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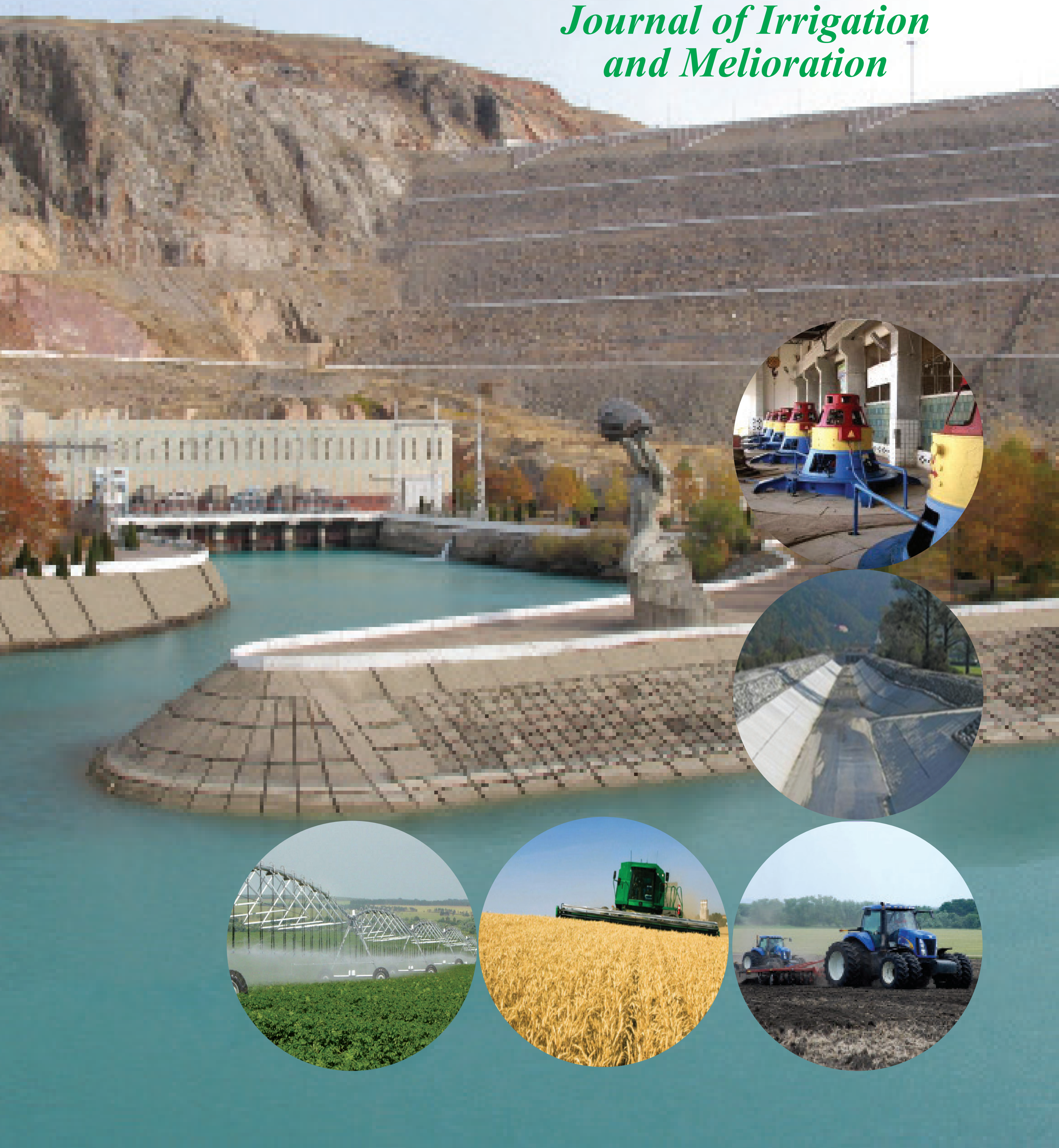

IRRIGATION AND MELIORATION

HYDRAULIC ENGINEERING STRUCTURES AND PUMPING STATIONS

ELECTRIFICATION AND AUTOMATION OF AGRICULTURE AND WATER RESOURCES MANAGEMENT

ECONOMICS OF WATER MANAGEMENT AND USE OF LAND RESOURCES

STUDY OF DYNAMIC BEHAVIOR OF EARTH DAMS CONSIDERING THE DAM BASE

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Abstract

The methods have been developed for solving various dynamic problems for inhomogeneous viscoelastic "structure-base" systems taking into account wave entrainment of energy, i.e. non-reflecting conditions at the boundary of the finite site of the base. The dynamic behavior and the stress-strain state of the inhomogeneous "viscoelastic dam-elastic base" system using wave entrainment of energy through the boundaries of the finite site of the base under short-term intense effect in the base are investigated.

