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NON-STATIONARY BEHAVIOR OF GROUND DAMS UNDER THE ACTION OF DYNAMIC LOADS

Abstract

Non-stationary dynamic behavior and stress-strain state of ground dam under kinetic action in the form of synthetic seismogram are studied in the paper by the method of finite differences. The regularity of deformation of dam material was taken as elasto-plastic one. Numerical results obtained are given in the form of graphs. The formation of residual deformations in the process of loading and zones undergoing plastic states are shown.

Introduction. Spectral method of design of earthquake-resistance of hydrotechnical structures such as ground dams is widely used in the practice of structure design in regions with high seismicity. However this method has a number of disadvantages one of them being not-accounting of real peculiarities of seismic influence as a dynamic one and the substitution of the latter by some conditional load. Using spectral method the designers get only an approximate assessment of reliability, the ways to principal reserves to increase an economical aspect of the constructions of hydrostructures are closed, as those reserves are revealed only when a wave nature of seismic influence is taken in account and are connected with a necessity of assumption of limited irreversible deformations of structures [1]. In [2,3] using the method of finite differences a stress-strain state of ground dams is theoretically studied with account of non-stationary processes occurring in the body of the dam. However a stress-strain state non-stationary behavior are considered only in initial moments of time during the process of dynamic loading, that is till the moment of time when a propagating wave reaches the crest of the dam. The problem is solved in plane statement, mainly when the material of the dam is in an elastic stage. The account of real (non-linear) characteristics of the ground opens a new approach to the prognosis of stress-strain state of the dam and it allows to project more economical and which is more important more reliable structures.

The authors of this paper have worked out in [4] a methods of numeric non-linear deformation of the material of the structure under dynamic loads within the frame of wave theory. In that paper they show the application of this method during several second in the process of dynamic loading.

The aim of the present paper is using the approach given in [4] to numerically study a dynamic behavior and stress-strain state of homogeneous ground dam with account of elasto-plastic deformation of the material of structure under kinetic influence (synthetic seismogram).

Statement of the problems. Let the prolonged uniform ground dam standing on a rigid foundation present a trapezoid outline in cross section. We will assume that from the moment of time $t \geq 0$ the dynamic action from the foundation begins to act on this dam. If an acting kinematic action is taken in the form of synthetic seismogram the law of change of rigid foundation has the following form

$$U_x = A_x \exp(-at)\sin(2\pi t/T), \quad U_y = B_y \exp(-bt)\sin(2\pi t/T) \quad \text{at} \quad t \geq 0 \qquad (1)$$

here A_x and B_y are the amplitudes of maximum horizontal and vertical displacements, a and b-parameters characterizing the degree of damping of these amplitudes, T - time of action load.

The equation of motion for plane-deformed ground dam in Cartesian system of coordinates has a form

$$\rho\frac{dv_x}{dt}=\frac{\partial S_{xx}}{\partial x}+\frac{\partial P}{\partial x}+\frac{\partial \tau_{xy}}{\partial y},\qquad \rho\frac{dv_y}{dt}=\frac{\partial S_{yy}}{\partial y}+\frac{\partial P}{\partial y}+\frac{\partial \tau_{xy}}{\partial x},\tag{2}$$

here ρ is a density of the material of the dam, v_x and v_y - velocities of the particles, P - hydrostatic pressure, S_{xx},S_{yy},τ_{xy} - deviators of the tensors of stresses. Total stresses σ_{xx}, σ_{yy} and σ_{zz} respectively are calculated from the relations

$$\sigma_{xx}=S_{xx}+P,\ \sigma_{yy}=S_{yy}+P,\ \sigma_{zz}=S_{zz}+P.\tag{3}$$

The regularities of deformation of ground dam material are taken in the form of elastic-plastic law [5,6]:

$$\frac{dP}{dt}=K\left(\frac{dV}{dt}\right)\Big/V,\tag{4}$$

$$\frac{dS_{xx}}{dt}+\lambda S_{xx}=2G\left(\frac{d\varepsilon_{xx}}{dt}-\frac{dV}{3Vdt}\right),\ \frac{dS_{yy}}{dt}+\lambda S_{yy}=2G\left(\frac{d\varepsilon_{yy}}{dt}-\frac{dV}{3Vdt}\right),\tag{5}$$

