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Classification of mathematical models of unsteady water movement in the main canals of irrigation systems

A.V. Kabulov, A.J. Seytov, A.A. Kudaybergenov

Professor, Department of “Information Security;,National University of Uzbekistan Tashkent, Uzbekistan
PhD, Scientific Research Institute of Irrigation and Water Problems, Tashkent, Uzbekistan
Senior teacher of department “Applied mathematics” of the Karakalpak State University. Nukus, Uzbekistan

ABSTRACT: The development of computerization around the world, and in recent years in the Central Asian states, makes it possible to use it for modelling and managing water resources in all water management systems and facilities, which include river sections, reservoirs and canals (gravity and with machine water lifting systems). Using a computer, you can simulate the behaviour of individual water facilities, their interaction, taking into account all the influencing factors in conditions close to real, in the process of their functioning. Therefore, this article provides an analytical review of modern studies of mathematical models of unsteady water movement in the main canals of irrigation systems, as well as their advantages and disadvantages.

KEY WORDS: mathematical model, unsteady flow of water, main canals, optimal control problems, fundamental solution, differential equations, hydraulic structures.

I.INTRODUCTION

Water management systems and facilities have a large spatial extent, a great number of technological and technical parameters, only quantitative and qualitative changes in their characteristics can be obtained using mathematical modelling methods.

Currently, there is no unified systematic approach to modelling the dynamics of water facilities, only there is a wide class of mathematical models of individual objects of varying degrees of complexity, therefore, the choice of mathematical models that will describe the required processes of water supply and water distribution in water systems and facilities with the required degree of accuracy, is a very problematic task.

Analysing the main models used to solve the problems of modelling the control of the canals of irrigation systems, they can be divided into three groups (Fig. 1): static models, dynamic models with lumped parameters, and dynamic models with distributed parameters. These models differ among themselves in the initial premises, the amount and type of primary information and the degree of detail of the results obtained.[1,2,3,4]

Static models use various algebraic dependencies between seeking and known parameters. These models include various modifications of regression models, models of hydrological series, etc. Parameters for these models are determined on the basis of observational data and full-scale experiments, using identification methods (least square methods, maximum likelihood methods, etc.).[5]

The advantages of static models are their simplicity, a small amount of initial information, the possibility of using computers, and the speed of obtaining information. With their help, it is possible, with a certain probability, to establish the characteristics of the watercourse in the sections where some observations were made.

The disadvantages of static models are the limited possibilities of applying these models to designed objects. Models provide little information about the process, are of little use outside the limits and ranges of observations.