Key words: earth dam, base, wave entrainment of energy, non-reflecting conditions, dynamic behavior, stress-strain state, inhomogeneous system.

Introduction. When assessing the dynamic behavior of earth dams considering their bases, the most often used model is the Winkler base one, which, despite its simplicity in calculation, does not allow taking into account a number of physical effects associated with the inertial properties of earth base. An elastic half-space model is devoid of this drawback, however, due to mathematical complexity it does not allow to obtain an analytical solution in a closed form, with the exception of a number of particular static problems.

In the study of dynamic behavior, along with all the above factors, it is also necessary to take into account the energy entrainment from the structure to an infinite earth base.

Many existing models that take into account the joint work of the structure-base system, even in the simplest case do not let to describe the dynamic process associated with the energy entrainment to an infinite base.

There are a sufficient number of publications [29,30], where it is proposed to use non-reflecting conditions at the boundary of the finite base site [1-14, 16-19, 26-27], which ensure energy entrainment from the structure to infinity.

Here are just some of the works devoted to the problem of studying the dynamic behavior of the "structure-base" system using artificial non-reflecting conditions at the boundary of the finite base site that provide energy entrainment.

Of particular note are the papers [1,2,14], devoted to the problem of setting the correct boundary conditions at the artificial boundaries of the computational domain, to mathematical justification, effectiveness analysis in solving specific problems. These papers also give a review of published works in which the artificial boundary conditions are analyzed on the boundary of the finite site.

From this it follows that the problem of assessing the dynamic behavior of inhomogeneous systems "structure-foundation-base" taking into account the wave entrainment of energy through the boundaries of the finite earth base is far from final solution and presents an urgent problem that needs to be solved.

Methods of research: This paper is devoted to solving the following issues:

- to study the dynamics of a structure, taking into account the base; it is proposed to replace the infinite base with finite sites using special conditions that ensure energy entrainment through the boundaries of the finite site;

- the variational problem is investigated with account for special conditions ensuring the energy entrainment and viscoelastic properties of structure material;

-the methods to solve the set dynamic problems for a finite site are proposed;

- the forced unsteady oscillations of earth dam are studied together with the base.

Consider a plane inhomogeneous system (structure + foundation + base), consisting of a deformable body occupying the volume $V=V_1+V_2+V_3+V_4$ and a deformable half-space (Fig. 1). The material of the deformable inhomogeneous body is viscoelastic, and the half-space is elastic. At the interfaces between the elements of the system (V_1, V_2, V_3, V_4) there are continuous displacements, stress components normal and tangential to the interface. The considered structure is presented as a massive structure; therefore, the calculation takes into account the mass forces $\vec{f}$ and various force effects applied to an arbitrary surface Σ_p.

The task is to determine the fields of displacements and stresses in the elements of the system (V_1, V_2, V_3, V_4) under various dynamic influences.

The problems under consideration are posed for a finite site (Fig. 1) of volume $V+V_5$ (V_5 is the volume cut out from a half-space) and restricted by surfaces $+\int_V \vec{f}\delta\vec{u}\,dV+\int_{\Sigma_p}\vec{p}\delta\vec{u}\,d\Sigma=0$, on which non-reflecting conditions are set.

