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B. E. Khusanov

Institute of Mechanics and Seismic Stability of Structures of Academy of Sciences of Republic of Uzbekistan, Uzbekistan

S. I. Umarkhonov

Institute of Mechanics and Seismic Stability of Structures of Academy of Sciences of Republic of Uzbekistan, Uzbekistan

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B.E. Khusanov
Institute of Mechanics and Seismic Stability of Structures of Academy of Sciences of Uzbekistan
Academgorodok, Tashkent -Uzbekistan-700147

S.I. Umarkhonov
Institute of Mechanics and Seismic Stability of Structures of Academy of Sciences of Uzbekistan
Academgorodok, Tashkent -Uzbekistan-700147

ABSTRACT

Ground dams as it is well known have a complex geometry and physical non-uniformity. Dams with inner (central) little-water-saturated obstacle in the form of a so-called core which is formed from little-water-saturated clayey soil are more frequent in Central Asia. Slope zones around this core are of water-saturated stone filled soils. Water infiltration, that is moisture content of soil leads to the change of strength parameters of the core material. Stress-strain state of ground dams is studied in this paper with account of moisture content of clayey parts of the dam on seismic action with records of real velosigrams. Dam material is modeled by elastic and elastic-plastic bodies. As this problem is a non-stational one, it is solved by finite-difference method. Based on the results of design a dynamic behavior of characteristic points of the dam (slope, crest and central parts) were defined at the moment of action of seismic load and zone of formation of plastic deformation.

INTRODUCTION

Long-term exploitation of ground dams situated in seismic region requires from researches the prediction of dynamic behavior of these dams on different dynamic and seismic actions. So knowing dynamic behavior of the dams we can determine their bearing capacity, strength of the slopes, etc. Dynamic methods of design of ground dam considering wave character of seismic action were worked out by many authors including Zaretskiy et al [1983], Lyather et al [1986]. Considering ground medium as a two-phase medium, these papers take into consideration moisture content of the ground. Solutions of such problems for different dams were obtained at initial time. In Khusanov et al [1998] using Wilkins' [1964] scheme the methods of dynamic design of ground dams on the action of different loads were worked out. In this method it is possible to apply more complex equations of state of ground bott as a two-phase and one-phase medium accounting moisture - content (for ground model presented in Khusanov [2000]). In this paper a dynamic behavior (stress-strain state) is numerically studied on the example of Charvak dam. The height of this dam is 168 m, the width - 12 m. The stepnesses of upper and lower slopes are 1:2 and 1:1,9 respectively at the centre of canal bed a core (an obstacle) is placed with a small-water-permeable characteristics in the form of trapezoid with the widthes of 200 m and 10 m (fig.1). When exploiting such ground dams in the body of these dams a filtration flow is formed. Rock filling of the dam shows a good water-filtration characteristics and its water saturation practically does not influence on mechanical characteristics and parameters of the rock mass. That is why filtration flow basically is formed on small-water-permeable core of the dam (curve AB, fig.1).

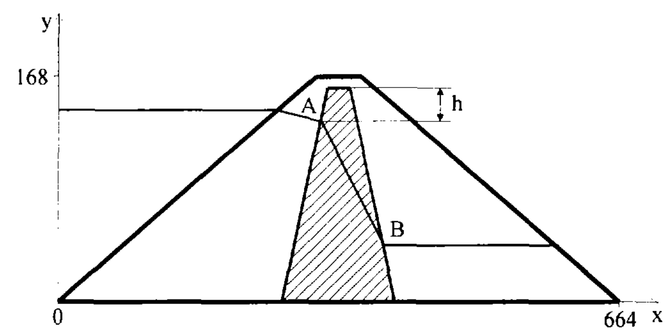


Fig.1. Cross Section of Charvak dam.

As the core of the dam consists of soft soils, their water saturation, that is moisture content considerably influences on physical and mechanical characteristics of ground. In Khusanov [2000] a model of deformation of structurally uninstable soils is offered which takes into account of the ground. Using this equation of state for the core of the dam let us consider a dynamic behavior of Charvak dam under the action of seismic load.

