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METHODOLOGICAL RECOMMENDATIONS

for independent and practical work
on the subject

«DESIGN OF HIGH-RISE BUILDINGS»

(for students of the second (master's) level of higher education all forms
of education specialty 192 – Building and Civil Engineering,
of education program "Industrial and civil construction")

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CONTENT

INTRODUCTION ……………………………………………………………………...4

1 CALCULATION OF STATIC STABILITY OF A BUILDING 5

   1.1 Example of assessing the static stability of a building 9

2 DYNAMIC STABILITY 18

   2.1 Example of assessing the dynamic stability of a building................................. 25

3 SPECIAL CASES 32

REFERENCES... 36

INTRODUCTION

The academic discipline "Design of High-rise Buildings" is one of the key ones in the training of specialists in the field of civil engineering and architecture. It covers a wide range of issues related to the design of complex high-rise facilities, taking into account structural, engineering, regulatory and safety requirements. A special role in mastering the discipline is played by practical classes and independent work, which contribute to the consolidation of theoretical knowledge and the development of engineering analysis skills.

These methodological recommendations are compiled in order to ensure the effective organization of the educational process, provide methodological instructions for performing typical tasks, as well as develop students' ability to independently make constructive decisions. The main attention is paid to the analysis of the stability of high-rise buildings - both in static and dynamic aspects.

The first section considers the methodology for calculating the static stability of a building, in particular with an example of its practical application. The second section is devoted to the analysis of dynamic stability, which is an important factor under the action of wind, seismic and other variable loads. The third section covers special cases that require a non-standard engineering approach.

The proposed materials are aimed at higher education students studying in architectural and construction specialties, and are aimed at increasing the level of professional training, practical validity of design solutions, and the ability to work in conditions of complex structural systems of high-rise construction.

1 CALCULATION OF STATIC STABILITY OF A BUILDING

The stability assessment is performed assuming the following hypotheses are true:

- a computational model of a building is a system of structures held in a state of stable equilibrium by a group of pylons (cantilever rods) clamped at the base;
- the stiffness of the pylons is constant along the height;
- the mass of the building is evenly distributed throughout its volume;
- The deformations of the floors in their plane are sufficiently small.

Under these assumptions, the problem reduces to integrating the following system of equations:

$$\begin{cases} B_y \dfrac{d^4U}{dx^4} - q_x = 0 \\ B_x \dfrac{d^4W}{dx^4} - q_y = 0 \\ B_{cr} \dfrac{d^2\varphi}{dx^2} - m = 0 \end{cases} \tag{1.1}$$

Under boundary conditions corresponding to Kirchhoff's conditions at the upper end of the core (roof):

$$M_x=M_y=0; \qquad V_x=V_y=0 \tag{1.2}$$

and at the edge of the foundation to the conditions of full clamping:

$$U=W=0; \qquad \frac{dU}{dx}=\frac{dW}{dx}=0; \qquad \varphi=\frac{d\varphi}{dx}=0 \tag{1.3}$$

where U, W, φ are the displacements of the core points along the X and Y axes and the twist angle;

B_x, B_y, B_{cr} – bending and torsional stiffness of the core;

q_x, q_y – transverse loads along the coordinate axes.

Taking solution (1.1) in the form:

$$U=C_1f(z) \qquad W=C_2f(z); \quad \varphi=C_3f(z), \tag{1.4}$$

$$\text{where } f(z)=1-\cos\lambda z; \quad \lambda=\frac{n\pi}{2H}, \tag{1.5}$$

C_1, C_2, C_3 – permanent;

N – building height:

the origin is located in the clamp, after the transformation we obtain that for $C_1 \neq 0,\ C_2 \neq 0,\ C_3 \neq 0$

$$\begin{vmatrix} (G_y - G_{кр}) & 0 & G_{кр} \cdot a_y \\ 0 & (G_x - G_{кр}) & -G_{кр} \cdot a_x \\ G_{кр} \cdot a_y & -G_{кр} \cdot a_x & (G_w - G_{кр}) \end{vmatrix} = 0 \tag{1.6}$$

The critical weight of the building (the minimum value of the three roots) is determined from the solution of equation (1.6):

$$A_1 G_{cr}^3 - A_2 G_{cr}^2 + A_3 G_{cr} - A_4 = 0, \tag{1.7}$$

where $A_1 = 1 - (a_x^2 + a_y^2)/\gamma$ (Fig. 1.1);

$$A_2 = G_{[} + G_y + G_\omega + G_x \frac{a_y^2}{\gamma} + G_y \frac{a_x^2}{\gamma};$$

$$A_3 = G_x G_y + G_x G_\omega + G_y G_\omega;$$

$$A_4 = G_x \cdot G_y \cdot G_\omega;$$

$$G_x = \frac{2{,}3 E_{cm} I_x}{H_o^2};\ G_y = \frac{2{,}3 E_{cm} I_y}{H_o^2};\ G_\omega = \frac{2{,}3 E_{cm} I_\omega}{\gamma H_o^2};$$

$H = 1.1H_0$; H_0 – height of the above-ground part of the building.

The factor 1.1 approximately takes into account the effect of the flexibility of the foundation on the critical weight of the building.

E_{st} – modulus of deformation of the 1st type of concrete of the base pylon (core);

For buildings with one stiffening core:

$$G_\omega = \frac{0{,}14 E_{cm} I_{кр}}{\gamma};$$

$$I_x = \sum^M I_{xi};\quad I_y = \sum^M I_{yi};\quad I_{xy} = \sum^M I_{xyi};$$

$$I_\omega = \sum^M I_{xi}(a_i - a_o)^2 + \sum^M I_{yi}(b_i - b_o)^2 - 2\sum^M I_{xyi}(a_i - a_o)(b_i - b_o) + \sum^M I_{\omega i};$$

a_0, b_0 – coordinates of the center of curvature of the building in an arbitrary coordinate system of the XOY (Fig. 1.1);

M – number of floors of rigidity in the building (pylons, cores);

a_i, b_i – coordinates of the center of curvature of the i-th pylon;

$$a_o = A_y\left(\sum^{M} I_{xi}a_i - \sum^{M} I_{xyi}b_i\right) - A_{xy}\left(\sum^{M} I_{xyi}a_i - \sum^{M} I_{yi}b_i\right);$$

$$b_o = A_x\left(\sum^{M} I_{yi}b_i - \sum^{M} I_{xyi}b_i\right) - A_{xy}\left(\sum^{M} I_{xyi}b_i - \sum^{M} I_{xi}a_i\right);$$

$$A_x = \frac{I_x}{I_x I_y - I_{xy}^2},\ A_y = \frac{I_y}{I_x I_y - I_{xy}^2},\ A_{xy} = \frac{I_{xy}}{I_x I_y - I_{xy}^2}$$

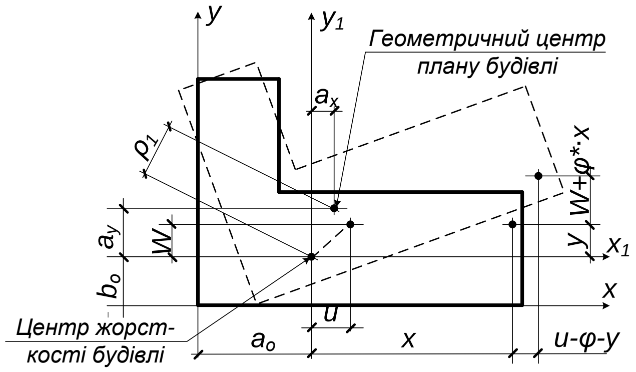


Figure 1.1 – Before calculating the stability of a building

If the main axes of the structure are parallel to the axes of the building:

$$a_o = \frac{\sum^{M} I_{xi}a_i - \sum^{M} I_{xyi}b_i}{Ix};\ b_o = \frac{\sum^{M} I_{xi}b_i - \sum^{M} I_{xyi}a_i}{Iy}.$$

If the main axes of all pylons are parallel to the axes of the building:

$$a_o = \frac{\sum^{M} I_{xi}a_i}{Ix};\ b_o = \frac{\sum^{M} I_{xi}b_i}{Iy};$$

I_{xi}, I_{yi}, I_{xyi} – the main reduced axial and centrifugal moments of inertia of the i-th pylon;

$I_{\omega i}$ – the reduced moment of inertia of the i-th pylon.

