The method of hydraulic flushing calculation of the reservoir of the "Indian type" hydrosystem

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Abstract. One of the main problems in the operation of hydrosystems for various purposes is the silting up of the reservoir. Silting up of a reservoir leads to a decrease in useful capacity. And, of course, the hydrosystem's facility will not be able to fully perform its functions. This issue is particularly relevant for territories located in the arid zone with transboundary rivers, whose water resources are used for industries with different types of water consumption (first of all, irrigation and energy). The most common type of cleaning of silted reservoirs currently used is hydraulic flushing. The advantages of this method are the simplicity of the technological process; the disadvantage is the large consumption of water for washing, which is especially important in conditions of constant shortage of water resources. A method for washing reservoirs of Indian-type hydrounits is proposed. The technique considers the characteristic design features of a particular hydroelectric complex, operating experience, and field research data (sediment characteristics). The proposed technique was applied at the Ravatkhodzha hydro complex.

1 Introduction

Siltation of reservoirs during operation is caused by sedimentation in its bowl of sediments coming from the catchment area along with liquid runoff [1-8]. Siltation of upstream reservoirs leads to a decrease in the useful capacity and contributes to sediment flow into the flow paths of hydropower facilities [2],]6-10]. Therefore, timely and high-quality washing of the upper stream is one of the important conditions for ensuring the rational operation of the hydraulic system. The nature and intensity of this process depend on the alluvial regime of the river and the design of the water support structure.

The Ravatkhoja Dam was built in 1929 on the foothill section of the Zerafshan River. The dam with a double-sided water intake from the settling pockets belongs to the Indian type of water intake. When designing the water intake, due to the lack of the designers' experience in designing water intake assemblies for the conditions of Central Asia, carrying a lot of sediment, the Sukkur barrage on the Ind River was taken as a prototype. The Indian type of water intake is designed for frontal water intake into the pocket, and further water supply from it to the canals, excess water is discharged through the weir dam [11-14].

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In the pockets, conditions are created for the deposition of bottom sediments due to an increase in depth and width, which causes a decrease in flow rates. After the layer of alluvial deposits in the pockets exceeds the permissible limits, they are washed off through a flushing sluice downstream. During the period of flushing the pockets, the water intake into the canals is stopped, and during the water intake into the pocket, the washers are closed tightly. The washers and the channel must not be opened at the same time. A distinctive feature of the Indian type of water intake, particularly of the node under consideration, is the accumulation of bottom sediments in front of the regulator (in pockets) with their subsequent flushing into the downstream through flushing locks. The pockets act as a single-chamber sedimentation tank, the dimensions of which are chosen, taking into account the conditions for the retention of pebble, gravel, and coarse sand in front of the regulator. This was achieved by setting the thresholds of the washing locks below the thresholds of all other openings of the structure. To ensure the normal operation of the pockets, frequent flushing is provided. For the growing season of an average water content year, the coefficient of total water withdrawal to the total river discharge is up to 50%. Due to the excessive width of the front of the structure and the introduction of the headwater to the weir's crest, water intake is carried out from separate channels. At the same time, the actual water intake coefficient sometimes reaches 90% of the total water consumption in the river. The hydraulic unit has serious drawbacks that complicate its operation [11-14].

1. Penetration of a large amount of sediment into the right-bank and left-bank regulators due to the lack of necessary costs for washing pockets.
2. Inclusion of flushing locks in the passage of flood flows. At a flow rate of more than 700 m^3/s, bottom sediments of coarse fractions flow into the head regulators and further into the channels.
3. Sediments the downstream of the unit, causing flooding of the flushing locks, making it difficult to flush pockets and greatly reducing its efficiency.

