSENTIAL HYDROGEOLOGICAL DATA FOR COMPREHENSIVE PLANNING OF THE ENGINEERING PROTECTION OF GROUNDWATER

4.1.1. ENGINEERING PROTECTION AGAINST CONTAMINATION

In planning new industrial and other installations which may be sources of groundwater contamination, comprehensive plans for the engineering protection of groundwater are developed as preventive measures. In regions where groundwater contamination has already taken place the measures are required for the prevention of the further spread of contaminants and for the elimination of the source of contamination.

The basic problem in hydrogeological planning of engineering protection at proposed industrial installations is the choice of location, which should ideally be on a site where groundwater enjoys natural protection against contamination. If such sites are absent, it is necessary to draw up a plan for the engineering protection of groundwater.

In the first stage of a three phase plan, the existing reference and archival materials about the natural conditions of the region (geological structure, hydrogeological conditions, climate, orography, etc.), the existing and planned industrial enterprises, and the water economy of the region – the reserves of groundwater and their use, rivers, the siting of water-intakes, sewage systems, etc. should be collected and studied. If necessary, a reconnaissance survey of the region should be carried out, as well as simplified explorations; drilling of individual wells, geophysical investigations, the measurement of the discharges of water in the waterways, the determination of the levels and chemical composition of ground and surface waters.

For each alternative site, the information obtained must provide answers to the following questions: what is the natural state of protection of the groundwater, is contamination of ground and surface waters possible here, which installations or bodies of water are threatened by contamination (water-intakes, rivers, lakes), what is the approximate travel time of contaminants to these installations? The rates of movement of contaminants forecast at this stage are based on considerations of the proposed siting and hydrodynamic interactions of reservoirs, water-intakes, rivers and other water facilities, using approximate values of the hydrogeological parameters of the aquifers.

At the next stage of the planning (the technical plan) detailed hydrogeological investigations are carried out at sites selected for the location of industrial waste lagoons, filtration beds, factories, and other facilities. These investigations are intended to clarify the hydrogeological conditions of the sites, to determine the main hydrogeological parameters and to calculate the infiltration losses of industrial wastes. These data must be such as to allow refirement of forecasts of the zone of spreading of wastes from the lagoon, including direction and flowrate, the evaluation of changes in the composition of groundwater during infiltration and the effect of this change on the groundwater intakes operating in the region, and on surface water. It is necessary also to estimate the volume of groundwater in the zone of influence from the waste water lagoon.

At this stage or at the third stage of planning (working drawings) hydrogeological investigations are undertaken in order to clarify technical questions connected with the planning of the individual structures which make up the pattern of engineering protection. In drawing up the program of hydrogeological research the information from the preliminary planning studies must be used, including the locations of the planned structures, indicating nearby water-intakes, waterways and bodies of water; data on the purpose and design of industrial waste reservoirs, and technical data of these installations; the quantity and composition of the industrial wastes disposed of in the reservoirs and removed from them; graphs showing changes in the level of industrial wastes in the reservoir, etc. The area of the enterprise or the reservoir should be covered by hydrogeological investigations, as well as adjacent territory up to the nearest aquifer boundaries or up to the theoretical lines within which the hydrogeological or hydrochemical influence of an enterprise may act as a potential source of contamination to ground and surface waters and water-intakes. The composition and extent of the research and investigations are determined in each specific case with respect to the composition and quantity of the waste waters which may enter the aquifer, as well as with respect to the climatic, hydrogeological, geological and hydrological conditions of the region. Because of the link between ground and surface waters, hydrological works must be carried out to describe the surface water regime and the quality of the surface water.

On sites where a source of contamination has been identified and the need for engineering protection of groundwater has been established more detailed studies may be required, which clarify the hydrogeological, hydrochemical, engineering-geological situation directly on the sites of location of the planned protective structures, to assist in the selection and planning of alternative protective measures. These include exploratory and experimental – infiltration field work, laboratory investigations, chemical analyses of water and soils, etc.

The main purposes of the additional studies carried out in the area of an established source of contamination of groundwater are: clarification of the geometry of the zone of contaminated groundwater within the aquifer,

detailed analysis of the composition and concentration of the contaminants in waste and groundwater, as well as the determination of the hydrogeological and physical – chemical parameters of the stratum, so that refined forecasts of the rate of contaminant movement can be made, and to substantiate plans for protective measures.

The determination of the hydrodynamic parameters is carried out at the same time as hydrogeological studies, which include experimental investigations (pumpings, injections), geophysical measurements and observations of the groundwater regime. For the determination of active porosity, and the true rate of movement of groundwater, tracer experiments are necessary, in which a tracer-marker is injected into the natural groundwater flow or into a disturbed flow (with pumping, injection, and the "doublet" method of pumping and injection) and the rate of its progress in the aquifer is tracked.

The aquifer and under – and overlying beds, as well as contiguous aquifers should be described in terms of their lithological, chemical, and granulometric compositions, and also with respect to the physical properties and conditions.

In investigating the chemical composition of waste waters and solid wastes, which cause contamination of groundwater, all the main sanitary and hygienic indicators must be determined quantitatively, as well as the components peculiar to the manufacturing process which gave rise to the wastes. Toxic components, whose levels in drinking and other water supplies are controlled by maximum permissible concentrations should reviewed special attention.

The natural quality of the groundwater should be characterized initially by the main sanitary and hygienic indicators. In a number of cases it is also advisable to establish the presence and concentration (background, natural) of the specific components which are present in waste waters and wastes.

All these data are used to establish, with respect to the installation under study, the need for experimental determinations of the parameters of physico-chemical interaction of contaminated water with the aquifer and to determine the methodology for investigations. In studying waste and groundwaters, in addition to the ionic composition, it is also desirable to identify the forms in which particular components are found. In many cases a comparison of the chemical composition of rocks, waste and groundwaters make it possible to form an opinion about the relative part played by particular processes of physico-chemical interaction, which will affect the migration of contaminants when waste waters enter the aquifer. The propagation of the main contaminating components of the waste waters of soda production, containing chlorides of calcium, magnesium and sodium, will be mainly affected by convective transfer, complicated by gravitational effects due to the high mineralization of these waste waters; the absorption of the contaminating components by the rocks in this case will be of secondary importance. Given a more complex composition of the contaminants, preliminary laboratory experiments should be carried out, from the results of which conclusions may be drawn on the possible physico-chemical interaction processes, and the main processes identified, which effect the propagation of contaminants in the water-bearing stratum. Following this, a decision is taken on the need for a more detailed study under laboratory and field conditions of the main physico-chemical process. The appropriate experiments are then carried out to study the laws and to determine the parameters of the process which may affect migration.

Analogue and numerical modelling techniques may be used to determine the necessary quantity of water pumped out, the duration of pumping and other parameters of protective installations (drainage wells, flow barriers, etc.).

4.1.2. ENGINEERING PROTECTION AGAINST DEPLETION

The potential of artificially-induced replenishment of supplies of groundwater for protection against depletion depends on a number of factors: geological and hydrogeological, geomorphological, climatic, the presence of a source of replenishment, etc. Geological and hydrogeological factors include the conditions of formation of groundwater at the site, the thickness and hydraulic properties of the aquifer intended to serve as a storage unit, its hydraulic nature, the three dimensional conditions, information on the natural drainage of the storage site, data on the direction and rate of subsurface flow and the intensity of water exchange under natural conditions; the thickness, structure, grains size composition and permeability of the zone of aeration, the quality of groundwater, and its regime. Data must be adduced on the interrelationship of ground and surface waters (for unconfined aquifers), the permeability and storativity characteristics of water-enclosing rocks, and the specific yield. In the case of induced replenishment system planned for river valleys or alluvial sediment, information would be needed on the relative permeability of stream bed deposits, including their clay content. With a thickness of the aquifer replenished of more than 20–25 m, information must be obtained about the vertical variations in permeability.

When evaluating the conditions for solving the question of the artificial recharge of groundwater for protection against depletion the first priority must be given to determining the thickness, grain size destribution and filtration properties of low permeability deposits, as well as to estimating the characteristics of the zone of aeration when that is composed of loose porous soils. It is essential to have data on the thickness, grain size distribution, structure and hydraulic properties of the rocks, the composition and quantity of solable minerals in individual beds and throughout the whole thickness of the unsaturated zone. When the thickness of the zone of aeration is more than 10 m, vertical variations of characteristics should be studied. In areas of marked facies change lateral variations should also be examined. The infiltration rate of water injected into boreholes must be adduced. The boundary conditions of the aquifer must be properly studied including the direction and rate of subsurface flow, the drainage of the site and the intensity of water exchange under natural conditions. A failure to take these factors into account may lead to a considerable part of the recharged water avoiding the water-intake works, and the efficiency of the collecting installation will be low. The groundwater regime of the site under investigation must be studied – the annual and longer term fluctuations in the groundwater levels, fluctuations in temperature, chemical composition, and sanitary state of groundwater.

It is also necessary to have data on groundwater quality, which make it possible to approach the question of the requirements for the quality of the recharge water.

As a rule, groundwater must satisfy the requirements for water which is abstracted (or will be abstracted) by a water-intake. For drinking purposes it must satisfy the Soviet (GOST 2874–73), America (PHS) or some other drinking water standard. In addition, water must be examined for the presence of harmful micro-components, as well as for its gaseous composition, in order reliably to predict the possibility of the formation of various sediments, films, slimes – iron and other mineral, colloidal, bacterial, etc., – the mixing of "natural" groundwater with the recharge water.

When evaluating the conditions for replenishment, considerable importance should be attached to the geomorphological description of the topography of the site, which must be given after appropriate investigations have been carried out. Broken terrain makes it difficult to install infiltration basins, canals, etc., or to bring water to the pumping wells through open canals, flumes, etc. At the same time, slopes within the limits of from 0.01 to 0.05 contribute to increased infiltration.

In determining the potential for artificial recharge of groundwater, climatic factors must be taken into account: the frequency and intensity of atmospheric precipitation, local evapotranspiration soil moisture deficits, the direction and velocity of the wind, the thickness, density and stability of snow cover, the depth of freezing of the soil and the duration of the period of freezing, the duration of the periods with a temperature below – 15 °C and information about extremely low temperatures. The climatic description should be based on data provided by the closest weather stations.

The description of the source for artificial groundwater recharge must include the features of its hydrological regime for a quantitative assessment of the potential availability for recharge, because sources are often the waters from rivers, lakes, and reservoirs. To analyze the regime of the source of recharge, the existing stations of the hydro-meteorological network must be used, and in their absence, independent hydrometrical work must be organized in conjunction with hydrochemical, hydrothermal and sanitary observations. The minimum duration of such observations both hydrological and water-balance must be such that lengthening the observed series by analogy is possible.

In investigating the variability of streamflow, the following must be determined: normal annual flow, annual flow over a long period, flow distribution within the year, minimum flow, maximum flow of spring floods and rainfall floods in the low-water period; flow probability, including flood flow; the duration of the period between floods in years of varying flow probability. If flow ceases at certain periods (drying up, freezing up) data must be obtained on the duration of the period of absence of flow and its probability. If river or other surface waters (in canals) are abstracted for other needs on the way to the recharge site of infiltration, data must be obtained on the quantity of the residual flow and its distribution in time and probability [34].

The quantitative characteristics of a surface source of recharge (river) are determined, on the whole, by the requirements placed on this source as if it were to satisfy the consumer by water taken from the source directly. If the waters of lakes or reservoirs, are used as a source of rechage the following must be determined: the volume of water in the lake (reservoir) and the water-balance between outputs and inputs (inflow of tributary rivers, atmospheric precipitation, evaporation, removal (discharge) of water, etc.). When considering the effluent waters of mines, condensation installations, purified industrial and household wastes, etc., it is necessary to have data on the stability or the nature of the periodicity of these sources of recharge.

An important factor which determines the potential for using artificial recharge of groundwater for protection against depletion is the quantity of the water of the recharge source. The requirements for water entering infiltration installations, adopted in the USSR, are set forth in section 4.7.1. In summary the requirements for the quality of stored water are determined: a) by the quality of "natural" groundwater, b) by the properties of the rocks of the aquifer which is to be recharged and c) by the national economic purposes for which the water will be used. Appropriate investigations should be carried out on the basis of these three main considerations. In studying the quality of the water of the recharge source it is necessary to bear in mind the potential for contamination of the water by some specific contaminants, for which no norms are set by the general standards for drinking water (GOST 2874–73, PH, etc.). Therefore, the question of the possibility of contamination of the surface waters by such potential contaminants must be considered and appropriate analyses of the water should be carried out.

4.2. IDENTIFICATION AND STUDY OF SOURCE OF CONTAMINATION

The contamination of groundwater is usually discovered as a result of exploratory work, by the deterioration of water quality in operating wells and in springs, or as a result of the study of the composition of groundwater carried out for other purposes.

The organization of systematic monitoring of the quality, and the level of contamination, of groundwater, in particular on the sites of possible sources of contamination, contributes to earlier discovery of sources of contamination. Regular study of the quality and the degree of contamination, of groundwater on the territory of the USSR has been carried out since the 1960's. The description of cases of contamination in the USA is given in a summary [59].

The methodology of identifying and studying sources of groundwater contamination is dealt with in the works of F.M. Bochever, N.N. Lapshin, A.E. Oradovskaia [7]; V.M. Goldberg [11, 12]; E.L. Minkin [23]; V.A. Mironenko et al. [25]. Presumptions about the presence of a source of contamination of groundwater arise if in one or more water-observation stations, analyses of the water show high values, when compared with background values of total mineralization or of the concentrations of individual components, the appearance of new substances, not characteristic of the natural groundwater of the aquifer under observation a deterioration of the bacterial composition.

The state of contamination of the soil layer and of the zone of aeration may also serve as an indirect indicator of the deterioration of groundwater quality. After confirmation of the anomalous indicators of groundwater quality, by several successive check analyses of water from the same and the nearest water observation stations, the advisability and urgency of a further more detailed study of the source of contamination should be established, because this may require the drilling of new, sometimes deep, exploratory and observation wells, and also the carrying out of other investigatory work.

In the solution of this issue we proceed from the following considerations: 1) the importance of the contaminated aquifer from the point of view of its existing or planned use for the purposes of water supply, irrigation, etc.; 2) the expected scale of groundwater contamination, which is determined by the size, nature and length of action of the presumed source of contamination; 3) the danger of an unfavorable effect of the contaminated aquifer on the quality of the groundwater of other aquifers of the region and on the quality of the surface waters.

The evaluation of these factors is based on data from earlier regional and detailed hydrogeological investigations, as well as on the basis of information on the water supply and the water consumption of the region concerned, etc. After the need for studying the source of groundwater contamination discovered is ascertained, a program of hydrogeological research and investigations is drawn up. The tasks under this program include: 1) the identification of the extent of the contamination, i.e., its area and the depth of the water-bearing stratum, the composition of the contaminants, the distribution of their concentrations in the area; 2) the study of the causes responsible for the contamination, with an identification of the priority to be assigned to sources of contamination; 3) an evaluation of the degree of danger of further development of the contamination of the water-bearing stratum; 4) the hydrogeological justification of measures to eliminate the consequences of groundwater contamination and to protect individual installations located nearby – groundwater intakes, rivers, etc.

The identification of the area of groundwater contamination begins with a hydrogeological survey of the site, accompanied by the selection and analysis of samples of surface, waste and groundwaters from all existing water stations – water-intakes, observation points, exploratory wells, rivers, brooks, lakes, canals, tailing dumps and sludge storage tanks, settling tanks, overflows of waste waters, etc. Taking account of these data a plan is then drawn up for hydrogeological research, which usually includes the drilling of exploratory wells for the clarification of the geological succession and the hydrogeological structure of the region, the determination of the direction of groundwater movement and the selection and investigation of groundwater samples from different depth in the aquifer. Useful results may be obtained from geophysical investigations of wells, to identify the heterogeneity of water-bearing rocks, as well as by areal geophysical surveys to delimit the confines of contaminated zones, which may be reorganised by the quantity of total mineralization, temperature and other properties. The composition and scope of this work depends not only on measurements of a wide range of geological, hydrogeological and geochemical parameters but also by the size, length of operation and nature of the source of contamination. Therefore, the examination and study of the source of contamination must precede the development of a program of hydrogeological research. If the source of contamination is presumed to be an industrial site, the surveys should deal with the following questions: 1) the quantity of surface quantity of surface and groundwater consumed by the establishment, the presence of a circulating water supply, the volume and chemical composition of total effluents and he effluents of individual production processes and the places of disposal of effluents; 2) the organization of the transportation and storage of water-soluble raw materials, industrial products and wastes the hydrogeological conditions of these sites, the possibility of rainfall dissolving wastes and raw materials and the entry of solutions into groundwater; the chemical composition of the air in the area of the industrial site, and of industrial waste storage tanks and evaporators; 3) the state of equipment, installations, sewerage systems, drainage gutters and other facilities from the point of view of the possibility of discovering sites of leakage of technological solutions and industrial wastes; 4) the existence of effluent treatment works, their characteristics, capacity, working conditions and degree of purification of waste water; 5) the sites and amounts of waste water disposal (in rivers, ponds, storage tanks, on filter beds, etc.) and sites for the storage of finely ground or pulverized production wastes (hydraulic ash dumps, slime storage tanks, etc.).

If the source of contamination is presumed to be a reservoir, storage tank or other receptacle for industrial wastes, the purpose, area, configuration of the installation and accessory facilities must be identified: along with their design (height of dikes, depth, existence and type of screen, etc.); the time of the start of operations and the arrangements for the disposal of waste or effluents; fluctuations of the water level in the receptacle as compared with water levels in the closest river-intakes, waterways and water reservoirs; the chemical composition of effluents fed into the receptacle; the amount of leakage losses (design – as estimated, and actual – as observed) and the existence of observation wells and the results of the observations of the propagation of industrial wastes in ground and surface waters near to the receptacle. The effect of leakage from the industrial waste tank on groundwater must also be evaluated.

The locations for exploratory wells should be chosen to take account of direction of groundwater flow, extending from the sectors of the aquifer known to be "dirty", closest to the source of contamination, towards the distant "clean" sectors, where the concentrations of contaminants are lower.

In selecting the depth of the exploratory wells, one should bear in mind the possibility of density stratification of the groundwater. The contaminated, often highly mineralized and therefore denser solutions may sink to the lower part of the aquifer at their point of infiltration, and subsequently migrate mainly through the lower, but sometimes the middle, part of the aquifer.

Cases are also possible of the accumulation of contaminants of low density chiefly in the upper part of the aquifer. The scale of pollution of the groundwater, singling out the zones which are notable for the intensity of their contamination (see section 2.1.1.), is evaluated from the results of the exploration and the study of the composition of the groundwater.

Data from the study of the source of contamination are then used to assessment the degree of danger of a spread of the contaminants in the direction of groundwater flow into rivers, into drains, mine workings, the nearest wells, and springs. The initial solution of this problem is obtained on the basis of an approximate forecast of the rate and time of movement of the contaminants through the aquifer using rough values of the main hydrodynamic parameters and the parameters of mass transfer (the coefficients of permeability and active porosity). The processes of physico-chemical interaction are not normally considered at this stage because of the absence of the corresponding parameters. Density differentiation may be taken into account at this stage, using

data on the density of the infiltrating waste and groundwaters.

In simple hydrogeological conditions, when it is possible to reduce the aquifer to a single or 2–3 layers homogenous types, and when it is also possible to schematize the boundary conditions, the configuration of the source of contamination, etc., the forecast is made by means of analytical calculations. In complex hydrogeological conditions, it is possible to use the analytical – tabular method and methods of analogue and numerical modeling. On the basis of the forecast of the rate and time of movement of contaminants, the danger of a deterioration in water quality in the water-intake wells and in other installations under consideration is estimated and the need is determined for the contaminant or elimination of the source of contamination, as well as the need for the protection of individual installations located near to the source of contamination.

4.3. PREVENTIVE MEASURES FOR THE PROTECTION OF GROUNDWATER AGAINST CONTAMINATION

4.3.1. THE PLACING OF OBSERVATION WELLS FOR THE MONITORING OF GROUNDWATER CONTAMINATION

The study of groundwater contamination and estimating the scale of such contamination are based on observations of the groundwater regime and of changes in its quality. Therefore, the most important task is the establishment of an observation network covering major industrial and agricultural installations, with whose activities groundwater contamination is connected, and large centralized water-intakes where a danger of contamination exists. The site of a fresh groundwater intake is the basic subject of protection.

As has been pointed out (section 2.1.3.), groundwater contamination to a considerable extent is brought about by the contamination of the natural environment – the atmosphere and atmospheric precipitation, surface waters and the soil. Therefore, monitoring groundwater contamination must be coordinated with the monitoring of pollution of the natural environment as a whole.

The purposes of a specialized network of groundwater quality protection are: 1) early discovery of contamination in groundwater (this is especially important at the sites of water-intake works); 2) the study of the dynamics of the zone of groundwater contamination in time and in space, i.e., the determination of the rate and direction of the propagation of contamination and the size of the zone of groundwater contamination; 3) the study of the movement of contaminants in groundwater taking account of the physico-chemical processes of interaction of these substances with the groundwater and the rocks and of the natural processes of self-purification; 4) the adjustment of forecasts of the propagation of contaminated water in the aquifer based on the results of observations of their actual movement, leading to improvements of forecasting methodology.

The justification of special protective measures is based or observations of changes in the hydro-chemical regime of groundwater and of forecasts of the development of the contamination zone.

Monitoring the level of contamination of groundwater and its development constitutes a hydrogeological water-conservation measure of a preventive nature.

The network of observation wells be designed to take account of the following factors: the location, nature and size of the sources of contamination; the configuration of the groundwater contamination zone; the structure of the aquifer (thickness, heterogeneity) and its boundary conditions; the direction of the natural movement of the groundwater; the rate of movement of the contaminated groundwater; the location of water-intakes and the ways contaminated or naturally substandard waters flow into them.

The number and arrangement of observation wells should be determined by the nature and rate of movement of the contaminated water and may require additional wells as time passes.

Because it is necessary to detect changes in the hydrochemical situation, one of the major factors governing the arrangement of observation wells is the rate of movement of the contaminated water through the aquifer. The time and rate of progress of contaminated water towards water-intakes are determined by the formulae given in section 2.5., while the progress of the contaminated water from the sources of contamination is given in section 2.2.1.

The observation wells should monitor the position of the boundary of contaminated or substandard water. For this purpose, some of the observation wells are set up in the contaminated water zone, while other observation wells are installed in the uncontaminated aquifer, in order to detect the transitional zone.

The observation wells should be placed in such a way as to provide an areal picture of contamination.

The observation network in the vicinity of industrial installations is built up starting from the source of contamination. In the area of a water-intake, the network should progress from the boundary of the zone of contaminated or substandard water towards the water-intake.

Since, in the area of industrial installations, the brunt of the contamination entering from the earth's surface is borne by the unconfined groundwater, the observation wells are fitted out chiefly to tap the unconfined groundwater aquifer. In constructing observation wells into deeper aquifers it is essential carefully to isolate the unconfined groundwater aquifer in the wells, in order to prevent cross-contamination between aquifers.

The analysis of water samples should be not only for the standard indicators, but also for contaminats characteristic of the region.

The observation network should include both wells located in the zone of influence of the source of contamination and background wells. The latter should be located at sites where the groundwater is unaffected by the process of contamination and is representative of the aquifer.

The equipping of observation wells depends on the hydrogeological conditions of the aquifer (thickness, heterogeneity of the section) and the properties of the contaminants. With a thick aquifer, it is advisable to mount a cluster of two or three observation wells equipped with filters in their upper, middle and lower parts. If the contaminants possess a longer specific gravity than the groundwater (for example, oil and oil products) they will mostly be associated with the upper part of the aquifer. In this case the observation wells should be set up to tap the upper half of the aquifer. If, however, the aquifer is contaminated by highly mineralized water with a specific gravity greater than that of groundwater, observation wells should be equipped also for the lower part of the aquifer, where the accumulation of a layer of highly contaminated water is possible.

In organizing a specialized network, it is essential to make maximum use of already existing observation boreholes, wells and other water-observation stations. For this purpose an analysis should be made of the status of the existing water-regime network and its suitability for the solution of groundwater protection problems.

At water-intake sites, observation wells are placed in 2 or 3 lines passing from the region of contaminated or naturally substandard groundwater towards the water-intake works. One of the lines must pass through the centre of the water-intake. The wells on the lines are placed close to the boundary of contaminated water and at a distance from it of 100–200 m. The direction and rate of movement of the contaminated water are identified from the results of observations during the initial period of operation and, on this basis, proposals are made for the further build-up of the observation network and the placing of new observation wells.

The initial number of observation wells between the water-intake and the contaminated groundwater zone is determined by the size of the water-intake works (for example, by the length of the linear row of wells) and by the extent of the contaminated water zone. In each specific case, the quantity of observation wells and their distance from the boundary of the contaminated water are established on the basis of the real conditions and the results of the forecasting evaluations of the time and rate of progress of the contaminated water.

The observation wells are placed in an operating aquifer, and in adjacent aquifers, if induced flow of contaminated or naturally substandard water is possible.

In placing observation wells adjacent to industrial or agricultural source of pollution and, particularly at the sites of waste storage reservoirs and filter beds, it is necessary to bear the following in mind: 1) the general direction and rate of movement of the groundwater; 2) the heterogeneity of the rocks and the position of the parts of the aquifer with the greatest water conductivity; 3) the location of the nearest groundwater intakes and their distance from the source of contamination; 4) the size and shape of the waste storage reservoirs; 5) the approximate arrangement of the industrial reticulation (pipelines, collectors), which also may be sources of groundwater contamination; 6) the distance from the waste storage reservoirs to the river; 7) the presence of geological faults, zones of fracturing and lithological windows.

The observation wells must be placed, firstly, along the direction of the natural movement of the groundwater, towards the nearest groundwater intakes. The presence of natural groundwater flow brings about irregular or non-uniform displacement of the contaminants in the aquifer, particularly the restricted propagation of contaminants upstream, and accelerated propagation downstream.

In the area of the waste storage reservoir (or system of reservoirs), several lines of observation wells are set up passing through the centre of the reservoir and along its periphery. The number of lines depends on the size of the waste reservoir.

The lines are laid out around the perimeter of the waste reservoir and are oriented parallel to the groundwater flow and in the direction of the nearest water-intakes.

2 to 4 wells are placed on each line. The distance between the wells is determined by the rate of spread of the contaminated water in the aquifer. If the wells are far from each other and from the reservoir, contamination of groundwater may not be noted for a long time. If, however, the wells are placed very near to the reservoir and at short distances from each other, the contaminated water may, after a comparatively short interval of time, overtake the observation wells and further observations of them will no longer describe the displacement of the front of contaminated water. Therefore, the choice of the distance between wells must be related to the rates of movement of the contaminated water in the aquifer. According to data in the literature, the rate of displacement of the front of contamination in the natural flow of groundwater amounts, on the average, to 30–50 m per year. Therefore, one may consider approximately that the distance of the well nearest to the waste reservoir on the line located along the groundwater flow or towards the water-intake must be not more than 50–100 m; subsequent wells must be placed at distances of 100–200 m from each other.

Provision should also be made for observation wells upstream and in lateral directions.

The further build-up of the observation network should be based on the results of the first stage of observations of the propagation of contaminated water.

Along with the setting up of the observation network on the sites of sources of contamination and at water-intakes, it is necessary to mount a specialized observation network within inhabited areas (cities), since the sources of groundwater contamination are not only industrial installations but also the urban area as a whole.

Within an urban area (city) there may be various sources of contamination: such as industrial enterprises, municipal garbage dumps, filling stations, fuel stores, chemical products stores, highways, burial grounds, etc. The entire city as a whole exercises a substantial effect on the chemical composition of groundwater and its temperature regime.

Against the background of the general anomaly of the chemical composition and temperature of groundwater affecting the municipal area as a whole, individual "super-anomalous" sites stand out, involving intensive local sources of contamination (chemical and thermal).

To justify the placing of observation wells for the purpose of monitoring the level of groundwater contamination within the limits of an inhabited area, it is first of all essential to identify the actual and potential sources of groundwater contamination and to draw up a pattern for the siting of the wells.

Observation wells on the area of an inhabited place should be arranged into several regional lines, oriented along the groundwater flow and passing through the entire city. Regional observation lines should be drawn through the main municipal sources of groundwater contamination and the municipal water-intakes, so as to take account of the cones of depression. To cover the sources of contamination not included in the regional lines, additional short longitudinal and transverse lines are laid out.

The system of observation wells at an industrial enterprise located within the city limits is an integral part of the municipal observation network.

In the same way, specialized observation networks for the monitoring of the groundwater contamination in urban areas, industrial enterprises and water-intake works should be considered as integral parts of the general system of groundwater monitoring.

4.3.2. EVALUATING THE STATE OF GROUNDWATER PROTECTION

The state of protection of groundwater against contamination is primarily a function of the presence of surface deposits of low permeability which hinder the percolution of contaminants from the surface of the earth into groundwater.

The state of groundwater protection should be taken into account when planning and siting industrial, agricultural and other installations affecting groundwater and in the justification of water-conservation measures for the protection of groundwater and water-intakes against contamination.

The state of protection of groundwater depends on many factors which may be broken down into three groups: natural, man-made, and physico-chemical.

The main natural factors include:

– the presence in the geological succession of low permeability deposits;

– the depth of occurrence of groundwater;

– the thickness, lithology and hydraulic properties of rocks (principally, strata of low permeability) overlying groundwater;

– the sorptive capacity of the rocks;

– the correlation of aquifer levels.

The man-made factors include the conditions under which contaminants are found on the surface (storage of wastes in accumulations, slurry ponds, disposal trenches, and in other industrial tanks, disposal of waste waters on filter beds, irrigation by waste waters, etc.) and the way in which the contaminants migrate to groundwater.

The physico-chemical factors include the specific properties of contaminants, their migration ability, sorption coefficients, chemical stability or time of decay of the contaminant and the interaction of the contaminants with the rocks and with the groundwater.

It is obvious that the greater the thickness of low permeability deposits overlying an aquifer, the lower their filtration properties and the greater the depth of the groundwater, the more favorable are the natural factors to their state of protection, and the higher the probability of protection of the groundwater in relation to any types of contaminants and the conditions of their penetration into groundwater from the earth's surface.

Therefore, in evaluating the state of groundwater protection one should proceed first of all from the natural factors, the most important of which is the presence of low permeability deposits.

Low permeability deposits (sandy loams, loams, clays, etc.) may be defined as those with coefficients of less than 0.1 m/daily.

It is advisable to carry out the evaluation of the state of groundwater protection in two stages. At the first stage most attention should be paid to the study of the natural factors of the state of protection. At the second stage it is essential to take due account also of the other factors – the man-made conditions and the specific characteristics of the contaminants connected with these installations.

The evaluation of the conditions of protection is comparative in nature.

Unconfined Groundwater

The evaluation of the state of protection of unconfined groundwater may be qualitative and quantitative. The qualitative evaluation is based on the natural factors, the quantitative – on the natural and man-made factors.

The qualitative evaluation of the conditions of unconfined groundwater protection is generally made on the basis of 4 indicators:

1) the depth of occurrence of the unconfined groundwater level (or, what is the same, the thickness of the zone of aeration);
2) the lithology of the rocks of the zone of aeration;
3) the thickness of the deposits of low permeability in the section of the zone of aeration;
4) the hydraulic properties of the rocks of the zone of aeration, particularly, of the low permeability deposits.

In the majority of cases the hydraulic properties of the rocks of the zone of aeration are unknown. Therefore, the qualitative evaluation of the state of protection of the unconfined groundwater is based on the first three indicators, with the hydraulic properties being estimated from lithological data. It is obvious that the greater the thickness of the zone of aeration, the greater the thickness of the deposits of low permeability and the lower their filtration properties, the better are the conditions for the protection of unconfined groundwater. Analysis has shown that what is decisive for groundwater protection is the presence in the zone of aeration of low permeability deposits. The thickness of the zone of aeration, when composed of highly permeable sediments, may also affect the state of protection of unconfined groundwater, but to a considerably lesser extent than the thickness of the low permeability deposits.

A qualitative evaluation of the conditions of unconfined groundwater protection may be carried out on the basis of a total number of points proceeding from the depth of occurrence of the unconfined groundwater level (or, what is the same, of the thickness of the zone of aeration), the thickness of the low permeability and their lithology, through which the filtration properties of these deposits are taken into account.

The total amount of points is determined from Table 4.1., which shows the gradations of the depths of the unconfined groundwater level, the thicknesses of the deposits of low permeability and their generalized lithology.

Table 4.1

DEPTH OF UNCONFINED GROUNDWATER LEVEL, THICKNESSES AND LITHOLOGY OF LOW PERMEABILITY DEPOSITS OF THE ZONE OF AERATION, AND NUMBERS OF POINTS CORRESPONDING TO THEM

Depth of unconfined groundwater level (H), m					Thickness (m_0) and lithology (a, b, c) of the slightly permeable layer											
1	2	3	4	5	6			7			8			9		
$H \leqslant 10$	$10 < H \leqslant 20$	$20 < H \leqslant 30$	$30 < H \leqslant 40$	$H > 40$	$m_0 \leqslant 2$			$2 < m_0 \leqslant 4$			$4 < m_0 \leqslant 6$			$6 < m_0 \leqslant 8$		
					a	b	c	a	b	c	a	b	c	a	b	c
1	2	3	4	5	1	1	2	2	3	4	3	4	6	4	6	8

Thickness (m_0) and lithology (a, b, c) of the slightly permeable layer (Continuation)

10			11			12			13			14			15			16		
$8 < m_0 \leqslant 10$			$10 < m_0 \leqslant 12$			$12 < m_0 \leqslant 14$			$14 < m_0 \leqslant 16$			$16 < m_0 \leqslant 18$			$18 < m_0 \leqslant 20$			$m_0 > 20$		
a	b	c	a	b	c	a	b	c	a	b	c	a	b	c	a	b	c	a	b	c
5	7	10	6	9	12	7	10	14	8	12	16	9	13	18	10	15	20	12	18	24

Five gradations of depths (H) of occurrence of unconfined groundwater are identified (columns 1–5 of the table): < 10 m, 10–20 m, 30–40 m, > 40 m. One point corresponds to the first gradation with minimum depth of occurrence of unconfined groundwater level (H < 10 m), two points – to the 2nd gradation, three points to the 3rd, four points to the 4th and 5 points to the 5th. These points are established on the premise that the whole zone of aeration is composed of sandy rocks.

The thicknesses of the deposits of low permeability (m_0) are subdivided into eleven gradations (columns 6–16): to 2 m, 4–6 m, ..., 18–20 m and above 20 m.

Three groups are identified by lithology:

Group 'A' – Sandy loams, light loams (coefficient of filtration – 0.1–0.01 m daily)

Group 'B' – Loams, sandy clays (coefficient of filtration – 0.01–0.001 m daily)

Group 'C' – Heavy loams, clays (coefficient of filtration – less than 0.001 m daily).

The total number of points determined by application of Table 4.1 allows six categories of unconfined groundwater protection to be identified (Table 4.2)

Table 4.2

CATEGORIES OF PROTECTION OF UNCONFINED GROUNDWATER
(according to total numbers of points)

Categories of conditions of protection	I	II	III	IV	V	VI
Total number of points (Σ)	$\leqslant 5$	$5 < \Sigma \leqslant 10$	$10 < \Sigma \leqslant 15$	$15 < \Sigma \leqslant 20$	$20 < \Sigma \leqslant 25$	> 25

The least favourable protection is the conditions corresponding to category I, the most favourable those corresponding to category VI.

The categories indicated (I–VI) provide only comparative evaluations of the state of protection.

Example 1: The unconfined groundwater lies at depth H = 7 m (1 point, column 1); in the zone of aeration there is a stratum of sandy loams and light loams (index 'A') with a thickness of m_0 = 3 m (2 points, column 7a). The total number of points is 3, the category of protection I.

Example 2: The unconfined groundwater occurs at depth H = 14 m (2 points, column 2); in the section of the zone of aeration there occurs a stratum of clays (index 'C') with a thickness m_0 = 5 m (6 points, column 8c). The total number of points is 8, the category of the state of protection II.

For the construction of a map of groundwater protection, working maps of the depths of groundwater and of the thicknesses of the low permeability deposits with an indication of generalized data about their lithology (indices A, B & C) are drawn up. The total number of points is evaluated at each borehole, well, test hole etc., or by breaking up the area into a grid and evaluating the number of points within each grid square. Then contours based on the total number of points (every

5 points) are drawn and, zones are identified which are characterized by categories of protection I–VI.

Zones of different depths of occurrence of unconfined groundwater, different thicknesses of low permeability deposits (taking account of their generalized lithology) and of the categories of conditions of protection may be plotted on the resulting map, to show the conditions of groundwater protection. The categories of protection may be designated by colours: categories I and II – by shades of yellow; categories III and IV – by shades of orange; categories V and VI – by shades of brown; the deepness of the colour increases with the category of the state of protection.

On medium and large scale maps, major sources of groundwater contamination (large industrial enterprises, surface waste reservoirs and disposal ponds, filtration beds, fields irrigated by waste waters, large stock raising units, etc.) and groundwater intakes may also be plotted. These installations are noted by conventional signs. Zones of karst development and river valleys are indicated on the map.

A quantitative evaluation of the degree of groundwater protection may be made on the basis of a determination of the transit time for contaminants filtering from the surface to the groundwater level (unconfined or confined). This indicator (time) depends not only on the natural factors, but also on man-made conditions at the surface. Comparing the time of filtration with the time of decay of the contaminant under study, one may form an opinion on the state of protection of the groundwater against penetration of the given contaminant.

The time is determined for 2 systems: filtration from surface waste water tanks with a constant level and filtration during the disposal of waste waters on the earth's surface (filter beds, fields for irrigation by waste waters) with constant discharge. The calculation of the time of filtration for the above systems is made according to formula (2.9)–(2.11).

As examples for the calculation of the time of movement of the contaminated water from the surface of the earth, the following are taken:

$$H_0 = 5 \text{ m and } q = 0.03 \text{ m/daily}$$

These values correspond to the real and widespread production conditions [11, 12].

Proceeding from the time it takes the waste waters to reach the unconfined groundwater level, the following categories of protection are identified:

I	$t \leqslant 10$ days	IV	$100 \text{ days} < t \leqslant 200$ days
II	$10 \text{ days} < t \leqslant 50$ days	V	$200 \text{ days} < t \leqslant 400$ days
III	$50 \text{ days} < t \leqslant 100$ days	VI	$t > 400$ days

The higher the category, i.e., the greater the time, the better the conditions of protection.

On the whole, unconfined groundwater is unprotected or partly protected. Unconfined groundwater may prove to be protected only in relation to individual types of contaminants, which are distinguished by a comparatively short lifetime and rapid decay. Such include a number of bacterial contaminants, individual pesticides, short-lived radioactive substances, etc.

On a map of the quantitative evaluation of unconfined groundwater protection the depths of occurrence of the unconfined groundwater, the thickness of the deposits of low permeability, the lithology of the rocks of the zone of aeration, filtration properties and, plotted, as a resultant indicator, the categories of protection. In addition, the same signs are plotted as appear on the map of the qualitative evaluation of groundwater protection.

Confined Groundwater

The evaluation of the conditions of protection of confined groundwater is carried out for the first confined aquifer in use from the surface. The contaminants may penetrate into this confined aquifer from above, from the overlying aquifer whose water, for one reason or another, is contaminated. In many cases, the overlying aquifer is an unconfined groundwater horizon.

Evaluation of the conditions of protection of confined groundwater can generally be carried out on the basis of the following indicators: 1) the thickness of the aquiclude; 2) the lithology of the aquiclude; 3) the filtration and migration properties of the aquiclude and 4) the correlation of the levels of the aquifer under investigation and the overlying aquifer.

In practice, the filtration and migration parameters of the aquiclude are unknown. Therefore, the evaluation of the state of protection is given on the basis of 3 indicators: thickness, lithology, correlation of the levels. The entry of contaminants from the overlying aquifer into the confined aquifer through an aquiclude may occur as a result of overflow (convective transfer) or molecular diffusion or the joint action of these processes. The major role in the entry of the contaminants belongs to convective transfer, the scale of which exceeds by many times purely diffusive transfer.

The indicator of the correlation of levels is important for an evaluation of the state of protection of the confined aquifer, which determines the mechanism of entry of the contaminants into the confined aquifer.

If the level of the overlying aquifer is designated by H_1, that of the underlying confined aquifer by H_2, for the evaluation of the state of protection of the confined aquifer, three cases may be identified: 1) $H_2 > H_1$, 2) $H_2 \approx H_1$ and 3) $H_2 < H_1$.

In the first one, when the levels of the confined aquifer are higher than the levels of the overlying aquifer, the confined aquifer considered may be deemed protected with great probability for any types of contaminants as long as that condition continues. Given such a correlation of levels when the vertical gradient is directed from the bottom upwards (positive gradient), the entry of contaminants from the overlying aquifer into the underlying by the convective method is impossible. The movement of contaminants through the aquiclude into the confined aquifer under these conditions may occur only through the mechanism of molecular diffusion. However, the diffusion transfer will be held up since the gradient of concentration, which brings about the process of diffusion, and the vertical gradient of head are set in opposite directions.

