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Improving the Methodology for Calculating the Capacity of Pipelines of Closed Irrigation Networks

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Abstract

The article deals with problem solving, determining the potential throughput of pressure pipelines of a closed irrigation network. A fundamentally new method of calculating the regularity of velocity distribution in round pipes was derived, on the basis of which a new formula was obtained for determining the flow resistance in pipes, which makes it possible to reliably determine the throughput capacity of pipelines in a closed irrigation network. Consequently, in the proposed equation (16) and resulting from it by expressions using formula (23), the existing drawbacks are completely absent, which indicates a more correct solution of the problem in this case. It should be recognized that the basic analysis conducted here is of interest in comparing the results of the calculation using formulas (20) and (23), since the carrying capacity of the pipeline largely depends on the value of the coefficient λ. For this, when calculating, we considered pipes of two diameters d=100 mm and d = 1000 mm. Note that pipes with a diameter of d=100 mm are most often used when installing progressive irrigation techniques. Calculation for pipes with d =1000 mm was considered for comparative evaluation of λ values.

Keywords: Irrigation system; Capacity; Law of velocity distribution

Introduction

The increasing use of closed irrigation systems in agricultural production necessitates a reliable determination of the carrying capacity of their pipelines, which is of interest in the design and construction of these systems. Known proposals for solving this problem have certain disadvantages, leading to various kinds of errors, the results they produce. This situation necessitates deep theoretical studies for more reliable determination of the throughput capacity of pipelines of closed irrigation systems. To solve this problem, first of all, we will focus on the analysis of known proposals for determining the capacity of pipelines of closed irrigation networks, which is described below. Existing methods for calculating the velocity distribution in the turbulent motion of water in a circular pipe. The regularity of the velocity distribution is of fundamental importance in solving practical and theoretical problems of flow hydraulics in pressure pipelines. It also serves as a starting point for assessing the resistance to flow in pipelines.

Experimental and theoretical studies of the distribution of velocities over the cross section of pipelines and resistance to flow in them are devoted to work [1-8].

Research courses

The logarithmic and steppe laws of velocity distribution have found wider application in practice, which will be discussed in some detail below.

Logarithmic velocity distribution law: As we see it, the most reliable basis for deriving the velocity distribution equation would be the Reynolds equations/10/. However, this system of equations does not allow solving the problem in view of the fact that it does not close and the solution of this issue is encountered with very great difficulties.

In principle, the logarithmic velocity distribution law is based on the hypothesis of Prandtai L [6] that the length of the displacement path (the relationship between the turbulent exchange coefficient and the velocity field) at the wall is directly proportional to the distance "y" from the wall, i.e. e = xy.

In turn, Landau LD and Livshits EM [9] / 11 /, from considerations of dimension, and Nikuradze I [10] /12/, on the basis of experimental data in pipes with artificial roughness, arrived at the velocity distribution equation in the following form:

$$U\ U* = 1\ x \ln r\ \Delta + N \qquad (1)$$

Where
U - Speed at various points of the pipe radius, m / s;
x is the Pocket coefficient; x = 0.4 r is the current radius of the pipe, m;
Δ is the height of the protrusions of the pipe roughness, m;
N is some constant number determined from experience;
$U* = \sqrt{\tau} / \rho$ - dynamic speed, m/s;
τ - friction stress on the wall, t/m^2;
ρ - density, t/m^3.

The value of the constant N in the formula (1) is expressed as the ratio

$$N = U\Delta\ U* \qquad (2)$$

Where: $U\Delta$ is the given speed at the height of the protrusions of the roughness of the pipe walls, m/s;

For pipes, various studies have obtained different values of constant N. The most reasonable of them can be considered N = 8.5, obtained by Nikuradze I [10] on the basis of extensive and carefully set experiments. Then, taking the value x = 0.4 and moving from natural to decimal logarithms, equation (1) can be reduced to

$$U\ U* = 5.75\ \ell g\ r\ \Delta + 8.5 \qquad (3)$$

The logarithmic law of velocity distribution (3), as shown by studies /13/, well satisfies the experience within the limits of the pipe radius closest to the wall 0.2. Therefore, its application throughout the entire stream is not always justified.