$$\frac{dS_{zz}}{dt}+\lambda S_{zz}=2G\left(0-\frac{dV}{3Vdt}\right),\ \frac{d\tau_{xy}}{dt}+\lambda\tau_{xy}=2G\frac{d\varepsilon_{xy}}{dt},$$

here $V=\rho_0/\rho$ is a relative volume, $\varepsilon_{xx},\varepsilon_{yy},\varepsilon_{xy}$ - components of deviators of stress, G and K - modulus of shear and volume compression respectively, λ - functional defined by Mises-Schleiher condition of yielding:

$$2J_2=S_{xx}^2+S_{yy}^2+S_{zz}^2+2\tau_{xy}^2\le 2\frac{Y^2(P)}{3},\tag{6}$$

$$\lambda=\left(2GW-dJ_2/dt\right)/\left(2J_2\right),$$

$$W=\sum S_{ii}\left(\frac{d\varepsilon_{ii}}{dt}-\frac{dV}{3Vdt}\right)+\tau_{xy}\frac{d\varepsilon_{xy}}{dt},$$

here $\lambda=0$ at $J_2<Y(P)^2/3$, $\lambda>0$, at $J_2=Y(P)^2/3$.

Here $Y(P)$ is a generalized condition of yielding and according to [5,6] is taken in the form

$$Y(P)=Y_0+\frac{\mu P}{1+\mu P/\left(Y_{PL}-Y_0\right)},$$

here Y_0- cohesion, μ - coefficient of friction, Y_{PL} - limit value of shear strength of ground.

The connection between the tensor of velocities of deformation and velocities of the particles is fulfilled by the relation:

$$\frac{d\varepsilon_{xx}}{dt}=\frac{\partial v_x}{\partial x},\quad \frac{d\varepsilon_{yy}}{dt}=\frac{\partial v_y}{\partial x},\quad \frac{d\varepsilon_{xy}}{dt}=\frac{1}{2}\left(\frac{\partial v_y}{\partial x}+\frac{\partial v_x}{\partial y}\right).\tag{7}$$

Using Eulerian variables it is necessary to add an equation of continuity to make a closed system of equations (2)-(7):

$$\frac{dV}{dt}=V\left(\frac{\partial v_x}{\partial x}+\frac{\partial v_y}{\partial y}\right).\tag{8}$$

Before the application of dynamic load (1) ground dam is taken as unstressed, unstrained one and in rest. If take this as an initial conditions, the system of differential equations (2)-(8) with free from stresses boundary conditions on slopes and crest and with condition (1) on the lower surface of the dam describes a complete picture of non-stationary behavior of ground dam.

Method of solution. Discussed problem is solved by the method of finite differences. In contrast to [2,3] we will use a finite differential scheme described in [7] and worked out in [4,8] for discussed types of problems. Here the following should be stated. In [4,6,8] the scheme [7] meant for the equations in Eulerian variables is used. As it is well known in Eulerian spaces the displacement of the particles are defined by the positions of the points in current and initial moments of time. That is why the coordinates of points of lower surface of dam in current moments of time are defined by relation

$$x = U_x + x_0, \;\; y = U_y + y_0,$$

here x_0, y_0 are initial coordinates of lower surface of dam, U_x, U_y- displacements defined by an expression (1).

Calculating the stresses by step method in [7] the values of increment of components of deformation (not a velocity of deformation) are used. So the connection of deformation with kinetic parameters in (7) is fulfilled by infinitely small increment. In this case the correlation (7) could be used to describe great deformations, summing up infinitely small changes. This shows an advantage of worked out method [4,8] comparing with [2,3] in study of dynamic behavior of ground dam. In conclusion we should state that the reliability of the method of design is substantiated in [8] by the solution of test example which shows a good coincidence of numeric result with an exact solutions for quadrangular structures.

Numeric results and their analysis. Consider the results of design on dynamic behavior on the example of Charvak ground dam under the action of loads (1). Geometrical characteristics of the dam were taken as following: the height of the dam of canal bed section equals 168m, the width of the crest- 12m, the steepness of the back of a dam- 1:2, and lower part- 1:1.9. Physical and mechanical characteristics correspond to an initial data: the density of the dam material equals 2300 kg/m^3 the velocities of longitudinal and cross wave propagation respectively 1500 m/s and 625 m/s the cohesion of the ground G/600, coefficient of the angle of friction 0.4, limit value of shear strength of dam material $22Y_0$. Here an averaged physical and mechanical characteristics of channel of the dam are given according to the results of Engineering Seismometric Service of the Institute of Seismology, Academy of Sciences of the Republic of Uzbekistan.