For a single stiffness core:

$$I_{\omega} = 0{,}05 I_{кр} \cdot H^2;$$

$$I_{кр} = \frac{\Omega^2}{\sum \frac{S_j}{\delta_j}};$$

Ω – double the area of the figure, which is limited by the midline of the pylon cross-section contour;

S_j – length of the section of the pylon contour of constant thickness δ_j.

Summing up the relationship $\frac{S_j}{\delta_j}$ extends over the entire contour of the cross section.

The geometric characteristics of the pylons (or/and their parts) are reduced in relation to the deformation modulus E_{st} of the base pylon:

$\gamma = \frac{\int_A (x^2 + y^2) dA}{A} = \frac{\int_A \rho^2 dA}{A}$ - description of the building plan;

A is the area of the building plan.

If the main axes of the building are not parallel to its axes (i.e. the centrifugal moment of inertia $I_{xy} \neq 0$) when calculating the critical weights G_x and G_y, instead of the moments of inertia I_x and I_y, I_{max} and I_{min} should be substituted, which are determined by the formula:

$$I_{\frac{\max}{\min}} = \frac{I_x + I_y}{2} \pm \sqrt{\left(\frac{I_x - I_y}{2}\right)^2 + I_{xy}^2}$$

The distance between the center of rigidity and the center of mass of the building is determined (Fig. 1.1):

$$\rho_1 = \sqrt{a_x^2 + a_y^2}\ .$$

It is permissible to take into account geometric nonlinearity approximately by introducing coefficients by which all longitudinal forces from the long part of the loads (~85%) should be multiplied. For three component displacements, these coefficients are equal to:

$$\eta_x^{long} = \frac{1}{1-\frac{G^n}{G_x}};\eta_y^{long} = \frac{1}{1-\frac{G^n}{G_y}};\eta_\omega^{long} = \frac{1}{1-\frac{G^n}{G_\omega}}$$

The most rational layout of the building involves the formation of a load-bearing system with a minimized distance between the center of mass and the center of rigidity, as well as minimal eccentricities of wind load.

1.1 Example of assessing the static stability of a building

As an example, let us consider the sequence and features of determining the critical forces of loss of stability of the load-bearing structure system of a 20-story residential building (18 residential floors and a 2-story parking lot). The plan and cross-section of the building are shown in Fig. 1.2 and 1.3. The building structures are made of monolithic reinforced concrete.