In addition, the distance from the wall, where the local averaged velocity equals the average velocity on the vertical from (1) is obtained in the form,

$$rb = ro\ e = 0.37ro \qquad (4)$$

Where:
ro - pipe radius, m;
e - Neprevo number (e = 2.72).

As can be seen from (4) the distance from the wall re, where the local averaged speed equals the average velocity on the vertical does not depend on the resistance of the pipe and, consequently, the shape of the velocity distribution diagram, which is also a disadvantage of the logarithmic law. As can be seen, the logarithmic law of velocity distribution along the pipe radius has a number of significant drawbacks that negatively affect the final results of the problems solved with its help. Power law of velocity distribution.

Along with the logithmic general fame, the power law of velocity distribution has the following general form /13/

$$U\ Uo = (r\ r0)\ 1 / \alpha \qquad (5)$$

Here,
Uo - speed on the pipe axis, m / s;
ro is the pipe radius, m;
r is the current radius, m;
α is the exponent of the velocity distribution curve equation.

Various studies have shown that using equation (5), one can solve a number of problems of kinematics and resistance to flow in pipelines. To do this, you need to know the value of the parameter α, 5 which is a variable.

To this end, a number of researchers dealt with the question of determining the value of α, which obtained various types of formulas for α. Moreover, some authors associated the value of α with the roughness coefficient, others with the Chezy coefficient, the third with the absolute value of roughness and depth of flow, etc.

Regarding equation (5), it can be noted that in addition to the different approaches to determining the value of α and the ambiguity of their end results, its essential drawback is that it responds well to the main thickness of the stream, does not correspond to it in the bottom layer.

From the above brief analysis of equations (1), (3) and (5), their shortcomings become apparent and, therefore, the need for further research in order to obtain more perfect patterns free from the shortcomings of both logarithmic and power velocity distribution laws. Such an attempt was made by Doctor of Technical Sciences Nureyev ChG [11] in work / 14 /. However, he obtained only a new equation for the distribution of velocities along the radius of a power-type pipe, which differs significantly from (5). In solving this problem, Nuriev ChG [11] emanates / 59 / from the parabola equation in general form (Figure 1):

$$(Ux - U\Delta)\ \alpha = 2P\ (ro-r) \qquad (6)$$

As a result of a series of transformations, he comes to the equation of the velocity distribution along the radius at the boundaries between the pipe axis and the height of the roughness protrusions on its wall. This equation has the following form:

$$U\eta = Uo - 3.75\ a\ a - 2\ [1- (1-\eta)\ 1/2] \cdot U * \qquad (7)$$

Here, the following notation:
Uo - speed on the pipe axis, m / s;
η = r / ro - relative radius:

$$a = (1+ 1\ 2)\ (2+ 1\ 2) \qquad (8)$$

The remaining notation is the same. Next Nuriyev CH G [11] found that the distance from the axis of the pipe, where the local averaged velocity equals the average velocity on the vertical, is expressed as:

$$\eta sr = 1 - (2\ d)\ 1 / \alpha \qquad (9)$$

If we bear in mind that the author (7) obtained the connection between α and the resistance coefficient (Darcy) λ in the form / 14 /:

$$1\ \alpha = \sqrt{2.25-4.07}\ \sqrt{1-2.99}\ \sqrt{-1.5} \qquad (10)$$

U Δ ro r Ux I Δ Uo

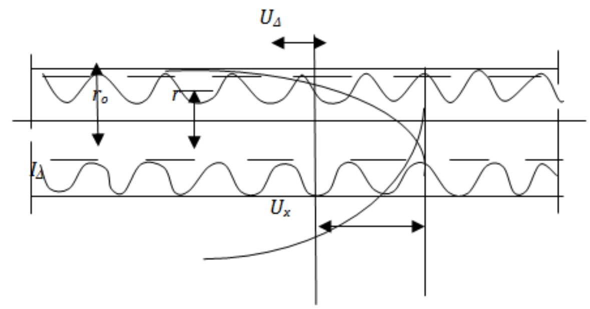

Figure 1: Calculation scheme.