Placing the beginning of the count of Cartesian coordinates on a lower point of the back of the dam and directing Ox axis the foundation and lower surface of the dam we will trace the dynamic behavior in the points situated in near-crest back zone (A), in back zone near the foundation (B), along the center of core near the foot of the dam foundation (C) and in the center of the core of the dam (D). Coordinates of these points are taken as equal: A{329m, 157m}, B{98m, 37m}, C{342m, 37m} and D{342m, 80m}. Numerical results obtained are presented in graphs (fig. 1-6). First of all the problem was solved under the action in vertical direction only, that is $A_x = 0$, $B_y = 0{,}25$, $b = 1$ and $T = 0{,}5$ sec in (1), the solutions are given in fig. 1-2. Fig. 3-6 correspond to solutions at $A_x = 0{,}15$ and $a = b$ and parameters of vertical action were not changed. Curves 1-4 fig. 1-5 correspond to points A,B,C and D respectively.

Fig.1 gives the changes of horizontal (a) and vertical (b) stresses in time, it shows the appearance of insignificant stresses in slope zone near the free surfaces of the dam. Here maximum stresses in modulus reach 2MPa. Significant stresses occur in central core zone of the dam (curves 1,2, fig.1). Maximum values of the amplitudes of the stresses appear near the foot of the foundation along the center of the core in initial moments in the process of loading. Later with time duration these stresses become less.

A great interest is risen by the occurrence of residual deformations. Changes of horizontal (a) and vertical (b) deformations in time are shown in fig.2. The values of deformations in all discussed points of the dam occur in initial moments of time. With duration of time their changes become stable causing here residual deformations. Maximum horizontal residual deformations are in near-crest back zone (curve 1, fig. 2,a). These deformations become negative, that is the loosening of dam ground against the direction of coordinate axis Ox occur. Insignificant residual deformations appear in lower parts of the dam including the back zone of the dam. In central part of the dam as well as in near-crest zone a residual deformations are significant (curve 4, fig. 2,a). The same picture is observed in changing of vertical deformations in time (fig. 2,b). Here the greatest compacting of the ground occur in near-crest zone of the dam. As it is seen from

