Non-stationary oscillations of high-rise axisymmetric structures

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Non-stationary oscillations of high-rise axisymmetric structures

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Abstract. Dynamic behavior of a high-rise structure is investigated in the paper taking into account linear and nonlinear strain and energy dissipation caused by internal friction in the building material under various kinematic effects. An algorithm and program have been developed for the dynamic calculation of high-rise axisymmetric structures under various dynamic effects. Unsteady oscillations of the high-rise axisymmetric structure represented by a one-dimensional model - a viscoelastic beam of the annular cross-section with a variable tilt of the generatrix and a variable thickness are considered. Dynamic behavior of a high-rise structure in the resonant mode under short-term pulsed effect, under prolonged pulsed effect, under the sinusoidal-damped effect, under real seismic impact is studied and the graphs of displacement changes over time of various points of the structures are plotted. It was found that the dynamic response of a high-rise structure is affected not only by the magnitude of the dynamic load but also by its frequency spectrum and duration. Under the impact of a frequency close to the natural vibration frequency of the structure, significant displacements occur, and, if the material does not possess dissipative properties, the structure could collapse, being in the resonant mode.

1. Introduction
The design and construction of various high-rise structures require ensuring their reliable operation under various dynamic influences. This, in turn, dictates the conditions for limiting and reducing the level of structure vibrations from harmful dynamic effects. To date, various methods and means of dealing with unacceptable vibrations of structures are known, in particular, the change in rigidity and inertial parameters of structures to detune from resonances, the increase in damping properties by using materials and structures with high absorption capacity, for example, special coatings, the use of vibration isolation and various types of vibration dampers [1, 2, 3]. Each of the mentioned methods has a rational range of applications.

In acting building codes, the dynamic calculation model of axisymmetric structures (high-rise pipes, cooling towers, etc.) is adopted in the form of an elastic cantilever with distributed or concentrated mass, which does not account for several factors — real geometry, design features, spatial nature of the structure, dissipative properties of the structure material and others that directly affect the sought for values of dynamic characteristics and the stress-strain state (SSS) of structures under various impacts. As a result, the found values of the sought-for quantities noticeably differ from the real ones.

Studies of the natural vibrations of high axisymmetric structures (smokestacks) showed that despite the differences in nature of the higher modes of vibrations of the structures under consideration, the fundamental modes (bending, longitudinal and torsional) and the corresponding eigenfrequencies are independent of the dimension of the selected calculation model. Therefore, the studies of structural dynamic behavior in an unsteady-state mode, taking into account elastic, viscoelastic, and nonlinear strain of the material, can be carried out in the framework of a one-dimensional model. The simplicity of the calculation model allows us to study in detail the effect of various laws of change under the impact and various laws of material strain on the dynamic behavior of structures.

The practice of modern construction in seismic regions requires studying the dynamic behavior of structures and improving the methodology for their calculation, taking into account not only geometrical features of structures but nonlinear and dissipative properties of the structure material as well. Practical methods of calculation are usually based on the dynamic analysis of structures as the linearly elastic systems. However, instrumental data and the results of engineering analysis on the nature of structures operation under strong earthquakes indicate that the structure rigidity does not always remain constant. Therefore, the parameters of the actual reaction of the structures should be determined only in non-linear analysis, which allows developing more reasonable methods of design and construction, increase the efficiency of structures while maintaining the required level of reliability. To date, several publications are known devoted to the study of the dynamics of structures, in which calculation methods, research results are highlighted, and an attempt is made to consider the dissipative properties of the material.

In [4, 5, 6, 7], the aerodynamic wind damping coefficients of modified square high-rise buildings were studied based on estimated results. The effect of aerodynamically modified cross-sections on the coefficient of aerodynamic damping under across-wind was investigated. The aerodynamic wind damping coefficients in high-rise buildings were determined using the eigensystem realization algorithm (ERA) method in combination with the random decrement technique (RDT).

Various basic and secondary modifications of the external forms of high-rise buildings and their advantages compared to conventional forms (square, triangular, round) were considered in [8, 9, 10]. The influence of wind-induced aerodynamic forces and moments was studied in buildings with a changed height; the space-time characteristics of vibrations were determined In [11, 12, 13, 14, 15, 16], the eigenfrequencies of a cylindrical shell under various boundary conditions were studied, the influence of uniform external pressure and symmetrical boundary conditions on the eigenfrequencies of homogeneous and multilayer isotropic cylindrical shells was studied, and the vibrational process of rigid composite cylindrical shells was investigated taking into account the bending behavior of the ribs.