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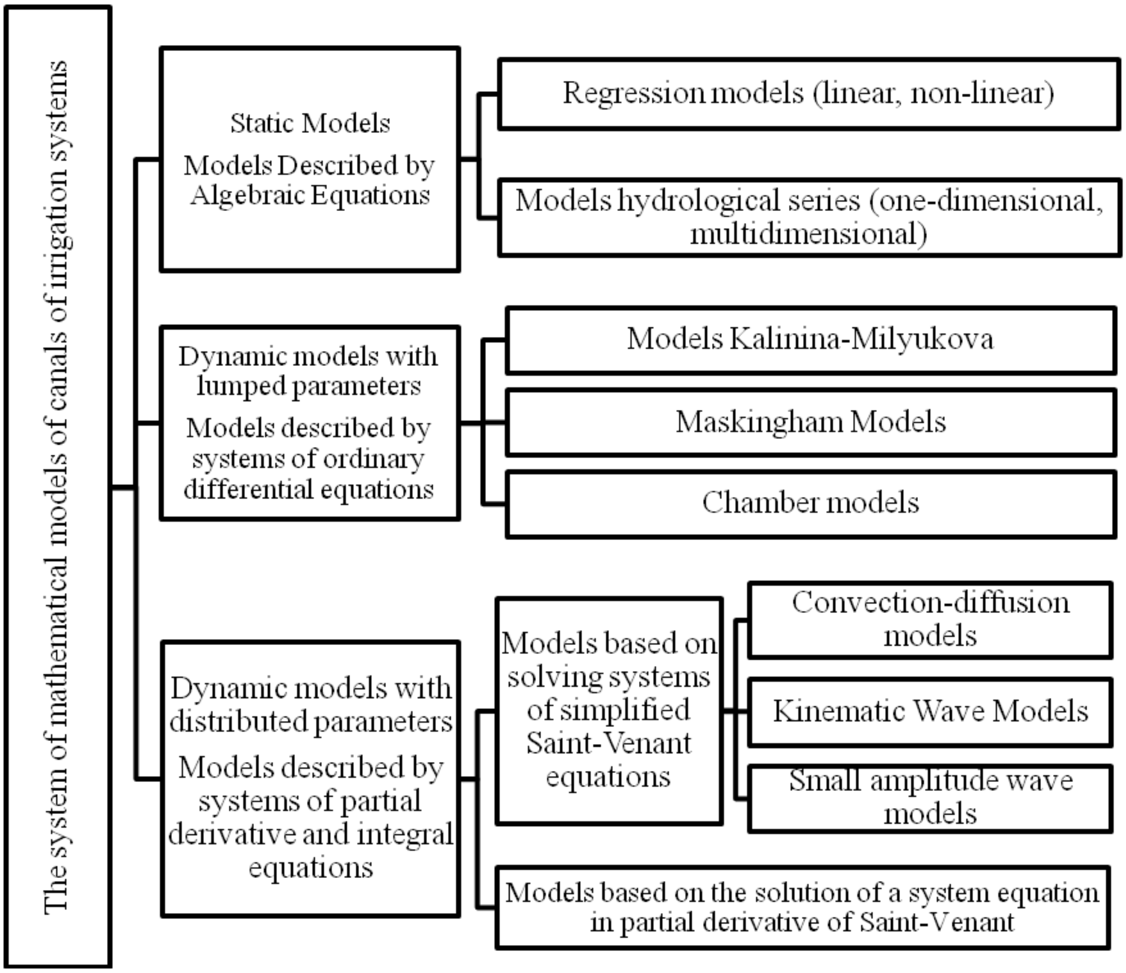

Fig.1. The system of mathematical models of canals of irrigation systems

II. METHODS AND RESULTS

Dynamic models with lumped parameters. Models with lumped parameters are based on the assumption that the length of the considered section is small compared to the wavelength. In these models, the size of the site is neglected and it is believed that its characteristics are concentrated at one point. Using these models, the characteristics of the water flow are calculated only in individual sections of the site. Models with lumped parameters also include hydrological models, such as linear and nonlinear reservoir models, Kalinin-Milyukov models, Maskingham model, etc. [1, 2, 3, 4].

The essence of the Kalinin-Milyukov model is the assumption that reach of riverbed has a length L (characteristic site) has the same regulatory effect on the release wave, as well as the reservoir equal in volume to it. In other words, with unsteady water movement, the uniqueness of the dependence of the volume of water in the area on the flow rate Q in its closing range: $W = f(Q_y)$

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The length of the characteristic profile is determined through the parameters of steady motion - flow rate Q_y, slope $-i_y$ and the ordinate of the free surface Z_y

$$L = \frac{Q_y}{i_y}\frac{dZ_y}{dQ_y} \tag{1}$$

At the same time, it is assumed that the site can be divided into n characteristic sites in which there is an unambiguous relationship between the volume of water and the flow rate in the closing sections. Additionally, assuming that this relationship is linear, the calculation of the hydrograph can be reduced to solving a system of equations

$$\left.\begin{array}{l} \frac{dQ_1}{dt} = \frac{1}{\sigma_1}(q - Q_1), \\ \frac{dQ_2}{dt} = \frac{1}{\sigma_2}(q - Q_2), \\ \dots\dots\dots\dots\dots\dots \\ \frac{dQ_n}{dt} = \frac{1}{\sigma_n}(q - Q_n), \end{array}\right\} \tag{2}$$

where q - inlet flow rate; Q_i - flow rate in the closing range i-th characteristic site; τ_i - coefficient of proportionality between the volume of water and the flow rate in the closing range i-th section (run-time on a characteristic section). This system can be solved by any operator method.