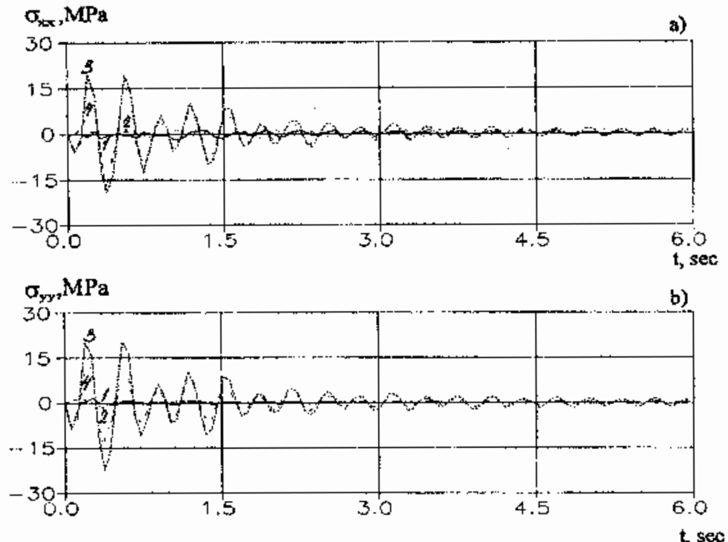

Fig.1

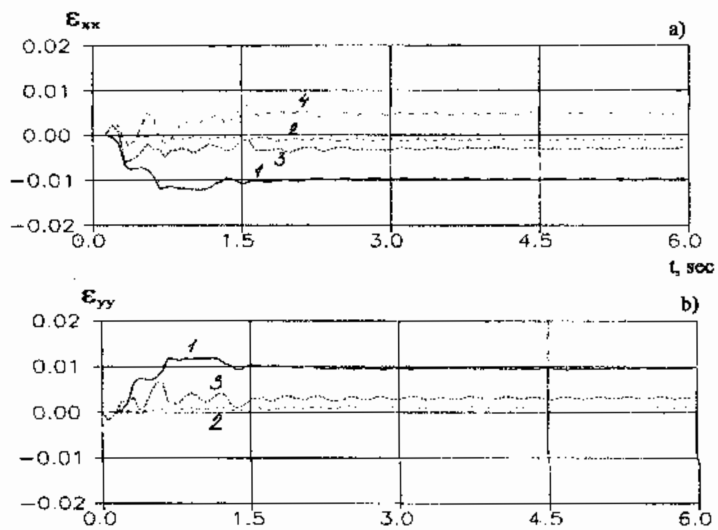

Fig.2

[Salyamova K.D., Khusanov B.E.]