In [17, 18], the determination of the dynamic characteristics of high-rise monolithic reinforced concrete structures was investigated, and the results obtained were recommended for certification of buildings.

Parametric vibrations of viscoelastic orthotropic plates of variable thickness were studied in [19, 20, 21, 22, 29, 30, 31], under external load, taking into account the influence of geometric nonlinearity, viscoelastic properties of the material, and other physicomechanical and geometrical design parameters on the dynamic instability region.

In [23, 24, 25, 26], vibrations of high buildings caused by wind and tornado waves were studied to assess the aeroelastic effects of high buildings using the wind tunnel tests. The aerodynamic damping coefficient and aerodynamic stiffness were determined by analyzing the aeroelastic force acting on the oscillating model. For a 347-meter-high building, the effect of aeroelastic parameters on wind-induced responses and equivalent static wind loads was analyzed. The results showed that during a return period of 100 years, aerodynamic damping was positive and aerodynamic stiffness was negative.

The current stage in the development of the theory of seismic resistance involves an account for nonlinear behavior of the structure material under dynamic loads. Any real objects possess non-linear properties to a different extent but in some cases the influence of non-linearity is negligible, in such cases linear models and the corresponding linear theories are used. The question of estimating the nonlinearity for real structures remains open due to several mathematical problems that arise when solving the problem and in the absence of nonlinearity parameters of the material. This dictates the relevance of the studies presented in this paper, where the dynamic behavior of a high-rise structure is studied taking into account the linear, nonlinear strain and energy dissipation caused by internal friction in the building material under various kinematic effects.

2. Methods

Unsteady-state forced vibrations of a high axisymmetric structure are considered; the structure is represented by a one-dimensional model - a viscoelastic beam of the annular cross-section with a variable slope of the generatrix and variable thickness. The lower end of the beam ($z = 0$) is rigidly fixed and the kinematic effect $w_0 (t)$ is set on it; the upper end ($z = L$) is free. The beam material is a nonlinearly viscoelastic one. Bending unsteady-state forced vibrations of points located at different levels of a structure under set kinematic effect is to be determined.

The mathematical statement of the problem includes the variational equation of the principle of virtual displacements, according to which the sum of work of all active forces, including inertia forces, on a virtual displacement δw, satisfying geometrical boundary conditions is zero

$$\delta A_M + \delta A_u + \delta A_P = 0 \tag{1}$$

Here δA_M, $\delta A_и$, δA_P are the virtual work of the bending moment, inertial forces and external forces, respectively, calculated by the formulas:

$$\delta A_M = -\int_0^L M(z)\delta\left(\frac{\partial^2 w}{\partial z^2}\right)dz\,,\ \delta A_u = -\rho\int_0^L F(z)\left(\frac{\partial^2 w}{\partial t^2}\right)\delta w dz\,,\ \delta A_P = \int_0^L P(z,t)\delta w dz \tag{2}$$

where ρ is the beam material density, L is the beam length, $w(z)$ is the beam deflection, $M(z)$ is the bending moment; $F(z)$ is the cross-sectional area; $P(z, t)$ – the external dynamic forces.

The kinematic boundary condition at the base is

$$z = 0 : w(z,t) = w_0(t) \tag{3}$$

where $w_0(t)$ is the known time function.

Initial conditions are

$$w(z,0) = u_0, \frac{\partial w(z,0)}{\partial t} = \dot{u}_0 \tag{4}$$

where u_0, $\dot{u}_0$ are the given constants.

To describe the relationship between the stress σ_z and the strain ε_z the nonlinear theory of viscoelasticity [4,29] is used, which has the form

$$\sigma_z = E\left\{\left[\varepsilon_z(t) - \int_0^t R_1(t-\tau)\varepsilon_z(\tau)d\tau\right] - \gamma\left[\varepsilon_z^3(t) - \int_0^t R_2(t-\tau)\varepsilon_z^3(\tau)d\tau\right]\right\} \tag{5}$$

where E is the instantaneous modulus of elasticity of the material; R_1, R_2 are the relaxation kernels; γ=const > 0 is the non-linearity coefficient, depending on the material of the beam.