The hydrodynamic situation is less favourable when the levels of the underlying and overlying aquifers coincide. In this case, the vertical gradient of head directed from the bottom upwards is absent, the gradient which hinders the movement of the contaminated water from above. But, at the same time, the gradient of head directed from above downwards is absent. The entry of contaminants into the confined aquifer from the overlying aquifer occurs as a result of diffusion, which under these conditions is not retarded by an upwards head gradient.

The most unfavourable hydrodynamic conditions are those where the levels of the confined aquifer are lower than the levels of the overlying aquifer.

In this case, a vertical head gradient downwards occurs and a hydrodynamic conditions favour arises for the flow of contaminated water from the overlying aquifer into the underlying aquifer. The transfer of contaminants through the aquiclude will occur due to the joint action of convection and diffusion, directed the same way.

Therefore, the correlation of levels influences the potential for the entry of contaminants into a confined aquifer and is of great importance for the conditions of its protection. However, the correlation of levels may be variable in time, and, accordingly, the conditions of protection of the confined groundwater aquifer also vary.

Maps of the state of protection of confined groundwater should be drawn to show the situation for a certain period of time, and the correlation of levels on the site investigated should be monitored over time. Clusters of observation wells should be installed on the confined aquifer under investigation and in the overlying aquifer in order to monitor the correlation of groundwater levels between the aquifers.

When evaluating the conditions of protection of a confined aquifer, the lithology of the aquiclude has definite importance. In the zone of fresh groundwater development, aquicludes are principally composed of clays, sometimes marls and calcareous deposits. Clayey aquicludes are considered the most secure. The screening properties of a clayey aquiclude will differ depending on the type of clay minerals; an aquiclude in which montmorillonite clays predominate is particularly secure.

On the basis of the combination of two indicators (the thickness of the aquiclude m_0 and the correlation of the levels H_2 – of the confined aquifer investigated and H_1 – of the overlying aquifer), the following main groups of confined groundwater protection may be singled out:

I. Protected – confined groundwater covered by an aquiclude constant in area with $m_0 > 10$ m, $H_2 > H_1$;

II. Conditionally protected – confined groundwater covered by an aquiclude constant in area without any disruption in continuity with:

a) $5\ \text{m} \leqslant m_0 < 10$ m and $H_2 > H_1$;

b) $m_0 > 10$ m, $H_2 \leqslant H_1$;

III. Unprotected – a) an aquiclude of small thickness $m_0 < 5$ m, and $H_2 \leqslant H_1$, b) an aquiclude not constant in area, with disruptions of continuity (lithological "windows", zones of intensive fracturing breaks), $H_2 \gtrless H_1$.

In group I, the state of protection of confined groundwater is ensured by the great thickness of the aquiclude and by hydrodynamic conditions under which overflow of contaminated groundwater from above is impossible. Within group I, the degree of protection will not be identical, depending on the thickness of the aquiclude and the differences in head levels; it will be higher the greater m_0 and ΔH.

It is especially emphasized that even when $H_2 > H_1$, but when the aquiclude is not constant in area and there are disruptions in its continuity, a confined aquifer cannot be considered as protected.

In addition to the criteria mentioned in the definition of group III, confined groundwater is also unprotected in the following cases: in river valleys when the aquiclude is cut through by the river in karstic regions, if the aquiclude is captured by karstic processes; in regions with complicated tectonic conditions in a zone of active water exchange and intensive neo-tectonic movements.

Since the singling out of the categories "protected" and partly "conditionally protected" includes the hydrodynamic indicator $H_2 \geqslant H_1$, this condition, as already been pointed out, should be constantly monitored.

On the map of qualitative evaluations of the conditions of protection of confined groundwater, the following are represented: the thickness of the aquiclude, its lithology, the difference of head levels of the protective groups I, II and III. The map shows: water-intakes, aquifers in use and under investigation; sites with confirmed supplies of groundwater sources of contamination; sites of contamination of aquifers; direction of groundwater movement.

4.3.3. LOCATING INDUSTRIAL INSTALLATIONS IN ORDER TO PROTECT GROUNDWATER AGAINST CONTAMINATION

The most significant sources of groundwater contamination are connected, as has already been pointed out, with large industrial enterprises, as well as with agricultural installations (stock-raising units, sectors irrigated by waste waters, agricultural areas treated with pesticides, etc.). Therefore, in locating industrial and agricultural installations, it is essential to strive to prevent their negative impact on groundwater or to minimise it.

It follows from the experience of studying the development of groundwater contamination that in many cases it is connected with an unfortunate location of industrial enterprises in the hydrogeological sense, when, in the choice of the site of industrial premises, no account was taken of the possibility of the systematic or accidental entry of contaminated waste or process waters into the zone of aeration and the aquifer. Thus, many enterprises which are sources of infiltration of contaminated waste waters are located in river valleys, on bottom lands and alluvial terraces, which, in choosing a site for industrial premises, appeared convenient because of the location of the industrial installation close both to the source of water supply (river, water-intake from alluvial water-bearing deposits), and close to the place of disposal of waste waters to the river.

The unfavourable consequences connected with the contamination of groundwater were either unknown at that time, or were not taken into account.

At the present time, when the scale of the deterioration of the quality of groundwater and its socio-ecological effects which have already occurred and which are continuing have become clear, the planning of the use of sites in rapidly developing regions should serve as an effective means for reducing this unfavourable impact on the quality of groundwater used for water supply. The choice of a site for one, or of a group of new industrial installations, the choice of the optimal alternatives for the development and location of industry in individual regions should be carried out on the mandatory condition of taking due account of the problems of protecting groundwater against contamination. This applies particularly to the choice of a location for sludge reservoirs, tailing dumps, non-ferrous metal refining works, oil refineries, plants for the production of chemical fertilizers, and other installations having a large quantity of waste waters in the process cycle and which potentially threaten groundwater contamination. Hydrogeological control of the locating of new industrial installations must be carried out on the basis of a comprehensive analysis of the hydrogeological conditions of the region as a whole and the site earmarked

for the location of an industrial installation, the analysis and evaluation of the significance of ground and surface waters of the region for water supply, irrigation and similar purposes, as well as the technological features of the planned enterprises.

Issues concerning the protection of the environment and the groundwater must be solved before at the stage of preparation of the plan for the industrial or agricultural installation. In planning such an installation, an evaluation should be carried out of its impact on the natural environment. In respect of groundwater, this evaluation includes: the determination of the quantity of discharge of waste waters filtering from the surface of the earth and entering the aquifer; the determination of the size of the zone of contamination of the aquifer at various points in time; the determination of the quantity of transport of contaminants by groundwater into rivers and the effect of this transport on changes in the chemical composition of the surface water; the determination of the possibility and time of the drawing up of contaminated groundwater to the nearest water-intakes. On the basis of such an assessment of the impact of the installation on groundwater, provision should be made in the planned enterprise for special protective measures.

In the same way, an integral and mandatory part of the general plan for the enterprise should be a plan for an observation network of wells in order to monitor the state of groundwater. The observation network should cover the installation as a whole and, above all, the main sources of contamination: sludge lagoons, ponds for the disposal of waste waters, tailing dumps, salt and ash dumps, garbage dumps, filtration beds, agricultural areas irrigated by waste waters and treated by pesticides, etc.

The main subjects of protection are fresh groundwater and in particular water-intakes for domestic and drinking purposes.

In choosing the location of an industrial or agricultural installation.

The prime objective is to prevent the contamination of operating or planned water-intakes, and there should be minimal (or zero) contamination of fresh groundwater in the aquifer.

The locating of an installation is carried out bearing in mind the geological and hydrogeological condition of the region, the natural state of protection of groundwater, the location of operating and planned groundwater intakes and the size of their sanitary protection zones. The industrial or agricultural installation must be located on sites which are characterized by relatively favourable natural groundwater protection conditions down groundwater gradient from the operating (planned) water-intake, or removed from it in such a way that the installation cannot be captured by the zone of influence of the water-intake or its zone of sanitary protection. The location of the installation higher upstream from the water-intake is inadmissible. On the contraty, the water intake should be located higher upstream than the industrial installation.

The distance from the water-intake works to the source of contamination should be more than the distance in the same direction from the water-intake to the boundary of the second belt of the sanitary protection zone of the water-intake (see section 4.4).

If the second belt of the sanitary protection zone coincides with the boundary of the water-intake recharge zone the distance (x_1) from the water-intake to the source of contamination located downstream should be more then the length of the recharge zone (see section 2.5). The quantity x_1 is estimated approximately, by the following formulae: for a single well or a group of well arranged in a concentrated way

$$x_1 > \frac{Q}{2\pi hki_e}; \tag{4.1}$$

for a linear row of wells

$$x_1 > \frac{l}{\pi} \operatorname{Arth} \frac{Q}{2lhki_e}, \tag{4.2}$$

where Q — is the yield of a single well or group of wells, Q_0 — is the yield of a single well of the row, l — is the distance between wells in the row, h — is the thickness of the aquifer, k — the coefficient of permeability of water-bearing rocks, i_e — is the gradient of natural groundwater flow.

One should avoid the construction of industrial or agricultural installations and, in particular, of surface waste reservoirs in groundwater recharge zones (at the top of rock slides or slopewash cones or benches, on the slopes of river terraces, on sites of highly fractured and karstified rocks), especially if the groundwater in these places is used for the water supply.

An important part in the choice of a site for the location of an industrial installation is played by the evaluation of the degree of natural protection of groundwater on the side under consideration against contamination from the surface. The methodology of evaluating the conditions of groundwater protection is set forth in detail above (section 4.3.2). Therefore, we consider below specific questions of the state of protection of groundwater in connection with the choice of a location for an industrial installation as a potential source of groundwater contamination. Because the main impact of man-made factors is usually borne by unconfined groundwater, industrial enterprises should not be located on sites where groundwater aquifers are developed which are suitable for use in the water supply. Also, contamination of groundwaters should not be allowed in those cases where the aquifer is not used directly for the water supply, but, being connected with deeper confined and unconfined aquifers, may serve as a cause of their contamination. This is exacerbated by the lowering of head in the deeper aquifer leading to induced recharge from overlying aquifers. But, on the whole, the deeplying confined groundwater possesses a great degree of protection, with the exception, of course, of cases of direct disposal of contaminants through wells.

One may count on the natural state of protection of groundwater against surface contaminants in cases where the aquifer is isolated from the surface contaminated water or if this water, in the process of infiltration, is fully purified of the contaminants contained in it.

In the first case, the contaminated surface water and wastes either do not enter the water-bearing stratum at all, or if they do reach it, then it is over such long periods of time (several tens or hundreds of years), that it far exceeds the usual periods of longterm use of groundwater. Such conditions are ensured when the roof of the aquifer is composed of unpermeable or low permeable rocks of great thickness, regional in extent, and not disrupted by lithological windows, fractures tectonic disruption. Contaminants cannot enter the water-bearing stratum if the latter possesses hydraulic heads above the ground level, the levels of water in waste water reservoirs and the levels of groundwater in contaminated aquifers, so

long as the hydraulic heads in the stratum under consideration are not lowered subsequently as a result of water withdrawal. Such a state of natural protection is inherent in deep-lying confined waters in many artesian basins. For unconfined groundwater, realiable natural isolation from surface waters is, generally unrealizable; surface wastes usually penetrate unconfined groundwater in a comparatively short time, which does not exceed tens of days or several months. Only under special circumstances do infiltrating surface waters fail to reach the level of unconfined groundwater; this may occur, for example, under conditions of a dry climate with periodic infiltration of a small amount of surface water through a thick zone of aeration, composed of low permeability rocks.

In the second case, where the surface waters entering the aquifer are neutralized and purified of contaminants as a result of filtration through the thickness of the overlying rocks, the natural state of protection of groundwater is linked with processes of "self-purification". In using this term, an analogy is usually implied with the self-purification of river, lake and other surface waters; however, this analogy is formal in nature, inasmuch as the essence and intensity of the natural processes leading to the purification of contaminated surface and groundwaters are different.

In the aquifer, where groundwater usually has a low temperature, there is a comparatively small supply of dissolved oxygen and a limited quantity of micro-organisms. Oxydation, decomposition, the mineralization of organic contaminants and similar processes are not so well developed as in surface waters. There is also substantially less potential for the dilution of contaminated waters. Only in the soil layer and the uppermost part of the zone of aeration do an abundance of micro-organisms and the presence of air afford the possibility of partial transformation of chemical and biological contaminants. In the zone of aeration and the water-bearing stratum, self-purification takes place in different ways for particular types of contaminants. The so-called neutral chemical components, as for example, chlorides, sulfates, etc. and similar substances which are not sorbed, do not decompose or decay, pass through the thickness of the rocks during filtration practically unchanged; groundwater is protected against such kinds of contamination only in the case of complete isolation of the water-bearing stratum or a very long duration of filtration evaluated in tens and hundreds of years (see above the first case). The calculations of the time of vertical percolation, given various filtration parameters of the rocks, show that the natural state of protection of unconfined groundwater, even with a thick and slightly permeable zone of aeration, is always insufficient for protection against contamination by neutral chemical components.

Active chemical substances, i.e., those which interact with the rocks and groundwater in particular, may be sorbed on the rocks given lengthy and continuous entry of contaminated water, may also spread over large distances, although they will progress more slowly than the neutral components of contamination. With brief entry of sorbent chemical contaminants, the latter may be completely retained in the rocks covering the aquifer in cases when the duration of filtration through these rocks is considerable and the rate of sorption exceeds the rate of desorption.

The natural state of protection of groundwater against the entry of surface waters containing radioactive substances is determined by the intensity of its sorption on rocks and by the time of decay; the natural state of protection of groundwater against biological contamination is determined, on the one hand, by the conditions of filtration and contamination; and on the other hand — by the time of survival, that is the duration of the existence of micro-organisms in groundwater. Thus, for a full evaluation of the state of natural protection of groundwater against contamination on the site of the proposed location of an industrial enterprise, it is necessary, in addition to the hydrogeological conditions, to take due account also of the quantity, composition, conditions and intensity of entry of contaminants. Inasmuch as at the first stages of the choice of a site, these data usually are poorly known, it is advisable to identify three main groups of aquifers based on the conditions of the natural state of protection of the groundwater: protected, conditionally protected and unprotected. The protected and conditionally protected include confined and unconfined interstratal waters, which, in the region under consideration, have a continuous water-resistant roof and which do not receive any recharge from overlying unconfined groundwaters, rivers and reservoirs through hydrogeological windows or dividing layers both under natural and disrupted conditions. The unprotected groundwater includes: a) unconfined groundwater, i.e. groundwater of the unconfined aquifer closest to the surface, which receives direct recharge, b) confined and unconfined interstratal waters, which, under conditions not disrupted by use, or in use, receive recharge in the region under consideration from overlying groundwaters, rivers and reservoirs through hydrogeological windows or dividing layers.

If for technical, economic or other reasons, it nevertheless becomes necessary to locate an industrial enterprise in an area where unprotected groundwater is developed, a more detailed study of the site is necessary, singling out those areas which are characterised by their degree of protection. The main hallmark of the degree of protection should be the duration of the vertical movement of contaminated water from the surface into the aquifer, and in the event that the contaminated water enters from a river and reservoir — the duration of the horizontal movement through the aquifer towards the water-intakes and similar installations. To make the corresponding calculation, it is necessary to have a sufficiently well documented description of the hydrogeological conditions as well as the hydrodynamic and migration parameters of the aquifer and the low permeability strata covering them. Even more detailed hydrogeological documentation is required for the preparation of a plan of special protective measures, if it becomes necessary to locate an industrial enterprise on the site of development of unprotected groundwater.

Therefore, in planning installations, which may be sources of contamination of groundwater and of the surface water connected with it, along with the usual array of geotechnical studies carried out to choose the design of installations, the methods of carrying out construction work and of ensuring the stability of constructions and water-management calculations, special hydrogeological and hydrochemical surveys must be carried out.

In choosing a site for an industrial installation, it should be borne in mind that blanket sandy loam and loam deposits, covering the aquifers, usually do not hinder the filtration of contaminated waste water into the aquifer; also hopes are not justified for the protective

effects of tailings, sludges and other sediments, which have accumulated in industrial waste tanks. Given an insufficient natural state of protection of the aquifer, industrial sites should be located at such a distance from groundwater intakes of a centralized water supply that the entry of contaminants into the water-intake and the surface waters can be completely ruled out. If the observance of these conditions is impossible, the problem of the location of the installations under consideration should be solved on the basis of the technical and economic calculations and comparisons, bearing in mind expenditures on measures for the protection of ground and surface waters and water-intakes against contamination, as well as the protection of the surrounding areas against water logging and swamping.

An example of planning the use of sites with a view to reducing the state of groundwater contamination is given in the work of Moosburner G.J., Wood E.F. [63]. For a forecast, use was made of a mass transfer model including bi-dimensional permeability, bi-dimensional dispersion and a first order chemical decomposition reaction, as well as of a model of multi-installation special-purpose optimizational programming as applied to 208 planned installations for construction. As a result of the investigation, the necessity has been demonstrated of limiting population growth in some parts of the area under consideration.

4.4. SANITARY PROTECTION ZONES OF WATER-INTAKES

4.4.1. PRINCIPLES OF SANITARY ZONES IDENTIFICATION

The protection of groundwater intakes is accomplished out by their removal to a safe distance from established or probable sources of contamination, which is achieved by the choice of the site of the water-intake and by the organization of special protective zones. The principles governing the definition of such zones and the regime within their limits are various (Miller D.W., Scaff M.R., [62]; Noring F. [64]; Exler H.J. [52], et al.) and are not always sufficiently justified. In some countries a safe distance is determined empirically, in accordance with traditions which have developed under conditions of a low level of urbanization and designed mostly for protection against bacterial contamination. The modern approach to the setting up of sanitary protection zones of water-intakes (SPZ) provides for protection against various types of contaminants, taking into account the hydrological conditions and the regulation of the economic use of the site of the SPZ (Minkin E.L. [21, 22]; Bochever F.M., Oradovskaia A.E. [4, 5, 6, 7] et al.).

The purpose of the sanitary protection zones of groundwater intakes (SPZ) is the prevention of bacterial and chemical contamination of water.

The main sources of microbial contamination are sewerage waters: municipal-domestic, surface (rain, melted, wash), stock-raising farms, as well as sewerage disposal and filter beds, leaks and accidental discharges from pipeline reticulations.

In an aquifer, micro-organisms more with the flow, and, therefore, the distance of propagation of micro-organisms, in the first place, is determined by the flow rate of the groundwater. At the same time, the distance of propagation of the micro-organisms is limited by the time of survival and preservation of the virulence of the micro-organisms. According to data supplied by special sanitary hygienic investigations, the time of survival of individual micro-organisms in groundwater depends on the type of micro-organism, the temperature and the chemical composition of the water, the initial quantity of micro-organisms and may reach 200–400 days. In fine-grained rocks, adsorption considerably hinders the propagation of micro-organisms. The parameters of adsorption, which depend on the properties of the rock and the type of micro-organism, are, however, insufficiently studied, and, therefore, in determining the distance of propagation of micro-organisms in connection with the calculations of the boundaries of SPZ, the adsorption of micro-organisms in the rocks, as a rule; is not taken into consideration. Taking account of adsorption and other physico-chemical processes affecting the propagation of micro-organisms in groundwater is possible only in those cases where the action of these factors is strongly expressed, and the laws governing their occurrence have been sufficiently studied. Thus, in justifying SPZ, the distance of propagation of micro-organisms in the aquifer is calculated basically according to the groundwater flow rate and the time of survival of the micro-organisms.

Sources of chemical contamination of groundwater are unpurified industrial waste waters, filtration from holding lagoons and other technical reservoirs and receptacles which collect industrial waste and waste waters, contaminated surface runoff from municipal and agricultural sites, etc. In calculations for SPZ, it is usually assumed that chemical contaminants are stable in groundwater, since they do not decompose and they are not transformed, so that together with the groundwater flow they may spread over considerable distances.

The ability of chemical contaminants to be transformed as well as to reduce their concentrations under the influence of physico-chemical processes occurring in the aquifer (sorption, sedimentation, decomposition, etc.) are taken into account only when the processes affecting the distance of propagation of chemical contamination are strongly expressed and their laws sufficiently studied. Such an approach ensures a sufficient "safety margin" in calculating the location of the boundaries of SPZ.

The limits of SPZ usually include two or three belts. In each of these belts, corresponding to their purpose, a special regime is established and comprehensive measures are carried out which exclude the possibility of contamination and, any deterioration of the water quality.

Let us consider the SPZ system, consisting of three belts: the first – a strict regime belt, the second and third are belts of restrictions. The purpose of the first belt of the SPZ is to eliminate the possibility of accidental contamination of groundwater directly through the water-intake works or by a disruption of the normal operation of the water-intake works, the water-raising devices, and installations for the purification and collection of water. Therefore, the size and shape of the first belt, in practice depend very little on the hydrogeological conditions and are determined chiefly by the composition and location of the installations protected.

The second and third SPZ belts (restriction belts) include an area intended for the protection against contamination of the source of water supply – the aquifer in the vicinity of the water-intake.

The boundaries of the first belt are located at a distance of no less than 30 m from the water-intake works using protected ground water and within a radius of not

less than 50 m when insufficiently protected groundwater is being used.

Protected groundwater includes confined and unconfined systems with a continuous surface cover of low permeability material and which do not receive any recharge from overlying unconfined groundwaters, waterways or reservoirs through hydrogeological windows and dividing layers. Insufficiently protected groundwater includes: 1) unconfined groundwater, i.e., ground water of the unconfined aquifer nearest the surface, which receives direct recharge; 2) confined and unconfined interstratal waters, which receive recharge from overlying unconfined groundwaters, waterways and reservoirs through hydrogeological windows and dividing layers.

For water-intakes of 1 or 2 working wells, located in areas where surface sources of contamination are absent, the distance to the boundary of the first belt may be reduced to 20–15 m. It is necessary to include a riparian area between the water-intake and the surface water source within the boundaries of the first belt of SPZ of bank filtration groundwater intakes, if the distance between tham is not more than 100 to 200 m. The boundaries of the first belt of the SPZ of groundwater intakes in systems of artificially-induced recharge should be set at the following distances; not less than 50 m, from the water-intake works – and not less than 100 m from the infiltration works.

A first SPZ belt is also established for rivers and resorvoirs used as sources for groundwater recharge, as well as for associated water pipelines and installations for water processing.

For circulating waterways, the boundary of the first belt of the SPZ is established not less than 200 m upstream and not less than 100 m downstream – from the water-intake; not less than 100 m from the line of the water's edge during summer-autumn low-waters; from the bank adjoining the water-intake – the whole water area and the opposite bank to a width of 50 m from the line of the water's edge at summer-autumn low-waters; towards the bank opposite a water-intake in the case of a river or canal with a width of less than 100 m – and a width of not less than 100 m for a river or canal of more than 100 m width. For nonflowing water bodies (reservoirs, lakes), the boundary of the first belt is determined by the sanitary and hydrological conditions; not less than 100 m from the water-intake in old directions from the intake; and not less than 100 m from the line of the water's edge of summer-autumn low-waters along the bank adjoining the water-intake.

At water intakes of the dipper type, the whole water area of the dipper is included within the first belt.

Certain demands are made on the sanitary state of the area of the first belt: all types of construction, residence of people, disposal of waste waters, use of toxic agro-chemicals, as well as the use of organic, and some types of mineral, fertilizers are prohibited; this area must be protected by a fence and planned for the removal of surface runoff.

The second belt of the SPZ adjoins the first, and the third adjoins the second belt, covering a wider area surrounding groundwater intakes. In the case of artificially-induced recharge systems the infiltration installations are included. The purpose of the second and third belts is to eliminate the possibility of the occurrence of sources of microbial and chemical contamination in that part of the water-bearing stratum from which groundwater is withdrawn to the water-intake.

In setting up the second and third SPZ belts of groundwater intakes, two hydrogeological questions are essential: firstly, the methodology of determining the size of SPZ and, secondly, the make-up of the sanitary and protective measures designed for the areas of the various SPZ belts. The larger the size of the SPZ, the greater the chances of preserving good quality groundwater. At the same time, if the SPZ is too large in size, the restrictions introduced in the economic use of this area may lead to certain economic damage. Correct solutions of the issue posed may be obtained given a differentiated, discriminating approach to the natural and economic situation in the area of each water-intake. At the same time, one should bear in mind the degree of danger in individual types of groundwater contamination under the existing or planned economic use of the area adjacent to the water-intake, the hydrogeological conditions of the aquifer in use, and in particular, the conditions of its natural protection against contamination, as well as the type, capacity and operating regime of the groundwater intake. Therefore, for protection against microbial contamination of limited extent, it is advisable to set up a second SPZ belt, and for protection against chemical contamination – a third SPZ belt.

The determination of the position of the boundaries of the second and third SPZ belts is based on the fact that the inflow of groundwater to the water-intake proceeds only from the recharge zone the form and size of which depend on the type of water-intake (individual wells, a group of wells, a linear row of wells, horizontal drain), the quantity of water withdrawn and the lowering of groundwater level, the hydrogeological parameters of the water-bearing stratum, and the conditions of its recharge and drainage.

The boundaries of the second and third belts are laid out by means of hydrodynamic calculations mainly along flow separation lines which form the boundaries of the zone of recharge of the water-intake.

In the case of open areas of the zone of recharge of the water-intake, e.g., upstream of the groundwater, and as for small water-intakes designed for a limited period of use the boundary of the second belt is located taking account of the estimated period T, selected on the basis of the following considerations.

In the area of the second belt of the SPZ, sources of microbial contamination must be absent. If, however, microbial contaminants enter the water-bearing stratum beyond the limits of the boundary of the second belt of the SPZ, they must not reach the water-intake; for this, it is necessary that the estimated time T_M of the progress of the contamination with the groundwater from the boundary of the second belt to the water-intake should be sufficient for the loss of viability and virulence of pathogenic micro-organisms.

The time T_M of the progress of microbial contamination in the groundwater flow to the water-intake should be considered as a basic parameter, determining the distance from the boundary of the second belt of the SPZ to the water-intake and, simultaneously, ensuring the epidemiological and hygienic safety of the boundaries of the SPZ.

To prevent the microbial contamination of groundwater, the time T_M is selected in the following way:

a) For unconfined groundwater under warm, humid climatic conditions and where contamination is possible via the zone of aeration the time T_M should be taken as not less than 200 days.

With a similar hydrogeological situation, but in conditions of a cold, temperate climate, as well as in all cases where there is a connection between the unconfined groundwater and a river or a surface reservoir, the time should be taken as not less than 400 days.

b) For insufficiently protected confined and unconfined interstratal waters, or where a close hydraulic connection between them and a surface reservoir exists, under cold, temperate climatic conditions, the time T_M should be taken as not less than 200 days; however, for the same hydrogeological conditions under a warm, or hot climate, but in the absence of any direct link with a surface reservoir, the time T should be taken as not less than 100 days.

On the area of a third belt of a SPZ, sources of a chemical contamination should be absent; if, however, persistent chemical contaminants enter the water-bearing stratum beyond the boundaries of the third belt of the SPZ, they will either not reach the water-intake, being displaced in the water-bearing stratum outside the zone of capture of the water-intake, or will reach it, but not earlier than the estimated time T_X, which should correspond to the usually accepted duration of the use of the water-intake.

Following the practice used in the quantitative evaluation of operating reserves of groundwater, the estimated time for the establishment of the size of the third belt of SPZ may be taken as equal to 25 years, although, in practice, this period of operation of the water-intake in fact is not a maximum in the majority of cases. In the groundwater of artesian basins, river valleys and alluvial fans, which are the main sources of water supply, after a 25-year period of operation, and often before the end of this period, the rate of draw-down of the reserves is markedly reduced and the movement of groundwater stabilized. Practically, under such conditions, we may speak of the probability of the quantity of groundwater forecast lasting an unlimited period.

The situation is different with respect to the quality of groundwater, since the possibility of the migration of contaminants and the danger of a deterioration in groundwater quality are maintained also under conditions of a stable water-intake operations regime. Therefore it is necessary to make sure that the sources of contamination do not occur also outside the limits of the third belt of the SPZ.

When there is a direct hydraulic connection of a bankside (infiltration) ground water-intake with a river or a reservoir, the bank of which is located within the calculated limits of the second and third belts of the SPZ of a bankside groundwater intake, it is necessary to establish second and third belts also for these surface sources of water.

In the second or third belts of the SPZ, the following limitations of economic activity are advised:

1) All types of construction are carried out with the permission and under the supervision of bodies of the public health and hydrogeological services; 2) sub-surface activities which may cause contamination of the aquifer are prohibited (the disposal and burial of waste waters, exploratory works for oil, gas, etc.); 3) inactive, defective, improperly operated wells and mine workings which create a danger of contamination for the aquifer are eliminated. These actions should be carried out with the restoration of the original state of protection of the aquifer according to an approved plan under the supervision of the public health and hydrogeological services; 4) Measures are taken for the sanitary protection of surface reservoirs which are hydraulically connected with groundwater.

In the second belt of an SPZ, where insufficiently protected groundwater is to be used for the water supply, the following are prohibited:

a) the use of non-neutralised organic (manure) fertilizers, agricultural and forest-protective toxic chemicals;

b) the locating of irrigation fields, stock-raising enterprises and farms, cemeteries, cattle burial-grounds, manure reservoirs, silo ditches, residential housing without a sewerage system, sewerage collectors and similar installations causing a danger of microbial contamination;

c) the locating of agricultural land, irrigated by highly mineralised or contaminated waters, stores for fuel and lubricating products, stores of toxic chemicals and mineral fertilizers, fuel-oil reservoirs and similar installations causing a danger of chemical contamination.

In the third belt of an SPZ where insufficiently protected groundwater is to be used for the water supply, the use of organic (manure) and mineral fertilizers, agricultural and forest-protective toxic chemicals is allowed, provided their use is limited and controlled. The locating of irrigation fields, stock-raising enterprises and farms, cemeteries, cattle burial-grounds, manure reservoirs, silo ditches, residential housing without a sewerage system, sewerage collectors and similar installations causing a danger of microbial contamination is allowed provided the surface and drainage flow from these installations is collected and removed beyond the limits of the area of the second and third belts of the SPZ. The locating of agricultural land, irrigated by highly mineralized or contaminated waters, storehouses of fuel and lubricating products, stores of toxic chemical and mineral fertilizers, fuel oil reservoirs and similar installations causing a danger of chemical contamination is allowed at a distance not closer than one kilometre from the boundary of the second belt, provided that special measures have been planned for protecting the aquifer against contamination under normal and emergency conditions of operation.

When using insufficiently protected groundwater sources of water supply, and given the presence beyond the limits of the third belt (at a distance of up to 3 kilometres) of sources of chemical contamination of groundwater, provision should be made for the organization of a network of observation wells along lines proceeding from the source of contamination to the boundaries of the third and second belts of the SPZ, and plans should be prepared for the elimination of such sources of contamination.

The positions of the boundaries of the second and third belts of the SPZ of the groundwater intakes are determined by hydrodynamic calculations, which should be carried out according to a methodology similar to that used for the evaluation of the capacity of the water-intake.

In complex hydrogeological conditions, the zone of capture of the water-intake may also have complex geometrical outlines, the delineation of which is possible only on the basis of analytical-tabular constructions using maps of water-table contours, drawn up from data provided by field observations, and from groundwater flow modelling.

An example of the construction of a Sanitary Protection Zone (SPZ) on the basis of an analytical-tabular method is shown in Fig. 4.1.

A river side groundwater-intake consists of nine wells; groundwater is abstracted at 27 thousand m^3/daily from fractured chalk-mark rocks of Cretaceous age, which characterized by considerable heterogeneity. Aquifer recharge is via infiltration of atmospheric precipitation on the valley slopes, as well as by the flow of river and flood waters through alluvial deposits on the flood plain of the valley. The position of the boundary of the second and third belts of the SPZ is calculated by the analytical-tabular method along characteristic flow lines separated out on a water-table contour map. This latter map was obtained by numerical modeling on a computer; the position of the water-table contours on this map coincides with the data from field observations. In modeling, account was taken of the heterogeneity of the water-bearing stratum, the complex outlines of the river, hydraulic connections between alluvial and chalk aquifers, infiltration recharge in periods of flooding of the low lying areas, recharge from the bed-rock slopes of the valley, etc.

Because of the high permeability of the rocks up to 100 m/daily) and low active porosity, the flowrate of groundwater reaches 1–2 m/daily in the vicinity of the water-intake. This determines the large size of the SPZ: the boundary of the second belt, intended for protection against bacterial contaminants, is 1.5–2 km distant from the water intake; the boundary of the third belt of the SPZ, designed for protection against chemical contaminants for a period of 25 years, is located at the following distances; upstream – 6 km, downstream – 1 km, in the direction of the bed-rock valley slopes – 3–4 km. According to modeling data, 21 % of the discharge of the water-intake is provided by recharge from alluvium and from the river.

If the hydrogeological situation can be schematized and averaged out according to the main planning parameters, the zone of capture of the water-intake and other unknown quantities, may be determined by means of analytical calculations (Minkin E.L., [21, 23]; Goldberg V.M., [10, 11, 13]; Planning Groundwater Intakes, [29]; Bochever F.M., Lapshin N.N., Oradovskaia A.E., [7] et al.) in order to confirm the soundness of the boundary of the SPZ of groundwater intakes. At the same time, the water-intakes themselves are schematized: usually they are represented in the form of unified group water intakes.

Fig. 4.2. shows a diagram of a zone of capture for one group water-intake with a yield Q in a homogeneous unlimited water-bearing stratum with the presence of unidimensional natural flow with a unitary discharge q. The zone of capture of the water-intake increases with time during operation. However, with some overestimation of the total area of the SPZ, its boundaries may be established from the maximum limits of the position of the dividing line of flow under steady state conditions. In the downstream direction of groundwater flow the boundary of the third belt is drawn through a dividing point with the coordinate – x_B. But in those cases where the distance from the water-intake to the point x_B is great, and the flow time of the water particles from it to the water-intake is greater than the estimated time T, the position of the third belt of the SPZ shifts closer to the water-intake – at distance r from the water-intake.

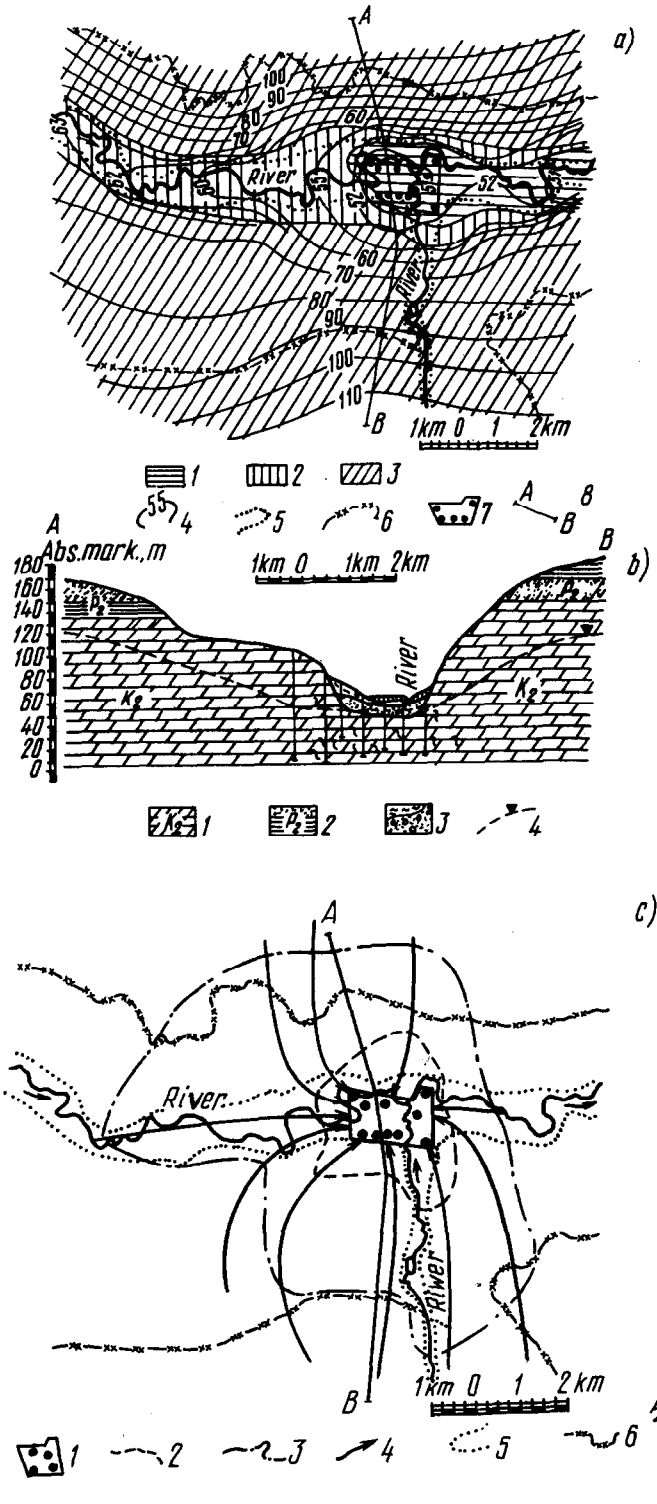


Fig. 4.1. Sanitary Protection Zones of a water-intake calculated by the analytical-tabular method.

a – map of the water-conductivity of rocks; 1,2, 3 – water conductivity: 1000–3000 m^2/daily and more, 100–1000 m^2/daily, less than 100 m^2/daily; 4 – water-table contour lines; 5 – boundary of flood plain deposits; 6 – boundary of Paleogenic deposits; 7 – water-intake; 8 – A–B – line of geological cross-section.

b – geological cross-section along line A–B
1 – fractured marle of the Upper Cretaceous period; 2 – clayey rocks of the Paleogenic period; 3 – alluvial deposits; 4 – groundwater level.

c – boundaries of SPZ; 1 – water-intake; 2 – boundary of 2nd belt of SPZ (for protection against bacterial contamination); 3 – boundary of 3d belt of SPZ (for protection against chemical contamination); 4 – line of flow; 5 – boundary of flood plain deposits; 6 – boundary of Paleogenic deposits; 7 – A–B–line of cross-section

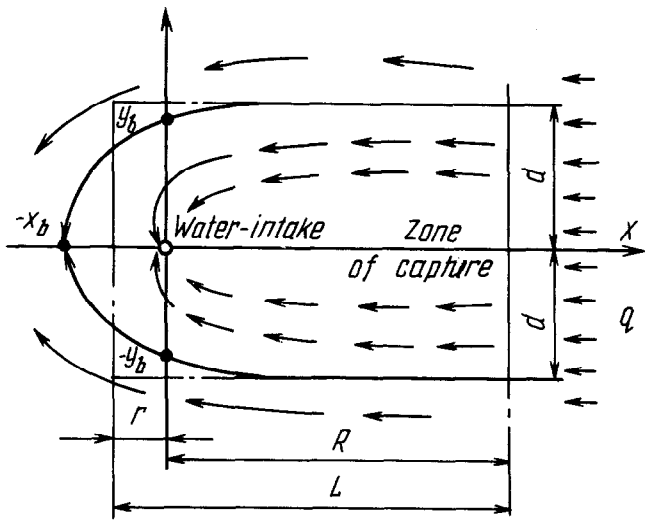


Fig. 4.2. Diagram of the zone of capture of a groundwater intake in an unlimited water-bearing stratum.

For practical calculations, it is advisable to schematize the zone of capture of the water-intake works in the form of a rectangle of width 2d and total length L, in which (see Fig. 4.2)

$$L = r + R \tag{4.3}$$

The quantity 2d is taken as equal to the maximum width of the zone of capture of the water-intake. The length of the SPZ upstream must be such that the water, removed from the water-intake at distance R, reach the water-intake only at the end of estimated time T, counted from the beginning of the operation of the water-intake, and estimated depending on the type of possible contamination and the degree of natural protection of the aquifer. In unconfined aquifers, as well as in shallow confined strata, covered by slightly permeable deposits (2-layer systems), in determining the position of the boundary of the SPZ depending on bacterial contaminations it is advisable to take account of the time t_0 of percolation of contaminated water vertically to the main operating stratum, i.e., to take

$$T = T_m - t_0 \tag{4.4}$$

Groundwater intakes consist for the most part of single wells and systems of interacting wells arranged in different ways. For purposes of calculation, the group water-intake may be represented in the form of some generalized systems. Given a compact arrangement of the wells within a limited area, they may be considered as one enlarged water-intake – "a big well" with a yield equal to the total yield of all operating wells of the water-intake. The axial point "of the big well" must be located in the "center of gravity" of the system of wells, i.e., in the point with the coordinates

$$x_0 = \sum_{i-1}^{n} x_i Q_i / Q;\ y_0 = \sum_{i-1}^{n} y_i Q_i / Q, \tag{4.5}$$

where Q – is the total discharge of the water intake; Q_i – are the yields of the individual wells (i – 1, 2, ...,), n – is the number of wells; x_i, y_i – are the coordinates of the wells in relation to an arbitrarily selected system of coordinates. The water-intakes in the form of linear rows of wells for the simplification of calculations are replaced by galleries (if their length is considerable) or by an enlarged water-intake, arranged in the center of the water-intake row (if the length of the water intake is comparatively small). Water-intake drains or trenches, for an approximate evaluation of the parameters of the SPZ, also may be replaced by galleries with approximately identical water uptake along the length of the installation. Radial groundwater intakes, arranged at groundwater divides or close to a river, are reduced for purposes of calculation to an enlarged water-intake works – "a big well". The axis of such an enlarged water intake is laid out along the center of the radial water intake.