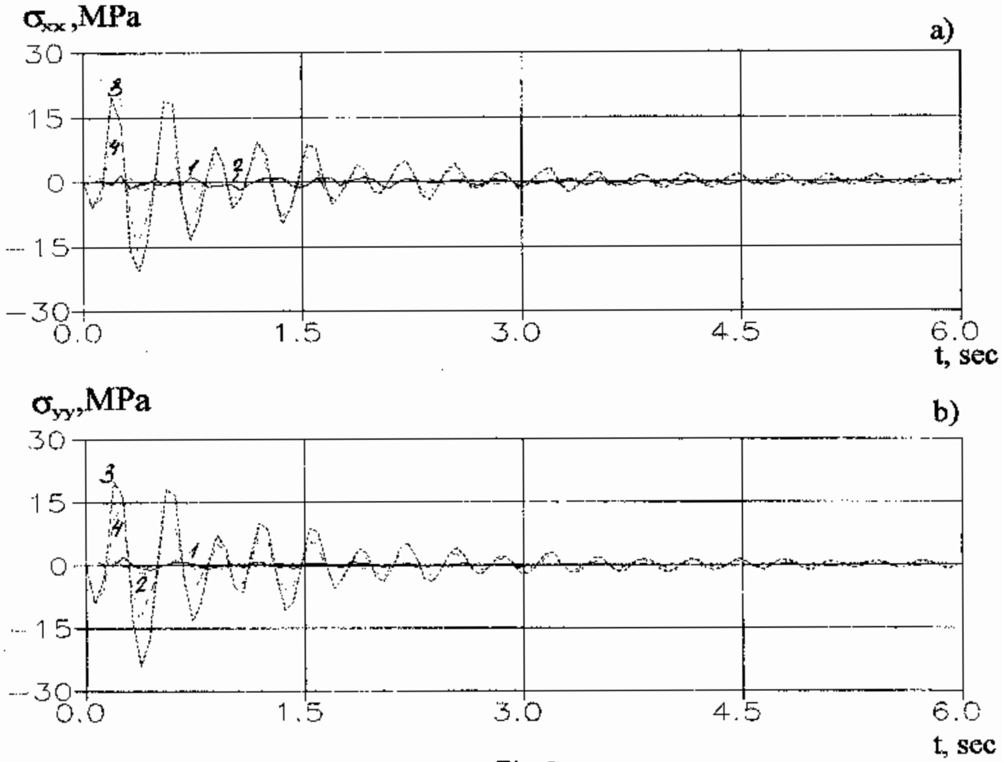

Fig.3

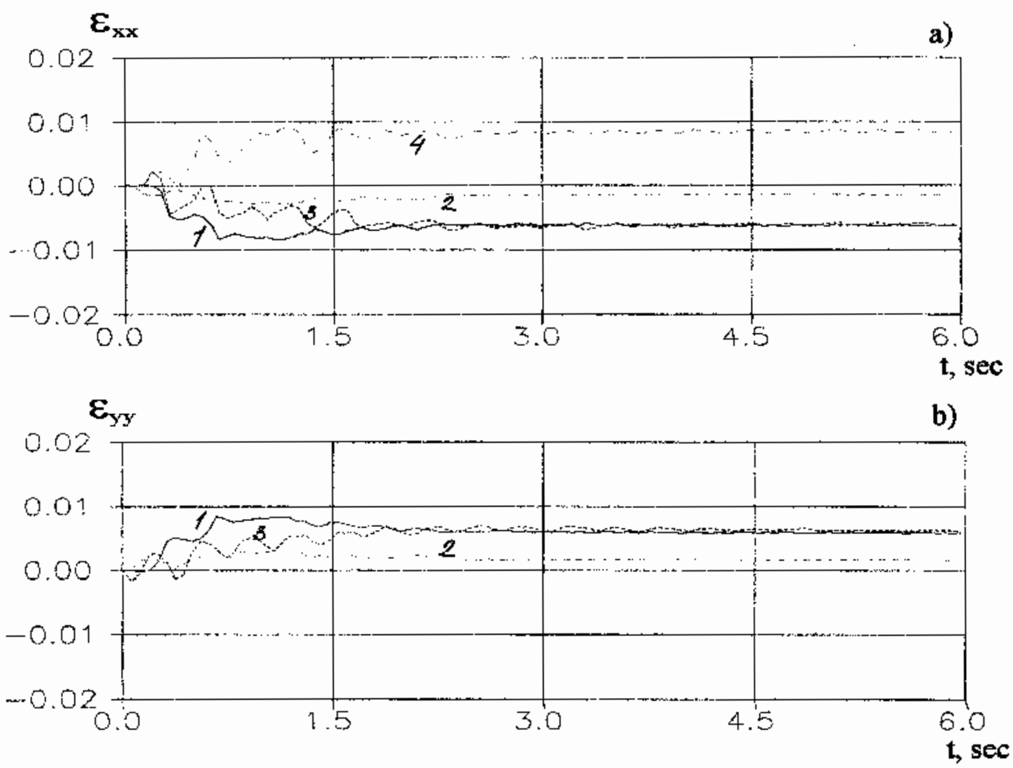

Fig.4

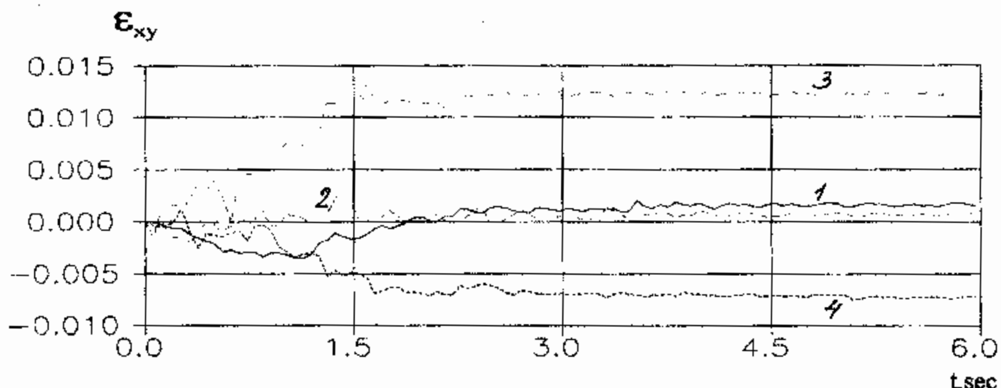
ε_{xy}
0.015
0.010
0.005
0.000
−0.005
−0.010
0.0
1.5
3.0
4.5
6.0
t,sec
1
2
3
4