The dependence between the deflection w and the strain ε_z is taken in the form

$$\varepsilon_z = -x\frac{\partial^2 w}{\partial z^2} \tag{6}$$

and the relationship between bending moment M_z and stress σ_z is

$$M_z = \int_F x\sigma_z dF \tag{7}$$

The problem of unsteady-state nonlinear forced vibrations of a beam consists in the following: for a given function $w_0(t)$ under initial conditions $u_0, \dot{u}_0$ - to find the deflection $w(z, t)$, strain $\varepsilon_z(z, t)$,), stress $\sigma_z(z, t)$ and bending moment $M_z(z, t)$, satisfying equations (1), (2), (5) - (7) and conditions (3), (4) for any possible δw.

To reduce the variational problem posed above to a system of resolving equations, the finite element method is used [11], where a one-dimensional element is selected as the finite element, taken in the form of a truncated cone that works on bending with four degrees of freedom.

For the displacement function *w* inside the *e*-th element, the cubic approximation is used:

$$w = \alpha_1 + \alpha_2 z + \alpha_3 z^2 + \alpha_4 z^3 \tag{8}$$

then for the first and second derivatives, the following expressions are obtained

$$\frac{\partial w}{\partial z} = \alpha_2 + 2\alpha_3 z + 3\alpha_4 z^2; \quad \frac{\partial^2 w}{\partial z^2} = 2\alpha_3 + 6\alpha_4 z \tag{9}$$

The dependence of the nodal displacements and rotation angles of the *e*-th finite element $\{w_i\}$ on the vector of arbitrary constants $\{\alpha_i\}$ in matrix form is written as

$$\{w_i\} = \begin{bmatrix} 1 & 0 & 0 & 0 \\ 0 & 1 & 0 & 0 \\ 1 & l & l^2 & l^3 \\ 0 & 1 & 2l & 3l^2 \end{bmatrix} \{\alpha_i\} \tag{10}$$

where $\{w_i\}^T = \{w_i, \beta_1, w_2, \beta_2\}$, $\beta_1 = \frac{\partial w}{\partial z}\Big|_{z=0}$, $\beta_2 = \frac{\partial w}{\partial z}\Big|_{z=l}$.

Hereinafter, the following notation is used: {} - vector, [] - matrix, ^{T} - transposition operation.

The transformation inverse to (10), i.e. the matrix dependence of $\{\alpha_i\}$ on $\{w_i\}$ is expressed as

$$\{\alpha_i\} = \begin{bmatrix} 1 & 0 & 0 & 0 \\ 0 & 1 & 0 & 0 \\ -\frac{3}{l^2} & -\frac{2}{l} & \frac{3}{l^2} & -\frac{1}{l} \\ \frac{2}{l^3} & \frac{1}{l^2} & -\frac{2}{l^3} & \frac{1}{l^2} \end{bmatrix} \{w_i\}, \text{ i.e. } \{\alpha_i\} = [A]\{w_i\} \tag{11}$$

Using the indicated transformations (11), we express the displacement function (8) and its derivatives in a matrix form in terms of nodal displacements $\{w_i\}$

$$w = \left[1 \cdot\cdot z \cdot\cdot z^2 \cdot\cdot z^3\right]\{\alpha_i\} = \left[1 \cdot\cdot z \cdot\cdot z^2 \cdot\cdot z^3\right][A]\{w_i\} \tag{12}$$

$$\frac{\partial w}{\partial z} = \left[0 \cdot\cdot 1 \cdot\cdot 2z \cdot\cdot 3z^2\right][A]\{w_i\} \tag{13}$$

$$\frac{\partial^2 w}{\partial z^2} = [0 \cdot\cdot 0 \cdot\cdot 2 \cdot\cdot 6z][A]\{w_i\} \tag{14}$$

We introduce the matrix [B]

$$[B] = [0 \cdot\cdot 0 \cdot\cdot 2 \cdot\cdot 6z][A][\ -\frac{6}{l^2} + \frac{12z}{l^3}; -\frac{4}{l} + \frac{6z}{l^2}; \frac{6}{l^2} - \frac{12z}{l^3}; -\frac{2}{l} + \frac{6z}{l^2}\ ],$$

then

$$\frac{\partial^2 w}{\partial z^2} = [B]\{w_i\} \tag{15}$$

Substituting expression (5-7) in (2), we obtain the virtual work of the bending moment for the *e*-th element

