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Using The Hereditary Theory Of Viscoelasticity In Dynamic Calculations Of Structures

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Abstract – This article describes the use of the hereditary theory of viscoelasticity in dynamic calculations of structures.

Keywords – Vibration, Energy Dissipation, Excitation, Resonant Vibrations, Solids, Structures, Deformation.

I. Introduction

The intensity of vibrations of structures under dynamic influences significantly depends on the degree of energy dissipation in them. It can be expected that the higher the energy dissipation in the structure, the less intense the resonant vibrations at a given level of excitation.

For a theoretical description of deformation processes during vibrations of solids and structures, taking into account internal friction, they often limit themselves to studying the general laws of the external manifestation of the dissipation mechanism. Hypotheses and linear models of frequency-independent internal friction have found wide application in solving problems of the dynamics of structures. These hypotheses, reflecting the manifestation of elastic imperfections of materials, do not describe the creep of deformations and relaxation of stresses, which are called "hereditary properties".

The foundations of the modern hereditary theory of viscoelasticity, reflecting almost all the features of the quasi-static and dynamic behavior of a material, were laid in the classical works of Boltzmann and Volterra. The hereditary theory of viscoelasticity is more general and more accurately describes the mechanism of dissipation in materials [1-9].

II. Literature Review

It is well known that the rheological properties of the medium have a significant effect on the deformation process as a whole, i.e. at all three stages: elastic, plastic and the stage of destruction. Therefore, the problems of the theory of viscoelasticity have attracted special attention of researchers in recent years. A great contribution to this area of research was made by the works of N.Kh. Arutyunyan, S.V. Aleksandrovsky, F.B. Badalov, V.V. Bolotina, V.M. Bondarenko, Zh.S. Erzhanova, A.A. Ilyushin, A.Yu. Ishlinsky, L.M. Kachanova, M.A.Koltunov, V.V. Moskvitin, P.M. Ogibalova, B.E. Pobedri, Kh.A. Rakhmatulina, Yu.N. Rabotnova, T.R. Rashidova, A.R. Rzhanitsin, L.N. Tolokonnikov, I.E. Troyanovsky, E.I. Shemyakina, T.Sh. Shirinkulova, G. H. Khozhmetov and many others.

At present, the most developed methods for solving linear quasi-static problems of viscoelasticity. Issues related to the behavior of viscoelastic structures under dynamic loading have been less comprehensively developed.

III. Analysis and Results

There is a sufficient number of experimental works on the study of the inelastic behavior of materials, in which the main place is given to the construction and analysis of the experimental creep or relaxation curve. To solve dynamic problems of viscoelasticity, it is necessary to have the values of the parameters of the kernels for models describing similar processes, which have been determined to date for an extremely limited class of materials. It is these factors that complicate the solution of dynamic viscoelasticity problems for inhomogeneous structures in a rigorous formulation.

Currently, one of the common models of the hereditary theory of viscoelasticity, which provides a good agreement between experimental and theoretical results, is the linear hereditary Boltzmann-Volterra theory with a combined exponential-power kernel of A.R. Rzhanitsin:

$$\sigma(t) = E\varepsilon(t) - E\int_0^t \Gamma(t-\tau)\,\varepsilon(\tau)\,d\tau, \qquad (1)$$

$$\Gamma(t-\tau) = \mathrm{A}(t-\tau)^{\alpha-1} e^{-\beta(t-\tau)}, \qquad (2)$$

$\Gamma(t-\tau)$ - relaxation core; $\mathrm{A}, \alpha, \beta$ - kernel parameters.

