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Dynamically stable sections of large soil canals taking into account wind waves

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Abstract. The article discusses the dynamic stability of the cross-sections of watercourses of transporting sediments in the conditions of stationary and non-stationary water flow. The results of special laboratory studies to study the set task are presented. Based on the dependence of S.Kh. Abalyants, modified dependences are obtained for calculating dynamically stable cross-sections of watercourses with stationarity and nonstationary of the water flow.

1. Introduction

The solving complexity of dynamic stability at the present stage is due to the fact that in order to improve the operation of canals, in some cases it is necessary to design hydraulic structures for different regimes of water flow [1, 2, 5, 8, 10, 11]. In some cases, the speed of the water flow exceeds the non-blurring value in relation to the underlying watercourses. This circumstance leads to the emergence of a number of negative problems associated with taking into account the transporting ability of streams in deformable watercourses and predicting deformations of their canals [1, 2, 3, 4, 9, 12, 16, 17, 18, 19, 22]. Also, the increased requirement for the design of large canals leads to the need to take into account unsteady currents (wind waves, ship waves, long waves, etc.). These currents significantly affect the erosion processes occurring in the canals and lead to a change in the transporting ability of the flows [23 – 25]. The question of the role of the effects of wind waves in the formation of the coastal slopes of the canals before the breaking of the waves remains poorly understood to date.

The following article analyzes the development of a simplified scheme for calculating a dynamically stable cross-section of large canals operating when wind waves are superimposed on currents with its subsequent verification by the author's experimental data. In what follows, we will assume that the amount of transported sediment and the flow rate of water in the canals determine the width, average depth and slope of the canal, and the shape of the canal's cross-section determines the distribution of tangential stresses in the canal, arising under the influence of stationary and unsteady currents. Therefore, the width, average depth and slope of dynamically stable canals are calculated based on the physical approach [25 – 29], and the cross-sectional shape - using the method of pulling force (tangential stresses) [30 – 34].



It is alleged that the issues of choosing a dynamically stable canal cross-section are of great interest. Thanks to the scientific works of researchers S.Kh. Abalyants, V.S. Altunin, V.S. Borovkov, Yu.A. Ibad-zade, I.F. Karasev, Kokhlin, M.A. Mikhalev and others in this direction for the conditions of stationary flows, dependences are obtained, which, when calculated, give satisfactory results. From the analysis of the existing calculation methods, the obtained experimental data showed their close agreement with the recommended calculation methods of various authors. Proceeding from this circumstance and taking into account the complexity of the question of the formation of the canal canal under conditions of non-stationary flow, we have chosen the dependence of S.Kh. Abalyan. We take this dependence as a basis for calculating a dynamically stable cross-section of canals under conditions when wind waves are superimposed on currents.

2. Method

Now we come to the mathematical solution of the problem. The problem of the shape of the canal of limiting equilibrium was solved in the calculation method of Forchheimer-Lane [13]. Here, the following assumptions are made: it is assumed that with approaching the shoreline, the water flow rate decreases to zero and the action of the water displacing the soil particle $\gamma h J$ is taken proportional to the local pulling force, where h is the local depth, J is the longitudinal slope of the water flow. In this case, the condition of the limiting equilibrium of particles on the slope leads to the equation

$$\frac{h}{h_m} = \sqrt{1 - \frac{(dh/dy)^2}{tg^2 \varphi_0}} \quad (1)$$

the integral represented by the cosine

$$h = h_m \cos\left(\frac{tg \varphi_0}{h_m} y\right) \quad (2)$$

where h_m - maximum water flow depth; y - transverse coordinate; φ_0 - the angle of internal friction of the bottom soil.

Comparison of the calculated values according to (2) of the transverse profiles of river canals and canals with the measured data showed [1] that in many cases the cosine shape of the canal is not characteristic of canal flows. This discrepancy is explained by the fact that the derivation of (2) does not take into account the real distribution of water flow velocities over the cross section. To take this distribution into account, it was undertaken in [1-2]. As a result of the analysis carried out by S.Kh. Abalyants, an equation was obtained in the following form:

$$\left(\frac{h}{h_m}\right)^{2\alpha} = \sqrt{1 - \frac{(dh/dy)^2}{tg^2 \varphi_0}} \quad (3)$$

where α - coefficient taking into account changes in speed from maximum to wall, $\alpha = 0,25$. As a result of the analysis of the actual cross-sections of canals in the regime of static and dynamic stability, S.Kh. Abalyants put forward a hypothesis about the possibility of using dependence (3) to calculate the cross-sections of canals of canals transporting sediments.

