

CALCULATION OF DISPLACEMENTS OF PLATE ELEMENTS OF BOX-SHAPE STRUCTURE OF BUILDINGS

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The article investigates the dynamic behaviour of a box-shaped building structure subjected to seismic excitation modelled as a sinusoidal base displacement. The structure is represented as a spatial system of interconnected plate and beam elements, accounting for bending, shear, and torsional effects. The problem is solved numerically using the finite difference method. Calculations are performed for three dimensionless excitation frequencies $\beta_1 = 16$, $\beta_2 = 17$, and $\beta_3 = 17.5$, with a base displacement amplitude of $A_0 = 0.02$ m. Material properties are taken as $E_b = 20000$ MPa and $E_c = 7500$ MPa, with geometric parameters $a = 3.5$ m, $b = 5$ m, and $h_c = 0.2$ m. Results show that as the excitation frequency approaches the natural frequency ($\beta \approx 17.5$), resonance leads to a significant increase in displacements. Maximum dimensionless deflections (W/A_0) occur in the upper regions of bending panels and are notably higher near openings compared to solid sections. The presence of openings increases displacement values, particularly along their edges, while beam and plate elements exhibit consistent deformation patterns.

Keywords: *Box-shaped building structure, displacements of constructions, dynamic impact.*

1. INTRODUCTION

Modern construction imposes high demands on the reliability and stability of structures, especially in seismic zones. When designing and analysing buildings, assessing the behaviour of their elements under external loads remains a relevant task. Plate elements of box-type building structures, such as walls, floors, and others, play a key role in ensuring the stability of a building. They are subjected to both static and dynamic effects, requiring precise calculations to determine their spatial state. Various methods, including the finite difference method, are used to evaluate the behaviour of plate elements in box structures under seismic activity, allowing for accurate numerical solutions.

The study [1] investigated the free and forced vibrations of a sandwich beam with a porous core and composite face layers, analysed the effects of various parameters. The paper [2] examined the nonlinear free vibrations of a fully geometrically exact Timoshenko–Ehrenfest beam and employed numerical methods to account for nonlinearities. In these articles, dynamic properties were determined, and the results were compared with existing literature, highlighting their significance in engineering and structural design.

Articles [3], [4] were devoted to dynamic calculations of elements of box-shaped buildings for seismic resistance, considering the spatial work of box-shaped elements. In this case, the dynamic effect was specified as harmonic oscillations of the base movements according to a sinusoidal law. The equations of motion were given for each of the plate and beam elements of the box-shaped building structure based on the Kirchhoff–Love theory.

Studies [5], [6] focused on improv-

ing the box model of a building structure, considering the contact conditions between panel and beam elements. Equations of motion for the box elements were derived, and displacement graphs for the plates and beams were constructed. The articles examined the problem of forced vibrations in a spatial box-type building, consisting of rectangular panels and interacting beams under dynamic impact, defined by a sinusoidal base displacement. The finite difference method was used to solve the problem.

In works [7]–[10], the dynamic characteristics and vibrations of various axisymmetric and flat structures were considered, taking into account various geometries, spatial factors and inelastic properties of materials. The solution of the problem was carried out by the finite element method and the expansion of the solution in terms of natural vibration modes. Various mechanical effects associated with the geometry of the structure and the inelastic properties of the material were identified.

The articles [10], [11] studied the dynamic analysis of elements under the influence of seismic and vibration loads. A method for taking into account inertial forces in dynamic analysis was proposed. The paper [12] considered classical analytical methods for calculating displacements based on the Kirchhoff–Love thin plate theory. The main assumptions and limitations of these methods were considered.

The paper [13] presented the modal analysis of a thin rectangular plate simply supported. The analytical solution of the differential equation was obtained by applying the method of separating the variables. Numerical methods such as the finite element method allowed approximating these functions with greater accuracy. The results

obtained by MEF through Ansys 15.0 were then compared with those obtained by the analytical method.

The present article aims at developing a theoretical calculation methodology for plate elements of the box structure in large-panel buildings under seismic effects, con-

sidering the spatial interaction of transverse and longitudinal elements. The building is modelled as a spatial box structure consisting of beam and plate elements, as shown in Fig. 1. It is assumed that the lower part of the building moves together with the foundation under dynamic loading.