This can be stated: formula (9) expresses the dependence of the distance on the pipe axis, where the local average speed is equal to the average speed on the vertical, on the drag coefficient λ, therefore, on the type of the velocity distribution diagram and therefore is variable. This and 7 is one of the positive distinctive features of the power law of velocity distribution from the logarithmic one.

On the other hand, when η = 1 (which corresponds to the height of the roughness protrusions on the pipe wall), the value of the bottom velocity at the height of the roughness projections is obtained from (7):

$$U\Delta = Uo - 3.75\ a\ a - 2\ U * \qquad (11)$$

This position in the previously proposed equation (5) corresponds to r = 0. At the same time, from the expression (5) we get UΔ = 0, which does not correspond to the real state of things, since the condition UΔ всегда 0 always holds. Thus, equation (7) derived by Nuriev ChG [11], is free from this lack of formula (5), which indicates a higher confidence (7).

It should be noted that the received H.G. The Nureyev pattern of velocity distribution (7) is compared with the data of laboratory studies of Nikuradze [10] / 12 /, which showed their very close coincidence.

Unfortunately, Doctor of Technical Sciences Nureyev Ch G [11] did not further develop work on the solution of this problem and limited himself to the above results, which can serve as a basis for further research in this direction. As noted above, Ch.G. Nuriev [11] considered the problem of the velocity distribution in the flow region between the pipe axis and the height of the roughness projections, i.e. without taking into account the movement of water within the height of the roughness of the ridges. We consider the next section of this paper to consider the solution to this problem. Regularity of the distribution of velocities, taking into account the roughness.

To solve this problem, we take the equation of the velocity distribution in the form (Figure 1)

$$Ur = Uo\ (1\text{-}\ r\ r0 + \Delta)\ 1 / \alpha \qquad (12)$$

Here
Δ - the height of the protrusions of the roughness on the pipe wall; m;
r - measured from the axis of the pipe.
At the same time, we assume that the speed at the base of the roughness protrusions is zero and that the velocity distribution diagram intersects the conditional line running along the tops of the roughness equal to the end of the segment, measured from the origin (Figure 1).
When r = ro, we have Ur = UΔ and equation (12) takes the form

$$U\Delta = Uo\ (\Delta\ ro + \Delta)\ 1 / \alpha \qquad (13)$$

Dividing (12) by (13) and deciding on Ur we get

$$Ur = U\Delta\ (1 + ro - r\ \Delta)\ 1 / \alpha \qquad (14)$$

Further, from the joint solution (12) with N = 8.45 and (11) we find

$$Uo\ U\Delta = 12.2a - 16.9\ 8.45\ (a - 2) \qquad (15)$$

In view of (15), equation (14) is reduced to the form

$$Ur\ Uo = 8.45\ (a - 2)\ 12.2\ a - 16.9\ (1 + ro - r\ \Delta)\ 1 / \alpha \qquad (16)$$

This is the velocity distribution equation in a circular pipe, taking into account the height of the roughness protrusions.