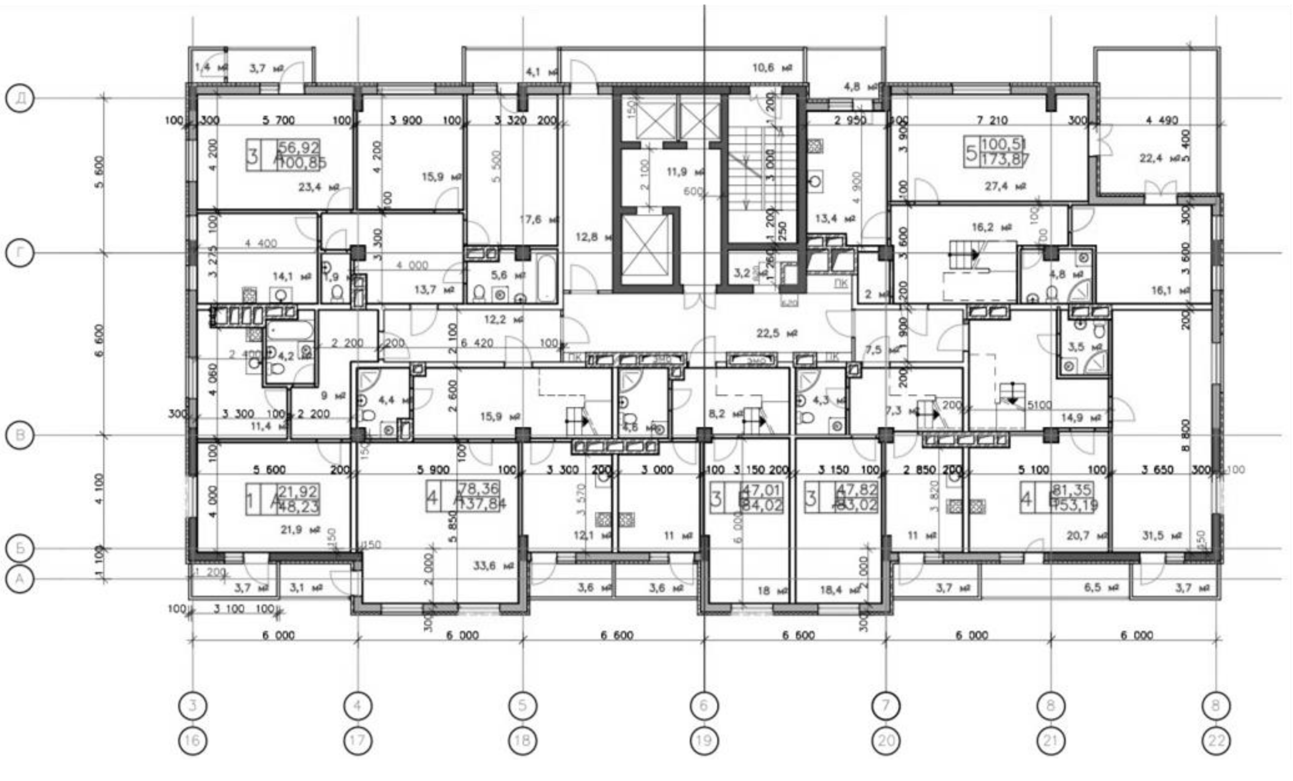


Figure 1.2 – Typical building floor plan

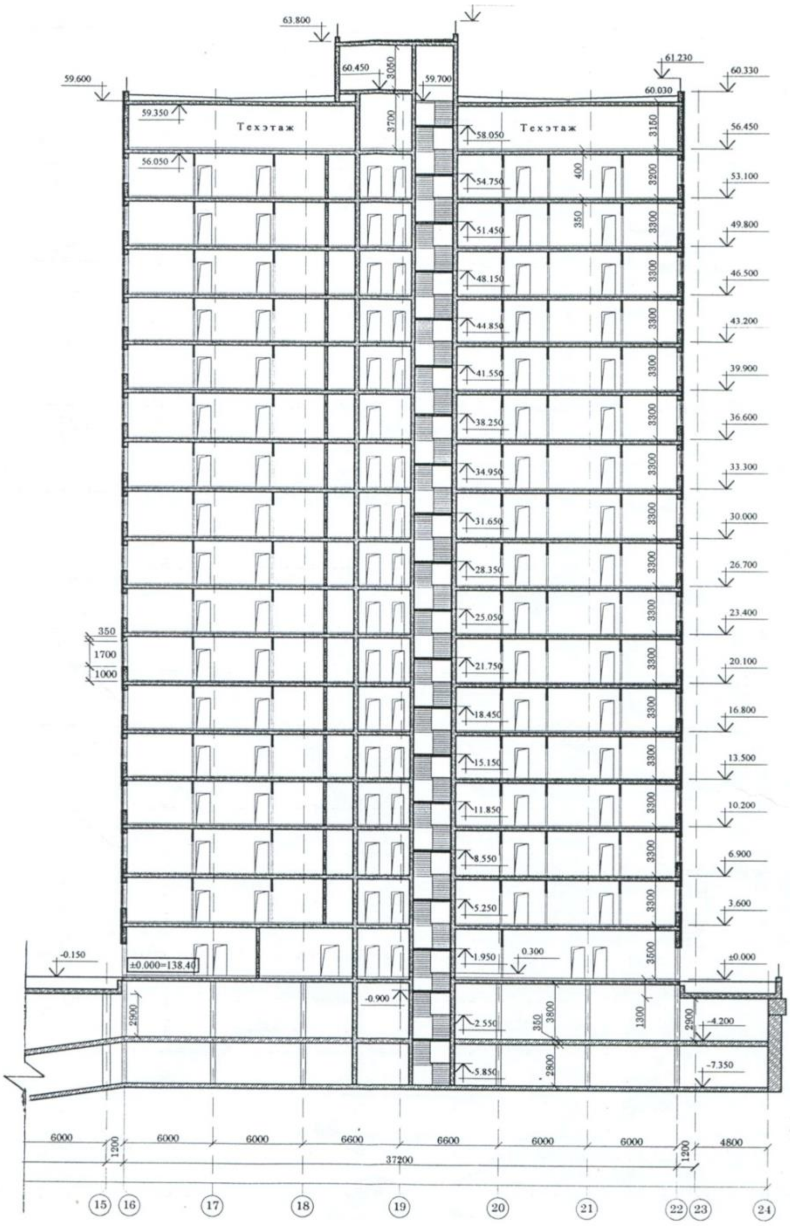


Figure 1.3 – Longitudinal section of the building

Let us define the geometric characteristics of the building's stiffness core:

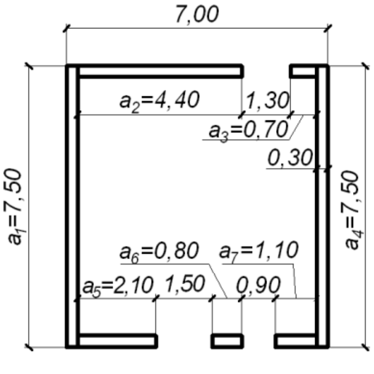


Figure 1.4 – Cross section of the stiffener core design

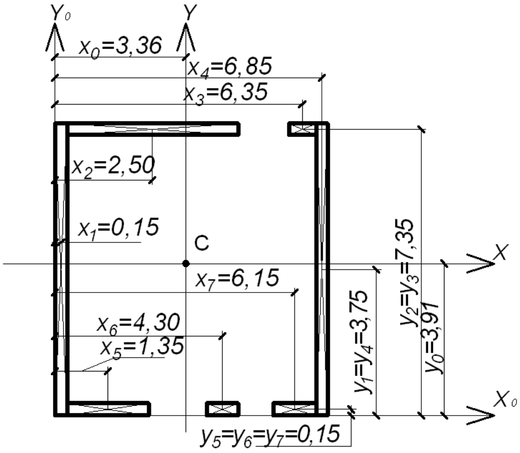


Figure 1.5 – Before determining the geometric characteristics of the stiffness core cross-section

Let us introduce the initial data to determine the main geometric characteristics of the cross-section (Figure 1.4, 1.5), m:

$a_1=7.5;$ $a_2=4.4;$ $a_3=0.7;$ $a_4=7.5;$ $a_5=2.1;$ $a_6=0.8;$ $a_7=1.1$
$y_1=3.75;$ $y_2=7.35;$ $y_3=y2;$ $y_4=y1;$ $y_5=0.15;$ $y_6=y_5;$ $y_7=y_5$ $d=0.3$
$x_1=0.15;$ $x_2=2.5;$ $x_3=6.35;$ $x_4=6.85;$ $x_5=1.35;$ $x_6=4.3;$ $x_7=6.15$

Let us determine the net cross-sectional area *A*, m^2:

$$A = d \cdot (a_1 + a_2 + a_3 + a_4 + a_5 + a_6 + a_7) = 7{,}23$$

Let us determine the static moments of the section S_x and S_y, m^3:

$$S_x = d \cdot (a_1 \cdot y_1 + a_2 \cdot y_2 + a_3 \cdot y_3 + a_4 \cdot y_4 + a_5 \cdot y_5 + a_6 \cdot y_6 + a_7 \cdot y_7) = 28{,}301$$
$$S_y = d \cdot (a_1 \cdot x_1 + a_2 \cdot x_2 + a_3 \cdot x_3 + a_4 \cdot x_4 + a_5 \cdot x_5 + a_6 \cdot x_6 + a_7 \cdot x_7) = 24{,}295$$

Let us determine the coordinates of the center of gravity of the section X_0 and Y_0, m:

$$X_0 = \frac{S_y}{A} = 3{,}36$$
$$Y_0 = \frac{S_x}{A} = 3{,}914$$

Let us determine the main moments of inertia of the section J_x, J_y and J_{kr}, m^4:

$$J_x = \frac{d \cdot a_1^3}{12} + a_1 d(Y_o - y_1)^2 + \frac{a_2 \cdot d^3}{12} + a_2 d(7{,}5 + Y_o - 0{,}15)^2 + \frac{a_3 \cdot d^3}{12} + a_3 d(7{,}5 - Y_o - 0{,}15)^2 +$$

$$+ \frac{d \cdot a_4^3}{12} + a_4 d(Y_o - y_1)^2 + \frac{a_5 d^3}{12} + a_5 d(Y_o - 0{,}15)^2 + \frac{a_6 d^3}{12} + a_6 d(Y_o - 0{,}15)^2 +$$

$$+ \frac{a_7 d^3}{12} + a_7 d(Y_o - 0{,}15)^2 = 56{,}3$$

$$J_y = \frac{d^3 \cdot a_1}{12} + a_1 d(X_o - 0{,}15)^2 + \frac{a_2^3 \cdot d}{12} + a_2 d(X_o - x_2)^2 + \frac{a_3^3 \cdot d}{12} + a_3 d(x_3 - X_o)^2 +$$

$$+ \frac{d^3 \cdot a_4}{12} + a_4 d(x_4 - X_o)^2 + \frac{a_5^3 d}{12} + a_5 d(X_o - x_5)^2 + \frac{a_6^3 d}{12} + a_6 d(x_6 - X_o)^2 +$$

$$+ \frac{a_7^3 d}{12} + a_7 d(x_7 - X_o)^2 = 61{,}219$$

$$J_{kr} = \frac{4\left[(7-d)(7{,}5-d)\right]^2 \cdot d}{(7{,}5-d) \cdot 2 + \left(a_2 + \frac{d}{2}\right) + \left(a_3 + \frac{d}{2}\right) + \left(a_5 + \frac{d}{2}\right) + a_6 + \left(a_7 + \frac{d}{2}\right)} = 115{,}872$$

The calculated moment of inertia of the pylons is determined by multiplying the initial moments of inertia by the coefficients of uniformity of the pylons:

$$J_{ji} = k_j \cdot \overline{J}_{ji}\,.$$

The pylon uniformity coefficient, which takes into account the reduction in pylon stiffness, can be represented as:

$$k_j = K_{sh} \cdot K_p \tag{1.8}$$

The coefficient K_{sh} takes into account the flexibility of the connections between the pylon elements. In this case, K_{sh}=1, and K_p takes into account the influence of deformations of the jumpers above the openings. These coefficients are determined by the formula:

$$K_p = \frac{1}{1 + p_j} \tag{1.9}$$

where p_j is a coefficient characterizing the increase in pylon displacements due to deformations of the lintels above the openings.