2. THEORETICAL BACKGROUND

Let us assume that the load-bearing elements of the building (plate elements 1, 3), located perpendicular to the direction of seismic impacts, work only on transverse dynamic bending. The deflections of the plate are designated by $W(x, y, t)$. The plate elements (plate elements 2, 4), located in the direction of the external impact, are subject to tension-compression and shear in their plane OXZ. The displacements of the plate elements working on shear are designated by $W(x, y, t)$. The floor (plate element 5) is

also deformable. The law of motion of the floor is determined in accordance with the forms of deformations of the upper edges of the vertical contacting plate elements.

The lower part of buildings under seismic impact in the direction of the OZ axis moves according to a given law [5], [7].

$$U_0 = A_0 \sin \omega_0 t, \quad (1)$$

where A_0 and ω_0 – the amplitude and frequency of forced oscillations.

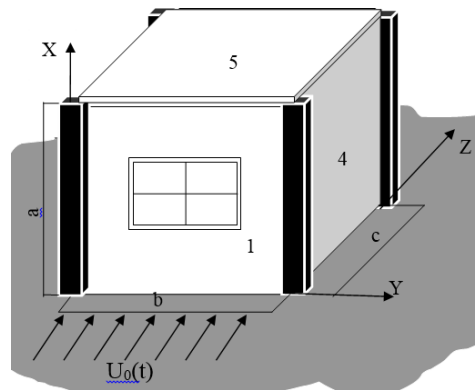


Fig. 1. Structural model of the building.

The article introduces the following designations for the plate elements of the building box: E_b , ρ_b , b , h_b and ν_b are the modulus of elasticity, density, width, thickness and Poisson's ratio of bending plate elements, E_c , ρ_c , c , h_c and ν_c are the modulus of elasticity, density, width, thickness and

Poisson's ratio of plate elements working in shear.

E_n , ρ_n , h_n and ν_n – elastic modulus, density, thickness and Poisson's ratio of the floor, G_n – shear modulus of the floor $G_n = \frac{E_n}{2(1+\nu_n)}$. All elements of box height are the same. It is considered that all beam

elements have a square section of size h_b , h_b from the same material. Elasticity and shear moduli of beams are designated E and G , Poisson's ratios ν and density ρ , moments of inertia of the beam section in bending

and torsion J and I_{kr} .

The expressions of bending and torque moments of bending plate elements M_{xx} , M_{yy} and M_{xy} are introduced using the formulas:

$$M_{xx} = -D \left(\frac{\partial^2 W}{\partial x^2} + \nu_b \frac{\partial^2 W}{\partial y^2} \right), M_{yy} = -D \left(\nu_b \frac{\partial^2 W}{\partial x^2} + \frac{\partial^2 W}{\partial y^2} \right), M_{xy} = -D(1 - \nu_b) \frac{\partial^2 W}{\partial x \partial y}, \quad (2)$$

where $D = \frac{E_b h_b^3}{12(1-\nu_b^2)}$ – cylindrical rigidity in transverse bending.

The expressions of the longitudinal and tangential forces of plate elements working

in shear are presented in the form:

$$P_{zz} = B \left(\frac{\partial u}{\partial z} + \nu_c \frac{\partial v}{\partial x} \right), P_{xx} = B \left(\nu_c \frac{\partial u}{\partial z} + \frac{\partial v}{\partial x} \right), P_{xz} = B(1 - \nu_c) \left(\frac{\partial u}{\partial x} + \frac{\partial v}{\partial z} \right). \quad (3)$$

where $B = \frac{E_c h_c}{1-\nu_c^2}$ – cylindrical rigidity of plate elements under tension and compression.

2.1. Method for Solving the Problem

A method for solving the problem of displacement of elements of a large-panel building box is proposed, taking into account spatial deformations with full contact conditions.