For an approximate determination of the position and size of the zone of capture of water-intakes in systems of artificial groundwater recharge, the whole unit of the installation (water-intake, infiltration installations) may be represented in a generalized form as a single enlarged water-intake with the yield

$$Q = Q_B - Q_\delta \tag{4.6}$$

where Q_B – is the total discharge of the water-intake wells; Q_δ – is the mean discharge of water entering the stratum from the infiltration installations.

Such a calculation does not give rise to substantial error provided there is a relatively large discharge of the water-intake as compared with the total inflow of water for infiltration and provided the size of the SPZ exceeds the distance between the water-intake wells and the infiltration installations.

4.4.2. SINGLE WELLS AND COMPACT GROUPS OF INTERACTING WELLS

Water-Intakes near Rivers

Groundwater intakes near rivers (canals, reservoirs and other surface waterways and bodies of water) usually draw upon shallow unconfined or slightly confined aquifers.

In the process of operation of rive bank water-intakes continuous recharge takes place of supplies of groundwater from surface waters and the groundwater rapidly acquires a steady state situation.

The basic diagrams of filtration to the rive bank intake, consisting of one well or a more or less compact group of interacting wells near a perfect river, are shown in Fig. 4.3. Below are given the equations for the determination of the main parameters of SPZ for each diagram as applied to confined flows. However, they may be used also for unconfined flows given the substitution of thickness m of the confined stratum by the mean thickness h_m of an unconfined stratum.

In the diagram of Fig. 4.3a natural flow is directed from the bank to the river and the yield of the water-intake Q is made up of the discharge of filtered river water Q_p and the discharge of groundwater natural flow Q_e:

$$Q = Q_e + Q_p \tag{4.7}$$

Depending on the correlation of these quantities, the maximum upgradient width of the zone of capture of the water-intake (2d) and on the line of the river ($2y_p$) is found. When $\bar{Q} > 1$

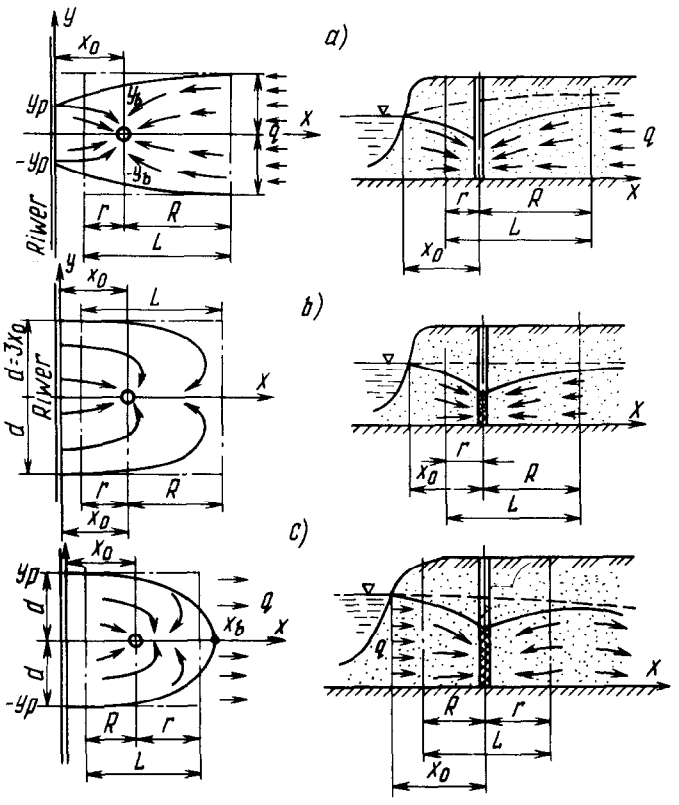


Fig. 4.3. Diagram of filtration to the bank-side water-intake near a hydraulically perfect river:
a) natural flow directed to the river; b) natural flow is absent; c) natural flow is directed away from the river.

$$d = Q_e/2q;\ y_p = x_0\sqrt{\bar{Q}-1}, \qquad (4.8)$$

where

$$Q_p = (2Q \operatorname{arctg} \bar{x}_0)/\pi + 2qy_p$$

$$Q_p = (2Q \operatorname{arctg} \frac{1}{x_0})/\pi - 2qy_p \qquad (4.9)$$

$$\bar{Q} = Q/\pi q x_0;\ \bar{x}_0 = x_0/y_p$$

The width of the zone of capture $2y_b$ on a line passing through a well ($x = x_0$), is determined for the given diagram according to the graph in Fig. 4.4.

To determine the distances R and r – upstream and downstream, from which the size of the SPZ is evaluated (see Fig. 4.3a) the following expression (10) and (11) should be used:

$$T = mnx_0[(\bar{Q}/\sqrt{\bar{Q}-1})\operatorname{arctg}(\bar{r}\sqrt{\bar{Q}-1}/(\bar{Q}-\bar{r})-\bar{r}]/q \qquad (4.10)$$

$$T = mnx_0[\bar{R}-(\bar{Q}/\sqrt{\bar{Q}-1})\operatorname{arctg}(\bar{R}\sqrt{\bar{Q}-1}/(\bar{R}+\bar{Q}))]/q \qquad (4.11)$$

where

$$\bar{r} = r/x_0,\ \bar{R} = R/x_0 \qquad (4.12)$$

m – is the thickness of the stratum, n – is active porosity, T – estimated time (25 years, 400 days, etc.), for which the position of the boundaries of the SPZ is determined.

The quantities $\bar{R}$ and $\bar{r}$ are given also in the form of graphs in Figs 4.5 and 4.6 (diagrams in Fig. 4.3a).

In the absence of natural groundwater flow (diagram in Fig. 4.3b), the width of the zone of capture of the shore water-intake 2d is established on the basis of the size of the river water filtration front, because the yield of the water-intake is completely provided by river waters: $Q = Q_p$. At the same time, the main recharge of the water-intake (more than 80 %) arises from the sector of the river 2d, equal approximately to $6x_0$. The length of this sector is taken as the width of the zone of capture of the water intake.

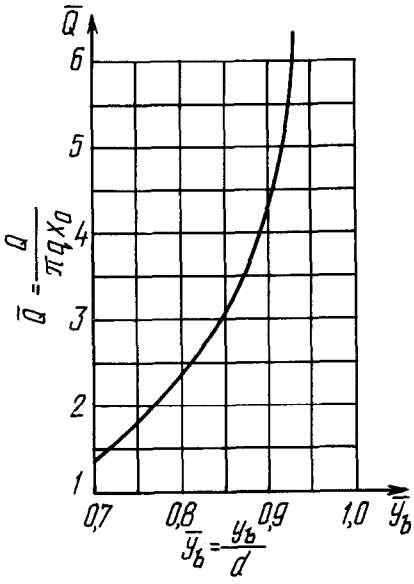


Fig. 4.4. Graph to determine the width of the zone of capture in a cross-section passing through the water-intake.

The extension of the SPZ into the bank in the present diagram (in the absence of domestic flow) is determined from correlations (13) and (14), or according to the graphs in Figs. 4.5 and 4.6 (when q = 0):

$$T = [\pi mnr^2\ (1-r/3x_0)]/Q \qquad (4.13)$$

$$T = [\pi mR^2 n\ (1+\bar{R}/3x_0)]Q \qquad (4.14)$$

In calculating for small intervals of time (when $T = T_M$) the width of the SPZ in the diagrams considered in Fig. 4.3a,b may prove larger than its length, i.e. $d > R$. For a single or group water-intake, this should not occur and d = R should be taken, while R is determined from T_M.

If the natural flow q is directed from the river towards the bank, the groundwater flow diagram acquires the form shown in Fig. 4.3c.

The length of the SPZ in the direction of the bank (r) and to the river (R) is determined in this case from formulae (15) and (16), as well as from the graphs in Figs 4.5 and 4.6 (see diagram 4.3c).

$$T = mnx_0[(\bar{Q}/\sqrt{\bar{Q}+1})\operatorname{arth}(\bar{r}\sqrt{\bar{Q}+1}/\bar{Q}-\bar{r}))-\bar{r}]/q \qquad (4.15)$$

$$T = mnx_0[\bar{R}-(\bar{Q}/\sqrt{\bar{Q}+1})\operatorname{arth}(\bar{R}\sqrt{\bar{Q}+1}/(\bar{Q}+R))]/q \qquad (4.16)$$

The width of the SPZ is established from the maximum width of the zone of capture $2y_p$ (on the river's edge). The latter may be determined from formula (17), or from the graph in Fig. 4.7:

$$y_p = x_0 \operatorname{ctg}(\bar{Q}y_0/x_0) \qquad (4.17)$$

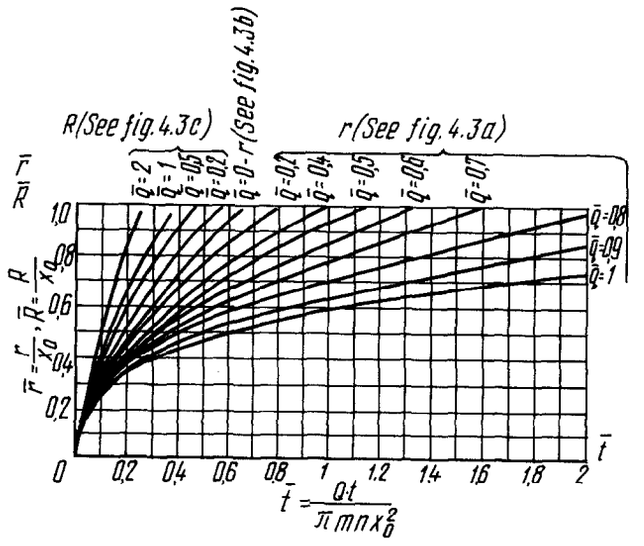


Fig. 4.5. Graphs to determine the length of the SPZ downstream of the groundwater (r in diagrams in figs. 4.3a, b, R in the diagram in fig. 4.3c).

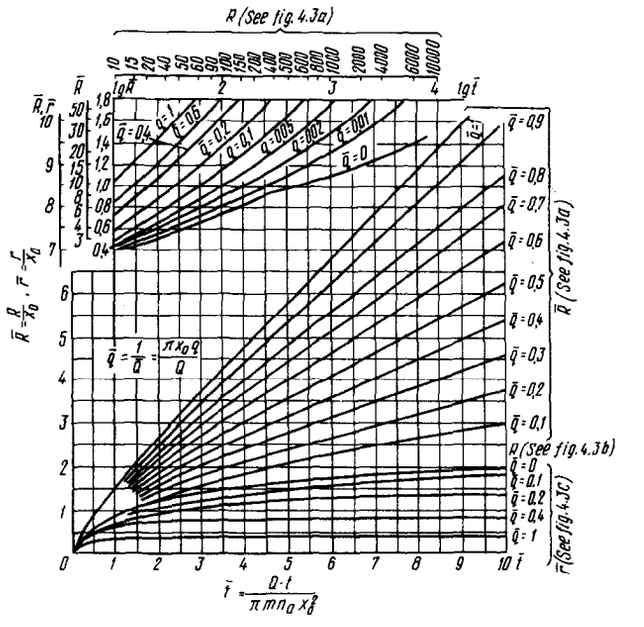


Fig. 4.6. Graphs to determine the length of the SPZ upstream of the groundwater (r in the diagram in fig. 4.3c, R in the diagrams in figs. 4.3a, b).

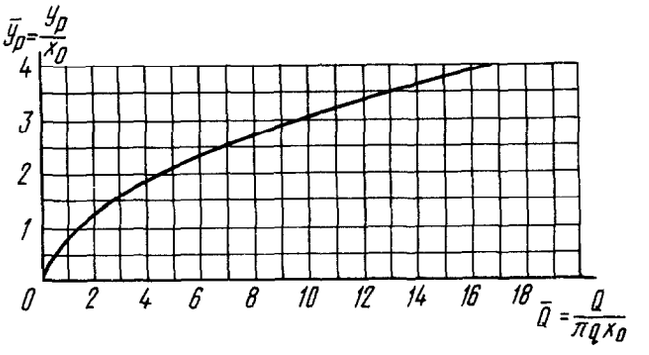


Fig. 4.7. Graph to determine the width of the zone of capture at the river's edge (diagram in fig. 4.3c).

The coordinate of x_b (see diagram in Fig. 4.3c) is determined from the expression

$$x_b = x_0 \sqrt{\bar{Q}+1} \qquad (4.18)$$

In the diagram for the determination of time T_{riv}, during of which the river water may reach the water-intake, one should take $\bar{r} = 1$ (for the diagrams in Figs 4.3a and 4.3b) and $\bar{R} = 1$ (for diagram 4.3c), in the corresponding formulae and on the graphs of Fig. 4.5.

When $T_{riv} < T$ (T – is the estimated period taken) the boundaries of the SPZ are drawn along the line of the river (r; $R = x_0$). This case should be considered as fundamental. Only when $T_{riv} \gg T$, in simple hydrogeological conditions, may the boundary of the SPZ be drawn between the river and the water-intake (r, $r < x_0$).

The diagram of the flow of groundwater to the shore water-intake near an imperfect river and of the natural flow directed to the river takes the form shown in Fig. 4.8.

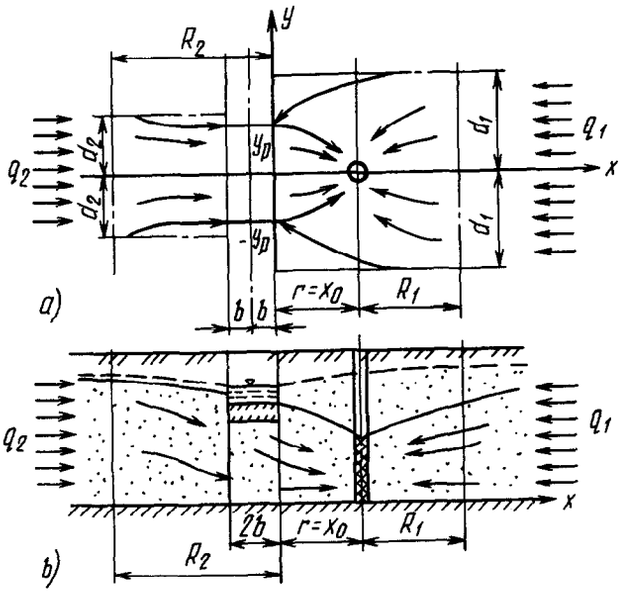


Fig. 4.8. Diagram of filtration to the bank-side water-intake near a hydraulically imperfect river:
a) plane; b) cross-section.

The zone of capture of the water-intake may include both banks of the river. At the same time, the discharge of the water-intake is divided into the following components:

$$Q = Q_{e1} + Q_{e2} + Q_p \qquad (4.19)$$

where Q_{e1} and Q_{e2} – are the discharges of natural flow, drawn by the water-intake on both banks, Q_p – is the inflow of river water to the water-intake. When $\bar{Q} > 1$, these discharges may be evaluated from the following equations, in which indexes 1 and 2 correspond respectively to the waterside sector of the location of the water-intake and to the opposite bank

$$Q_{e1} = 2Q \operatorname{arctg} \bar{x}_H/\pi + 2q_1 y_p, \qquad (4.20)$$

$$Q_{e2} = (Q - Q_{e1})/\operatorname{ch}(2b\alpha), \qquad (4.21)$$

$$Q_p = (Q - Q_{e1})\,[1 - 1/\operatorname{ch}(2b\alpha)], \qquad (4.22)$$

where

$$x_H = x_0 + \Delta L, \tag{4.23}$$

$$\bar{x}=x_H/y_p;\ y_p=x_H\sqrt{\bar{Q}-1};\ \Delta L=\mathrm{cth}(2b\alpha)/\alpha, \tag{4.24}$$

α — a parameter indicating the degree of hydraulic connection between the river and the aquifer, determined by experimental data, b — half the width of the channel.

The position of the boundaries of the SPZ on the bank on which the water-intake is located is determined by the equations:

$$d_1 = Q_{e1}/2q_1 \qquad r = x_0 \tag{4.25}$$

R_1 is calculated from the graph in Fig. 4.6 and from expression (11), in which instead of $\bar{R} = R/x_0$, $\bar{Q}=Q/\pi x_0 q$ and q one should use $\bar{Q}=Q/\pi x_H q_1$ and q_1. The position of the boundaries of the SPZ on the opposite bank is determined by the expression:

$$d_2 = Q_{e2}/2q_2;\ R_2 = q_2 T/mn \tag{4.26}$$

The calculation and setting up of an SPZ on the opposite bank should be carried out because of the existence of a danger of groundwater contamination and in relation to a high value of the discharge Q_{e2} in the total discharge of the water-intake Q.

Water-Intakes at a Distance from the River

A characteristic of isolated strata, i.e., those which do not have external recharge sources (infiltration of atmospheric precipitation, overflow of surface waters or groundwaters from nieghbouring strata, etc.) is the unsteady nature of the groundwater flows throughout the whole time of operation of the water-intake. However, an approximate evaluation of the size of the zone of capture, is determined in the same way as in conditions of steady or quasi-steady groundwater flow. For an enlarged water-intake in an unlimited isolated stratum, the equation of the dividing line has the appearance (Fig. 4.9)

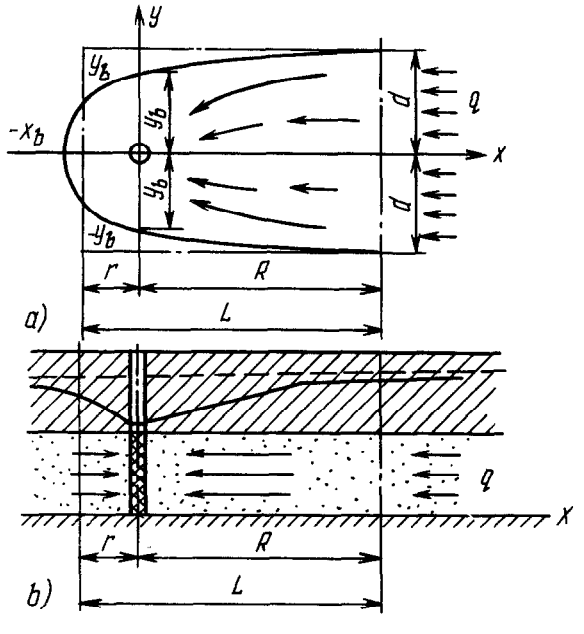


Fig. 4.9. Diagram of filtration to water-intake distant from river: a) plane; b) cross-section

$$x = -y\,\mathrm{ctg}\,(y/x_b), \tag{4.27}$$

where x_b — is the distance from the water-intake to the water-dividing point, while

$$|x_b| = Q/2\pi q \tag{4.28}$$

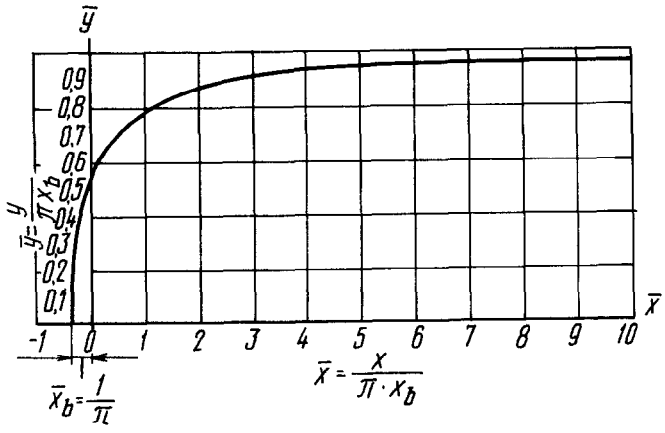


Fig. 4.10. Graph to determine the coordinates of the dividing line of flow.

In Fig. 4.10 a graph is given for the determination of the coordinates of the points located on the dividing line. According to this graph, one may find the width of the zone of capture (2_y) in any cross-section, located at distance x from the water-intake. The maximum width of the zone of capture

$$2d = Q/q \tag{4.29}$$

is established at a comparatively small distance from the water-intake, when

$$x > (3\text{–}4)\ |x_b| \tag{4.30}$$

The width of the zone of capture in the cross-section passing through the centre of the water-intake (x = 0), is determined in the following way:

$$2_{y_b} = Q/2q \tag{4.31}$$

The length of the SPZ upstream of the groundwater from the water-intake and the time of travel to the water-intake may be found from the equation

$$T=mn\,R-|x_b|[\ln(|x_b|+R)-\ln|x_b|]\,/q, \tag{4.32}$$

or according to the graph in Fig. 4.11.

In determining distance r to be boundary of the second or third belts of the SPZ downstream the following formula is used:

$$T=mn\,|x_b|[\ln(|x_B|-r)-\ln|x_b|]-r\,/q \tag{4.33}$$

as well as the graph in Fig. 4.11.

In the absence of natural groundwater flow, the zone of capture of the water-dividing water-intake in an isolated stratum constitutes a circle with the radius

$$R = \sqrt{QT/\pi mn} \tag{4.34}$$

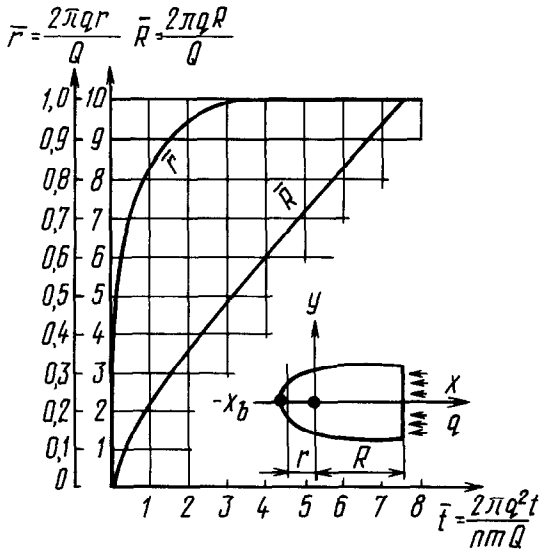


Fig. 4.11. Graph to determine R and r for the filtration diagram shown in fig. 4.9.

Formula (25)–(34), are valid for bank side water-intake and for confined strata with constant thickness m, as well as for unconfined aquifers with varying thickness h, given the condition of averaging of the latter, i.e., when $m = h_{cp}$.

4.4.3. WATER-INTAKES IN THE FORM OF LINEAR ROWS OF WELLS

Water-Intakes near the River

As a rule, the length of bank side water-intakes in the form of linear rows of wells considerably exceeds their distance from the river ($l > x_0$) and as a result of this, unidimensional flows occurs in the larger part of the zone of influence of the water-intake. Fig. 4.12 shows the diagrams of bank-side water-intakes in the form of linear rows of wells at a perfect waterway or body of water. These diagrams also correspond to bank-side water-intakes in the form of horizontal galleries, drains, etc.

With natural discharge q, directed to the river (diagram in Fig. 4.12a), the yield of the linear row of wells, as in the case of a single group water-intake, is made up of the waters from the river (Q_p) and from the bank, i.e., from the natural flow (Q_e). The quantity Q_e may be found from the graph in Fig. 4.13. The coordinate of the point Q_e is determined according to the expression.

$$y_0 = \sqrt{l^2 - x_0^2 + 2x_0 \operatorname{ctg}(2\pi ql/Q)} \qquad (4.35)$$

The calculating equations for the determination of d, r and R are given in formulae (36)–(39):

$$d = Q_e/2d \qquad (4.36)$$

$$r = QT[1-(2ql/Q)]/2mnl;\ (r\ max = x_0) \qquad (4.37)$$

$$T = mn x_0\ \bar{R} - (\bar{Q}\sqrt{\bar{Q}}+1)\operatorname{arctg}[\bar{R}\sqrt{Q}+1/(\bar{Q}+1+\bar{x}_0(\bar{R}+x_0))]\ /q \qquad (4.38)$$

where

$$\bar{R}=R/\sqrt{l^2-x_0^2},\ \bar{x}_0=x_0/\sqrt{l^2-x_0^2},\ \bar{Q}=Qx_0/\pi q(l^2-x_0^2) \qquad (4.39)$$

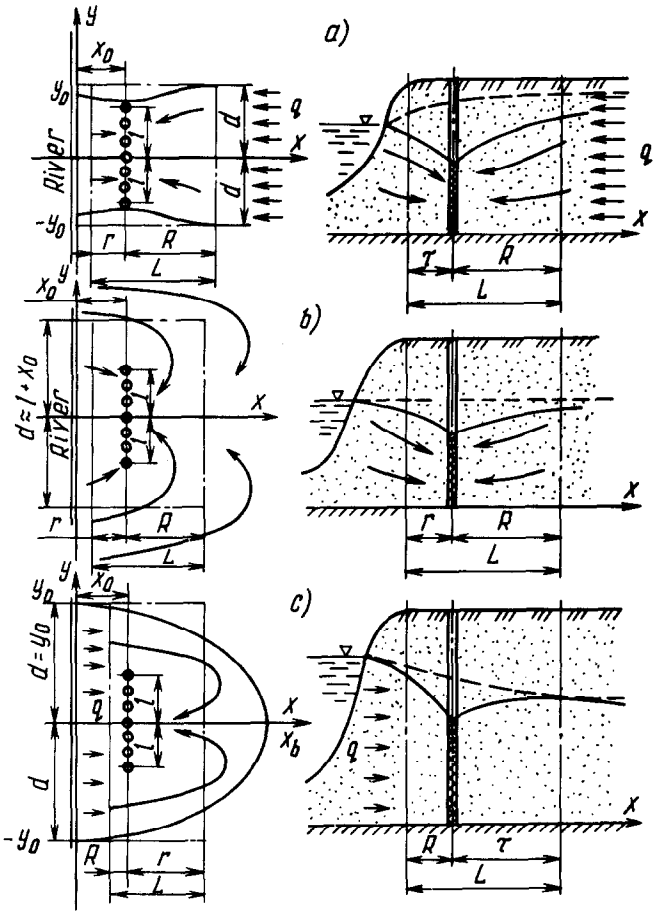


Fig. 4.12. Diagram of filtration to the bank-side linear water-intakes.
a) natural flow directed to the river; b) natural flow is absent; c) natural flow is directed away from the river.

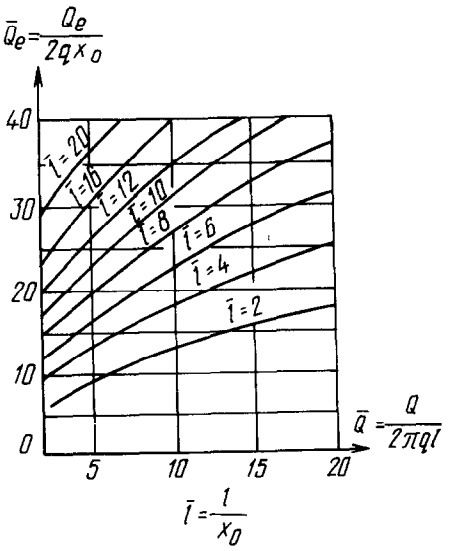


Fig. 4.13. Graph to determine natural flow Q_e, captured by a linear water-intake (to go with the diagram in fig. 4.12a).

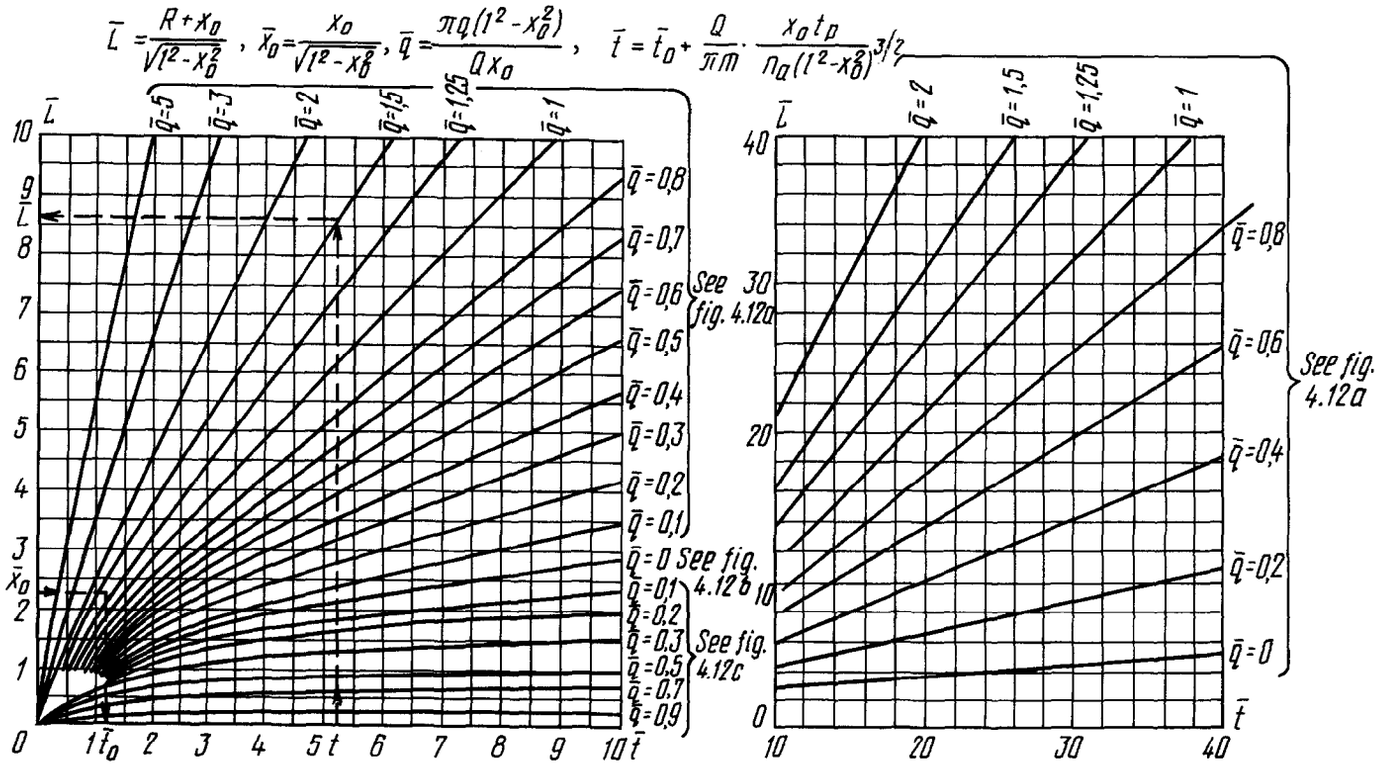


Fig. 4.14. Graphs to determine the length of the SPZ for bank-side linear water-intakes (to go with diagrams in fig. 4.12).

For the determination of R the graph in Fig. 4.14 may also be used.

In using the graph in Fig. 4.14 one should first determine the dimensionless parameter $\bar{T}_0$. To do this the following quantity is laid out on the axis of the ordinates

$$\bar{x}_0 = x_0/\sqrt{l^2-x_0^2} \qquad (4.40)$$

and the quantity q on the axis of the abscissae

$$q = \pi q\,(l^2-x_0^2)/Qx_0 \qquad (4.41)$$

from which the corresponding value $\bar{T}_0$ is estimated.

Further, the quantity is calculated

$$\bar{T} = \bar{T}_0 + Qx_0 T_{per}/\pi mn\,(l^2-x_0^2)^{3/2} \qquad (4.42)$$

and the quantity $\bar{T}$ corresponding to the given value L is found from the graph in Fig. 4.14:

$$\bar{L} = (R+x_0)/\sqrt{l^2-x_0^2}, \qquad (4.43)$$

and then the unkown distance R to the upper border of the SPZ.

The time of entry of the river waters into wells of the water-intake in the diagram considered may be determined approximately from

$$T \approx 2mnlx_0/Q[1-(2ql/Q)-2x_0 l/\pi(l^2-x_0^2)] \qquad (4.44)$$

In the absence of a natural flow (q = 0) the diagram of groundwater flow to the water-intake assumes the form shown in Fig. 4.12b. The main part of the yield of the water-intake (> 80 %) in this case is compensated by the inflow of surface waters from the river on a front of width 2d = 2 (1 + x_0). On this basis, the width of the SPZ is determined. The values r and R are determined from

$$r \approx QT/2mnl \quad (r_{max} = x_0) \qquad (4.45)$$

$$T = \pi mnl^2 R\,[1+Rx_0/l^2+R^2 3l^2]/Qx_0 \qquad (4.46)$$

In addition, R may be determined from the graph in Fig. 4.14.

The time of inflow of river water to the wells of the water-intake is determined, as in the preceding diagram, from formula (44), but with q = 0.

With natural groundwater flow, directed away from the river, the zone of capture of the linear bank-side water-intake acquires the form represented in Fig. 4.12b and Q = Q_p. The width of the SPZ here is determined by the maximum width of the zone of capture at the river's edge and may be found from the graph in Fig. 4.15 when

$$\bar{Q} = Qx_0/\pi q\,(l^2-x_0^2) \qquad (4.47)$$

The coordinate of the dividing point is equal to

$$x_b = \sqrt{x_0^2-l^2+2x_0\,\mathrm{ctg}\,(2\pi ql/Q)} \qquad (4.48)$$

The quantity R is determined from the expression

$$R \approx QT(1+2ql/Q)/2mnl \quad (R_{max} = x_0) \qquad (4.49)$$

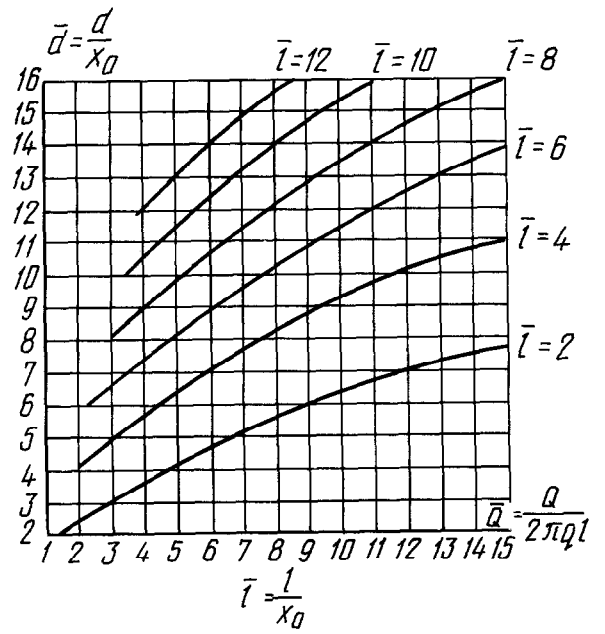


Fig. 4.15. Graphs to determine the width of the SPZ of a linear bank-side water-intake (to go with the diagram in fig. 4.12c).

For the determination of the r the graph in Fig. 4.14 may be used (in which R should be replaced by r), as well as formula (50):

$$T=mnx_0\ (Q/\sqrt{Q-1})\ \mathrm{arth}\left[(\bar{r}\sqrt{\bar{Q}-1})/(\bar{Q}-1-\bar{x}_0(\bar{r}+\bar{x}_0))\right]-r\ /q, \tag{4.50}$$

where $\bar{r}=r/\sqrt{l^2-x_0^2}$ ($r_{max}=x_b-x_0$)

The time of flow of river water to the water-intake may be determined from formula (44), given a negative value of q.

The diagram of groundwater flow to linear water-intakes, located near to a hydraulically non fully penetrating river, is shown in Fig. 4.16. The main parameters of SPZ on a site of a water-intake in the diagram are determined approximately from the same equations as for a perfect (fully penetrating) river, but by substituting in them the real distance x_0 by quantity x_H determined from formula (23). In the diagram the discharge of the water-intake Q_p is equal to:

$$Q=Q_{e1}+Q_{e2}+Q_p \tag{4.51}$$

where Q_{e1} and Q_{e2} — are the discharges of natural flow taken up by the water-intake on both banks; Q_p — is the flow of river water to the water intake. The discharge Q_{e1} is determined from the graph in Fig. 4.13 when $x_0=x_H$ (x_H — see formula 23). The discharges Q_{e2} and Q_p are determined from:

$$Q_{e2}=(Q-Q_{e1})/\mathrm{ch}\ (2b\alpha) \tag{4.52}$$

$$Q_p=(Q-Q_{e1})\ [1-1/\mathrm{ch}\ (2b\alpha)] \tag{4.53}$$

The size of SPZ on the bank on which the water-intake is located is determined by the expressions

$$d_1=Q_{e1}/2q_1;\ r\approx x_0 \tag{4.54}$$

R_1 is calculated according to formula (38) and from the graph in Fig. 4.14, in which instead of R, x_0, q should use R_1, x_H, q_1.

The size of the SPZ on the opposite bank may be calculated from the expressions

$$d^2\approx Q_{e2}/2q_2 \tag{4.55}$$

$$R_2\approx q_2T/mn \tag{4.56}$$

Water-Intakes at a Distance from the River

Linear water-intakes are most often arranged perpendicular to the direction of natural water flow (Fig. 4.17). The zone of capture in this way is limited by a groundwater divide at a distance from the water-intake. At the same time

$$x_b=l\ \mathrm{ctg}\ (2\pi ql/Q) \tag{4.57}$$

The length of the SPZ upstream and downstream of the groundwater (R and r) is determined from the graph in Fig. 4.18. The maximum quantity of SPZ downstream of the groundwater

$$r_{max}=x_b \tag{4.58}$$

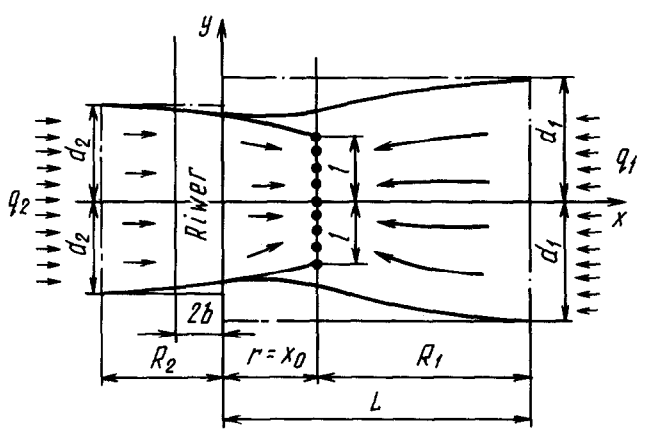


Fig. 4.16. Diagram of filtration to a linear water-intake near a partially penetrating river.

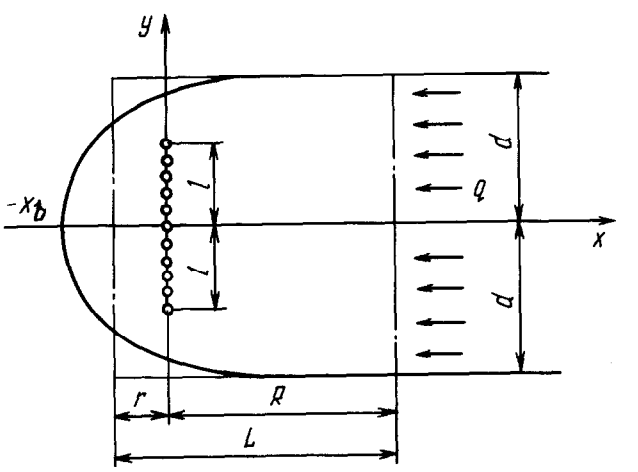


Fig. 4.17. Diagram of a linear water intake distant from the river (unlimited aquifer).

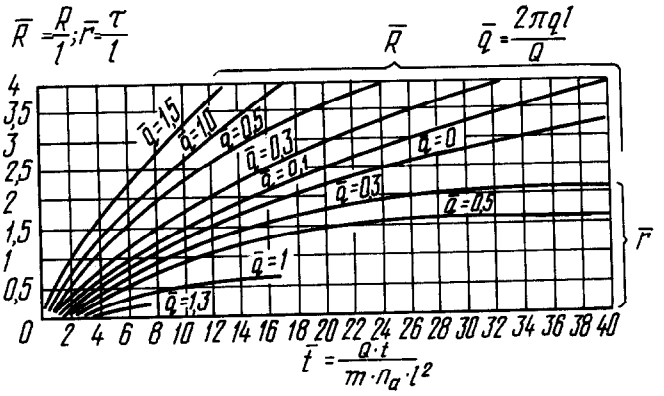


Fig. 4.18. Graphs to determine the length of the SPZ for linear rows of wells distant from the river (to go with the diagram in fig. 4.17).

The width of the SPZ is determined by the ratio of the yield of the water-intake to the quantity of the natural groundwater discharge:

$$d = Q/2q \qquad (4.59)$$

In this case the discharge of the water-intake works is fully provided by the natural groundwater flow, i.e., $Q = Q_e$.

4.4.4. TAKING ACCOUNT OF ADDITIONAL RECHARGE OF WATER-BEARING STRATA IN CALCULATING SPZ

The most widespread and substantial sources of additional recharge of water-bearing strata are overflow of groundwater from adjacent aquifers and the increase in the rate of surface water infiltration in the process of water-intake operation. To avoid the danger of underestimating the area of the SPZ, calculations of an SPZ taking account of the additional recharge of an operating stratum should be carried out only for chemical contamination, and if there are sufficiently accurate initial data on the parameters characterizing the intensity of the additional recharge. In evaluating the possibility of bacterial contamination, carried out for relatively small intervals of time, the water-bearing stratum should be considered in the calculations as isolated, i.e., the additional recharge is not taken into account.