As a control of compliance with the boundary conditions from (16) with $r = ro + \Delta$, i.e. at the base of the roughness protrusions, it turns out Ur = 0;

When r = ro, which corresponds to the height of the protrusions of the roughness, Ur = UΔ, and equation (16) takes the form

$$U\Delta\ U\ o = 8.45\ (a{-}2)\ 12.2{-}16.9 \qquad (17)$$

One of the important parameters in the velocity distribution equation (16) is the exponent α, which, as noted above, has a variable value. Therefore has a great theoretical e and practical value determining the value of this parameter. To solve this problem, we use the boundary conditions of equation (16). So, when r = 0, the speed Ur corresponds to the speed on the pipe axis, i.e. Ur = Uo holds and Eq. (16) is expressed as follows:

$$1 = 8.45\ (a{-}2)\ 12.2a{-}16.9\ (1 + ro\ \Delta)\ 1 / \alpha \qquad (18)$$

Logarithm of expression (18) leads us to the equation

$$\ell g\ (ro - \Delta\ \Delta) = \alpha \ell g\ 12,\ 2a{-}16.9\ 8.45\ (a{-}2) \qquad (19)$$

Still a = (1+ 1) (2+ 1 α) Equation (19) makes it possible to determine the value. However, due to the complexity of this expression, an analytical determination of the value is difficult, since solving equation (19) is a selection. Therefore, equation (19)

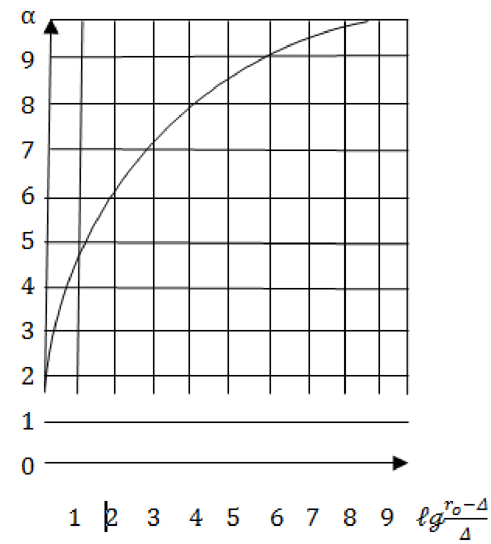

Figure 2: Graph of α = ƒ (ro − Δ Δ) for pressure pipelines.

Resistance to flow in round pipes.
Reliable determination of the coefficient of resistance to flow movement depends on the degree of accuracy of the velocity distribution source for this equation along the pipe radius. Up to the present time for a fully developed turbulent flow, i.e. for the square domain of resistance, the logarithmic velocity distribution law was generally taken as the initial one.

From this law, taking into account Nikuradze's [10] experimental refinement, an expression for the coefficient λ is obtained in the form:

$$1\ \sqrt{\lambda} = 2\ell g\ ro\ \Delta + 1.74 \qquad (20)$$

This dependence will be used by us in the future to estimate the throughput of the pipeline in comparison with the dependence we recommend for determining the Darcy coefficient λ.

To solve this problem, we use the well-known expression / 13 /

12 Uo = Ucp + 3.75U * (21)

Then equating (11) to (13) subject to (21) and the ratio

$$Ucp\ U* = 2\sqrt{2}\ \sqrt{\lambda} \quad (22)$$

After simple transformations, it is possible to obtain a formula for determining the value of the coefficient of hydraulic friction (resistance) λ in the following form:

$$1\ \sqrt{\lambda} = 1.33\ \{a\ (\alpha - 2)\ [1-(\Delta\ ro + \Delta)\ 1\ /\ \alpha - 1\} \quad (23)$$

Here α is determined from the graph
Figure 2 with a known value of $\ell g\ ro-\Delta\ \Delta$, and the value of α is from expression (8). Thus, we obtained a completely new equation (23) for determining the value of the coefficient of hydraulic friction λ, which makes it possible to obtain more reliable and real values of this coefficient.

Equation (23) makes it possible to determine the value of the coefficient of hydraulic friction (Darcy) for pipes of any diameters at known heights of protrusions of their roughness.

The reliability of equation (23) is indirectly confirmed by the fact that the velocity distribution equation (7) quite accurately corresponds to the real velocity distribution over the cross section of the pipe, obtained by Nikuradze [10] as a result of carefully set up experimental studies on their measurement. As for equation (16), it is a development of equation (7), taking into account the flow zone within the height of the roughness protrusions.

