SEISMIC STABILITY CALCULATIONS OF EARTH AND ROCKFILL DAMS

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The following subjects related to seismic stability evaluation of earth and rockfill dams [1] are treated in the paper: (1) the determination of the inertia loads through the seismic coefficients adopted in the Standards and with the use of spectral charts of the dynamic coefficient; (2) the examination of the slope seismic stability with account for the structural behaviour represented as a plane problem [4-6] and a three-dimensional problem [8].

On the basis of the acting Standards the calculations of a number of earth and rockfill hydraulic structures were made including the evaluation of the 296 m high Nurek rockfill dam being constructed in area with seismicity IX.

The determination of the inertia loads acting upon structures was made by the shear wedge (one-dimensional) approach and the finite element (two-dimensional) procedure. In these cases the inertia load S_{ikj} acting in point "k" in the j-th direction upon a structural mass of the weight Q_K for the i-th mode of the structural natural vibrations was determined from the expression

$$S_{ikj} = K_c \, m \, Q_K \, m^0 \beta_i \, \eta_{ikj} \tag{1}$$

in which η_{ikj} is the coefficient of the i-th mode of natural vibrations for the structural point "k", equal to

$$\eta_{ikj} = u_{ikj} \, C_i(\vec{u}_o) = u_{ikj} \frac{\sum\limits_K Q_K \sum\limits_{j=1}^{n} u_{iKj} \cos[\widehat{u_{iKj}, \vec{u}_o}]}{\sum\limits_K Q_K \sum\limits_{j=1}^{n} u_{iKj}^2} \tag{2}$$

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u_{ikj} are the projections of the displacement of points "k" along the three mutually orthogonal directions ($j = 1, 2, 3$); $\cos(u_{ikj}, \hat{\vec{u}}_o)$ are the cosines of the angles between the directions of the seismic influence vector $\vec{u}_o$ and the displacements of structural point "k", β_i is the spectral dynamic coefficient defined through Diagram 2 of the Design Standards (1) or from the expression $\beta_i = 1/T_i$ (here T_i is the period of the i-th tone of the initial vibrations adopted in the range between 3.0 and 0.8).

Eq.(1) contains the following coefficients: m which considers the risk of the ultimate state for the dam and which is taken equal to 1.3 for the 1st class retaining earth fill hydraulic structures and equal to 1.0 for the same structures of the 2nd, 3rd and 4th classes; m^o which depends on the kind of structural material and structural type; for earth and rockfill dams $m^o = 0.7$. In this case the value of $\beta_i^o = m^o \beta_i$ is taken no less than 0.8.

The design inertia loads S_{pkj} in point "k" of the structure for the prescribed number of natural vibration modes are calculated from expression

$$S_{pkj} = \sqrt{\sum_{i=1}^{n} S_{ikj}^2} \qquad (3)$$

Accordingly the seismic accelerations S^*_{pkj} in point "k" are determined from the same equations (1)-(3) with the only difference that the factor Q_k is omitted in (1).

The determination of the horizontal seismic accelerations (inertia loads) in a one-dimensional shear wedge representation was performed with account for the first four natural vibration modes. The calculation of the horizontal S_x and vertical S_y components of the inertia loads (or the accelerations S_x^*, S_y^*) in a two-dimensional finite element idealization were made with inclusion of a various number of natural vibration modes (up to 40).

The three-dimensional slope stability evaluation was performed with account for the linear variation (decline) of seismic accelerations (the inertia loads) from the values obtained in the central section (of the maximum height) evaluated by the above shear wedge and finite element representations down to the appropriate Standard values of these parameters at the structure-foundation interface.

The dynamic material characteristics were received from the geophysical measurements [2] of the propagation velocities of the elastic longitudinal, V_p and transverse, V_s, waves in the core and shells of the Nurek dam. For this purpose a number of production explosions was produced in the vicinity of the dam.

In the shear wedge calculations the values of $\rho = 2.3$ t/m^3 and $V_s = 700$ m/sec averaged along the whole structural section were adopted and in the finite element representation two versions of the dynamic characteristics were taken - the values averaged along the whole structural section (1st version) and the values different for various zones (the core, the shells and the stone fill of the slopes) but constant within each of these zones (2nd version).

