Hydraulic resistances of eroded riverbed and assessment of their sustainability

Kenesbay Baymanov*, *Gulmurat* Shaniyazov, and *Torebek* Uzakov, and Ruslan Baymanov

Karakalpak State University, Nukus, Karakalpakistan

Abstract. In the world, due to the shortage of water resources, the most important issues are the rational use of water resources, the assessment, and the improvement of the capacity of riverbeds and channels. In this regard, research developments on improving the methods of the hydraulic calculation of the resistance and the stability of mobile river channels and channels are of particular importance. The results of some theoretical and experimental studies of hydraulic resistances and the stability of watercourses by scientists in the CIS and abroad are considered. The analysis of massive field data on the Darcy coefficient (Shezi) of earth channels covered in a sandy mobile bed is performed. With a change in the flow velocity field, the roughness of the channel surface changes, changing its size and shape, which determines the channel's resistance to flow movement. It is established that in stable channels with certain widths, the flow velocity is evenly distributed, having one maximum along the channel axis, and the movement is carried out in the form of a compact single jet along the flow cross-section. With changes in hydraulic conditions, the curvature of the isotopes is observed, the flow tends to split, and several closed systems of rotating the jet appear, which are signs of the development of channel deformations.

1 Introduction

Many works have been devoted to studying hydraulic resistances in open watercourses, both in the CIS countries and abroad. However, the inconsistency of the results does not allow us to determine the quantitative regularities of changes in the coefficient of hydraulic resistance from the main parameters and qualitative relationships. Some researchers believe that pressure losses in channels caused by artificial roughness are significantly affected by the Froude number and the slope of the water surface [1-2]. Among the dependences that reflect changes like resistances under different water flow regimes, graphs of the following types are known: Nikuradze-Zegzhda, which are not widespread for watercourses in the natural ground. Researchers [1-4] believe that in channels with artificial roughness, several

*Corresponding author: kenesbaybaymanov44@gmail.com

characteristic flow modes can be distinguished depending on the flow conditions, which have their own regularities of changes in the coefficient of hydraulic resistance. At the same time, it was shown that flows with large slopes, including those with increased bottom roughness, have features associated with the occurrence of hydraulic resistances.

At the bottom of the eroded channels, a complex ridge relief is formed, which changes the roughness of the bottom surface and the nature of its deformation. To consider this feature of the channel process in practical calculations when designing channels, it is necessary to determine the parameters of ridges that arise at the bottom of the channel from the hydrodynamic characteristics of the flow. Currently, there are several theories of the occurrence of bottom ridges in streams [5-9], which can be divided into four main directions: bottom ridges are generated by large-scale turbulent formations. The length of ridges coincides with the size of structures [1, 6]; the reason for the occurrence of sand waves is "attached" eddies arising from the flow of random irregularities of the bottom of the stream [8]; in the stream, there are fixed relative to the bottom of the vortex formation (their causes are not parsed) generating the ridge structure of the bottom; the bottom of the stream with the ridge topography is only robust to random perturbations [7].

Testing of existing theories in practice has shown. The fact that the calculations performed based on the proposed models differ for one stream [8] and for full-scale flows, such calculations indicate the imperfection of physical models of sand ridge generation in streambeds [5-9].

When discussing the problem of hydraulic resistances, discussions continue about the defining criteria. The role of number Reynolds' role in this problem is obvious. The fact λ that λ depends on the Froude number and the slope of the free surface discovered by several researchers is controversial. The question of which of the criteria to give preference to remains open.

In our research, we proceeded from the fact that the key issues of the problem of hydraulic resistances are: the ratio of roughness and drag coefficient λ and the relationship of hydraulic drag with riverbed processes.

2 Results and discussion

According to research, resistances along the length of a natural channel consist of two main parts: 1) resistances of the granular bottom surface and 2) resistances of the bottom ridges. To these two types of resistance along the length, we have the following equalities:

$$\lambda_R = \lambda_\Delta + \lambda_g \qquad (1)$$

or

$$\frac{1}{C^2} = \frac{1}{C_\Delta^2} + \frac{1}{C_g^2} \qquad (2)$$

In the written equations, the index Δ indicates the values related to the resistance of granular roughness, and the index g indicates the resistance of ridges.