To do this, in (3), instead of the angle of repose, (φ_0) its reduced value is introduced:

$$\varphi_g = \frac{\varphi_0}{1,65} \quad (4)$$

where φ_g and φ_0 - are the angles of internal friction of the soil at dynamic and static stability.

We apply the approach of S. Kh. Abalyants for dynamically stable canal canals operating under the influence of wind waves. To do this, we will divide the slope of the canal conditionally into two zones, the first of which corresponds to the breaking zone of wind waves $h < 1.28h_v$, where h_v is the height of

wind waves in the breaking zone. In the first zone, the bottom profile acquires features similar to the beach relief and is calculated according to the method [14]. In the second, deeper zone, we will assume tangential stresses from the transverse coordinate caused by wind waves. In the future, we will assume that under the influence of wind waves, the destruction of the coastal slopes occurs, which will find their expression in the formation of a flatter bottom profile in this zone. The process of flattening of the slopes occurs under the influence of wind waves. To quantitatively account for this process in equation (3) we introduce a new value of the angle of repose, which has the following form, taking into account the dimension:

$$\sqrt{tg^2\varphi_g} - K \frac{u_{*m}^2}{(vg)^{2/3}}, \quad (5)$$

where φ_g - determined from (4); V - kinematic viscosity coefficient; S - relative density of a given soil; u_{*m} - dynamic velocity arising at the bottom of the canal flow under the influence of wind waves, which is determined by the Egleson formula [14]:

$$u_* = \left(\frac{8\pi v H^2}{\tau^3 s h^2 2\pi \frac{d_{cp}}{\lambda}} \right)^{2/4} \quad (6)$$

Where H is the mark of the wave bottom; τ - wave period; λ is the wavelength.

d_{cp} is the average diameter of bottom sediments; h_v , λ , τ - height, length and period of wind waves, determined for average values of depth and current velocity according to the method [14]. From expression (5) it can be seen that the coefficient K remains unknown. In order to study the hydrodynamic stable sections under the influence of wind waves and to determine the coefficient K , experimental studies were carried out on the canal flume of the laboratory of the Karshi engineering-economics institute.

In the hydraulic flume, a model of a trapezoidal canal of incoherent ($d_m = 0.67$ mm) soil was built under the conditions of slopes $m = 2$, $m = 3$, $m = 3.5$.

In the hydraulic flume, a model of a trapezoidal canal of incoherent ($d_m = 0,67$ mm) soil was built in the conditions of slopes laying $m = 2$, $m = 3$, $m = 3,5$.

The experiments were carried out at such rates of the water flow, in which they exceeded the values corresponding to the non-erosion rates. The experiments were carried out to study the deformations of the coastal slopes of the canals in the conditions of stationary and non-stationary (superposition of waves on currents) currents. In the experiments, the nonstationarity of the flow was reproduced with the help of a portable wave generator of the rotating shield type.

3. Results and Discussion

The obtained experimental data of the conducted experiments are shown in Table 1. Deformations of the cross-sections of the model canal under the conditions of stationary and unsteady flows after the experiment are shown in Fig. 1. Let us consider the process of deformations of the canal cross-sections. Under stationary conditions of currents, an excess of current velocities of non-eroding velocities leads to erosion of the coastal slopes in their upper part and the subsequent deposition of eroded sediments in the zone where the slopes meet the canal bottom, and flattening of the slopes is observed, which reach their maximum in their deepest part. In these zones of maximum flattening of the bottom, zones of elevated dynamic flow rates are confined.

Table 1. Summary of Findings from Experimental Studies

№ experience	Q l/s	h_m m	v_m m/s	d_m mm	T hour	h_b m
3	25.49	0.20	0.25		II	
3 ^b	25.49	0.20	0.25		5	0.01
I	30.45	0.20	0.29	0.67	8	
I ^b	30.45	0.20	0.29		3.5	0.01
2	39.91	0.215	0.35		5	
2 ^b	39.91	0.215	0.35		1.5	0.008
5	18.5	0.161	0.24		1.0	
5 ^b	18.5	0.161	0.24		6	9.01
4	22.47	0.170	0.29	0.67	12.7	
4 ^b	22.47	0.170	0.29		7.75	0.009
6	28.0	0.176	0.35		6.75	
6 ^b	28.0	0.176	0.35		5.70	0.008
8	15.88	0.134	0.27		II	
8 ^b	16.0	0.134	0.27		6.75	0.01
7	20.0	0.143	0.31	0.67	6.7	
7 ^b	20.0	0.143	0.31		5.5	0.008
9	23.0	0.146	0.345		3.6	
9 ^b	23.0	0.146	0.345		2.65	0.009