Under conditions of overflow of groundwater from neighbouring aquifers, (Fig. 4.19b), the modulus of additional recharge is

$$\epsilon = K_0 \Delta S/m_0, \qquad (4.60)$$

where K_0 and m_0 — are the flow coefficient and the thickness of the layer separating the operating stratum from neighbouring aquifers, ΔS — is the difference of decreases in level in these strata. Total discharge of the water-intake in the diagram considered

$$Q = Q_e + Q_{nep}, \qquad (4.61)$$

where Q_e — is the discharge of natural flow in the operating stratum, seized by the water-intake; Q_{nep} — is the discharge of water from neighbouring aquifers.

For an approximate evaluation of the discharges Q_e and Q_{nep}, and of the boundaries of the SPZ, it is assumed that the configuration of the zone of capture of the water-intake in the given case is similar to the zone of capture in the isolated stratum and for compact groups of water-intake wells, and is expressed by equation (27), in which the distance x_b determined by the parameters of the overflow:

$$x_b = \frac{Q}{2\pi q(1+\upsilon)}\left[\frac{x_b}{B}K_1\left(\frac{x_b}{B}\right) - \upsilon\right], \qquad (4.62)$$

where B — is the parameter of overflow

$$B = \sqrt{\frac{K_0}{m_0}\left[\frac{1}{(km)_1} + \frac{1}{(km)_2}\right]} \qquad (4.63)$$

$(km)_1$ and $(km)_2$ — are the hydraulic conductivity of the operating and adjacent strata; $\upsilon = (km)_1/(km)_2$; q — is the linear discharge of a natural flow in the operating aquifer (it is taken that $q \approx \text{const}$); $k_1\left(\frac{x_b}{B}\right)$ — is Bessel's function from an imaginary argument of the first kind. The quantity x_b is determined from (62) by selection. The component of the water-intake discharge from the natural flow is $Q_e = 2dq$, while, in accordance with the premise accepted, $d = \pi x_b$. Accordingly

$$Q_{nep} = Q - 2\pi x_b q \qquad (4.64)$$

The quantity Q_{nep} constitutes the change in the intensity of groundwater overflow during the operation of the water-intake as compared with the overflow under natural conditions. To calculate the length of the SPZ upstream and downstream in the diagram considered,

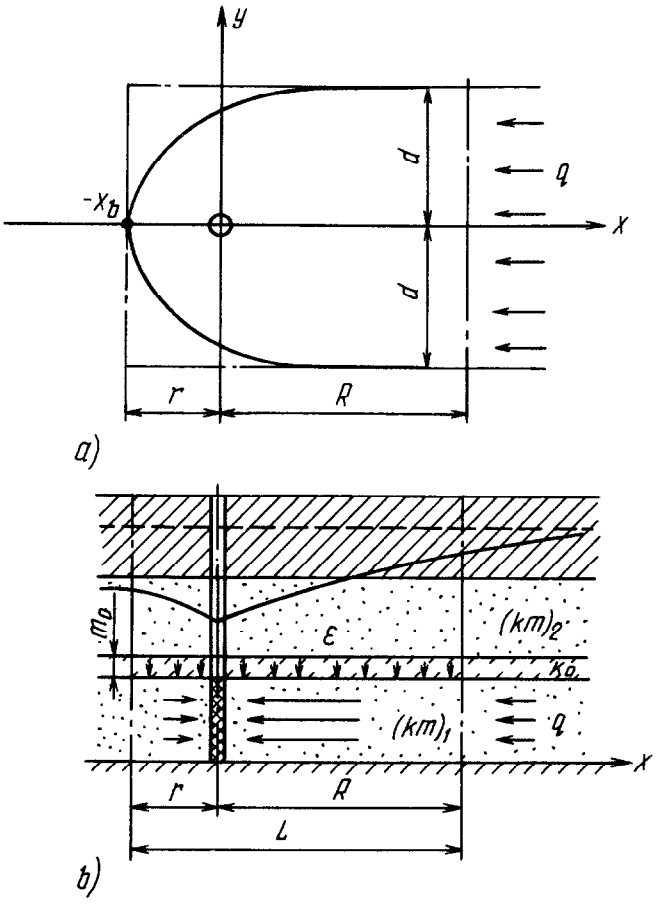


Fig. 4.19. Diagram of filtration to the water-intake distant from the river:
a) plane; b) cross-section for a stratum with overflow.

equations (32) and (33) and the graph in Fig. 4.11 are used, which is the same as for an isolated stratum, but with x_B, determined according to (62).

4.5. WASTE MANAGEMENT AND PROTECTION MEASURES AGAINST GROUNDWATER CONTAMINATION

4.5.1. PRINCIPLES OF DESIGN OF WASTE-MANAGEMENT PROGRAMMES

Because the Earth's environment is the ultimate receptacle for wastes, environmental protection and wastes management are associated, mutually affecting endeavors that must be considered jointly. Continuing population expansion and economic growth worldwide will produce even greater quantities of wastes requiring disposition [69].

The volume of urban sewage alone is projected to increase about 16-fold during the next few decades as sewerage systems are installed and extended [61].

The generation of wastes is a normal and unavoidable end product of human activities, and that the disposition of wastes inevitably accompanies every productive undertaking.

Full efforts to protect groundwater from wastes entail consideration of alternate disposal environments. Inasmuch as all Earth environments are susceptible to pollution, waste-management planning should be comprehensive enough to evaluate every potential disposal environment in order to yield the most favorable programme design.

The quantities and kinds of wastes generated are key considerations in the design of waste management programmes. Minimization of the volume and toxicity of wastes generated is obviously good basic policy from the standpoint of environmental protection. Inasmuch as waste creation to some levels is unavoidable, recycling and reuse of waste residuals may be among the most effective means of accomplishing these policy goals. Briefly stated, the term "residuals" refers to the residues of production, after all useful values have been extracted. Extracting the full values of wastes through recycling and multiple usage may yield, in addition to reduction in the volume of waste pollutants discharged to the environment, significant economic and resource-conservation benefits. Economical methods of reclamation of waste water generated by industry, agriculture, and cities, for example, may offer all three [73]. New and innovative ways to reduce the volume and chemical and biological hazards of wothless "residuals" are being sought.

Aquatic environments, including groundwater, are among the beneficiaries of reduction in magnitude and improved environmental assimilation of residual wastes of human activities. Inasmuch as the most effective way to protect groundwater quality is to avoid pollution in the first place, the concept of minimization of wastes source is especially applicable to groundwater protection.

The two basic classes of methods for protecting and improving the environment through residuals management are: (I) measures that reduce the volume and harmful characteristics of residues released to the environment, and (II) measures that make use of, and increase the capacity of, waste assimilative processes in nature. The table 4.3 subdivides these two principal classes and it lists practical examples of management procedures applicable to each subclass.

TABLE 4.3. PHYSICAL MEASURES FOR IMPROVING ENVIRONMENTAL QUALITY (AFTER ROWER – 4)

I. *Measures that reduce the discharge of residuals to the environment*

A. Measures for reducing generation of residuals
   1. Change raw material inputs
   2. Change production processes
   3. Change mix of product outputs
   4. Change product output specifications
   5. In-plant (on-site) recirculation of water

B. Measures for modifying residuals after generation
   1. Materials or energy recovery (direct recycle)
   2. By-product production (indirect recycle)
   3. Residuals modification without recovery for reuse of any material or energy
   4. In-plant recirculation of water

II. *Measures that utilize assimilative capacity of the environment*

A. Measures for making better use of assimilative capacity
   1. Redistribute effluent from a given activity over space and/or over time
   2. Change the time scheduling of activities
   3. Change the spatial location of activities

B. Measures for increasing assimilative capacity
   1. Add dilution water to water bodies
   2. Use multiple discharge outlets from reservoirs at various depths
   3. Artificially mix water in reservoirs and lakes
   4. Artificially aerate streams, lakes, and estuaries with air or oxygen, by surface or subsurface diffusers

If protection of groundwater quality is given high priority in the design of a waste management programme, the weight assigned to each of the above factors will need to be distributed accordingly. For instance, emphasis given to protection of groundwater quality may necessitate modification of the institutional structure and perhaps the sacrifice of some economic benefits in order to accomplish the special protective measures that strict protective policy requires. A reverse management policy that allows liberal use of groundwater reservoirs for disposal of wastes would require different (perhaps less) institutional controls and possibly enable economic benefits not attainable under greater restrictions on such usage of the reservoirs.

4.5.2. GROUNDWATER COMPONENTS OF WASTE PROGRAMME DESIGN

Many of disposal practices are employed for both solid and liquid wastes. Waste disposal practices may directly affect the groundwater resources.

Restoration of the chemical or bacterial quality of polluted groundwater generally is neither physically nor economically practicable. The porous texture of rocks and the inaccessibility of groundwater resources are among the features that hamper cleanup of the resource once it is polluted. For these reasons, *avoidance of pollution in the first place is the most effective management policy.*

However, avoidance of groundwater pollution is difficult and a certain amount is an inherent result of productive human activity. Technologically and agriculturally advancing nations may need to accept certain

minimum levels of groundwater pollution as inevitable. Policies and programmes of waste management should seek to minimize the harmful effects of surface activities on groundwater quality, with the recognition, however, that complete isolation from contaminants is unattainable.

With that caveat in mind, an effort was made to list in table 4.4 the principal methodologies for prevention, monitoring, control, and abatement of groundwater pollution.

Preventive measures. Measures taken to prevent groundwater pollution obviously must be applied at sources or potential sources of the pollutants. Because as polluting substances may be derived from, or transported by, the full suite of Earth environments — the atmosphere, oceans, land masses, and water in and on the land — a wide array of preventive actions may be necessary, including institutional as well as technological measures. Regional, interregional, and even international collaboration in the solution of mutual problems is a difficult but important step. Regional and international cooperation that may be required to cope successfully with acid rain pollution is a case in point. Many groundwater pollution cases are only of local extent, but collectively they are of regional concern and necessitate regional planning and protective actions.

Even after groundwater pollution has taken place, effective preventive efforts at the sources may limit the seriousness of the problem.

Monitoring provides information useful in hydrologic investigations and in the management of water, wastes, and pollution. The principal measures are listed in table 4.4. Although monitoring is often cited in the literature as a preventative or corrective measure for groundwater problems, it should be recognized as only a means of collecting information.

TABLE 4.4. PRINCIPAL METHODS AND MEASURES FOR PREVENTION, MONITORING, CONTROL, AND ABATEMENT OF GROUNDWATER POLLUTION

Methods	*Measures*
PREVENTION	**Prevention measures can be applied only at the sources of pollutants.**
A. Conducto hydrogeological studies and other investigations necessary for planning and design of preventive actions, monitoring, and control and abatement measures.	
B. Design and implement comprehensive land-utilization and waste-management plans that prevent or reduce pollution.	
C. Exercise control over atmospheric sources of pollution (saline rain, factory stack emissions, etc.) detrimental to groundwater quality.	
MONITORING	**Monitoring provides information**
A. Specify the purpose of the monitoring programme.	
B. Coordinate monitoring design with hydrologic knowledge of the groundwater system.	
C. Monitor major sources and potential sources of pollutants.	
D. Monitor the spread and advance of contaminants in both the unsaturated and saturated zones.	
E. Monitor water levels, pumpage, artificial recharge, and all other significant hydraulic influences on directions and rates of pollutant migration.	
CONTROL AND ABATEMENT	**Control and abatement measures are expensive and seldom fully successful.**
A. Investigate effectiveness of natural chemical and biological renovation processes on pollutants flowing through unsaturated and saturated zones. These processes usually are slow, but under favorable hydrogeologic conditions may be reliable, long-term control and abatement measures for groundwater pollution.	
B. Conduct tests of the effectiveness of artificially initiated **in situ** chemical and biological reactions in both the unsaturated and saturated zones in order to accelerate immobilization or neutralization of pollutants.	
C. Extraction, flushing, redirection, dilution, or containment of polluted groundwater through use of pumping and injection wells, infiltration ponds, hydraulic barriers and physical barriers to flow, and other water-control measures.	
D. Excavation and removal of contaminated portions of aquifers. May be practicable for small volumes of aquifer material. Suitable disposal site for excavated material must be available.	

Monitoring of surface activities, above the groundwater reservoir, may provide an early warning of potential pollution. This advance information may permit early preventative measures prior to contamination of the soil zone, unsaturated rocks, and groundwater reservoir, which is the most effective management step for protection of groundwater quality. Pollutants that threaten groundwater originate in and move through all Earth environments, and as in the case of preventative measures, a complex and extensive array of monitoring facilities may be necessary. Once pollutants are detected in a groundwater reservoir, monitoring emphasis shifts from advance warning efforts to tracking the movement and chemical state of the contaminating liquids. Following is a listing of the general types of data commonly acquired by monitoring programmes and necessary associated investigations:

- o Demographic and economic conditions and trends
- o Past and current land usage and trends
- o Water resources — trends in development, utilization, and disposition
- o Patterns of wastes generation and disposal, past, present, and potential
- o Atmospheric sources of pollutants
- o Soil, unsaturated zone, and groundwater hydrology — hydraulic, geochemical, and biological factors that influence fate and migration of pollutants
- o Surface-water hydrology — relation to unsaturated zone and groundwater in terms of hydraulic and geochemical factors influencing pollutant migration and chemical and biologic reactions

The intended purposes of the monitoring effort should be clearly identified and firm goals for the undertaking established at the outset. The reasons for establishment of a monitoring programme will be the primary influence on the scope and nature of the undertaking: rational design is not possible without a clearly identified purpose for the undertaking. Generally, the purpose of a local monitoring installation associated with a single potential source of pollution, such as a waste lagoon, is obvious. Clear justification for broader monitoring programmes intended to serve multiple purposes may be

more difficult to formulate. Often the purposes and goals of areal, regional, and national monitoring systems are vague; in the absence of satisfactory justification, the worth of the undertaking is suspect.

Control and abatement measures include physical, hydraulic, and biological or chemical methods to isolate, contain, divert, dilute, extract, or treat polluted groundwater. The principal methods are listed in table 4.4. The most common measures include efforts to eliminate polluted water by accelerated rates of extraction; by injection or infiltration of water at hydrologically strategic points in an effort to restrict the migration of, or flush out, the polluted water; and by installation by hydraulic or physical barriers to pollutant flow (Figure 4.20).

Figure 4.20c illustrates an impermeable subsurface barrier erected parallel to the coast in order to obstruct landward flow of saline water. The barrier may be constructed of sheet piling, poddled clay, emulsified asphalt, cement grout, betonite, silica gel, or plastics. Construction costs are high and the barrier is subject to damage from earth movements, erosion, and chemical attack and deterioration.

4.6. TYPES OF SPECIAL PROTECTIVE MEASURES TO CONTAIN AND ELIMINATE GROUNDWATER POLLUTION FOCI

Certain countries have already passed laws directed at protecting groundwater from pollution. This does not

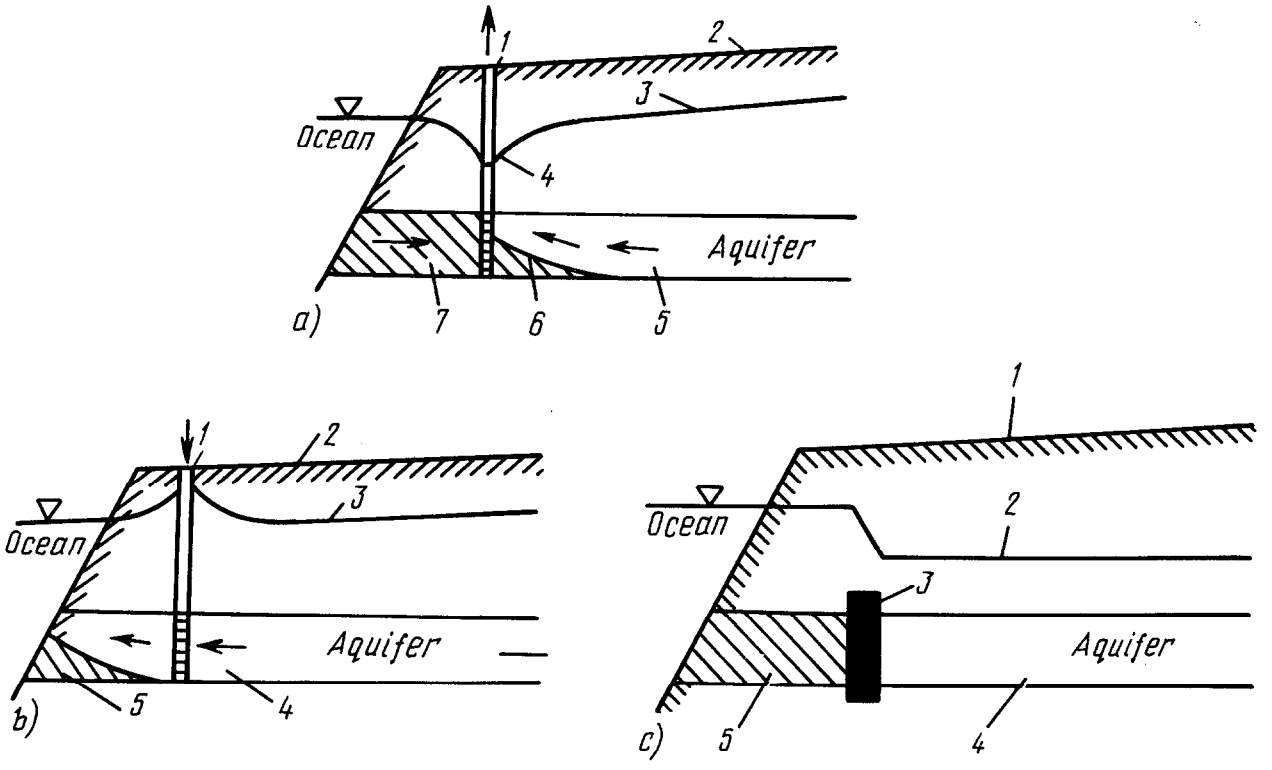


Fig. 4.20. Barrier control measures for seawater intrusion (after TODD, 1980).

The hydraulic theory on which barrier concepts are based is sound, but methods are expensive and results highly variable. The examples depicted in Figure 4.20 pertain to the control of seawater intrusion (Todd, [68]), but the principles illustrated apply to any groundwater contaminant. Figure 4.20a is a cross-sectional drawing across a line of continuously pumping wells, paralleling the coast, which form an elongate pumping trough (a negative barrier) that attracts both the intruding seawater and freshwater from the aquifer.

Figure 4.20b illustrates a reverse principle; freshwater is injected into the line of wells to form a pressure ridge (a positive barrier) paralleling the coast. As indicated by the arrows, injected water moves both seaward and landward, pushing the seawater tongue toward the ocean and recharging the freshwater part of the aquifer. The injection system requires high-quality water which normally must be conveyed to the area, usually at high cost.

mean, however, that special antipollution engineering measures are necessarily being implemented on the required scale. Such measures are required principally in regions where groundwater constitutes the main source of water supply and where its pollution implies a need for expensive groundwater treatment facilities or even for complete water intake shutdown.

Special protective measures are directed both at preventing groundwater pollution at construction sites as possible pollution sources, and at containing and/or eliminating pollution foci which have already arisen in an aquifer. These are various types of engineering structures whose nature, type and design are determined by hydrogeological conditions, by the nature of the pollution source, by the scale of the existing pollution focus, and by the special characteristics of the properties to be protected (parts of an aquifer, individual groundwater intakes, rivers, etc.). The choice of schemes and types of

special protective measures should be effected on the basis of protection-facility feasibility studies which must analyze and compare the scale of detriment occasioned by groundwater pollution on the one hand and the cost of various protection options on the other. Such studies must likewise consider the choice of alternative water supply sources, evaluate the availability of groundwater in the given area, and make proper allowance for the social and environmental aspects of the case considered.

In the case of local pollution foci, special protection measures are directed at: a) the isolation of pollution sources from the remaining parts of an aquifer by means of antiseepage walls, screens (including hydraulic screens) and shields of various types; b) the interception of polluted groundwater at the place of origin by means of vertical and horizontal drains. With area pollution by agricultural fertilizers, poisonous chemicals, antiskid materials spread on roadways and other similar factors, the requirement is to intercept the polluted rain and drain runoffs while restricting the use of the pollutants.

In order to prevent pollution and to protect unconfined aquifers from random leaks and industrial water seepage from production sites as well as leakage of liquid industrial raw materials and stored effluents, the first requirement is to ensure that the bases of such structures and storage tanks are finished with reliable waterproofing. It is advisable in many instances to provide drainage in the form of a sand layer bedded on a waterproof shield made of clay puddle, polymer film or some other seepage-proof material. In the event of accidental spills the waste water and industrial liquids are then intercepted by the filter elements of the drain layer and removed through drainpipes for processing in treatment facilities.

Line-type drains of similar design are provided along water-carrying conduits such as pulplines, pipelines and intrafactory transport trunk lines, i.e. wherever accidental leaks of waste and process waters are a possibility. The foundations of areas where loading, unloading and warehousing of soluble raw materials, products, and industrial and household wastes will be performed must also be equipped with a drain layer overlying a shield of tamped clay, clay puddle, bituminous concrete or other impervious material. With a shallow-bedded waterproof layer, individual structures or even an entire industrial site can be insulated from the remaining portion of the aquifer by means of an annular waterproof screen or wall running down to the waterproof layer and combined with drainage to carry away the infiltrating effluent and atmospheric precipitation (Figure 4.21). In the case of a thick aquifer, groundwater protection at an industrial site can be provided by a vertical drainage system consisting of abstruction boreholes. The depression cone must be large enough to cover all of the area in which leaks and polluted water seepage can occur. In order to ensure the removal of pollutants as fast as possible in the event of an accidental spill, the discharge of the drainholes near the spill must be increased; in the absence of major leaks the slightly polluted or practically clean water can be used as a source of industrial water supply. Monitoring of protection effectiveness and drainage system operation is facilitated by a system of observation wells permitting visual determination of groundwater levels and quality.

Torrential rainwater falling in the vicinity of industrial sites with elevated air and soil pollution is also subject to collection and treatment.

Sludge extractors, collectors and evaporators, hydraulic ash and salt dumps, and other industrial containers and

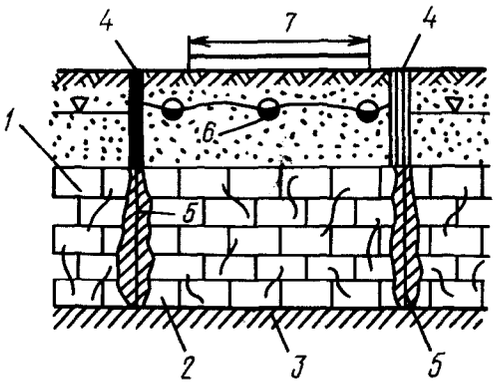


Fig. 4.21. Combined circular anti-filtration curtain and drainage for the protection of groundwater against infiltrating waste flows.
1 — sandy-gravelly rocks, 2 — hard fractured-water-permeable rocks; 3 — water-resistant rocks, 4 — anti-filtration partition; 5 — cement curtain; 6 — drainage; 7 — site of industrial premises — source of groundwater contamination.

pools for storing polluted water and other wastes must be seepageproof. In some cases such proofing can be ensured simply by finding a proper location. Generally, however, it is ensured by providing various antiseepage and drainage devices in the body, base or periphery of a tank, or, in the case of a sludge reservoir dam or dyke, at the reservoir bowl and tail bay. The antiseepage elements of dams and dykes (shields, harrows, upstream puddle blankets, cores, diaphragms) must be made of practically impervious materials such as polymer films, bitumen (asphalt) coatings, clay puddle, etc. Only in certain special cases is it permissible to have them made of compacted clay soil of extremely low water permeability. The antiseepage elements cutting across water-permeable ground must be reliably grouted. The drainage devices employed with sludge tanks and collectors (sloping, piped, and straight-line drains, holes, culverts etc.) in the lower part of a dam body or at a tail reach are intended to intercept any waste water which has managed to seep through the body, base, or sides of the dam or dyke for pumping into a sludge tank of closed-circuit industrial water supply system.

In order to ensure against polluted water seepage through the bottom of a sludge tank, collector or other industrial basin in cases of a shallow-bedded waterproofing, the antiseepage devices provided take the form of an annular impervious screen or shield with appropriate drainage (Fig. 4,22 a, b) with a deep-lying waterproof layer and a low natural groundwater table, a sheet shield made of waterproof material covered over with a protective layer of local soil is provided over the entire surface of the sludge container bowl (Fig. 4.22c). The use of almost but not entirely waterproof clay shields is permissible only when dealing with low-toxicity wastes whose seepage within certain limits can be considered tolerable in the given hydrogeological setting.

In certain instances one may use a hydraulic screen in the form of a water-filled trench running all the way to the waterproofing, or else of a row of rechange wells in which the water level must exceed the polluted groundwater level. One disadvantage of hydraulic screens is that clean water must be continuously pumped into them; another is that certain conditions call for the concurrent pumpout of both polluted and clean groundwater.

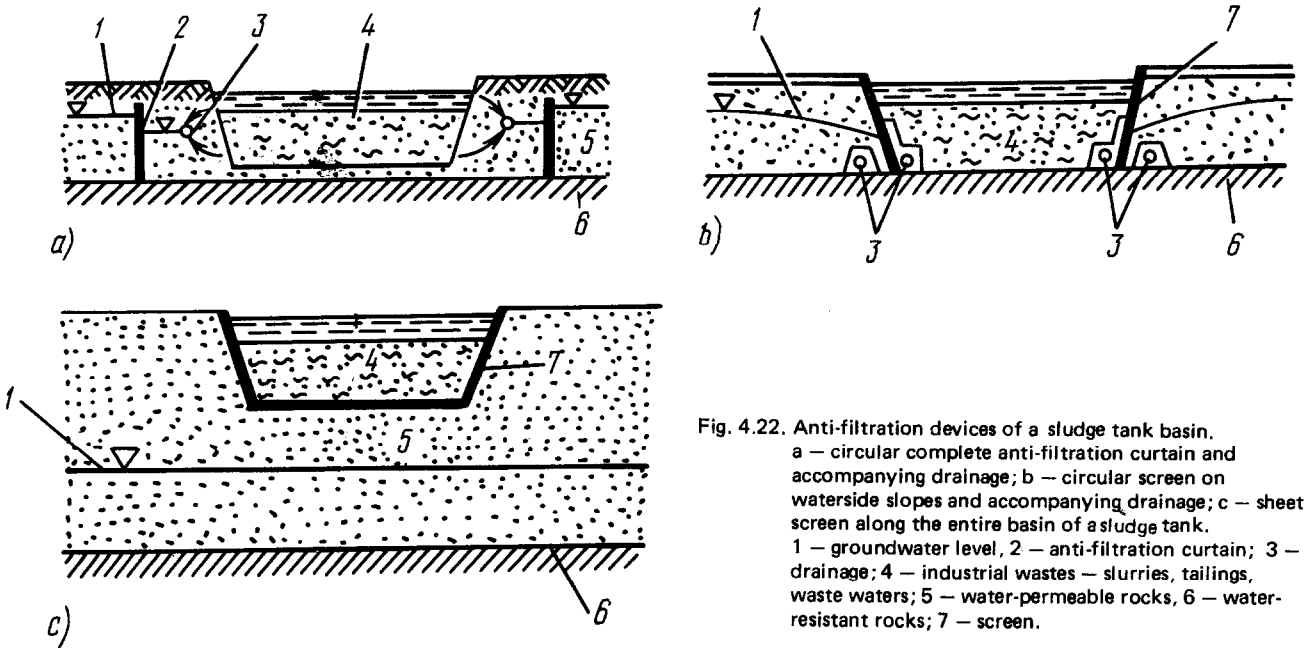


Fig. 4.22. Anti-filtration devices of a sludge tank basin.
a — circular complete anti-filtration curtain and accompanying drainage; b — circular screen on waterside slopes and accompanying drainage; c — sheet screen along the entire basin of a sludge tank.
1 — groundwater level, 2 — anti-filtration curtain; 3 — drainage; 4 — industrial wastes — slurries, tailings, waste waters; 5 — water-permeable rocks, 6 — water-resistant rocks; 7 — screen.

Vertical waterproof barriers placed in the path of polluted groundwater can take the form of injection screens. These are produced by the injection through wells of solutions which fill pores and cracks and thus render local rocks impermeable. Materials suitable for injection to produce a waterproof screen include cement, silicates, electrosilicates, resins, and clays. Freezing is another possibility. A further type of vertical screen is the barrage wall produced by extracting rock from the ground and replacing it with waterproof material. Antiseepage barrage walls can take the form of a continuous row of large-diameter boreholes or of a narrow trench filled with concrete, clay puddle, resin or other waterproof material. Thin barrage walls can also be made of ferroconcrete elements, i.e. sheet pilings which are either cemented into the ground or extracted, after which the remaining slot is filled with a cement-clay or other solution. It is also possible to provide a compound antiseepage barrier. In this instance the upper part of a water-bearing stratum consisting of noncoherent rock types is fitted with a wall, while the lower part of the aquifer consisting of coherent rock is furnished with an injection screen or curtain.

At sites where antipollution measures had not previously been implemented in a timely fashion and where groundwater pollution has already occurred, the most urgent step to be taken is usually that of removing the pollution source. This means banning the further use of seepage fields, dumps and other polluted water and waste accumulations, as well as redesigning and rebuilding of industrial plant, alteration of manufacturing processes, etc. However, in those instances where a pollution source has been massively polluting groundwater for quite some time, its simple removal does not eliminate the threat of continued propagation of pollutants throughout an aquifer. When this is the case what is called for is containment of the pollution focus which can be provided either by installing one or more of the various pollution barriers described above (waterproof barrages, antiseepage screens, hydraulic curtains), or else by creating a depression cone surrounding the pollution focus (Christensen [49]; Bochever, Lapshin, Oradovskaya [7] et al.). Set out below are the conditions which prompted the implementation of various protection schemes designed and installed at various sites in the USSR.

1) A high fluvial terrace above a flood plain is composed of alluvial sands with loam and loamy sand interlayers. The thickness of the alluvium does not exceed 10 m. Lying below are practically impermeable moraine boulder clays and loams. Groundwater pollution by petroleum products occurred in the vicinity of a carbon black factory. Groundwater laden with petroleum products seeped out along the steep river bank and polluted the river water which served as a water supply. Investigations carried out to determine the contours of the pollution focus included the drilling of exploratory holes, experimental bailing of such holes, prospecting pit filling, taking of ground samples from the aeration zone as well as of groundwater, samples which were then tested for petroleum product concentrations, geophysical studies, etc.

It was decided to stop the flow of polluted water into the river and to gradually evacuate the petroleum products present in the aquifer by means of a horizontal intercepting drainage system installed along the line of minimum bedding depth of underlying aquiclude (Fig. 4.23). The drained waters, after clean-up to remove the petroleum products, will be used as a process water supply for the plant. The production site was likewise equipped with technical facilities designed to prevent further petroleum product leakage.

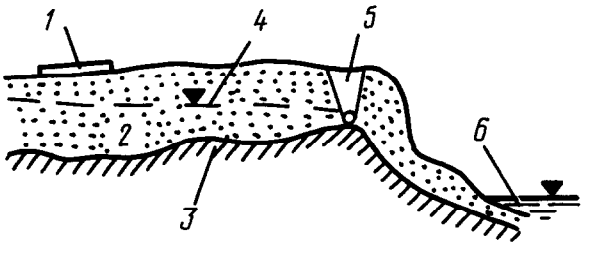


Fig. 4.23. Interception of contaminated groundwater by a horizontal drain.
1 — industrial sites; 2 — water-permeable rocks; 3 — aquiclude; 4 — groundwater surface; 5 — horizontal drain; 6 — river.

2) The pollution sources (Fig. 4.24) were the production area of a chromium compound plant (5), an unscreened sludge tank (4), and a hydraulic ash dump (2) situated on top of an alluvial terrace. The sand-gravel-shingle bed alluvial aquifer tapped for water supply purposes developed a focus of groundwater pollution by hexavalent chromium due to leaks from the production site and sludge collector tank. This resulted in the contamination of the nearest water intakes (6), (7), (8) and of the river draining the aquifer; also threatened were groundwater intakes situated downstream. Hydrogeological surveys and studies subsequently revealed the character and degree of groundwater pollution, suggesting a system of special protection measures directed at securing the polluted groundwater perimeter and polluted groundwater removal. The drainage holes selected where those of water intake (6) situated in the immediate vicinity of the industrail site, as well as a specially designed downstream line of boreholes (1).

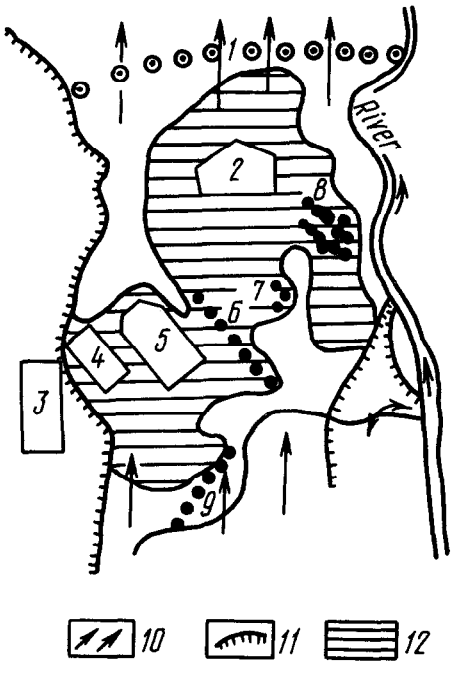


Fig. 4.24. Diagram of special protective measures in the area of contamination of an alluvial aquifer by chromium.
1 — drainage contour curtain; 2 — hydraulic ash dump; 3 — new screened sludge tanks; 4 — old unscreened sludge tank; 5 — industrial site of chromium compounds plant; 6 — water-intake used as protective drainage; 7, 8, 9 — groundwater intakes; 10 — direction of natural groundwater flow; 11 — boundary of a valley slope; 12 — source of groundwater contamination.

Predictive estimates of the velocity and arrival time of polluted water flow towards the drainholes as well as determinations of the concentrations and quantities of chromium associated with a given drainhole configuration were arrived at with the aid of hydrodynamic nets obtained by seepage simulation. The simulations made allowance for the variation of the hydraulic conductivity of the aquifer per unit area, seepage losses from the hydraulic ash dump and seepage fields, water uptake by operating water intakes and planned drainholes, hydraulic links between groundwater and river water, etc.

Following its purification and desalination, the most chromium-polluted water (pumped out at a rate of 8 000 m^3/day) will be put to use for technological purposes, while the slightly polluted water (27 000 m^3/day), will contribute to the industrial water supply following its purification by reagents. The seeping hydraulic ash dump (2) will be eliminated, and new sheilded sludge collectors (9), constructed of waterproof clays have already been installed on the bedrock bank.

3) Unpurified industrial effluents from a synthetic rubber factory were discharged for a substantial time onto seepage fields (1) situated close to a river on an alluvial terrace consisting of a thick (45–50 m) layer of alluvial and fluvioglacial sand (Fig. 4.25). The seepage value for the upper part of the alluvium was 20 m/day, and that of the lower part 30–40 m/day. The alluvial aquifer is under considerable pressure as a source of domestic drinking and industrial water.

At the time when the seepage fields were provided for it was assumed that effluents to be dealt with would undergo natural biological purification. Observations showed, however, that the surfactant emulsifiers present in the effluents exhibited low oxidability, which meant that pollution rapidly took over the entire ground layer underlying the seepage fields, eventually spreading towards intakes (2), (3) and (4). Water intake (2) was put out of commission entirely, while at water intake (4) it was the wells lying closest to the river which became polluted. As a consequence of hydrogeological studies and determinations of the degree of groundwater pollution the seepage fields were decommissioned. On the other hand, the existing groundwater pollution focus, which was of large size and which presented a threat to operational water intakes (3) and (4) as well as to a new planned intake, had to be contained and eliminated.

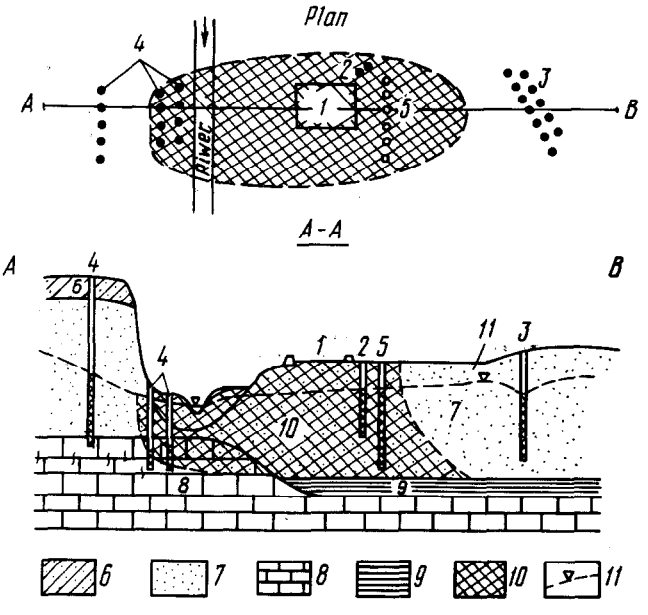


Fig. 4.25. Diagram of the localization and elimination of a center of groundwater contamination in the vicinity of filtration beds.
a — plan, b — cross-section
1 — filtration beds; 2, 3, 4 — groundwater intakes; 5 — protective drainage; 6 — sandy-clayey thickness; 7 — sands; 8 — limestones; 9 — layer of clay; 10 — contaminated groundwater; 11 — groundwater level

Hydrogeological studies, seepage simulation, and forecast-type mass transfer calculations led to the selection of a drain-type protection system with evacuation and subsequent treatment of the polluted groundwater in order to eliminate the surfactants. Following consideration of seven versions of such a protective drainage system it appears that the option of choice is a rectangular hole system (5) placed near the contaminated water intake (2). The drain discharge rate is to be 20000 m^3/day. The drainage configuration and operating mode were chosen on the basis of the following criteria: maximum pollutant removal over the first few years of system operation, shortest possible time of water decontamination, and minimum evacuation rate.

4) On emerging from a gorge a river traverses a debris cone constructed of pebble and boulder deposits. The thickness of the high-permeability rocks of the debris cone (the seepage factor of its upper part being 40–160 m/day and of its lower portion 20–50 m/day) increases rapidly with distance away from the head of the debris cone where it constitutes 10–20 m/day and towards its central portion (100–200 m and over). The pebble and boulder deposits contain fresh groundwater fed by seepage from the river at the head of the debris cone, as well as by the losses from the main canal and irrigation ditches. The river drains groundwater a small distance away from the debris cone head. A hydrogeologically oriented feasibility study of a chemical plant site to be used for agricultural fertilizer production had not been carried out, with the result that the facility was situated incorrectly, at the debris cone head where the groundwater stream is formed (Fig. 4.26). The leaks of waste effluent at the plant site as well as from collector tanks for domestic sewage and nominally clean water which nevertheless contained high levels of nitrogen compounds led to intensive pollution of the local groundwater, aggravated by high rock permeability and the considerable slope of the local topography. The sector with the most polluted groundwater situated under the plant site and downstream from it has an area of 4 km^2; the total area polluted amounts to about 30 km^2. The area engulfed by polluted groundwater is advancing downstream in the direction of a household drinking water supply intake (8). The presence of the groundwater pollution focus makes it impossible to profit from the entire fresh groundwater deposit which exists within the debris cone on the left and right banks of the river.