As a result of solving this system, the dependence of water flow on time and riverbed parameters is obtained, which can be used for the calculation of control actions.
The Maskingham model is based on the solution of the water balance equation of the following form

$$\Delta t\frac{\left(W_{н}^{t} - W_{к}^{t}\right)}{2} + \Delta t\frac{\left(W_{н}^{t+1} - W_{к}^{t+1}\right)}{2} = \\ = K\left[\lambda\left(W_{н}^{t+1} - W_{н}^{t}\right) + (1-\lambda)\left(W_{к}^{t+1} - W_{к}^{t}\right)\right], \tag{3}$$

where $W_н^t$ and $W_н^{t+1}$ - volume of water in the initial section at a time t and $t+1$;
Δt - estimated time interval between t and $t+1$;
K – coefficient depending on the hydraulic parameters of the site for prismatic riverbeds with a constant slope $K = \frac{Cl}{\sqrt{i}}$; l – the site length.

Solving equation (3) with respect to W_K^{t+1}, we get

$$W_K^{t+1} = \frac{(\Delta t/2 + K\lambda)W_H^t - [(\Delta t/2 - K(1-\lambda)]W_K^t + [\Delta t/2 - K\lambda]W_H^{t+1}}{\Delta t/2 + K(1-\lambda)} \tag{4}$$

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or

$$W_K^{t+1} = C_1 W_H^{t+1} + C_2 W_H^t + C_3 W_K^t. \quad (5)$$

Here

$$C_1 = \frac{\Delta t/2 - K\lambda}{\Delta t/2 + K(1-\lambda)}, C_2 = \frac{\Delta t/2 + K\lambda}{\Delta t/2 + K(1-\lambda)}, C_3 = -\frac{\Delta t/2 - K(1-\lambda)}{\Delta t/2 + K(1-\lambda)}.$$

Selection K and Δ - the main problem for this method. They can be determined from observational data using equations (4) - (5).

This method allows you to find the relationship between the flow rate and volume of water in the riverbed and can be used for management purposes. Modifications of this method are the method of Kunzh and Paps [5].
In [2, 5], models were used to calculate the unsteady movement of water in river sections. In these models, river sections are divided into a number of chambers and a balance differential equation is compiled for each of the chambers.

The advantage of the above dynamic models with lumped parameters is their simplicity, a small amount of initial information, relatively small amounts of calculations, deeper than that of stochastic models, penetration into the essence of the described process, a greater degree of detail of the calculation results, the possibility of calculation and application in real time.

The disadvantages of models with lumped parameters are the need for careful justification in each specific case of their application in connection with the use of assumptions that are not always obvious and do not follow from any more general laws.
Dynamic models with distributed parameters are mainly described by the Saint-Venant equation and take into account the distribution of flow motion parameters in river and canal sites along the length and time, which are based on the following assumptions:

- the flow is one-dimensional, i.e. the speed in the cross-section is the same and the water level in the transverse direction is horizontal;
- the curvature of the streamlines is small and the vertical accelerations are negligible, therefore, the pressure is hydrostatic;
- the effect of friction at the boundaries and turbulence of the flow can be taken into account by the laws of resistance, which are similar to those used in the steady-state flow of water;
- the average slope of the riverbed bottom is small and the cosine of the angle between the bottom line and the horizontal can be taken equal to unity.

