International Journal of Progressive Sciences and Technologies (IJPSAT)
ISSN: 2509-0119.
© 2021 International Journals of Sciences and High Technologies
http://ijpsat.ijsht-journals.org
Vol. 25 No. 2 March 2021, pp. 51-62

The Stress State Of A Soil Dam Under Dynamic Action, Taking Into Account The Dissipative Properties Of The Soil

K. D. Salyamova[1], A.A.Yangiev[2*], X. Fayzirv[3*], X. X. Tyrdikulov[1*], D.S. Adjimuratov[2*]*

[1] Institute of Mechanics and Seismic Stability of Structures of the Academy of Sciences of the Republic of Uzbekistan, Tashkent, Uzbekistan

[2] Tashkent Institute of Irrigation And Agricultural Mechanization Engineers,

[3] Tashkent Architectural and Construction Institute.

(cc) BY

Abstract – This article presents solutions to a non-stationary dynamic problem for a soil dam, taking into account the elastic and inelastic characteristics of the soil, discloses the applied methods, identifies areas of instability, justifies the use of a numerical method to solve such problems. The main goal of this work is to develop a methodology for solving dynamic problems for soil hydraulic structures (dams, dikes, reservoirs) in a flat setting, taking into account the elastic and inelastic properties of the soil material. The calculation results are presented in the dependences form of stresses and displacements make it possible to determine the vulnerable zones of an earth dam, where the stable operation loss of the structure is possible.

Keywords – Earth Dam, Rezaksay Dam, Finite Element Method, Newmark Method, Stresses, Flat Elastic Setting.

I. INTRODUCTION

The design and construction of high earth dams requires attention to long-term processes taking place in their body and affecting the quality of construction, since there are known disasters that took place during the destruction of the arched dam of Malpase in France or during a landslide in the reservoir of the arched dam of Vayont in Italy, the destruction of the earthen dam of Titon in the USA. In 2017, there was a panic in California (USA) when the threat of destruction of the Oroville Dam appeared; more than two hundred thousand residents were evacuated from the settlements near the dam. This indicates the need to meet the requirements for the safety of dams, including the safety of earth dams.

The current state of the theory of seismic resistance of hydraulic structures erected from local materials is characterized by the use of refined prerequisites and reliable information regarding the choice of design models that take into account real geometry, design features, piecewise heterogeneous soil properties that change under the influence of filtration flows, the nature of static, hydrostatic and dynamic influences, features of the stress-strain state of the structure. Revealing the main regularities of the stress-strain state and behavior of hydraulic structures under seismic influences, taking into account the above factors, will make it possible to create and carry out effective anti-seismic measures that ensure the trouble-free operation of currently operating and projected earth dams[1-4].

Scientists of the national school of mechanics of Uzbekistan made a great contribution to the development of calculation methods and the study of the statics and dynamics of earth dams[5-16]. In these works, on the basis of flat and spatial design schemes, issues related to the seismic resistance of the designed and operated structures in the regions of Uzbekistan subject to high seismic risk are considered. Various soil properties are taken into account - plasticity, moisture content, nonlinear properties, as well as structural features of structures.

As you know, the spectral method for calculating structures for seismic effects, regulated by regulatory documents, is performed for conditional seismic loads determined using the assumption of elastic deformation of a cantilever model with masses concentrated along the height[17-18]. Such a model justifies itself when calculating high, compact in terms of structures, the vibration modes of which have a pronounced bending character. Oscillations of massive earth dams are more complex in nature and include not only horizontal shear, but also vertical displacements, which cannot be determined using the cantilever model. Therefore, in the cited works, calculations of the static and dynamic behavior of ground hydraulic structures (ground dams) were performed using:

- flat (plane-deformable) model (Fig. 1 a), representing the cross-section of the dam [7];

- spatial model [8-9], representing the real body of the dam (Fig. 1.b);

- as well as a model consisting of two intersecting planes, one of which represents the transverse, and the second - the longitudinal section of the dam, which coincides with the cross section (Figure 1 c).

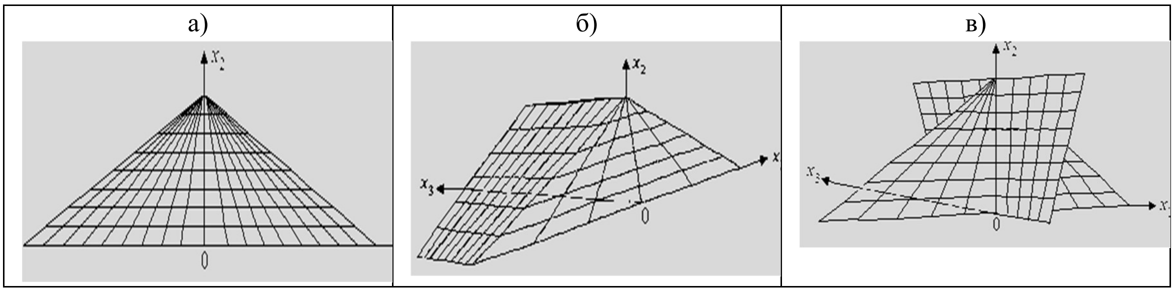


Fig.1. Calculation models of a soil dam on a rigid foundation

The lower edges of the design models (Figure 1) are fixed, i.e. the base of the dam is assumed to be absolutely rigid. However, to take into account uneven deformation, subsidence, bulging of a part of the soil or any other negative manifestation of a weakened foundation, construction models are used together with a foundation that has sliding side faces (Figure 2), i.e. only the vertical displacement of the infinite strip of the base is taken into account.