In the calculations of channel resistance, only the values of the coefficients λ_Δ and C_Δ, which reflect the resistance of granular roughness, are easily and reliably determined. A theoretically valid way to determine them is to use logarithmic resistance formulas. Let's write this formula for sandbeds (median particle diameter d_{50}≤1.5mm) in the form

$$\sqrt{\frac{2}{\lambda_\Delta}} = \frac{C_\Delta}{\sqrt{g}} = 5.66\, lg\frac{R}{1.6 d_{50}} + 6.0 \qquad (3)$$

Along with logarithmic formulas, the empirical power formula is widely used in calculations Manning-Strickler

$$\frac{C_\Delta}{\sqrt{g}} = 6.67\left(\frac{R}{d_{50}}\right)^{1/4} \qquad (4)$$

Strickler's formula (4) is based on laboratory and in-situ measurements (on Swiss rivers) covering a wide range of bottom particle sizes.

As a result of his extensive series of experiments, V.S. Knoroz obtained the following formula for ridge resistance

$$\frac{C_g}{\sqrt{g}} = 3.16\left(\frac{l_g}{h_g}\right)\left(\frac{R}{h_g}\right) \qquad (5)$$

Formulas based on laboratory experiments, when applying them to the natural flow, were ineffective since the ridge relief in laboratory trays, with their small width-to-depth ratios and vertical flat walls, underestimated the resistance. Therefore, it is possible to try to find a way out of the difficulty by insinuating dependence with the help of full-scale research. B.F. Snishchenko made measurements on several rifts of the Volga, Oka, Don, and Polometi, got the formula

$$\sqrt{\frac{2}{\lambda_g}} = \frac{C_g}{\sqrt{g}} = \sqrt{\frac{l_g}{h_g + 0.033\, l_g}} \qquad (6)$$

where h_g and lg are the height and length of ridges, respectively; h_g/h *and* h_g/l_g are dimensionless geometric characteristics of ridges – their steepness h_g and l_g and relative height h_g/h.

The main argument of the ridge drag formulas, which is not explicitly related to the size of ridges, can naturally be considered the degree of stability of bottom particles. G. Einstein [13] proposed to take as such an argument the value of:

$$\Psi = \frac{(\rho_s + \rho) g d}{\tau_\Delta} = \left(\frac{\rho_s}{\rho} - 1\right)\frac{d}{RJ_\Delta} \qquad (7)$$

where p_s is the particle density, p is the liquid density, and the value Ψ is called the stability coefficient of bottom particles.

In this paper, the problem of hydraulic resistance of earth channels is solved based on an analysis of mass homogeneous field data from studies of irrigation channels in Central Asia using the Darcy coefficient (Shezi) and the degree of stability of bottom particles [3,5]. The data corresponding to the most stable state of the riverbed corresponding to the conditions of stability of bottom soil particles are selected:

$$\bar{u} + 3\sigma_u = u_o \qquad (8)$$

where $\bar{u}$ is the average velocity, $\sigma_u = 1.4u_*$ is the root-mean-square deviation from the average velocity, $u_* = \sqrt{gRJ}$ is the dynamic velocity, $u_o = 0.64\sqrt{hd}$ is the critical velocity, at which gravity is balanced by the lifting force, h is the depth, m, d is the particle diameter, mm.

We rewrite G. Einstein's formula (7) in the following form:

$$\Psi = \left(\frac{\rho_s + \rho}{\rho}\right)\frac{gd}{u_*^2} = a\frac{gd}{gRJ} = a\frac{d}{RJ} \qquad (9)$$

from here

$$J = \frac{1}{\Psi} * \frac{ad}{R} \tag{10}$$

Substituting J into the Shezy formula, we obtain

$$u = C\sqrt{RJ} = C\sqrt{R\frac{1}{\Psi} \cdot \frac{ad}{R}} = \frac{C}{\sqrt{g}}\sqrt{\frac{a}{\Psi} \cdot gd} \tag{11}$$