$$\delta A_M^e = \int_0^l \left[\frac{\partial^2 w}{\partial z^2}\delta\left(\frac{\partial^2 w}{\partial z^2}\right)\int_F x^2 dF\right]dz - E\int_0^t \left\{R_1(t-\tau)\int_0^l \left[\frac{\partial^2 w}{\partial z^2}\delta\left(\frac{\partial^2 w}{\partial z}\right)\int_F x^2 dF\right]dz\right\}d\tau$$

$$-E\gamma\int_0^l\left[\left(\frac{\partial^2 w}{\partial z^2}\right)^3\delta\left(\frac{\partial^2 w}{\partial z^2}\right)\int_F x^4 dF\right]dz+E\gamma\int_0^t\left\{R_2(t-\tau)\int_0^l\left[\left(\frac{\partial^2 w}{\partial z^2}\right)^3\delta\left(\frac{\partial^2 w}{\partial z}\right)\int_F x^4 dF\right]dz\right\}d\tau \tag{16}$$

Substitution of (15) into (16) and integration over the cross-sectional area leads each term of expression (16) to the following form:

the first term

$$\delta\{w_i\}^T E\int_0^l J^e(z)[B]^T[B]dz\{w_i\}=\delta\{w_i\}^T\left[K^e\right]\{w_i\}\ , \tag{17}$$

where

$J^e(z)=\frac{\pi}{4}\left[(R_H(z))^4-(R_B(z))^4\right]$ is the moment of inertia of the cross-section of the *e*-th element; R_H, R_B are the outer and inner radii of the element, respectively;

$\left[K^e\right]=E\int_0^l J^e(z)[B]^T[B]dz$ is the stiffness matrix of the *e*-th element.

the second term

$$E\int_0^t\left\{R_1(t-\tau)\int_0^l\left[\frac{\partial^2 w}{\partial z^2}\delta\left(\frac{\partial^2 w}{\partial z^2}\right)\int_F x^2 dF\right]dz\right\}d\tau=\delta\{w_i\}^T E\int_0^t\{R_1(t-\tau)\times$$

$$\times\int_o^l J^e(z)[B]^T[B]dz\}d\tau\{w_i\}=\delta\{w_i\}^T\int_0^t R_1(t-\tau)\left[K^e\right]\{w_i\}d\tau \tag{18}$$

the third term

$$E\gamma\int_0^l\left[\left(\frac{\partial^2 w}{\partial z^2}\right)^3\delta\left(\frac{\partial^2 w}{\partial z^2}\right)\int_F x^4 dF\right]dz=\delta\{w_i\}^T E\gamma\int_0^l J_1^e(z)[B]^T[B]\{w_i\}[B]\{w_i\}[B]\{w_i\}dz, \tag{19}$$

Here $J^e(z)=\frac{\pi}{8}\left[(R_H(z))^6-(R_B(z))^6\right]$.

Expanding the expression under the integral sign in (19):

$$\begin{gathered}\left\{V^e\right\}=J_1^e(z)\left[\ -\frac{6}{l^2}+\frac{12z}{l^3};-\frac{4}{l}+\frac{6z}{l^2};\frac{6}{l^2}-\frac{12z}{l^3};-\frac{2}{l}+\frac{6z}{l^2}\ \right]^T\times\\ \left[\ (-\frac{6}{l^2}+\frac{12z}{l^3})w_1+(-\frac{4}{l}+\frac{6z}{l^2})\varphi_1+(\frac{6}{l^2}-\frac{12z}{l^3})w_2+(-\frac{2}{l}+\frac{6z}{l^2})\varphi_2\ \right]^3\end{gathered} \tag{20}$$

we see that it is a vector whose coordinates are cubic polynomials from nodal displacements. As a result of integration over the length of the element, the third term (19) is

$$E\gamma\int_0^l\left[\left(\frac{\partial^2 w}{\partial z^2}\right)^3\delta\left(\frac{\partial^2 w}{\partial z^2}\right)\int_F x^4 dF\right]dz=\delta\{w_i\}^T E\gamma\left\{V^e\right\} \tag{21}$$

where the index "e" indicates that the vector {Ve} is defined for the *e*-th element.