Based on representation (1), the general kinematic law of motion of the box is presented as the sum of the base displacement function $U_0(t)$ and the relative displacements of the plates and beams:

$$u_3 = U_0(t) + W(x, y, t), \quad u_1 = U_0(t) + u(x, z, t), u_2 = v(x, z, t), u_3^{(i)} = U_0(t) + W^{(i)}(x, y, t). \quad (7)$$

The displacements of the plate and beam elements are written in the form:

$$W = W(x, y) \sin(\omega_0 t), \quad u = u(x, z) \sin(\omega_0 t), v = v(x, z) \sin(\omega_0 t), \quad W^{(i)} = W^{(i)}(x, y) \sin(\omega_0 t), \alpha^{(i)} = \alpha^{(i)}(x, y) \sin(\omega_0 t). \quad (8)$$

Let us consider the theoretical calculation of the box of a large-panel building under seismic impact, taking into account the spatial work of the elements. Let us compose the equations of motion for each plate and beam element of the box structure

of the building [5]–[7].

The plate elements are considered to be thin elastic plates, subject to the Kirchhoff-Love hypothesis. When constructing the equations of motion and boundary conditions of the plates and beams, we will use

the expressions of moments (2)–(3).

The system of equations for bending, torque and shear forces is presented in the form:

$$\frac{\partial M_{xx}}{\partial x} + \frac{\partial M_{xy}}{\partial y} - Q_x = 0,$$

$$\frac{\partial M_{xy}}{\partial x} + \frac{\partial M_{yy}}{\partial y} - Q_y = 0, \quad (9,a)$$

$$\frac{\partial Q_x}{\partial x} + \frac{\partial Q_y}{\partial y} = \rho_b h_b \ddot{W} + \rho_b h_b \ddot{U}_0. \quad (9,b)$$

The system of equations of motion of plate elements working in shear is taken as:

$$\frac{\partial P_{zz}}{\partial z} + \frac{\partial P_{xz}}{\partial x} = \rho_c h_c \ddot{u} + \rho_c h_c \ddot{U}_0,$$

$$\frac{\partial P_{zx}}{\partial z} + \frac{\partial P_{xx}}{\partial x} = \rho_c h_c \ddot{v}. \quad (10)$$

The system of equations for bending

and torsional vibrations of beams has the form:

$$\frac{\partial M^{(i)}}{\partial x} - Q^{(i)} + \frac{h_b}{2} P_{zx}^c = 0, \quad (11,a)$$

$$\frac{\partial Q^{(i)}}{\partial x} + \rho^{(i)} F W^{(i)} = R_y - P_z^c - \rho_{(i)} F \ddot{U}_0, \quad (11,b)$$

$$\frac{\partial M_{kr}^{(i)}}{\partial x} = \rho_i I_{kr} \ddot{\alpha}^{(i)} + M_{yy}^b + \frac{h_c}{2} R_y. \quad (11,c)$$

where GI_{kr} – beam torsional rigidity.

The boundary conditions on the base of the building box are specified as for rigid clamping. The lower part of the building moves together with the base, and there is no rotation. The boundary conditions at the base of the building frame are specified as fixed (rigid clamping). The lower part of the structure moves together with the foundation, and rotation is absent.

$$u_1 = u_3 = u_3^{(i)} = U_0(t), u_2 = 0, \frac{\partial W}{\partial x} = 0, \frac{\partial W^{(i)}}{\partial x} = 0, \alpha^{(i)} = 0. \quad (12)$$

The boundary conditions (12) taking into account (7), are rewritten as:

$$W = 0, \frac{\partial W}{\partial x} = 0, u = 0, v = 0, W^{(i)} = 0, \frac{\partial W^{(i)}}{\partial x} = 0, \alpha^{(i)} = 0. \quad (13)$$

In the zone of connection of plate elements, full contact conditions are specified, ensuring equality of displacements

and stresses. Contact conditions in the zone of connections of beam and plate elements working in shear are written in the form:

$$u(x, z, t)_{z=c_i} = W^{(i)}(x, t), v(x, z, t)_{z=c_i} = \pm \frac{h_b}{2} \frac{\partial W^{(i)}(x, t)}{\partial x},$$

$$W(x, y, t)_{y=b_i} = W^{(i)}(x, t), \left(\frac{\partial W(x, y, t)}{\partial y} \right)_{y=b_i} = -\alpha^{(i)}. \quad (14)$$