Where from:

$$\frac{C}{\sqrt{g}} = const\frac{u}{\sqrt{gd}} \tag{12}$$

After processing and analyzing the data of field studies conducted by S.H. Abalyants, I.I. Goroshkov, A.D. Savarenskiy and L.B. Levanovskiy, and K.I. Baymanov [1-9], we constructed a graph of the dependence $C/\sqrt{g} = f(Fr_d)$ and obtain the dependence in the form (Fig. 1):

$$\frac{C}{\sqrt{g}} = 0.86\, Fr_d = 0.86\frac{u}{\sqrt{gd}} \tag{13}$$

From the formula (12) and (13), we determine the limit value of the channel bottom mobility parameter – Ψ. Here the parameter $a=(p_s-p)/p=1.65$ is for the Amu Darya alluvial deposits, then we have the following equality:

$$\sqrt{\frac{\Psi}{1.65}} = 0.86, where\, \Psi = 1.22 \tag{14}$$

From formula (15), we determine the dynamic flow velocity u_*:

$$u_* = \sqrt{\frac{a}{\Psi}} * \sqrt{GD} = 1.66\sqrt{gd} \tag{15}$$

Substituting formula (16) into the Shezi formula, we obtain the critical value of the non-diluting velocity:

$$u_o = \frac{C}{\sqrt{g}} * u_* = \left(\frac{C}{\sqrt{g}}\right)_= C\, GD\ * 1.66\sqrt{gd} = 17.5 * 1.66\sqrt{gd} = 20\sqrt{gd} \tag{16}$$

Where:

$$(C\ /\ \sqrt{g})\_(\,PREV) = 17.5 \tag{17}$$

Is the limit value of the dimensionless Shezi coefficient established from field measurements [15].

Thus, it can be assumed that the stability loss of bottom particles occurs at $u/\sqrt{gd} \geq 20$. At the same time, at the bottom of the riverbed, the geometry of bottom shapes formed at the bottom surface changes over time.

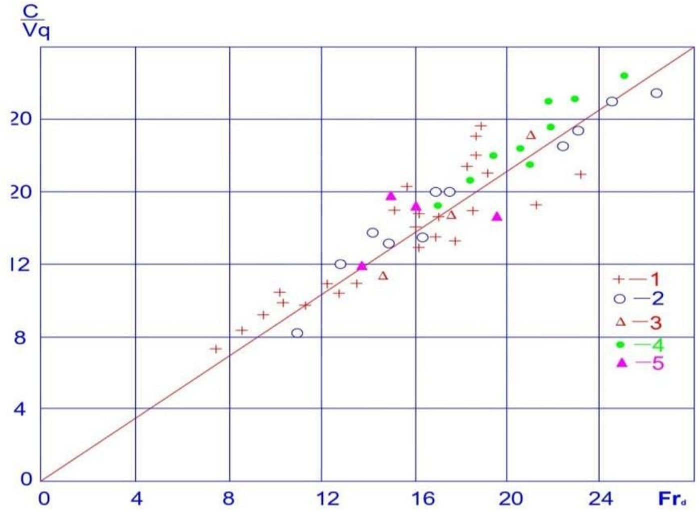


Fig. 1. Functional dependence $C/\sqrt{g} = f(Fr_d)$.1 is channels The Kyzketken irrigation system (1954-1955); 2 is channels The Kyzketken irrigation system, 3 is Karakum channel (1967); 4 is Supply channel of the Kyzketken irrigation system(1983-1986 y.); 5 is Tashsaka channel (1954).

Considered data from field studies [3, 14, 15] refers to channels with regular straight channels, maintained in satisfactory conditions and not overgrown on the wetted part of the slopes. A significant part of the data relates to channels with a bed of fine silted sand, sometimes with banks of sandy loam or loam, while the rest of the data relates to channels with sandy loam and loam. These channels tend to attract a certain amount of fine sand along the bottom in the form of small ridges. In relatively wide channels, the bottom roughness is the decisive factor for hydraulic losses [3]. Thus, more than 500 data with hydraulic radii and slopes within $0.20 \leq R \leq 4.0$ m and $0.000021 \leq J \leq 0.00198$ were taken as the basis for analysis and generalization *J*. In the conditions of the lower course of the Amudarya River, it is important to consider the influence of the flow saturation with suspended sediments on the drag coefficient.