Table 1

Versions of characteristics	Structural zones	Density ρ, t/m^3		The Poisson coefficient	Elastic moduli E [MПa] for:	
		ρ_{sk}	ρ_{sat}	μ	ρ_{sk}	ρ_{sat}
I	the whole of the section	ρ_{av} = 2.3		0.361	E_{av} =	3070
II	core	1.96	2.33	0.361	2150	2400
	shells	2.20	2.40	0.361	2760	3070
	stone fill of the slopes	1.85	2.18	0.361	2760	3070

The values of T_i obtained from the calculations of the Nurek dam in a one-dimensional shear wedge representation are summarized in Table 2 and those obtained from a two-dimensional finite element idealization with a uniform mesh and for the 1st version (Table 1) of the material dynamic characteristics are given in Table 3.

Table 2

The parameters under definition	Natural vibration modes			
	1	2	3	4
T_i, c	1.08	0.482	0.308	0.226

Fig. 1 shows the distribution of the overall inertia loads along the dam height obtained through the shear wedge and finite element calculations.

In the shear wedge calculations the horizontal load S_x^{swm} was defined by multiplying the mass of a soil layer (with the unit height and thickness) located at the considered depth below the crest by the design acceleration in the same direction obtained with account for four natural vibration modes. In the finite element calculations the horizontal (S_x^{fem}) and the vertical (S_y^{fem}) components of the inertia load with account for various number (up to 40) of the natural vibration modes were defined by their summation over all the nodes located at the considered depth below the crest and by deviding the obtained result by the height of the layer the mass of which was referred to these nodes.

Table 3

No.No. of modes	1	2	3	4	5	6	7	8	9	10
T_i, c	1.23	0.801	0.683	0.598	0.563	0.543	0.482	0.455	0.419	0.406
No.No. of modes	11	12	13	14	15	16	17	18	19	20
T_i, c	0.393	0.369	0.361	0.344	0.33	0.322	0.317	0.309	0.296	0.290
No.No. of modes	21	22	23	24	25	26	27	28	29	30
T_i, c	0.281	0.274	0.271	0.261	0.260	0.256	0.254	0.249	0.244	0.239
No.No. of modes	31	32	33	34	35	36	37	38	39	40
T_i, c	0.236	0.233	0.231	0.228	0.224	0.221	0.220	0.218	0.217	0.212

With the same initial data the two-dimensional finite element calculations result in a denser spectrum and longer natural vibration periods compared to the shear wedge evaluations (Table 2 and 3). The inclusion of the shear wave velocities V_p, V_s (or the elastic moduli) obtained by the geophysical methods under the higher frequencies ($f \geqslant$ 30-80 cps) and lower relative strain amplitudes ($\varepsilon \leqslant 10^{-5}$) as compared to the values of the same parameters (f = 0.25-15 cps and $\varepsilon = 10^{-5} \to 10^{-2}$) observed in earth structures under earthquakes of intensity 7-9, results in the overestimated values of the structural initial vibrations and the inertia loads acting upon these structures under earthquakes.

The limiting condition $\beta_i^o = m^o \beta_i \geqslant 0.6$ adopted in the Design Standards, may result in some overestimation of the inertia loads (and accelerations) for the super high dams (of the Nurek type) when calculations are performed by the i-th modes with the natural vibration period $T_i \geqslant 1.25$ s. In evaluating the inertia loads on the lower part of the dam (in the vicinity of the base) the accelerations in the foundation were taken equal to the Standard values of $m k_c$. Fig. 1 reveals that the main contribution into the horizontal component of the load S_x is produced by the 1st, 5th, 7th, 11-12th and 14-15th natural vibration modes. The inclusion of the 16-20th and 21-25th natural vibration modes results in a negligible (not exceeding 2-3%) increase of the inertia load in the upper section of the dam (to the depth of 150 m) and in a more remarkable increase of S_x (7–10%) in its lower section. The inclusion of 26–40th natural vibration modes into the calculations shows practically no further change in the values of S_x.

The main contribution into the vertical load value is also produced by the first fifteen natural vibration modes. With the further increase in a number of natural vibration modes taken into account (up to 26), the values of S_y increase to a greater extent (10–20%) than the values of S_x. However, a still further increase of the natural vibration modes considered (up to 40) shows practically no change in the value of the vertical component of the load S_y.

The results of these and other calculations show that in the evaluations of the inertia loads acting on the dams of the type under study, the selection of the appropriate number of natural vibration modes to be considered may be performed on the basis of the convergence of the horizontal component S_x. The number of the natural vibration modes included into the calculations may be restricted to 15. The horizontal loads S_y^{swM} obtained by the shear wedge one-dimensional approach for the various sections of the dam proved to be 10–30% higher compared to the appropriate horizontal components of the inertia loads S_x^{fem} obtained from the finite element calculations with inclusion of 40 natural vibration modes.

