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ARAŞTIRMA MAKALESİ /**RESEARCH ARTICLE**

NUMERICAL APPROACH FOR TUPOLANG DAM INVESTIGATION

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ABSTRACT

The technique of strength and sustainability analysis of earth and underground hydro technical structures taking into account physical and mechanical parameters of soil properties and design features of Tupolang dam (Uzbekistan) offer in this issue. Structures behaviours under dynamic loading by elastic and nonlinear approach by 2D schemes are considered.

Keywords: Dam, Plasticity, Soil, Rock.

NÜMERİK YAKLAŞIMLA TUPOLANG BARAJININ İNCELENMESİ

ÖZ

Bu bildiride, zemin özelliklerinin fiziksel ve mekanik parametreleri ve Tupolang Barajının (Özbekistan) tasarım özellikleri göz önüne alınarak, yer üstü ve yer altı hidro teknik yapıların mukavemet ve dayanıklılık analizlerinin teknikleri sunulmaktadır. İki boyutlu elastik ve doğrusal olmayan yaklaşımlar, dinamik yüklemedeki yapı davranışları esas alınmıştır.

Anahtar kelimeler: Baraj, Plastisite, Zemin, Kaya.

1. INTRODUCTION

The majority of valleys in Uzbekistan are located in arid zones and, as the result, an irrigation farming agriculture is widely spread. Dams play the role of water accumulators. Nowadays the construction of new dams and the modernization of old ones are carried out. The territory of Central Asia is a seismic active zone and it is necessary to consider this factor in design and operation of dikes, dams, etc. The accumulated experience in monitoring of earth dams shows that the approach to the majority of aspects of the failure state are determined by features of the construction stress-strain state. Calculated estimations of stress-strain states are effectively used on a blueprint stage for the comparison and the choice of embodiments. Besides, it is important to provide the estimation of reliability and definition of the necessity of providing safety of weirs.

The account of real material properties in definition of stress-strain state of earth dam is related to investigation of non-linear soils properties that encloses the new approach to its

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prognosis and allows to design energy conserving and reliable characteristics of dams.

Tupolang dam, which is still under construction on the Surkhandarya River, has been considered under dynamic load, behavior of spillway outlet channel in rock massive was studied in static statement. In calculation of dam the non-linear (elastic-plastic) properties and soil saturation effect have been accounted.

2. DAMS INVESTIGATION

Statement of a problem

Geometrical parameters of dam are:
The dam height is 192.5m;
The dam length is 1000m;
The crest width is 10m;
Upstream escarpment is 1:1.96;
Downstream escarpment is 1:1.84.

Length of spillway outlet channel is 800м. The angle of the tunnel downgrade is supposed to be zero.

Expansion of the dam and channel allows considering them as the plane strain problem.

The common set of equations looks like:

Equations of motion in deviator form are

$$\begin{cases} \rho \dfrac{dV_x}{dt} = \dfrac{\partial S_{xx}}{\partial x} + \dfrac{\partial P}{\partial x} + \dfrac{\partial \tau_{xy}}{\partial y} \\ \rho \dfrac{dV_y}{dt} = \dfrac{\partial S_{yy}}{\partial y} + \dfrac{\partial P}{\partial y} + \dfrac{\partial \tau_{xy}}{\partial x} - \rho g \end{cases} \tag{1}$$

Here ρ is a soil density, P is a hydrostatic pressure, V_x, V_y are the mass velocities, g is acceleration due to gravity.

Table 1: Dam characteristics

Zones	Unit weight KPa/m^3	Poisson's ratio	Young's modulus KPa	Angle of internal friction
Loam core	16.677	0.3	1500	30
Transition filter	21.582	0.3	1500	40
Downstream and up-stream shells	21.582	0.3	1500	45

Constitutive relation of soil is accepted as (Grigoryan, 1967):

$$\frac{ds_{xx}}{dt} + \lambda S_{xx} = 2G\left(\frac{d\varepsilon_{xx}}{dt} - \frac{1}{3V}\frac{dV}{dt}\right)$$
$$\frac{ds_{yy}}{dt} + \lambda S_{yy} = 2G\left(\frac{d\varepsilon_{yy}}{dt} - \frac{1}{3V}\frac{dV}{dt}\right)$$
$$\frac{ds_{zz}}{dt} + \lambda S_{zz} = -\frac{2G}{3V}\frac{dV}{dt}, \quad \frac{\tau_{xy}}{dt} + \lambda \tau_{xy} = 2G\frac{d\varepsilon_{xy}}{dt}$$
$$\frac{dP}{dt} = -K(dV/dt)/V \tag{2}$$