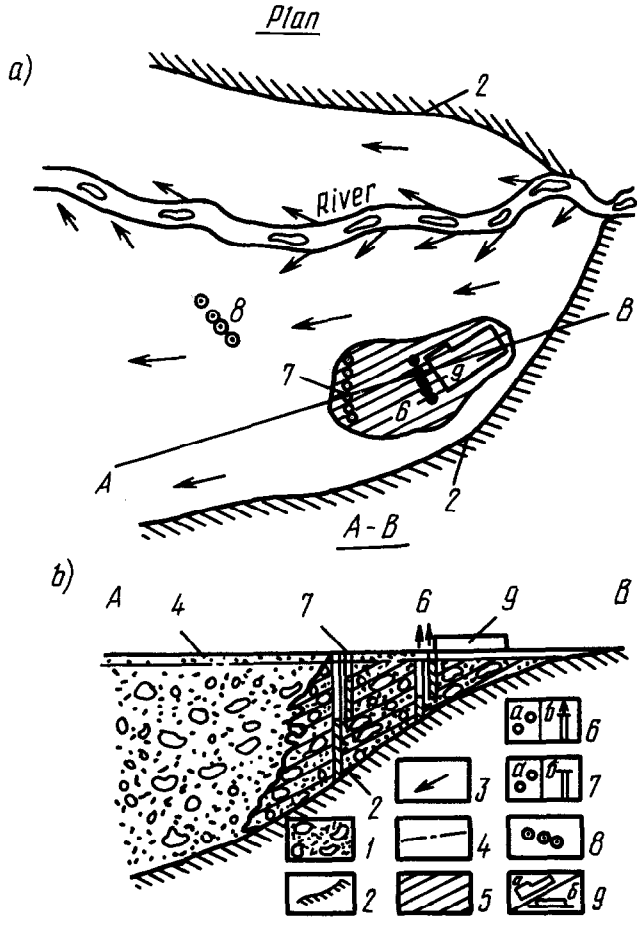


Fig. 4.26. Localization of the source of contamination of the groundwater of an alluvial fan
a — plan, b — cross-section
1 — gravelly-pebbly-rubbly deposits of the alluvial fan; 2 — boundary of the water-permeable rocks; 3 — direction of groundwater flow; 4 — groundwater level; 5 — area of propagation of the most contaminated groundwater; 6 — intercepting wells; 7 — protective wells; 8 — domestic-drinking water intake; 9 — chemical plant

Upon the completion of hydrogeological studies which included, among other measures, lengthy test pumping of wells of varying depth situated near the industrial site, a plan to contain and then eliminate the pollution focus was devised. In addition to general engineering measures (stoppage of waste effluent leaks onto the plant site, surface relocation of effluent pipelines formerly located underground, purification of waste and nominally clean waters, refurbishing of the household waste cesspool, termination of gas and smoke release into the atmosphere, etc.), the plan includes the installation of an interceptor water intake immediately downstream of the plant, with a protective water intake still further downstream. The interceptor intake is to take the form of a straight row of wells providing for polluted water withdrawal from two aquifer thickness intervals, namely from the groundwater surface down to a depth of 30 m and from the 30–50 m interval. The protective intake is also being built in the form of a straight row of wells, and will have groundwater withdrawal intervals of 50–70 m and 80–100 m. The protection of the household drinking water intake will be ensured by supplementary aquifer feed from open influent seepage pools now being excavated near the intake, as well as by a reduction in the depth of the water intake wells. The effectiveness of all of these protective measures will be evaluated by means of a special observation network. This will make it possible to develop a more precise notion of the required well configuration and interceptor and protective well placement. After purification, the polluted groundwater removed by the interceptor row will be used as a process water supply to the plant. The water from the protective row will be fed into filterless evaporators situated outside the debris cone.

The polluted groundwater interception system now being planned for the area of Rocky Mountain Arsenal in the state of Colorado in the USA (see Water and Water Eng., 1980) will provide for treatment of the water drawn from the local aquifer for subsequent return to that aquifer. The chief components of the system are as follows: 1) 54 drainage wells interconnected by pipes and equipped for selective well activation to make possible separate cleanup of three zones where the groundwater is known to be polluted; 2) a hydraulic screen of 38 injec-

tion wells to contain the purified water; 3) an antiseepage wall running down to the base of the aquifer to prevent "blotting up" of the purified water which is pumped in; 4) treatment equipment consisting of granulated activated charcoal filters to remove organic pollutants and activated aluminum columns to remove fluorides; 5) observation wells installed at various points throughout the system which make it possible to maintain surveillance of its operation. A system installed in Switzerland to protect an alluvial aquifer from petroleum products in the vicinity of an oil refinery includes: 1) drainholes running down to the impervious bed; 2) horizontal drains intercepting the topmost portion of the groundwater flow; 3) antiseepage walls either running all the way down to the impervious bed or else stopping short of it. (Dracos Th. [50]). Analysis of projects are designs for pollution focus containment and elimination shows that barrier-type devices are not an effective means of controlling pollution when one is dealing with a pollution focus of large size affecting a thick aquifer; moreover, the creation of a depression cone poses the difficult problem of utilization and/or expensive treatment of the polluted waters which are pumped out. Notwithstanding cost/benefit optimization of various pollution control measures and facilities, these remain an expensive proposition, a fact attributable largely to the need for treating the pumped water and to the large size of pollution foci. For the same reasons, the reestablishment of groundwater quality through the total removal of pollutants (rock excavation and removal, flushing of the aquifer with clean water and cleansing solutions, etc.) is a practical prospect only when smaller pollution foci are involved.

4.7. MEASURES FOR THE PROTECTION OF GROUNDWATER AGAINST DEPLETION

As pointed out earlier in the present monograph (see Chapter 3), the steadily increasing use of groundwater in many parts of the world is resulting in the gradual depletion of groundwater reserves. This process is a particularly rapid one in arid zones, where large volumes of groundwater go into irrigation, as well as at many centralized water intakes situated in platform-type artesian basins. The exhaustion of supplies in an aquifer necessitates the cutting back of water uptake and thus engenders difficulties in water supply. Another possible consequence, which arises when a considerable drop in the dynamic water table occurs (such a drop may range from several dozen meters to a few hundreds of meters) leads to a deterioration of operating conditions, increased water costs, and a lowering of groundwater use efficiency. Quite often a water shortage will provoke groundwater usage at a rate exceeding usable groundwater reserves, which in turn tends to draw in brackish water and to give rise to other undesirable processes. Aquifer depletion along sea and ocean coasts entails its invasion by salt water (this has occurred in Tallinn, Pyarnu and Klaipeda on the Baltic Sea, at Krasnoperekopsk in the Crimea, and at various other locations along the Black Sea coast in the USSR, in the Los Angeles and New York City areas in the USA, along the North Sea coast in the Netherlands, at the port of Latakia in Syria, and elsewhere). Groundwater depletion thus constitutes a serious water supply problem in various parts of the world.

4.7.1. ARTIFICIAL REPLENISHMENT OF GROUNDWATER STORAGE AS AN ACTIVE FORM OF GROUNDWATER PROTECTION AGAINST DEPLETION

One of the principal ways of combatting aquifer depletion and pollution is artificial groundwater replenishment. This term refers to an entire range of engineering measures directed at increasing groundwater feed, the enhancing or maintaining usable stocks available in an aquifer or groundwater deposit, as well as improving groundwater quality [40]. The desirability of implementing artificial groundwater replenishment has to do with the comprehensive character of this approach and with the various advantages which it offers as a highly effective solution to problems involved in the rational use and maintenance of groundwater in the context of a given economy. The range of groundwater protection measures directed against groundwater depletion includes the passive expedient of reducing groundwater uptake. This will not be discussed in the present monograph.

As for artificial groundwater replenishment, it is a set of measures which in most cases enables one to counteract depletion processes and to ensure normal operation of water intakes, or, at the very least, considerable extension of their useful lifetime.

In considering artificial groundwater replenishment from the standpoint of antidepletion protection, it is necessary to make full allowance for the distinction between surface flow and groundwater seepage: the latter entails seepage resistance which impedes water flow in an aquifer [26]. This is why in order to obviate high losses in terms of subsurface water and low-efficiency operation of influent seepage installations, artificial replenishment must usually be conducted near groundwater intakes or at least within a water intake depression or upstream from the intake. This has an important implication, namely that problems of intensifying groundwater feed and countering groundwater depletion must be considered with reference to a specific scheme whereby the water taken up by an intake is disposed of. The prospect of creating artificial groundwater storage without simultaneous or prior groundwater withdrawal does not therefore appear to be very realistic, except in the special instance of artificial formation of groundwater lenses in deserts.

The minimum thickness of the water-free portion of a ground aquifer must be assumed to be approximately 10 m with a seepage factor of not less than 15–20 m/day. In other words, the minimum hydraulic conductivity of the aeration zone in the horizontal direction must be not less than 150–200 m^2/day [20]. Roughly the same hydraulic conductivity must characterize the replenished aquifer. The possibility of artificial replacement of the water contained in artesian aquifers (i.e. of creating artificial elastic storage) is determined by the hydraulic conductivity being not less than 100–150 m^2/day for a single stratum and 250–300 m^2/day for multilayer aquifers, provided that the pressure head does not fall below its normal value by more than 30–40 m, given a fairly low value of the elastic water yield. The lower limit of the gravity yield should be set at 2–3 %. If the replenished aquifer is quite thick (20–30 m and over), data concerning the variation of seepage properties over its cross section must be furnished.

Geomorphological and climatic factors play an important role in the assessment of the feasibility of artificial groundwater replenishment. Rugged terrain precludes the installation of major influent seepage facilities (pools, canals, etc.), just as it precludes the delivery of

water to such facilities along open canals, flumes, etc. On the other hand, slopes ranging from 0.01 to 0.05 are conductive to influent seepage. The amount and probability of atmospheric precipitation, evaporation rate, saturation deficit, wind direction and velocity, and the thickness and persistence of snow cover are all factors which have a bearing on the character of influent seepage facilities and their operating modes. The most salient of these factors are the behavior of the temperature curve in the low-temperature range, the duration of periods with a mean temperature below -10 to $-15\ ^{\circ}C$, together with behavior at extremely low temperatures, which may be relevant for seepage facility design.

The problem of the artificial replenishment feasibility cannot be resolved without identifying the artificial replenishment source, i.e. the source from which the replacement water will be drawn [39]. The principal water replacement source is river flow. Other possible sources are ephemeral surface waters in arid and semiarid regions which flow during a brief but intense spring flooding period; local water resources (torrential-rainfall and snowmelt waters, etc.); mine drainage waters, as well as condensation plant and air conditioning waste waters, water from water-level lowering installations, etc., provided that these are of sufficiently high quality.

Another important factor which must be taken into account when evaluating replenishment feasibility is water quality. This applies both to the groundwater and to the water delivered to the seepage facilities. Depending on the degree of self-purification which might be expected to occur in the aquifer, the water supplied to seepage facilities may need to comply with existing drinking water quality standards such as GOST 2874–73 in force in the USSR.

In order to create a regional hydrogeological planning basis and to introduce artificial groundwater replenishment systems into the national economy, it is advisable to undertake preliminary zoning of the territory in question with respect to artificial replenishment feasibility. Such zoning is a prerequisite for more detailed prospecting operations in general and is an especially important requirement in the case of regions with an overall deficit in water resources or where aquifer depletion is an obvious fact.

Zonation following the procedure established in the USSR and developed by the author of the present section [39] is based on analysis of the following factors: the presence or absence of a collector, i.e. a subsurface "vessel" capable of being filled by influent seepage and subsequent topping up with certain volumes of water, or else of a highly water conductive stratum permitting higher-intensity water intake feed; the need for artificial groundwater replenishment; the presence or absence of a replenishment source, i.e. of volumes of water which can be used for injection into the aquifer and the economic feasibility of implementing artificial groundwater replenishment measures.

The presence or absence of a natural collector (subsurface volume) must be considered as an absolute consideration, for if no such volume (a lithological-stratigraphic rock layer possessing appropriate seepage and water-holding properties and exhibiting certain appropriate boundary characteristics) is not available, its artificial creation is virtually impossible, entailing as it would the artificial provision of several million or even tens of millions of cubic meters of water-holding capacity.

The need for artificial replenishment as dictated by both present and future needs is a variable factor, since growth in production activities may give rise to it in places where no such need existed previously. Moreover, artificial replenishment must be viewed in broad terms, as an option which offers a) the elimination of water shortages through the creation of new artificial water supplies; b) groundwater protection against depletion; c) improvement of water quality; d) the securing of material benefits. Artificial replenishment is less costly than the building of a river water purification plant and constitutes a saving in valuable agricultural land which would have to be sacrificed to a water intake installation facility. This list of benefits could be extended.

The presence of replenishment sources which might be collected in underground containing rocks is a variable. In areas with no replenishment source but with conditions otherwise conducive to replenishment, the necessary water can be delivered from quite remote outside locations. World experience and hydraulic engineering practice (e.g. in the USSR and the US) shows convincingly that it is indeed possible to transport large amounts of water (tens and hundreds of cubic meters per second) over distances of 500–1000 km and even further.

The economic effectiveness (cost/benefit competitiveness) of this method in comparison with other methods of coping with specific water resources problems, including the protection of groundwater against depletion, is also variable, since it depends on advances in engineering and evolves along with the latter. Allowance for the cost/benefit factor in territorial zonation on the basis of artificial groundwater replenishment conditions is one of the more difficult propositions, sinc, unlike cost/benefit comparisons of possible solutions to economic problems for a given unit (town, plant or factory, etc.), the assessment of the economic advisability of implementing artificial replenishment on a regional level entails accounting for a very large number of disparate influences which it is often difficult to interpret in an unambiguous way.

It is precisely the interrelation of those factors, whose relative importance can vary in accordance with the hydrogeological, climatic, economic and other conditions prevailing in each specific case, that enables the assessment team charged with special-purpose zonation to confirm their identification of particular taxonomic zoning ranks when considering large territories characterized by a great variety of natural settings and conditions.

By virtue of these criteria, the zonation procedure entails making three maps. These are the map of the principal aquifers which appear promising from the standpoint of artificial groundwater replenishment, a groundwater replenishment source map, and their resultant, namely the map known in the USSR as the "Zonal map of artificial groundwater replenishment conditions". The zonation and mapping effort for the USSR was completed by the mid-1970s, which means that the above three map types exist for the entire territory of the Soviet Union on a scale of 1:2 500 000. The example here is a fragment of a map of the principal aquifers in the northern part of the European USSR which are promising from the standpoint of artificial groundwater replenishment (Fig. 4.27). The zonation program just described made it possible to delineate regions where the implementation of artificial groundwater replenishment, especially for the purpose of antidepletion protection,

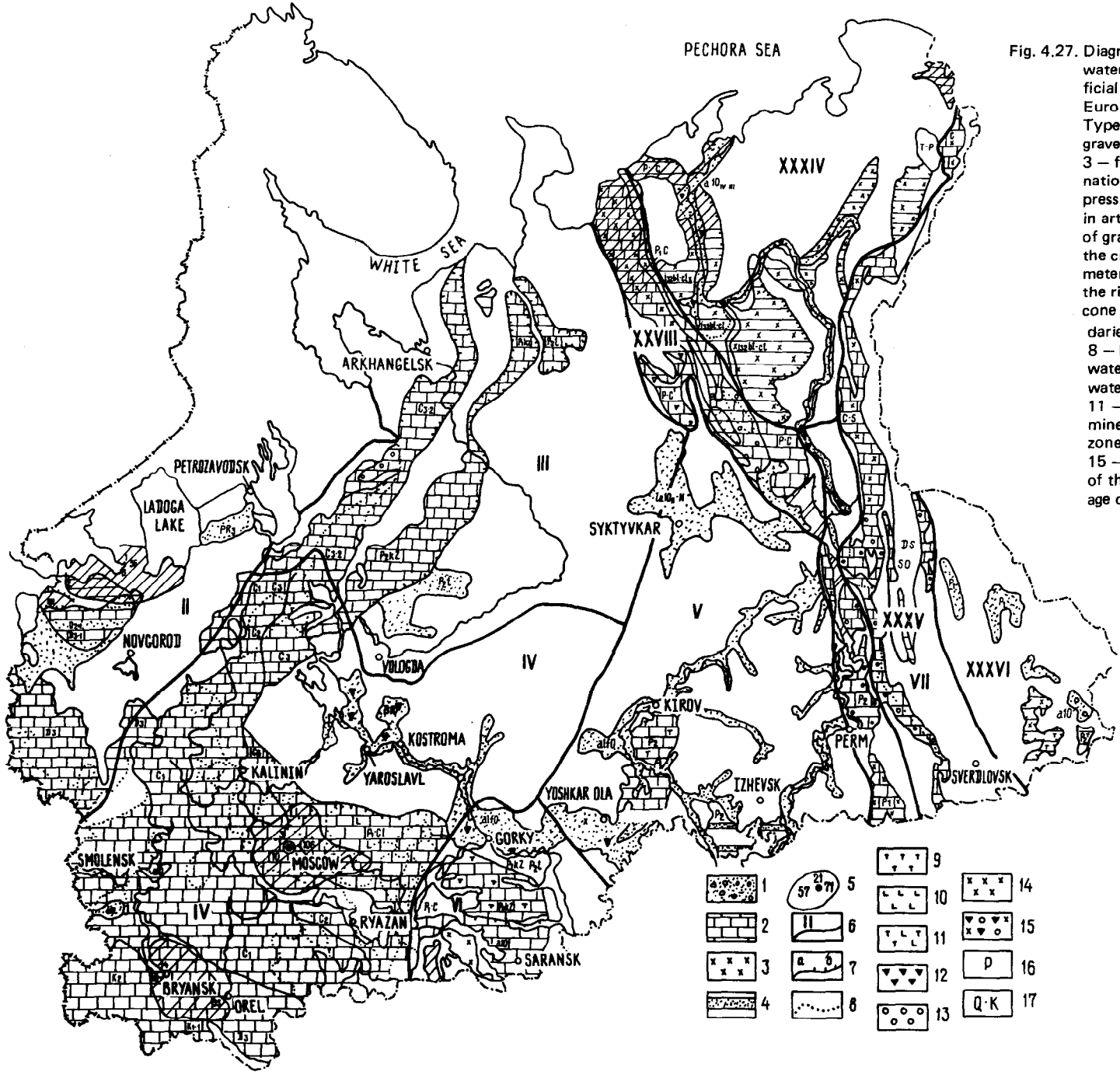


Fig. 4.27. Diagram of the distribution of the main aquifers and water-bearing formations which are promising for artificial groundwater recharge in the northern part of the European territory of the USSR.
Type of collector: 1 — porous in loose deposits (sands, gravel, pebbles); 2 — karstic, chiefly in calcareous rocks; 3 — fractured in rocks of varied composition; 4 — alternation of various types of collectors; 5 — zones of depressions formed in the operation of large water-intakes in artesian basins of the platform type; 6 — "the center of gravity": of the water uptake, the numbers around the circle — on the left — the lowering of the level in meters as of 1.1.78 in the center of the depression, on the right — the approximate capacity of the depression cone in millions of m^3 as of the same date; 7 — boundaries of hydrogeological regions of the first order; 8 — boundaries of aquifers promising for artificial water recharge; 9 — boundaries of propagation of waters with varying mineralization; 10 — 1-3 g/l; 11 — more than 3 g/l; 12 — waters with variegated mineralization; designations of the thickness of the zone of aeration; 13 — 5–15 m; 14 — 15–30 m; 15 — more than 30 m; 16 — values of the thickness of the zone of aeration from 5–30 m and more; 17 — age of the aquifer; 18 — age of interrelated aquifers.

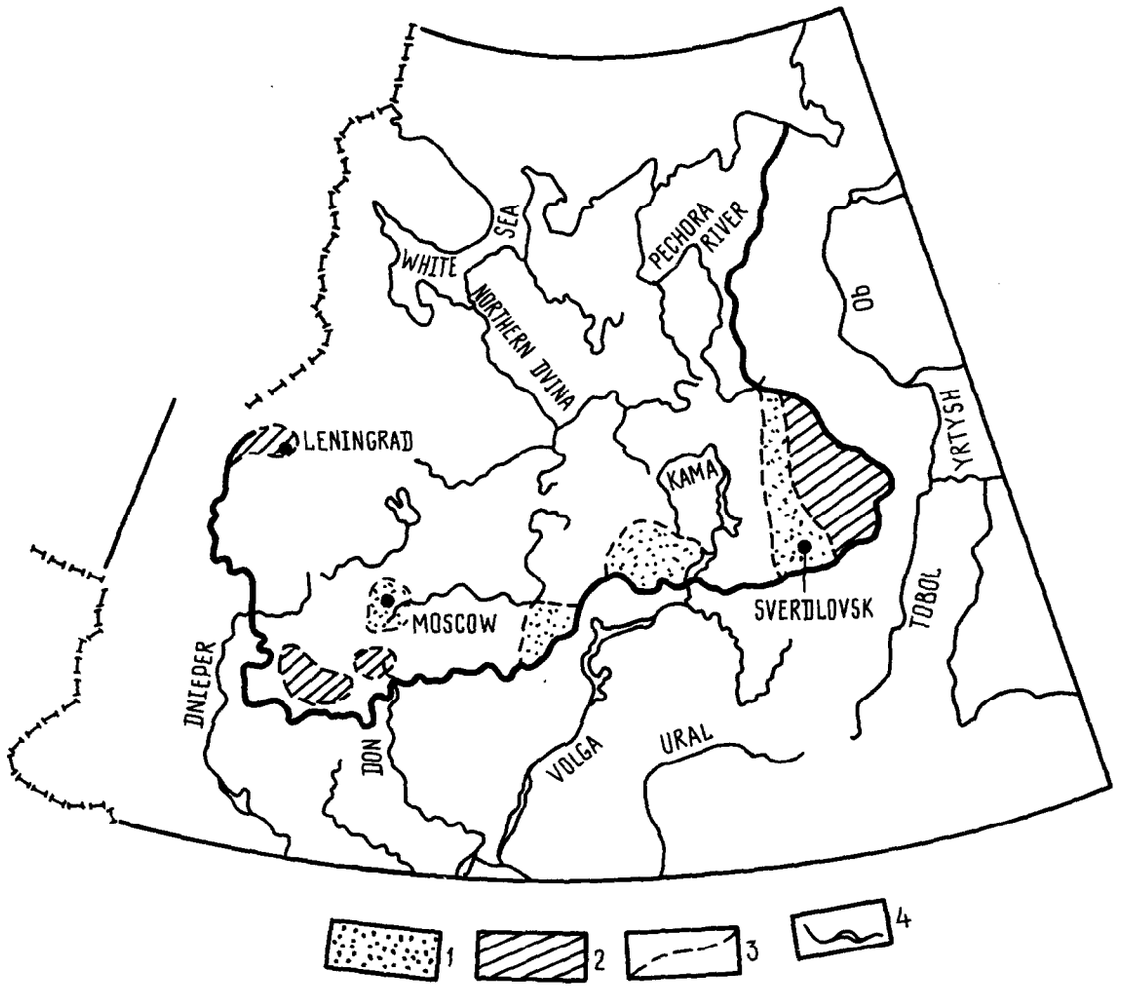


Fig. 4.28. Diagram of distribution of regions in the northern part of the European territory of the USSR promising for the use of artificial groundwater recharge in the near future: 1 — regions where the distribution method will predominate; 2 — the same with the injection method predominating; 3 — boundaries of promising regions; 4 — boundary of the area

appears to be most timely. Figure 4.28 illustrates this with a map of such regions situated in the northern part of the European USSR.

The assessment of possible replenishment sources entails their detailed description in terms of availability over a period of several years and within a single season of the year. When the replenishment source is surface water, the requirements as to its quantitative description is the same as when describing surface waters in the prospecting and assessment of seepage water intake supplies. In the case of mine water, condensation water from air conditioning equipment and purified industrial and domestic effluents, representative data are required concerning the steadiness and periodicity of operation of the water sources. As already noted, one of the main indicators of the quality of water used for artificial water replenishment, a factor which has a direct bearing on the output of influent seepage installations, is the turbidity of water. The permissible turbidity in the case of open systems is 20 mg/liter; for closed systems it is from 2 to 4 mg/liter, the extreme allowable concentration being 5 mg/liter. This explains why the principal source of water for artificial replenishment is river flow. Figure 4.29 illustrates this with a map of the mean annual turbidity of river water for the nothern part of the European USSR [44].

The two chief methods of artificial replenishment used in the world today are surface spreading and pumped injection and their various modified versions (Table 4.5). The spread method is used to replenish groundwater in phreatic aquifers when the top part of the aeration zone is composed of high-permeability deposits or where the overlying low-permeability loam or clay layer is no more than 3–4 m thick. The influent seepage facilities provided in such cases are referred to as being of the "open" type. The pumping method is used to force water into artesian aquifers, or else when the surface has a thick (10–12 m and thicker) layer of low-permeability rocks and soil. The spreading method can be implemented in various ways such as the provision of seepage pools, canals, ditches and pits; the flooding of selected areas of the natural surface or of specially prepared (e.g. furrowed) patches; the clearing of permanent and intermittent watercourse channels for the purpose of intensifying influent seepage from a river (this is the "riverbed method").

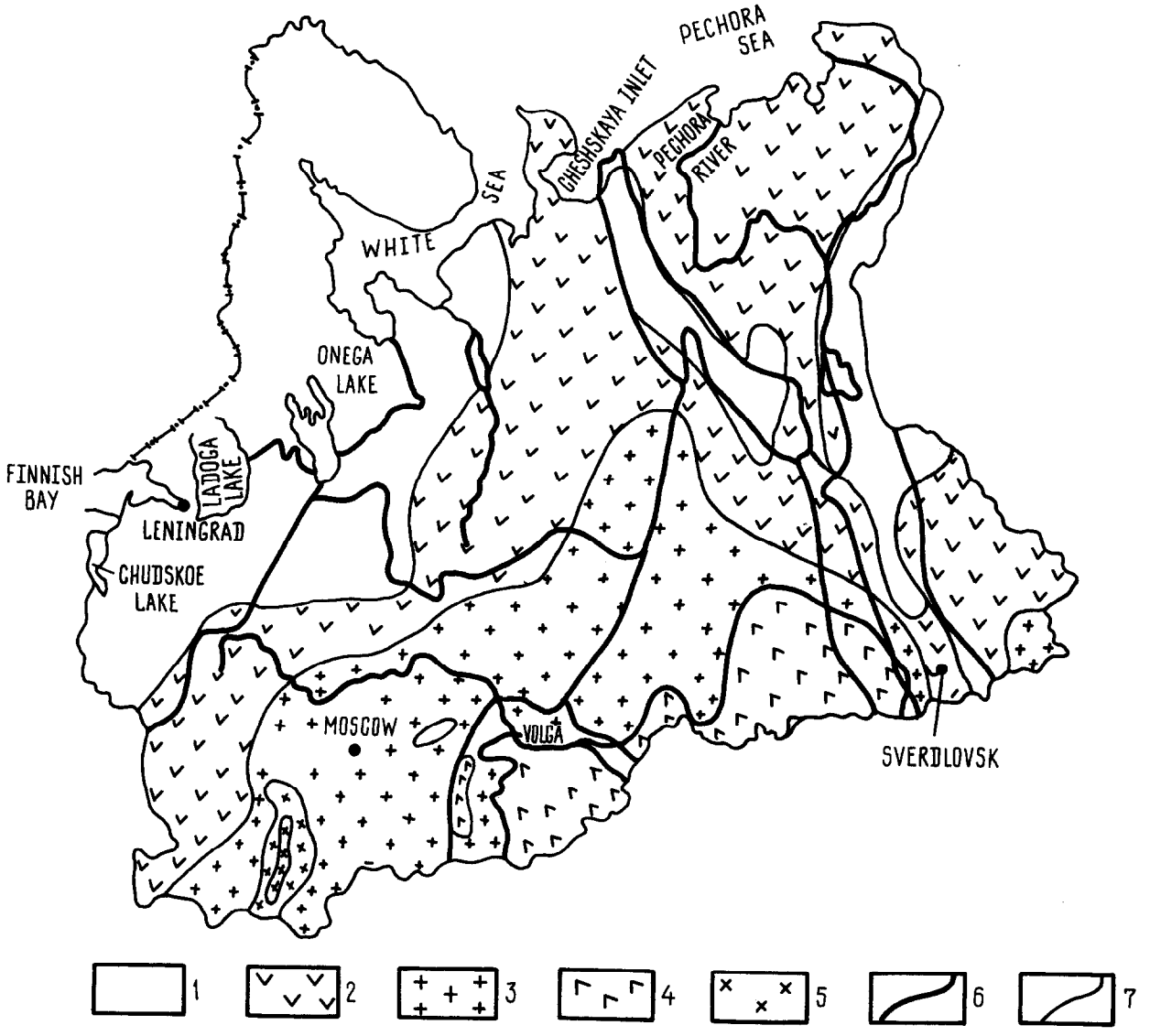


Fig. 4.29. Diagram of the mean annual turbidity of river waters of the northern part of the European territory of the USSR (according to G.I. Shamov [30] with clarifications and additions by the author). The turbidity of water in rivers:
1 – up to 20 mg/l; 2 – from 20 to 50 mg/l; 3 – from 50 to 100 mg/l; 4 – from 100 to 200 mg/l; 5 – 200–500 mg/l.
Boundaries: 6 – of hydrogeological regions (artesian basins, shields, systems of fractured-karstic waters); 7 – of zones with varying mean annual turbidity of river waters. The territory of the Kaliningrad region is not shown on the map.

Table 4.5

GUIDE TO PROPER SELECTION OF ARTIFICIAL GROUNDWATER STORAGE REPLENISHMENT METHODS

Type	Facilities or measures	Result	Conditions of implementation
1	2	3	4
A. Free influent seepage by water spreading	1. Pools (pits, open workings) a) capital-construction type b) lightweight-construction type 2. Ditches and furrows 3. Canals, trenches 4. Banked seepage areas	1. Buildup of artificial storage 2. Formation of artificial stocks 3. Water quality improvement (in some cases)	Phreatic aquifers of groundwater deposits in the following basic settings: river valleys, sand massifs, fluvio-glacial deposits above moraine, bounded structures in fissured and fissured karst rocks. The thickness of the low-permeability cover must not exceed 5–6 m

Table 4.5, continued

1	2	3	4
	5. Riverbed method (creation of pools, reservoirs and other water bodies) a) capital-construction dams b) ferroconcrete backwater dikes c) rock fills d) earth dams and banks	1. Surface flow regulation 2. Buildup of artificial storage 3. Artificial reserves formation	Groundwater contained in the alluvial deposits of river valleys with permanent and intermittent flow
	6. Flooding of a) areas of operating well deployment b) brackish lagoons, meadows c) gullies, small flat-bottomed valleys d) dead river channels, lakes 7. Land irrigation, sprinkling	1. Buildup of artificial storage 2. Buildup of artificial reserves	Groundwater in the principal types of groundwater deposits
B. Influent seepage under pressure by water pumping	1. Pumping into holes, wells and bore pits	1. Enhancement of elastic storage 2. Creation of "hydraulic barriers" to impede invasion by marine, or saline and/or polluted groundwater 3. Water quality improvement (in some cases)	Artesian aquifers of platform-type basins and folded areas with intermoraine fluvio-glacial deposits, old buried valleys, interpermafrost and subpermafrost pillars
	2. Decantation of groundwater from one aquifer to another via unloading boreholes	1. Increases in natural (elastic) storage 2. Water quality improvement (in some cases)	Presence of a pressure head drop between two adjacent aquifers
C. Improvement of natural groundwater feed	1. Snow retention in feed area	Increased influent seepage of snowmelt water and natural stocks formation	Presence of high-permeability cover and relief conducive to snow retention
	2. Improvement of the filtration properties of the covering ground a) harrowing (loosening) b) bulldozing of low-permeability layer c) flushing of silted layer by means of a hydraulic motor	Improved rates of seepage of rain, snowmelt and surface water	Thin impervious layer, largely loam or loamy sand, with high-permeability underlying layer
	3. Rivet channel widening	Increased influent seepage area	River valleys with channels bedded in loose alluvial deposits
D. Regulation of subsurface runoff and groundwater evaporation	1. Underground dams, waterproof screens	1. Cutoff or reduction of groundwater outflow beyond water intake area 2. Increased aquifer thickness or pressure head 3. Protection against the "intrusion" of seawater and the attraction of saline and/or polluted groundwater	Shallowness of ground or artesian flow; restricted seepage cross section coupled with considerable slopes and seepage rates
	2. Pumping of groundwater from water intake wells (downstream) into feed-zone wells (upstream)	Braking of subsurface flow	Groundwater flows of restricted width and thickness

Where low-permeability loam and clay extends to a depth of 5–20 m below the surface, a possible solution is the construction of many large diameter (1 m and over) gravel-filled seepage wells. The pumping method calls for the use of injection wells or culverts into which water is delivered under pressure. These are the seepage facilities of the "closed" type. One special method of artificial replenishment is the enhancement of groundwater feed into the aquifer from adjacent underlying or overlying aquifers through the drilling and fitting of special holes connecting the feeder and replenished aquifers. The pressure heads in the feeder aquifers must of course be higher.

For example, in the plains area of the Crimean Peninsula the intensified use of the principal Pontian-Meotian shell limestone aquifer resulted in a sharp lowering of the groundwater table and in the onset of groundwater exhaustion symptoms. To achieve natural transfer of water from the Middle Miocene aquifer lying 160 m below the Pontian-Meotian deposits it was decided to make use of the pressure head difference of 28–30 m [16]. The Middle Miocene aquifer is less suitable for direct supply due to its depth. The main reason for its unsuitability, however, is the fact that it consists of fine sand, which gives rise to blockage of water intakes during operation. The trial holes perforated both aquifers and the rate of transfer from the underlying aquifer into the Pontian-Meotian limestone reaching 23.5–26.4 liters/sec in the case of one well. The transfer proceeded in a virtually steadystate, which meant that no sand was transported out of the Middle Miocene aquifer. These actions made it possible, in large measure, to neutralize the adverse effects of over-intensive groundwater uptake from the Pontian-Meotian aquifer.

The scale of influent seepage facilities in each individual case are determined by the relevant conditions and objectives of the artificial replenishment system [8, 28, 29, 31, 34]. The criteria determining both the size and number of influent seepage installations when using the spread method are as follows: 2) the amount of water required to be fed in; b) the geological-hydrogeological setting; c) the replenishment source regime; d) the geomorphological conditions in the area. Influent seepage pools usually have a length ranging from 30–50 to 400 m and a width (measured along the bottom) from 10–15 to 50–80 m. The indicated optimal pool dimensions are determined largely by the ease of their cleansing after each seepage cycle, and especially by the ease of removal of the clay film and silted ground layer. The pool depth is usually 2–3 m. The dimensions of other influent seepage facilities used in the spread technique are determined by the same criteria as are applied to pools. At times they cannot even be clearly distinguished: for example, a canal can be regarded as a narrow pool, etc. The length of influent seepage canals, ditches and furrows runs from 200–300 m to 1.5 km and over, their depth ranges from 3 to 5 m; the bottom width is 5–20 m for canals and 1–5 m for ditches. The size of the channel bed area cleared is determined by calculation, depending on the amount of seepage water required, the seepage rate and the width of the stream bed. The length of the stream segment subject to clearing is determined from these data. The area of a flooded zone is determined by the magnitude and intensity of the recharge required, by local geomorphology, and by the rate of influent seepage. The size of a flooded area can range from 1–5 to hundreds hectares.

The construction of a injection well is basically no different from that of an ordinary water intake well [28]. Forcing can be effected: a) through a pump; b) through the gap between the pump column and filter (both of these methods are employed without withdrawal of the pump out of the well); c) with the pump withdrawn (Fig. 4.30). The diameter of the well depends on the required absorption rate or pump flow rate, on the seepage characteristics and receptivity of the aquifer and its active thickness. It is necessary in any case to opt for the maximum technically justifiable well diameters given a gravel fill thickness of 15–20 cm.

As already noted, a consideration of prime importance in the assessment of artificial replenishment conditions is the character of the aeration zone. The specifications to be determined are the thickness, grain size distributions [28, 35], structure, hydraulic characteristics of the rocks and the composition and quantity of salts present, especially if the site in question is situated in an arid zone. If the aeration zone is over 10 m thick, it becomes necessary to study its characteristics as they vary with depth; in the event of marked lateral facies changes, the variation of characteristics in plan must also be ascertained. Infiltrometry pit measurements must be carried out in order to determine the influent seepage rate. Similar measurements and data are required for the saturated zone of the aquifer which is to be artificially recharged.

So far there are not officially established requiremets as to the quality of the water delivered to influent seepage facilities. In assessing the quality of the water one can make use of the tentative guidelines issued by the Public Water Supply and Treatment Institute of the Academy of Public Utilities in Moscow as set out in [36] (Table 4.6). The requirements for water pumped into closed influent seepage systems have not been laid down so far. It should be stressed, however, that due to the considerable difficulty of cleaning operations and filter surface maintenance these are invariably more rigid than those for water delivered to open systems. As already indicated, the maximum permissible water turbidity is 2–4 mg/liter. In any case the quality of water delivered to influent seepage facilities muts ensure that only water

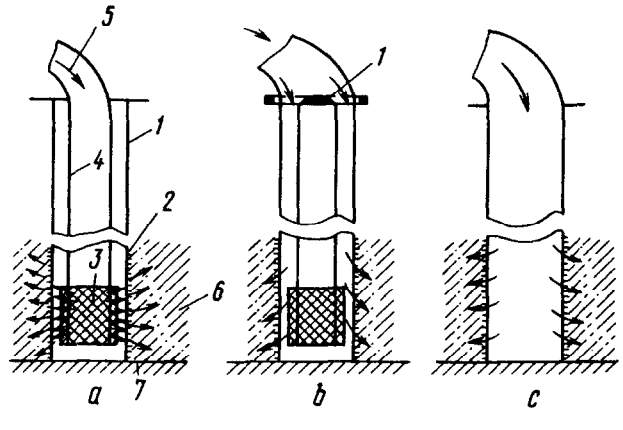


Fig. 4.30. Diagram of injection of water into a well: a – through a pump; b – through the clearance between the column of the pump and the walls of the well; c – during the extraction of the pump; 1 – casing column; 2 – filter; 3 – pump; 4 – column of the pump; 5 – inflow of water under pressure; 6 – aquifer; 7 – water-resistant base.

which satisfies the appropriate health standards such as the GOST standards applicable in the USSR [15] is subsequently withdrawn from the aquifer.

An important factor in addition to the seepage parameters is the filtration capacity of the aquifer, i.e. the ability of the aquifer materials to retain and accumulate fine suspensions in their pores in the course recharge [28]. The filtration capacity parameter is expressed in kg/m^2 and is equal numerically to the amount of mechanically suspended matter trapped in the rock pores or cracks per unit area of filter surface. The filtration capacity of a granular rock depends on its effective diameter. The approximate relationship between filtration capacity and effective rock grain size is shown in Fig. 4.31, which is based on the data of T.V. Burchak and Bogolyubov [2]. Another characteristic of aquifers which it is important to consider in assessing the feasibility of artificial groundwater replenishment is their receptivity, i.e. their ability to absorb and transmit water.

The principal negative factor acting to reduce the efficiency of influent seepage installations over time is the silting up of the filtering surfaces by suspended

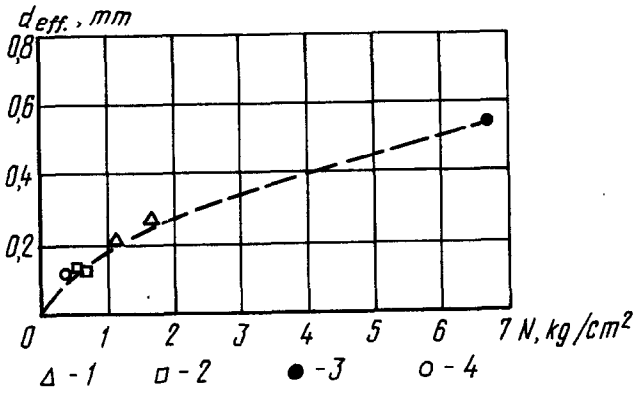


Fig. 4.31. Graph of the relation between the filtration-capacity of loose soils N and the effective diameter of the particles $d_{эф}$ [14].