STATEMENT OF THE PROBLEM

We'll place the origin of Cartesian system of coordinates on the lower point of upper slope of the dam and direct the axis Ox along its foundation and lower surface. From the moment of time $t>0$ a dynamic action from the foundation begins to act on the dam. If accept acting effect according to the record of real velocigram of the earthquake on the entire lower surface of the dam the necessity no longer arises to stude the behavior and action of the dam foundation. An equation of motion of Plane - deformed dam has the following form

$$\rho\frac{dU_x}{dt}=\frac{\partial S_{xx}}{\partial x}+\frac{\partial P}{\partial x}+\frac{\partial\tau_{xy}}{\partial y},\quad \rho\frac{dU_y}{dt}=\frac{\partial S_{yy}}{\partial y}+\frac{\partial P}{\partial y}+\frac{\partial\tau_{xy}}{\partial x},\quad (1)$$

here ρ - is the density, U_x, U_y - velocities of the particles, P - pressure and $S_{xx}, S_{yy}, \tau_{xy}$ - components of deviators of stresses. Naturally full stresses are calculated by formulae

$$\sigma_{xx}=S_{xx}+P,\qquad \sigma_{yy}=S_{yy}+P,\qquad \sigma_{zz}=S_{zz}+P.\quad (2)$$

The model of deformation of rock filling of the dam is taken in the form of Grigoryan's [1967] equation:

$$P=\frac{K}{n}\left(\left(\frac{\rho}{\rho_0}\right)^n-1\right);\qquad (3)$$

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

$$\frac{dS_{yy}}{dt}+\lambda S_{yy}=2G\left(\frac{d\varepsilon_{yy}}{dt}-\frac{dV}{3Vdt}\right),\qquad (4)$$

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

giving more general dependence of the strength limit on the pressure in Mises's generalized condition

$$2J_2=S_{xx}^2+S_{yy}^2+S_{zz}^2+2\tau_{xy}^2\le 2Y(P)/3,\qquad (5)$$

$$Y(P)=Y_0+\frac{\mu P}{1+\mu P/(Y_{PL}-Y_0)},\qquad (6)$$

here K and G - are the moduli of volume compression and shear respectively, $V=\rho_0/\rho$ - relative volume, Y_0 - cohesion, μ - coefficient of friction, Y_{PL} - limit value of shear strength of rock filling, λ - functional defined by the following formulae

$$\lambda=0\qquad \text{at}\qquad J_2<Y(P)^2/3,$$

$$\lambda=\frac{2GW-dJ_2/dt}{2J_2}\qquad \text{at}\qquad J_2=Y(P)^2/3,\qquad (7)$$

here $$W=\sum_{j=1x,y,z}S_{jj}\left(\frac{d\varepsilon_{jj}}{dt}-\frac{dV}{3Vdt}\right)+\tau_{xy}\frac{d\varepsilon_{xy}}{dt}.$$

The core of the dam consists, in general, from loess soils as it was mentioned earlier the moisture content of these soils considerably influences on strength characteristics of the core material. According to Khusanov [2000] mechanical parameters of ground consider as the functions depending on the degree of moisture content in the following form

$$K(I_W)=K_{sat}\exp(\alpha_K(1-I_W)),$$

$$G(I_W)=G_{sat}\exp(\alpha_G(1-I_W)),\qquad (8)$$

$$c(I_W)=c_{sat}\exp(\beta(1-I_W)),$$

$$\mu(I_W)=\mu_{sat}\exp(\gamma(1-I_W))\qquad I_W=W/W_{sat},$$

Here K_{sat}, G_{sat}, c_{sat} and μ_{sat} - are moduli of volume compression and shear, cohesion force and coefficient of the angle of internal friction completely moistured ground respectively; α_K, α_G, β and γ - empirical dimensionless coefficients characterizing the degree of change of a corresponding mechanic characteristics of ground; I_W - parameter of the degree of ground moisture content; W- current ground moisture content; W_{sat} - moisture content corresponding to a complete water saturation of ground pores. So, the equation of state of the core of dam is taken the same as (3)-(5) with relationships (8) and the function of plasticity in the following form

$$Y(P,I_W)=c(I_W)+\mu(I_W)\cdot P\qquad (9)$$

To the system of equations (1)-(9) it is necessary to add the relationships, connecting the components of deformation velocities with mass velocities and equation of ground continuity (11)

$$\frac{d\varepsilon_{xx}}{dt}=\frac{\partial U_x}{\partial x},\qquad \frac{d\varepsilon_{yy}}{dt}=\frac{\partial U_y}{\partial y},\qquad (10)$$

$$\frac{d\varepsilon_{xy}}{dt}=\frac{1}{2}\left(\frac{\partial U_y}{\partial x}+\frac{\partial U_x}{\partial y}\right);$$

$$\frac{dV}{dt}-V\cdot\left(\frac{\partial U_x}{\partial x}+\frac{\partial U_y}{\partial y}\right)=0.\qquad (11)$$