To determine the calculated axial moments of inertia J_x and J_y, the coefficients p_j (p_x and p_y) can be determined by the formula [1]:

$$p_j = \frac{h\cdot l^3}{3\cdot J_n\cdot H^2}\cdot\frac{F_1\cdot F_2}{F_1+F_2}\cdot\left(1-\frac{J_1+J_2}{\overline{J}_j}\right) \qquad (1.10)$$

where F_1 and F_2 are the given cross-sectional areas of the pylon “branches”;

J_1 and J_2 – the reduced moments of inertia of the pylon branches;

J_j – the moment of inertia of the pylon, calculated without taking into account the influence of seams and holes on its stiffness;

h – floor height;

H – pylon height;

J_n – moment of inertia of the jumpers over the holes.

To determine the calculated moment of inertia during rotation, in formula (1.10) instead of p_j we substitute the coefficient p_{kr}.

$$p_{\text{кр}} = \frac{8\cdot\overline{J}_{\text{кr}}}{(a+b)^2}\cdot\left(\frac{c}{F}+\frac{h\cdot\nu}{30\cdot J_n\cdot\sum\frac{b_s^2}{l^3}}\right)-c \qquad (1.11)$$

where c – is the ratio of the pylon height to the floor height;

F – area of the pylon cross-section weakened by holes;

b_s – the distance between the centers of gravity of adjacent branches of the pylon in the direction parallel to the wall that has the opening (Figure 1.6);

a and b – dimensions of the pylon;

ν – is a coefficient whose numerical value depends on the ratio of the height of the opening to its width (table 1.1) [2].

Table 1.1 - Ratio of opening height to its width

hn/l	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0
ν	1.03	1.11	1.26	1.46	1.73	2.05	2.43	2.87	3.37	3.93

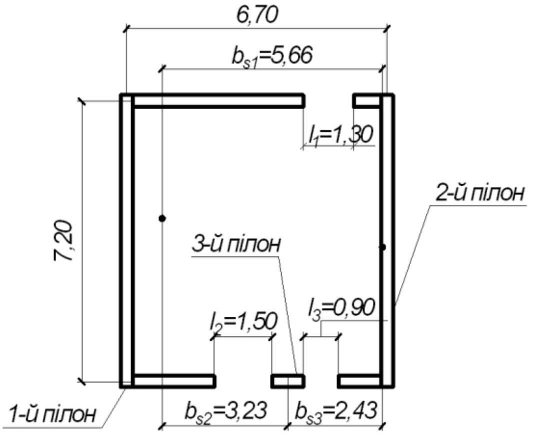


Figure 1.6 – Stiffness core

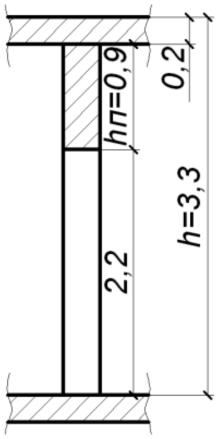


Figure 1.7 – Hole cross-section

To determine the coefficients p_j (p_x and p_y), the “worst” (having a large number of weakenings) pylons were selected: namely 5 and 6 (Figure 1.5).

$F_1=a_5 \cdot d\ m^2$; $h=3.3\ m$; $h_{\mathrm{n}}=0.9\ m$; $H=70\ m$; $F_1=0.63\ m^2$;

$F_2=a_6 \cdot d\ m^2$; $l=1.5\ m$; $F_2=0.24\ m^2$;

$$J_n = \frac{b \cdot h_n^3}{12} = 0{,}018 м^4; \qquad J_1 = \frac{d \cdot a_5^3}{12} = 0{,}232 м^4;$$

$$J_2 = \frac{d \cdot a_6^3}{12} = 0{,}013 м^4$$

$$J_{x_{брутто}} = \frac{7 \cdot 7{,}5^3}{12} - \frac{6{,}4 \cdot 6{,}9^3}{12} = 70{,}889 м^4;$$

$$J_{y_{брутто}} = \frac{7^{.3} \cdot 7.5}{12} - \frac{6{,}4^3 \cdot 6{,}9}{12} = 63{,}642 м^4;$$

$$p_x = \frac{h \cdot l^3}{3J_n \cdot H^2} \cdot \frac{F_1 \cdot F_2}{F_1 \cdot F_2} \cdot \left(1 - \frac{J_1 + J_2}{J_{x_{брутто}}}\right) = 7{,}2 \cdot 10^{-3};$$

$$p_y = \frac{h \cdot l^3}{3J_n \cdot H^2} \cdot \frac{F_1 \cdot F_2}{F_1 \cdot F_2} \cdot \left(1 - \frac{J_1 + J_2}{J_{y_{брутто}}}\right) = 7{,}197 \cdot 10^{-3}.$$

To determine the coefficient p_{kr}, you first need to determine the necessary geometric attributes (Fig. 1.8, 1.9).

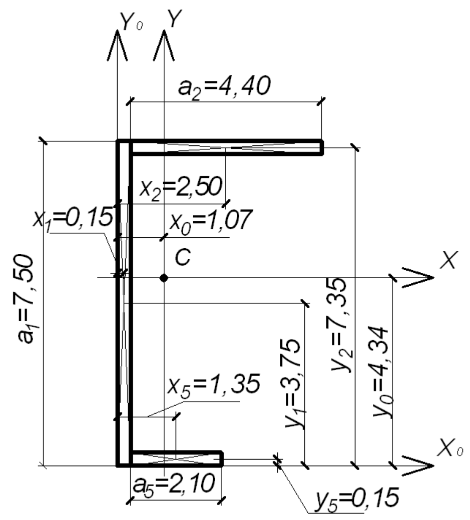


Figure 1.8 – Determination of the center of gravity of the first pylon, which is part of the stiffness core

$$A_1 = d(a_1 + a_2 + a_5) = 4{,}2м^2;$$

$$S_{x_1} = a_1 \cdot d \cdot y_1 + a_5 \cdot d \cdot y_5 + a_2 \cdot d \cdot y_2 = 18{,}234м^3;$$

$$S_{y_1} = a_1 \cdot d \cdot x_1 + a_2 \cdot d \cdot x_2 + a_5 \cdot d \cdot x_5 = 4{,}488м^3;$$

$$Y_{o_1} = \frac{S_{x_1}}{A_1} = 4{,}34м;$$

$$X_{o_1} = \frac{S_{y_1}}{A_1} = 1{,}07м;$$

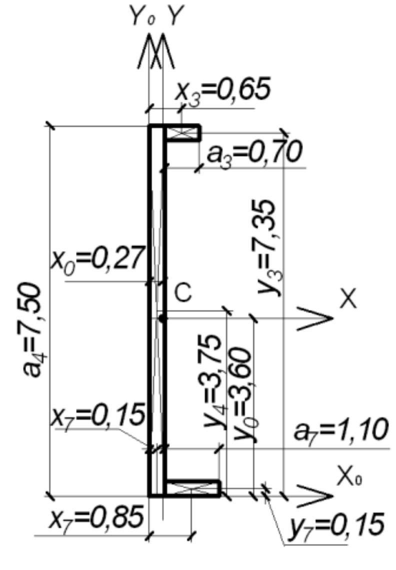


Figure 1.9 – Determination of the center of gravity of the second pylon, which is part of the stiffness core

$A_2=d(a_4+a_7+a_3)$; $x_7=0.85\ m$; $x_4=0.15\ m$; $x_3=0.65\ m$; $A_2=2.79\ m^2$;

$$S_{x_2}=a_4\cdot d\cdot y_4+a_7\cdot d\cdot y_7+a_3\cdot d\cdot y_3=10{,}03м^3;$$

$$S_{y_1}=a_4\cdot d\cdot x_4+a_7\cdot d\cdot x_7+a_3\cdot d\cdot x_3=0{,}755м^3;$$

$$Y_{o_1}=\frac{S_{x_2}}{A_2}=3{,}595м;$$

$$X_{o_1}=\frac{S_{y_2}}{A_2}=0{,}27м;$$

$a=7.2m$; $b=6.7m$; $c=\frac{2{,}2}{h}$; $v=3.93m$; $l_1=1.3m$; $l_2=1.5m$

$l_3=0.9m$; $b_{s1}=5.66m$; $b_{s2}=3.23m$; $b_{s3}=2.43m$

$$p_{kr}=\frac{8\cdot J_{kr}}{(a+b)^2}\cdot\left[\frac{c}{A}+\frac{h\cdot c}{30\cdot J_n\cdot\left(\frac{bs_1^2}{11^3}+\frac{bs_2^2}{12^3}+\frac{bs_3^2}{13^3}\right)}\right]-c=4{,}191\cdot10^{-3}$$