Such models were considered in [10], where the influence of a weakened fractured area on the stress-strain state of a high soil dam and the surrounding rocky base with a weakened fractured area was studied, the soil parameters of which were obtained from the results of experimental drilling. The performed studies had a practical way out for issuing recommendations for further increasing the height of the considered soil dam, taking into account the data of experimental drilling.

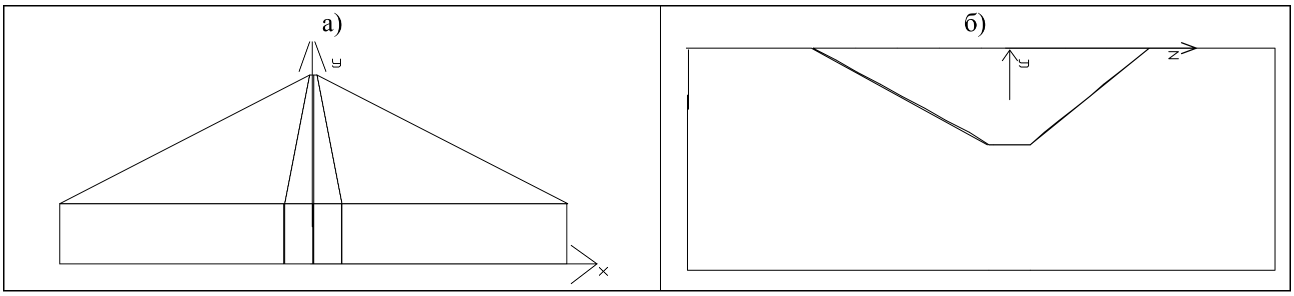


Fig.2. Dam models with base: transverse - a; longitudinal (in the canyon) - b

When studying the stress-strain state and dynamic behavior of an earth dam using the above models for static, hydrostatic and dynamic actions, a numerical method is used, in most cases it is a finite element method. This method allows one to take into account the real geometry, design features of objects and various properties of soils, manifested during loading and wetting [10-11], i.e. plasticity, fracturing, as well as geometric and physical nonlinearity. To take into account the nonlinear properties of the material (soil), special calculation methods were developed based on the solution of linear and nonlinear systems of algebraic, differential and integro-differential equations of high order.

Investigation of the stress-strain state of soil structures (dams) is an extremely difficult task, since the deformative properties of the soil depend on many factors: the current average stress; stress deviator component; applied load; humidity; degree of destruction, etc. Nonlinear laws of soil deformation in a structure, implemented in the specified calculation programs, include taking into account: structural destruction during volumetric deformation; humidity; geometric (for high dams) and physical nonlinearity under loading, as well as dry and viscous friction. The developed methods of static and dynamic calculation and the solution of complex problems on VAT and the dynamic behavior of soil structures made it possible to analyze the influence of hydrostatic pressure, viscosity, nonlinear deformation and soil moisture on the VAT of specific soil dams in Uzbekistan[12-14]. Analysis of the zones of possible destruction arising in dams under various static and dynamic influences allows a reasonable and economical approach to the operation of the selected soil protective structures - this determines the relevance and practical value of the work carried out.

II. Method

To create a mathematical model, use [19]:

1) - variational principle of minimum total energy of the system:

$$\delta\Pi - \delta' W = 0 \qquad (1)$$

where $\delta\Pi$ - system potential energy increment, and $\delta' W$ - the sum of the work of external forces on possible displacements;

2) - equation of state expressing the relationship between the components of stresses σ_{ij} and deformations ε_{ij} for an elastic medium

$$\sigma_{ij} = \lambda\varepsilon_{kk}\delta_{ij} + 2\mu\varepsilon_{ij} \qquad (2)$$

3) – Cauchy relations connecting deformations with displacements

$$\varepsilon_{ij} = \frac{1}{2}\left(\frac{\partial u_i}{\partial x_j} + \frac{\partial u_j}{\partial x_i}\right) \qquad (3)$$

4) - boundary conditions on a rigid foundation, meaning the absence of possible displacements $\delta\overline{u}$:

$$\delta\overline{u} = 0 \qquad (4)$$

5) - external influence is represented by volumetric forces - $\overline{P}$ (weight), applied throughout the entire volume of the structure, and surface forces $\overline{f}$, acting on a part of the structure's surface and being, in essence, hydrostatic pressure.

Here $\overline{u} = \{u_i, u_j\}$ - horizontal and vertical movements of the point of the body with coordinates $\{x_i, x_j\}$; $\sigma_{ij}, \varepsilon_{ij}$ - stress and strain tensor components; $\lambda,\ \mu$ - Lamé constants.

For the numerical solution of the problem, the finite element method [6-16, 19-20] is used, which is currently one of the main numerical methods of solid mechanics. This method is invariant with respect to the geometry of the investigated object and the mechanical characteristics of materials (soil and concrete). In addition, the finite element method is characterized by the simplicity of taking into account the interaction of the structure with the external environment, various mechanical (static and dynamic) loads, and various types of boundary conditions.