The fourth term

$$\begin{gathered}E\gamma\int_0^t\left\{R_2(t-\tau)\int_0^l\left[\left(\frac{\partial^2 w}{\partial z^2}\right)^3\delta\left(\frac{\partial^2 w}{\partial z}\right)\int_F x^4 dF\right]dz\right\}d\tau=\\ =\delta\{w_i\}^T E\gamma\int_0^t R_2(t-\tau)\int_0^l J_1^e(z)[B]^T[B]\{w_i\}[B]\{w_i\}dzd\tau\end{gathered} \tag{22}$$

Considering (20), we get

$$E\gamma\int_F x^5\int_0^l R_2(t-\tau)\left(\frac{\partial^2 w}{\partial z^2}\right)^3\delta\left(\frac{\partial^2 w}{\partial z^2}\right)dzdF=\delta\{w_i\}^T E\gamma\int_0^t R_2(t-\tau)d\tau\{V^e\} \quad (23)$$

The use of the finite element method procedure leads the variational problem (1) and (3) to a nonlinear system of integro-differential equations, which has the following matrix form [32]:

$$[M]\{\ddot{w}(t)\}+[K]\{w(t)\}=\{P(t)\}-\int_0^t R_1(t-\tau)[K]\{w(t)d\tau+E\gamma\{V(t)\}\}-E\gamma\int_0^t R_2(t-\tau)d\tau\{V(t)\} \quad (24)$$

Here *[M], [K]* are the matrices of mass and rigidity of the entire structure; {w} is the displacement vector of all the nodal points of the structure; *{V}* is a vector whose coordinates are determined by cubic polynomials of system displacements, *{P}* is a vector of external influences.

This equation is solved by the Newmark method [12, 32]. Equation (24) at given initial conditions (4) is solved by direct integration using a numerical step-by-step procedure. We used the Newmark method to solve the system of equations (24), based on independent expansions of $w(t_i+\tau)$ and its derivative into the series in powers τ, while holding the terms containing the third derivative w_i. The coefficients for the residual terms α and β are selected from the condition for ensuring the unconditional convergence of the integration process:

$$w(t_i+\tau)=w_i+\tau\dot{w}_i+\frac{\tau^2}{2}\ddot{w}_i+\alpha\tau^3\dddot{w}_i;\ \dot{w}(t_i+\tau)=\dot{w}_i+\tau\ddot{w}_i+\beta\tau^2\dddot{w}_i \quad (25)$$

Substituting $\dddot{w}_i=\frac{\ddot{w}_{i+1}-\ddot{w}_i}{\tau}$, expressions for displacements and velocities (25) are written as

$$w_{i+1}=w_i+\tau\dot{w}_i+\frac{\tau^2}{2}\ddot{w}_i+\alpha\tau^3(\ddot{w}_{i+1}-\ddot{w}_i) \quad (26)$$

$$\dot{w}_{i+1}=w_i+\tau\ddot{w}_i+\beta\tau^2(\ddot{w}_{i+1}-\ddot{w}_i) \quad (27)$$

Then the acceleration obtained from (26)

$$\ddot{w}_{i+1}=\frac{1}{\alpha\tau^2}(w_{i+1}-w_i)-\frac{1}{\alpha\tau}\dot{w}_i+\left(1-\frac{1}{2\alpha}\right)\ddot{w}_i \quad (28)$$

is substituted into the velocity expression (27)

$$\dot{w}_{i+1}=\frac{\beta}{\alpha\tau}(w_{i+1}-w_i)+\left(1-\frac{\beta}{\alpha}\right)w_i+\frac{\tau}{2}\left(2-\frac{\beta}{\alpha}\right)\ddot{w}_i \quad (29)$$

To find a solution w_{i+1} for time t_{i+1}, the general equation of motion is written as follows:

$$[M]\ddot{w}_{i+1}+[C]\dot{w}_{i+1}+[K]w_{i+1}=\{P_{i+1}\} \quad (30)$$

After substituting expressions for accelerations (28) and velocity (29) into (30) an algebraic system of equations is obtained

$$[A]\{w_{i+1}\}=\{R_{i+1}\} \quad (31)$$

Where

$$[A]=[K]+\frac{1}{\alpha\tau^2}[M]$$

$$\{R_{i+1}\}=\{P_{i+1}\}+[M]\left(\frac{1}{\alpha\tau^2}\{w_i\}+\frac{1}{\alpha\tau}\{\dot{w}_i\}+\left(\frac{1}{2\alpha}-1\right)\{\ddot{w}_i\}\right)+\{W_i\} \quad (32)$$

where

$$\{W_i\}=\int_0^t R_1(t-\tau)[K]\{w_i\}d\tau+E\gamma\{V_i\}-E\gamma\int_0^t R_2(t-\tau)d\tau\{V_i\} \quad (33)$$

To solve the resulting system of equations (31), it is necessary to specify at the initial moment the values of displacements $\{w_0\}$, velocity $\{\dot{w}_0\}$, and accelerations $\{\ddot{w}_0\}$. Usually $\{\ddot{w}_0\} = 0$ is taken.