Comparison of the results of the calculation of the resistance coefficient for various formulas as noted above, one of the main parameters of the reliable determination, which determines the accuracy of determining the flow of water, is the channel resistance coefficient to flow (Darcy coefficient) λ. This coefficient, based on the logarithmic velocity distribution law (16), is derived from equation (23). If we analyze these formulas, we can see that for smooth pipes, when Δ = 0, from logarithm (20) we get λ = 0, which is impossible, since there are no absolutely smooth surfaces, especially the walls of pipes. As for our equation (23), here, at Δ = 0, we have an expression for determining the resistance of the flow in smooth pipes in the form

$$1\ \sqrt{\lambda} = 2.66\ a - 2 \quad (24)$$

In this expression, the viscosity characteristics of the flow come into play and the value of λ included in (8) must be determined taking these characteristics into account. However, this issue is not considered in this paper. This circumstance also testifies to the imperfection of the logarithmic velocity distribution law and formula (20) that follows from it for definition.

But in our equation (16) and expression (23), which follows from it, these deficiencies are completely absent, which indicates a more correct solution of the problem in this case. Based on this brief but fundamental analysis, it is of interest to compare the calculation results using formulas (20) and (23), since the carrying capacity of the pipeline largely depends on the value of the coefficient λ. For the calculation we consider pipes of two diameters d = 100 mm and d = 1000 mm. Note that pipes with a diameter of d = 100 mm are most often used when installing progressive irrigation techniques.

Calculation for pipes with d = 1000 mm 14 is considered for comparative evaluation of λ values.

Pipes of such diameter in our country are also often used for water supply and other purposes. In the calculations we consider several options with different heights of the protrusions of the roughness Δ; 0.01; 0,005; 0.001; 0.0005; and 0, 0001 m. To determine the value of λ of the formula (20) and (23), we will respectively give the form:

$$\lambda = 1\ (2\ell g ro\ \Delta + 1.74)\ 2\ (25)\ \lambda = 0.565\ \{(\alpha - 2)\ [1-(\Delta\ ro + \Delta)\ 12]\ a-((\alpha - 2)\ [1\ (\Delta\ ro + \Delta)\ 12]\}\ 2 \quad (26)$$

Then for a pipe with d = 100 mm (ro = 50 mm) with Δ = 0.01 m, by the formula (25) we find λ = 1 (2ℓg0.05 0.01 +1.74) 2 = 0.1 to determine the value λ by the formula (26) it is necessary to know the value of the index α. Therefore, knowing ro = 0.05 m and Δ = 0.01 m, we find $\ell g\ ro + \Delta\ \Delta = \ell g$ 0.05 + 0.01 0.01 = 0.778 knowing $\ell g\ ro + \Delta\ \Delta$ = 0.778, from the graph in figure 2 we find α = 2.15.

Then from (8) we will have α = (1 + 1 2, 15) (2+ 1 2, 15) = 3.6

Next, using equation (26), we determine the value of the coefficient λ: λ = 0.565 {(3.6−2) [1− (0.01 0.05 + 0.01) 1 / 2.15 3.6− (3, 6−2) [1− (0.01 0.05 + 0.01) 1 / 2.15} 2 = 0.064 15 Calculate, thus the values of λ by formulas (25) and (26) with d = 100 m are given in Table 1

Δ. м	$\frac{r_o}{\Delta}$	$\frac{r_o+\Delta}{\Delta}$	ℓ□$\frac{r_o+\Delta}{\Delta}$	**α**	**a**	**formula λ (25)**	**(26)**
0.01	5	6	0.778	2.15	3.60	0.1000	0.064
0.05	10	11	1.040	2.50	3.20	0.070	0.043
0.001	50	51	1.708	4.00	2.81	0.038	0.027
0.0005	100	101	2.000	4.40	2.73	0.030	0.025
0.0001	500	501	2.700	5.60	2.57	0.020	0.017

Table 1: Calculate, thus the values of λ.