In this case, when solving dynamic problems for the structures under consideration, the hereditary theory of viscoelasticity (1) with a weakly singular kernel of the form (2) is used, for which, for any t

$$\int_0^t \frac{\mathrm{A}e^{-\beta t}}{t^{1-\alpha}}dt \le 1, \qquad 0 \le \alpha \le 1.$$

The resolution of this kernel for describing creep processes (bounded, steady-state, unbounded) has the form [10]

$$\mathrm{K}(t) = \frac{e^{-\beta t}}{t}\sum_{n=1}^{\infty}\frac{\left[\mathrm{A}\Gamma(\alpha)\right]^n t^{\alpha n}}{\Gamma(\alpha n)}. \qquad (3)$$

In [10], functions (1), (3) and their integrals are tabulated in detail and a method for determining the parameters $\mathrm{A}, \alpha, \beta.$ These parameters of the influence function for specific materials are determined by matching the experimental curve

$$\bar{\varepsilon}_Э = \frac{\varepsilon(t)}{\sigma_K}$$

and theoretical creep curve $\bar{\varepsilon}_\mathrm{T} = 1 + \int_0^t \mathrm{K}(t-\tau)d\tau$,

built in logarithmic coordinates.

For determining $\mathrm{A}, \alpha, \beta$ experimental material creep curves are plotted on standard logarithmic paper and aligned with the corresponding theoretical curve $\bar{\varepsilon}_\mathrm{T}$ by shifts along the abscissa axes t and ordinate $\bar{\varepsilon}_\mathrm{T}$.

When determining the parameters of the core for concrete and soil, several experimental creep curves given in [11] were processed. Below are selectively the values of the obtained kernel parameters $(A_Э, \alpha_Э, \beta_Э)$ for concrete and soil:

1) .For twin specimens, first loaded at the age of 28 days to constant voltage of various levels

$$\alpha_Э = 0.075; \quad \beta_Э = 0.00000014; \quad A_Э = 0.0194;$$

2) For concrete at constant stresses of various initial levels, loaded at the age of 35 days

$$\alpha_Э = 0.1; \quad \beta_Э = 0.00000011; \quad A_Э = 0.01594;$$

3) For soil - fine-grained sandstone with clay cement in an air-dry state

$$\alpha_Э = 0.25; \quad \beta_Э = 0.00243; \quad A_Э = 0.01674.$$

The found values of the kernel parameters were further used to solve specific problems.

In Fig. 1, a, b. and Fig. 2.a, b. graphs of dependences of frequencies and modes of vibration of soil and concrete dams are presented. At the same time, in each case, the influence of these types of material on the frequency of the dam's oscillation was investigated.