To get a closed equations (1)-(11) it is necessary to add a differential equation of the process comparing with discussed dynamics of dam on the action of seismic loads is so slow (stationary) that according Mustafayev [1989] we may use the solution of this equation. First we determine the curve of depression of filtration flow. Let us assume that in discussed dam the curve of depression is a straight line AB passing in the core through the points of normal support level (A) and water level of lower spole (B) of the dam (fig.1). In a lower part of this line the moisture content is naturally equal to a complete water saturation of pores and in higher part-thanks to water capillary rise in ground - we obtain variable moisture content corresponding to the solution of stationary problem of the process of moisturizing of ground. Using formulae experimentally obtained by Mustafayev [1989], describing the numeric solution of a stationary problem.

$$W(y) = W_0 + (W_{sat} - W_0)\cos\frac{\pi y}{2h_k} \qquad (12)$$

giving the change of moisture content of ground within the limits of the height of water capillary rise from the line AB. In equation (12) W_0 - is a natural moisture content, h_k - the least height (fig.1). Boundary conditions are as follows: on the lower surface (y=0) of the dam the seismic load changing in time is given in the form of real velosigram recorded during Tashkent earthquake (1966) by Ivatscenco et al [1988] (fig.2); on the slopes and the crest of the dam the conditions are taken as free of stresses. Initial conditions are taken as zero.

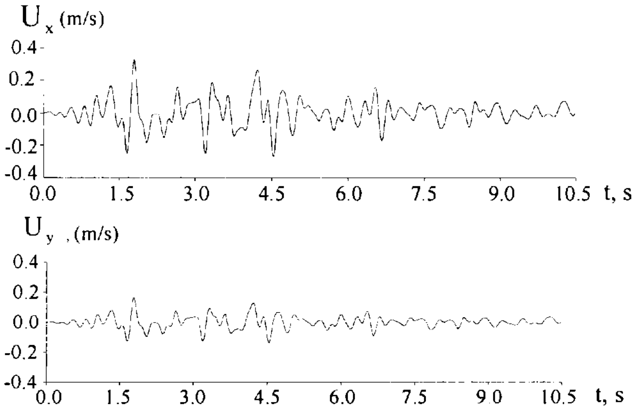
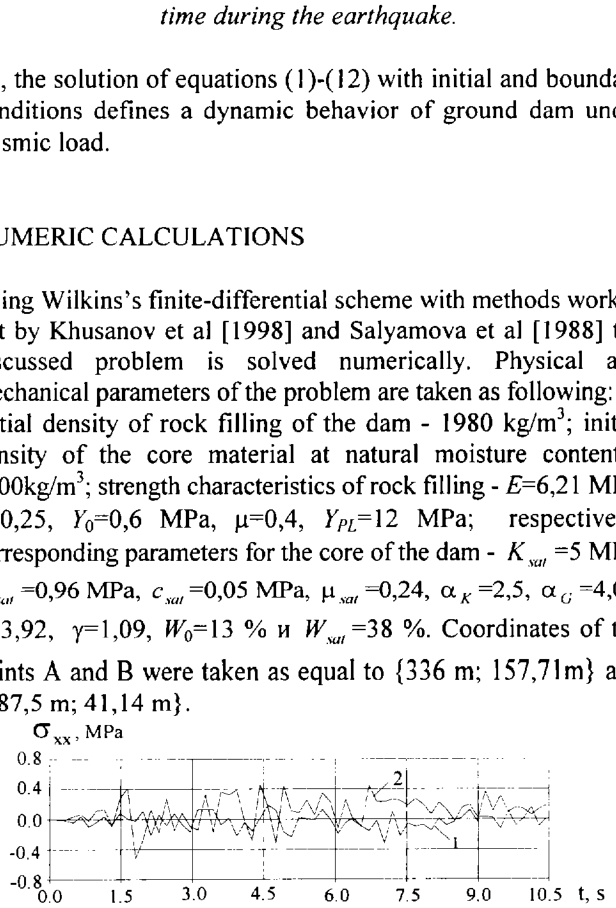


Fig.2. Change of velocity of foundation particles of the dam in time during the earthquake.

So, the solution of equations (1)-(12) with initial and boundary conditions defines a dynamic behavior of ground dam under seismic load.

