

# Design of unsteady filtration in the upper water-resistant prism of earthfill dams given horizontal drainage layers in the base

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**Abstract.** The article presents the results of the study of unsteady filtration in the upper water-resistant prism of dams given horizontal drainage layers in the base. Design methods have been developed for unsteady filtration in the upper water-resistant prism of dams, given horizontal drainage layers in the base. Research main results have been presented for the number of schemes of drainage in the upper water-resistant prism of dams. Experimental research has been carried out to check the validity of accepted assumptions when deriving relationships for the filtration design of earthfill dams with horizontal drainage layers and assess the accuracy of the solutions obtained. Experimental research on processes under consideration for physical models has shown a good resemblance with the results obtained from numerical calculations. This makes it possible to confirm the validity of chosen design schemes and accepted initial assumptions in numerical studies.

## 1 Introduction

The main tendency towards the construction of water-retaining hydraulic structures in a region under consideration, as well as in domestic and foreign practice, is the predominance of the construction of dams from soil materials. At the same time, low-permeability materials are being widely used as filtration control elements (screens and cores) and in the body of water-resistant prisms of dams [1-4].

Concerning the stability of dams from low-permeability soils, the flooded slopes of upstream water-resistant prisms of earthfill dams and water reservoir banks pose the greatest danger in cases of rapid dropping (drawdown) of upstream water level. In the

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operation practice of water reservoir hydrosystems, these cases are not uncommon and, as a rule, are caused by uneven loading of hydropower stations, periodic water intake into diversion systems, a pre-flood drawdown of the water reservoir, various emergencies, etc. [5,6,7].

In dams under consideration, the mentioned conditions lead to the significant lag of phreatic line of filtration flow from upstream water level and, as a result, the occurrence of dangerous hydrodynamic water pressures directed in upper water-resistant prisms mainly towards the reservoir, and their neglect in design can lead to failure of slope stability [8,9,10,11,12,13]. In this case, the stability of slopes is set depending on the extent to which the upstream water level changing rate differs from the rate of change of phreatic line position within the structure [8,9,12]. Slope stability of dams must be ensured for various positions of reservoir water level.

To exclude or, rather, noticeably decrease arising hydrodynamic forces in the body of the dam under given circumstances and to increase the stability of flooded slopes, various drainages, including horizontal drainage layers, are used, which are arranged in tiers along the entire height of the dam at given distances from each other. The justified choice of location scheme of so-called unloading drains and their correct structural solution must significantly increase upstream slope stability [14,15,16]. Besides, the installation of such drains significantly decreases pore pressure in low-permeability soils, makes it possible to erect dams of overwatered soils, contributes to the completion of the main part of deformation during the construction period, and opens up opportunities for the increase of the rate of structure erection, i.e., reduction of construction time [17,18,19].

However, despite the relatively wide use of unloading drains, their placement issues, filtration design methods, validation of structural solutions, etc., remain poorly studied. As a result, there are cases of unreasonable structural solutions of drains, ineffective and incomplete use of their possibilities, etc.

Therefore, in the context of the increase of the scales of earthfill dam construction, especially with the account of wide application of low-permeability materials for their construction, the problem with the solution of tasks of unsteady filtration in earthfill dams with drained and undrained upper water-resistant prism with the account of filling and draining regime of water reservoirs for various hydrogeological conditions are relevant for design and construction of dams, as well as for evaluation of stability during their operation [20,21].

The issues of using drainage devices of various designs in dams with unsteady filtration are mentioned in the works of various researchers: A.O.Sheetopal, E.E.Lysov, and A.A.Sinyavskaya [22], Y.B.Patel, A.B.Krishai, K.L.Arora [25], M.R.Bakiev [26], S.N.Babakaev [27], H.Fayziev, Sh.A.Rahimov, Sh.H.Baymatova [29,30,31].

A.O.Sheetopal conducted research with the EGDA method for various designs of drains installed to increase the stability of the upstream slope of the pressure dam to increase the stability of the upstream slope of the pressure dams of the PHEs reservoir.