To describe the dynamic processes occurring in the system (Fig. 1), the principle of virtual displacements is used, according to which the sum of the work of

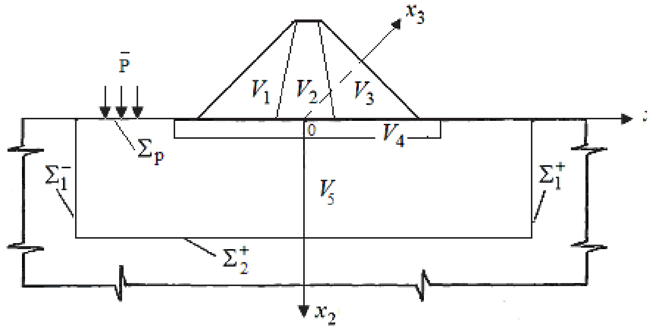


Figure 1. Calculation model of the system

all active forces, including inertia ones, on virtual displacements is zero:

$$\delta A = -\int_{V+V_5} \sigma_{ij}\delta\varepsilon_{ij} dV - \int_{V+V_5} \rho_n \ddot{\vec{u}}\delta\vec{u}\, dV + \int_{\Sigma_1^- + \Sigma_1^+ + \Sigma_2^+} \sigma_{ij}\nu_j \delta\,\vec{u} d\Sigma + \int_V \vec{f}\delta\,\vec{u}\, dV + \int_{\Sigma_p} \vec{p}\delta\,\vec{u}\, d\Sigma = 0 \qquad (1)$$

When setting the task, the following assumptions are used:

- it is assumed that volume strain occurs according to the elastic law, and shear strain - according to the viscoelastic law; physical relations connecting the stress tensor σ_{ij} with the strain tensor ε_{ij} [20,22] have the form

$$S_{ij} = \mu_n\left[e_{ij} - \int_o^t \Gamma_n(t-\tau)e_{ij}(\tau)d\tau\right]; \sigma = K_n\theta \qquad (2)$$

So, K_n, μ_n are the instantaneous moduli of volume and shear strain; S_{ij}, e_{ij} are the components of stress and strains deviators; σ is the spherical part of the strain tensor; θ is the volume strain.

For the elastic material of the *n*-th element of the system (Fig. 1), the values of μ_n are the shear modulus of elasticity, and for a viscoelastic material μ_n are the Volterra integral operators [20,22] of the form:

$$\mu_n[\varphi(t)] = \mu_n\left[\varphi(t) - \int_0^t \Gamma_n(t-\tau)\varphi(\tau)\, d\tau\right] \qquad (3)$$

Cauchy relations are used [24] to connect the components of the strain tensor e_{ij} and the components of the displacements vector $\vec{u}$:

$$\varepsilon_{ij} = \frac{1}{2}\left(\frac{\partial u_i}{\partial x_j} + \frac{\partial u_j}{\partial x_i}\right) \qquad (4)$$

- and non-reflecting conditions of the form [1,3],

$$\vec{x} \in \Sigma_1^{\pm}: \quad \frac{\partial u_i}{\partial x_1} \pm \frac{1}{c_R}\frac{\partial u_i}{\partial t} = 0, \qquad \vec{x} \in \Sigma_2^{+}: \quad u_i = 0. \qquad (5)$$

at energy entrainment through the boundaries $\Sigma_1^{\pm}$ of the finite site V_5.

Here: $\vec{u} = \{u_1, u_2\}$ are the components of the displacement vector; $\delta\vec{u}$, $\delta\varepsilon_{ij}$ - are the isochronous variations of displacements and strains; ρ_n is the density of the material of the *n*-th element of the system; $\vec{f}$ is the vector of mass forces; $\vec{p}$ is the vector of external loads; Γ_n are the relaxation kernels; $\varphi(t)$ is the arbitrary function of time; ν_j - are the direction cosines of the outer normal; c_R are the propagation velocities of the Rayleigh wave in a half-space; *n* = 1,2,3,4,5 is the numbering of the system elements; *i, j* = 1,2.

The steady-state and unsteady-state forced vibrations of an inhomogeneous system are considered (Fig. 1). All the problems under consideration are solved by the finite element method (FEM) with a discretization of $V+V_5$ site into a number of finite elements. When solving specific problems, the discretization of $V+V_5$ site (Fig. 1) into finite elements is carried out considering the structural features and physico-mechanical properties of the material of different parts of the system.

Algorithms for solving the problem: Unsteady-state forced vibrations of a structure occur as a result of non-periodic effect, and significantly depend on the initial configuration and loading velocity. This allows us to determine the maximum values of displacements, strains and stresses in any part of the dam for the entire process of external influences, to reveal dangerous sections of structures in terms of strength and to develop ways to reduce the stress-strain state (SSS) taking into account certain material parameters and design features of structures.