Table 4.6

PERMISSIBLE QUALITY OF WATER DELIVERED TO INFLUENT SEEPAGE POOLS FOR DOMESTIC DRINKING WATER SUPPLY PURPOSES

Indicators of water composition and properties	Prescribed indicator values	Remarks
Turbidity, in mg/liter	For seepage through ground with α_{eff} = = 0.5–1 mm: 20; for seepage through ground with α_{eff} = = 0.15–0.3 mm: 10	If $\alpha_{eff.} > 1.5$ mm the pool floor should be covered with a layer of finer sand. Turbidity values higher than those indicated preseepage clearing of the feed water
Color, in degrees	The permissible water color is determined by the degree of dilution by natural groundwater to produce a water of standard quality In the absence of dilution by natural water the following restrictions apply: a) with a 50 % humic acid content in the color composition: 60 b) with a lower humic acid content: 40	
Odor and taste, in points	Directly perceptible: not over 3	Preseepage aeration of water to ensure maximum dissolved oxygen content is required
Dissolved oxygen O_2, in mg/liter	Maximum water saturation for a given temperature	
Organic matter, in mg/liter, O_2:		
permanganete oxygen consumption	Not over 15	
bichromate oxygen consumption	Not over 30	
Bacterial pollutants:		
Coli bacillus index, in units	Not over 100 000	
Total bacteria (overall count), in units per milliliter	for d_{eff} = 0.5–1 mm: 1 000 for d_{eff} = 0.15–0.3 mm: 5 000	
Mineral composition (in dry residue), in mg/liter	Set in accordance with design data. Must satisfy drinking water requirements after mixing with natural groundwater	Calculated in the course of planning and design
Chlorides, in mg/liter	Ditto	
Sulfates, in mg/liter	–"–	
Overall hardness, in mg/equiv/liter	–"–	
Fluorine, in mg/liter		Insufficient fluorine calls for fluoridation prior to water supply delivery
Chromium, in mg/liter	Not over 0.1	
Arsenic, in mg/liter	Not over 0.5	
Mercury, in mg/liter	Not over 0.005	

Table 4.6, continued

Indicators of water composition and properties	Prescribed indicator values	Remarks
Lead, in mg/liter	Not over 0.1	
Zinc, in mg/liter	Not over 1	
Copper, in mg/liter	Not over 1	
Total iron, in mg/liter	Up to 3	
Manganese, in mg/liter	Up to 0.3	
Ammonia, in mg/liter	Not over 1.5	
Phosphates, in mg/liter	Not over 1.0	
Petroleum and petroleum products, in mg/liter	Not over 0.5	
3,4-benzopyrene, in mg/liter	Not over 0.001	
Pesticides, in mg/liter:		
DDT	Not over 0.1	
Hexachlorocyclohexane (HCCH)	Not over 0.02	
Polychloropyrene	Not over 0.2	
Chlorophos	Not over 0.05	
Sevin	Not over 0.1	
Sodium trichloroacetate	Not over 0.5	
Surface-active agents (surfactants) of the anion-active group, in mg/liter	Not over 0.5	
Lignin-sulfanic acid, in mg/liter	Not over 2.0	
Water temperature, °C	Up to 25, with brief rises to 30	
Plankton, cells/milliliter	Not over 10 000	

particles. In addition to suspensions other factors contributing to silting are bacteriological slime formation, bound gases, air, elevated iron content, etc. Influent seepage facilities must therefore undergo periodic cleaning. The period between cleaning operations can range from three or four times a year to once every few years, the most common interval being once every one or two years. The total period of operation of an influent seepage facility between two successive cleanings is called the "seepage cycle". In the case of open-type facilities the methods for restoring filter surfaces can be divided into two groups — those entailing preliminary pool emptying and those where emptying is not required. When using methods of the latter-type, water-filled and continuously operating pools are cleaned by means of small-sized suction dredgers, or else by means of special equipment for underwater hydraulic cleansing. Such cleaning operations are very similar to the work involved in clearing river channels in order to intensify river water seepage. The group of methods in question does not yield very good results. Uniform removal of clogging substances over the entire floor of a pool cannot be guaranteed. The effectiveness of cleaning is likewise adversely affected by the rapid and intensive resilting of the cleaned portions of the bottom surface which ensues immediately under the considerable seepage pressure of the pool water. Nor is reliable control over cleaning quality possible. This is why preference is given today to methods of the former group, i.e. those which do require pool to be emptied. These include the loosening and removal of the slime and highly silted sand layers, either manually or with the aid of appropriate tools. The slime and sand layer removed is usually 1–3 cm thick. The dirty sand is removed from the pool, passed through a sand washer and reused. The advantage of the manual cleaning method is the high quality of its results. At the same time it is highly labour-consuming and not very efficient (a single worker can clean about 40 m^2 per hour) while requiring considerable physical effort. Mechanical cleaning of a pool bottom can also be effected with the aid of various machines such as those developed in Czechoslovakia or at the Dresden Technical University in the GDR [8]. It is also possible to employ standard and widely used machinery such as scrapers (Great Britain), excavators and bulldozers (USSR, USA). A bulldozer can clear away 12 000–15 000 m^2 of silted surface in a single man-shift. The advantages and disadvantages of machine clearing are the inverse of those involved in manual cleaning: on the one hand there is their high efficiency and cheapness, and on the other low-quality results, the impossibility of using them to remove only a precisely judged layer of silted sand, and therefore the increased use and waste of good sand. In addition, in carrying out mechanical cleaning it is necessary to see to it that the machinery does not drip oil products, oil and other contaminants onto the cleaned surface. The restoration of the filter surface by loosening, plowing and harrowing is usually an expedient suitable for shallow pools provided that the pool bottom is soil or impervious ground. The loosening usually extends down to a few centimeters and is carried out by means of smaller tools and machines in order to avoid compaction of the filter surface under the weight of the machine.

Restoration of the receptivity of closed influent seepage facilities (wells, culverts) is a more difficult task, the method to be employed being chosen in keeping with the silting or clogging process to which they are subject [31]. The consequences of mechanical silting of well filters and the spaces around the filter can be eliminated by flushing, by vibration, and by hydraulic percussion. It is recommended [31] that flushing be carried out by segments, i.e. over packer-isolated filter segments not over 3 m in length. The use of low-frequency vibrators with a vibration rate of 800–1 000 per minute makes flushing considerably more effective. Gas clogging in the near-

filter zone can be eliminated by flushing coupled with vibration and the use of so-called implosive action, i.e. instantaneous evacuation of the air by means of a special apparatus placed next to the filter. Where biological or chemical clogging is the problem (or where both are involved), well receptivity can be restored by means of periodic cleansing with a reagent (usually of an acid type) and compound treatments. Biological clogging can be successfully countered by the periodic cleaning of the well and the equipment with which it is fitted with chlorine. Chemical clogging can be corrected by means of various solvents, the latter being chosen to correspond to the clogging substance involved. Highly resistant clogging crusts on filters can be removed by percussive action on the well filter. This can be effected by means of a torpedo-type detonating fuse (TDF), pneumatic and electro-hydraulic shock treatment (EHS), implosive action and ultrasonic chipping or flaking [31].

From this, the prediction of the operation (output capability) of influent seepage facilities will be seen to be one of the most salient aspects of artificial groundwater replenishment as a method of countering groundwater depletion. In the method of its execution and in its basic underlying theory this prediction is similar to the hydrogeological forecasting carried out in assessing usable groundwater storage in the absence of artificial replenishment. The chief methods of predicting the operation of influent seepage facilities are the hydrodynamic technique on the one hand and mathematical modeling on the other. Approaches to the prediction of influent seepage facilities operation in groundwater protection and the computation formulas proposed by various authors are numerous and varied, but all of them are based on the method of flow superposition and are ultimately directed towards one end, that of determining the resultant lowering of the table at some point due to the operation of a pumping installation and the table rise at that same point due to the operation of influent seepage equipment or facilities. It follows from this that the conditions of implementation of artificial replenishment for groundwater protection, the prediction of the effectiveness of measures as well as the attainment of the objectives set, must be considered by examining the operation of the influent seepage system and the water intake facilities as a closely interrelated whole.

It should be borne in mind that the use of influent seepage facilities of the open or closed type renders boundary conditions more complicated. In addition, the arrival of seepage water in the aquifer proceeds nonuniformly, undergoing attenuation along the time axis due to silting and/or clogging, but this nonuniformity can be regulated and compensated by human engineering activity.

Prediction of filter surface silting and clogging at open-type facilities can be performed in the field by means of direct measurements with the aid of appropriate instruments – clogging gauges (colmatators), gamma densitometers, etc. The operation of silt gauges is based on the principle of passing a stream of liquid similar in its suspended matter content to that delivered to the influent seepage pool through soil enclosed in a seepage column. Clogging gauges make it possible to predict the dynamics of slime and silt formation on the seepage surfaces. One example of a successful clogging gauge design is that of the "S-u Colmatator" developed by the firm of Geotest in Brno (CSSR). The clogging and silting of seepage surfaces with time can also be predicted by means of radiation measuring instrumentation. Silting increases the natural radioactivity of rocks in the silting zone due to the filling up of pores by a higher-radioactivity clay fraction. This makes it possible to use the method of gamma logging, i.e. the measurement of natural soil radioactivity. A further method of measurement involves the determination of density variations in the silting zone by gamma-ray absorption or scattering methods. With increased clogging and silting, the readings given by a gamma densitometer will usually decrease. This is useful in the study of the seepage properties of the porous media constituting the water-receiving portions of the well under assessment.

Most of the computation parameters involved in predicting the output of both closed- and open-type influent seepage facilities are standard filtration or capacity variables used extensively in hydrogeology. We shall therefore not describe the methods for their determination, just as we shall not discuss the determination of such quantities as process time, quantity of water delivered, etc. We shall consider, rather, the filtration capacity and the complex parameter A which is the product of the seepage factor of the slime film by the density of its skeleton. The dimensions of filtration capacity are kg/m^2 and those of the parameter A are $kg/m^2/day$ [31]. The filtration capacity is usually determined experimentally by 1) direct investigation of the silted sand; 2) the use of the suspension balance equation; 3) evaluation of the duration of the silting/clogging period. The parameter A can likewise be determined by any of three methods, 1) measurement at an experimental pool using influent seepage gauges, clogging gauges (colmatators) or filtration instruments; 2) direct determination of the seepage factor and the density of the slime film skeleton in the course of evacuated pool inspection; 3) with the aid of empirical curves describing the relationship between the complex parameter A (or its components) on the one hand, and the properties of the water used for influent seepage [31]. In the preliminary stages of investigation the parameter A can be determined using a graph (Fig. 4.32) linking A with the content by weight percent (P) of clay and fine particles in the water delivered for influent seepage. Because of limited space we cannot describe in greater detail the techniques involved in determining these parameters, referring the interested reader to [31].

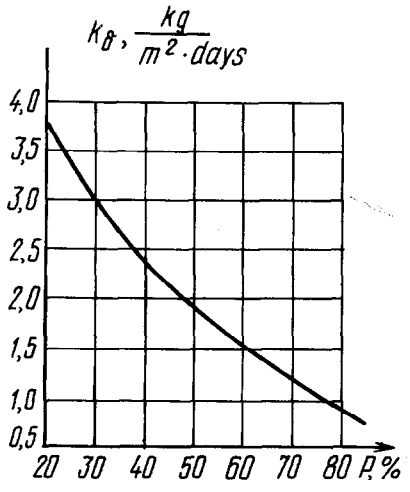


Fig. 4.32. Graph of the relation of the generalized parameter K_δ to the percentage (by weight) content in the water supplied for infiltration of fine-sandy and clayey fractions (P).

When organizing of a program of artificial replenishment of fissure and fissured karst water it is necessary to bear in mind a number of factors arising from the special nature of groundwater flow in such aquifers. For example, the water turbidity requirements in such cases are much less rigid, since the danger of silting and clogging is very small [41]. On the other hand, the media in question are characterized by very low purifying ability, which makes it necessary either to place a layer of sand on the seepage surfaces, or else to impose stricter requirements (except turbidity) on the quality of the water to be used for influent seepage.

Predictions associated with groundwater protection by artificial replenishment are usually carried out by mathematical modeling methods when dealing with complex hydrogeological conditions characterized by considerable variation of the seepage properties of the aquifer or by complicated boundary conditions which tend to vary over the useful lifetime of water intakes and boundary-type influent seepage facilities. Here, as in groundwater storage calculation, what is required is the solution of a number of inverse problems formulated in steadystate terms with a view to determining seepage parameters and boundary conditions, and in nonsteady-state terms in order to ascertain the conditions of interaction of surface and groundwater and aquifer interconnections, as well as to gain a more precise idea of seepage parameter values and boundary conditions, etc. It might be mentioned in this connection that in the USSR, simulations involving artificial replenishment have been carried out for water intakes already in operation (the Baltezers intake in Riga, the Zhartas intake in the Karaganda region, et al.). The solution of fundamental problem, i.e. the prediction of system operation at maximum output and with artificial replenishment, is obtained to take account of the size and number of influent seepage installations, their positions relative to intakes, and for the need for information concerning the future operation of the seepage facilities. All of this must be done with the final objective in mind, i.e. with a view of halting aquifer depletion or the reestablishment of groundwater storage if depletion has already taken place. Thus, the prediction of artificial replenishment system operation includes calculating the appropriate number of influent seepage installations as well as their initial and predicted final output. ("Final" in this context refers to the output towards or at the end of the seepage cycle). It also involves calculating the distance (spacing) between seepage facilities on the one hand and pumping facilities on the other, predicting variations in pumped water quality (although this may remain constant) as well as assessing usable water storage with allowance for artificial groundwater replenishment [19, 28].

Antidepletion-oriented groundwater protection studies are carried out in the same stage-by-stage sequence as are prospecting operations. It is therefore necessary at an early stage to answer the question as to whether or not given user needs can be satisfied without artificial replenishment. In the former instance special studies on the feasility of artificial replenishment are begun by taking stock of all data obtained in earlier stages of prospecting, usually starting with assessment of the operational deposit when the water intake installation satisfying first-phase user needs is already working. In the latter case, the study of the issues set out above in very concise form must be initiated from the outset.

Examples of the successful use of artificial groundwater storage replenishment as a means of countering groundwater depletion are quite numerous. Groundwater intakes situated within the debris cones of the piedmont alluvial-proluvial plain of Kopet-Dag, where discharge rates are as high as 1 m^3/sec and which together draw approximately 10–12 m^3/sec (the intakes is question are those of Ashkhabad, Bezmein, Sekizyab, Altyyab, Kel'techinar, Firyuza and other locations) operated in an unsteady mode. The rate of water table decline in the least favorable case (that of the Altyyab intake with a discharge of 32 000 m^3/day) reached 3.5 m/year. Moreover, the quality of the water drawn was declining due to the attraction of brackish waters from the sides and from below the aquifer [26]. The possibility of drawing upon the water supplies delivered by the Karakum V.I. Lenin Canal was considered as an anti-depletion measure. At present the situation at many of the intakes has improved due to the provision of influent seepage of water from the canal. There are plans for building special seepage facilities with a view to intensifying water intake feed.

Serious measures to regulate flow and provide for artificial groundwater storage replenishment are being contemplated in the Ortoalysh Depression with the aim of intensifying the renewal of groundwater being drawn by one of the intakes of Central Asia. Influent seepage due to the surface flow of the Alaarcha River in conditions undisturbed by hydraulic construction permitted recharge to the alluvial-proluvial aquifer complex and provided for the operation of a water intake with an output as high as 2.5 m^3/sec. However, in the mid-1970s a major portion of the Alaarcha flow had been diverted away from the aquifer complex along a concrete channel built to satisfy irrigation needs, with the result that intake operation became unsteady and entailed a continuous lowering of the groundwater table. At the present time the Allarcha Valley is under study as a site for the installation of artificial replenishment systems intended to increase groundwater feed.

The groundwater from the Pontian and Meotian aquifer complexes in the plains portion of the Crimean Peninsula was for many years withdrawn at rates in excess of the usable replenishment available in the area in order to satisfy local irrigation needs. This resulted in groundwater depletion, a lowering of the water table, and, what is most serious, the invasion of sea water into the aquifers at various locations. In the vicinity of the town of Krasnoperekopsk, for example, the rate of advance of the salt water contour was as high as 420 m annually. With the commissioning of the Northern Crimean Canal the situation improved considerably, as groundwater uptake for irrigation was reduced and a source of fresh water for artificial groundwater replenishment provided. The pumping of 14 000 m^3/day of fresh water into a straight-line row of wells made it possible first to halt the advancing salt water front, and then to drive it back to the shoreline [16].

Similar experiences in other countries may also be mentioned. In the USA the perennial uptake of groundwater for irrigation in certain valleys in California and in the Great Plains area (particularly in Texas and in the eastern part of New Mexico) resulted in a marked (up to 40–80 m and even greater) water table drop, increased irrigation water lift height, and reduced profits from agriculture. This made it necessary to take measures aimed at restoring groundwater storage through artificial groundwater replenishment [47].

Efforts in the Great Plains area include a program of hydrological and hydrogeological surveying of temporary lakes ("playas") in the high-altitude plains country of the state of Texas, where there are over 30 000 of them. The purpose of the survey was to harness a new source for the artificial replacement of groundwater in aquifers already depleted by intensive irrigation uptake. Experience with the decoding of space photographs taken from aboard the Apollo-9 spacecraft shows that it is possible to use satellite data for identifying hydrological and hydrogeological features of the terrain below [60]. Satellite photography makes it possible to: 1) determine water supplies in lakes and the times when this can be used for artificial groundwater replenishment; 2) indentify the lakes which feed groundwater (the volume of water these contain tends to diminish more rapidly than that in other lakes); 3) estimate the permeability of bottom deposits. Selected temporary lakes are equipped with water level recorders which transmit signals to the satellite, which retransmits them to an earth station. The resulting information arrives at the earth station once every 17 days making it possible to map areas which are promising from the standpoint of possible artificial groundwater replenishment.

A system of replenishment by the pumping of water into alluvial deposits near Barcelona (Spain) works on an original recovery principle. The facility consists of a central injection well surrounded by 16 small-diameter (0.2 m) wells arranged in a circle of 3 m radius. Whenever the pressure head drop in the pumping well reaches a critical value, a pump system is activated for a brief period (up to 15 min) to pump water into the peripheral wells on the basis of an alternating schedule, i.e. first into one group of eight wells and then into the eight other wells. The artificial replenishment source is river water which has been treated by microfiltration and chlorination only. The output of this facility is on the order of 7000 m^3/day [47].

The technology of replenishment through influent seepage facilities of other types (ponds, ditches, etc.) can vary. Thus, according to E. Trüb, Swiss water resources managers make extensive use of influent seepage ponds, of which approximately 40 % in each district are periodically filled with water, while the remaining 60 % of ponds are left dry to permit recovery. The flooding phase lasts 14 days, and the recovery phase roughly three weeks. The ponds are planted with trees (willow, poplar, alder), which makes it possible to leave the ponds for a long time without the need for desilting [74].

The largest artificial groundwater replenishment system in Switzerland is the Hardwasser facility with an output of approximately 140 000 m^3/day. Replenishment is here effected by means of a system of ditches whose overall length is 3 500 m. However, the average seepage velocity for these ditches (5 m/day) turned out to be lower than that achieved with ponds (8 m/day) with their considerably higher pressure head. Purification of raw water drawn from the Rhine involves coagulation followed by fast filtration. In places where there is a pollution hazard due to contact between the Rhine water and the bank filtrate, the solution is selective release of water from the ditches and ponds back into the Rhine in order to expel waters of inferior quality.

Examples of the use of artificial groundwater replenishment as a means of combatting groundwater pollution are less numerous and less impressive. In our view this is due largely to the fact that this method is a more passive one when it comes to pollution control than it is in the field of antidepletion protection. If pollutants have already entered an aquifer, there is no way that they can be eliminated by the methods of artificial replenishment. All that is feasible is the installation of hydraulic screens to prevent any further advance of polluted water and to keep it away from operating water intakes. Another way of achieving this is to construct protective barrages. One example of the indirect impact of human engineering activity which had the effect of protecting groundwater against pollution concerns the city of Voronezh. Industrial effluents in that area penetrated into the principal water supply aquifer complex of Upper Pliocene/Lower Quaternary age, thus threatening the normal operation of local water intakes. Eventually, however, a water reservoir was built on the Voronezh River, one of the purposes of this project being to halt the advance of polluted water towards the water intakes. This measure was in fact successful in stabilizing local water quality.

The subject of artificial groundwater replenishment is a broad and complex one, and its consideration from the standpoint of antidepletion and antipollution protection is merely one of its aspects in relation to the needs and demands of the national economy. Nor must one lose sight of the fact that groundwater depletion and the deterioration of groundwater quality are quite often interrelated process. After all, the attraction of brackish water or water which is substandard because of one or another component is often due to water uptake being in excess of usable storage. This interrelation of the two phenomena is a particularly intimate one in coastal areas.

Artificial groundwater replenishment can be helpful in dealing with water supply problems under conditions where other solutions are either entirely lacking or would prove excessively costly, as is the case, for example, along the Black Sea coast of the Caucasus from the town of Gelendzhik to Sukhumi. Artificial groundwater replenishment is therefore an exceedingly effective way of rationalizing and intensifying the use of groundwater in particular and of water resources in general. In other words, it must be viewed as an integral part of water resources development. The protection of groundwater against pollution and, what is still more important, against depletion, are thus two important aspects of a more general problem.

4.7.2. COMPUTATION FORMULAS AND EQUATIONS FOR THE PREDICTIVE ASSESSMENT OF ARTIFICIAL REPLENISHMENT

The design of artificial groundwater replenishment schemes with a view to ensuring protection against depletion or increasing water intake discharge involves the step known as "predictive assessment of artificial replenishment". Such assessment makes it possible to determine the output of influent seepage facilities (pools, wells), the required number of such facilities, and the total discharge of water intakes once the artificial replenishment system is in place and operational. All of these determinations are carried out with specific reference to the boundary conditions and parameters characteristic of a given aquifer. In the final analysis the predictive assessment of artificial replenishment makes it possible to establish more rational utilization of available resources and of water protection measures which might be shown to be economically justifiable and desirable.

The hydrogeological computation procedure set out below is described with reference to the two most widespread types of influent seepage installations or facilities, namely seepage pools and injection holes, although the same procedure is also suitable for calculating seepage canals, absorption culverts, wells, pits and shafts.

Calculation of influent seepage pools

The basic aim in calculating influent seepage pools is to determine their output, influent seepage area, principal dimensions, and pool number. The required output Q_{is} of influent seepage facilities such as a pool system is given in general form by the formula

$$Q_{is} = \frac{Q_{wis}}{L} \qquad (4.65)$$

where Q_{wis} is the fraction of the water intake discharge due to artificial replenishment (in m^3/day), which corresponds to the discharge attained when dealing with a totally depleted aquifer; L is the efficiency of the artificial replenishment system which indicates (in fractions of unity) which portion of the total discharge of the water delivered for replenishment is attracted to the water intake facility.

The value of the factor L for each specific case can be determined by a hydrodynamic calculation on the basis of the boundary conditions characterizing the given location, the aquifer parameters, the water intake facility type, the artificial replenishment system contemplated, and the mode in which the system is intended to operate.

The principal quantitative indicator of artificial replenishment of unconfined groundwater due to influent seepage pool operation is the influent seepage velocity V_{is} (in m/day).

The mean influent seepage velocity can be estimated by analogy with already operating pools situated in similar hydrogeological and other natural conditions and/or on the basis of data obtained by operating a trial pool at the prospective site, provided the duration of the trial is not less than one whole seepage cycle.

When such data are not available, the average velocity can be approximated with the aid of analytic relations using seepage parameters which have been obtained on the besis of shorter-term observations (of one or two months' duration) with experimental pools or seepage columns.

Present theories concerning the dynamics of seepage of natural waters through rocks are based on certain simplifying assumptions whereby water is regarded as an ideally "pure" liquid or, at best, that the seepage process is accompanied by the mechanical silting of the rocks and soil. It is common practice in this connection to distinguish between two types of silting which have a bearing on the seepage process, and to deal with each type separately. These are, on the one hand, deep silting, which results in the formation of a silted zone within the rock formation and surface silting.

Analysis of field and laboratory studies shows that the seepage patterns of natural water are extremely varied, depending on lithology, the character of suspension of the seepage water, its chemical and biological composition, and the water temperature and gas factor.

Mathematical models to describe natural water seepage have been constructed on the basis of the theoretical assumption, confirmed by experiment, that seepage resistance to water flow is determined by the intensity of the rock and soil silting process. The character of silting itself varies according to whether pore clogging is complete or gradual, with a single pore or fissure being silted up by a large number of suspended particles. It has been established that one type of seepage can gradually assume another form with time. Moreover, there exist intermediate forms of water seepage which are accompanied both by deep silting of rocks and the formation of silt sediment on rock surfaces.

It has been established that there are fundamental differences in silting processes as they occur in different media. In this connection various types of rocks can be classified as follows:

1) highly fissured and karst rocks characterized by gradual deep silting with considerable diffusion of suspended matter in the voids with practically no "film" seepage, turbid water flowing just as it would through a pipe or a channel;

2) boulder and shingle beds either of the well sorted variety or with gravel filler are characterized by a process with a protracted cycle of gradual deep silting, with individual pores clogged by several particles from the suspension, and gradual transition to silt sediment formation;

3) sand-gravel-cobble bed deposits, large- and medium-grained sand, cobble beds, and boulder-and-cobble beds with a sand filler in which seepage takes on an intermediate character, with deep silting occurring in a shorter time than it does in rocks of the previous type, and with the possibility of each pore being clogged by a single suspension particle;

4) slightly fissured coherent rocks, porous sandstones and conglomerates in which suspended particles can penetrate to considerable depths by traveling along individual open cracks and pores, while the remainder is left behind as a sediment on the rock surface, so that pore silting occurs concurrently with sedimentation;

5) fine- and small-grained sand, clayey sand, loam, loamy sand and very slightly fissured coherent rocks in which the process of deep silting can be very brief, with surface silt deposition playing the principal role (seepage through a silt layer previosly formed in, for example, sand can take on a similar form during the second seepage cycle following cleaning operations which have affected the silt sediment only). Various computation equations and formulas are available for approximating the velocity of water seepage from pools with allowance for the deep and/or surface silting of the rocks in question.

If rocks and soil happen to be sufficiently permeable with respect to suspended matter present in surface water, then the period of deep silting for a pool in which the water is continuously topped up to a constant level (h = const) is given by the formula

$$V_t = V_0 e^{-\beta t_K} \qquad (4.66)$$

where V_0 and V_t are, respectively, the initial and the time-variable seepage velocity, in m/day; t_K is the duration of deep siling, in days; β is the coefficient of seepage velocity decrease as determined experimentally under field or laboratory conditions for two instants in time,

$$\beta = \frac{\ln V_1 - \ln V_2}{t_2 - t_1} \qquad (4.67)$$

The duration t_K of the deep silting period is in turn given by the approximate formula

$$t_K = \frac{G}{V_{av} \cdot M} \qquad (4.68)$$

where G is the filtration capacity or specific area saturation of the rock by the silt mass during seepage of turbid water (this quantity, expressed in g/m^2, is determined experimentally using seepage columns or operational/trial pools); V_{av} is the average influent seepage velocity during the deep silting period, in m/day; M is the turbidity of the water entering the pool, in g/m^3 (or mg/liter).

If the period of surface silting (silt deposit formation on the rock surfaces) has already begun, then given a relatively constant turbidity of the water in the pool and h = const one can make use of the following formula (see H. Zöffler, W. Pietsch and W. Huhn, 1973):

$$V_t = \frac{V_{0K}}{\sqrt{1 + (K)\, t_s}} \qquad (4.69)$$

where V_{0K} is the velocity of seepage at the end of the silting period, in m/day; t_s is the duration of surface silting of the pool, in days; (K) is an empirically determined parameter for the period of surface silting of the pool or of a membrane filter, in days^{-1}.

The value of the parameter (K) is usually established at the end of the trial by means of the formula

$$(K) = \frac{V_{0K}^2 - V_t^2}{V_t^2 - t_s} \qquad (4.70)$$

Influent seepage pools are often operated in a complex mode consisting of three basic periods (Fig. 4.33).

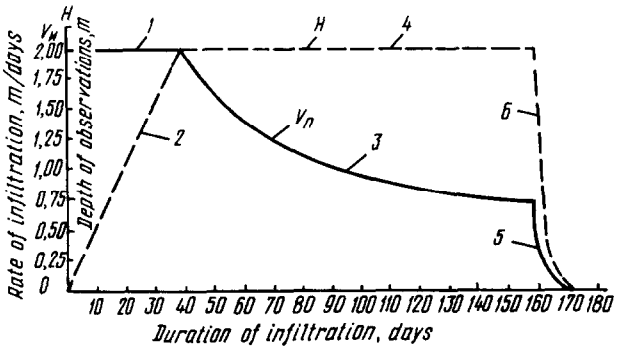


Fig. 4.33. Graphs of the change in time of the rate of infiltration and the depth of filling of the basin with water under a complex operating regime

1st period: 1 — v = const; 2 — h = f (t)
2nd period: 3 — v = f (t); 4 — h = const
3rd period: 5 — v = f (t); 5 — h = f (t)

During the first period as the pool is being filled, seepage proceeds at a fairly constant velocity as the water level in the pool continuously rises.

During the second period following attainment of the maximum water level h_{max} the basin operates at a constant pressure head while the seepage speed decreases with time.

During the third period, once water delivery to the pool has stopped, there is a decrease both in the water depth and in the seepage velocity.

The formulas given below make it possible to predict influent seepage pool operation and to determine silted layer parameters for the case where the water seeps through an increasingly thick silt deposit.

1. For artificial replenishment with a constant influent seepage velocity (V = const),

$$t_1 = \frac{10^3 K\delta h_{max}}{V^2 M} \qquad (4.71) \quad \text{and} \quad K_\delta = \frac{V^2 Mt}{10^3 h_t} \qquad (4.72)$$

2. For artificial replenishment with water depth maintained constant (h = const),

$$V = \sqrt{\frac{10^3 hK_\delta}{2Mt}} \qquad (4.73) \quad \text{and} \quad K_\delta = \frac{2V_t^2 Mt}{10^3 h} \qquad (4.74)$$

3. For a compound mode of artificial replenishment for the second period of influent seepage, where h = const,

$$V_t = \frac{V}{\sqrt{1 + \frac{2hM(t - t_1)}{10^3 K_\delta A_0^2}}} \qquad (4.75) \quad \text{and} \quad K_\delta = \frac{2hM(t - t_1)}{10^3 A_0^2 \left(\frac{V^2}{V_t^2} - 1\right)} \qquad (4.76)$$

4. In the pool emptying mode where V = f(t) and h = f(t),

$$V_t = V_K e^{-A_0^{-1} K t} \qquad (4.77) \quad \text{and} \quad h_t = h_K e^{-A_0^{-1} K t} \qquad (4.78)$$

$$\text{when } A_0 \approx \frac{h_t}{V_t} \qquad (4.79)$$

The above formulas contain the following letter symbols:

V_t and h_t are the time-variable influent seepage velocity (in m/day) and the depth of the water in the pool (in m); h_{max} is the maximum permissible water depth for the constant seepage velocity mode; V and h are the constant seepage velocity and water depth; t and t_1 are the total duration of seepage and the duration of the first period (in days); V_K and h_K are the final seepage velocity and final water depth in the pool at the instant that the water supply to the pool is turned off; A_0 is the coefficient of resistance to seepage of the silted layer at the end of the first period (in days); A_{0K} is the same coefficient at the instant of turn-off of the water supply to the pool (in days); $K_\delta = 10^3 \cdot K_0 \cdot \delta$ is a compound seepage parameter associated with the silt sediment, equal to the product of the sediment seepage factor and the specific weight of the skeleton (kg/day·m^2).

Seepage velocity variations in the h = const mode are determined by taking specific time intervals and using formulas (4.73) and (4.75) to calculate the corresponding velocity at each instant, after which it is possible to plot the function V = f(t). The latter is then used to find the average velocity over the entire period covered by the calculation. For the mode in which the level of water in the pool is dropping as the pool is emptied, the seepage velocity and pool water depth can be determined by means of formulas (4.79) and (4.80). This involves choosing brief time intervals, e.g. one or two days, and then using the calculation points to construct the curves of V = f(t) and h = f(t), which can then be used to determine the average seepage velocity over the given period.

The average seepage velocity over the entire seepage cycle, V_{av}, is determined from the resulting values of V_i and t_i for each seepage period,

$$V_{av} = \frac{V_1 t_1 + V_2 t_2 + V_3 t_3}{t_1 + t_2 + t_3} \qquad (4.80)$$

where V_1, V_2, V_3 are the average velocities over the first, second, and third periods of corresponding duration t_1, t_2, and t_3, respectively. The annual-average seepage velocity can be determined from the formula

$$V_{an.av} = \frac{\Sigma V_i \cdot t_i}{365} \qquad (4.81)$$

When the values of the output Q_{is} (in m^3/day) and the average seepage velocity V_{av} (in m/day) are known, the total seepage area F (in m^2) can be found from the formula

$$F = \frac{Q_{is}}{V_{av}} \tag{4.82}$$

It should be noted that seepage through pool slopes usually occurs during the initial period of operation only, so that the area given by formula (4.82) is largely that of the pool bottom.

In those instances where the designer or builder of influent seepage pools does not have experimental data or an analogy-based value of the water seepage velocity available to him, he can rely on approximate average values based on the known granulometric composition (effective particle diameter) of the loose soil at the site under investigation, namely

0.5 m/day for d_{eff} = 0.15–0.25 mm,

0.5–1.0 m/day for d_{eff} = 0.25–0.5 mm,

1–2 m/day for d_{eff} = 0.5–1.5 mm.

The indicated velocity values roughly correspond to a pool water depth of 2 m, a six-month seepage cycle, and an average water turbidity no higher than 10–20 mg/liter.

Rough calculations of the average seepage velocity in the absence of experimental data can be carried out on the basis of a correlation function established with a probability of 68 % and without allowance for determination error,

$$V = 0.054\ K^{0.75} \tag{4.83}$$

where K is the seepage factor calculated on the basis of test pumping data.

Statistical analysis shows that on the average, the seepage factor or water-bearing rocks exceeds by a factor of 40–50 the seepage velocity which characterizes vertical water movement.

The size of influent seepage pools in plan is determined from the assumption that they are placed along the line of water intake wells, with due allowance for convenient operation. The length l_δ (in m) of an individual basin is often set at 100–400 m with a distance between pools of at least 20 m.

If the total seepage area F (m^2) is known and if the length of a single pool is set at l_δ, then the average pool width b_δ (in m) can be found from the formula

$$b_\delta = \frac{F}{\Sigma l_\delta} \tag{4.84}$$

The number of pools required is determined in accordance with selected pool size and the total seepage area.

The total water yield W (m^3) of seepage pools into an aquifer over a seepage cycle t_f (in days) is given by the formula

$$W = Q_{is} \cdot t_f = F \cdot \Sigma V_i t_i \tag{4.85}$$

Hydrogeological feasibility studies of seepage pool operation often entail estimates of the possible duration of a single seepage cycle, and therefore of the frequency of pool bottom cleaning operations.

The total duration of a single seepage cycle, t_f (in days), is given by

$$t_f = t_K + t_s \tag{4.86}$$

where t_K is the duration of deep silting as determined with the aid of data on the filtration capacity of the rocks and soil using formula (4.70); t_s is the permissible duration of surface silting in the pool. The quantity t_s can in turn be determined by the following equation:

$$t_s = \frac{10^3 K \delta A_0}{2hM} \left(\frac{V^2}{V_{per}^2} - 1\right) \tag{4.87}$$

Experience shows that the permissible final seepage velocity can lie in the 0.5–0.3 m/day range. However, if the quality of the water used for influent seepage is not very high, then the final seepage velocity can be set even below 0.3 m/day in the interests of more effective purification. The decisive factors in this context may include the achieved effect of "raw" water purification and the economic constraints involved in artificial replenishment.

Calculation of injection wells

In the hydrogeological assessment of artificial groundwater storage replenishment by the injection well method it is usual to assess variations in the specific water absorption discharge over a single seepage cycle. These variations are due to the silting of the filter and soil in the immediate vicinity of the well zone and determine the total water yield discharge of the system.

The injection mode most frequently used is characterized by a uniform discharge with a continuously varying water level in the well.

In this case the variation of the specific water absorption discharge q_t, i.e. of the level rise per meter, can be expressed by means of the exponential function

$$q = q^0 \cdot e^{-\beta t} \tag{4.88}$$

where q_0 is the specific absorption discharge at the start of a trial (in m^3/day·m) and β the coefficient of discharge decrease during forcing, determined experimentally for two instants using a formula similar to (4.67),

$$\alpha = \frac{\ln q^1 - \ln q^2}{t_2 - t_1} \tag{4.89}$$

The specific water absorption discharge of wells is usually double the specific discharge of the same wells obtained for similar groundwater level rises and falls ($\Delta H = S$) when bailing.

The output of injection wells can likewise be computed with the aid of a theoretical formula in which partial penetration due to incomplete exposure and filter/rock silting in the well zone are estimated in terms of the dimensionless indicator of total resistance ξ_t,

$$Q_r = \frac{4\pi km\ \Delta H}{\ln \frac{2{,}25\ at}{r_0^2} + 2\xi_t} \tag{4.90}$$

where km is the hydraulic conductivity of the aquifer (in m^2/day); ΔH is the injection pressure head (in m); α is the piezoconductance coefficient (in m^2/day); r_0 is the well radius (in m).

The total dimensionless resistance ξ_t can be determined from formula (4.90). The resistance associated exclusively with the degree of silting of the filter and well zone soil, ξ_k, is defined as the difference between the total resistance ξ_t at any instant after the start of well filling or pumping into the well and the initial resistance ξ_0 as determined from trial pumping data or for the initial instant of trial pumping. The values of ξ_k for various instants during pumping are then used to determine the

time variation of the resistance due to silting, which is then used for predictive estimation of absorbing well output.

In the absence of experimental or operational data on the pumping of natural water into wells, the specific well water absorption q_p (in m^3/day) per meter of pressure head ΔH can be approximated using a correlation formula without allowance for determination error,

$$q_p = 5.62(km)^{0.46} \qquad (4.91)$$

The resulting values of q_p are then used to estimate the required number of absorbing wells N,

$$N = \frac{\Sigma Q_{is}}{q_p \cdot \Delta H} - \frac{\Sigma Q_{is}}{Q_w} \qquad (4.92)$$

where ΣQ_{is} is the required total discharge of the water pumped into the wells (in m^3/day); Q_w is the average discharge of a single well (in m^3/day).

The total water yield to the aquifer, W (in m^3) provided by the absorbing well system over the forcing or pumping period t is given by

$$W = Q_{is} \cdot t = N \cdot Q_w t = \Sigma Q_i t_i \qquad (4.93)$$

In assessing water absorption and the total water yield to the aquifer due to the system of injection wells one should bear in mind that well receptivity is not usually completely reestablished by cleaning.

Calculation of water intakes operating in conjunction with artificial groundwater replenishment systems

The predictive assessment of water intake discharge with allowance for artificial groundwater replenishment requires that allowance be made for the initial and boundary conditions associated with the aquifer, the type and location of water intake and influent seepage facilities, as well as the possible mode of artificial replenishment. It should be borne in mind, moreover, that analytical computations of water intake output under artificial replenishment conditions are carried out for areas where boundary conditions can be represented schematically as linear or circular contours in water-bearing rocks which have relatively uniform seepage conditions. In complex hydrogeological settings where the seepage parameters of the rocks and soil vary considerably and also in the case of highly complex water intake and influent seepage systems, output estimates must be made by modeling. Two artificial replenishment modes are generally singled out: a constant mode in which only individual installations within a larger system of influent seepage units are shut down for cleaning (recovery) while the rest of the system continues to operate without interruption, and a periodic mode where the replenishment system is shut down when there is no water available in the replenishment source, when there is a temporary decline in water quality, or when all of the influent seepage units are being cleaned.

The simplest instance of predictive water intake assessment is that where one relies on the method of analogies, provided a given region already has an operational artificial replenishment system in action.

The first step then is to determine the single-unit discharge q_w (in m^3/day·m) of an analogue water intake during artificial replenishment by dividing the total discharge Q_{wl} (in m^3/day) by the length l_1 (in m).

The predicted discharge of the new water intake unit with allowance for artificial replenishment is then given by the formula

$$q = q_{w^1} \frac{(kmS \cdot F)_2}{(kmS \cdot F)_1} l_2, \qquad (4.94)$$

where $(kmS \cdot F)_1$ is the product of the hydraulic conductivity, groundwater table fall, and artificial seepage area for the reference water intake site; $(kmS \cdot F)_2$ is the same product for the new water intake site; S is the permissible table drop; F is the seepage area as established on the basis of local conditions; l_2 is the length of the new water intake unit (in m).

Clearly, the method of analogies can be applied legitimately if the water intake sites in question are characterized by like or closely similar conditions of groundwater storage formation and by similar boundary conditions. The formulae set out below make it possible to estimate water intakes within artificial replenishment systems with reference to seven of the most typical hydrogeological and natural/technological settings (Fig. 4.34).

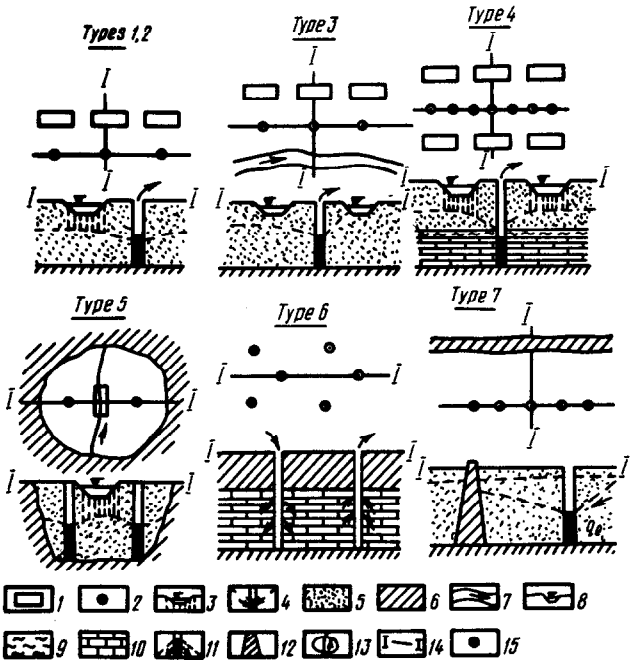


Fig. 4.34. Typification of hydrogeotechnical conditions of water-intake sites with systems of regulating groundwater operating supplies:
1, 3 — infiltration basin; 2,4 — operating well; 5 — loose water-bearing deposits; 6 — water-impermeable rocks; 7, 8 — river; 9 — slightly permeable separating layer; 10 — water-bearing calcareous rocks; 11, 15 — water-absorbing well; 12 — underground dam; 13 — closed boundary of a sector of water-enclosing rocks; 14 — cross-section

Type I. A long row of seepage pools and a water intake installations in the form of a straight-line row of wells are situated in an aquifer which is infinite in area (Fig. 4.34). Such conditions occur in artificial groundwater replenishment within large river terraces, interfluvial watershed areas, outwash and piedmont plains, and subaerial river deltas. The water intake discharge and the efficiency of the artificial groundwater storage replenishment system are here given by the formulas

$$Q_w = \frac{(2kmS+q_u \cdot L_\delta)l}{L_w+2L_{inc}} \quad (4.95) \qquad \alpha = \frac{L_\delta}{L_w+2L_{inc}} \quad (4.96)$$

where

$$L_w = 1.128\sqrt{at} \quad (4.97)$$

$$L_\delta = \sqrt{at}\,\text{ierfc}\left(\frac{x}{2\sqrt{at}}\right) \quad (4.98)$$

$$L_{inc} = \frac{\sigma}{2\pi}\ln\frac{\sigma}{2\pi r_0} \quad (4.99)$$

$$q_{is} = V_u \cdot b\frac{\Sigma l_\delta}{l}, \quad (4.100)$$

where L_w is the radius of influence (in m) of a water intake in the row in plane flow; L_δ is the conventional reduced radius of influence (in m) of the influent seepage pools; L_{inc} is the linear seepage resistance (in m) characterizing the hydrodynamic incompleteness of the water intake facility; q_{is} is the unit influent seepage discharge (in m^3/day·m) of the row of pools reduced to the length l (in m) of the water intake facility; k and m are the seepage factor and the thickness of the aquifer, respectively; S is the table drop (in m) in the operating wells; a is the coefficient of hydraulic conductivity (in m/day); t is the duration of system operation (in days); b is the average width of the pools (in m); Σl_δ is the sum of the pool lengths (in m); x is the distance (in m) between the water intake facility and the midpoint of the row of influent seepage pools; 2σ is the spacing between the operating wells (in m); r_0 is the well radius (in m).