We note that the types of flow under consideration are realized in riverbeds, the cross sections of which can be of any shape and vary along the axis, but this change is limited by the condition that the curvature of the flow lines is small.
The integral form of the Saint-Venant equations consists of mass conservation law [1,26, 27]:

$$\int_{x_1}^{x_2} \left(\omega_{t_2} - \omega_{t_1}\right) dx + \int_{t_1}^{t_2} \left(Q_{x_2} - Q_{x_1}\right) dt = 0 \quad (6)$$

and momentum conservation

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$$\int_{x_1}^{x_2}\left(Q_{t_2}-Q_{t_1}\right)dx=\int_{t_1}^{t_2}\left[\left(\upsilon^2\omega\right)_{x_1}-\left(\upsilon^2\omega\right)_{x_2}\right]dt+$$
$$+g\int_{t_1}^{t_2}\left(I_{x_1}^1+I_{x_2}^2\right)dt-g\int_{t_1}^{t_2}\int_{x_1}^{x_2}\left[\rho I^2+\omega\left(I_0-I_f\right)\right]dxdt, \tag{7}$$

where $Q=\upsilon\omega$;

$$I^1=\int_0^{h(x)}\left[h(x)-\eta\right]\sigma(x,\eta)d\eta$$
$$I^2=\int_0^{h(x)}\left[h(x)-\eta\right]\left[\frac{\partial\sigma}{\partial x}\right]_{h=h_0}d\eta;$$

$I^0=-\frac{\partial Z_q}{\partial x}$; I^f – friction slope; $\sigma(x,h)$- cross section width; moreover $\sigma(x,h)=\sigma(x)$ - top flow width; ω - living area; υ – uniform speed across the cross section; x – coordinate along the length of the site; t – time; h - water level; Z_q – bottom mark.

The differential form of the system of one-dimensional Saint-Venant equations has the form

$$i-\frac{\partial h}{\partial x}=\frac{v}{g}\frac{\partial v}{\partial x}+\frac{1}{g}\frac{\partial v}{\partial t}+\frac{|v|v}{C^2R}+\frac{qv}{g\omega},$$
$$\frac{\partial\omega}{\partial t}+\frac{\partial Q}{\partial x}=q; \tag{8}$$

where C – Shezi coefficient; R – hydraulic radius; q – lateral inflow.

The advantage of the above models is the use of a small number of generally accepted and repeatedly tested initial positions, a clear and strict mathematical formulation of the emerging problems.

A significant advantage of hydraulic models is their versatility. They are applicable both in the design and operation of sections of rivers and canals. Hydraulic models allow almost always to obtain the characteristics of processes with the required detail and acceptable error, to interpolate and extrapolate the characteristics over a wide range. Therefore, they, are widespread despite the high cost of creation. This primarily refers to one-dimensional mathematical models [6, 7, 8].

The disadvantages of hydraulic models are mainly associated with processes in riverbeds, where the emergence of the so-called non-transit zones-shrubbed or other sections of the river, where the water hardly moves. Non-transit zones play the role of storage capacities; therefore, such zones should not be taken into account in the live section of the flow. Methods for distinguishing transit zones have not yet been developed, as a result of which they are not taken into account in the commonly used one-dimensional equations of water motion. In canals, with proper maintenance, the appearance of non-transit zones is almost not observed, as a result of which the indicated disadvantages of hydraulic models are insignificant.

Thus, it is hydraulic models that are of the greatest interest for the study of dynamic processes in water facilities and systems.

The presented models can be classified according to the solution methods used. Existing methods for solving Saint-Venant equations are conditionally divided into three groups. The first includes solutions obtained as a result of attempts to find the general integral of the Saint-Venant equations using strict mathematical analysis when the method of differential characteristics is applied with the subsequent use of equations in finite differences.

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The second group consists of solutions found using mathematical analysis using the theory of small-amplitude waves.

The third group includes solutions obtained as a result of approximate integration of the Saint-Venant equations with preliminary replacement of them with equations in finite differences.

The sources of mathematical modeling of unsteady movement of water in open streams were the studies of Saint-Venant, S. A. Khristianovich [9], A. A. Arkhangelsky [6], J. Stoker and others.

The method of characteristics proposed by S.A. Khristianovich was developed by A.A. Arkhangelsky, Y.D. Guildenblatt and N.T. Meleschenko. The essence of this method is to replace the Saint-Venant system of equations with an equivalent system of four ordinary differential equations, which are two equations of characteristics (direct and inverse) and two equations connecting the flow elements along with these characteristics. A system of four equations is solved infinite differences and allows us to calculate the coordinates t and s of the nodes of the grid of characteristics in the wave plane. Further, the calculated parameters determine the values of the flow parameters: $Q(s,t)$, $z(s,t)$.