The displacements of the upper points of beam and plate elements operating in

bending and shear are denoted by

$$W_a(y, t) = W(a, y, t), u_a(y, t) = u(a, z, t), v_a(z, t) = v(a, z, t). \quad (15)$$

Based on notation (15), the law of distribution of the displacement of overlap points will be defined by the expressions:

$$\begin{aligned} u_n(z, y, t) &= W_a(y, t) + u_a(z, t) - W^{(i)}(a, t) \\ v_n(z, y, t) &= v_a(z, t). \end{aligned} \quad (16)$$

The contact conditions at the joints of the floor and plate elements operating in bending are of the form:

$$-R_x + \eta_0 \rho_{\Pi} h_b h_{\Pi} \ddot{W}_a = h_b h_{\Pi} \frac{\partial \tau_{zy}^n}{\partial y} - \eta_0 \rho_{\Pi} h_b h_{\Pi} \ddot{U}_0, M_{xx}^a = 0, \quad (17)$$

where: $\tau_{zy}^n = G_n \left(\frac{\partial W_a}{\partial y} \right)$ – shear stress of the floor at its edge $z = c$, $\eta_0 = \frac{2(bh_b + ch_c)}{bc}$.

The contact conditions at the joints of the floor and the post-and-beam elements are written as:

$$M^{(i)} = 0, -Q^{(i)} + \eta_0 \rho_{\Pi} h_b h_{\Pi} \ddot{W}^{(i)} = -\eta_0 \rho_{\Pi} h_b h_{\Pi} \ddot{U}_0, M_{kr}^{(i)} = 0. \quad (18,a)$$

The contact conditions at the joints of the floor and plate elements operating in shear, with respect to the contact tangential and normal stresses, are written in the form:

$$\begin{aligned} -ch_c \tau_{zx}^c + \eta_0 m_{\Pi} \ddot{u}_a &= ch_c h_{\Pi} \frac{\partial \sigma_{zz}^n}{\partial z} - \eta_0 m_{\Pi} \ddot{U}_0, \\ -ch_c \sigma_{xx}^c + \eta_0 m_{\Pi} \ddot{v}_a &= ch_c h_{\Pi} \frac{\partial \sigma_{xz}^n}{\partial z}, \end{aligned} \quad (18,b)$$

where $\sigma_{zz}^n = E_{\Pi} \left[\frac{\partial u_a}{\partial z} \right]$, $\sigma_{zx}^n = G_{\Pi} \left[\frac{\partial v_a}{\partial z} \right]$ – normal and shear stresses of the floor at its edge $y = b$, $\sigma_{xx}^c = E_c \left[\frac{\partial v}{\partial x} + \nu_c \frac{\partial u}{\partial z} \right]_{x=a}$, $\tau_{zx}^c = G_c \left[\frac{\partial v}{\partial z} + \frac{\partial u}{\partial x} \right]_{x=a}$, – normal and shear stresses of plate elements working in shear at its upper edge $x = a$.

As boundary conditions on the contour of the window opening, we set the same as for the free edge (Fig. 2):

$$M_{xx} = 0, R_x = 0, \text{ at } x = \text{const (on the contour AB and CD)}, \quad (19,a)$$

$$M_{yy} = 0, R_y = 0, \text{ at } y = \text{const (on the contour AC and BD)}. \quad (19,b)$$

At each internal corner point of the opening A, B, C, D, we have five boundary conditions:

$$M_{xx} = 0, R_x = 0, M_{yy} = 0, R_y = 0, M_{xy} = 0, \text{ at } x = \text{const}, y = \text{const}. \quad (20)$$

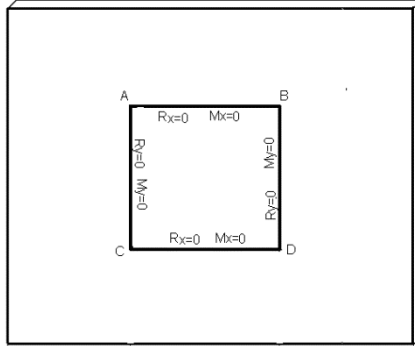


Fig. 2. Flexible plate element with a cut-out.

The vibration modes (8) must satisfy the equations of motion (9)–(11), boundary conditions (13), (19) and (20), and contact conditions (14), (17) and (18).