In accordance:

$$Kp_x=\frac{1}{1+px}=0{,}993;\qquad Kp_y=\frac{1}{1+py}=0{,}993;\qquad Kp_{kr}=\frac{1}{1+p_{kr}}=0{,}993;$$

$$J_x=J_x\cdot Kp_x=55{,}897м^4;\ J_y=J_y\cdot Kp_y=60{,}781м^4;$$

$$J_{kr}=J_{kr}\cdot Kp_{kr}=22{,}32м^4;$$

Under the influence of external loads, the building deviates from its original vertical position. With a large mass of the building and insufficient rigidity, the increase in deformations can be significant and cause a loss of the overall stability of the building. The weight of the building at which a loss of overall stability becomes possible is determined by solving the 3rd degree equation (1.12):

$$A_1\cdot G_{кр}^3-A_2\cdot G_{кр}^2+A_3\cdot G_{кр}-A_4=0, \tag{1.12}$$

where

$$A_1=1-\frac{(a_x^2+a_y^2)}{\gamma}; \tag{1.13}$$

$$A_2 = G_x + G_y + G_w - G_x \cdot \frac{a_y^2}{\gamma} - G_y \cdot \frac{a_x^2}{\gamma}; \qquad (1.14)$$

$$A_3 = G_x \cdot G_y + G_x \cdot G_w + G_w \cdot G_y; \qquad (1.15)$$

$$A_4 = G_x \cdot G_y \cdot G_w. \qquad (1.16)$$

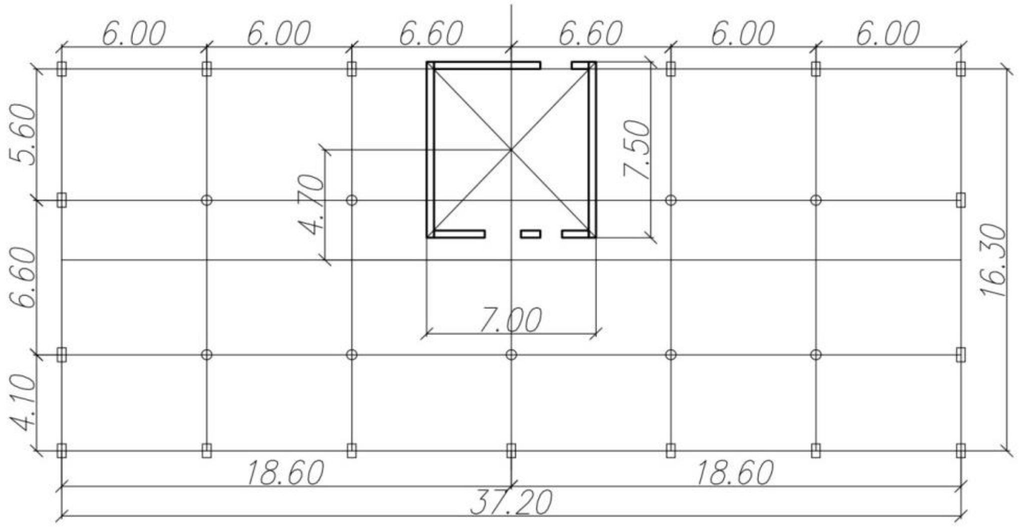


Figure 1.10 – Design (simplified) plan of a building with a stiffening core

$E=2.75\ 10^6 MPa;\quad a_y=4.7m;\quad a=37.2m;\quad b=16.3m$

$$\gamma = a_y^2 + \frac{a^2 + b^2}{12} = 159{,}551\text{м}^2;$$

$$G_x = \frac{2{,}3 \cdot E \cdot J_x}{H^2}; \quad 0{,}9 \cdot G_x = 6437{,}896m; \quad G_x = 72153{,}217m;$$

$$G_y = \frac{2{,}3 \cdot E \cdot J_y}{H^2}; \quad 0{,}9 \cdot G_y = 70611{,}734m; \quad G_y = 78457{,}482m;$$

$$G_w = \frac{0{,}14 \cdot E \cdot J_{kr}}{\gamma}; \quad 0{,}9 \cdot G_w = 48472{,}907m; \quad G_w = 53858{,}786m;$$

$$A_1 = 1 - \frac{a_y^2}{\gamma} = 0{,}862;$$

$$A_2 = 0{,}9 \cdot G_x + 0{,}9 \cdot G_y + 0{,}9 \cdot G_w - 0{,}9 \cdot G_x \frac{a_y^2}{\gamma} = 175031{,}809m;$$

$$A_3 = 0{,}9 \cdot G_x \cdot 0{,}9 \cdot G_y + 0{,}9 \cdot G_x \cdot 0{,}9 \cdot G_w - 0{,}9 \cdot G_y \cdot 0{,}9 \cdot G_w = 1{,}116 \cdot 10^{10} m^2;$$

$$A_4 = 0{,}9 \cdot G_x \cdot 0{,}9 \cdot G_v \cdot 0{,}9 \cdot G_w = 2{,}223 \cdot 10^{14} m^3$$

Substituting all the determined parameters and solving the 3rd degree equation (1.12), we obtain:

$$A_1 \cdot G_{kr}^3 - A_2 \cdot G_{kr}^2 + A_3 \cdot G_{kr} - A_4$$

$$G_{kr} = \begin{pmatrix} 40765 \\ 64938 \\ 97457 \end{pmatrix} = 40765m$$

$$G_n = 37{,}2 \cdot 16{,}3 \cdot 1{,}2 \cdot 21m; \quad G_n = 15280{,}272m;$$

$$\frac{G_{kr}}{G_n} = 2{,}668 \quad \text{which is more than 1.5}$$

Conclusion: The stability of the building is ensured.

2 DYNAMIC STABILITY

As previously indicated, in the case where the height of a building exceeds the smaller size of its plan by seven or more times, the possibility of aeroelastic phenomena of various nature should be assessed.

Let us indicate the most characteristic of them [3].

Divergence. Divergence refers to the static (quasi-static) loss of stability of a structure flowing around a gas flow. The phenomenon of divergence can be compared with the Euler loss of stability of a structure. These include: bending-torsional divergence of structures and divergent instability of bridge structures. When studying the process of divergence, it is obviously possible to neglect the inertia forces of structural elements.

Classic flutter. By classical flutter we mean the established self-oscillations of a structure with two or more degrees of freedom in a potential gas flow.

A characteristic of classical flutter is the exchange of energy between oscillatory movements corresponding to different generalized coordinates.

Disruptive flutter. Shear flutter – self-oscillations of structural elements under the action of shear flow, accompanied by a periodic change in the angle of attack with a change in the nature of the flow shear. These include: bending-torsional vibrations of tall buildings and elements of bridge structures, expressed in their twisting in the wind flow, etc. Thus, shear flutter is associated mainly with torsional deformations.

Aeolian oscillations. They belong to self-oscillations and occur in the presence of alternate breakdown of vortices of the Karman “chess track” type. Characteristic here is the fact that the process of breakdown of vortices to a certain extent depends on the parameters of the oscillatory motion. The energy of the gas flow is transferred to the system as if in “portions” under the action of individual vortices.

This type of oscillation is sometimes called "wind resonance"; it is characteristic of poorly streamlined structures.