The method of characteristics makes it easy to obtain the boundaries of the region of existence of a given solution (a given wave), in particular, to trace the moment and target of a wave going into a continuous one. In addition, the equations in characteristic form are used in grid methods to calculate the elements of the mode in boundary sections.

The variational method was developed by N. A. Kartvelishvili [10] and is based on the application of the Bubnov-Galerkin method for calculating the unsteady movement of water. This method allows us to reduce the solution of the system of partial derivative equations (in this case, the system of Saint-Venant equations) to the solution at the beginning of the system of ordinary differential equations, and then the system of algebraic equations [11]. It turned out to be convenient when using modern computers for calculations.

A certain difficulty in this method is the selection of variation coefficients based on the careful processing of topographic and hydrometric materials.

The straight line method is considered as the limiting case of the grid method when, when applying a rectangular grid, one of its linear dimensions tends to zero, and the set of nodes in the limit fills a certain system of rectilinear parallel segments. Moreover, partial differential equations are reduced to a system of ordinary differential equations by replacing derivatives with respect to the corresponding unknown by difference relations [1].

The method of small-amplitude waves was developed by N.T. Meleschenko [1, 2]. The main assumption of this method is that all changes in the parameters of the mode are assumed to be small in comparison with their values at the initial steady state, therefore, the squares and products of these quantities are neglected. A good application of this method was proposed by E.E. Makovsky [12]; his analytical solution of the linearized Saint-Vanant equation is currently widely used in engineering calculations [13, 14].

In [15], an unconventional approach to the study of the kinematics of complex flows based on the Le Chatelier principle is developed. As applied to the problems of one-dimensional modeling, the approach used allows us to clarify the throughput capacity of river-basin and canal objects without using the hydrodynamics of turbulent flows. V.I. Koren and L.S. Kuchment conducted research on the creation of mathematical models of unsteady water movement [16, 17, 18, 19]. They consider a wide range of problems associated with the creation of mathematical models of river flow.

Many works are devoted to the mathematical formulation of boundary problems, the creation of numerical methods for solving using mainly explicit difference schemes, the development of methods for determining morphometric and hydraulic characteristics, their approximation, identification and determination by solving inverse problems. N.A. Kartvelishvili [10, 11] developed, as applied to one-dimensional flow diagrams, the fundamental problems of deriving one-dimensional equations from multidimensional equations of motion, which made it possible to elucidate the theoretical aspects of one-dimensional schematization of the process, to clarify the mathematical meaning of the adjustments to the momentum and energy, which are differently introduced into the equations of motion by many the authors considered the possibility of simplifying and schematizing watercourses.

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The main advantage of the explicit scheme is the relative simplicity of constructing a solution algorithm and software implementation. But it does not allow calculations at large calculated time steps, because in order for the circuit to be stable, a certain relationship must be maintained between the time step and the calculated length step — the so-called Courant-Friedrichs-Levy condition [20]

$$\Delta t = \frac{\Delta x}{|v| + \sqrt{gh}} \qquad (9)$$

This limits the use of explicit schemes.
As shown by S.K. Godunov [20], there is another limitation

$$\Delta t \le \frac{c}{g}\sqrt{\frac{R}{i}} = \frac{K}{g\omega\sqrt{i}} \qquad (10)$$

where v - flow rate; g - gravitational constant; h - ordinate of the free flow surface; c - Shezi coefficient; R - wetted riverbed perimeter; i - bottom slope; ω - living cross-sectional area; K – flow rate module.

For implicit schemes, restriction (11) disappears, and the condition of S.K. Godunov (12) can be avoided by taking the value of the flow modulus K from the upper layer of the wave surface. According to experts, the finite element method is the most effective method for solving hydraulic problems, but it has been little studied [21, 22].