Where $V=\rho_0/\rho$ is the relative volume; ε_{xx}, ε_{yy}, ε_{xy}, - the components of the strains deviator; G, K - shear modulus and bulk modulus respectively; λ is a functional, defined by von Mises-Shleicher (Grigoryan, 1967) generalized criterion:

$$2J_2 = S_{xx}^2 + S_{yy}^2 + S_{zz}^2 + 2\tau_{xy}^2 \le \frac{2Y(P)}{2}$$
$$\lambda = (2GW - dJ_2/dt)/(2J_2)$$
$$W = \Sigma S_{ij}\left(\frac{ds_{ij}}{dt} - \frac{1}{3V}\frac{dV}{dt}\right) + \tau_{xy}\frac{d\varepsilon_{xy}}{dt} \tag{3}$$

Where if $\lambda = 0$ then $J_2 < Y^2(P)/3$, and if $\lambda > 0$ then $J_2 = Y^2(P)/3$

Here Y (P) - the generalized yield criterion depending on pressure:

$$Y(P) = Y_0 + \frac{\mu P}{1 + \dfrac{\mu P}{Y_{PL} - Y_0}} \tag{4}$$

Where Y_0 – soil cohesion; μ is a friction coefficient; Y_{PL} - the limit of shear strength. At $Y_0 \to 0$ in (3) and (4) coefficient λ is equal to zero and the considered model transfers into elastic one. Cauchy relations carry out the tensor of strain velocities and mass velocities ratio.

$$\varepsilon_{xx} = \frac{\partial U_x}{\partial x}, \varepsilon_{yy} = \frac{\partial U_y}{\partial y}, \varepsilon_{xy} = \frac{1}{2}\left(\frac{\partial U_x}{\partial y} + \frac{\partial U_y}{\partial x}\right) \quad (5)$$

The system of the differential equations (1, 2, 5) and relations (3, 4) are closed and with initial and boundary conditions they describe the stress-strain state of the considered structure under dynamic loading. Initial conditions are accepted equal to zero. Boundary conditions are: crest and dip-slopes of a dam are stress free zones and the seismic load in view of real velocigram (Figure 1) (scenario earthquake) is applied to foundation. This velocigram was recorded by Hydroproject (Moscow) and Institute of Seismology (Tashkent) (Ivashenko and Nurtayev, 1988). At the numerical calculations we consider the foundation of dam as an undeformable rigid rock.

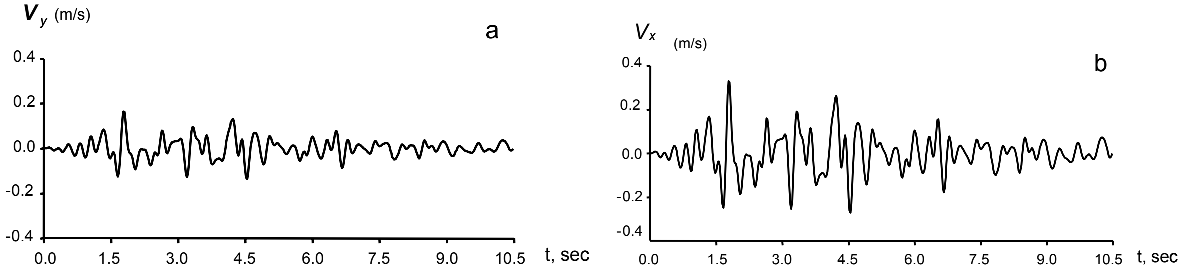
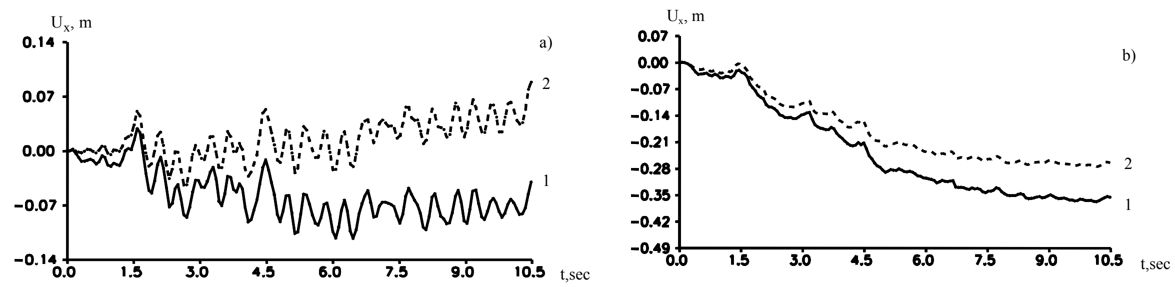