Figure 2 shows the dependence graph in the rectilinear coordinate system *C=f(R)* according to field studies [3, 14, 15] conducted in the period from 1930 to 1985 in the channels of the lower reaches of the Amu Darya.

As shown in Figure 2, the location of the change in the Shezi coefficient is *C* depending on the hydraulic radius *R*. The spread of points is so large that without distinguishing points by different flow modes, it is difficult to speak about any resistance pattern. The dependence curves shown in Fig. 2 *C=f(R)* are constructed based on calculations using the formulas R. Manning, N.N. Pavlovskiy, I.I. Agroskin, and S.H. Abalyants at the roughness coefficient *n*=0.020 corresponding to earth channels in the normal state of the riverbed. Comparison of the plotted curves with full-scale points shows that they do not consider the sediment movement mode, and therefore most full-scale points are located above and below these curves. The study of hydraulic resistances of the suspension-bearing stream bed requires mandatory consideration of the sediment flow regime in the channel under consideration and elucidation of the relationship between the flow kinematics and the bottom relief [14-15].

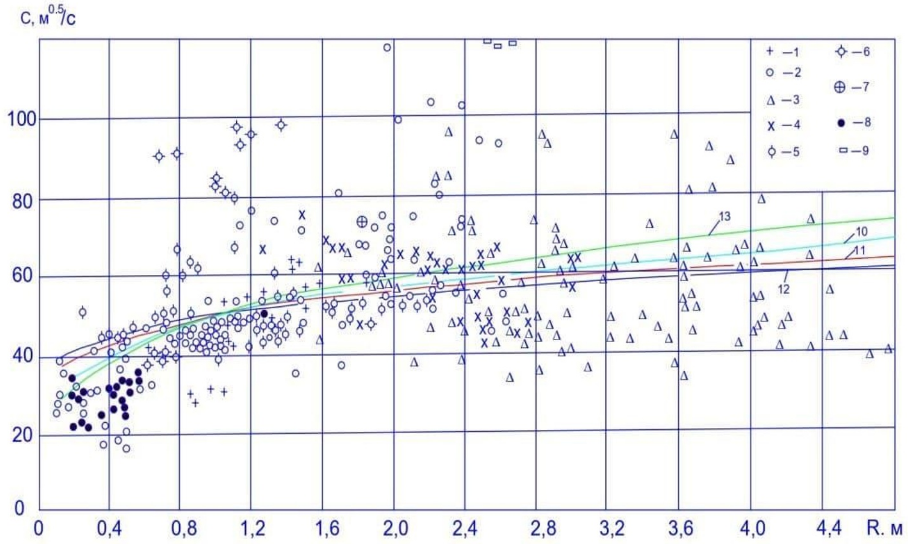


Fig. 2. Generalization of aggregate field data and comparison of calculated values for the Shezy coefficient. 1, 2, 3 are Kyzketken irrigation system (1932,1952-1953,1975-1986 years); 4 is Suyenli irrigation system (1976-1982); 5, 6 are South Khorezm channel (1930-1952); 7 is Khuankhe river channel; 8 is SANIIRI industrial channel (1952); 9 is Amudarya River (1976); Curves C=f(R) according to the formulas R. Manning (10), N.N.Pavlovsky (11), I.I.Agroskin (12), S.H.Abalyants (13).

Thus, the empirical formulas used in calculation practice for determining the Shezi coefficient do not sufficiently reflect the physical nature of the hydraulic resistance of the riverbed. The flow structure in the channels significantly depends on the degree of saturation of the flow with suspended sediments, with an increase in which the density and viscosity of the flow increase, which also affect the intensity of turbulence. Therefore, it is necessary to introduce additional indicators that consider these factors in the formulas for determining the hydraulic resistance of the riverbed.

Experiments show that hydraulic resistances in moving channels are directly related to their stability. Sediment transport is the main factor of riverbed processes in rivers and channels. In particular, excess sediments leads to instability of the planned forms of the channel, its widening, wandering, and multi-armedness [3, 5, 14]. To ensure the stability of the riverbed, it is necessary that sediments entering the irrigation system are freely and completely carried by the stream.

When calculating the cross-section of channels in the eroded soil, the average flow velocity and depth values are usually used and do not consider differences in the nature of the deformation (separation of particles) of the slopes and the bottom of the channels.