The instant mode method was proposed by N.M. Vernadsky and improved (for manual calculation) by V. A. Arkhangelsky and Y. D. Gildenblat, I. V. Egiazarov [3, 6]. The algorithm and programs for computer calculations were developed by B. L. Historik [23]. This method is one of the applications of the implicit difference scheme for the grid method. Here, for each moment of time corresponding to the end of the estimated time interval, the instantaneous values of expenses and levels at the boundaries of the calculated sections are determined (“instantaneous modes”).

In the early 60s, under the guidance of Professor O.F. Vasilyev, a scientific school was formed in the IG SB RAS (A.A. Atavin, A.F. Voevodin, M.G. Gladyshev, S.M. Shugrin), the works of which belong to results in mathematical modeling of unsteady water movement [8, 24, 25]. Here, equations of motion were obtained in a more general form, mathematical formulations of problems on smoothly changing and discontinuous unsteady motion of water in watercourses and their systems are given. Numerical methods have been developed for solving one-dimensional problems in stationary and moving grids without and with a gap, methods for calculating and identifying many parameters of equations. Studies have been carried out to increase the degree of automation of mathematical modeling. Much attention is paid to the formulation, analytical and numerical methods for solving problems, including the movement of water along a dry riverbed. Methods for solving two-dimensional problems were developed and appropriate tools were developed.

Approximation of differential equations using implicit difference schemes on each time layer leads to the solution of a difference boundary value problem having essentially the same structure as the main boundary value problem. Algorithms for solving difference equations that take this structure into account are considered the most promising [7, 8]. When solving such problems, the main difference equations are used, which are linearized, then using one of the variants of the sweep method, unknown coefficients are determined through which the state parameters of the system under study are determined using balance relations.

A complete model of the transient flow of water in a site of a canal. The condition of the main canal site is characterized by an unsteady flow of water and is described by the system of differential equations of Saint-Venant in the form of energy conservation laws [27]

$$B\frac{\partial z}{\partial t} + \frac{\partial Q}{\partial x} = q, \qquad (11)$$

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$$\frac{1}{g\omega}\left(\frac{\partial Q}{\partial t}+2v\frac{\partial Q}{\partial x}\right)+\left[1-\left(\frac{v}{c}\right)^2\right]\frac{\partial z}{\partial x}=$$

$$=\left[i+\frac{1}{B}\left(\frac{\partial \omega}{\partial x}\right)_{h\,=\,const}\right]\left(\frac{v}{c}\right)^2-\frac{Q|Q|}{K^2},$$

where $v=\frac{Q}{\omega}$; $c=\sqrt{\frac{g\omega}{B}}$; $Q=Q(x,t)$ – water flow rate; $z=z(x,t)$ – ordinate of free surface; g – gravitational constant; i - bottom slope; $B{=}B(z)$ – flow width over a living section surface; $\omega=\omega(z)$– living cross-sectional area; $c=c(z)$ – propagation speed of small waves; $K=K(z)$– flow module.

Partial differential equations of the hyperbolic type in system (13) are the equations of conservation of mass and momentum of the flow, and are a mathematical model of the unsteady movement of water in the open canal.

As functions that determine the flow, the flow rate is selected here *Q(x,t)* and the ordinate of the free surface *z(x,t)*. Independent variables are the longitudinal coordinate *x* and time *t*.

The riverbed is set by the ordinate of the bottom $z_0(x)$ and cross-sectional width *B(x,t)*on distance z (vertical) from the bottom of the riverbed .
Then:

flow depth: $h(z,t)=z(x,t)-z_0(x,t)$; flow cross-sectional area: $\omega(x,t)=\int_0^h B(x,z)dz$; average speed of flow: $\upsilon=\frac{Q}{\omega}$;

small wave propagation speed: $c=\sqrt{g\omega/B}$; bottom slope $i=-dz_0/dx$.