The Newmark method is unconditionally stable if:

$$\beta \geq 0.5, \qquad \alpha \geq 0,25(\beta + 0,5)^2$$

3. Results and discussion

The dynamic behavior of a high-rise smokestack of the Novo-Angren hydro-electric power plant (HEPP) is considered; its actual dimensions are presented in [27, 29], the smokestack is rigidly pinched at the base. The problem of forced unsteady-state vibrations of this structure is solved under various laws of kinematic disturbance of the lower base. The material of the structure is assumed to be a linearly elastic one.

In this case, the resolving equation (24) contains no viscous and nonlinear components and it has the following form

$$[M]\{\ddot{w}(t)\} + [K]\{w(t)\} = \{P(t)\} \qquad (34)$$

under corresponding initial conditions (4). For the problems considered in this section, the initial conditions are assumed to be homogeneous, i.e.

$$w(z,0) = 0, \frac{\partial w(z,0)}{\partial t} = 0 \qquad (35)$$

It is necessary to determine the displacements of structure points at different points in time. The obtained linear system of differential equations (34) with initial conditions (35) is solved by the Newmark method.

3.1. Resonance mode.

The lower base of the pipe is subjected to a kinematic effect of the type

$$z = 0, \ddot{u}_0 = \sin(pt) \qquad (36)$$

The frequency of impact *p* was chosen close to the eigenfrequency ω_0 of the pipe bending vibrations.

The nature of the impact and the solutions obtained using the developed algorithm and computer program for the transverse displacements of pipe points located close to the middle of the pipe at a height of *z* = 200 m and at the top - at a height of *z*=325 m are shown in figure 1.

An analysis of the obtained displacements shows that under the indicated impact of a frequency $P \approx \omega_0$, the amplitude of the points displacements increases unlimitedly over time, i.e. the pipe is in a resonance mode. The oscillations occur with the period of the fundamental tone of natural vibrations. These results confirm the reliability of the developed algorithm and calculation program.

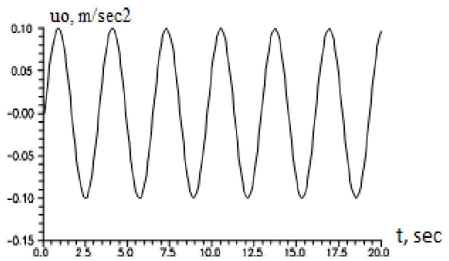
3.2.Short-term impulse effect.

The lower base of the pipe is subjected to a kinematic effect of the type

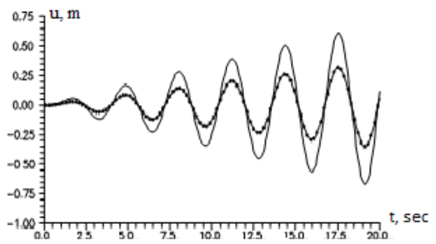
$$z = 0 : \ddot{u}_0 = \begin{cases} 1 & t \leq 1sek \\ 0 & t > 1sek \end{cases} \qquad (37)$$

Here, the pulse duration is approximately equal to a quarter of the fundamental period. The nature of the impact and the displacements of various points of the pipe obtained using the developed algorithm and computer programs are shown in figure 2.

As seen from the results, in this case free vibrations of the pipe occur at an amplitude established at the time of impact termination.

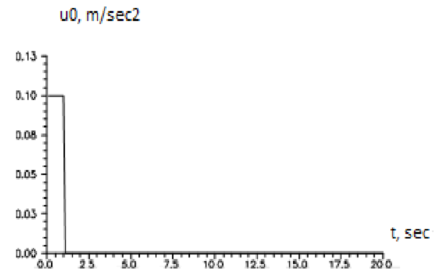


a

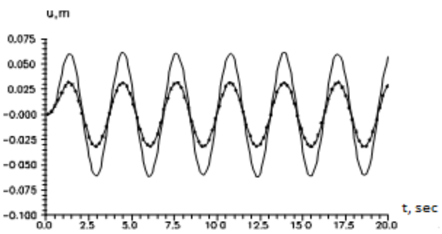


b

Figure1. Forced vibrations of various points of the pipe at resonance: a) the nature of the impact; b) oscillations of the pipe points: -*-*-*-* - at a height of z = 200m; ____ - at a height of z = 325m



a



b

Figure2 Forced vibrations of various points of the pipe with a short pulse: a) the nature of the impact; b) oscillations of the pipe points: -*-*-*-* - at a height of z = 200m; ____ - at a height of z = 325m