In experiments conducted at the hydro-physical laboratory of Moscow State University on eroded models of various cross-sections made from full-scale Amu Darya sand d_{CP}=0.2 mm, the regularities of the formation of dynamic stable channels were studied [13-15].

At the initial moment of channel formation, regardless of the shape of its cross-section, the channel deformation is basically the same. Initially, the ridges of ridges formed at the water's edge form an acute angle with it. At the same time, the bottom of the channel remains almost stationary.

The movement of sand ridges causes intense deformation of the slopes while the area occupied by the ridges expands. The direction of the ridge crest relative to the water cut is preserved. The ridges move parallel to each other. Along the basement ridges, the movement of sand grains to the flow axis is observed. Sand grains that have rolled down the slopes also form ridges at the bottom of the channel, which, unlike the ridges on the slope, are perpendicular to the flow axis. Since the formation of ridges at the bottom of the channel, the intensity of particle movement here increases sharply. As a result, the amount of sand set in motion exceeds the transport capacity of the stream, which leads to sediment deposition at the bottom in an amount determined by the difference between the solid flow rate and the transport capacity of the stream. This leads to an increase in the bottom level. From this point, intensive erosion of the slopes begins, leading to the expansion of the channel (Fig. 3).

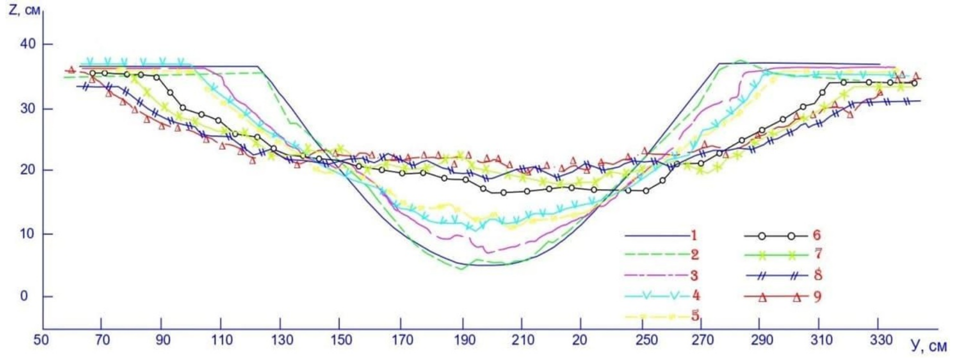


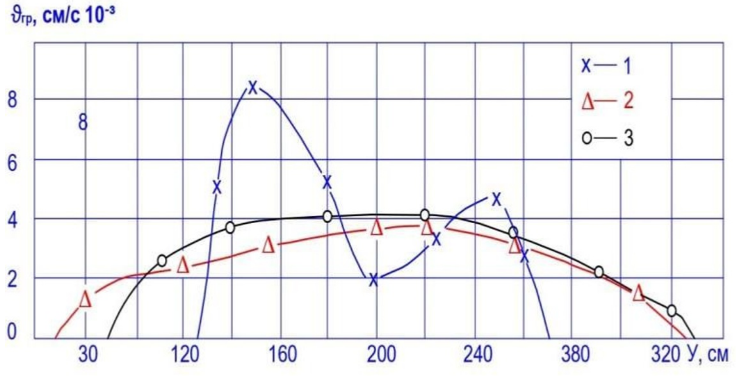
Fig. 3. Transverse channel profiles [17]. 1-9 are, respectively, before the experiment through 6; 26; 40; 47; 89; 159; 187 h after start of experiment.

Based on the results of this experiment, for different stages of channel formation, the distribution of the flow width of the speed of movement of sand ridges, which is an important characteristic of the intensity of the channel process, is obtained (Fig. 4).

In domestic practice, the most widely used method is the calculation of stable channel channels based on the permissible speed. The essence of this method is to assign the average cross-sectional velocity of the water flows less than the permissible value for this category of soil:

$$u < u_o \quad (18)$$

Where u and u_o are, respectively, the average and permissible water flow velocities determined for the average depth h_{av}.



Figю 4. Distribution of velocity of ridges. 1-3 along the channel width in 2; 78; 217 h from beginning

of experiment [17].