NUMERIC CALCULATIONS

Using Wilkins's finite-differential scheme with methods worked out by Khusanov et al [1998] and Salyamova et al [1988] the discussed problem is solved numerically. Physical and mechanical parameters of the problem are taken as following: an initial density of rock filling of the dam - 1980 kg/m^3; initial density of the core material at natural moisture content - 1700kg/m^3; strength characteristics of rock filling - E=6,21 MPa, ν=0,25, Y_0=0,6 MPa, μ=0,4, Y_{PL}=12 MPa; respectively; corresponding parameters for the core of the dam - K_{sat} =5 MPa, G_{sat} =0,96 MPa, c_{sat} =0,05 MPa, μ_{sat} =0,24, α_K =2,5, α_G =4,02, β=3,92, γ=1,09, W_0=13 % и W_{sat} =38 %. Coordinates of the points A and B were taken as equal to {336 m; 157,71m} and {387,5 m; 41,14 m}.



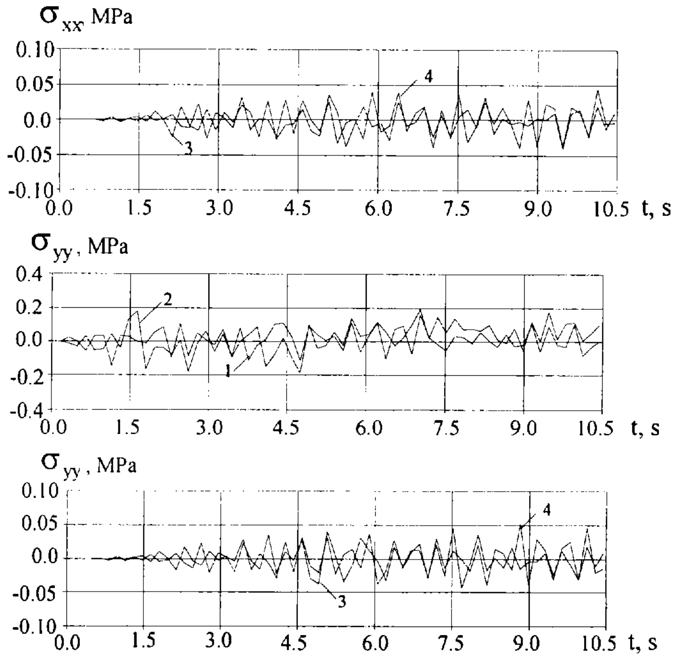


Fig.3. Change of stresses in time.

Coordinates of the points A and B were taken as equal to {336 m; 157,71m} and {387,5 m; 41,14 m}. Numeric results are given in the form of graphs. Figure 3 shows the change of horizontal and vertical stresses depending on stresses in characteristic fixed points 1-{329 m; 127 m}- in a near-crest upper slope zone; 2 -{98 m; 37 m} - upper slope zone near the foundation; 3 - {342 m; 37 m}- in the center of the core near the foot of the foundation; 4 -{342 m; 80 m}- in the center of the core of the dam. As it was expected maximum stresses under seismic action occur on rock filling of the dam. Here the values of horizontal stresses comparing with vertical ones are approximately two time greater. Because of decrease of strength parameters of the core of the dam, the stress state varies in the limits ~ 0.05 MPa. It should be stated that these stresses in the core of the dam in design on load action shown in fig.2 are pre-limit ones and plastic flow in material of dam core was not observed.

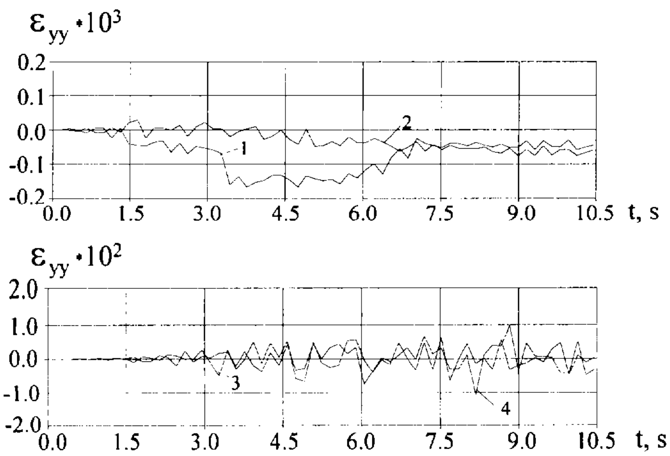


Fig.4. Change of deformation in time.