The characteristic form of equations (13) has the form [7]

$$\frac{\partial Q}{\partial t}+(v\pm c)\frac{\partial Q}{\partial x}-B(v\mp c)\left[\frac{\partial z}{dt}+(v\pm c)\frac{\partial z}{\partial x}\right]=$$
$$=\left(\varphi-\frac{Q|Q|}{K^2}\right)g\omega-(v\mp c)q. \qquad (12)$$

Here $\varphi=\left[i+\frac{1}{B}\left(\frac{\partial\omega}{\partial x}\right)_{h\,=\,const}\right]\left(\frac{v}{c}\right)^2$.

The initial conditions are specified as

$$z(x,0)=z_0(x),\qquad Q(x,0)=Q_0(x), \qquad (13)$$

where $Q_0(x)$, $z_0(x)$ – known functions.

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Boundary conditions at points $x_1 = 0$ and $x_2 = l$ are written as follows

$$Q(0,t) = u_1(t), \quad Q(l,t) = u_2(t). \tag{14}$$

Water flows at the points of water intake of the canal site, the right side of equation (15) under the conditions of discreteness of water distribution, has the form

$$q(x,t) = -\sum_{i=1}^{5} q_i \delta(x - a_i) l(t - T) \tag{15}$$

In these models, an analytical solution of equations (13), (15) and (16) under the indicated boundary conditions is absent, since the hydraulic parameters of the water flow is a nonlinear function depending on the shape of the cross section of the canal site.

From the expression of the lateral water intakes (17) it is seen that consumers are provided with a discrete water supply in time in the form of a step function. With stepwise functions, to solve the problem of optimal control of water distribution, it is necessary to formulate criteria for the quality of water distribution in the canals of irrigation systems under conditions of discrete water supply to consumers and a system of restrictions.

III. CONCLUSION

An analytical review of modern and classical studies on mathematical models of the main canals of irrigation systems has been carried out, the results of which make it possible to determine specific areas of research for its implementation. The mathematical models of the main canals of irrigation systems are analyzed, taking into account the uniform water supply to consumers, the essence of which is to uniformly change the flow rate through the canal's hydraulic structures. This is of great importance to the national economy.

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AUTHOR'S BIOGRAPHY

Kabulov Anvar Vasilovich, Professor of National University of Uzbekistan named Mirzo Ulugbek, 15.03.1952.
Kabulov Anvar Vasilovich, doctor of technical Sciences, Professor of the Department "Information security", National University of Uzbekistan named after Mirzo Ulugbek, member of the Council of presidents of the International engineering Academy, head of the Engineering Federation of Uzbekistan, specialist in mathematical modeling and information technology. Currently, he conducts theoretical and practical research in the field of creating algorithmic and automatic models for managing complex systems and information security.
Address: Department "Information security" National University of Uzbekistan named after after Mirzo Ulugbek, Tashkent, Uzbekistan

Seytov Aybek Jumabayevich, was born in 1979, teacher. Author of more than 25 scientific papers. Researcher at the Laboratory "Modern Information Technologies in Water Management and Water Resources Management" Research Institute of Irrigation and Water Problems Ph.D. in technical sciences, received the B. E. Degree in Applied Mechanics-mathematics from National University Uzbekistan, Tashkent, Uzbekistan in 2000. He is pursuing PhD in Management, systems analysis and computer science at Tashkent University of Information Technologies, Tashkent, Uzbekistan. Her research areas include Mathematical modeling, optimal management of distributed parameters, management of water resources in canals, rivers and water facilities, Numerical implementations of nonlinear differential equations, information processing, Programming and Software Engineering.
Address: Laboratory "Modern Information Technologies in Water Management and Water Resources Management" Research Institute of Irrigation and Water Problems Ph.D. in technical sciencesTashkent, Uzbekistan

Kudaybergenov Adilbay Abatbaevich, was born in 1979. Researcher at Berdaq Karakalpak State University.Author of more than 10 scientific papers. Received the B. E. Degree in Applied Mathematics from Karakalpak State University, Nukus, Uzbekistan in 2000.
Her research areas include Artificial Intelligence, Programming, Graph Theory and Software Engineering.
Address: Department of Applied mathematics, Karakalpak State University. Nukus, Uzbekistan