According to the results of studies carried out in [22], it is noted that horizontal drainage layers in the upstream wedge, compared to other considered devices, significantly increase the stability of the upstream slope of the dam under a rapid drop of upstream water level.

G. Schnitter and J. Zeller [25] (on a slotted flume), E. Reinius [23] (on EGDA), and J. Brauns [24] (analytically) studied the upper wedge of a dam with a core and horizontal drainage under a rapid drop of reservoir water level. The results of the authors' research have shown the high efficiency of horizontal drainage layers to increase the stability of the upstream slope.

## 2 Methods

Below there is a revision of a dam with a single-tier perfect horizontal drainage layer at the base, i.e., drainage, which cuts through the entire prism in the horizontal direction from the slope line to the core edge.

When studying filtration processes, the following assumptions are made:

- 1) the soil of the dam is considered to be a non-deformable, homogeneous, and isotropic porous medium;
- 2) the movement of groundwater in the body of the dam is laminar, complying with Darcy's law;
- 3) capillary rise of water in soil pores and evaporation from the depression surface is neglected;
- 4) the base is considered to horizontal and waterproof.

For the dam scheme shown in Fig. 1, the water movement to the horizontal drain due to a drop in upstream water level is almost vertical. This makes it possible to calculate the drop in the phreatic line of filtration flow at the upstream prism of dams under a one-dimensional problem for individual sections.

Under these conditions, to decrease the rate of groundwater level dropping in the upstream prism of dams with horizontal drainage, the following equation can be written [4]:

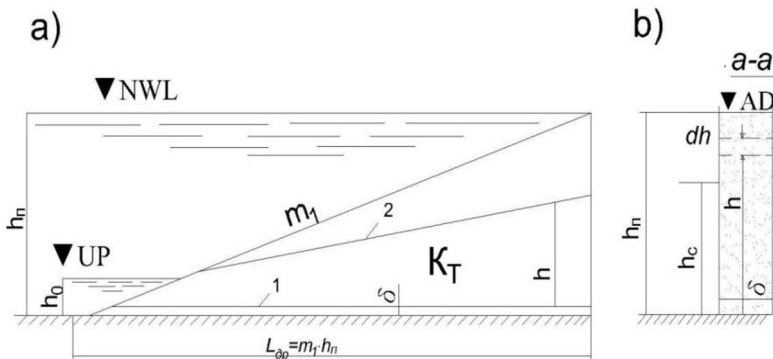
$$\frac{\partial h}{\partial t} = -\frac{k_t h - h_c}{\mu h - z_n} \tag{1}$$

where  $h$  is depth of water in the body of the dam;  $h_c$  is depth of water in the reservoir after dropping;  $k_t$  is permeability coefficient of soil in the body of the dam;  $\mu$  is coefficient of storage of soil in the dam body;  $Z_n$  is the distance from the dam bottom to an upper surface of drainage under consideration, which in one layered drainage is equal to drainage thickness.

To solve equation (1), it is necessary to set the law of change of the rate of reservoir water level. In case of the linear law of water level dropping at a constant rate, equal to  $v=h_n/t$ , the depth of the water during the time interval under consideration will be equal to the following:

$$h_c = h_n - v \cdot \Delta t = h_n(1 - \frac{t_c}{t}) \tag{2}$$

Where:  $h_n$  is depth of water in the reservoir at normal water level;  $t$  is time of full dropping of water level in the reservoir.



**Fig 1.** Scheme to design of filtration flow in upper prism of dam.1 is horizontal drainage, 2 is

depression surface of filtration flow when decreasing upstream water level.