Thus, when evaluating seismic stability of the embankment dams, it should be kept in mind that the value and distribution of the inertia loads are highly affected by the choice of the dynamic characteristics of the soils, the structural calculation schemes and procedures, the number of the naturalvibration modes taken into account, the coefficients [1] adopted in the Standards and other factors.

The slope stability evaluations of earth and rockfill dams with account for the behaviour of the slopes in a plane strain condition under the seismic loads obtained from the above calculations were performed by the methods of Krey and the Mozhevitinov method of the "inclined interrelation forces" [4] satisfying all the three equilibrium conditions. The calculations were performed using a specially developed computer program [5]. On the basis of the above method algorithms and a program for calculating the three-dimensional stability of hydraulic structures of the embankment type under earthquake effects were also developed [8]. These programs allow to perform the slope stability analysis with any type of distribution of the accelerations and inertia loads obtained from calculations, experimental observations and field investigations of dam vibrations under earthquakes.

The slope stability calculations were carried out for the cases of the normal headwater level and the tailwater level of 0.2H (H – pressure head upon the dam) with two versions of the shear resistance characteristics of the rockfill shell material (Table 4).

Table 4

Dam material	Volumetric soil density [T/M³]			Shear resistance characteristics: 1st version		Pressure	2nd version	
	γ_{sk}	γ_{sat}	γ_{susp}	$\varphi°$	C [kПа]	σ [MПа]	$\psi°$	C [kПа]
Stone facing of slopes	1.85	2.18	1.18	39	60	–	42	–
						$\sigma \leq 0.5$	40	–
Boulder shells	2.20	2.40	1.40	39	60	$0.5 < \sigma \leq 1.0$	38	–
						$\sigma > 1.0$	35	–
Loamy core	1.96	2.33	1.33	28	40		$\varphi = 28$	40

Besides the shear resistance characteristics of the shell materials (Table 4) represented by the values of ψ varying stepwisely with the pressure σ (2nd version) and by the parameters φ, C in the Koulomb-Mohr relationship (version 1), the equivalent values $\psi = \text{arc tg}[\,\text{tg}\,\varphi + \frac{C}{\sigma}]$ continuously varying with the pressure (version 1a) were used for the calculations.

Fig. 2 shows the most dangerous plane and three-dimensional surfaces of the slopes of the Nurek hydropower complex. Table 5 presents the minimum safety factor values corresponding to these surfaces.

The analysis of the calculation results discloses that:

- the lowest values of the safety factor (0.81 for the upstream slope and 0.96 for the downstream slope) were obtained when determining the inertia loads by a one-dimensional shear wedge scheme using the 2nd version of the shear resistance characteristics (neglecting the cohesion);
- the determination of the inertia loads on the dam by the two-dimensional finite element scheme using the same shear resistance characteristics produced the larger values of the safety factor (1.04 for the upstream slope and 1.15 for the downstream slope);
- the determination of the inertia loads by the finite element scheme using the 1st version of the strength characteristics of the shell material (with account for cohesion) results in still higher values of the safety factor (1.30 for the upstream slope and 1.40 for the downstream slope);
- the increase in the number of the initial vibration modes taken into account (from 20 to 40) does not practically influence the value of the slope stability safety factor;
- the seismic stability calculations of the dam slopes with account for the three-dimensional slope behaviour in the valley brings about an increase in the safety factor by 10% in cases when the accelerations along the width of the valley are constant and by 20% in cases when account is taken of their decrease in the direction from the design section to the interface of the dam and the banks;
- under the identical versions of the strength characteristics (1 and 1a) the values of the safety factor are practically the same. Therefore in the cases of a large range of the variations of the normal pressure σ the stability calculations may be performed by direct usage of the nonlinear experimental relation $\tau = f(\sigma)$ without its approximation by sections of the Coulomb-Mohr straight lines. The results obtained prove that the proper selection of calculation methods and schemes adequately representing the structural behaviour in the field, of the number of natural vibration modes taken into account, of the strain and strength material characteristics as well as the inclusion of some other factors discussed earlier in the paper, play an important role in the substantiation of the seismic stability evaluations of hydraulic structures.