Analysis of field and laboratory studies shows that channel beds undergo deformations when the average flow rate is set even 25% less than the permissible speed, that is u, at u/u_o=0.75. The cross-section of channels traced in disjoint soils is most re-formed. Based on their experimental experiments, N. A. Mikhailova and M. M. Selyametov believe that calculations based on the permissible speed do not make it possible to build a channel with a cross-section that would not deform. Asserts that the calculation of the dynamic stable cross-section of the channel should be carried out taking into account the average and pulsation characteristics of the flow.

Experiments carried out (14, 17) showed that the assignment of average velocities does not ensure the fulfillment of condition (18) on all verticals of the channel cross-section. Based on field studies, it was confirmed that the stability of the riverbed is most fully determined by the distribution of the Froude parameter [18].

When analyzing the data of studies on the formation of channels by self-washing methods with increased water consumption [14], it was found that the local value of the Froude parameter Fr_o , i.e.,

$$Fr_b = \frac{u_b^2}{gh_b} < Fr_o = \frac{u_o^2}{gh_{CP}} \qquad (19)$$

where u_B is the average vertical velocity $h_в$; HB is the vertical depth; g is the acceleration of gravity; u_o is the permissible flow velocity; h_{CP} is the average flow depth.

Analysis of the process of formation of the channel shows that depending on the shape parameter of $\beta=B/h_{CP}$ in the deformation of the channel, the distribution width of the value of Fr_B all dimensions of cross-sections has the following pattern of changes (Fig.5): if β=19 maximum value of Fr_B is in the cut-off area and the flow axis; when β=30 a maximum value of Fr_B has a cut-off area and ¼ in the right and left of the axis of the stream, where there is maximally flow rate. When the dynamically stable cross-section is finally formed, the maximum value of Fr_B falls on the flow axis, and conditions (19) are met. If we compare the measured cross-sections Fr_B with the permissible value of Fr_o in different periods of the study (from 1975 to 1978), we can trace a detailed description of the process of forming a dynamically stable channel by self-flushing with an increase in water flow from 180m^3/s to 360m^3/s.

In recent years, low water levels have been observed, and the channels have begun to receive small water flows that do not correspond to the formed cross-section. Thus, there was a change in the regime of water velocities and flow rates in the direction of decrease, and siltation of the channel and an increase in the bottom mark were observed. At the same time, due to the strong deformation of the channel slopes at its bottom, there was a further increase in the channel expansion. Consequently, channels with the passage of time under the influence of deep regulation of runoff approach in their forms to natural channels.

It is theoretically and experimentally established that the formation of a longitudinally helical motion with transverse circulation flows. The structure of the direction of rotation of the transverse circulation, the position and number of these helical movements change depending on the head of the well, the speed, the width of the stream, and the shape parameter of the channel.