For this case, the considered problem for the system (Fig. 1) using FEM procedure is reduced to solving a system of linear integro-differential equations

$$[M]\{\ddot{u}(t)\} + [C]\{\dot{u}(t)\} + [K]\{u(t)\} = = \{F\} + \{f(t)\} + \int_0^t \Gamma(t-\tau)[K]\{u(\tau)\}d\tau \qquad (6)$$

With initial conditions

$$\{u(0)\} = \{u_0\}, \ \{\dot{u}(0)\} = \{v_0\} \qquad (7)$$

Here the matrices [M], [K] are the mass and stiffness matrices of the system; [C] is the matrix that accounts for wave entrainment of energy; $\{u(t)\}$ is the vector of the sought for displacement amplitudes; $\{f(t)\}$ is the vector of dynamic load; $\{F\}$ is the total vector of static loads (mass forces, hydrostatic pressure of water, etc.).

The system of integro-differential equations (6) is solved under initial conditions (7) by the Newmark method [25].

Research results: Using the above methods, a plane problem of assessing the dynamic behavior of inhomogeneous system (Fig. 1) is solved; the base is elastic, and the structure is viscoelastic under non-stationary dynamic effect, changing according to the law:

$$\vec{x} \in \Sigma p: \quad P(t) = \begin{cases} 100000 & t = 0 \\ -250000t + 100000 & in \quad 0 \le t < 0{,}4 \text{ sec} \\ 0 & t \ge 0{,}4 \text{ sec} \end{cases} \qquad (8)$$

The load *P(t)* in kN is plane and applied at a distance of 25 m from the foot of the dam on the base surface, i.e. at the site Σ_P (Fig. 1). It is necessary to determine the fields of displacements and stresses in the body of the dam at various points of time that occur under instantly applied loads (8).

In calculations the following values were accepted:

- for the dam: height H = 168.0 m, the coefficients of the upper and lower steepness of slopes $m_1=m_2=2.2$m; the crest width b = 10.0 m; material properties: the elasticity modulus *E* = 3000.0 MPa; the Poisson's ratio ν = 0.3; the specific gravity of soil γ = 2.2 tf/m³. To take into account the viscoelastic properties of soil, the A. Rzhanitsyn kernel was used [23]

$$\Gamma(t) = Ae^{-\beta t}t^{\alpha-1}, \qquad (9)$$

with parameters [15]: A = 0.0146; α = 0.2; β = 0.0000057.

- for the base: the elasticity modulus E = 3600.0 MPa; the Poisson's ratio ν = 0.3; the specific gravity of soil = 2.8 tf/m³.

The problem solution with stated parameters revealed that the waves resulting from the applied load *P(t)* create a non-uniform field of displacements in the dam body. The beginning of the motion of each point of the structure corresponds to the time of arrival of the wave front, determined by the distance of the point from the place of load application and the velocity of wave propagation in soil.

The results showed that not all the points of structure enter the motion simultaneously. The beginning of motion of each point corresponds to the time of arrival of the blast wave front. This time is uniquely determined by the distance of the point from the source of explosion and the velocity of wave propagation in soil.

Figure 2 shows the isolines of horizontal displacements distribution in the dam section at different points of time. A wave from a source located in relative proximity