Thus, sediment control is carried out by keeping them in settling pockets located in front of the channel regulator, with periodic flushing downstream. Sump pockets have a relatively small capacity and, with the amount of bottom sediment carried by rivers in the foothill zone, they naturally do not cope with their task; they require large water consumption for washing the sediments. The Indian type of all water intakes of the dam type used in Central Asia (Fergana, European with layer-by-layer washing galleries) is the most unsuccessful and problematic. This type of water intake provides non-drip water intake only at 15-20% (Fergana water intake - at 50-70%, and with the device of various thresholds, sediment walls up to 90-95%, with layer-by-layer water intake up to 30-40%). The choice of the layout was based on the simplicity of the design without considering the peculiarities of the channel process on the rivers of Central Asia. Under such conditions, the organization of rational leaching of the headwater is of paramount importance. [4,5,7,8,9,10,14] The department "Use of water energy and pumping stations" of the Tashkent Institute of Irrigation and Agricultural Mechanization Engineers purposefully deals with the issues of improving the efficiency of operation of hydraulic structures and equipment of hydropower facilities (pumping stations and hydroelectric power plants) [15,16,17,18,19]. This article proposes a method for calculating the duration of washing the headwater of Indian-type hydrosystems to improve the conditions for water intake into the supply channels of the hydroelectric power station.

2 Methods

Many prominent scientists have dealt with the issues of siltation and washing of the headwaters of water intake structures: A.M. Mukhamedov, S.T. Altunin, N.F. Daneliya, H.A. Ismagilov, V.S. Lapshenkov, R.K. Kromer, G.V. Sobolin, M. Radoan, L. Matchavariani, S. Bennett, etc. [1, 2, 3, 4, 5, 7, 20, 21]

Under operating conditions, engineers are required to quickly calculate the leaching and silting of the headwater without particularly complex computational and graphic works to at least approximately establish the duration and length of the leaching propagation to remove a given volume of sediment. The most well-known and frequently used methods, especially hydrodynamic methods (methods of I. I. Levy, M. A. Velikanov, A. I. Goncharov, K. I. Rossinsky-Debolsky, and many others) when solving even for a flat problem, lead to complex mathematical expressions that take a lot of time. The use of computer tools allows you to solve this problem [20-24].

When calculating the wash, the theory of slowly changing fluid motion in eroded channels, proposed by prof. I. I. Levy was taken as a basis [25].

Main assumptions:

1. the water flow rate within a short time interval ΔT is constant: $Q = const$;
2. the channel has a constant width and a rectangular shape; the erosion of the bottom flows evenly along the entire width of the channel with unchanged banks.
3. during the time ΔT, the curve of the free surface remains unchanged.

I levy considers the flow under slowly changing modes, i.e., the flow is considered as one dimensional and hydraulic flow elements *h*, *b*, and *v* depend only on the coordinates of the length *s* and time *t*. The problem is solved by successive approximation.

The basic equations:

1. the equation of unevenly slowly varying fluid motion.
2. the equation of channel erosion in the form given by prof. I.I. Levy [25].
3. equation of sediment discharge.

Experiments of I.I. Levy, A.M. Mukhamedov, F.Sh. Mukhamedjanov and others showed that the decisive factors determining the duration of washing [11,13,14,23,26]:

1. Value of flushing flow;
2. Lowering of the level in the section of the dam;
3. Volume of accumulated sediments;
4. Geometric coarseness of sediments and their heterogeneity;
5. Concentration of bottom sediments in the natural flow;
6. Longitudinal slope of the river bed;
7. Curvature of the supply channel, etc.

When calculating, it is almost impossible to take all these factors into account in due measure. Therefore, in different methods, many factors are taken into account through empirical coefficients. The priority in developing dependences for flushing calculation belongs to I.I.Levy [25], who obtained a formula for determining flushing duration at known flow rates, specific water flow rate, sediment volumes, and average diameter. The flushing scheme is as follows: the critical depth h_{cr} is established above the sediment ridge, and from this place upstream, the depth $h>h_{cr}$, and downstream $h<h_{cr}$. As the duration of flushing increases, the slopes become flat; at the end of flushing, they are equal, and further, they approach the slope of the uniform regime. The amount of sediment washed out by the stream from the upstream pool along the length in section 1-1 with p thickness should be equal to the volume of sediment carried out in the downstream pool through section 2-2 during the time *dt* is equal:

$$\gamma * q_{m*}\, dt = \gamma^{1} * B * p * dx, \tag{2}$$

where *B* is the width of the channel,

q_t is sediment flow rate.