It can be seen that in all cases the value of the friction resistance coefficients λ by our formula (26) is less than that by formula (25), although with a decrease in the magnitude of the roughness protrusions, λ determined by both formulas approach each other.
Consequently, the throughput of the same pipelines according to our formula (26) is obtained more than by formula (25), since λ when determining the water flow is included in the denominator of the equation

$Q = W \cdot Vcp = W \sqrt{8\, g\, \lambda \cdot iro}$ (27)

Similar calculations are made for pipes with a diameter of d = 1000mm. The results are shown in Table 2.

Δ. м	$\frac{r_o}{\Delta}$	$\frac{r_o+\Delta}{\Delta}$	$\ell g\frac{r_o+\Delta}{\Delta}$	α	a	λ by the formulas (25)	(26)
0.01	50	51	1.708	4.08	2.80	0.037	0.027
0.05	100	101	2.004	4.60	2.70	0.030	0.023
0.001	500	501	2.700	5.60	2.57	0.020	0.017
0.0005	1000	1001	3.000	6.05	2.52	0.017	0.015
0.0001	5000	5001	3.700	7.00	2.45	0.012	0.012

Table 2: Similar calculations are made for pipes with a diameter.

In this case, the value of λ according to our formula (26) is obtained significantly less than by formula (25) and only for very small Δ (in this case, at Δ = 0.0001 m), the value of λ by formulas (25) and (26 a) match with each other.

For greater clarity, the data in Tables 1 and 2 are graphically depicted in Figure 3 and 4 below.

From these figures it is clearly seen that by the formula (26) the value of λ always turns out to be less than by the formula (25) and only for very small values of Δ do they approach each other. However, these approximations are random in nature, since for smooth pipes, i.e. when Δ = 0 by the formula (25) λ = 0, which cannot be, and our formula (26) is simplified and takes the form:

$\lambda = 0.14\,(a-2)\,2$ (28)

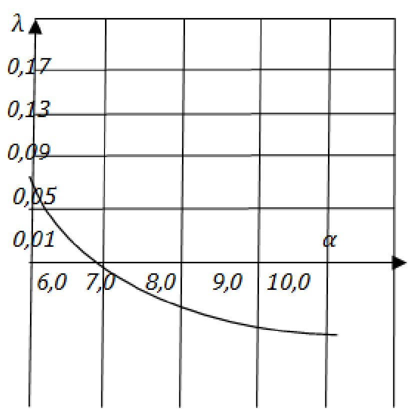

Figure 3

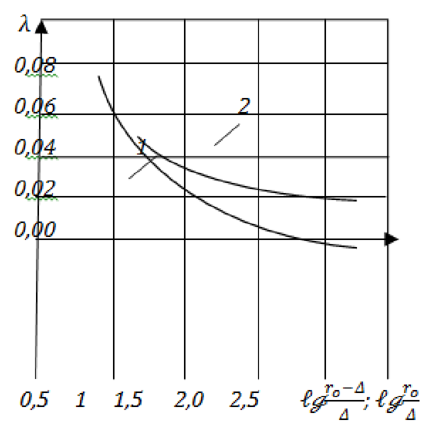

Figure 4

Figure 3: Graph of the function λ = ƒ (α) for smooth pipes.
Figure 4: Curves of λ = ƒ (ℓg $ro - \Delta\ \Delta$; ℓg $ro\ \Delta$) according to formulas (36a) - 1 and (39a) - 2 for pipes with a diameter of d = 100mm.

The work of Schlichting G [12] / 13 / presents graphs comparing the results of calculations for smooth pipes according to the power law of velocity distribution with experimental data, where the values of α were selected taking into account the best fit of the calculated and experimental values of speeds to each other.