Galloping. Galloping refers to the self-oscillations of a structure that arise under the influence of shear flows and deformations and are mainly of a bending nature.

Galloping occurs for power line wires ("dancing" of wires) or sections of pipelines when they freeze; for various elements of light tall buildings and structures that are in the wind flow.

Buffing. Buffing – forced oscillations occurring under the influence of gust currents. Buffing of tall structures occurs if individual parts of it are washed by a wind flow that breaks off from other parts or nearby structures. Obviously, when studying buffeting, it is important to determine the resonance zones, the location of which depends on the parameters of the natural oscillations of the structure.

Parametric oscillations of structural elements in wind flow. Such oscillations occur in the presence of a pulsating aerodynamic load, if it can be taken into account that it changes periodically in time and if it causes forces that determine the stiffness of the system.

Each of the aeroelastic phenomena can cause undesirable deformation of an engineering structure. A wide range of aeroelastic dynamic processes determines the variety of types of damage accumulation in the elements of the system. In some cases, fatigue cracks may develop, leading to premature exhaustion of the structural resource. In other cases, under "turbulent" transient conditions, the system's load-bearing capacity may be exhausted due to brittle collapse in the material or after a very short process of low-cycle fatigue development.

There are known examples where aeroelastic deformations resulted in accidents and catastrophes of large ground structures, as well as bridges.

Assuming that the hypotheses, assumptions, and notations (most of them) adopted for the static stability problem are valid, we will now consider the problem of aerodynamic stability of a building located in a stationary air flow.

In this regard, we use a system of differential equations of oscillations:

$$\begin{aligned} & EI_x \frac{\partial^4 \eta}{\partial z^4} - \frac{\gamma I_x}{g} \cdot \frac{\partial^4 \eta}{\partial z^2 \partial t^2} + \frac{\gamma A}{g} \cdot \frac{\partial^2 \eta}{\partial t^2} + \frac{\partial^2}{\partial z^2}(M_y \theta) + K_\eta \theta = 0, \\ & M_y \frac{\partial^2 \eta}{\partial z^2} + EI_\omega \frac{\partial^4 \theta}{\partial t^4} - GI_d \frac{\partial^2 \theta}{\partial z^2} - \frac{\gamma I_\omega}{g} \cdot \frac{\partial^4 \theta}{\partial z^2 \partial t^2} + \frac{\gamma A r^2}{g} \cdot \frac{\partial^2 \theta}{\partial t^2} + \left[q_x e_x + K_\theta \right] \theta = 0, \end{aligned} \tag{2.1}$$

where $\eta(z, t)$; $\theta(z,t)$ – horizontal displacements and the angle of twist;

$r^2 = \dfrac{\int (x^2 + y^2) dA}{A} + a_x^2 + a_y^2$,- description of the building plan;

a_x; a_y – coordinates of the center of rigidity;

γ – specific gravity of reinforced concrete;

g – acceleration of free fall;

I_x; I_y – moments of inertia;

A – building plan area;

e_x – eccentricity of wind load.

As a calculation model, we will take a box-shaped shell with a rigid profile (Fig. 2.1).

System (2.1) describes the oscillations of this shell.

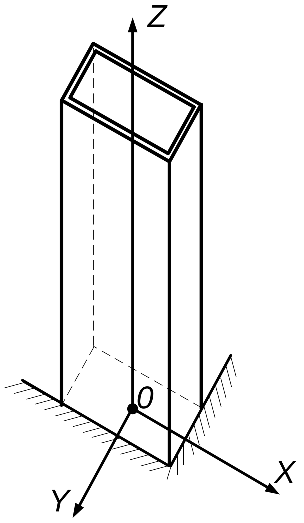


Figure 2.1 – Calculation model of the building (aeroelastic problem)

In this case, the geometric characteristics included in (2.1) are taken in accordance with section 1.

Without reducing the generality of the statement, let us assume that the aerodynamic coefficients

$$\begin{aligned} K_\eta(V) &= K_\eta = const; \\ K_\theta(V) &= K_\theta = const; \end{aligned} \qquad (2.2)$$

where V – the velocity of the incoming flow, and

K_η and K_θ depend on the shape and geometric dimensions of the building in plan and are average values [3.9].

In turn, the headwind pressure depends on the flow speed.

$$q_x = -KV^2, \qquad (2.3)$$

and the aerodynamic coefficient "K" also depends on the shape and geometric dimensions of the streamlined object (building) [4].

Assuming, in a first approximation, the vibrations of the building to be harmonic, we accept

$$\begin{aligned} \eta(z,t) &= \eta(z)\sin\omega t; \\ \theta(z,t) &= \theta(z)\sin\omega t, \end{aligned} \qquad (2.4)$$

where ω is the circular oscillation frequency.

Substituting (2.4) into (2.1), after reducing by a common factor *sinωt*, we obtain a system of ordinary differential equations:

$$\begin{aligned} &EI_x\eta^{IV} + \frac{\gamma I_x}{g}\omega^2\eta^{II} - \frac{\gamma A}{g}\omega^2\eta + (M_y\theta)^{II} + K_\eta\theta = 0, \\ &M_y\eta^{II} + EI_\omega\theta^{IV} - \left(GI_d - \frac{\gamma I_\omega}{g}\omega^2\right)\theta^{II} - \left[\frac{\gamma Ar^2}{g}\omega^2 - q_x e_x - K_\theta\right]\theta = 0. \end{aligned} \tag{2.5}$$

Let us further assume that

$$\begin{aligned} &\eta(z) = B\psi(z); \\ &\theta(z) = C\chi(z), \end{aligned} \tag{2.6}$$

where *B* and *C* are constants, and the functions included in (2.6) are taken as the deflection function of a cantilever beam loaded with a uniformly distributed load:

$$\psi(z) = \chi(z) = z^4 - 4Hz^3 + 6A^2z^2, \tag{2.7}$$

here *H* is the height of the building above the ground.

The wind load distribution is assumed to be uniform conditionally. However, introducing into consideration a real, usually trapezoidal distribution, does not fundamentally affect the result of this illustrative solution, but will only lead to an increase in algebraic transformations.

The solution written in the form (2.7) satisfies the boundary conditions

$$\text{in a pinch} \quad \begin{aligned} &\eta(0) = \eta^{I}(0) = 0; \\ &\theta(0) = \theta^{I}(0) = 0; \end{aligned}$$

$$\text{at the free end} \quad \left.\begin{aligned} &\eta^{II}(H) = \eta^{III}(H) = 0; \\ &\theta^{II}(H) = \eta^{III}(H) = 0; \end{aligned}\right\} \quad \text{Kirchhoff's conditions.}$$

In addition, expression (2.7) determines the magnitude of the moment included in (2.5):

$$M_y = \frac{KV^2(H-z)^2}{2} \tag{2.8}$$

The Lagrange equations for the forces determined by the left-hand sides of (2.5) have the following form:

$$\begin{aligned} &B\left[EI_x\int_0^H(\psi^{II})^2dz-\frac{\gamma I_x}{g}\omega^2\int_0^H(\psi^I)^2dz-\frac{\gamma A}{g}\omega^2\int_0^H\psi^2dz\right]+\\ &+C\left[\int_0^H M_y\chi\psi^{II}dz+\int_0^H K_\eta\chi^2dz\right]=0, \end{aligned} \tag{2.9}$$

$$\begin{aligned} &B\int_0^H M_y\chi\psi^{II}dz+C\left\{EI_\omega\int_0^H(\chi^{II})^2dz+(GI_d-\frac{\gamma I_\omega}{g}\omega^2)\int_0^H(\chi^I)^2dz-\right.\\ &\left.-\left[\frac{\gamma Ar^2}{g}\omega^2-q_xe_x-K_\theta\right]\int_0^H\chi^2dz\right\}=0. \end{aligned}$$

Equating the determinant of system (2.9) composed of coefficients with unknowns *B* and *C* to zero, we obtain:

$$\begin{vmatrix} C_{11}C_{12} \\ C_{21}C_{22} \end{vmatrix}=0, \tag{2.10}$$

or $\qquad C_{11}C_{22}-C_{12}C_{21}=0,$

where
$$\begin{aligned} &C_{11}=a_1+a_2+a_3; && C_{12}=a_4V^2+a_5;\\ &C_{21}=a_6V^2; && C_{22}=a_7+a_8+a_9+a_{10}+a_{11}V^2+a_{12}; \end{aligned} \tag{2.11}$$

Note that the inequality $C_{12}\neq C_{21}$ serves as a sign of the non-conservative nature of the system.