3.3. Long-term impulse effect.

The kinematic effect is set on the lower base of the pipe according to the law:

$$z = 0 : \ddot{u}_0 = \begin{cases} 1 & t \le 5sek \\ 0 & t > 5sek \end{cases} \tag{38}$$

Here, the pulse duration exceeds the fundamental period of natural vibrations. The nature of the impact and the displacements of various points of the pipe obtained using the developed program are shown in figure 3.

An analysis of the results obtained and their comparison with the results obtained previously shows that in this case, at a constant (up to 5 sec) impact, the pipe makes free oscillations relative to the deviated position, and then - after the termination of the impact, the oscillations continue relative to the neutral - vertical position with the amplitude established at the time of impact termination.

3.4. Sinusoidal-damping effect.

Damping kinematic effects can occur when the structure is loaded with seismic [30,32], explosive, and other long-term unsteady impacts. Therefore, the kinematic effect set on the lower base of the pipe is considered according to the law of a damping sinusoid:

$$z = 0 : \ddot{u}_0 = A\sin(pt)\exp(-\gamma t) \tag{39}$$

Here A is the amplitude of the impact; $p \approx \omega_0$ is the impact frequency equal to the first frequency of the bending eigenmodes of the pipe; γ=0.1 is the coefficient characterizing the impact attenuation over

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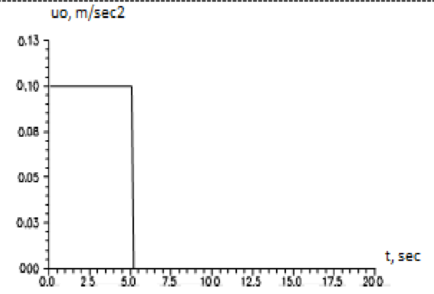
time. The amplitude of the impact A is chosen so that its maximum is 0.1. This is done to be able to compare the displacements obtained in all cases and to analyze the effect of impact nature, but not its intensity.

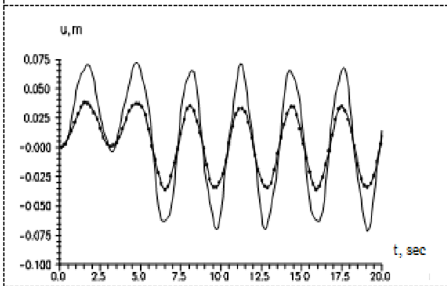
The amplitude of action thus obtained is equal to

$$A = \frac{\sqrt{p^2 + \gamma^2}\exp\left(\frac{\gamma}{p} arctg \frac{p}{\gamma}\right)}{p} \tag{40}$$

The nature of the impact and the displacements of various points of the pipe obtained using the developed calculation programs are shown in figure 4.

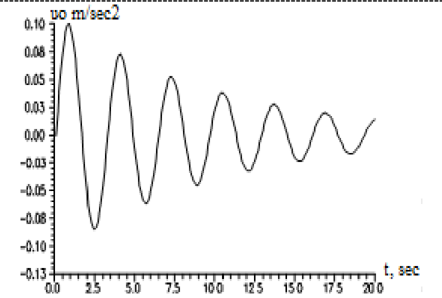


a

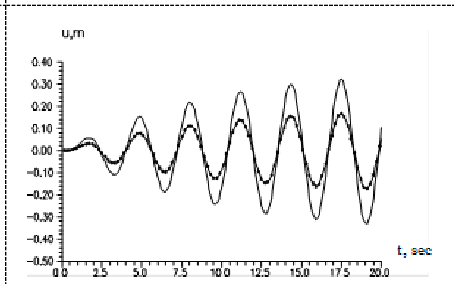


b

Figure 3. Forced vibrations of various points of the pipe at a long pulse (T=5 sec): a) the nature of the impact; b) oscillations of the pipe points: -*-*-*-* - at a height of z = 200m; ____ - at a height of z = 325m



a



b

Figure 4. Forced vibrations of various points of the pipe under damping effect: a) the nature of the impact; b) oscillations of the pipe points: -*-*-*-* - at a height of z = 200m; ____ - at a height of z = 325m

Analyzing the results obtained and comparing them with the results in figures 1-3, it can be noted that in this case, the amplitude of points oscillations is much greater than the amplitude under pulse impacts, but, unlike the case of the resonance mode, it does not increase infinitely, but is set at a certain mark (for the point z = 325 m it is ≈3 m), after which the free vibrations of the pipe continue with such amplitude.