Since the composition of sediments contains a significant amount of gravel-pebble fractions moving along the bottom of the riverbed, we will use the formula proposed by V. S. Knoroz as a formula for determining the flow rate of sediments [20,25]:

$$q_t=q*\gamma((u-u_0)/3.5w_0)^4*(d/h)^{1.6}, \quad (3)$$

after the transformations, we get

$$q_T = \frac{u^4}{(3.5w_0)^4}\left(\frac{d}{h}\right)^{1.6} q\gamma B = Aq\gamma Bu^4, \quad (4)$$

And,

$$A = \frac{c}{w_0{}^4}\left(\frac{d}{h}\right)^{1.6} = \frac{0.00675}{w_0{}^4}\left(\frac{d}{h}\right)^{1.6}, \quad (5)$$

The flow rate is determined by the formula:

$$u = \varphi\sqrt{2gz_0}, \quad (6)$$

Where z_0 is the difference over the sediment ridge, it is determined:

$$z_0 = 1.5*h_{кр} + p + i_н*L - h_2 - i_к*L, \quad (7)$$

Substituting expressions (5), (6), and (7) into (2) and making the appropriate integration we obtain:

$$t = \frac{1}{A_d(i_k - i_n)}\left[\ln\frac{D}{D-(i_k-i_n)L} - \frac{(i_k-i_n)L(1.5h_k - h_2)}{D(D-(i_k-i_n)L}\right], \quad (8)$$

Where

$$A_d = (\gamma/\gamma_1)AQ\varphi^4(2g)^2, \quad (9)$$

$$D = p_0 + 1.5h_{kr} \quad (10)$$

The same initial equations of I.I. Levy were used to calculate the parameters of hydraulic flushing of settling pockets. Still, the formula of I.V. Yeghiazarov [11,20,25] was used as the formula for sediment flow rate:

$$q_t = \gamma q i^{1/2} K_1 (\frac{s}{s_0} - 1), \tag{11}$$

Where
S is= γhi is drag force;
S_0= f (γ_s-γ) d_{cp} is initial attractive force;
D_{aver} is average diameter of sediment
γ_s is specific gravity of sediment;
γ is specific weight of water;
f is friction coefficient f = 0.03
i is slope of the stream bottom;
K is linear dimensionless numerical proportionality coefficient (0.015).

$$q_T = \frac{A}{h^4} - \frac{B}{h^{5/3}} , \tag{12}$$

Substituting the value of q_t into the deformation equation and taking the calculation per

$$\left(\frac{A}{h^4} - \frac{B}{h^{5/3}}\right)dt = l\gamma^1 dh,$$

unit width

of the channel, and separating the variables, we obtain

$$\frac{1}{l\gamma^1} dt = \frac{dh}{\left(\frac{A}{h^4} - \frac{B}{h^{5/3}}\right)} , \tag{13}$$

Where
$$A = \frac{\gamma^2 q^4 n^3 K_1}{(\gamma_н - \gamma) d_{cp} f}, \quad (14)$$

$$B = K_1 \gamma q^3 h, \quad (15)$$

To obtain the relationship h=f(t), it is necessary to integrate the equation within the given depths from h_1 to h_2 at a known length of the headwater l.

$$T = \frac{3l\gamma^1}{k_1 \gamma q^2 n}\left[\frac{1}{8}h^{8/3} + ch^{1/3} - c\sum_{n=0}^{\infty} \frac{h^{\frac{7n+1}{3}}}{(7n+1)c^n}\right], \quad (16)$$

Where
$$c = \frac{A}{B} * \frac{\gamma q^2 n}{(\gamma_н - \gamma) d_{aver} f}; \quad (17)$$

3 Results and Discussions

Input data for the calculation. In flushing, the value of the flushing flow and the value of the backwater are important.

The calculation consists of the following steps:

- selection of the calculated flow rate;
- determination of the possible volume of flushing;
- determination of the optimum flushing time and the required amount of water.

As input data for the calculations were used materials of field surveys, the Office of Hydrometeorological Service, and operation of the hydroscheme.

Input data for the calculations were used:

- Hydrographs of recent years (2010-2015).
- The average annual flow rate for 1999-2015 is 151.58 m^3/sec.
- The average discharge for three months of the flood (when the river carries the main amount of sediment) $Q_{average\ flood}$ = 300 -350 m^3/sec. For the low-water flow, the average flow of the other months is taken as $Q_{mean\ low\ water}$= 50 m^3/sec.