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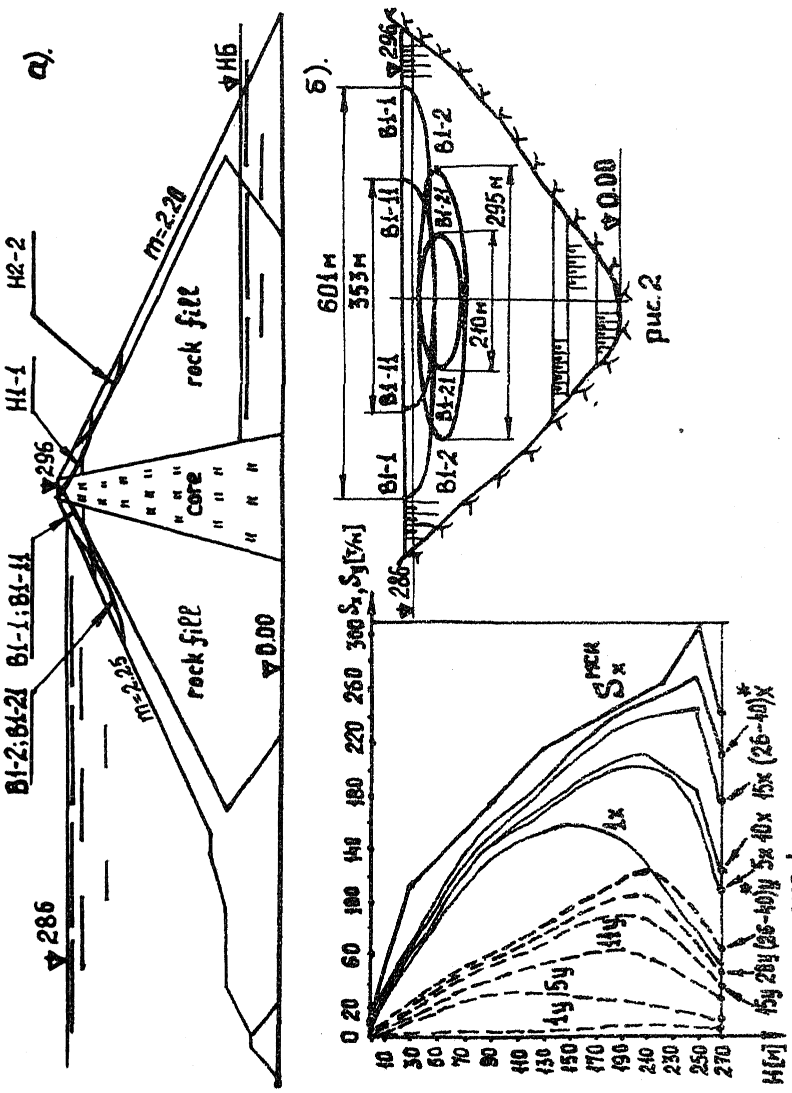

Fig.1. Distribution of the inertia loads along the dam height

Fig.2. Most dangerous failure surfaces of the dam slopes
a) section
b) view from the upstream

Table 5

Slopes	Versions of strength characteristics	Inertia load calculation procedure	Number of natural vibration modes taken into account	Most dangerous shear surface	Safety factors $(K_2)_{seism}$	$(K_2)_{static}$	$(K_3)_{seism}$	$(K_3)_{static}$
Upstream	1st version (φ, c)	shear wedge	4	BI-I BI-II	1.07 1.07	2.81 2.81	1.21 1.34	3.18 3.52
	1st version (φ, c)	finite element	20	B4-I	1.31	2.32	-	-
		finite element	40	B5-I	1.30	2.32	-	-
Upstream	2nd vefsion	shear wedge	4	BI-2 BI-21	0.81 0.81	1.92 1.92	1.00 1.15	2.21 2.47
	$\psi = \psi(\sigma)$	finite element	20	B2-2	1.05	2.06	-	-
	$c = 0$	finite element	40	B3-2	1.04	2.06	-	-
Downstream	1st version	shear wedge	4	HI-I	1.11	1.98	1.27	2.22
	(φ, c)	finite element	20	H3-I	1.41	2.16	-	-
		finite element	40	H4-I	1.40	2.16	-	-
	2nd version	shear wedge	4	H2-2	0.96	1.78	1.16	2.09
	$\psi = \psi(\sigma)$	finite element	20	H2-2	1.16	1.59	-	-
	$c = 0$	finite element	40	H3-2	1.15	1.59	-	-