Mathematically, the problem of unsteady forced oscillations is reduced to solving a matrix system of inhomogeneous differential equations of the second order with the right-hand side, which is a function of time

$$[M]\{\ddot{q}\} + [C]\{\dot{q}\} + [K]\{q\} = \{P(t)\} \qquad (5)$$

where $[M]$ – mass matrix; $[K]$- stiffness matrix; $\{P(t)\} = [M]\{\ddot{u}_0\}$; attenuation matrix $[C]$- describes the internal friction due to the viscosity of the medium. The question of choosing the type of matrix $[C]$ is discussed below..

The solution of the system of equations (5) with the corresponding initial conditions can be obtained by the Newmark method [20]. Newmark's method is based on expansions of $q(t_i+\tau)$ and $\dot{q}(t_i+\tau)$ in series in powers of τ (integration step):

$$q(t_i+\tau)=q_i+\tau\dot{q}_i+\frac{\tau^2}{2}\ddot{q}_i+\alpha\tau^3\dddot{q}_i\,;$$

$$\dot{q}(t_i+\tau)=\dot{q}_i+\tau\ddot{q}_i+\beta\tau^2\dddot{q}_i\,,$$

where α and β are chosen such that the unconditional convergence of the integration process is ensured: $\beta\geq0{,}5;\ \alpha\geq0{,}25(\beta+0{,}5)^2$.

The solution for nodal displacements q_{i+1} at the end of the i^{th} time step is determined from the algebraic system of equations [20]:

$$[A]\{q_{i+1}\}=\{P_{i+1}\},$$

which is solved by the Gauss method.

Here

$$[A]=[K]+[C]\beta/(\alpha\tau)+[M]/(\alpha\tau^2);$$

$$\{P_{i+1}\}=\{R_{i+1}\}+[M]\left[\frac{\{q_i\}}{\alpha\tau^2}+\frac{\{\dot{q}_i\}}{\alpha\tau}+\left(\frac{1}{2\alpha}-1\right)\{\ddot{q}_i\}\right]+[C]\left[\frac{\beta\{q_i\}}{\alpha\tau}+\left(\frac{\beta}{\alpha}-1\right)\{\dot{q}_i\}+\frac{\tau}{2}\left(\frac{\beta}{\alpha}-2\right)\{\ddot{q}_i\}\right],$$

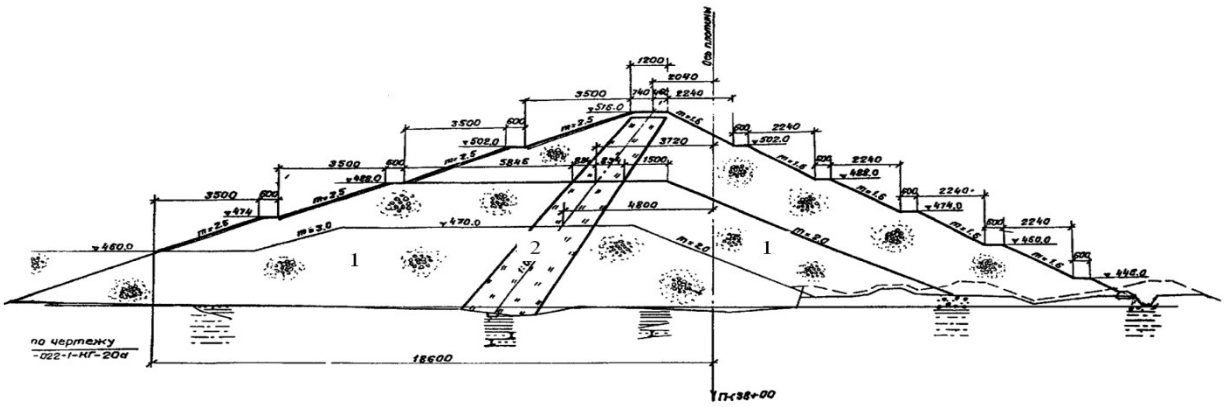
$\{q_i\}, \{\dot{q}_i\}, \{\ddot{q}_i\}$ - displacements, velocities and accelerations of the nodal points found in the previous time step.

The described algorithm of the Newmark method is applied to solving problems of unsteady forced vibrations of the considered soil dams, while the dynamic effect for each structure is selected individually.

The calculations were made on the example of the Rezaksay reservoir located in Rezaksay valley (a tributary of the Syrdarya river) in Chust district of Namangan region of the Republic of Uzbekistan. The dam, creating the reservoir, is made of a deaf earth embankment with an inclined core of sandy loamy soil and persistent prisms of gravel and pebble soils (Fig. 3). The maximum height of the dam in the channel part is 80 m, the length along the ridge is 3323 m, and the highest rise of the reservoir level is 77 m.

The thrust prisms of the dam are filled up from gravel-pebble soils with sand filling and rolling according to the density of dry soil ρ=2,1t/m^3. The embedding of the slopes of the dam was determined by calculations and are for the upstream slope m_1=2,5, for the downstream slope - m_2=1.9.

The inclined core is made of a mixture of loam and sandy loam with rolling up to a dry soil density of ρ=1,7t/m^3. The base of the core along the entire length of the dam is an array of siltstones and sandstones, covered with layers of soil of varying thickness, consisting of pebble, loam and sandy loam.