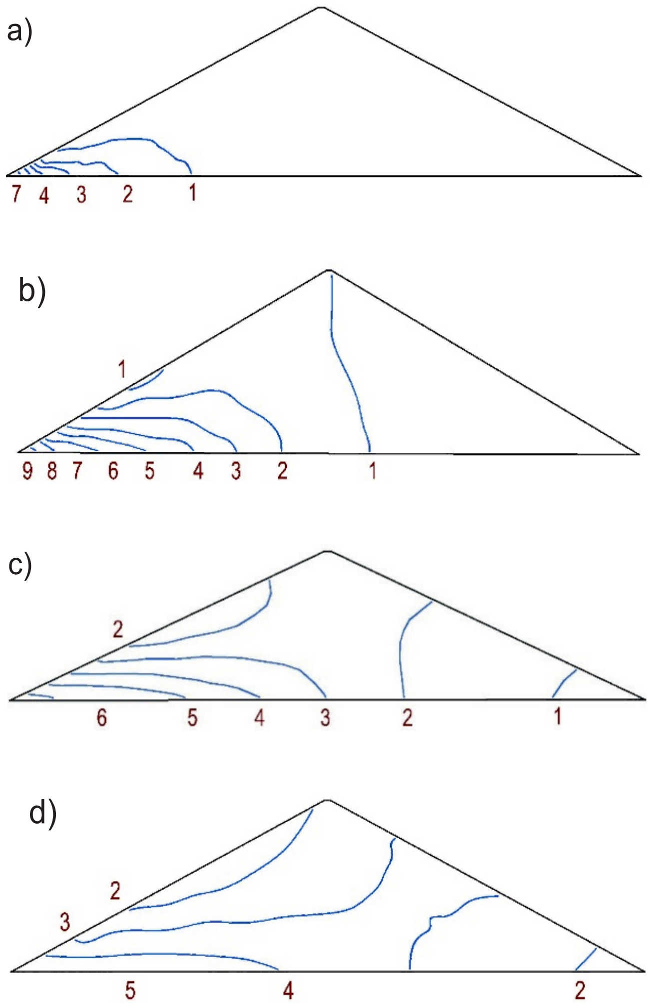


Fig. 2. Isolines of the distribution of horizontal displacements (m) in the dam section at different points of time t: (a) 0.2 sec, (b) 0.32 sec, (c) - 0.52sec, (d) - 0.60sec

to the bottom of the dam, propagating along the base, first causes a displacement in the foot of the upper slope (Fig. 2a), and eventually covers more distant sections of the structure (Fig. 2c, d).

Here, the lower section of the upstream slope, limited by isoline “1”, remains stationary as a result of wave diffraction at the base-slope contact point. The isoline with the same index on the lower slope (Fig. 2b) corresponds to the position of the wave front, before an unperturbed (at *t* = 0.46 sec) section of the dam (right side of the figure). In subsequent moments, the load disturbance *P(t)* completely covers the dam body and the distribution of horizontal displacements in it is represented by isolines in Figs.2 a-d. After wave propagation, the strained state of the dam gradually

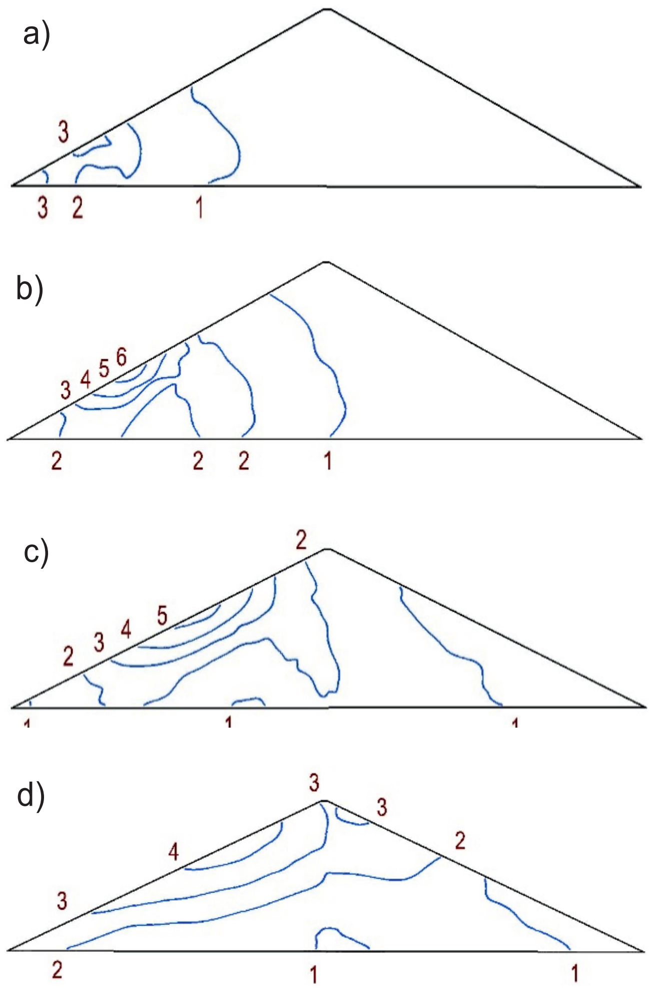
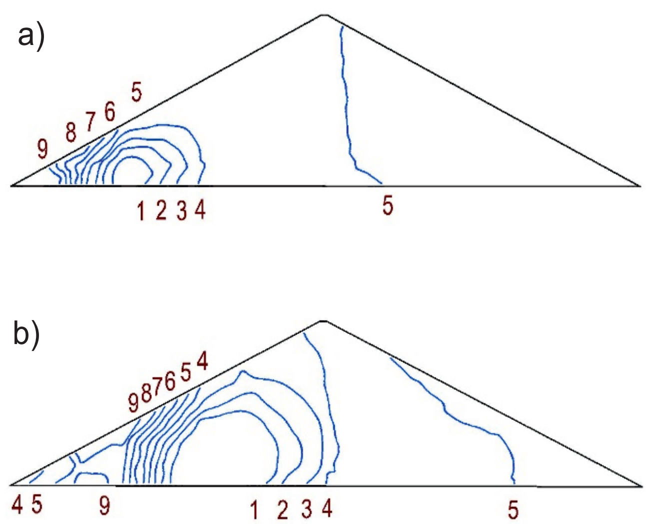