When making the transition in the above formulas from artesian to phreatic water, it should be borne in mind that $m = (h - \frac{S}{2})$.

Type 2. This case is characterized by the same boundary and hydrogeological conditions (Fig. 4.34,), but differs in respect of the artificial replenishment mode, which is periodic. The hydrodynamic computations here entail estimates of:

a) the potential discharge of the water intake facility without artificial groundwater replenishment during periods when the influent seepage units are being cleaned (or in winter);

b) the required discharge of the influent seepage facility ensuring both maintenance of the water intake output and the concurrent total restitution of water withdrawn from the aquifer during the preceding period.

The discharge of the straight-line row of wells fed by natural storage contained in an infinite aquifer is given by the formula

$$Q = \frac{2kmSl}{L_w + 2L_{inc}} \quad (4.101)$$

In the period following the exhaustion of the previously available storage the influent seepage discharge from the pools must be

$$Q_{is} = q_w l\,\frac{L_{w1} + L_{w2} + L_{inc}}{L_\delta} \quad (4.102)$$

where L_{w1} and L_{w2} are the conventional radii of influence (in m) of the water intake facility during the exhaustion period t_1 and the period t_2 of groundwater replenishment, as determined from formula (4.97).

Using data on the required artificial replenishment discharge Q_{is} and the average influent seepage velocity V_{is}, one can determine the required area of influent seepage from the pools, as well as their size and number.

Type 3. The most typical case is that where a straight-line row of wells runs between a river with a perfect connection with groundwater and a parallel row of influent seepage pools (Fig. 4.34). The site boundary conditions in this instance can be represented schematically as a semifinite stratum with a single constant pressure head contour (the river) and a sprinkler strip along the line of influent seepage pools. The discharge of the water intake facility and the efficiency of the artificial groundwater replenishment system are then given by

$$Q_w = \frac{(kmS + q_{is}L_\delta)\cdot l}{2(L_c+L_{inc})}, \quad \alpha = \frac{L_\delta}{2(L_c+L_{inc})} \quad (4.103)$$

where $L_c = L_0 + \Delta L$ is the virtual distance (in m) from the water intake facility to the river; L_0 is the true distance from the water intake facility to the river; ΔL is the linear hydraulic resistance of the river channel bed determined from clustered trial pumping data or from observations of the groundwater regime in the immediate vicinity of the river. Formulas (4.103) are valid for $L_\delta \geqslant L_c$. For a steady seepage mode one must assume that $L_\delta = L_c$.

Type 4. The boundary conditions in this case can be schematically represented in cross section as a three-layer system consisting of two aquifers separated by a low-permeability layer (Fig. 4.34). Artificial replenishment by means of a system of influent seepage units delivers water to the upper aquifer, while the water intake system in the form of a straight row of wells draws water from the lower layer of the aquifer sandwich. In the general case, i.e. that where artificial replenishment keeps the water table constant in the upper aquifer, the discharge of the water intake which taps the lower aquifer (provided there is continuous inflow from above) and the influent seepage system efficiency are

$$Q_w = \frac{l[(km)_1 + (km)_2]S + lq_{is}B(R_{is}-R'_{is})}{B\,(R_w + \frac{1}{\nu}R'_w)} \quad (4.104)$$

$$\alpha = \frac{R_{is} - R'_{is}}{R_w + \frac{(km)_2}{(km)_1}R'_w} \quad (4.105)$$

The dimensionless hydraulic resistances R and R′ are given by the formulas

$$R = \frac{\sqrt{at}}{B}\,\text{ierfc}\left(\frac{x}{2\sqrt{at}}\right) \text{ and } R' = \frac{1}{2}l^{-\frac{x}{B}} \quad (4.106)$$

Here $(km)_1$ and $(km)_2$ are the transmissivities of the operating and replenished aquifers (in m^2/day); B is the leakage parameter (in m) determined from trial seepage data or by studying the water intake operating mode; $\nu = (km)_1/(km)_2$; α' is the generalized value of the piezoconductance factor which makes allowance for groundwater decantation between aquifers. Its dimensions are m^2/day and it is given by the formula

$$\alpha' = \frac{(km)_1 + (km)_2}{\mu_1 + \mu_2 + \mu_0}, \quad (4.107)$$

where μ_1, μ_2, μ_0 are the water yields of the lower and upper aquifers and of the impervious separating layer, respectively.

Type 5. The conditions characterizing this case (Fig. 4.34) occur in areas with highly developed close structures, some of which may be situated in local blocks of fissured and karst rock and thawed ("talik") zones within permanently frozen rock underlying loose water-bearing deposits. The water intake facility consists of a

group of pumping wells. Artificial groundwater replenishment is possible during the summer (or flooding) period and is effected by means of influent seepage pools which supply injection wells. The boundary conditions are represented schematically in the form of a disk-shaped stratum bounded by an impervious contour. The water intake facility discharge due exclusively to natural groundwater storage use is given by Masquette's formula

$$Q_W = \frac{2\pi kmSN}{\ln \frac{R_k}{r_0} + \frac{2at}{R_k^2} - 0.75} \qquad (4.108)$$

where $R_K = \sqrt{F/\pi}$ is the radius (in m) of the block reduced to a circular contour (F being the area of the block in m^2). The remaining symbols are defined as before.

In the period following exhaustion of the storage the required artificial seepage discharge must be

$$Q_{is} = \frac{Q_W (t_1 + t_2)}{\alpha t_2} \qquad (4.109)$$

where t_1 is the duration of the natural groundwater storage use period (in days); t_2 is the duration of artificial replenishment during the period when the surface water source is available (in days); α is the efficiency of the system, which can be taken equal to roughly 0.9 in the case of closed structures, with allowance for a certain amount of water being lost on rock wetting.

Type 6. In this case artificial replenishment is effected by pumping water into injection wells. The water intake facility takes the form of a group of abstraction wells. Such conditions are typical of artesian aquifers in mountainous folded regions, for platform-type artesian basins, and for piedmont plains.

Water intake facilities of the type in question operate according to the "pump out/pump in" principle, when groups of withdrawing and water-injecting wells are operating concurrently (Fig. 4.34). For an aquifer assumed to be infinite in plan (provided that $L^2 \geqslant 3-4\alpha t$, where L is the distance to the impervious boundary), the usable groundwater storage and the efficiency of the pumping type replenishment system are given by the formulas

$$Q_W = \frac{4\pi kmS + Q_H \ln \frac{2{,}25at}{r^2}}{\ln \frac{2.25at}{r_r^2} + R_C}; \alpha = \frac{\ln \frac{2.25at}{r^2}}{\ln \frac{2.25at}{r_r^2} + R_C} \qquad (4.110)$$

Formulas (4.110) are valid provided that $r > r_r$. Here Q is the total hydraulic absorption discharge of the recharge wells (in m^3/day); R_C is the internal hydraulic resistance in the water intake wells.

For a straight-line row well system $R_C = \frac{2}{n} \ln \frac{\sigma}{2\pi r_0}$,

and for a rectangular or square system $R_C = 2 \ln \frac{0.27\sqrt{F}}{r_0}$

where r is the distance (in m) between the centers of the water intake facility and the recharge system; n is the number of wells in the straight-line row; r_r is the reduced radius for various well systems, i.e. $r_r = 0.19l$ for a straight-line row system and $r_r \approx 0.61 R_0$ for a rectangular or square system; R_0 is the radius of the rectangular system when the latter is reduced to a circle. It is given by $R_0 \approx 0.56\sqrt{F}$, where F is the area (in m^2) of the well deployment site.

Type 7. The aquifer is associated with the valleys of smaller rivers and temporary watercourses. Its extent and thickness are both small. In order to increase water intake discharge a decision is made to install artificial subsurface barriers to inhibit the outflow or natural drainage of influent seepage water. The hydrogeological system is represented schematically as a strip stratum with parallel impervious boundaries and a transverse constant-discharge contour, i.e. as an underground dam to intercept subsurface runoff. The water intake facility in the form of a straight row of wells is placed parallel to the underground dam (Fig. 4.34).

The contour of the underground dam can be represented conventionally as an absorbing culvert with a unit natural flow discharge of q_n (m^3/day) distributed uniformly over the flow width, $q_n = Q_n/L$. The output discharge of the water intake facility and the efficiency of the subsurface runoff regulation system are given by

$$Q_W = \frac{\pi km(2S + J_0 L_n)}{\ln \frac{\sigma}{2\pi r_0} + \frac{3.55\sqrt{at}}{\sigma}}; \alpha = \frac{\pi L_n}{L\left(\ln \frac{\sigma}{2\pi r_0} + \frac{3.55\sqrt{at}}{\sigma}\right)} \qquad (4.111)$$

where J_0 is the slope of the subsurface flow under natural conditions; L_n is the reduced radius of action (in m) of the underground dam (constant-discharge contour).

The quantity L_n is defined similarly to L_δ (the radius of action of an absorbing culvert), i.e. it is given by formula (4.98) where x is the distance (in m) from the underground dam to the transverse row of intake wells.

Assessment of water quality variations associated with artificial replenishment

The quality of the water withdrawn by a water intake operating in conjunction with water replenishment depends on a number of physicochemical and biological processes. It is also necessary to anticipate the presence of multiple zones with differing purifying capabilities. For example, replenishment via influent seepage pools is associated with the formation of three filtration zones in the aquifer: a silting zone (I), a zone of incomplete water saturation (II), and a zone of complete water saturation (III). Each of these zones is characterized by its own set of hydrodynamic and physicochemical characteristics which affect the process of water self-purification during seepage.

As may be concluded from the results of field and laboratory trials, considerable changes in the quality of the influent water occur in the silt sediment and deep-silted rock zone, as well as lower down, in the zone of incomplete rock saturation. These include the retention of the greater part of the suspended particles, up to 90–98 % of bacteria, the biochemical decomposition of organic matter, and the sorption of anionic elements, electrically neutral organic compounds, and a number of heavy metals. A considerable role in the overall water purification process is played by absorptive processes, namely chemosorption, ion exchange, and adhesion to active silt particles, to sand grains, and to the sandy filler of shingle beds. All of this makes it necessary to study and to predict water quality separately for each of the filtration zones identified taking account of their hydrodynamic characteristics and conditions.

The silted zone (I) includes the silt deposit or sediment and a layer of silted or clogged rock which acts as a secondary filter. Water movement in this zone, whose thickness usually does not exceed 0.3–0.5 m, occurs under conditions which are close to full saturation, the natural moisture content of the silt deposit being as high as 80–90 %. The physicochemical regime in this zone is

determined by the presence in the arriving water of dissolved oxygen which is expended in the course of microbial decomposition of organic matter. Owing to the large specific surface area of the silt and clay particles, the silted zone exhibits enhanced sorptive capacity as compared with the underlying layer of unsilted ground.

Data obtained by determining the equilibrium concentration of micropollutants with allowance for the sum total of their absorption have made it possible to develop the following formula for the predictive estimation of the degree of water self-purification in the silted layer:

$$\Delta C_1 = C_{01} - \frac{n_1 N_1}{\frac{K_0 t_1}{m_0 - h_0 \ln \frac{m_0 + h_0}{h_0}} - n_1} \qquad (4.112)$$

where C_{01} is the initial concentration of micropollution in the surface water; n_1 is the active porosity; N_1 is the sorptive capacity; m and K_0 are the thickness and filtration factor of the silted layer; t_1 is the duration of seepage through the silted layer; h_0 is the depth of water in the pool.

Zone II, which lies beneath the silted zone, is a layer where seepage is not accompanied by complete filling up of the pores. The thickness of this zone often varies from 1 to 10 m over a single artificial replenishment site. Field measurements have indicated that with an initial moisture content of 2–5 % in the unsaturated zone prior to artificially induced influent seepage, a seepage velocity of 0.4–3 m/day from the pools raised that moisture content between 10 and 18 %. With a porosity in the aeration zone of 0.3–0.35, it turns out that roughly 50–70 % of the pores are filled with air, and occasionally with enriched carbon dioxide and nitrogen due to the biochemical decomposition of organic matter.

In contrast to the horizontal flow in an aquifer, which is characterized by fully saturated conditions the movement of pollutants through the unsaturated zone under the action of gravity must be determined with allowance for the moisture conductivity factor of the rocks. The following formula is recommended for assessing water quality in connection with sorption processes in the zone of incomplete saturation:

$$\Delta C_2 = C_{02} - \frac{n_2 N_2 h_\alpha}{t_2 \sqrt[3]{K_m V^2} - h_0 n_2} \qquad (4.113)$$

where C_{02} is the quality index of the water arriving in the partially saturated zone from the zone of silting; N_2 is the sorptive capacity of the rocks and soil of the partially saturated layer; n_2 and h_α are its active porosity and thickness, respectively; t_2 and V are the seepage time and velocity; K_m is the moisture conductivity factor of the rocks. The remaining symbols represent quantities already defined.

Thus, the prediction of variations in the quality of grundwater which is artificially replenished with the aid of seepage pools requires a determination to be made of the filtrational, migrational and sorptive parameters of each filtration zone.

In the case of the horizontal water seepage zone (zone III) the relative concentration of matter at any point with the coordinate x(t) which is required for assessing water quality with allowance for the sorption, dispersion and absorption occurring in plane flow away from the linear contour of artificial replenishment is given by the formula

$$\bar{C}(t) \approx 0.5 \operatorname{erfc} \left(\frac{x - V^* t}{2\sqrt{D^* t}}\right) \qquad (4.114)$$

$$\bar{C} = \frac{C - C_n}{C_a - C_n} \qquad (4.115)$$

where C is the concentration (in mg/liter) of a given component in the artificially replenished groundwater; C_a is the concentration (in mg/liter) of the same component in the artificial replenishment feed water; x is the distance (in m) from the replenishment contour to the calculation point; $V^* = \frac{V \cdot \beta}{n(1+\beta)}$ is the velocity (in m/day) of the advancing pollution front with allowance for sorption; V is the seepage velocity (in m/day); n is the active porosity of the rocks; $D^* = \frac{D \cdot \beta}{n(1+\beta)}$ is the dispersion coefficient (in m^2/day) with allowance for sorption; D is the coefficient of convective dispersion (in m^2/day).

The dispersion coefficient is related to the seepage velocity by the formula

$$D = \lambda V \qquad (4.116)$$

where λ is a parameter (in m) characterizing the geometric structure of the pore space.

In the case of a linear Henry isotherm the coefficient of distribution β of the substance in question with allowance for its sorption (absorption) is given by

$$\beta = \frac{C_{eq}}{N} \qquad (4.116a)$$

where C_{eq} is the equilibrium concentration (in mg/liter) of the pollutant in the water; N is the sorptive capacity (in mg/liter) of the rock, which represents the limiting amount of the component susceptible to sorption for a given concentration of that component per unit volume of the rock.

Variations in water quality with respect to certain parameters in the course of influent seepage can also be predicted with the aid of empirical relations obtained on the basis of filtration column experiments (H. Zöffler, W. Pietsch, W. Huhn, 1973). According to these data the general equation describing the decrease in the amount of organic matter in water seeping through rock can be written as

$$\Delta pV = \lambda \ln \frac{L}{L_0} \qquad (4.117)$$

where ΔpV is the decrease in the amount of potassium permanganate (in mg/liter) along the seepage path; L is the computation seepage path length (in m); L_0 is the ideal seepage path length (in m) corresponding to $\Delta pV = 0$ in a small-scale experiment; λ_g is the decomposition reaction rate constant (in mg/liter) according to a small-scale experiment involving the determination of ΔpV values corresponding to two filtration path lengths (0.3 and 0.6 m).

Similar relations are available for recalculating experimental conditions on the basis of different temperatures, seepage rates, and effective diameters of ground particles.

In establishing the feasibility of artificial replenishment, and more particularly in predicting eventual groundwater quality one must bear in mind the fact that the changes which occur in the course of seepage can be subdivided into two basic groups of physico-chemical indices: those which do not vary greatly with seepage (this applies to the total mineral content, sulfates, chlorides, and hardness), and those which vary as a result of

sorption and other processes (turbidity, oxidability, bacterial indices, as well as iron, nitrogen compounds, and oxygen content). In general, if the processes of dispersion, sorption and absorption are neglected, then the predicted concentration of an individual component in the groundwater withdrawn by a water intake due to bulk mixing with water arriving from an influent seepage pool is given by

$$C_W = \frac{C_0 \alpha_1 Q_{is} + C_n (Q_W - \alpha_1 Q_{is})}{Q_W} \quad (4.118)$$

If, one the other hand, seepage from a river contributes to the water intake discharge, then the concentration of a given component, provided mixing is the only factor, is given by

$$C_W = \frac{C_0 \alpha_1 Q_{is} + C_r \alpha_2 Q_r + C_n (Q_W - \alpha_1 Q_{is} - \alpha_2 Q_r)}{Q_W} \quad (4.119)$$

where C_W is predicted concentration (in mg/liter) in the groundwater withdrawn by the water intake; C_0, C_r, C_n are the concentrations of the given component in the surface water, in the river, and in the natural groundwater, respectively; Q_W is the planned water intake output (in m^3/day); $\alpha_1 \cdot Q_{is}$ is the discharge (in m^3/day) drawn to the water intake and made available for influent seepage; $\alpha_2 Q_r$ is the seepage discharge (in m^3/day) attracted to the water intake from the river.

The values of αQ for various sources contributing to the discharge of the water intake are determined by the hydrodynamic method for specific boundary conditions, artificial replenishment systems, and water intake types.

It should be noted in conclusion that sufficiently reliable prediction of the influence of all processes on the purification of natural water in the course of seepage is a highly complex matter. This is why experience in the operation of water intakes already working in conjunction with artificial replenishment systems assumes such importance along with meticulous and thorough physicochemical and biological observations.

4.7.3. THE CONCURRENT USE OF GROUNDWATER RELEASED IN MINE AND LAND SURFACE DRAINAGE AS AN ACTIVE FORM OF GROUNDWATER PROTECTION AGAINST DEPLETION

The extraction of useful minerals, and especially mineral ores, is accompanied by the release of large amounts of groundwater. The water content of mine and quarry ground can reach values of 10 m^3/ton and even higher, i.e. the mining of a single ton of mineral ore means removing 10 m^3 (metric tons) of water. In the mid-1970s, for example, the average water content in the USSR coal industry was 3 m^3/ton [9]. The issues relating to the use of mine waste water for antidepletion and antipollution protection of groundwater have been dealt with in some detail with reference to USSR experience by M.S. Gazizov and coauthors [9]. According to this source, the mining enterprises administered by the USSR Ministry of Ferrous Metallurgy alone produced 500 million m^3 of water annually over the period 1971–1975. Still larger amounts of groundwater were released by the coal industry. Water inflow into coal mines and cuttings reached 1 870 million m^3 (60 m^3/sec) in 1970; the corresponding estimate for 1980 is 2 140 million m^3 (68 m^3/sec). The increased depth of mining excavations expected by the year 2000 will increase electric power expenditures on dewatering approximately five times when compared with figures for 1970. If to this we add rough estimates for the amounts of water released by other branches of the mining industry (nonferrous metallurgy, the building materials industry, etc.), we find that in 1980 the total amount of water pumped from mining excavations amounted to 3.0–3.5 billion m^3 (or 95–110 m^3/sec) [35]. It is likely that the total amount of groundwater released per one million tons of mined minerals is roughly the same in any of the world's countries with a developed mining industry. Water inflow into the wettest mining excavations and their systems is in the low thousands of cubic meters per hour, reaching ten to twelve thousand cubic meters per hour during flood periods (as happens, for example, in the shale basin mines in the Baltic area). Such high rates of mine water release are associated with the formation of extensive areas of groundwater table depression whose radius can be as large as several tens and even a hundred kilometers. The formation of such extensive depression cones, whose depth at the points of maximum ground water table drop averages several hundred meters, inevitably leads to aquifer depletion in certain areas. This is why the rational use of mine waste water is a problem of paramount importance for all countries with a developed mining industry.

The use of mine water for supply purposes (industrial, irrigation, and sometimes even domestic use) makes it possible to reduce groundwater uptake, thus preventing aquifer depletion, or else to organize artificial groundwater replenishment operations.

However, all of this requires serious consideration to be given to the quality of groundwater pumped out in evacuating mine excavations. As far as the USSR is concerned (although, as we see it, the same characteristics must be more or less valid for mining areas in other parts of the world), the groundwater occurring in mining areas is either neutral or alkaline. The fact that it flows along exposed surfaces means that it is contaminated with ground particles, coal dust, chemicals and bacteria [9]. Mine water likewise often exhibits specific forms of pollution, e.g. by petroleum products (grease and oil drop the hydraulic systems of mining machinery, mechanical shoring, etc.), phenol (coal and shale combustion products in worked spaces or at surface dumps), and other substances. A description of mine water composition based on data for the principal coal mining basins of the USSR is presented as Table 4.7 (see [9]).

Mine water in coal basins is contaminated by suspended matter (this applies to over 90 % of the total amount of water released), sulfates and chlorides. It often contains a considerable amount of iron and exhibits acidity and bacterial contamination. Approximately 53 % of the total amount of groundwater released is mineralized, and about 10 % is of acidic composition. The principal factors contributing to the formation of acidic waters are oxidation and the leaching out of sulfide and organic sulfur, as well as the metabolic activity of Thiobacillus ferri oxidens. Acidic mine water is characterized by a high sulfate concentration (over 3 000 mg/liter in the Kizel basin and over 7 000 mg/liter in the Donetsk basin) as well as iron (over 1 000 mg/liter in the Kizel basin and up to 30 mg/liter in the Donetsk basin) [9].

The very brief description of the quality of groundwater released as a result of mining activities (especially coal mining) shows that its use in the economy requires considerable caution. At present mining groundwater use is largely confined to industrial applications. In order to be rendered fit for use for other purposes (domestic needs, irrigation, artificial groundwater replenishment,

etc.), mine water must first be subjected to extensive purification, which is far from being economically feasible in every instance. On the other hand, mine and ore water in such areas as the Estonian coal basin, the Northern Urals bauxite area or the Mirgalimsai mines in the Karatau regions, where hundreds of thousands of cubic meters of high-quality fresh water are pumped out daily, must certainly be put to sound economic use. In the 1970s roughly 90 % of mine water was dumped into rivers, with only 10 % used by ore dressing plants. In the 1980s new government regulations coupled with generally improved water quality are shifting this ratio towards the greater use of mine water in the economy. The same trend is evident in waste water treatment: whereas in the mid-1970s only 40 % of mine water was treated while

Table 4.7

DESCRIPTION OF THE CHEMICAL COMPOSITION OF MINE WATER ON THE BASIS OF DATA FOR THE PRINCIPAL COAL AND SHALE BASINS*

Basin	pH	Average water composition data (in mg/liter)				
		suspended matter	dry residue	sulfates	chlorides	total iron
Donetsk	3–9	20–13000	300–6000	130–6000	120–3200	0–30
Kuznetsk	6.6–8.5	30–12000	300–1500	20–300	10–100	0.21–1.60
Karaganda	6.9–8.4	100–6700	1000–9900	300–3000	100–3400	–
Pechora	7.3–7.7	20–1000	460–1640	20–1640	20–400	0.40–1.00
Kizel	2.8–4.0	160–5400	1050–5400	1050–5400	49–3800	54–1170
Moscow	4.5–8.2	20–2500	300–1200	24–700	3–50	0.1–9.00
Baltic (shale)	7.2–8.7	1–300	300–1300	10–600	10–300	0–1.20

*Data supplied by the Perm' Coal research Institute

60 % was simply dumped (mainly into rivers), by the early 1980s the ratio became roughly fifty-fifty, with the ratio of treated mine water continuing to increase. Appropriate water legislation adopted in the early 1970s played an important role in this regard. The coming ten to fifteen years will see the implementation of new, more modern methods of mine water treatment for the removal of suspended matter. Demineralization is being conducted by means of distillation facilities (for mineral contents exceeding 5 g/liter) and electrodialysis plants (for a mineral content up to 3 g/liter). Plans are underway for treating acid mine water by the dual-mode neutralization method using slaked lime, with the use of sulfate-reducing bacteria being considered as another possibility [9]. All of this will make it possible in future to achieve a marked increase in the use of mine water in the national economy. Due notice should also be taken of a certain fact which has had an important bearing on the use of waters from mineral ore mines: when the latter are situated in an arid or semiarid zone characterized by short supplies of water (e.g. the Donetsk and Karaganda areas) the mine water is generally unusable due to its excessive salinity, high iron content, etc. On the other hand, the humid zones where much high-quality mine water is usually available (as in the Baltic shale basin, the Moscow basin and the Northern Urals bauxite area) are not as a rule in need of such water. It is only at Karatau (in southern Kazakhstan) that all of the water released at the Mirgalimsai mines is used for irrigation.

Control of water regimes in mining areas and water regulation with a view to protecting groundwater against depletion by deep drainage are for the most part implemented by providing antiseepage screens. These constitute a barrier to protect the natural stocks contained in an aquifer (subsurface runoff) and ensure the confinement of dumping to the groundwater actually present in the ore mining area. Unfortunately, antiseepage screens cannot be made to extend much deeper than 100–200 m, which somewhat restricts their use in mining. On the other hand, the use of antiseepage screens is certainly justifiable in cases where the mineral ore in question lies beneath a shallow aquifer. As was shown by M.S. Gazizov [9], the use of such screens at the Kursk Magnetic Anomaly mines would make it possible to forestall the depletion of fresh groundwater stocks and the pollution of watercourses by pit water. According to the same author, the same device at the iron ore pits in the Kustanai area would permit the preservation of the natural saline groundwater regime and prevent the contamination of the surface water of the river Tobol by the salty pit waters.

An active form of groundwater protection against depletion consists in the concurrent use not only of mine water, but of other drainage facilities as well. Groundwater drainage is fairly common practice in modern engineering. We are referring, however, not to modern drainage practices associated with needs such as table lowering in civil engineering, but rather to continuous drainage by means of reservoirs to protect urban areas from flooding, as well as to the drainage of irrigated acreage. Examples of the use of table-lowering facility waste waters may be seen in the barrage wells constructed in certain cities and towns along the Volga (Ul'yanovsk, Kazan', Gor'kii et al.) to prevent the possible flooding of urban districts due to the building of reservoirs along that river. The output of these table-lowering systems is sometimes in excess of 100 000 m^3/day, with some of the water used for industry and the remainder dumped. The water in question is of high quality and low mineral content. It is free of harmful impurities and can be used to satisfy various water needs, including artificial groundwater replenishment [40]. An important aspect of groundwater protection against depletion in irrigated areas is the recycling of drainage water. (As we know, irrigation is

probably the most water-intensive branch of the economy: wherever irrigation is based on the use of groundwater without its concomitant artificial replenishment, the inevitable result is gradual aquifer depletion). In the arid regions of Soviet Central Asia where irrigation is developing at extremely high rates, the water resources utilization factor sometimes exceeds 1.5, which results in considerable savings in terms of satisfying the need for "fresh" water and at the same time helps to protect groundwater from depletion.

It should also be noted that an important role in the protection of groundwater against depletion is played by the use of treated effluents, largely of the household and domestic variety. In Los Angeles (USA), for example, the creation of a barrier against salt ocean water and the replenishment of aquifer groundwater used for public water supply needs was assisted by complete biological recycling of city sewage water at the Hyperon treatment plant [58]. The quality of the water entering the aquifer is in conformity with US Public Health Service standards [65]. The treated effluent water does not affect the sanitary condition of the groundwater and has no aggressive effect on equipment. A 19-month treated-effluent pumping trial showed that certain components and characteristics of the water improved in the course of ground filtration (this applies to phenols, nitrogen compounds, chemical oxygen demand, color, bacterial count), while others remained within stable limits (carbon compounds, fluorine, borates, detergents). The iron concentration dropped sharply with preliminary chlorination of the recharged water, while detergents (alkyl benzene, sulfonates) could be removed by filtering through activated charcoal. Considerable improvement in water quality resulted from preliminary fast filtering at speeds not exceeding 3.6 m/hour. The cost of conditioning effluents for aquifer replenishment is 1.5 US cents per cubic meter, with activated charcoal treatment rendering the conditioning process more expensive by a factor of 1.5. The quality of the water was already considerably improved (compared with that of the feed water) at a distance of just 45 m from the injection well, while at 90 m it satisfied virtually all US Public Health Service requirements.

Urban effluents subjected to complete biological treatment are used in Israel to replenish groundwater in the sands of the Coastal Plain [67]. It is intended to increase the use of effluents from Tel Aviv for this purpose to a level of 300 000 m^3/day. An interesting example of successful groundwater replenishment with industrial effluents in the USSR is reported by I.V. Kononov [18], the water in question coming from a wool washing factory. All of these facts indicate that properly treated effluents may well become an important factor in the rational use of water resources, making it possible to conserve groundwater and to protect it from depletion.

Groundwater protection against depletion and the economical use of groundwater (and water in general) necessarily entail the integrated and most complete possible reliance on all potential water sources. This includes the concurrent use of the groundwater pumped out in the course of mine evacuation (in cases where this is possible, with due regard for the quality of the pumped water), the repeated use of drainage water, including irrigation drainage water, for irrigation needs (provided its quality remains sufficiently high), as well as the use of treated sewage effluents. These issues are most acute in the arid areas of the world with their perpetual water shortages. It is necessary in all circumstances, however, to insist on and enforce appropriate water quality requirements, especially when the water in question is eventually intended to be a source of public water supply.

4.7.4. THE INTEGRATED USE OF SURFACE AND GROUNDWATER, WITH ALLOWANCE FOR THEIR INTERRELATION

Groundwater protection, especially against depletion, consists in the rational and integrated use of the water resources of a given area of the world. This is especially true of areas where water is scarce, and most particularly in the arid zone. It is legitimate under some circumstances to opt for a temporary depletion of aquifers, provided this does not result in excessive deterioration of the quality of the water withdrawn. Such a situation arises when no sufficiently large and powerful sources of groundwater replenishment are available, while the needs of the national economy must be met without delay. In such circumstances one has no choice but to accept the depletion of available storage capacity until such time as a new water source comes into play in the area. For example, even though it resulted in a drop in the tables of certain groundwater deposits by several tens of meters in some places, the extensive use of groundwater in the vicinity of Dzhezkazgan in the USSR made it possible to satisfy fully the needs of the local metal ore mining industry and of the local population until such time as water from the Irtysh River could be delivered along the Irtysh-Karaganda Canal first to Karaganda and then to Dzhezkazgan [23]. However, such an expedient is necessarily one which is not very desirable. In fact it must be considered exceptional and be resorted to only when losses to the economy due to water shortage are assessed as being greater than the cost of water delivery from outside to satisfy today's demand.

There is no longer any need now, as it was necessary to do even in the 1960s, to prove that water is something which is "one and indivisible", and that the water resources of a given region must be considered in an integrated way rather than as something which can be divided into "groundwater" on the one hand and "surface water" on the other. In fact, the system of regional-level assessment of groundwater resources in the USSR [30] incorporates a set of estimation and assessment methods which covers the possibility of making allowance for the same water twice over, once in the form of natural groundwater stocks, and later as part of the surface runoff of rivers, even though this part is nothing other than the same groundwater already accounted for by hydrogeologists, but which has now made its way into a river or rivers.

We note that water resources construction, and in particular the building of dams and reservoirs along rivers, usually tends to enhance groundwater feed, at least in the upper reach. Thus, the building of the Kuibyshev Reservoir on the Volga increased groundwater tables by 10–17 m due to the additional pressure exerted in the shore area. This made it possible to increase the amount of water withdrawn by intakes several times over [26, 32].

On the other hand the more intensive uptake of groundwater, particularly by bank-side intakes, tends to reduce river flow.

When assessing the reduction of river flow occasioned by the withdrawal of groundwater by water intakes, hydrogeologists, hydrologists, and water resources managers are concerned with a limited number of groundwater

deposits, namely deposits situated in river valleys, in structures of limited area constructed of fissured and karst rocks, and those which are associated with rivers and lie in fluvioglacial deposits and outwash plains. The decrease in surface flow which accompanies groundwater withdrawal from such deposits can be markedly high, and (in the case of smaller rivers) quite considerable indeed. Estimates of river flow reduction due to groundwater abstractions are not very important with other deposit types. Examples of such deposits are platform-type artesian basins, debris cone deposits, etc. Groundwater use in a debris cone setting does not affect surface flows above the toe of the cone due to the depth of groundwater and the isolation of the groundwater table from the surface level. As far as the area below the toe of the cone is concerned, the absence of an effect is due to the fact that water in such areas is used largely locally and chiefly for irrigation, so that even though springs in this zone, as well as various "karasu", do tend to run dry as a result of groundwater uptake, the returning irrigation water tend to supply surface flow below the irrigated area, so that injury to river flow is scarcely noticeable. By "river flow reduction" we mean the total decrease in the transit flow and subsurface feed of a river (or rivers) as a result of groundwater uptake within the area in question.

It should be stressed that problems of estimating the losses to river flow due to groundwater uptake are of importance only in relation to smaller rivers (of length less than 200 km, with a catchment area not exceeding 3–4 thousand km^2). It is only very rarely that they assume significance for "medium" rivers (i.e. rivers whose length and catchment area are larger than the indicated values). Some quite instructive figures applicable to Belorussia have been supplied by researchers of the TsNIIKIVR [33]. The researchers at the Institute investigated the impact on river flow of 144 groundwater intakes with an output of up to 108 000 m^3/day (many water intakes have an output of 40–80 thousand m^3/day). The detriment to river flow due to the major centralized withdrawal of groundwater depended on river size (water content) ranged from 0.01 % up to 50–80 %, and occasionally even 100 % of the average long-term discharge. In the case of smaller rivers the effect of groundwater intakes is indeed considerable (over the territory of the Belorussian SSR the reduction to river flow was found to range from 10 % to 50 %, equalling 100 % and 84 % in two individual cases). For medium and large rivers with discharges exceeding 7–8 m^3/sec the loss to river flow does not exceed 1–4 %, while in the case of such rivers as the Dnieper, Berezina, Sozh, Viliya, Neman, Shchara, Western Dvina, Pripyat', Mukhovets et al. the loss to river flow due to groundwater intake operation in their valleys ranged from 0.01 % to 1.0 %. It should be noted that groundwater uptake can even increase river flow due to the dumping into rivers of used groundwater not associated with surface waters. These data demonstrate once again that the problem of assessing the losses to river flow due to groundwater intake operation in river valleys is necessary only for smaller rivers whose maximum size has already been indicated. In arid regions where water shortages are most acute the amount of water needlessly lost is inordinately high, since almost 100 % of the annual runoff is associated with flash floods when much of the water evaporates in salt marshes and drainless saline depressions. Even in places where river flow is regulated, the available reservoirs cannot retain the entire supply Over a three-year period (1971–1973), for example, 550 million cubic meters of fresh flood water had to be let go over the dam at Dzhezkazgan [42]. On the other hand, the building of river reservoirs is not always the most rational way of regulating surface flow in arid regions where there may be considerable evaporation losses, which can exceed 1 000–2 000 mm annually. This is why the first experiments on the conversion of surface runoff to subsurface flow were undertaken in the arid zone. For example, in the late 19th century farmers in California (USA) cleared dry creek beds in order to increase floodwater absorption. Another example comes from Turkmenian SSR, where measures were taken with a view to intensifying the accumulation of water formed on the "takyrs" during the spring season due to snowmelt and precipitation for storage in the underlying sands ("subtakyr lens" formation). Such measures required an increased use of groundwater, since, as is shown in Section 4.7.1 of the present monograph, the storage of water in subsurface cavities is usually impossible without preliminary or simultaneous water withdrawal.

The need to rationalize the water economy, especially in the arid regions, and the need for integrated and optimal use of ground and surface water resources, have been the guiding considerations in the creation of water resources management systems, Indeed, the optimization-oriented planning of water resources use within the context of such systems establishes their rightful claim to being one of the most effective measures directed at protecting groundwater against depletion. Examples of countries where such systems are being successfully operated include the USSR (the Crimea, whose case is discussed above, Karaganda [17], Tbilisi [43], Dzhezkazgan [42]), the USA (California and a number of other areas [47]), and the Netherlands [51]). One important element in the rationalization of water resources use and groundwater protection against depletion are interbasin runoff transfers. Examples include the Karakum Canal and the Irtysh-Karaganda-Dzhezkazgan water delivery system with its overall length of 1 000 km and a system head feed of several tens to a few hundred cubic meters per second. Also noteworthy are the runoff transfer operations in the USA (e.g. from northern to southern California, or from the Havasu reservoir on the Colorado River to the Los Angeles area [47]). Plans are underway in Spain to transfer water from the river Ebro to the industrial areas of Barcelona and the Costa Brava, as well as from the river Tajo into the Júcar River basin in the southern part of Valencia via a 200-km canal with a throughput of several dozen cubic meters per second.

The facts and data cited in the present section are intended, as far as possible, to take stock of the principal trends in the development and improvement of the integrated use of water resources from the standpoint of groundwater protection against depletion. They indicate that given expanding production and growing demand for water, coupled with limited water resources (especially in the arid zone), the future belongs to integrated systems of water supply and allocation providing for the simultaneous and optimal use of all water resources (local, surface and subsurface, supplied from outside via interbasin transfer systems, etc.). Moreover, since groundwater is generally best from the standpoint of mineral content, physical characteristics, sanitary condition, temperature and other criteria, it must be used first and foremost to satisfy the most important of needs – that of public

water supply. This imperative in turn prescribes regimes of groundwater use which preclude the possibility of groundwater depletion.

4.8. METHODOLOGY FOR CONSTRUCTION OF GROUNDWATER PROTECTION MAPS

A successful solution of groundwater protection problems requires very specialized cartographical methods producing special purpose maps usually called groundwater protection maps or aquifer-vulnerability maps or maps of aquifer – contamination potential or contamination hazard. Groundwater protection maps provide the best graphic representation of hydrogeological and other factors related to groundwater pollution and protection. These maps should serve above all as a guide for water-economic, planning and administrative authorities to overcoming the problems of haphazard, uncontrolled development, deteriorating quality and other impacts on groundwater. They also should be helpful to planners, hydrogeologists, municipal and industrial officials in locating safe waste disposal sites, ascertaining potential groundwater pollution problem areas and selecting proper precautionary measures to be taken before serious groundwater contamination can occur. Synoptical maps of groundwater pollution should provide moreover a basis for development of environmental impacts assessments on groundwater storage (number, concentration and extension of pollutants).

Considering the fact that team work in solving groundwater protection is often indispensable, such maps are the best means of communication between experts of different professions. Another advantage is that maps speak across the barriers of language by the use of standard colours, patterns and symbols. From this point of view the world-wide standardization in this field is very desirable.

Classification of maps is a standard first step in the process of scientific analysis and synthesis, making it possible to assess the degree to which existing maps or maps in preparation meet their scientific and practical purposes. It also provides guides to the types of maps best suited to particular purposes. Maps in general can be classified in many ways and no one classification is satisfactory for all purposes.

The maps of groundwater pollution or protection are usually classified as special purpose maps within the range of environmental maps. The content and technical processing of environmental maps are still developing. At present neither national nor international standardizations in this field exist. The boundary between special and applied maps is sometimes difficult to determine. The applied maps differ from special ones principally by selection of information presented.

Another way to classify the maps related to groundwater protection arises from the fact that groundwater forms an important element of the hydrological cycle. From that point of view we can incorporate these maps into the wide rank of hydrological maps. A schematic matrix for classifying hydrologic maps on the basis of purpose, reliability, scale and data geometry has been adopted by UNESCO (1977).

A classification of maps related to groundwater protection as such does not exist at present. As for content we can recognize so called maps of groundwater vulnerability, maps of groundwater contamination or maps of groundwater protection. We can also differentiate between maps serving for preventive protection and for decontamination of groundwater already polluted. As for the geographic extent of an area under study, maps for solving problems of global, international, national, regional or local characters can be distinguished.

A proposed classifying scheme based on scales, contents and purposes of maps in question is presented in Table 4.8.

In the middle of the sixties the conceptual and methodological processes needed to compile groundwater protection maps were being developed.

The map of aquifer contamination potential in Illinois, USA presented by W.H. Walker (1968) is an example compiled at that time with a special view to groundwater protection problems.