In expressions (2.11):

$$\begin{aligned} &a_1=EI_x\int_0^H(\psi^{II})^2dz=\frac{144}{5}EI_xH^5;\\ &a_2=-\frac{\gamma I_x}{g}\omega^2\int_0^H(\psi^I)^2dz=-\frac{72}{7}\cdot\frac{\gamma I_x\omega^2H^7}{g};\\ &a_3=-\frac{\gamma A}{g}\omega^2\int_0^H\psi^2dz=-\frac{104}{45}\cdot\frac{\gamma A\omega^2H^9}{g};\\ &a_4=q_x\int_0^H M_y\chi\psi^{II}dz=-\frac{4}{15}KH^9;\\ &a_5=K_\eta\int_0^H\chi^2dz=\frac{104}{45}K_\eta H^9;\\ &a_6=a_4=-\frac{4}{15}KH^9; \end{aligned} \tag{2.12}$$

$$a_7 = EI_\omega \int_0^H (\chi^{11})^2 dz = \frac{144}{5} EI_\omega H^5;$$

$$a_8 = GI_d \int_0^H (\chi^1)^2 dz = \frac{72}{7} GI_d \cdot H^7;$$

$$a_9 = -\frac{\gamma I_\omega}{g} \omega^2 \int_0^H (\chi^1)^2 dz = \frac{72}{7} \cdot \frac{\gamma \cdot I_\omega}{g} \omega^2 H^7;$$

$$a_{10} = -\frac{\gamma A r^2}{g} \omega^2 \int_0^H \chi^2 dz = -\frac{104}{45} \cdot \frac{\gamma A r^2 \omega^2 H^9}{g};$$

$$a_{11} = q_x e_x \int_0^H \chi^2 dz = \frac{104}{45} K \cdot e_x \cdot H^9;$$

$$a_{12} = K_\theta \int_0^H \chi^2 dz = \frac{104}{45} \cdot K_\theta H^9.$$

In (2.12) it is taken into account that

$$\int_0^H (\psi^{II})^2 dz = \int_0^H (12z^2 - 24Hz + 12H^2)^2 dz = 144 \int_0^H (z - H)^4 dz = \frac{144}{5} H^5;$$

$$\int_0^H (\psi^{'})^2 dz = \int_0^H 16(z^3 - 3Hz^2 + 3H^2 z)^2 dz = 16 \int_0^H (z^3 - H^3)^2 dz = \frac{72}{7} H^7;$$

$$\int_0^H \psi^2(z) dz = \frac{104}{45} H^9;$$

$$\int_0^H M_y \chi \psi^{II} dz = \frac{4}{15} H^9;$$

Assuming that $a_4 \neq 0$, we introduce the parameters:

$$\beta = \frac{(a_1 + a_2 + a_3) a_{11} + a_5 a_6}{a_4^2};$$

$$\gamma = \frac{(a_1 + a_2 + a_3)(a_7 + a_8 + a_9 + a_{10} + a_{12})}{a_4^2}.; \qquad (2.13)$$

As a result, we get: $$V^4 - \beta V^2 - \gamma = 0; \qquad (2.14)$$

Let us note further: $$V^2 = \varphi; \qquad (2.15)$$

then $$\varphi^2 - \beta\varphi - \gamma = 0;$$

$$\varphi_{1,2} = \frac{\beta}{2} \pm \sqrt{\frac{\beta^2}{4} + \gamma}, \qquad (2.16)$$

Adding to (2.16) the condition of the extremum of the square of the velocity (Fermat's theorem):

$$\frac{\partial^2 \varphi}{\partial \omega^2} = 0, \tag{2.17}$$

it is possible to find the corresponding circular oscillation frequency, and then the critical wind flow speeds

$$V_{1,2,3,4} = \pm\sqrt{\frac{\beta}{2} \pm \sqrt{\frac{\beta^2}{4} + \gamma}}\,. \tag{2.18}$$

Analysis of the roots (2.18) makes it possible to establish the type of aeroelastic phenomena. Here we mean the forms of the roots (real, complex) and their signs.

2.1 Example of assessing the dynamic stability of a building

Consider the 48-story building shown in Figures 2.2 and 2.3. Geometric relationship $\frac{H}{b} = \frac{192,0}{24} = 8,0 > 7,0$ In this regard, it is necessary to assess the possibility of aeroelastic phenomena.