Data on the sediment regime:

According to S.T. Altunin [11, 14, 20], the volume of sediment deposited in front of the junction is 200 000-250 000 m^3. We will take 240 000 m^3 for the calculated volume (120 000 m^3 for each channel). Operating experience shows that an optimal number of leaching is 5-7 times in flood period. Let's take several leaching equal to 5, i.e., for one leaching; it is necessary to flush about 120000/5 = 24 000 m^3.

Slope of water surface I=0.0040(after silting) - 0.0048(domestic).

Hydraulic elements of such a river as Zerafshan cannot have absolute values. All parameters have conditional (averaged) values.

The total width of the supply channel is 220 m. Due to the excessive width of the channel, the flow approaches the hydroelectric complex by two channels. The right channel is along the right bank, the left channel - respectively, along the left bank. The width of both channels varies from 30 to 60 m. For calculation, we will take an empirical formula of the relation between width and flow rate: $B= 7{,}43*Q^{0.36}$;

Depending on water discharge, water depth varies H=0.6-3.0 m (right channel) and H=1.0-4.0 m (left channel). To determine the depth of the channel, we will use the graphs in Figures 1-4. The volumetric weight of sediment is 2.35 t/m^3. Volume weight of sediment in water γ= 1.35 t/m_3.

n= 0.040 is the roughness coefficient at flushing [14,21].

Algorithm and calculation program. The flushing calculation algorithm is as follows:

1. Determination of unknown geometric parameters of the channel: average depth, width.
2. Calculation of hydraulic characteristics: specific flow rate, critical depth above the sediment bed, compressed depth by the selection method from the flow formula, and hydraulic sediment size.
3. Determination of the parameters of the main equation: A_1, D, A_d.
4. Setting the flushing time with fully clarified water, with average turbidity of the flow and taking into account the curvature of the supply channel.
5.

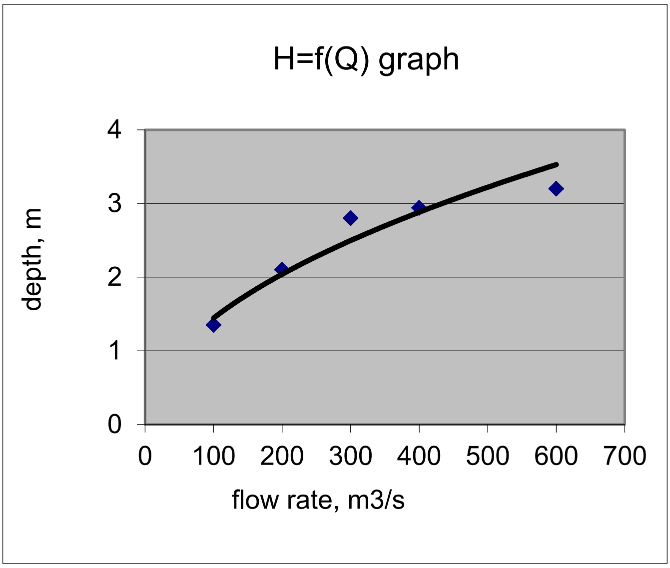
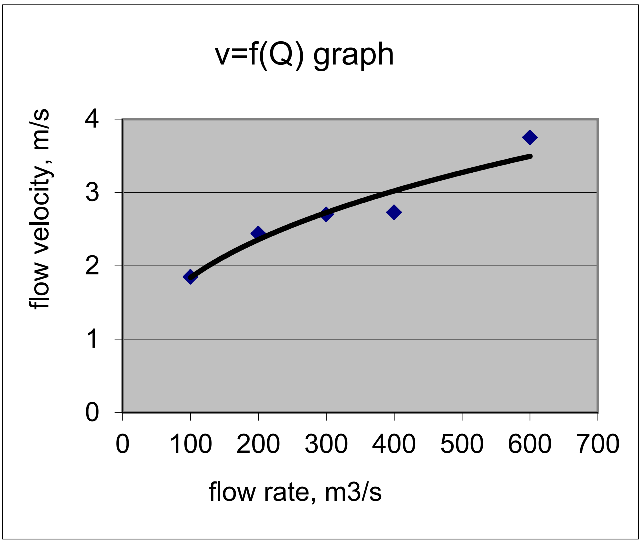