Fig. 3. Isolines of the distribution of the principal stresses σ1 in the dam cross section at different points of time t: (a) 0.2sec, (b) 0.32sec, (c) 0.52sec and (d) 0.60sec

stabilizes.

The values of horizontal displacements on isolines (Fig. 2) increase at an equal interval of 0.005 m beginning from 0.0 m - on isoline "1". The maximum displacements are 0.042 m and are observed in the site



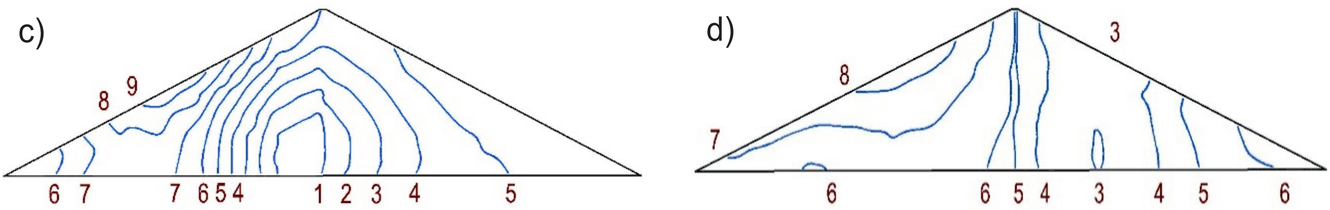


Fig. 4. Isolines of the distribution of shear stresses σ12 in the dam cross section at different points of time t: (a) 0.2sec, (b) 0.32sec, (c) 0.52sec and (d) 0.60sec.

restricted by the line with the index "9", on the line itself the displacements are 4 cm.

The stress state of the dam, represented by the principal stresses σ1 at various points of time (at the beginning, in the middle and at the end of the process) is shown in Fig. 3. The stress is measured in MPa.

At the initial time of the process, the lower part of the upstream slope is strained, a tensile zone [28] with positive stresses σ1 (line “2” in Fig. 3a) appears, which then propagates up the slope along with the wave propagation (Fig. 3b, c) and over the entire internal site of the dam (Fig. 3c, d). The magnitude of the stresses σ1 at the isolines (Fig. 3) varies with the same step of 0.05 MPa: from 0.0 MPa - on the line "1" to 0.3 MPa - on the line "6"

The maximum tangential stresses (σ12) arise on the surface of the upstream slope (Fig. 4): first, at its foot, and then along the entire height, which is fraught with the possibility of a landslide on the slope (Fig. 4).

The stresses σ12 on the isolines (Fig. 4) vary with a step of ± 0.025 MPa from 0.0 MPa on the line “5”to ± 0.1 MPa on the lines “1” and “9”.

Conclusions:

1. The methods have been developed for solving various dynamic problems for inhomogeneous viscoelastic "structure-base" systems, taking into account non-reflecting conditions at the boundary of the finite base site.
2. The study of the dynamic behavior of the inhomogeneous viscoelastic dam-base system with non-reflecting boundaries under short-term intensive action at the base showed that:

- maximum values of principal stress σ1 arising in the lower part of the upstream slope gradually extend to the entire slope and the central part of the dam;
- maximum values of tangential stresses σ12 are reached on the surface of the upstream slope, first at the foot of the dam, then along the entire surface of the slope. There are no tangential stresses in the center of the dam;
- when the wave propagates along the dam, asynchronous movement of its parts occurs, damping due to energy entrainment and viscoelastic properties of the structure material.

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