The general solution to the problem of

forced vibrations of a bending plate element of a box is described by a function represented as the sum of the solution to the problem of forced and natural vibrations:

$$W(x, y, t) = A_0 W_v(x, y) \sin \omega_0 t + \sum_{i=1}^N C_i W_i(x, y) \sin p_i t. \quad (21)$$

The solution to the problem of forced oscillations of a plate element operating in

bending is expressed through the natural modes of oscillations:

$$W_v(x, y) = \sum_{i=1}^N A_i W_i(x, y), \quad (22)$$

where $A_i (i = 1, 2, 3, \dots)$ – expansion coefficients.

Based on the expression $C_1 = -A_0 A_1 \frac{\omega_0}{p_1}$ and taking into account (22), we obtain a gen-

eral solution to the problem for a bending plate element in the form [5]–[7]:

$$W(x, y, t) = A_0 \left(\sin \omega_0 t - \frac{\omega_0}{p_1} \sin p_1 t \right) W_v(x, y). \quad (25)$$

The expression for the displacement of a plate element working in shear has the form:

$$\begin{aligned} u(x, z, t) &= A_0 \left(\sin \omega_0 t - \frac{\omega_0}{p_1} \sin p_1 t \right) u_v(x, z), \\ v(x, z, t) &= A_0 \left(\sin \omega_0 t - \frac{\omega_0}{p_1} \sin p_1 t \right) v_v(x, z). \end{aligned} \quad (26)$$

The problem of determining the unknown coordinate functions in expressions (25)–(26) is solved by the finite difference method. For convenience of calculations, the frequency of forced oscillations ω_0 is presented in the form:

$$\tilde{\omega}_k = \frac{\beta_k}{b^2} \sqrt{\frac{D}{\rho H}},$$

3. RESULTS AND DISCUSSION

In the calculations, the values of the frequency of the external impact were carried out for three values of the dimensionless frequency of forced oscillation $\beta_1=16$, $\beta_2=17$, $\beta_3=17.5$, and the amplitude of the base displacement $A_0=0.02$ m.

Note that the value of the third frequency is very close to the first natural frequency and the building's movement goes into resonant mode.

The following parameters are specified as the initial data for the box elements.

Moduli of elasticity of plate elements operating in bending and shear are $E_b=20000\text{MPa}$, $E_c=7500\text{MPa}$. Poisson's ratio of the materials of plate elements is $\nu_b = \nu_c = \nu_c = 0.3$.

Set the following geometric dimensions: for bendable plate elements 1 and 3, thickness, height $a = 3.5$ m and length $b = 5$ m, and for plate elements 2 and 4, thickness $h_c = 0.2$ m; length $c = 4$ m. Let us proceed to the discussion of the obtained numerical results on the bending moment on bending plate elements. The transverse dimensions of the beam elements are equal, respectively, to the thicknesses of the bending plate elements and the plate elements working in shear.

where the values of β_k are specified, and the frequency of the external action is determined through the parameters of the panels.

The study develops a method for dynamic calculation of the box structure of buildings based on the method of finite-difference schemes.

In the graphs, the solid lines W_v show the results of plate elements with an opening, and the dotted lines W show the results without openings, n is the interval (step) of finite differences. Figure 3 shows a graph of the change in the dimensionless deflection W/A_0 along the vertical (Fig. 2 $M_y=0$, $R_y=0$) at the right edge of the opening of the bending plate elements. The calculation results show that the deflection values at the edges of the opening of the plate elements are greater than the results without an opening. The values of the frequencies of forced vibration vary near the values of natural frequencies. The maximum deflection value is found at the second upper half of the height of the bending panel.

Figure 4 shows the change in dimensionless deflection by width at the lower edge of the opening (Fig. 2 $M_x=0$, $R_x=0$) of the bending plate elements. Figure 5 demonstrates the laws of change in deflection by width (deflection diagram) at the upper edge of the opening of the bending plate elements. The solid lines $W(k)v$ show the results of plate elements with an opening, and the dotted lines $W(k)$ reveal the results without openings, n is the interval (step) of finite differences.