Table 2. Summary of Findings from Experimental Studies

№ experience	h_v m	τ s	λ m	$(tg\varphi_g)_{meas}$	$\frac{6h_m}{B}$
3				0.548	0.553
3 ^b	0.19	0.85			
I				0.602	0.583
I ^b	1.19	0.85	2		
2				0.658	0.525
2 ^b	1.23	0.85			
5				-	-
5 ^b	1.07	0.85			
4			3	0.449	0.456
4 ^b	1.10	0.85			
6				0.385	0.414
6 ^b	1.12	0.85			
8				0.343	0.386
8 ^b	0.97	0.85			
7			3.5	0.333	0.358
7 ^b	1.01	0.85			
9				0.320	0.331
9 ^b	1.02	0.85			

Under conditions of superposition of waves on the flow, the process of flattening of the transverse profile of the bottom increases. In these profiles, the coastal slopes can be divided into three zones. In the deep zone, sediments of bottom sediments are observed, which are carried out from the middle and upper parts of the slopes. In the upper part of the coastal slope corresponding to the zone

maximum transformation of waves in shallow water, relief forms are observed, bearing signs of the beach - insignificant values of the bottom slopes in the waterside part. In the middle part of the slopes, bottom erosion is observed, which, in combination with the accumulation of sediment in the lower part, causes a general flattening in the outer part of the coastal slopes.

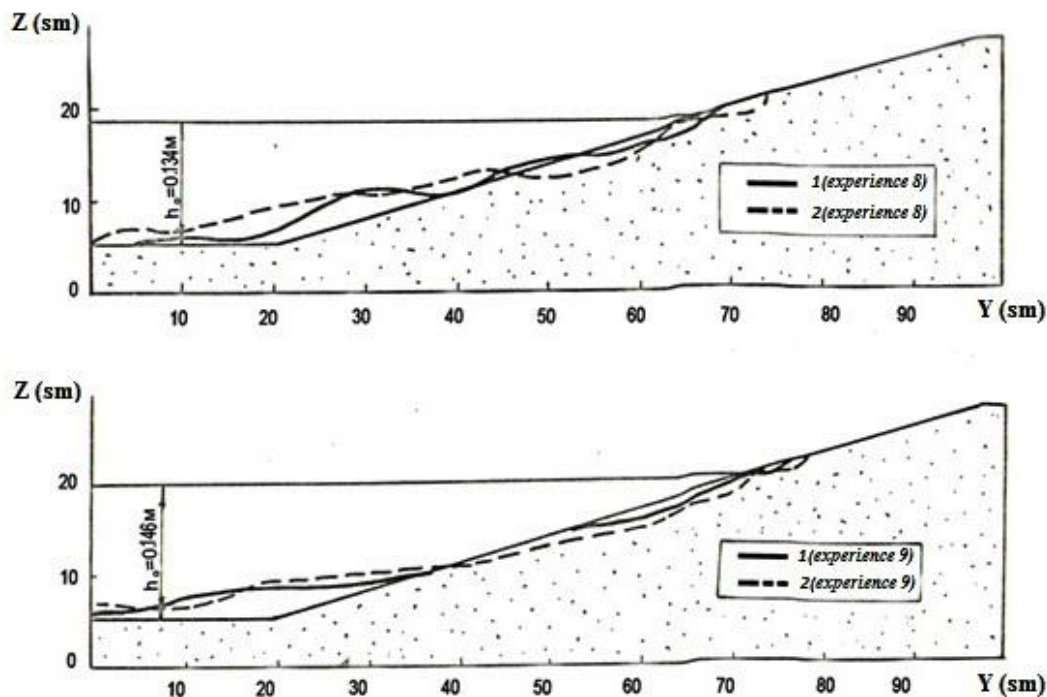


Figure 1. Formation of the canal cross-section in a stationary flow (1) and when waves are superimposed on the flow (2)

Determine the empirical coefficient K in (5). We integrate equation (3) at $\alpha = 0,25$ and the boundary condition at $h = 0, y = 0$. After mathematical transformations, we have:

$$\frac{h}{h_m} = 1 - \left[1 - \frac{\sqrt{tg^2 \varphi_g - K \frac{u_{*m}^2}{(vg)^{2/3}}}}{2h_{mb}} y \right]^2 \tag{7}$$

where h_{mb} is the maximum flow depth in the canal when waves are superimposed on the flow.