1-gravel-pebble soil with sandy filling; 2 - mixture of loam and sandy loam

Fig.3. Rezaksay earth dam

III. RESULTS

Consider unsteady forced vibrations using the example of the above soil dam. In this case, the initial conditions are taken to be homogeneous (zero): at $t=0$: $\{q_0\}=0$, $\{\dot{q}_0\}=0$.

Let the kinematic effect be represented as a harmonic function with the frequency of natural vibrations of the structure, and its duration is 2 sec. The entire time interval is 3-4 seconds:

$$\ddot{u}_0 = \begin{cases} A\sin(2\pi pt) & 0 \le t \le 2c \\ 0 & t > 2c \end{cases} \qquad (6)$$

After the termination of the impact, the free vibration mode is established in the structure. For 2 seconds, the structure in the resonant mode makes 6-7 vibrations, according to the analysis of which it is possible to draw the necessary conclusions about its dynamic behavior in the seismic process.

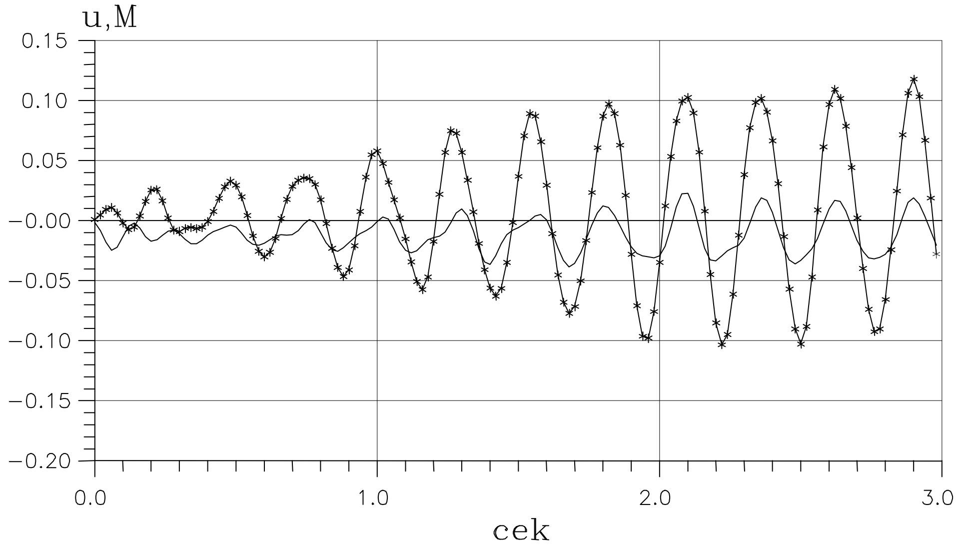
This kinematic effect with a frequency equal to the fundamental frequency of natural vibrations of the structure ($p=\omega_1$=3,1Hz) is used to demonstrate the dynamic behavior of the structure in a dangerous resonance mode. In addition, it should be noted that such a harmonic effect - with a period of T=0,05÷0,3 sec - can be classified as seismic, since its frequency range coincides with the frequency range of seismic effects. For example, the prevailing period of the 1976 Gazli earthquake was about T=0,1sec. Thus, an artificially selected impact can be a substitute for a real accelerogram.

In the absence of energy dissipation ([C]=0), system (2) describes the motion of an elastic dam without damping. The horizontal and vertical displacements of a point near the crest of the dam, which is under its own weight and under the indicated kinematic action, obtained in this case by the Newmark method under zero initial conditions, are shown in Fig. 4.a. The frequency of the kinematic impact (harmonic) at the base of the dam is equal to the frequency of natural oscillations of Rezaksay dam ω_0=3,1сек$^{-1}$, and the amplitude of the impact A=0,1 corresponds to the acceleration of the ground during a seven-point earthquake.

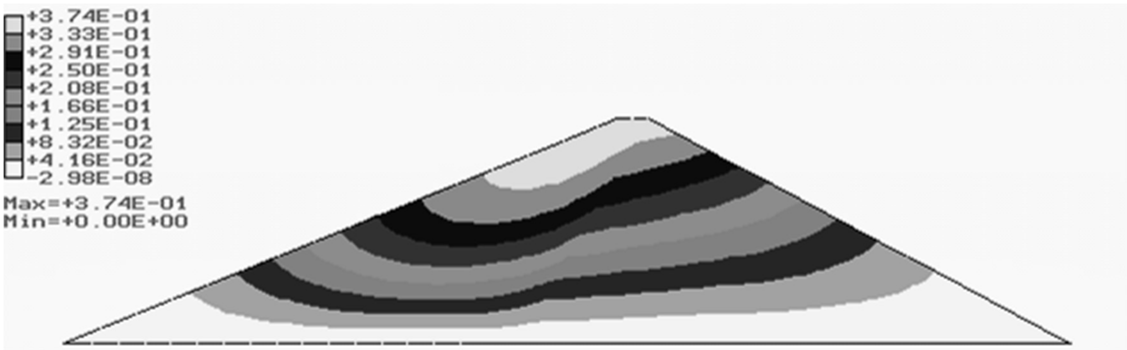
As expected, when exposed to a frequency equal to the fundamental frequency of natural vibrations of the structure, i.e. $p=\omega_1$, the latter oscillates with a linearly increasing amplitude, which by the end of the impact reaches 0,1 m (10 cm) for horizontal oscillations and 0.02 m (2 cm) for vertical ones. Such a linear increase in the amplitude when the frequency of the impact coincides with the fundamental frequency of the structure's oscillations (resonance) is predictable and proves the reliability of the results obtained according to the developed program.