Fig.5

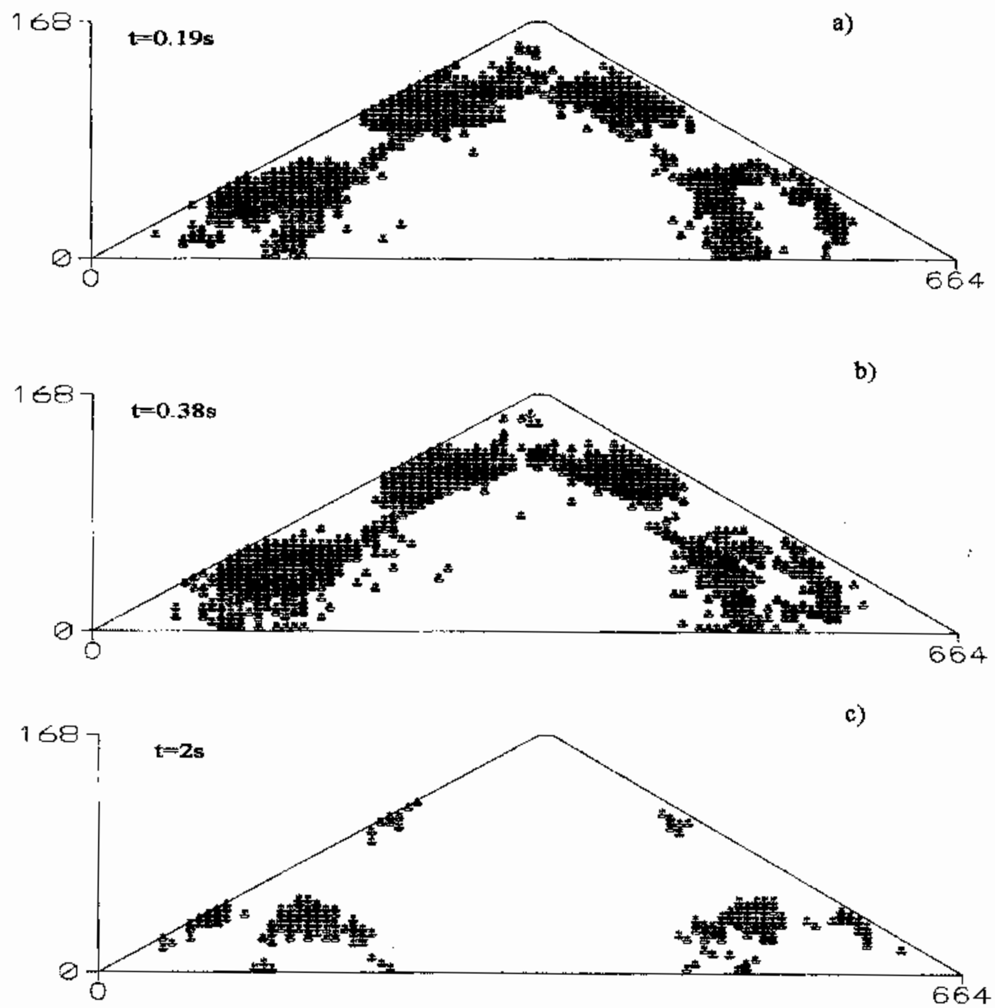
a)
t=0.19s
168
0
0
664
b)
t=0.38s
168
0
0
664
c)
t=2s
168
0
0
664

Fig.6

fig. 1 and 2 even without the motion along the axis Ox given by boundary conditions in (1) there appear a horizontal components of stress and strain. Naturally it is caused by the complexity of the dam (the waves are reflected from slope boundaries).

By the same way the stresses and strain are changed in discussed points of the dam in time at a given values of foundation displacements in both directions (fig.3 and 4). Here maximum values of the amplitude of stresses appear in central part of the dam, and significant residual deformations- in near-crest zone of the back of the dam. The dependence of shear deformation in time in observed points A,B,C and D is shown in fig. 5. According to the results of design on the back slope zone of the dam appear an insignificant shears. The greatest shear occurs in the ground situated in the center of the core.

Design results and fig.1 and 3 show that maximum values of stresses in discussed points appear in the $t = 0{,}19$ sec (compressing) and $t = 0{,}38$ sec (tensile ones). From the moment of time $t = 2$ sec the values of stresses are intensively damping and curve shape $\varepsilon_{xx}(t)$, $\varepsilon_{xy}(t)$ and $\varepsilon_{yy}(t)$ become stable. In fig.6 the zones of formation of plastic flows in the moments of time $t = 0{,}19$ sec (fig.6,a), $t = 0{,}38$ sec (fig.6,b) and $t = 2$ sec (fig.6,c) are given. As it was assumed an extensive zones of plastic flows appear in initial moments of time (fig.6,a and b). Plastic characteristics of ground at these moments of time develop in slope zones of the dam. With duration of time stress state of the dam is damping and so the zone of particles undergoing plastic flow is decreasing (fig. 6,c). Thus under the action of dynamic load in the form of synthetic seismogram (1) on ground dam from the foundation the most dangerous are the initial moments of time as it could influence the stability of the slopes.

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