From the dependence (7) we determine the empirical coefficient K :

$$K = \frac{(vg)^{2/3}}{u_{*m}} \left[tg^2 \varphi_g - \frac{4h_{mb}^2}{y^2} \left(1 - \sqrt{1 - \frac{h}{h_{mb}}} \right)^2 \right] \tag{8}$$

To determine K from relationship (8), you first need to determine the value of the angle of internal friction of the soil φ_g with dynamic stability. To do this, we will make the following mathematical transformations. Taking in (3) $u_{*m} = 0$ (stationary flow), we determine the average depth of a dynamically stable canal:

$$h_{cp} = \frac{2h_m}{B} \int_0^{B/2} \left[1 - \left(1 - \frac{tg \varphi_g}{2h_m} \right)^2 \right] d_{cp} = \frac{tg \varphi_g}{4} B \left(1 - \frac{tg \varphi_g B}{12h_m} \right) \quad (9)$$

From the dependence (7.4.56) $u_{*m} = 0$ when we find:

$$h_m = \frac{Btg \varphi_g}{4}. \quad (10)$$

Substituting expressions (10) into (9.4) we obtain:

$$tg \varphi_g = \frac{6h_{cp}}{B} \quad (11)$$

where h_{cp} and B are determined according to the methodology described in [13] in accordance with the given water flow rates and the transporting capacity of the designed canal.

Substituting (11) into (7) we have a transverse profile of the bottom of a dynamically stable canal at a steady flow:

$$\frac{h}{h_m} = 1 - \left(1 - \frac{3h_{cp}}{B} \cdot \frac{y}{h_m} \right)^2 \quad (12)$$

h_{cp} and B is the average depth and width of the canal at a stationary flow, which are determined by the method described in [13] in accordance with a given water flow rate and the number of pumps entering the canal.

Comparison of the obtained dependence (12) (Fig. 2) for a stationary flow shows good similarity with the author's experimental data.

According to laboratory tests, the following values of the angle of internal friction of the soil have been established: in a dry state $-\varphi_{0c} = 33^{\circ}09'$ ($tg \phi_{0c} = 0.653$); in wet $-\varphi_{0Bt} = 31^{\circ}22'$ ($tg \phi_{0wet} = 0.610$).

The analysis of the obtained data of the studies of the transverse profiles shows that the values of the angle of repose of the bottom soil in the model canal varied within $tg \varphi_g = 0,320...0,658$ and were in good agreement with the one calculated by the formula (11). In this case, dependence (4), which shows for the initial soil $tg(\varphi_{0wet}/1.65) = 0/344$, should not be generally considered valid for the accepted experimental conditions. So, the analysis shows the reliability of the hypothesis on the use of dependence (3) for dynamically stable canal canals. But for this case, the angle of internal friction of the bottom soil should not be reduced by 1.65 times, but set in accordance with the initial flow rates of water and sediment entering the canal. These parameters entering the canal in accordance with [13] determine the width and average depth of the canal transporting sediment, which in turn determine the shape of the transverse profile of the bottom of the dynamically stable canal (see dependence 11).

Application of the dependence (11) made it possible to determine the coefficient K from the given transverse profiles of the bottom under the action of waves on the flow, the values of which varied within the range of $K = 0.5 \dots 0.1$ (average value $K = 0.3$). As a result, we obtain a transverse profile for a dynamically stable canal under conditions when waves are superimposed on currents and dependence (7) takes its final form:

$$\frac{h}{h_{mv}} = 1 - \left[1 - \frac{\sqrt{\left(\frac{6h_0}{B}\right)^2 - \frac{0,3u_{*m}^2}{(vg)^{2/3}}}}{2h_{mv}} y \right]^2 \quad (13)$$

where B is the canal width at a purely drain flow.

To determine the canal width under conditions of wave superposition on currents, an assumption was made, which in many cases is confirmed by the data of experimental studies (Fig. 2): wave superposition on the flow, leading to an increase in the canal width, does not significantly change its cross-sectional area. This assumption is based on the fact that was obtained in the course of the experiment that the erosion of the canal in the near-waterside part is accompanied by an approximately equal intensity of sediment accumulation in the zone of conjugation of the coastal slopes with the central part of the canal. Using this assumption, written in the form

$$\int_0^{B/2} hdy = const \quad (14)$$

allowed to get the ratio:

$$tg\varphi_g B^2 = tg\varphi_v B_v^2 \quad (15)$$

where φ_g, B is the angle of natural slope of the bottom soil and the width of a dynamically stable canal at a stationary flow; φ_v, B_v - the angle of repose of the bottom soil and the width of the dynamically stable canal in conditions of superposition of waves on the current.