The same figure also shows the distribution of maximum displacements (horizontal - Fig. 4.b and vertical - Fig. 4.c) over the entire section of the dam for the entire period of impact.

a)



b) distribution of maximum horizontal displacements



c) distribution of maximum vertical displacements

Fig.4. Horizontal (—*—*—) and vertical (———) displacements of the central point (a) and the distribution of the maximum horizontal (b) and vertical (c) displacements of the dam section over the entire period of impact

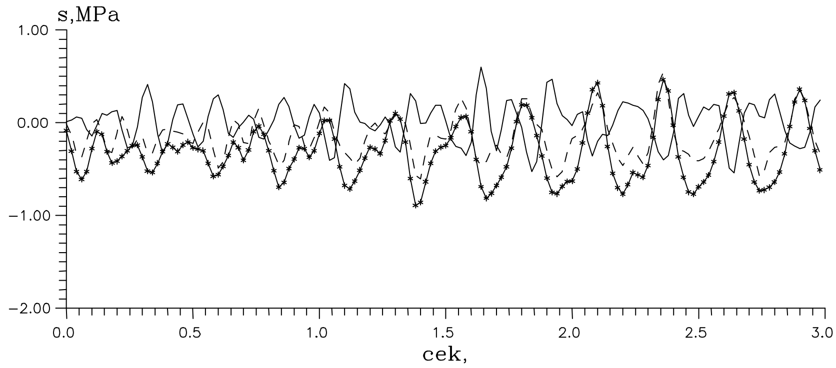
After the cessation of the impact, the dam goes into a mode of free oscillations, and vertical oscillations occur relative to the position of static equilibrium, determined by the weight of the structure (Fig. 4 - thin line).

Horizontal displacements at the first resonance, as the results of Figs 4b, c show, reach a maximum value at the crest of the dam and decrease uniformly towards the rigid foundation. The vertical displacements of the upper prism are positive, and the lower ones are negative, i.e. during a shift, the slope and the ridge zone of the upper prism move upwards and the part of the lower prism symmetrical to it falls down.

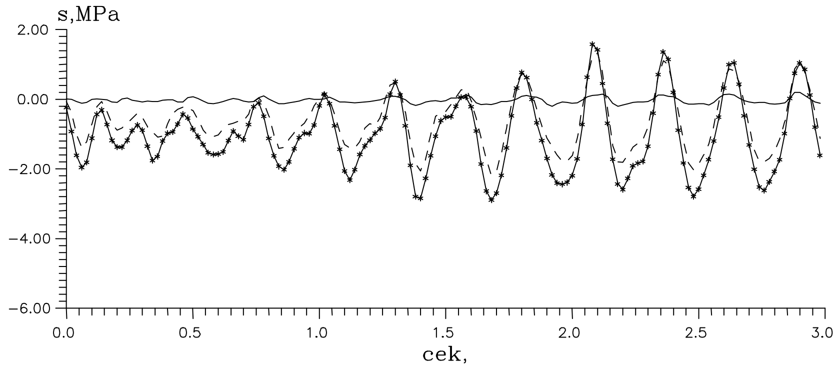
The results are shown in Fig. 4 obtained without taking into account dissipation in the soil, therefore, free vibrations of the dam occur with a constant amplitude.

Graphs of changes in normal - horizontal and vertical, as well as shear stresses for the entire considered interval are shown in Fig. 5.

a) horizontal stresses σ_x ((MPa) in the central section:



b) vertical stresses σ_y, (MPa) in the central section



c) shear stresses τ_{xy} (MPa) in the central section

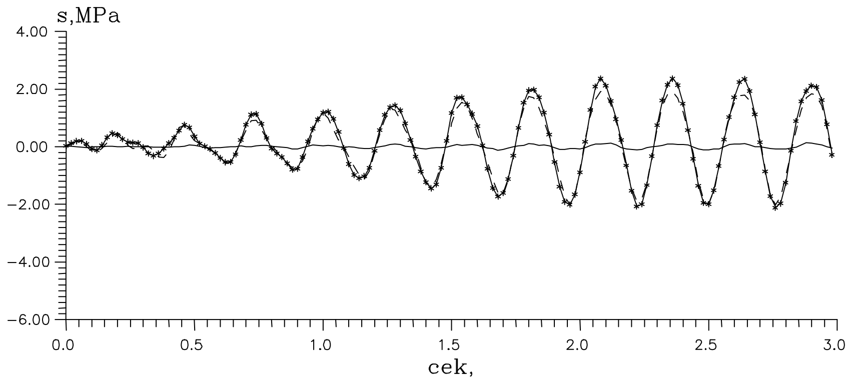


Fig. 5. Change in normal - horizontal (a) and vertical (b), as well as tangential (c) stresses at the points of the central section

(—*—*— - down; - - - admist; ——— up) at resonant vibration mode

IV. DISCUSSION

Graphs of stress changes over time demonstrate an increase in vertical (σ_y) and tangential (τ_{xy}) stresses in this mode during the entire time the load is applied. The maximum stress values (taken in absolute value) are observed at points near the base. Let us put it that the prevalence of negative values of vertical stresses means that the vibrations of the soil structure occur relative to the level of static equilibrium, characterized by the compression of the structure under its own weight. Over time, the amplitudes of vertical stresses at the bottom and in the center (line —*—*— and - - - - in Fig. 5b) go into a positive half-plane, which means the appearance of positive tensile vertical stresses here, weakening the bond between the soil and the base.