It turned out that a smaller value of the Reynolds number Re corresponds to a smaller value of α. So, for Re = 4 · 103, the value α = 6; Re = 2.3 · 104, α = 6.6; Re = 1.1 · 105, α = 7.0; Re = 1.1 · 106, α = 8.8; and when Re = 2 · 106 and Re = 3.2 106, α = 10.

Determine the value of the Darcy coefficient λ for smooth pipes with reduced values of α, obtained from experiments. To do this, first determine the value of a. according to expression (8) with α = 6, we have: a = (1 + 1 6) (2 + 1 6) = 3.11 18 thus determined values of a and α by formula (28) are given in Table 3.

α	6.0	6.6	7.0	8.8	10.0
a	3.11	2.48	2.45	2.35	2.32
λ	0.172	0.0323	0.0284	0.0172	0.0135

Table 3: Graphically depicted.

As can be seen from this figure, the value of λ has the greatest value at λ = 6, and then drops sharply and has real values for smooth pipes with α ≥ 7, since in smooth pipes the drag coefficient λ cannot also have large values that occur when λ> 7.0.

Thus, our equation (23) or (26) is of a general nature as compared with formula (25) and differs from it in its higher accuracy and reliability of the results produced.

Returning to the Table 1 and 2 we will depict their results graphically in Figure 4 and 5.

From these figures it can be seen that the values of λ according to our formula (26) are always less than those obtained from formula (25) and only for very small values of Δ do they approach each other. This situation suggests that, ceteris paribus, with the same diameter of the pipe, its throughput according to our formula is obtained significantly more than using formula (25) or the same water flow in our case can be skipped through the pipe with a small diameter than the formula (25).

We will show what was said on the example of calculation.

An example of calculation, when calculating pipelines, the value of the coefficient of resistance λ is determined first. We considered this question above; the results of the calculations are given in Table. 1 and 2, respectively, for pipes with d = 100 19 mm and d = 1000 mm.

Next, we will consider the solution of the problem of the pipe carrying capacity with d = 100 mm with the value of the height of the roughness projections Δ = 1.0 mm.

It is known that the water velocity in the pipe is determined from the expression V = √ $8g\ \lambda\ iR$ (29)
Where R is the hydraulic radius, m

If we denote the value of the resistance coefficient obtained from the logarithmic formula (25) λ, and according to our formula (26) λ2, then with the same slope of the pipeline we can respectively write

$$V1 = \sqrt{8g\ \lambda 1\ iR1} \qquad (30)$$

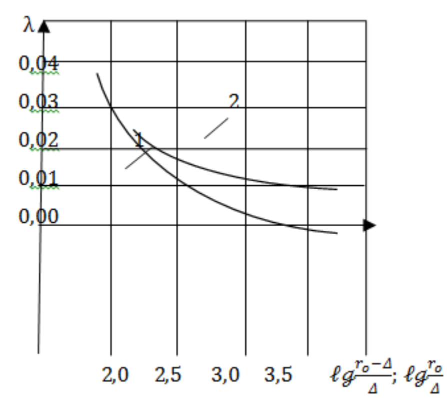

Figure 5: Curves of λ = ƒ ($\ell g\ ro - \Delta\ \Delta$; $\ell g\ ro\ \Delta$) according to formulas (36a)-1 and (39a)-2 for pipes with a diameter of d = 100mm.

$$V2 = \sqrt{8g\ \lambda 2\ iR2} \qquad (31)$$

Then dividing (31) by (30) we will have:

$$V2\ V1 = \sqrt{\lambda 1\ \lambda 2 \cdot R2\ R1} \qquad (32)$$

From this expression you can get:

$$V2 = V1\ \lambda 1\ \lambda 2\ R2\ R1 \qquad (33)$$

Suppose that with the help of the pipeline water is supplied by the sprinkler, the flow rate of which must be determined.

The diameter of the pipeline d1 = 100 mm (outer diameter) dн = 108 mm with a wall thickness t = 4.0 mm), the height of the roughness projections Δ1 = 1.0 mm.