Figure 1. Velocigram of earthquake

Strength indexes of dip-slopes are: Y_0=1 Mpa, μ=0.4, $Y_{PL}=20Y_0$ and for core: Y_0=0.6 Mpa, μ=0.4, $Y_{PL}=20Y_0$. The full adhesion requirements are accepted on the boundary of the core and dip-slopes of the dike. Designers take into account the line of seepage in solving the stability problem for the slopes of dam. In the figures the dash lines (without account of saturation zones) and the solid lines (in view of the line of seepage) are the results of calculation.



a) crest zone

b) slope zone

Figure 2. Displacement of dam body
1 – with taking into account of soil saturation
2 - – without taking into account of soil saturation

One can see that taking into account the soil saturation gives some difference in calculations. The time alteration of horizontal displacements in crest and slope zones of the dam is shown on Figure 2. Up to 1.5 seconds after the action of seismic loads the influence of soil saturation does not lead to considerable divergence of incremental horizontal displacements and after that time reduce these values approximately up to 2 times.

The same situation is observed with the strains in crest zone. But in slope zones the divergence is not considerable.

The horizontal strains in the centre of core reach 50% of the divergence in comparison with the calculation without saturation effect. At foundation of the dam core such divergence rises up to 75 %.

The problem of the direction effect of seismic loads for dynamic dam behaviour is solved with the same physical and mechanical characteristics for the dam.

3. TUNNEL IN ROCKY MASSIVE

Underground hydrotechnical structures are the most complicated and labour-consuming types of the buildings which are part of weirs. The elementary schemes of rod systems for calculations of underground structures have been used until recently. The problem concerning the stress-strain state of spillway channel of Tupolang dam under gravity loading is solved here by boundary element techniques.

Calculation of such tunnels should be carried out on the basis of rock mechanics models and with account of the failure and lamination of soils.

The indirect boundary element techniques are more convenient in this case (Crouch and Starfield, 1983).

Fundamental Kelvin's solution is used for the integral expression of Navier's equation solution. As the consequence of the displacement and stresses expression via fictive loads is:

$$
\begin{aligned}
u_x &= \frac{P_x}{2G}\left[(3-4\nu)f + yf_{,y}\right] + \frac{P_y}{2G}\left[-yf_{,x}\right] \\
u_y &= \frac{P_x}{2G}\left[-yf_{,x}\right] + \frac{P_y}{2G}\left[(3-4\nu)f - yf_{,y}\right] \\
\sigma_{xx} &= P_x\left[(3-2\nu)f_{,x} + yf_{,xy}\right] + P_y\left[2\nu f_{,y} - yf_{,xx}\right] \\
\sigma_{yy} &= P_x\left[(2\nu-1)f_{,x} - yf_{,xy}\right] + P_y\left[2(1-\nu)f_{,y} + yf_{,xx}\right] \\
\sigma_{xy} &= P_x\left[2(1-\nu)f_{,y} - yf_{,xx}\right] + P_y\left[(1-2\nu)f_{,x} - yf_{,xy}\right]
\end{aligned}
\qquad (6)
$$

Here

$$
f(x,y) = -\frac{1}{4\pi(1-\nu)}\left[y\left(arctg\frac{y}{x-a} - arctg\frac{y}{x+a}\right) - (x-a)ln\sqrt{(x-a)^2+y^2} + (x+a)ln\sqrt{(x+a)^2+y^2}\right]
$$

and P_i are fictive forces.

Tunnel calculation under gravity loading can be solved more conveniently by introducing initial stresses:

$$
\sigma_y^0 = -\gamma h; \quad \sigma_x^0 = \frac{\nu}{(1-\nu)}\sigma_y^0
$$

However in practice the next expression for gravity stresses are used:

$$
\sigma_y = \lambda_1\gamma h; \quad \sigma_x = \lambda_2\gamma h; \quad \sigma_z = \gamma h,
$$

Here h is the depth of digging, γ is a unit weight and λ_i are the coefficients of lateral pressure.

But as a rule this approach is more engineering and all input data are accepted from their preliminary analysis of a concrete case. A more strict approach supposes the use of elasticity. The solution used is shown below. The numerical solution has been made without the account of the layered structure of rock continua. Generally, the full solution demands the 3D analysis; however in some cases the two-dimensional analysis gives a comprehensible pattern.

Rock massive is accepted as homogeneous continua (clay schist) and E=80.8Gpa; ν=0.29; γ=27076.6Gpa/m. The mountain altitude from the foundation of tunnel is 200m. The results of numerical solution are presented below (Figure 3). However, more comprehensive analysis must take into account the initial stresses, fracturing and non-homogeneous properties of soil.