In Czechoslovakia M. Olmer (1967) compiled a map showing the threat to groundwater by oil hydrocarbons and M. Vrána (1968) presented a groundwater protection map of the Czech Socialist Republic (Bohemia and Moravia) on a scale of 1 : 500 000.

Table 4.8

PROPOSAL FOR GROUNDWATER PROTECTION MAPS CLASSIFICATION

Scale		Contents	Purpose	Examples
Extra large	1 : 5 000 and larger	sanitary and other protective zones of drinking water sources and natural healing resources, hydrogeological data of special importance in local safeguarding measures, data related to sanation of oil hydrocarbons incidents	special-purpose maps for limited number of users such as water-works and spa administrations, oil hydrocarbon industry officers, hydrogeologists etc.	maps of local importance compiled mostly regardless to the uniformity of methodical procedures (majority of them solves the problem partly); examples not indicated because of their great variety
large	1 : 10 000 1 : 25 000 1 : 50 000	water bearing properties of aquifers and their contamination hazard; potential and existing sources of point contamination; water-economic objects; sanitary zones of ground water; waste deposits; hydrogeological, hydrological, hydrochemical	1) multipurpose maps of areas with first-class hydrogeological and water-economic importance; to be used by wide range of research institutes, enterprises,	Zeman et al. (1973) – Czechoslovakia Vrána (1976) – Czechoslovakia (proposal) I.G.M.E. (1979) – Spain Lavie – Putallaz (1974) – France

Scale		Contents	Purpose	Examples
		and meteorological data relative to respective protection rules	authorities etc. 2) special purpose maps for locating waste disposal sites, oil hydrocarbon stores, pipe lines etc.	Beauduc-Lienhardt-Rousselot (1976) France Kleczkowski-Myszka-Pyrich (1973), Poland
medium	1 : 100 000 1 : 200 000 1 : 500 000	contamination potential (contamination hazard) of aquifers, required stage of regional groundwater protection, large groundwater catchments, sanitary zones of drinking water, protective zones of natural healing resources, sources of non-point contamination	synoptical maps satisfying requirements of water-economic, water-planning and administrative authorities on district, provincial and/or on a nation-wide scale	Walker (1968) – USA Vrána (1968) – Czechoslovakia Olmer-Řezáč et al. (1974) – Czechoslovakia Lemaire-Martin (1973) – France Tosan-Lienhardt-Collin (1975) – France Rogovskaya (1976) – Soviet Union
small	1 : 1 000 000 to 1 : 10 000 000	analogous to that of medium scale, sometimes with certain simplification, usually displaying less than three elements related to groundwater protection	general synoptical maps to be used for solving problems on a nation-wide and an international scale	Albinet (1970) – France Antonov-Rajkova (1978) – Bulgaria Macioszczyk-Plochniewski (1979) – Poland
extra small	smaller than 1 : 10 000 000	maps based on estimation and generalization of items above mentioned; mostly depicting one element only	maps solving problems on continental and global scale	not constructed to-date

A similar map was compiled by M. Banský et al. (1970) on the same scale for the territory of the Slovak Socialist Republic.

The greatest progress in construction of groundwater protection maps occurred in France in the beginning of seventies. The map of aquifer vulnerability from pollution in France on a scale of 1 : 1 000 000 compiled by M. Albinet (1970) can be considered a classic example of such maps. The French experience with groundwater protection promptly gained access to South America as illustrated by R. Kreimer (1970) from Buenos Aires.

The examples presented demonstrate the methods used in the first phase of the development of groundwater protection maps. The authors tried to solve the problems by constructing synoptical maps on a small scale which covered almost the entire state territory. This method may be considered logical, taking into account the fact that the governmental authorities urgently needed such a basis to solve the most pressing and important tasks of groundwater protection.

The second phase of this methodical development is characterized by a transition to maps on a medium and large scale. These maps have been developed because of the requirements of groundwater protection in smaller territorial units.

In France, there were edited by B. Lemaire and P. Martin (1973) two map sheets on a scale of 1 : 100 000 showing groundwater contamination hazard (sheet Montpellier, sheet Basse Valle de l'Aude), later on four sheets of a vulnerability map on a scale of 1 : 50 000 by J. Lavie and J. Putallaz (1974). Next year there appeared map sheet Lyon of groundwater vulnerability map on a scale of 1 : 250 000 compiled by R. Tosan, G. Lienhardt and J.J. Collin (1975). With the same sheet Lyon but at a scale of 1 : 50 000 by P. Beauduc, M.J. Lienhardt and D. Rousselot (1976) a new edition of groundwater vulnerability maps has been initiated, of which four sheets appear every year.

The groundwater protection map of the Czech Socialist Republic on a scale of 1 : 200 000 comprising 18 sheets was compiled by M. Olmer, B. Řezáč et al. (1974). The legend for large scale groundwater protection maps (1 : 25 000 – 1 : 50 000) was proposed by M. Vrána (1976).

Other maps related to groundwater protection were compiled in that period in Soviet Union (N.N. Rogovskaya, 1976), in People's Republic of Poland (A.S. Kleczkowski, J. Myszka and J. Pyrich, 1973; A. Macioszczyk and Z. Plochniewski, (1979) and in People's Republic of Bulgaria (Ch. Antonov and B. Rajkova, 1978). The method used to compile these maps consisted mainly in depicting natural safeguarding conditions of groundwater.

A very specialized map on a large scale (1 : 50 000) was developed by the Geological and Mining Institute of Spain (1979) for the purpose of planning municipal solid waste disposal.

The methodology for constructing groundwater protection maps is still developing. As the problem is now considered to be an integral part of environmental protection, such an approach requires certain modifications of methods to be used.

The need for more satisfactory methods of safeguarding groundwater has arisen with the recognition that

the problem forms integral part of natural environment protection as a whole. Such an approach requires modified cartographical methods to be used for construction of groundwater protection maps. The legend for large scale maps (1 : 10 000 – 1 : 50 000) proposed by M. Vrána (1976) may be considered methodical contribution for the preparation of such maps. These multipurpose maps should be compiled in all areas having first-rate hydrogeological and water-economic importance and which are subject to a high contamination hazard.

The principles of the map consist of coloured representation of water-bearing properties of 1–2 main aquifers and of depicting their contamination hazard. Seven categories are distinguished by colours arranged in a conventional order (red for high contamination hazard, orange and yellow for moderate and blue and green for low). The occurrence of two superimposed aquifers is expressed by stripping. Transmissivility of aquifers is depicted by shading of respective colour in four categories. The superimposed hatching illustrates the permeability of the subsoil covering layer, a factor that is very important for estimating the vertical movement velocity of pollutants towards aquifers. Usually it is sufficient to recognize three categories, but other divisions may be adopted for local conditions. Basic data can be derived from pedological and engineering-geological maps, but some simplification is often required.

A wide range of point and line symbols (about 60 items) represent sources of groundwater contamination, water supply objects, drained and irrigated areas, groundwater protective zones etc.

The outlook for the development of groundwater protection maps over the next few years could be characterized as follows: methodical principles for compilation of large and medium scale maps will further progress because of increasing demands for local and regional groundwater protection. In the course of increased collaboration between related scientific disciplines the integration of groundwater protection maps in the range of environmental maps will become necessity. Later on, when an abundance of material from special purposes maps on large scale will be available, the attention of map compilers will probably be focused to the preparation of small scale synoptical maps to be used for international collaboration in this field.

A further development is anticipated the use of remotely sensed and digitalized data, above all for maps on small and extra small scales. Some of the classical form of maps in which features are represented by conventional symbols will eventually be supplemented or replaced to a large extent by photoimage maps or sensor-image maps. Map formats and symbolisation will be changed radically to accommodate techniques of cartographic automation. Cartographic information will be compiled and stored in digital form at a Central Data Bank which can be interogated at distant points through computer terminals to provide information (in either digital or graphic form) or to provide special map products.

The first step in this methodical process would require the world-wide standardization of sets of symbols and ways of depicting a wide variety of features characterizing groundwater protection. The constitution of Commission for Groundwater Protection within the IAH in 1978 can be considered as an important approach to this aim. The preparation of an international standard for groundwater protection maps similar to those already edited by UNESCO/IAH/WMO for hydrogeological (1970) and hydrological (1977) maps may be declared the prime requisite of all cartographers.

4.9. INVESTIGATION AND DECONTAMINATION OF OIL-POLLUTED GROUNDWATER

Field surveying is aimed at determining the extent of pollution. Clean-up activities should restrict the focus of pollution and should gradually decontaminate the rock or soil formations. The two kinds of activities are frequently combined in practice, in particular in the case of emergency. Investigation or surveying may be carried out as follows, from the standpoint of the mechanism of pollution

a) as a preventive measure at points of existing resources of potential pollution, where no spill of oil products has actually taken place,
b) at points of short-term pollution, such as the site of accidents to railway and round tankers, defects in oil pipelines, etc.,
c) at points of frequent and repeated spills associated with large-capacity storage facilities for liquid fuels, refineries, airports, etc.

PREVENTION OF SPILLS

The most effective prevention of groundwater pollution is the siting of plants used for processing, storing or transporting oil products at points where leaks do not endanger hydrogeological structures or exploited groundwater resources.

Generally speaking, the size of the area at risk increases with topographic elevation of the point of spill with respect to the collecting area. At the same time, the natural hydraulic resistivity of rock is overcome more readily by the increasing potential energy of the contaminant, or by that of the polluted groundwater.

For technical and economic reasons, it is impossible to eliminate all the potential sources of groundwater pollution from certain areas. For example, we cannot replace railways, airports, oil pipelines, etc., and these installations may have been constructed in the past without hydrogeological investigation. In that case, the preventive hydrogeological investigation has the purpose of recommending the optimum protection, including construction of observation facilities, and possibly also of clean-up systems.

Whereas observation wells should be made close to all facilities potentially dangerous to groundwater quality, clean-up measures are undertaken in the preventive stage only in cases of potential danger of large leaks, or in the proximity of significant groundwater resources. The inspection systems consist of a network of inspection wells or observation boreholes, which are specially equipped for detecting the presence of contaminants in water and possibly also for actuating alarm systems.

The first stage of investigation is aimed at obtaining data for an optimum selection of the site, and to define the requirements for groundwater protection. If it is concluded, that potential spills of contaminants do not represent a danger to groundwater or surface water, the investigation can be concluded in this stage [5]. The working method includes the study of archive data and field surveys. The technical work should be designed so as to be useful in later stages of the investigation.

The object of the second phase is to gather data for the final stage of groundwater protection. The second stage includes test boring, pumping tests and laboratory

analyses. Observation and monitoring wells are constructed at this stage, possibly together with facilities for hydraulic or static protection. The density of the observation well network depends on the physical conditions and on the significance of the structure to be protected. There is the important rule that each well should sample only one aquifer. Correct function of an observation well requires direct contact of the water table with the well screen throughout the monitoring period. Only in this way, can the presence of an oil layer on the groundwater table be detected correctly. The material of the well lining should not react with water and with oil products. The best observation wells therefore contain stainless steel linings, while the lining and screens of the other wells may be ceramics, earthenware or unpainted steel. Protective coatings which do not resist water and oil are not suitable; the same applies to plastics, which may be subject to extraction or absorption of hydrocarbons.

The inner diameter of the casing pipe, 200 mm, permits correct sampling procedures to be followed, and when necessary, can be employed in clean-up pumping.

The third stage of investigations comprises short-term monitoring for the purpose of determining the values of natural background concentrations of hydrocarbons to establish optimum intervals for groundwater table measurements, for measuring selected physico-chemical parameters and for taking water samples.

Short-term spills are dangerous owing to their unpredictability, and because they generally endanger previously unpolluted groundwater.

The first stage is concerned with determining the basic hydrogeological and geotechnical parameters of the aquifer and of the overlying stratum, in particular the direction and rate of flow. The horizontal and vertical extend of pollution by hydrocarbons as a separate phase, and in solution, is determined at the same time. Further spreading of contamination is assessed. The hydrogeologist usually cooperates closely in the proposal for the immediate clean-up activity, which is carried by the fire brigade or by technical squads provided by the customer. The second stage of the investigation is concerned with constructing a system for restricting the spread of, and then eliminating the pollution, if this has already not been cleaned up in the first stage.

The clean up operations and measures are carried out in close cooperation with the organization responsible for the spill. An effective method for rendering the aftereffects of spills less dangerous is a rapid removal of the polluted soil, pumping off the product from temporary sumps, trenches, etc. In the second stage, a network of hydrogeological wells is bored; to verify the hydraulic and hydrochemical properties of the polluted aquifer, and for used in the clean-up operations.

Hydrochemical investigations determine the quantitative and qualitative pollution of groundwater. Representative water samples are a necessary precondition for reliable analyses, but great difficulties may be encountered. As a result of the diverse specific gravity of hydrocarbons, they tend to form stratified layers in the aquifer. At higher concentrations, a zone comprising layers of immiscible hydrocarbon, emulsion and solution is created at the groundwater table, so that the taking of a homogeneous sample is virtually impossible. Boreholes containing a layer of oil product are not sampled for analytical purposes, unless auxiliary sampling tube wells are available.

Very low concentrations of hydrocarbons require not only a high degree of purity of the sampling equipment, but also inert materials for all the components coming into contact with water, so as to rule out sorption of hydrocarbons, or vice versa, elution of organic compounds. Oxidation, which may take place during sampling, will distort the analytical results, because of the more extensive biodegradation taking place in the sample bottle. There have so far been no sampler on the world market, which would meet all the requirements. The most reliable samples are those taken in the course of pumping, either directly by the pump or by means of an auxiliary sampling device. Among the available instruments, it is possible to recommend, with certain reservations, the cylindrical type Friedinger sampler, although these have the disadvantage of plastic components, and bottle vacuum type samplers, which suffer from tedious handling procedures.

For mapping purposes, it is sufficient to follow the surface layer, and this requires simpler equipment: cylinders of a stainless metal with a clean cylinder being available for each sample. In some cases, the samples may be taken directly into the transport bottles, thus eliminating the necessity of pouring the water and cleaning the equipment.

The sample bottle should have a perfectly sealing stopper and in principle should be filled without creating any head space. The samples should be dispatched to the laboratory as quickly as possible, preferably in a refrigerated box. Aluminium sulphate or preferably an extracting agent, such as tetrachloromethane or Freon, is sometimes added to the sample directly after recovery, to reduce the loss of hydrocarbons due to biodegradation and diffusion to a minimum.

The choice of the analytical laboratory method depends on the composition of the infiltrated product and on the age of pollution. Ultraviolet spectrophotometry, which is sensitive to aromatic hydrocarbons, is used for spills of light distillation fractions, in particular petrols, and for all fresh spills of oil products ranging from petrol to light fuel oils. Infrared spectrophotometry, which is more sensitive to alkanes, is used in all the other instances. The detection limit of the two methods is given at 0.01–0.05 mg l^{-1}.

When it is necessary to identify pollution caused by several possible sources use has to be made of the more complex methods of gas chromatography or mass spectrometry. These methods are more sensitive (0.001 mg l^{-1}), and selective, but also much more expensive than the analysis in the UV or IR spectrum. The economic factor should be considered when ordering the analyses; a combination of the two analytical methods is generally chosen to take account of the local situation.

The methods of measuring in field are less sensitive by one order of magnitude than the laboratory methods. One of the few available field analyzers, Horiba OCMA 200, has a detection limit of 1.0 mg l^{-1} and is not sensitive to aromates. The field analysis has the advantage of being rapid, which is valuable in particular in the investigation of fresh spills, and in the absence of errors due to transport and storage of samples.

A promising indirect field method is based on determining the presence of hydrocarbons in groundwater by measuring ORP and the content of dissolved oxygen. These parameters monitored in a chalk aquifer to the east of Prague have predicted the arrival of an oil plume escaped from a storage facility for petrol and diesel oil

(Švoma, Houzim, 1981). The change in the oxidative medium was monitored in 8 boreholes. The results from one of those are listed in Table 4.9 (the decrease of the $_rH$ value indicated transition into the reductive region).

Table 4.9.

MONITORING PHYSICAL-CHEMICAL PROPERTIES OF GROUNDWATER

Borehole No. 57	3.8.78	6.10.78	20.12.78	22.2.79
$_rH$	22.9	22.4	17.18	–
O_2	0.95	1.0	0.6	–
Hydrocarbons	0.06 mg/l	0.74 mg/l	oil film	oil layer

$_rH = \frac{Eh}{0.029} + 2\,pH$ Eh – measured potential (mV)

When a layer of free phase hydrocarbons is present at the groundwater table, its thickness is measured by means of mechanical and electronic gauges.

Spills from underground storage facilities and pipelines only rarely show any perceptible signs on the ground level. There is an exception of accidental rupturing of high-pressure pipelines, which have catastrophic effects on the natural environment. However, the quality of groundwater is much more endangered by "smaller" hidden leaks, which have been going on for long periods of time. The huge amounts that can escape within a layer from a hole 2 mm in size at a pressure of 0.6 MPa can be easily calculated.

It is obviously desirable to detect hidden leaks as soon as possible. In addition to observation wells serving for prevention, the occurrence and lateral spread of oil contamination may be determined by means of test pits. This method is usually demanding with respect to costs and time, and in addition also questionable in fissured rocks. For this reason alternative methods for the detection of hydrocarbons are sought, which would supplement or even replace investigation boreholes.

The oil product accumulated in rocks can be reliably detected by means of *gasometry*. The low-boiling oil products evaporate even at temperatures prevailing in the subsurface. (At 10 °C the vapour pressure of petrol and aviation kerosene amounts to 140 mm Hg and 1.3 mm respectively). With a shallow groundwater table, gaseous hydrocarbons easily migrate above the ground level, where they can be determined by means of a simple suction device using glass detection tubes. The concentration of hydrocarbon vapours is assessed according to the intensity of the colouring of the reactive filling. Reproducible results are provided by portable analysers, e.g. Portafid, which work on the principle of flame ionization detection FID , having the range from 1 ppm up to 6.0 vol. % CH, and which are selective with respect to oil hydrocarbons. The equipment is provided with a suction device, so that in addition to ground level detection, it may also be used for gasometric measurements in shallow boreholes, prepared by a light mobile rig.

GEOBOTANICAL METHODS

When the roots of the vegetation cover reaches into the zone contaminated with oil hydrocarbons, changes evoked in the plants can be utilized for detecting the pollution (Pyšek, Švoma 78). The effect of oil hydrocarbons on plants is complex, and depends on species, concentration, and time of contact: it may be inhibiting, teratological, toxic or stimulative. The presence of contaminants may be revealed by a whole scale of indications: the number and density of species, growth anomalies, a shift in phenophases, and the general health of the plants.

Detection of pollution by means of infrared films makes use of the decrease in the reflectance of most plants in the infrared region (700–100 mm) when their general health has deteriorated. The colour infrared film has one layer sensitive to infrared light and one or two coloured layers sensitive to the visible region, usually to the red and green part of the spectrum.

The wavelengths, on which the changes in reflectance of plants undergo the most distinct changes, differ with plants species. The intensity of the decrease also depends on numerous physical and physiological circumstances, so that interpretation of the picture is sometimes not explicit.

Better differentiation and identification is given by multispectral photography. This method uses simultaneous exposure of the vegetation by several cameras using black-and-white panchromatic and infrachromatic material. The optimum effect is attained when using a suitable filter-film combination. Compared to the previous method, the latter is more demanding with respect to the exposure proper and in particular with that to picture interpretation. It is also possible to monitor pollution of groundwater by a free oil phase after its uptake by plants. The photographic methods, and in particular aerial photography has the advantage in its documentation character, in the possibility of monitoring the development of contamination at preselected time intervals, and the great areal coverage.

However, there is no method for direct photodetection of pollution, when this is not manifested in the vegetative cover or in drains. The only existing non-destructive method for the determination of an oil component underground is provided by *geophysics*. The best results in the determination of the areal extent of an oil body are provided by electrical methods, which in sands show a theoretical limit of 10 m in depth and 10^{-2}–10^{-1} m of the hydrocarbon layer thickness. A promising method for the investigation of shallow oil contamination is the use of radar which, however, has so far remained in the stage of research in contrast to the geoelectrical methods.

THE CLEAN-UP ACTIVITIES

The clean-up activities may be undertaken in all the types of protection.

a) to remove the spillt contaminants,
b) to prevent further spreading of harmful substrates, which have already penetrated to the groundwater table and are propagating in the direction of the hydraulic and concentration gradients.

In the infiltration of contaminants from the ground surface a positive role is played by the sorptive ability of soil and of the weathered bedrock. Further contamination can be prevented by rapid excavation of the polluted soil. When the polluting substance has penetrated deeply into the rock formation or to the groundwater table, it has to be removed by pumping together with the contaminated water. Problems arise with suitable and safe dumping sites for the contaminated soil.

The contaminant can be prevented from spreading with the groundwater flow by constructing a suitable vertical obstacle in its path, or by interrupting the continuity between the pollution source and its environment by creating a hydraulic depression. The vertical impermeable curtain can be situated perpendicular to the direction of flow, and in that case the groundwater table rises and has to be pumped at suitable points.

When the obstacle has been placed obliquelly to the direction of flow and when the natural hydraulic gradient is adequately high, the contaminated water will change its course. The complete underground wall is anchored in an impermeable bedrock and crosses the groundwater flow throughout the aquifer thickness. An incomplete wall is terminated in the aquifer, usually only several metres below the groundwater table, and can thus only prevent the spreading of hydrocarbons flowing on the groundwater table. The applicability of underground walls depends on geological conditions: the degree of consolidation of rocks, the aquifer thickness and the water table depth. The walls can be best established in alluvial valley sandy gravels. The impermeable underground wall is created by filling a vertical groove with a sealing material, either clay or clay-cement concrete. The excavation and filling of the groove is carried out by sections 2–8 m in length. The odd sections are excavated first, and when they are filled, the even sections are finished. The wall is usually 50–60 cm in thickness. The impermeability of the wall has to be given special attention throughout its length, because failure of a single segment may endanger the protective effect of the entire isolation curtain.

An effect similar to that of underground walls is obtained by grout sealing. The material used in grouts are clay-cement mixes, or special organic compounds resistant to oil hydrocarbons. The grout curtains are used for final sealing of underground walls, for replacing sections of the wall where this cannot be constructed because of some obstacle such as a rock formation, building or power lines. Grouting is also used for the sealing of fracture-permeable formations, and in particular when eliminating spills of oil products into underground structures.

Hydraulic infiltration curtains perform a function similar to that of impermeable walls. Artificial elevation is produced by continuously supplying water by means of infiltration trenches, reservoirs or wells, so that the groundwater flow is changed. The method has the advantage of being readily operative, and for this reason is used in the cleaning-up of some large emergency spills. It has the drawback of being relatively inefficient in less permeable and inhomogeneous rocks, and of rapid ageing resulting from clogging of the infiltration objects. For this reason the infiltration curtain should be used only in exceptional cases.

The universal, most operative and most widely employed protective method is that of clean-up pumping. A protective hydraulic depression is formed by continuous pumping of water from a system of wells, or possibly dug pits, or drainage trenches.

A significant part in the choice of methods for groundwater protection is played by economic aspects and by the time necessary for performing the clean-up operations. The initial costs of solid impermeable curtains are high, but their operation is not technically demanding nor expensive. The hydraulic protective systems exhibit quite opposite aspects.

The rehabilitation measures are carried out either in immediate proximity to the location of spill, when it is called point or line protection, or the protection is solved from the regional points of view and is therefore situated at a greater distance from the source of pollution. This is the so-called area protection, which may also be used as a second zone supplementing another protective system.

The boreholes in the individual lines are situated so that their depression cones overlap by 30–50 %. In this way the protective effect of pumping is ensured for the time required for maintenance or replacement of the pumping or measuring equipment. The well materials should resist corrosion; use is made of stoneware, microceramics, polypropylene, etc. When using steel casing pipes, the application of protective paint coats is undesirable, unless they are capable of resisting the corrosive effects of oil hydrocarbons.

When designing clean-up boreholes, one should always take into account the possibility that the free oil phase may float on the groundwater table. The perforated casing pipe reaches about 1 m above the maximum established water table level when the protective line is composed of single separate boreholes either for reasons of economy or because of the small quantity of oil involved. In that instance, two pumps are usually installed in one borehole. The stronger pump is placed deep below the dynamic table, and serves to create the depression. The weaker pump, often provided with a separating strainer serving for the separation of the free phase, removes relatively small amounts of strongly contaminated water or of the oil product proper. When great amounts of water are expected to be pumped out pairs of boreholes are made. The deeper borehole is equipped for pumping the basal part of the aquifer and creates the depression. The other borehole, hydraulically incomplete, reaches only a few metres below the depressed table and serves for removing the free hydrocarbons and strongly contaminated water.

The method of pumping described above has several advantages:

a) continuous removal of oil substances from the groundwater surface suppresses the possible formation of emulsions and solutions, which could pass into the deeper part of the well by transverse flow and diffusion;
b) the method reduces considerably the amount of strongly contaminated water, which requires difficult and expensive treatment before discharging at the surface;
c) the relatively pure water from the deep well can be utilized e.g. for irrigation to speed up the decontamination process, as well as for other pur-

poses such as cooling and other industrial uses. In extreme cases the water may be used for public supply systems.

Whereas the drilling of wells is carried out in essentially the same way for hydrogeological surveys and in hydrogeological investigations and clean-up operations following oil spills, investigation-clean-up pumping presents certain complications. The corrosive effects of hydrocarbons tend to attack, and rapidly damage rubber cables and insulations, while even plastic coatings do not provide reliable long-term protection. Explosion-proof types of pumps should be generally used in the pumping of oil-containing water.

In contrast to hydrogeological surveys aimed at finding suitable water resources, when the boreholes are situated in the most permeable parts of the hydrogeological structure, this is usually not the case in clean-up operations. When complying with the principle of situating the first-line boreholes in close proximity to the point of spill, it is difficult to take into account the permeability of rocks. The well yield must often be artificially raised, for instance by evacuation. The suction piping of a vacuum pump and vacuum gauges are fitted in the hermetically sealed terminal of a specially equipped well. The discharge main from the pump is likewise fitted in the terminal in an airtight manner. In a locality composed of low-permeable kaolinized arkose sandstones an underpressure of 51–53 kPa has increased the yield four-times the original value. The content of free hydrocarbons in the pumped water increased simultaneously with increasing underpressure. Another method for increasing the yield is artificial infiltration from sprinklers, or by pumping in water through injection boreholes. The recharge of water within the range of the depression cone is advantageous in particular in dry seasons, when it partially eliminates the unfavourable side effect of protective pumping on the watersupply situation of the adjacent region. The protective systems based on collector trenches are restricted to unconsolidated sediments and weathering products. The success of application depends on the depth of the groundwater table, and such systems are very effective and virtually unrivalled in shallow aquifers of low permeability. Also in hilly, badly accessible countrysides, drainage ditches are often the only possible solution for rehabilitation measures.

The strongly contaminated water removed from all types of hydraulic protection cannot be discharged into sewers, nor into rivers or lakes. In the case of contamination with heavy oils, the water need only be settled in gravity separators. These are quite satisfactory for this purpose, of simple design and servicing. Even the most perfect separators, however, cannot remove dissolved hydrocarbons from water, and most of the emulsified hydrocarbons cannot likewise be removed. In the case of contamination with Diesel oil, aviation kerosene or petrol, the concentration of hydrocarbons at the exit from gravity separators may amount to tens mg per litre. The gravity separators have therefore to be supplemented with a further purification stage. Final treatment of water on vapex filters have been proved satisfactory in hydrogeological investigation practive (Czechoslovak Patent No. 177461/1979). The filters are cylindrical vessels, filled with hydrophobized expanded perlite-vapex. It has been proved by long-term tests, that a water purification plant consisting of two gravity separators of rectangular plan and two vapex filters is capable of purifying water contaminated with hundreds mg per litre of oil hydrocarbons to a maximum residual concentration of 5 mg/l. Water contaminated with petrol or benzene has generally to be additionally treated by aeration, mostly by means of simple sprinklers.

Experience shows that the natural purification of an aquifer contaminated with thousands cubic metres of oil substances takes many years. For this reason it is desirable to artificially accelerate the natural decontamination processes. Introduction of oxygen and nutrients into the aquifer stimulates the biodegradation, of the oil hydrocarbons by microorganisms. However, application of controlled biodegradation, particularly in deep aquifers, encounters a number of technical difficulties, which have so far prevented this potential method from being more widely employed.

ГЛОССАРИЙ ПО ОХРАНЕ ПОДЗЕМНЫХ ВОД

GLOSSARY OF GROUNDWATER PROTECTION

ВВЕДЕНИЕ

Настоящий глоссарий составлен в соответствии с решением редакционной коллегии по монографии „Гидрогеологические основы охраны подземных вод" и является дополнением к этой монографии. Заседание редакционной коллегии проходило в Москве 30 ноября — 2 декабря 1982 г. Глоссарий включает 35 наиболее распространенных терминов в области охраны подземных вод. Другие гидрогеологические термины и их определения приводятся в ранее опубликованном Международном глоссарии по гидрогеологии (International Glossary of Hydrogeology, Unesco, Paris, 1978). Трактовка терминов и определений дается на русском и английском языках.

Настоящий вариант глоссария подготовлен в институте ВСЕГИНГЕО в декабре 1983 г. — январе 1984 г. Л.Г. Лукьянчиковой под руководством профессора В.М. Гольдберга (ответственный редактор русского текста монографии). Перевод на английский язык выполнен Л.В. Куренной.

INTRODUCTION

This glossary is made up in accordance with the decision of an editorial board of the monography "Hydrogeological Base of Groundwater Protection", and is a supplement to this monography. The meeting of the editorial board was held in Moscow on 30 November — 2 December 1982. The glossary contains 35 concepts most frequent in the sphere of groundwater protection. Other hydrogeological terms and their definitions are given in a previous issue of the International Glossary of Hydrogeology (UNESCO, Paris, 1978). The concepts and definitions are presented in Russian and English.

This version of the glossary was prepared in VSEGINGEO (All-Union Research Institute of Hydrogeology and Engineering Geology) by Luk'yanchikova L.G. under the supervision of Professor Goldberg V.M. (chief editor of the Russian version of the monography) in December 1983 — January 1984. English translation is made by Kurennaya L.V.

Б

1.
Бассейн сточных вод.
Waste water basin.
Специально оборудованные инженерно-технические сооружения и естественные бассейны, в которые отводятся сточные воды на постоянное или временное хранение, а также для последующей их очистки и обезвреживания.
Син.: хранилища сточных вод; шламохранилища; хвостохранилища; пруды-накопители; пруды-испарители и пр.
Special engineering structures and natural basins for a permanent or temporary storage of waste water, as well as for its further purification.
Syn.: waste water storage; tailings storage; storage ponds; evaporation ponds, etc.

В

2.
Водозабор инфильтрационный.
Infiltration water intake.
Инженерно-техническое сооружение (скважины, колодцы, подземные водосборные галереи), используемое для забора подземных вод и привлекаемых из рек и водоемов инфильтрующихся поверхностных вод.
В.и. располагается вдоль берега реки (водоема).
Син.: водозабор береговой; водозабор в речных долинах.
Engineering structure (well, borehole, underground gallery, adit) used for extraction of groundwater and surface water percolating from streams and water bodies.
I.w.i. locates along the river bank or a water body shore.
Syn.: longshore water intake; water intake in river valleys.

3.
Водоохранные мероприятия.
Water protection.
Комплекс мероприятий, направленных на предотвращение загрязнения и истощения подземных вод.
Син.: защитные мероприятия.
Complex of measures intended to prevent from groundwater pollution and depletion.
Syn.: protection.

З

4.
Загрязнение подземных вод.
Groundwater pollution.

Обусловленные антропогенной деятельностью изменения качества воды (физических, химических, биологических свойств) по сравнению с ее естественным состоянием и предельно допустимыми нормами качества.
Man-made changes in water quality (physical, chemical and biological properties) as compared to a natural water state and maximum permissible quality standards.

5.
Загрязнение химическое.
Chemical pollution.
Изменение физико-химических свойств подземных вод в результате вызванного деятельностью человека поступления в водоносный горизонт химических веществ, естественных или антропогенных.
Changes in physical and chemical properties of groundwater as a result of a man-made aquifer contamination by chemical matters either natural or artificial.

6.
Загрязнение тепловое.
Thermal pollution.
Изменение физико-химических свойств подземных вод вследствие повышения, а также понижения их температуры, обусловленных антропогенной деятельностью.
Changes in physical and chemical properties of groundwater due to an increase, as well as a decrease in water temperature attributed to human activity.

7.
Загрязнение (подземных вод) бактериальное (микробное).
Bacterial (microbial) (groundwater) pollution.
Загрязнение вод в результате поступления в них патогенных и санитарно-показательных микроорганизмов.
Water contamination by pathogenic and sanitary-indicative microorganisms.

8.
Загрязняющее воду вещество.
Water pollutant.
Вещество в воде, вызывающее нарушение естественного состояния подземных вод и норм качества воды.
A matter which exists in water and disturbs the natural state of groundwater and hinders water quality standards.

9.
Защищенность подземных вод (естественная).
(Natural) groundwater protection.
Совокупность гидрогеологических условий, обеспечивающих предотвращение проникновения загрязняющих веществ в водоносные горизонты.
К гидрогеологическим условиям относятся: глубина залегания подземных вод, мощность и литология зоны аэрации, наличие водоупоров, изолирующих водоносный горизонт и пр.
Combination of hydrogeological factors that prevent from aquifer contamination by pollutants. These hydrogeological conditions are as follows: depth of groundwater table, thickness and lithology of an aeration zone, presence of aquifers which isolate an aquifer, etc.

10.
Зона санитарной охраны (водоисточников).
Sanitary zone of (water source) protection.
Территория, в пределах которой устанавливаются санитарные ограничения и проводятся мероприятия, обеспечивающие защиту водоисточников от загрязнения.
An area subject to particular sanitary limitations and measures that provide water source protection against pollution.

11.
Зона аэрации.
Unsaturated zone.
Часть литосферы, располагающаяся между земной поверхностью и зеркалом грунтовых вод.
A part of lithosphere between the land surface and groundwater table.

12.
Интенсивность загрязнения.
Level of pollution.
Превышение содержания загрязняющих воду веществ по сравнению с естественным состоянием подземных вод и предельно допустимыми нормами.
An excess of water pollutant concentration as compared to the natural state of groundwater and maximum permissible standards.

13.
Интрузия соленых вод.
Salt water intrusion.
Поступление соленых вод, поверхностных или подземных, в водоносный горизонт, содержащий пресные подземные воды.
Син.: внедрение соленых вод; подсос соленых вод.
An inflow of salt (surface- or ground) water into an aquifer containing fresh groundwater.
Syn.: salt water encroachment; salt water invasion during fresh groundwater pumpage (upconing).

14.
Инфильтрационные бассейны.
Infiltration basins.
Инженерные сооружения площадного типа, предназначенные для искусственного пополнения запасов пресных подземных вод путем инфильтрации в водоносные горизонты поверхностных вод, заполняющих данные сооружения (бассейны, каналы).
Area-type engineering structures for an artificial recharge of fresh groundwater storage through an infiltration of surface water filling these structures (basins, canals) into aquifers.

15.
Искусственное пополнение запасов подземных вод.
Artificial recharge of groundwater storage.
Увеличение или создание новых запасов подземных вод путем направления части поверхностных вод в водоносные горизонты через скважины, бассейны, траншеи и т.д.
An increase in or development of new groundwater storage through an induced flow of surface water into aquifers or by direct recharge via basins, ditches or boreholes, etc.

16.
Исследования опытно-миграционные.
Migration tests.
Натурные (полевые) эксперименты, в процессе которых проводится индикация подземных вод с целью определения миграционных параметров.
Син.: опытно-миграционные работы.
In-situ (field) experiments accompanied by groundwater indicators aimed at determination of migration parameters.

17.
Источник загрязнения (подземных вод).
Source of (groundwater) pollution.
Источник, вносящий в поверхностные или подземные воды загрязняющие воду химические вещества, микроорганизмы, тепло.
A source which inserts pollutants, i.e. chemical matters, microorganisms, heat, into surface- or groundwater.

18.
Истощение (подземных вод).
(Groundwater) depletion.
Уменьшение запасов подземных вод вследствие их отбора в количествах, превышающих питание (пополнение) подземных вод.
Decrease in groundwater storage as a result of groundwater withdrawal which is in excess of the groundwater recharge.

К

19.
Качество воды.
Water quality.
Характеристика состава и свойств воды, определяющая пригодность ее для конкретных видов водопользования.
Description of water composition and properties that determines availability of water for particular use.

20.
Контроль за охраной подземных вод.
Groundwater control.
Система мер, направленных на получение систематической информации о загрязнении и истощении подземных вод, их прогнозирование и обеспечение соблюдения всеми организациями установленного порядка охраны подземных вод от загрязнения и истощения.
A system of measures intended to obtain regular information on groundwater pollution and depletion, as well as to predict these events, and to provide the observance of the established order in groundwater protection against pollution and depletion by all consumers.

М

21.
Масштабы загрязения.
Scales of pollution.
Количественные показатели, характеризующие размеры области загрязнения и интенсивность загрязнения.
Quantitative characteristics indicating the sizes of a polluted area and the rate of contamination.

22.
Миграционные параметры.
Migration parameters.
Количественные показатели, характеризующие условия и процессы массопереноса в водонасыщенной пористой среде.
Quantitative characteristics governing the conditions and processes of mass transfer in saturated porous medium.

23.
Миграция вещества в подземных водах.
Solute mass transport.
Процесс перемещения растворимого вещества в водоносном горизонте в условиях его физико-химического взаимодействия с подземными водой и породами.
The transfer of dissolved materials in an aquifer under physical-chemical interaction between groundwater and rocks.

Н

24.
Нефтяное насыщение.
Oil saturation.
Насыщенность порового объема нефтью как компонента двухфазной системы „нефть—вода".
Pore volume saturation with oil as a component of the two-phase "oil—water" system.

25.
Нормы качества воды.
Water quality standards.
Установленные в соответствии с национальными или международными нормами значения показателей качества воды по видам водопользования.
Син.: предельно допустимые нормы; предельно допустимые концентрации (ПДК).
Water quality characteristics established by national or international agreement for the different types of water consumption.
Syn.: maximum permissible rates; maximum permissible concentration (MPC); maximum acceptable concentration (MAC).

О

26.
Область загрязнения подземных вод.
Area of groundwater pollution.
Часть водоносного горизонта, содержащая подземные воды, физические и химические свойства которых под влиянием антропогенного воздействия отличаются от естественных.
Portion of an aquifer containing groundwater with physical and chemical properties altered due to a man's impact.

27.
Охрана подземных вод.
Groundwater conservation.
Система мер, направленных на предотвращение и устранение загрязнения и истощения вод.
A system of measures to be taken for prevention from and liquidation of water pollution and depletion.

С

28.
Санитарные показатели (качества подземных вод).
Sanitary characteristics of groundwater quality.
Лимитирующие показатели качества воды, определяющие пригодность ее для питьевых и бальнеологических целей (С.п. качества подземных вод частный вид „Норм качества воды").
Water quality standards defining water adequacy for drinking and balneological purposes (S.c. of groundwater quality as a particular type of "Water quality standards").

29.
Сброс сточных вод.
Sewage effluent disposal.

Отведение сточных вод в бассейны сточных вод, на поля фильтрации, на сельскохозяйственные поля орошения и в природные водотоки.
Waste water disposal into sewage basins, filtration fields, agricultural irrigated lands and natural water courses.

30.
Состояние подземных вод.
Groundwater state.
Характеристика подземных вод по совокупности их количественных и качественных показателей применительно к видам водопользования.
Groundwater description by using combination of groundwater quantitative and qualitative characteristics toward the types of water consumption.

31.
Специализированная наблюдательная сеть.
Specialized observation network.
Специально заложенная в зоне влияния источника загрязнения и водозабора сеть наблюдательных скважин для контроля за состоянием подземных вод.
A network of observation wells specially implemented at the influence zone of a pollution source and water intake to control the groundwater state.

32.
Сточные воды.
Waste water.
Воды, отводимые после использования в бытовой и производственной деятельности человека.
Син.: промышленные сточные воды; жидкие промышленные отходы; бытовые сточные воды.
Water disposed after its usage in domestic and industrial fields.
Syn.: Industrial sewage; liquid industrial wastes; domestic sewage.

Т

33.
Техногенные факторы.
Technogenic factors.
Факторы, обусловленные деятельностью человека.
Син.: антропогенные факторы.
Factors attributed to human activity.
Syn.: Man-made factors; anthropogenic factors.

Ф

34.
Фазовая проницаемость.
Phase permeability.
Проницаемость горной породы относительно одного из нескольких (двух, трех) одновременно содержащихся в ней флюидов (текучих фаз).
Rock permeability relative to one of several (two or three) fluids (liquid phases) simultaneously present in this rock.

Я

35.
Язык соленой воды.
Salt water tongue.
Краевая часть соленых (природных) вод или загрязненных сточных вод, интрудировавших в водоносный горизонт с пресной водой, имеющая в вертикальном разрезе вид клина или языка.
Marginal portion of salt (natural) water or polluted wastes intruded into a fresh aquifer and presenting in the form of a tongue or wedge in a vertical section.

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Внешторгиздат. Изд. № Е-2405
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