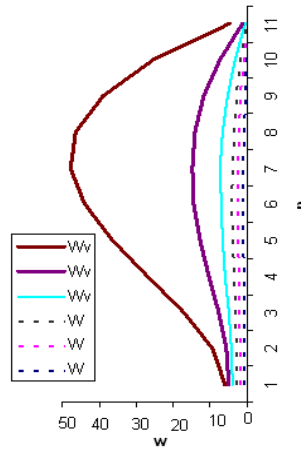


Fig. 3. Laws of change in the vertical dimensionless maximum deflection W/A_0 , at the right edge of the opening for three values of the dimensionless frequency of forced oscillation of plate elements operating in bending in resonant mode.

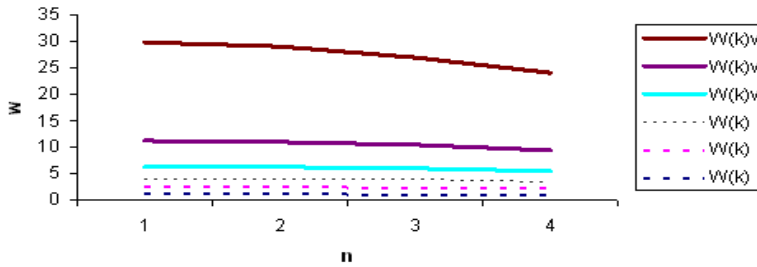


Fig. 4. Laws of change in the vertical dimensionless maximum deflection W/A_0 , at the lower edge of the opening for three values of the dimensionless frequency of forced oscillation β of plate elements operating in bending in resonant mode.

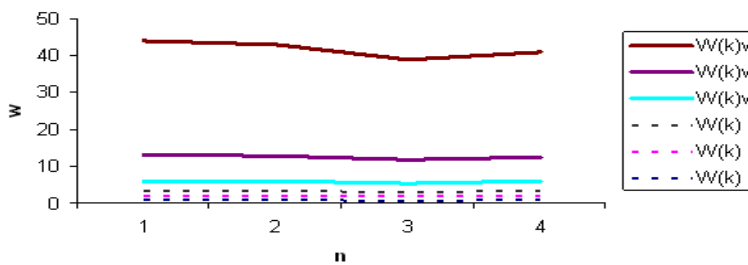


Fig. 5. Laws of change in the vertical dimensionless maximum deflection W/A_0 , at the upper edge of the opening for three values of the dimensionless frequency of forced oscillation β of plate elements operating in bending in resonant mode.

Figure 6 shows a graph of the dimensionless deflection of a vertical beam element, the solid lines $W_{(m)v}$ demonstrate the results of plate elements with an opening,

and the dotted lines $W_{(m)}$ show the results without openings, n is the interval (step) of finite differences.

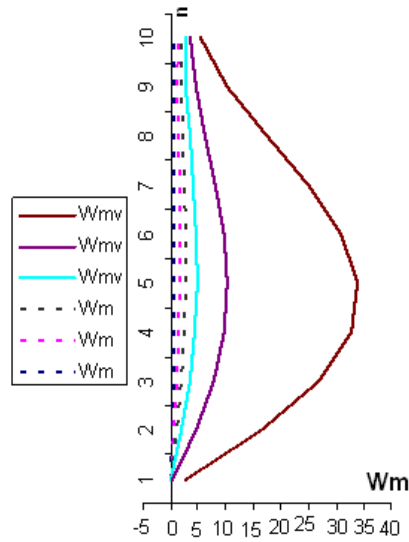


Fig. 6. Changes in the vertical dimensionless maximum deflection W/A_0 at three values of the dimensionless frequency of forced vibration β of beam elements in the resonant mode.

Comparing Fig. 3 and Fig. 6, we conclude that the deflection of the vertical edge of the bent panel is less than the deflection at the edge of the opening. This is explained by the fact that the edges of the panel are

held by transverse plate elements that work in shear. This means that the bending shapes of the beam elements and plate elements are identical.

4. CONCLUSION

The equations of motion of points of plate and beam elements of the building box, boundary, contact and initial conditions of the problem of forced vibrations are provided in the paper. Within the frame-

work of finite difference methods, a technique for dynamic calculation of displacements of plate and post-and-beam elements of box structures of buildings has been developed in the research.

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