From (15) we have:

$$B_v = B \sqrt{\frac{tg\varphi_g}{tg\varphi_v}} = B \sqrt{\frac{\frac{6h_0}{B}}{\sqrt{\left(\frac{6h_0}{B}\right)^2 - \frac{0,3u_{*m}^2}{(vg)^{2/3}}}}} \quad (16)$$

The calculated value of the canal width, formed as a result of the combined action of waves and currents according to the formula (16), showed (Fig. 2) a good agreement with the results of laboratory measurements. From this we can conclude the reliability of the earlier assumption (14).

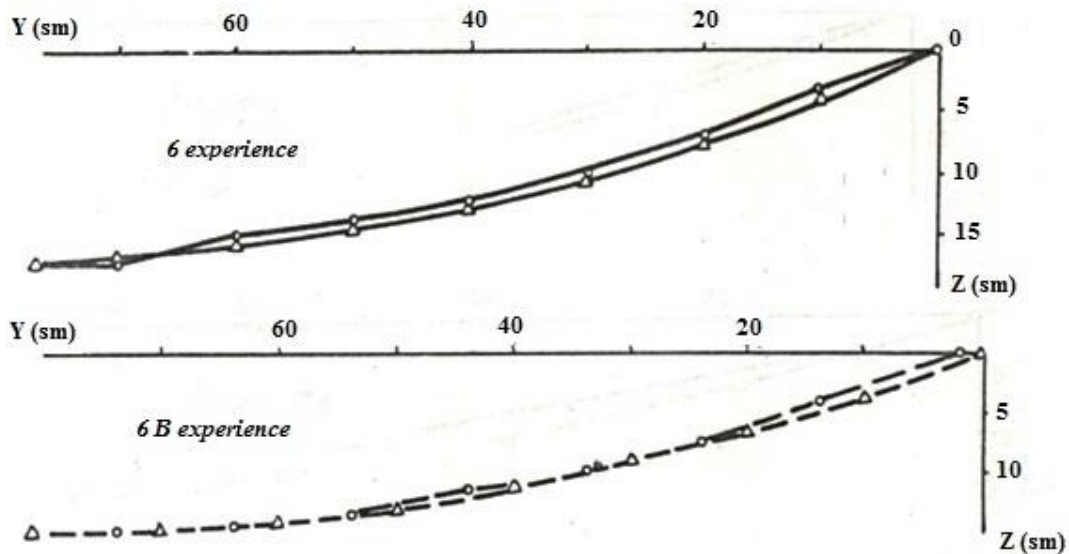


Figure 2. Comparison of the calculated (1) and measured (2) transverse profiles of the experimental canal: o - at a stationary flow; Δ - at unsteady flow.

Substituting further into B_v and $tg\varphi_v$ in (10), we have the dependence for calculating the maximum canal depth during waves:

$$h_{mv} = \frac{B_v tg\varphi_v}{4} \tag{17}$$

The calculation of the slopes of the trapezoidal canal with known values of the width and maximum depth is determined by the formula:

$$m = \frac{B_v}{h_{mv}} \left(1 - \frac{h_0}{h_{mv}} \right) = 0.3 \frac{B_v}{h_{mv}} \tag{18}$$

4. Conclusion

As a result of the studies carried out, dependencies were obtained for calculating the parameters of large earth canals at stationary currents and the superposition of waves on currents.

It should be noted that the great methodological difficulties of the process of propagation of surface gravity waves in a counter current (blocking of waves, rapid attenuation of their amplitude, etc.) did not allow an experimental study of the deformation of earth canals in this mode. However, drawing an analogy between the contribution of wind waves on the head and counter currents to the overall balance of tangential stresses at the bottom of the canal flow makes it possible to make an assumption, according to which, in the case of waves on the counter current, as well as on the head one, the canal cross section will be zones with multidirectional canal deformations. Thus, in the near-water part of the canal, the dominance of the tangential stresses caused by wind waves will cause erosion of this part of the canal and the subsequent accumulation of erosion products in the central zone of the canal. In this case, as in the case of a passing current, flattening of the slopes of the canal will be observed and the shape of its cross-section will be identical to the shape of the canal during waves on the passing current. This assumption gives reason to believe that the calculation method described in this article is applicable to the case of waves on the opposite current.

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