The precise solution of equation (1) with analytical methods is related to great mathematical difficulties. Therefore, we shall limit ourselves here with the solution based on using the finite differences method [5]. Then the equation (1) shall be rewritten in the following form:

$$\frac{h_{j+1}-h_j}{2\Delta t} = \frac{k_T}{\mu} \frac{h_{j+1}-h_c}{h_n-Z_n} \quad (3)$$

$$h_{j+1} - h_j = -\frac{2\Delta t k_T}{\mu} \frac{h_{j+1}-h_c}{h_n-Z_n} \quad (4)$$

$$h_{j+1} - h_j = -\frac{2\Delta t k_T}{\mu} \left( \frac{h_{j+1}}{h_n-Z_n} - \frac{h_c}{h_n-Z_n} \right),$$

$$h_{j+1} - h_j + \frac{2\Delta t k_T}{\mu} \frac{h_{j+1}}{h_n-Z_n} = \frac{2\Delta t k_T}{\mu} \frac{h_c}{h_n-Z_n},$$

$$\left( 1 + \frac{2\Delta t k_T}{\mu(h_n-Z_n)} \right) h_{j+1} - h_j = \frac{2\Delta t k_T}{\mu} \frac{h_c}{h_n-Z_n},$$

$$\left( \frac{\mu(h_n-Z_n)+2\Delta t k_T}{\mu(h_n-Z_n)} \right) h_{j+1} - h_j = \frac{2\Delta t k_T}{\mu} \frac{h_c}{h_n-Z_n},$$

$$\frac{\mu(h_n-Z_n)+2\Delta t k_T}{\mu(h_n-Z_n)} h_{j+1} = h_j + \frac{2\Delta t k_T}{\mu} \frac{h_c}{h_n-Z_n} \quad (5)$$

Solving equation (5) concerning  $h_{j+1}$ , we shall obtain the formula, which determines the position of the phreatic line in the vertical section under consideration with linear droppage of reservoir water level:

$$h_{j+1} = \frac{\mu(h_n-Z_n)}{\mu(h_n-Z_n)+2\Delta t k_T} h_j + \frac{2\Delta t k_T h_c \mu(h_n-Z_n)}{[\mu(h_n-Z_n)+2\Delta t k_T] \mu(h_n-Z_n)} \quad (6)$$

For further analysis of processes under study, it is convenient to take the generalized criteria of unsteady filtration in the form of the ratio of dam body coefficient of permeability  $K_t$  to the rate of droppage of phreatic line  $\vartheta$  multiplied by the coefficient of storage  $\mu$ , i.e.

$$K_v = \frac{K_T}{\vartheta \mu} \quad (7)$$

which, as seen, is a dimensionless parameter.

### 3 Results and Discussion

To determine the position of the phreatic line of filtration flow design program is created in C#. Diagram was constructed based on the design results, which allows determining phreatic line position, which complies with a determined moment of water level droppage in the reservoir, using known values of reservoir water level dropping rate, coefficients of permeability, and storage of dam body soil.

With a linear law of reservoir water level droppage the ordinates of the phreatic line were determined depending on  $K_v$  parameter (7), which varies between 0.1 and 10. These change limits of  $K_v$  correspond to the dam body soil coefficient of permeability from 10 to

1·10<sup>-4</sup> m/day and reservoir water level dropping the rate from 0.5 to 40 m/day.

The diagram to determine the phreatic line ordinates for this scheme of the cross profile of the dam with linear droppage of reservoir water level is given in Figure 2. To find the ordinates of the phreatic line from the diagram, for example, in section a-a,  $K_v = Kt/\mu v$  is calculated first, then  $t = h_n/v$  and  $t_1 = \Delta t/t$  are determined using the value of  $\Delta t$ . Then the reduced values from values of  $h/h_n$  are found using  $K_v$  and  $t_1$ , and finally, from the last relationship,  $h$  is determined by simple calculations. For instance, in the case of  $K_v = 1$  and  $t_1 = 0.6$ , the relative height of the phreatic line in section a-a will equal to  $h/h_n = 0.83$ , as it is shown with arrows in Figure 2. Using the diagram, one can determine the ordinates of the phreatic line for any other section. The time  $\Delta t$  is counted so that reservoir water depth after each time  $\Delta t$  shall be equal for these sections.