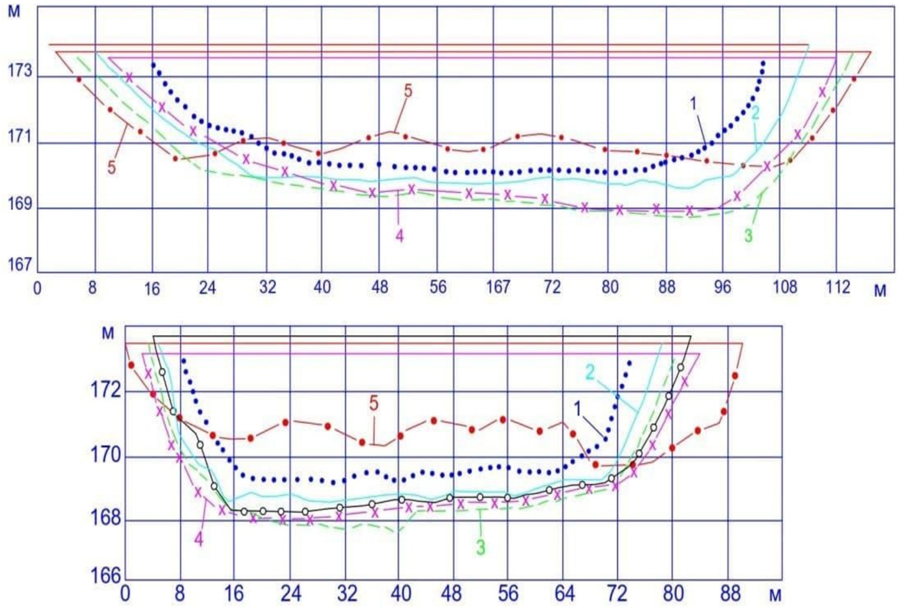


Fig. 5. Combined paper profiles of Kyzketken channel in following characteristic sections: PK135 (β=30); PK160 (β=19). 1-16.09.1970 y; 2-15.07.1975 y; 3-06.07.1978 y; 4-22.08.1985; 5 – 26.07.1999.

Analysis of the measurement data on the distribution of averaged velocities over the depth and width of the flow with the indication of isotach on the transverse profiles [19] shows that in stable channels with a certain width, a symmetrical arrangement of isotach is observed parallel to the contour of the wetted perimeter. As the hydraulic conditions change and the width increases, there is a curvature of the isotopes, a tendency of the flow to split and bend the dynamic axis of the flow, and the appearance of several disordered closed systems of a rotating jet (screws), or several extremes of averaged velocities are observed, which are signs of the development of channel deformations. Thus, the analysis of the velocity field structure in the transverse flow profile leads to the conclusion that a stable channel is characterized by its own definite cross-sectional shapes and the channel shape parameter β and the corresponding characteristics of the turbulent flow structure.

To assess the homogeneity of the velocity field, I. F. Karasev recommended $\theta_B = \beta\sqrt{\lambda}$ of quasi-homogeneity, the kinematic structure in the cross-section of the flow, for which he established the criterion θ=4.5 for a river flow, and this criterion requires clarification when applied to channels. In this formula of replacing $\sqrt{\lambda}$ with the values of the dimensionless Shezy coefficient excluding$\sqrt{2}$, we obtain:

$$\theta_B = \beta\frac{\sqrt{g}}{C} = \beta\frac{u_*}{u} \tag{20}$$

where $\beta=B/h_{CP}$ is the channel shape parameter; $u_* = \sqrt{gh_{CP}J}$ is the dynamic velocity; θ_B is the quasi-uniformity of the flow turbulence; B is the width of the flow at the top; u is the average flow velocity.

Analysis of the data from studies of the channel operation mode and calculations performed [15] showed that the isotach is most symmetric at β=18÷22; θb_B=1.0. For β<14; θ_B <0.8, the flow seems to be compressed by the side walls. And at β=25÷35; θb_B=1.0÷2.0, the isotach bifurcation is observed in the flow with the allocation of independent dynamic axes of the flow. At β>35; > θ_B>2.0, the flow is completely divided into separate structurally isolated forms.

Thus, the analysis of the velocity field structure in the transverse flow profile concludes that a stable channel is characterized by its own specific cross-section shapes and channel shape parameters β and the corresponding characteristic of the turbulent flow structure.

3 Conclusion

Based on the analysis of existing theoretical, laboratory, and field studies of the hydraulic resistance and stability of riverbeds by scientists from the CIS and abroad, as well as their own field studies on the channels of the lower reaches of the Amu Darya River, the following conclusions were obtained:

1. The regularities of occurrence of ridge forms on the bottom of eroded channel beds and the nature of its deformation are Revealed.

2. According to research, the resistance along the length of the riverbed consists of two main parts: grainy surface resistances and bottom ridges resistances. Formulas for determining these resistances (4) and (6) are proposed. To determine the total resistance of mobile channels of earth channels based on the formula of G. Einstein, the formula (14) is developed.

3. Data from long-term studies on channels make it possible to establish that hydraulic resistances change over time following the degree of stability. The design task should consist in determining stable channel sizes, at which subsequent reshaping will proceed gradually and will not cause a violation of the channel capacity.

4. Sediment Transport is the main factor of riverbed processes in rivers and channels. In particular, an excess of sediments leads to instability of the planned forms of the riverbed, its widening, wandering, and multi-armedness. To ensure stability, it is necessary that the sediment entering the system is carried freely and completely by the flow.

5. To establish quasi-uniformity of the kinematic flow structure, it is necessary to observe the conditions of uniformity of the General turbulent flow structure proposed by formula.

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