Fig. 1. Graph of the dependence H =f(Q) for the left-hand flow



Fig, 2. Graph of the dependence v =f(Q) for the left-hand flow

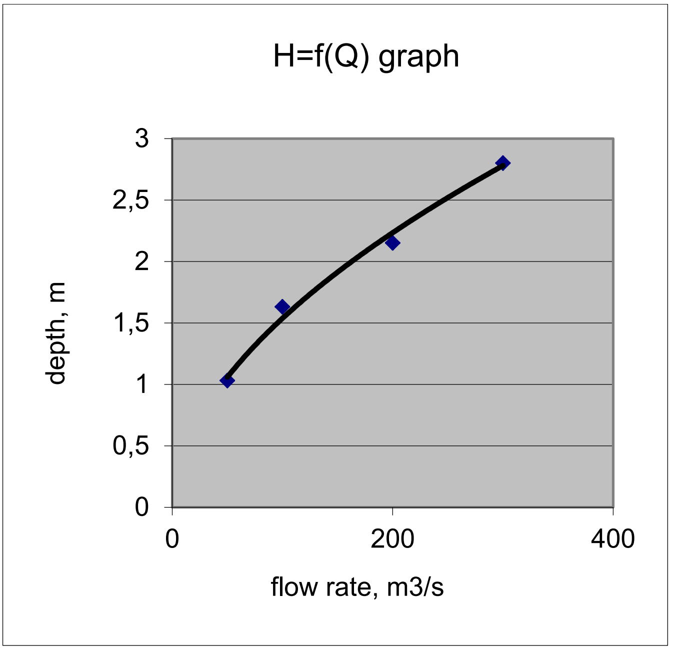


Fig. 3. Graph of the dependence H =f(Q) for the right-hand flow

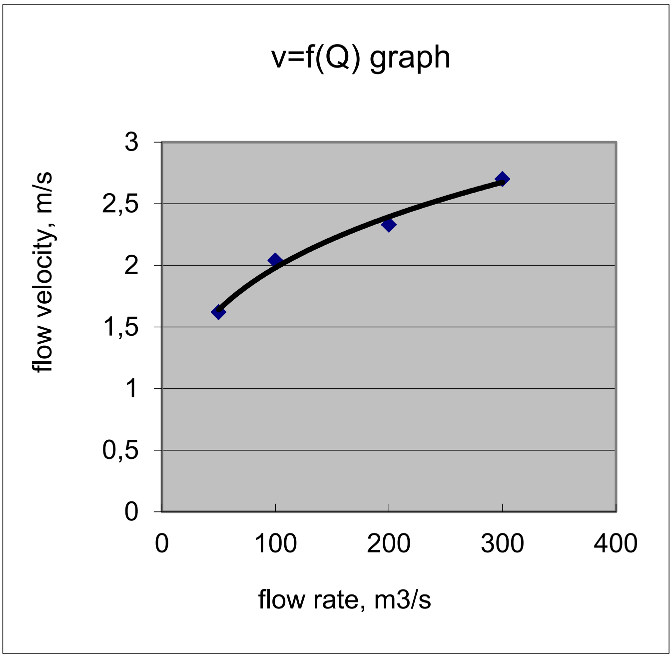


Fig. 4. Graph of the dependence v =f(Q) for the right-hand flow.

This algorithm was used to compose a program in the Turbo Pascal 7.0 language. In most methods, the water flow rate is assumed to be fully clarified. To take into account, the concentration of bottom sediments in the natural flow entering the erosion of accumulated sediments, R.K. Cromer [21] introduces a correction coefficient K_ρ into the formula for determining the flushing time, which can be calculated using the empirical formula:

$$K_P = 4(\gamma_H / \rho)^{0.22} \bullet i^{0.6}, \tag{18}$$

Also, a coefficient is entered into the formula that considers the shape of the supply channel of the K_f. This coefficient takes values in the range of 1.0 - 2.0, depending on the channel's curvature (for a straight supply channel, $K_f = 1.0$). For the calculation, we take $K_f = 1.6$.