For example, taking the height at section b-b equal to  $h_n(b-b) = 0.8h_n(a-a)$ , the following should be determined first:

$$t_{(b-b)} = \frac{h_n(b-b)}{v} = \frac{0.8}{1.0} = 0.8$$

and then we determine the time

$$\Delta t_{b-b} = \Delta t_{a-a} - [t_{a-a} - t_{b-b}] = 0.6 - (1 - 0.8) = 0.4$$

and

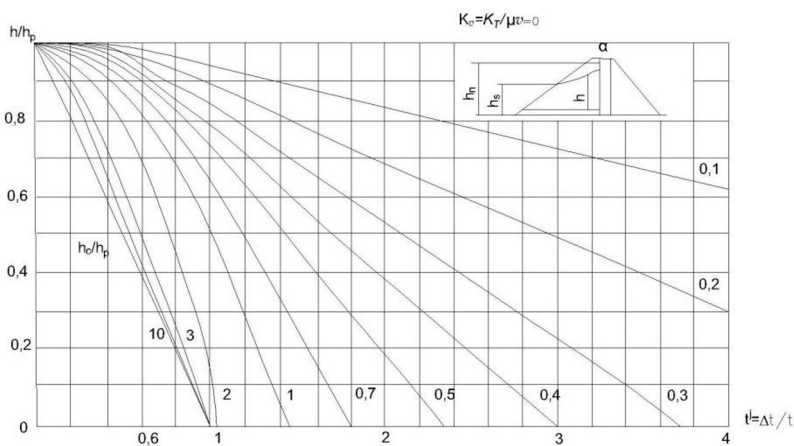
$$t^1 = \frac{\Delta t_{b-b}}{t_{(b-b)}} = \frac{0.4}{0.8} = 0.5$$

and, hence, the relative height of the phreatic line will be equal to

$$\frac{h(b-b)}{h_n(b-b)} = 0.86$$

from where it is easy to determine  $h$  for section b-b.

The diagram in Figure 1 shows that the rate of droppage of the phreatic line significantly depends on  $K_v$  parameter, with the increase of which the rate of droppage of the phreatic line significantly increases



**Fig. 2.** Diagram of water level droppage in upper prism of dam with linear decrease of reservoir water level in section a-a.

From the drainage scheme, the dam body at  $K_v < 0.1$  stays fully saturated. At  $K_v = 1$  the lag of the phreatic line from reservoir water level at time  $t' = 1$  is equal to  $0.52h_n$ , and at  $K_v = 5$ , the horizontal drainage can almost completely drain the dam body, and the lag of the phreatic line is equal to only  $0.02h_n$ .

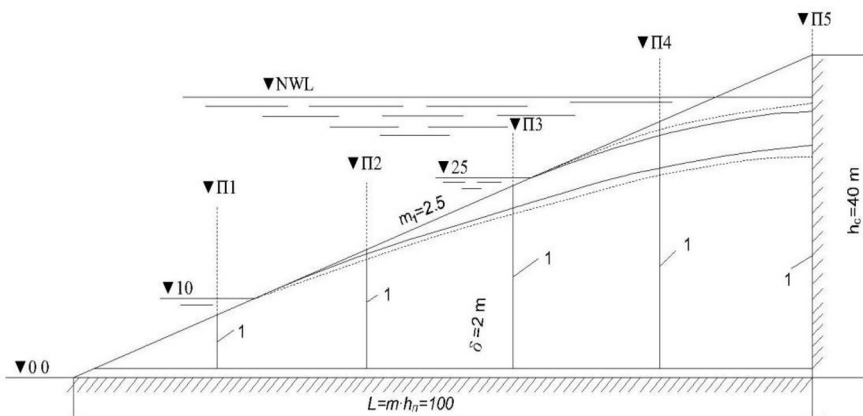
To verify the validity of accepted assumptions in finding the relationships for filtration design of the upper wedge of a dam with horizontal drainage and assess the degree of accuracy of obtained solutions (6), experimental studies were carried out.

The experiments were held in a filtration flume. The flume is a rectangular-shaped metallic reservoir with sizes  $0.8 \times 0.12 \times 4.0$  m. The front wall is glazed. The ends of the flume have spillways, which are intended for water supply. Piezometers are installed in the dam's body to monitor the filtration process. For dams with one perfect drainage, piezometers are used at the base, made of a fine metal mesh, the size of which ensures that it is not clogged with sand, attached to the front wall of the flume. In the study of unsteady filtration in the body of the dam, its upstream was filled with water from a pressure tank installed next to the flume. On the transparent side of the tray, a  $10 \times 10$  cm coordinate grid was applied vertically and horizontally. The grid makes it possible to take readings of the configuration and dimensions of the model elements and the position of the water levels in the piezometers. All measurements of experimental data are tied to it.