For these data, the value of $\lambda 1$, according to the formula (25) is $\lambda 1$ = 0.038, and according to the formula (26) $\lambda 2$ = 0.027 (Table 1).

Accept the value of the slope i = 0.01.

Then by the formula (30) the water velocity in the pipeline will be

$$V1 = \sqrt{8g\,\lambda 1\, i\, d\, 4} \qquad (34)$$

Substituting the corresponding values of the parameters, we get: V1 = √ 8 · 9.81 0.038 · 0.1 4 0.01 = 2.27 m/s
Taking into account the value $\lambda 2$ = 0.027 according to our formula (26) from (33) with $R1 = R2$ we will have: V2 = 2.27 √0.038 0.027 = 2.69 m/s

Now we determine the carrying capacity of the pipeline when calculating according to the logarithmic law of velocity distribution (V1 = 2.27 m/s) and 21 of the power law we obtained (V2 = 2.69 m/s).

Accordingly, we will have: Q1 = WV1 = πr 2 · V1 = 3.14 0.052 · 2.27 = 0.00786 m 3 / s Q2 = WV2 = r 2 V2 = 3.14 0.052 · 2.69 = 0 , 00931 m^3/s

Thus, the capacity of the pipeline according to the logarithmic law is Q1 = 7.86 l/s and according to the power law we propose, Q2 = 9.31 l/s.

And this means that in our case the pipeline capacity is obtained by 18% more than by a logarithmic formula.
If we take for pipeline capacity Q1 = 7.86 l/s, obtained on the basis of calculations by logarithmic dependencies, then the same flow rate at the flow rate in the pipeline with d = 100 mm is equal to V2 = 2.69 m/s, obtained by our a power formula with some minor error can be passed through a pipeline with a diameter of d2 = 2√Q $\pi V2$ = 2√0.00786 3.14 2.69 = 0.061 m. This pipe diameter with outer diameter dн = corresponds to this internal diameter of the pipe according to GOST 10704-76 70 mm and wall thickness t = 4.0 mm.

This thickness t = 4.0 mm is assumed to be the same with the pipe wall thickness d = 100 mm (dн = 108 mm), adopted above, for ease of comparison. If we now compare the weight of one running meter of pipe with dн = 108 mm and d2 = 70 mm, then we can establish the economic effect of using our formula (26) in the calculations.

According to GOST 10704-76 1 pm pipe with dn = 108 mm weighs 10.26 kg, and from dn = 70 mm-6.51 kg. Then the saving of metal per 1 pm of pipe in the calculations using our formula (26) as compared to formula (25) is 10.26 - 6.51 = 3.65 kg. 22

Thus, if in the country for one year during the construction of closed irrigation networks, other communications, 500 km of pipes with d = 70 mm are used instead of pipes with d = 108 mm, then the total amount of saved metal will be 500,000. 3.65 = 1,825,000 kg or 1825 tons. Currently there are no stable prices for materials, they often change mainly in a big way. However, if we take the cost of 1 ton of metal at current prices (roughly) 20,000 manat, then the overall economic efficiency from using pipes with dн = 70 mm pipes instead of d = 100 mm according to our recommendations will be E = 1825 · 20000 = 36500000 manat, or 36.5 million manat. These savings are in materials and their cost. However, it is possible to calculate the savings in another embodiment, namely, by supplying water through a pipe with d = 108 mm in an amount of 9.31 l / s, using our calculation method together with 7.86 l/s using logarithmic formulas, you can increase the area irrigated crops and obtain additional agricultural products, provide a larger number of settlements, industrial enterprises with drinking and industrial water and calculate in this connection the economic effect, which will be very significant.

It is possible to carry out calculations for other pipe diameters and for each diameter to determine the economic efficiency of applying our calculation method. Thus, the application of our proposed method of calculation to determine throughput, pipelines of a closed irrigation network, shows their high reliability and, therefore, greater economic efficiency of agricultural production and other industries.

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