3.5.Real accelerogram of the Gazli earthquake.

Dynamic behavior of a high-rise structure under real seismic impact is studied, the impact is taken in the form of the accelerogram of the Gazli earthquake (1971), represented by the horizontal component [28]. This accelerogram refers to high-frequency ones and corresponds to an earthquake of 8 points, has a predominant period of T = 0.1 seconds, and a duration of 8.74 seconds. A record of the accelerations of points on the earth's surface in one of the horizontal directions is shown in figure 5a. The resulting data obtained using the developed program for calculating the pipe points displacements are shown in figure 5b.

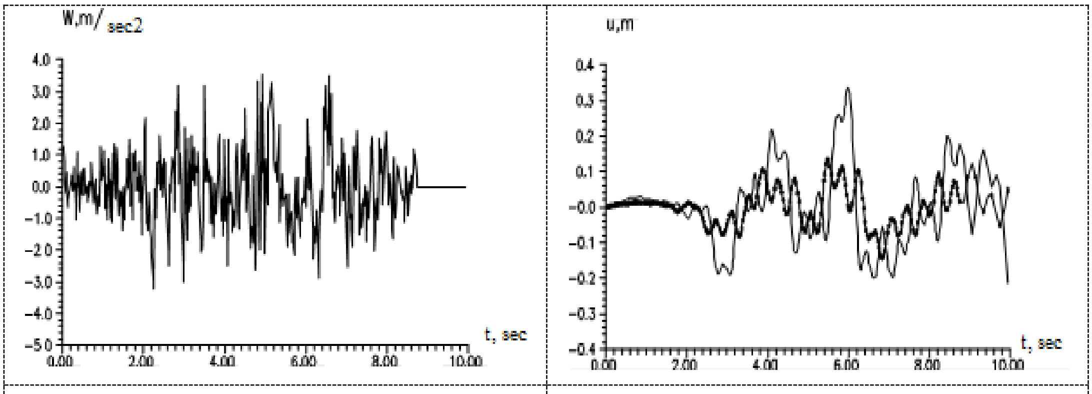


Fig.5. Forced vibrations of various points of the pipe under the real seismic impact (accelerogram of the Gazli earthquake): a) -impact; b) oscillations of the pipe points: -*-*-*-* - at a height of z = 200 m; ____ - at a height of z = 325 m

An analysis of results shows that the horizontal displacements of the points of a high-rise structure, in this case, are characterized by a rather long transition period with insignificant deviations of points from the neutral position (up to about 2 seconds), then, with a further increase in the impact amplitude, the point oscillations increase to the maximum, coinciding in time with the maximum accelerogram values. Toward the end of the impact, the displacements of the pipe points acquire the character of free vibrations, with a significant effect of higher modes of vibrations.

This is due to the difference in the period of the fundamental tone of structural vibrations and the predominant period of the accelerogram, i.e. the most energy-intensive vibration modes corresponding to lower frequencies practically do not respond to such an effect. At the same time, high-frequency eigenoscillations corresponding to higher frequencies respond to such an effect, but since they are less energy-intensive, the point oscillations are insignificant, as can be seen in the figure.

4. Conclusions

Based on the above studies, the following conclusions can be drawn:
1. A generalized approach to solving problems of unsteady forced vibrations of a high-rise structure was developed taking into account the finite wave propagation velocity and non-linear strain of the material.
2. Based on the developed algorithm and calculation program, a dynamic calculation of high axisymmetric structures was carried out taking into account their real geometry.
3. Based on the results of the studies, to analyze the behavior of the high-rise structure under various kinematic effects, several practical problems have been solved.
4. As a result, it was found that the dynamic response of a high-rise structure is affected not only by the magnitude of dynamic load but also by its frequency spectrum and duration: under the impact with a frequency close to the eigenfrequency of the structure, significant displacements occur in the latter and if the material does not possess dissipative properties, the structure can collapse during the resonant mode.

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