The consideration of rock fracturing gives corrected results, the distribution of stresses is given below (Figure 4). In this case rectangular cave in rocky layered massive on dam's vicinity area is considered. Regular fracturing in rock is considered as anisotropic continua. Are physical characteristics accepted: $E_x=10^7$ KPa, $E_y=1.2E7$ KPa, $v_{xy}=0.2$, $G_{xy}=5E6$ KPa, angle of anisotropy are 45^0, initial stresses on infinity are $\sigma_y=-5000$ KPa, $\sigma_x=-2500$ KPa. Foundation is a rigidly fixed (undeformable, based on bedrock) and upper boundary is stresses free.

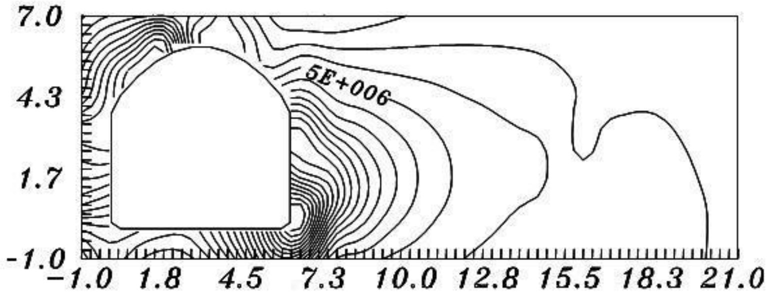
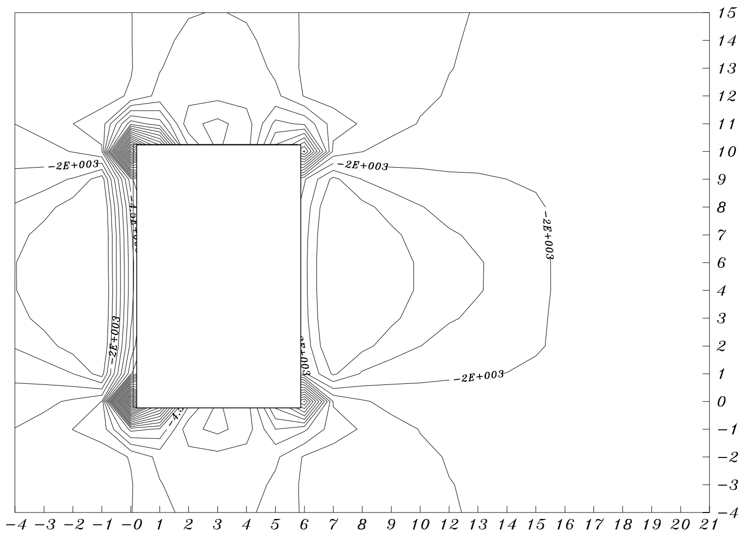
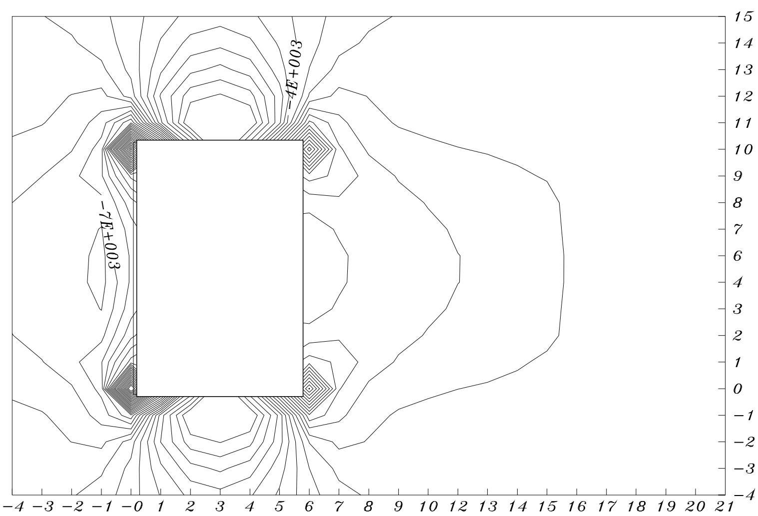


Figure 3. Effective stresses distribution in tunnel



a) σ_x distribution



b) σ_y distribution

Figure 4. Stresses distribution in rocky massive

4. CONCLUSION

As the result the horizontal impact on the dam is the most dangerous for failure and faults formation in soil. 2D dynamic analysis and account of nonlinearity allow to predict the strain-stress state, the location of incremental zones, tension zones, failure and creeping zone of slopes.

Underground structures related to dams have to be calculated with account of all fracturing which had arisen at design and operation stage. One of the approaches is anisotropic representation of fracturing rock massive.

The above mentioned factors confirm the necessity of the account of soil saturation and rock properties for calculation and operation of earth dams in seismic regions.

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