Figure 4. shows the change of vertical deformations in time in the same points shown in fig.3. Here from fig.3 it is seen that the state of rock filling of the dam becomes beyond-limit; and in slope and near-crest zones of the dam there appear residual deformations. Residual deformations in near crest are negative; that leads to compacting of the dam material. The greatest compactness along the vertical is reached in upper near-crest part of the dam. Changes of deformations in the core of the dam are non-stationary. Their values till a pre-limit state are considerably greater than the deformation values of rock filling of the dam.

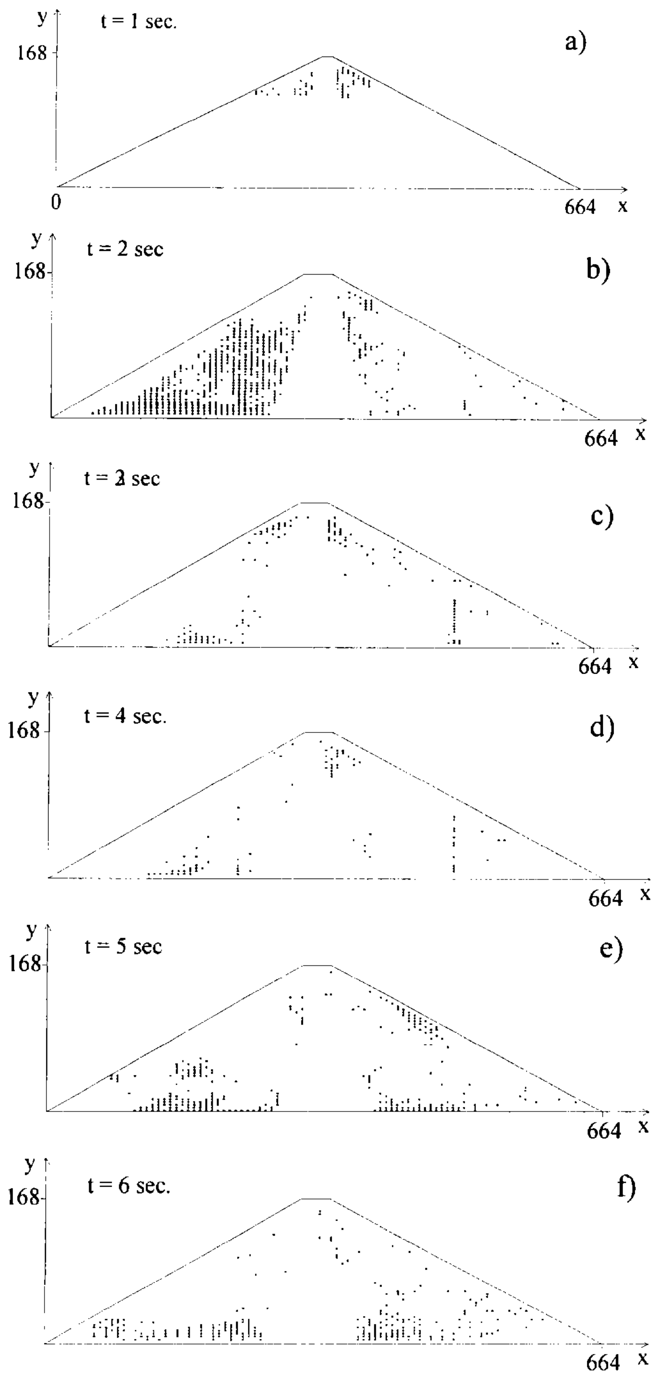


Fig.5. Zones of plastic flows.

Figure 5 shows those sections of the dam which undergo beyond-limit states at the moments of time t = 1, 2, 3, 4, 5 and 6 sec ((a)-(f)) respectively). As it is seen from fig.5 plastic deformation begins in near-crest zone (possibly because of reflection from free crest). At the moment t=2 sec (fig.5,b) the region of plastic deformation practically covers the whole upper slope zone. Then with time the occurrence of beyond limit states in the particles of the dam becomes less.

CONCLUSION

Design shows that stress-strain state at initial moments of time (approximately at t=2 sec) mostly unfavorable. High values of stresses and residual deformations and displacements in slope and near-crest zones of the dam may lead to destruction or soil slip of the slopes of the dam. To approve this, it is necessary to carry out design with account of hydrostatic and dynamic effect of water on upper slope of the dam and to take into consideration its own weight and the forces of gravity.

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