Next, we will consider the dynamic behavior of this soil dam, taking into account the dissipative properties of the soil under the same dynamic influences.

The reasons leading to energy dissipation can be caused by energy losses into the environment (“external” friction), as well as losses caused by internal processes in the material of the system (“internal” friction). In the first case, it is assumed that dissipative forces are proportional to inertial forces; the second case is associated with the viscous behavior of the material during deformation.

To describe the absorbing, dissipative properties of the soil, and to obtain a resolving system of equations, we apply a dynamic model of a Kelvin-Voigt viscoelastic medium

$$\sigma_{ij} = \lambda\theta\delta_{ij} + 2G\varepsilon_{ij} + \lambda'\dot{\theta}\delta_{ij} + 2G'\dot{\varepsilon}_{ij} \tag{7}$$

Where σ_{ij} - stress tensor components;

ε_{ij} , $\dot{\varepsilon}_{ij}$ - strain tensor and strain rate components;

λ, G - Lame's constants;

λ', G' - the corresponding viscosity coefficients of the medium;

$\theta = \frac{1}{3}(\varepsilon_1 + \varepsilon_2 + \varepsilon_3)$ $\dot{\theta} = \frac{1}{3}(\dot{\varepsilon}_1 + \dot{\varepsilon}_2 + \dot{\varepsilon}_3)$ - average strain and strain rate;

$\delta_{ij} = \begin{cases} 1, i = j \\ 0, i \neq j \end{cases}$ - Kronecker symbol.

The use of such a model in the calculations of structures made of soil materials for seismic effects makes it possible to take into account the energy absorption in the soil due to the viscosity of the material, friction between solid particles, the interaction between water and the soil skeleton, in case of irreversible plastic deformations, etc. In addition, this model makes it possible to evaluate absorbing capacity of soil structures depending on the frequency spectrum of the structure.

The description of viscoelastic behavior is achieved by presenting the components of deformations and mean deformations in a complex form $\varepsilon_{ij} = \varepsilon_{0ij}\exp(i\omega t)$, $\theta = \theta_0 \exp(i\omega t)$, as a result of which the rate of deformation and rate of change in volume will be equal to $\dot{\varepsilon}_{ij} = i\omega\varepsilon_{0ij}\exp(i\omega t)$, $\dot{\theta} = i\omega\theta_0 \exp(i\omega t)$.

In order to obtain an explicit expression for the dissipation matrix [C] included in equation (5), we transform (7) using complex modules

$$\lambda(i\omega) = \lambda - i\omega\lambda'; \qquad G(i\omega) = G - i\omega G'$$

Then equation (26) with complex moduli takes a form similar to Hooke's law

$$\sigma_{ij} = \lambda(i\omega)\theta\delta_{ij} + 2G(i\omega)\varepsilon_{ij}$$
,

and the explicit expression for the dissipation matrix [C] entering into equation (5) before the derivatives of the nodal displacements is obtained in the form

$$[C] = \eta[K]$$

where $\eta = \lambda' + 2G'$ - is a positive constant.

Thus, the use of this model in the finite element discretization of the structure leads to a resolving system of differential equations

$$[M]\{\ddot{q}\} + \eta[K]\{\dot{q}\} + [K]\{q\} = \{P(t)\} \quad (8)$$

where η - is a viscosity index.

In (8), the matrix of damping coefficients is proportional to the matrix of quasi-elastic coefficients; this case is called internal friction and is associated with the manifestation of the viscous properties of the material. If we transform (8) by multiplying it on the left by the matrix inverse to the mass matrix ($[M]^{-1}$), we get

$$\{\ddot{q}\} + \eta[M]^{-1}[K]\{\dot{q}\} + [M]^{-1}[K]\{q\} = [M]^{-1}\{P(t)\},$$

or, taking into account that $[M]^{-1}[K] = diag(\omega_i^2)$ - diagonal matrix of squares of natural frequencies - we obtain a system of separate equations –

$$\{\ddot{q}\} + \eta diag(\omega_i^2)\{\dot{q}\} + diag(\omega_i^2)\{q\} = [M]^{-1}\{P(t)\} \quad (9)$$

To select the value of η, we use the known data, according to which the following values $0{,}2 \le \psi \le 0{,}35$ are given for the coefficient of internal soil absorption ψ. Then from the formula connecting the coefficient ψ with the coefficients of friction (coefficients at the derivative of the displacements η) and frequencies (ω_i), we obtain

$$\psi = \frac{2\pi\eta_i\omega_i^2}{\omega_i},$$

from $\eta_i = \dfrac{\psi}{2\pi\omega_i}$ $\psi(0{,}2 \le \psi \le 0{,}35)$

Taking into account the range of change $\psi(0{,}2 \le \psi \le 0{,}35)$ and the spectrum of fundamental frequencies (3÷5.4Hz), we obtain the following limits of variation of the coefficient η for the soils of Rezaksay dam

$$0{,}006 \le \eta \le 0{,}0175,$$

whence we choose the average value η=0,01, which we will further use in calculating the dynamic behavior of a soil structure (Rezaksay dam) for a dynamic load. In this case, the impact, as before, is harmonic with a frequency that coincides with the fundamental vibration frequency of the structure. The initial conditions are zero. The method for solving system (8) is the Newmark method [20]. The result of the solution is the displacement of the nodal points of the structure, shown in Fig. 6.