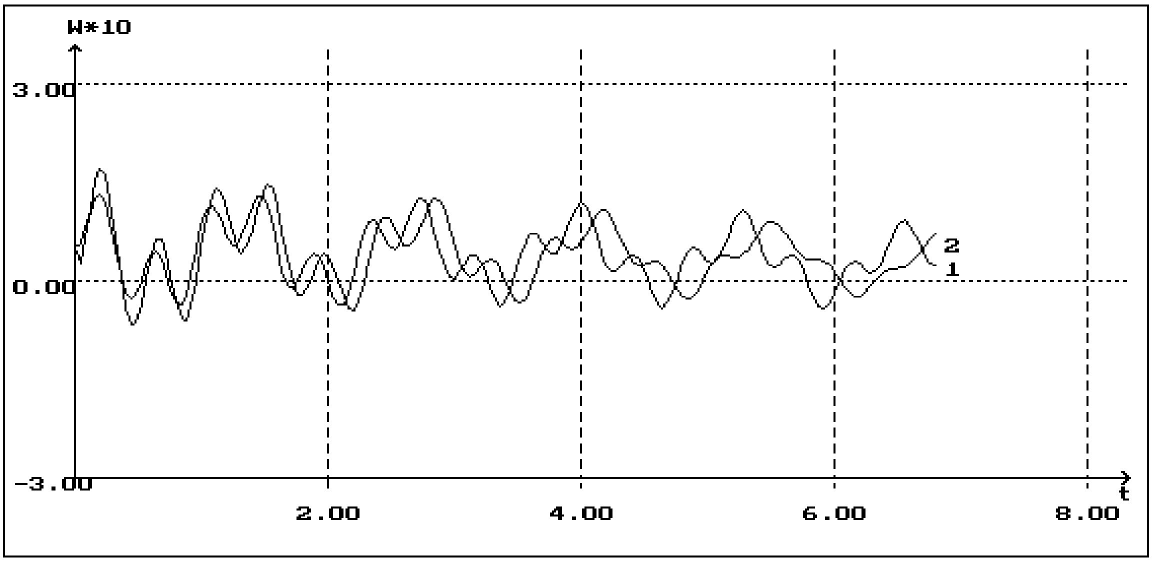

Fig. 1 a. μ=0.3, ρ/ρ_1=1/2.4, λ=2, α^*=0.5

1. A=0.01674, α=0.25, β=0.00243
2. A=0.01594, α=0.1, β=0.00000011

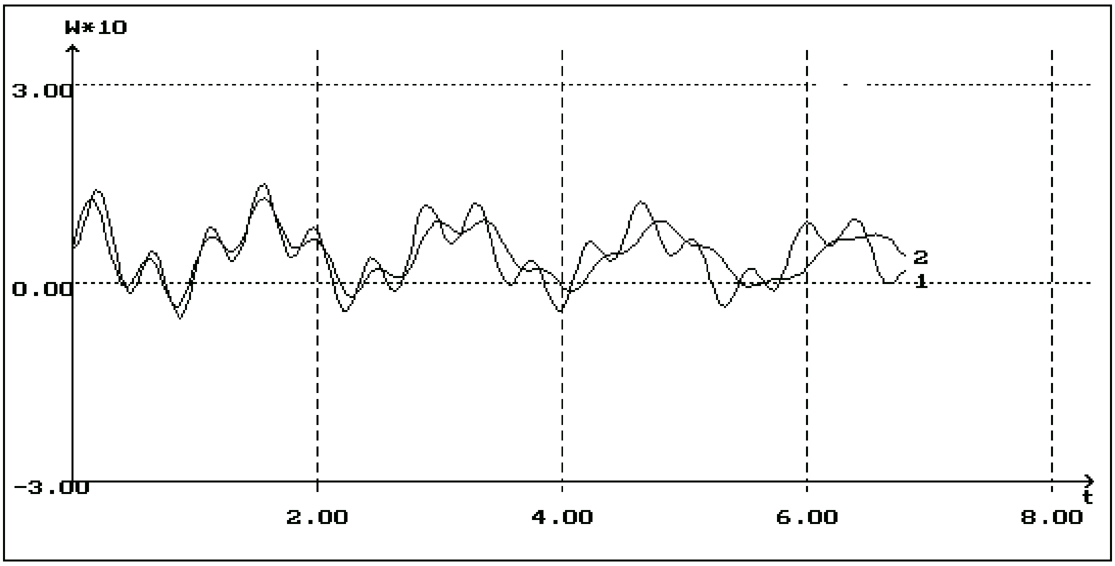

Fig. 1 b. μ=0.3, ρ/ρ_1=1/2.4, λ=2, α^*=0

1. A=0.01674, α=0.25, β=0.00243
2. A=0.01594, α=0.1, β=0.00000011

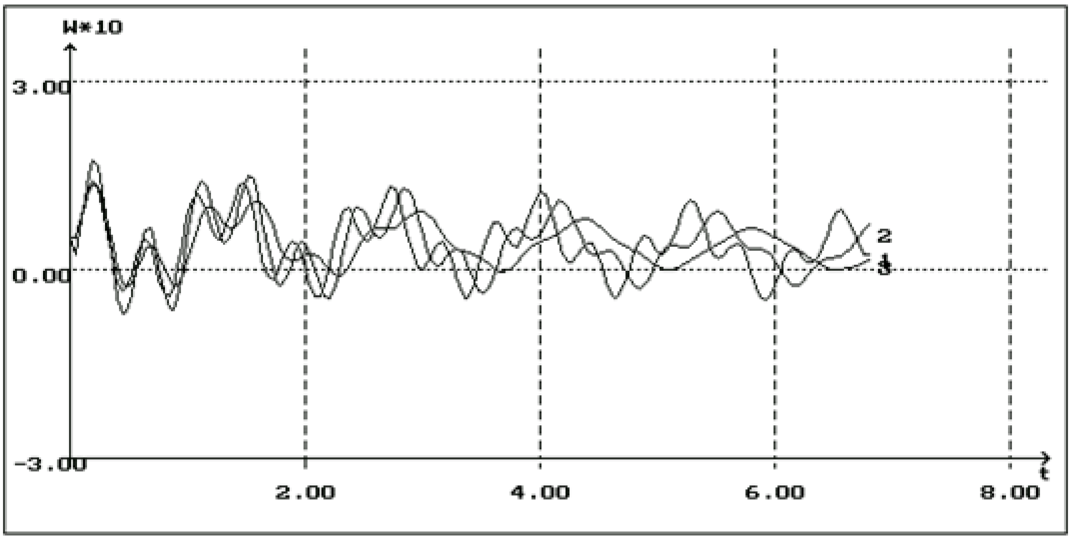

fig. 2 a. μ=0.3, ρ/ρ_1=1/2.4, λ=2, α^*=0

1. A=0.01674, α=0.25, β=0.00243
2. A=0.01594, α=0.1, β=0.00000011
3. A=0.0194, α=0.075, β=0.00000014

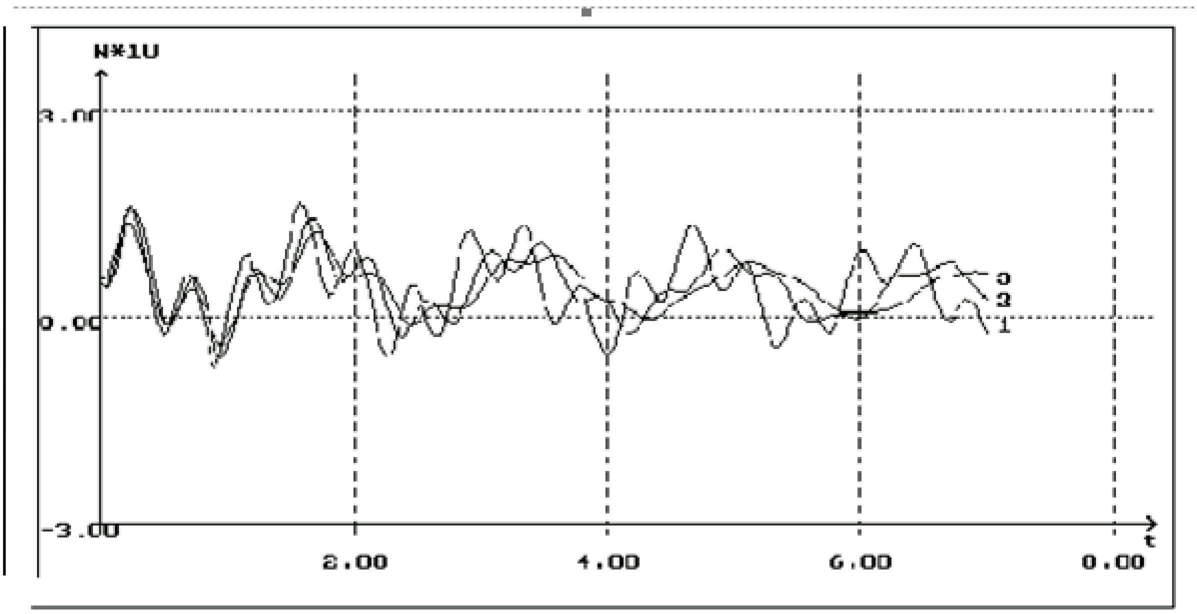

fig. 2 b. μ=0.3, ρ/ρ_1=1/2.4, λ=2, α^*=0.5

1. A=0.01674, α=0.25, β=0.00243
2. A=0.01594, α=0.1, β=0.00000011
3. A=0.0194, α=0.075, β=0.00000014

IV. Conclusion/Recommendations

From the presented graphs it follows that the frequency and amplitude of vibrations of concrete dams decreases faster than the frequency and vibrations of earth dams. Comparison of the results shows that the frequencies and amplitudes of vibrations of a soil dam are 14% higher than the amplitudes and frequencies of vibrations of a concrete dam, the amplitude and frequency of vibrations of a concrete dam are 10% higher than those of twin samples, and a soil dam is 24% higher than amplitudes and frequencies of oscillations of twin samples.

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