Dune sand was used as the dam model material in the flume. Gravel-pebble soil of 10-20 mm was used as a drainage material. The soil was laid in the flume in 2 cm thick layers by tamping, with a moisture content of 5% and a density of  $1.55$ - $1.6$   $t/m^3$ . The permeability coefficient of soil at this density was initially determined by filtration studies and was equal to 5-10 m/day. The coefficient of storage of soil was defined as the ratio of the volume of water flowing freely from a certain volume of soil under the action of gravity for a certain period to the entire volume of this soil, initially saturated with water.

The experiments were carried out as follows. Water was poured upstream of the water reservoir, being modeled to the level corresponding to the normal water level. The water level was maintained at this level until the phreatic line was established. After that, the faucets ensuring the drawdown water lever from normal water level were turned on.

The upstream water level kept dropping, and when it reached  $0.75h_n$ ,  $0.625h_n$ ,  $0.5h_n$ ,  $0.375h_n$ , and  $0.25h_n$ , the readings of the phreatic line position were taken from piezometers. Then upstream water level was set to  $0.875h_n$  again, and the process was repeated with different rates of reservoir water level droppage.



**Fig. 3.** Comparison diagram of phreatic line position, obtained with theoretical relationships (6) and using the model in soil flume. ----- is theoretical curves; \_\_\_\_\_ is experimental curves

Modeling was carried out for a dam with a core and single-layer horizontal drainage at the dam's base. 5 experiments were carried out with slope rates  $m1 = 2.5; 3.0$  at  $K_v=0.67; 0.89; 1.18; 1.44; 1.8$ . Fig. 3 shows the positions of the phreatic lines corresponding to  $625h_n$  and  $0.25h_n$  for  $K_v=0.89$ ; where  $K_v$  is determined from relationship (7). For the same values of  $K_v$ , phreatic lines were calculated using the formula (6). A comparison of the phreatic lines obtained by calculation and experimental methods is given in Table 1.

**Table 1.** Comparison of the results of design of phreatic lines with experimental data with  $K_v=0.89$

$hc$ "M"	Piezometers					
	Experimental	Design	Discrepancy, %	Experimental	Design	Discrepancy, %
35	35	35	0	35	35	0
30	30	30	0	30	30	0
25	25	25	0	25	25	0
20	20	20	0	23.75	23.52	-0.97
15	15.8	15.92	+0.76	22.5	22.44	-0.27
10	15.5	14.96	-3.48	21.0	20.4	-2.86

**Continuation of table № 1.**

$hc$ "M"	Piezometers					
	Experimental	Design	Discrepancy, %	Experimental	Design	Discrepancy, %
35	35	35	0	35	35	0
30	31.4	31.8	+1.27	34.3	34.63	+0.96
25	30.8	31.2	+1.30	33.6	33.69	+0.27
20	29.7	29.92	+0.74	32.5	32.186	-0.966
15	28.5	27.84	-2.32	31.4	30.13	-4.05
10	26.0	25.6	-1.54	28.8	27.48	-4.58

As can be seen from these tables, the maximum discrepancy between the experimental and calculated data does not exceed 4.58%.

## 4 Conclusions

➤ Based on the finite differences method, an algorithm for solving the problem of unsteady filtration and instructions for the program in dams with a core and horizontal reservoir drainage at the base have been developed.

➤ It has been established that horizontal reservoir drainage in the upper wedge significantly changes the direction of streamlines, while the direction of hydrodynamic forces is close to vertical. Due to this, the stability of the slope of the dams increases with a rapid decrease in the water level in the reservoir.

➤ Experimental studies of the processes under consideration on physical models showed a good resemblance of experimental data with the results of numerical calculations. This makes it possible to affirm the validity of the chosen design schemes and the accepted initial assumptions in numerical studies.

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