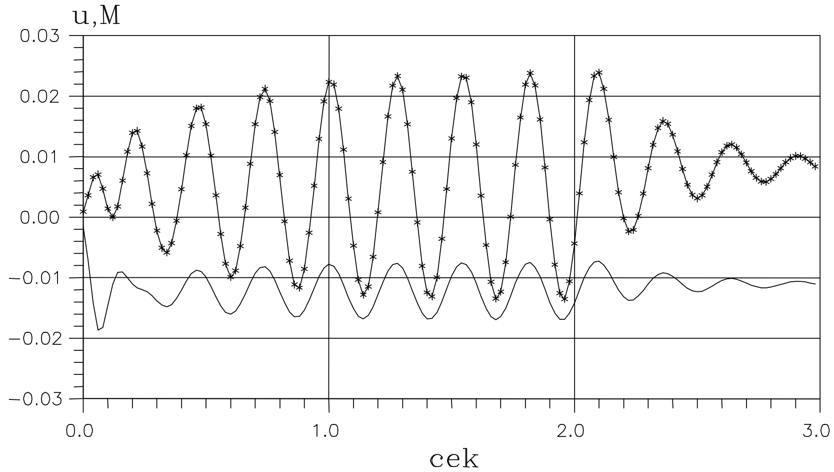


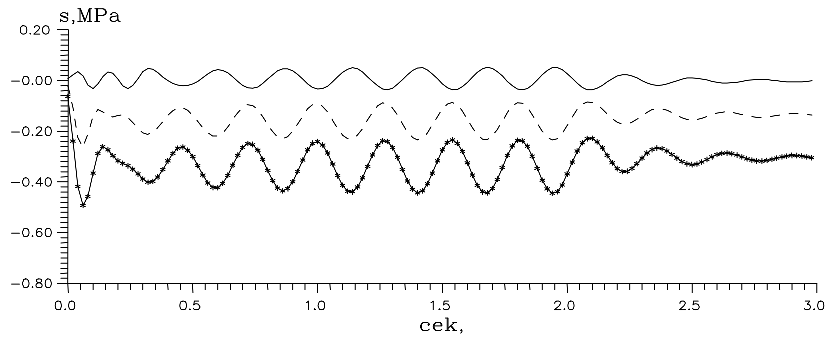
Fig. 6 Horizontal (—*—*—) and vertical (———) movements of the ridge of the Rezaksay dam taking into account internal friction

A comparative analysis of the behavior of an elastic dam (Fig. 5) and a dam with internal friction in the material (Fig. 6) shows that in the second case, both horizontal and vertical oscillations occur with amplitude that is significantly lower in magnitude than the amplitude of the elastic case. Moreover, even during the kinematic action, the amplitude of the dam's oscillations with internal friction does not constantly increase. This indicates a change in the frequency spectrum of a structure with damping properties of the soil. In other words, the absence of resonance in this case, i.e. unlimited growth of the amplitude of oscillations, indicates a change (decrease) in the fundamental frequency of the system, caused by rheological processes in the soil of the dam.

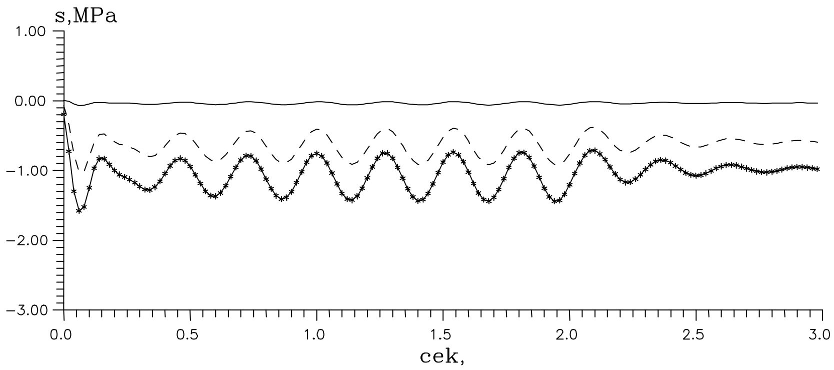
Horizontal vibrations occur relative to the neutral position (line—*—*— in Fig. 6), and vertical vibrations - relative to the position of static equilibrium determined by the displacement of the structure under its own weight (line ——— in Fig. 6). After the termination of the load (*t*>2 sec), the oscillations decay rapidly, and the deformed state of the dam at the end of the process is characterized by a horizontal axis shift (horizontal ridge displacements are set above the neutral line), and vertical settlement at the level of static equilibrium.

The change in the stress state components under kinematic action with the frequency of the main vibrations of an elastic structure, taking into account internal friction in the soil, are shown in Fig. 5.