The corrected formula for determining the washing time has the form:

$$T_3 = T_2 K_f = (K_f / K_P) \bullet T_1, \tag{19}$$

Where T_1 is the washing time without taking into account the concentration of bottom sediments in the natural flow;

T_2 is washing time, taking into account the sediment concentration, but without taking into account the shape of the supply channel;

T_3 is washing time, taking into account both the sediment concentration and the shape of the riverbed.

Table 8 shows the results of the calculation according to the proposed program for one channel. From the table with the results of calculating the flushing time of one channel, it can be seen that for flushing one supply channel at a conditional flood flow rate of 350 m^3/s, the required time for flushing will be 5.54 h (or 9.08 h, taking into account the load of sediments of the natural flow of about 2- 3 kg/m^3, or 14.53 h, taking into account the curvature of the supply channel). The required amount of waste water will be 6978.65x10^3 m^3 (or (11440.8 or 18307.8)x10^3 m^3). The larger the flushing flow rate, the less time spent on flushing a given volume of deposits.

Table 1. Results of calculation of flushing time of one duct by the proposed methodology.

Q, m^3/s	H m	B m	H_{kr} m	A^1 x10^5	D	T, sek	T, hour	T_2, hour	T_3, hour	W, x10^3, m^3	W_2, x10^3, m^3	W_3, x10^3, m^3
100	1.25	39.5	0.92	5.421	1.4485	48489	13.46	22.08	35.34	4848.0	7948.8	12722.4
150	1.65	45.7	1.10	4.102	1.465	39399	10.944	17.95	28.71	5909.9	9693.0	15503.4
200	2.1	50	1.24	3.362	1.488	32999	9.16	15.02	24.04	6599.8	10814.4	17308.8
250	2.45	55	1.36	2.883	1.5151	27908	7.75	12.7	20.3	6977	11430	18270
300	2.81	59	1.47	2.547	1.5434	23593	6.55	10.7	17.19	7077.9	11556	18565.2
350	2.87	62	1.57	2.293	1.5715	19939	5.54	9.08	14.53	6978.65	11440.8	18307.8
400	2.95	65	1.67	2.098	1.6012	16603	4.61	7.56	12.10	6641.2	10886.4	17424
450	3.10	68	1.75	1.938	1.6288	13755	3.82	6.26	10.00	6189.75	10141.2	16200
500	3.20	71	1.83	1.804	1.6555	11222	3.11	5.11	8.17	5611	9198	14706
550	3.25	73	1.91	1.684	1.6769	9203	2.55	4.19	6.71	5061.65	8296.2	13285.8
600	3.30	75	1.98	1.590	1.7039	6986	1.94	3.18	5.09	4191.6	6868.8	10994.4
650	3.40	78	2.05	1.504	1.7269	5133	1.42	2.33	3.74	3336.45	5452.2	8751.6
700	3.50	80	2.11	1.431	1.7513	3304	0.99	1.50	2.41	2312.8	3780	6073.2

4 Conclusions

1. To ensure guaranteed water intake, it is advisable to periodically flush the headwater through the channels. First, the right-bank, then the left-bank channels with a flow rate of at least 250-300 m^3/s. It is advisable to repeatedly perform flushing with jerks, i.e., several successive rises and decreases in water levels in the pool. For an approximate appointment of the time and other parameters of flushing, can use a computer program for calculating the flushing time developed based on the basic differential I.I. Levy's equations and the formula for the sediment rate of V.S. Knoroz, the dependence B=f(Q) and the graphs of the dependence H=f(Q) and v=f(Q) for different channels of the supply channel obtained from the calculation of sediment transport.
2. Experience of operation of the dam for 80 years shows that there are often situations when for one reason or another (especially in dry years due to the shortage of washout flow) is impossible to implement complete washing of the upper pool in such cases, it is

often necessary to perform partial flushing of the upper pool. The remaining volume of sediments rinse under more favorable conditions. According to the proposed program, the data of previous experiments and the results of calculations show that the intensity of the sediment washout first increases sharply. Towards the end of the wash, the intensity of the wash fades. Usually, up to 60-70% of the total volume of deposits can be washed off in half of the washing period.

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