а)



б)



в)

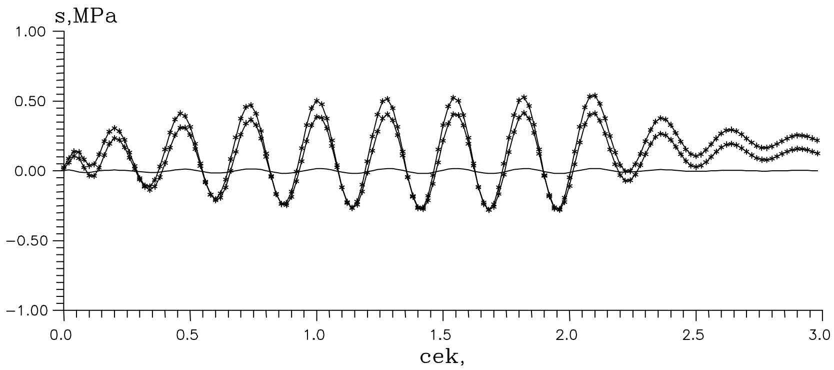


Fig.7. Change in horizontal (a), vertical (b) and tangential (c) stresses in the central section (—*—*— down; - - - amidst; —— up) of Rezaksay dam with internal friction in the soil under harmonic action with the frequency of natural vibrations

Consequently, taking into account the internal friction in the soil reduces the amplitude of vibrations and reduces the level of stresses in soil structures. In addition, the effect of weakening the connection between the structure and the base, noted in the elastic case, disappears, as indicated by the output of the amplitude of vertical displacements into the positive half-plane. Here (in Fig. 7 b) such a phenomenon is not observed.

V. Conclusion

The formulation of the problem of forced vibrations of a soil dam under dynamic action with and without taking into account the internal in soils is given; an algorithm for solving non-stationary problems by the numerical finite element method using the Newmark method is presented; Analysis of the numerical results - the dependences of displacements, stresses of vulnerable points of a soil structure in relation to the time of dynamic action showed that taking into account internal friction in the soil leads to a decrease in the amplitude of forced vibrations in sections.

References

[1] F. Zabala, R.D. Rodari,L.O. Oldecop, Seismic analysis of earthdams using a lagrangian particle method, The 14 World Conference on Earthquake Engineering October 12-17, Beijing, China xx (2008).

[2] V.B. Glagovsky, E.V. Kourneva, Numerical modeling of concrete face rockfill dam at seismic impact The 14 World Conference on Earthquake Engineering October 12-17, Beijing, China xx (2008).

[3] Salyamova K D., Rumi D.F., TurdikulovKh.X., Analysis of seismic stability of retaining earth structures with account of dissipative properties of soil. Europeansciencereview 11–12 November–DecemberVolume 1 Vienna 81-84 (2018).

[4] Goldin A.L., Rasskazov L.N., Design of soil dams. Moscow,.384 (2001).

[5] Dalmatov B.I., Soil mechanics, foundations and foundations. M .: Stroyizdat.319 (1981).

[6] Mirsaidov MM, Theory and methods of calculating soil structures for strength and seismic stability. Tashkent: Science., 312(2010).

[7] Mirsaidov M.M., Matkarimov P.Zh., Study of the stress-strain state of earth dams in a three-dimensional formulation Zh.FerPI, Fergana2**.**57-61 (2003).

[8] Mirsaidov MM, Matkarimov P.Zh., Yuldashev B.Sh., Investigation of the dynamic behavior of soil structures taking into account various deformation properties and water saturation of soils J. Earthquake-resistant construction. Safety of structures. Moscow**.** (2005).

[9] Rumi D.F., Salyamova K.D., Influence of rock fracturing on the stress-strain state of a soil dam., Zh. Bulletin of Tashkent State Technical University.**3**.253-256 (2015).

[10] Sultanov K.S., Mirzaev A.G., Model of deformation of collapsing soils taking into account moisture, DANRUz 7.32-35(1999).

[11] Sultanov K.S., Khusanov B.E., Equations of the state of collapsing soils taking into account moisture. Foundations, foundations and soil mechanics.**3**. 7-11. (2001).

[12] Salyamova KD, Rumi DF, Dynamics of a soil hydraulic structure taking into account the rheological properties of soil, Scientific-theoretical journal Bulletin of BSTU *ISSN2071-7316*.. **3**.16-35 (2015).

[13] Salyamova K.D. Rumi D.F., Dynamics of soil structures taking into account dry and viscous friction in the soil, Scientific-technical and production journal Gorny Vestnik of Uzbekistan.**1**.78-61 .(2015).

[14] Salyamova K.D., Rumi D.F., Dynamics of earth dams*,LAPLAMBERTAcademicPublishing,* 150 . (2015).

[15] Mirsaidov MM. Sultanov TZ, Rumi DF, Assessment of the dynamic behavior of the “structure-foundation” system taking into account the wave entrainment of energy Engineering and construction journal - Russia, St. Petersburg,**4.**94-105(2010).

[16] Salyamova KD, Rumi DF, Numerical analysis of the stress-strain state of the system "soil structure-foundation", J. of the Siberian Federal University. SibFU.Technics.Technology.**9**. 516-535(2016).

[17] *Guidelines for accounting for seismic effects in the design of hydraulic structures (to SNiP II-A.12-75). – L.: VNIIG named after Vedeneeva*,.164.(1977).

[18] *SHNK 2.06.11-04 Construction in seismic areas. Hydraulic structures. Gosarkhitekstroy .. Tashkent,54.*(2004).

[19] Zenkevich O. The finite element method in technology, Moscow, Mir.541 (1975).

[20] Obraztsov I.F., Savelyev L.M., Khazanov Kh.S., The finite element method in the problems of structural mechanics of aircraft. M .: Higher school, 392 (1985).