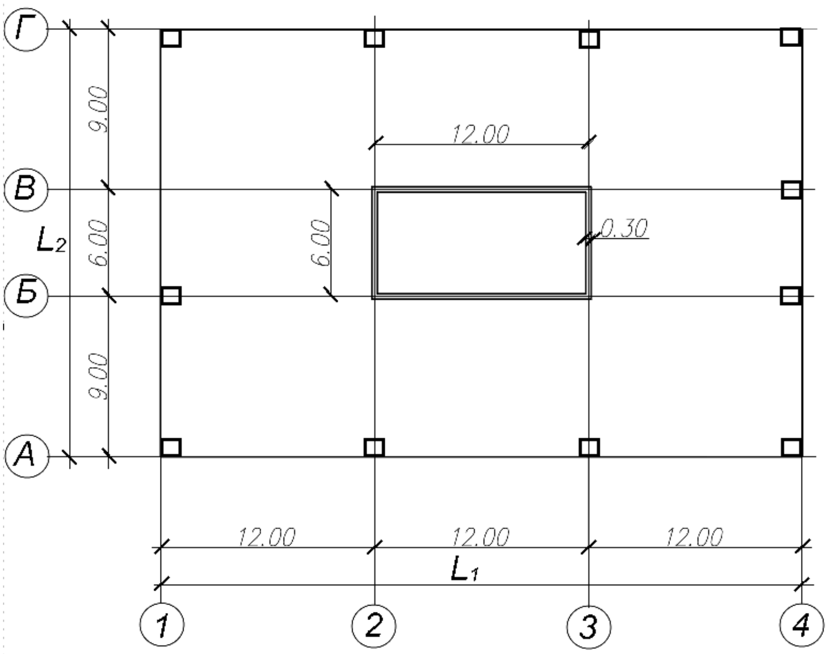


Figure 2.2 – Building plan

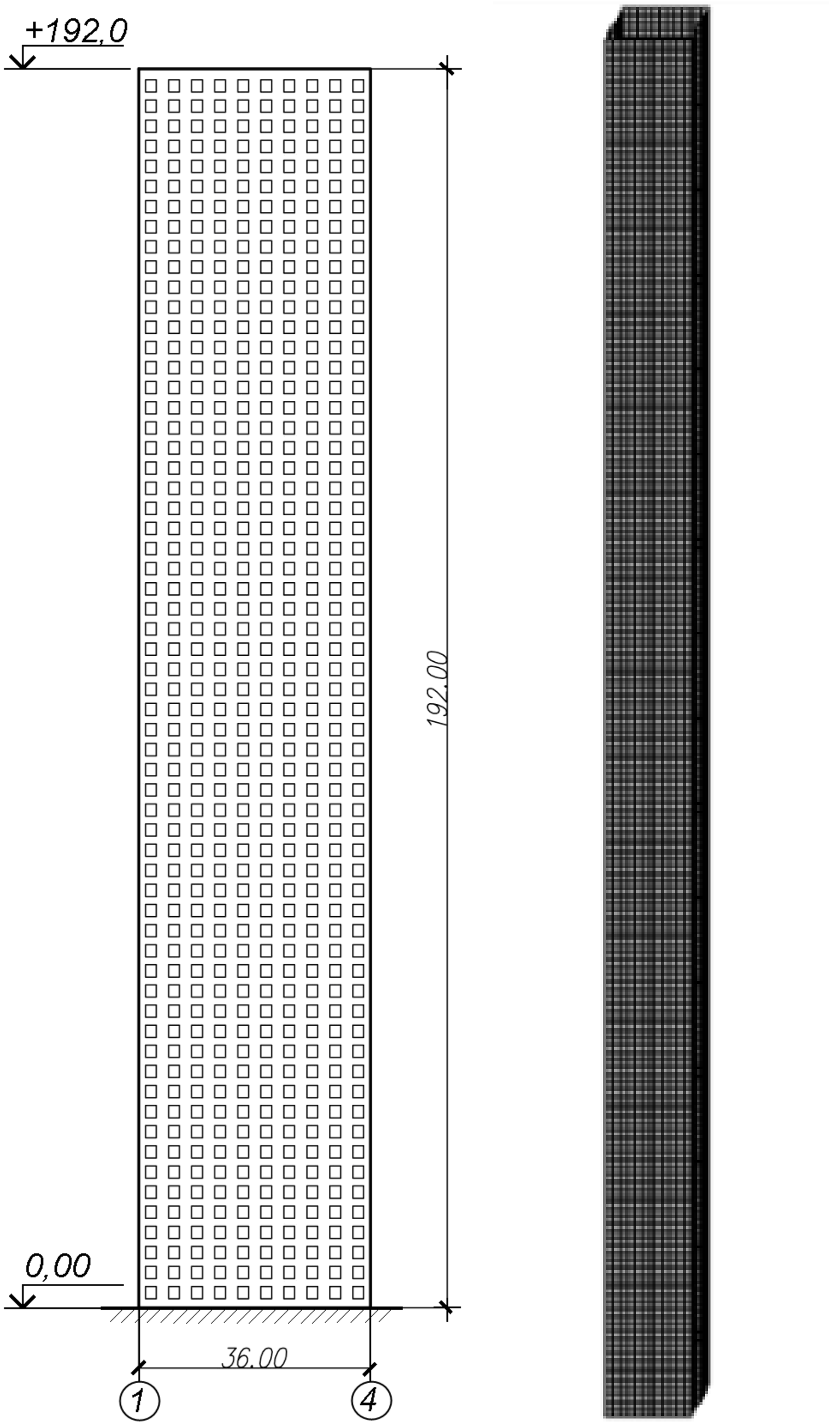


Figure 2.3 – Building facade and finite element model of the building

We will determine in advance the geometric characteristics of the core of the building section, namely the moments of inertia (relative to the "X", "Y" axes, torsional and sectorial) of the core of the building's rigidity, the area and the coefficient characterizing the building's plan. The building material is reinforced concrete, consisting of concrete of class C25/30.

Let's define the geometric characteristics necessary for the calculation:

$$I_x = \frac{d_1 \cdot d_2^3}{12} - \frac{(d_1 - 2t)\cdot(d_2 - 2t)^3}{12};$$

$$I_y = \frac{d_2 \cdot d_1^3}{12} - \frac{(d_2 - 2t)\cdot(d_1 - 2t)^3}{12};$$

$$A = d_1 \cdot d_2 - (d_1 - 2t)\cdot(d_2 - 2t);$$

$$I_d = \frac{A^2 \cdot t}{2\cdot(d_1 + d_2)};$$

$$I_\omega = A \cdot d_1^2 \cdot d_2^2;$$

$$r = \sqrt{\frac{A^2 + B^2}{12}};$$

$$G = \frac{E}{2\cdot(1+\mu)}.$$

The results of determining the geometric characteristics are summarized in Table 2.1.

Table 2.1 – Output data

Characteristic	Value	Characteristic	Value	Characteristic	Value
d_1, m	12	I_x, m^4	66.41	A, m^2	10.44
d_2, m	6	I_y, m^4	197.31	e_x, m	0
t, m	0.3	I_d, m^4	0.91	k	0.145
L_1, m	36	I_ω, m^6	1127.51	k_η	1
L_2, m	24	r^2, m^2	156	k_θ	1
H,m	192	γ, kN/m^3	25	g, m/s^2	9.81
E, kN/m^2	$2.9\cdot10^7$	μ	0.17	G,kN/m^2	$1.24\cdot10^7$

Where d_1, d_2, are the dimensions of the building's stiffness core in plan, respectively;

t – the thickness of the walls of the building's rigid core;

L_1, L_2 – dimensions of the building in plan;

I_x, I_y, I_d, I_ω – moments of inertia axial, torsional and sectorial, respectively;

N – height of the building above the ground;

A – area of the building's stiffness core;

E, G – modulus of elasticity and shear modulus of reinforced concrete, respectively;

r^2 – description of the building plan;

γ – specific gravity of reinforced concrete;

g – acceleration of free fall;

μ – Poisson's ratio;

Ex – eccentricity;

k, k_η , k_θ – aerodynamic coefficients.

According to formulas (2.12), we determine the values of $a_1 – a_{12}$ (Table 2.2).

Table 2.2 – Values of coefficients a1 – a12

Parameter	Value	Parameter	Value	Parameter	Value
a_1, kN m^7	$1.477\cdot10^{22}$	a_5, kN m^8	$8.195\cdot10^{20}$	a_9, kN m^9	$-2.846\cdot10^{20}\cdot\omega^2$
a_2, kN m^7	$-1.676\cdot10^{19}\cdot\omega^2$	a_6, kN m^6 s^2	$-1.371\cdot10^{19}$	a_{10}, kN m^9	$-3.405\cdot10^{24}\cdot\omega^2$
a_3, kN m^7	$-2.182\cdot10^{22}\cdot\omega^2$	a_7, kN m^9	$2.457\cdot10^{23}$	a_{11}, kN m^7 s^2	0
a_4, kN m6 s^2	$-1.371\cdot10^{19}$	a_8, kN m^9	$1.114\cdot10^{24}$	a_{12}, kN m^9	$8.195\cdot10^{20}$

Equation (2.14) will take the form:

$$V^4 + \beta\cdot V^2 + \gamma = 0; \tag{2.19}$$

where $$\beta = \frac{(a_1 + a_2 + a_3)\cdot a_{11} - a_5\cdot a_6}{a_4^2} = 59{,}77\left[\frac{\text{м}^2}{\text{с}^2}\right];$$

$$\gamma = \frac{(a_1 + a_2 + a_3)\cdot(a_7 + a_8 + a_9 + a_{10} + a_{12})}{a_4^2} =$$
$$= -3{,}956\cdot10^8\cdot\omega^4 + 4{,}202\cdot10^8\cdot\omega^2 - 1{,}047\cdot10^8\left[\frac{\text{м}^4}{\text{с}^4}\right]; \tag{2.20}$$

Let's make a substitution:

$$V^2 = \varphi. \tag{2.21}$$

We get:

$$\varphi^2 + \beta \cdot \varphi + \gamma = 0. \qquad (2.22)$$

From here:

$$\varphi_{1,2} = -\frac{\beta}{2} \pm \sqrt{\frac{\beta^2}{4} - \gamma},$$

or

$$\varphi_{1,2} = \pm 0{,}5 \cdot \sqrt{1{,}583 \cdot 10^9 \cdot \omega^4 - 1{,}681 \cdot 10^9 \cdot \omega^2 + 4{,}189 \cdot 10^8} - 29{,}885. \qquad (2.23)$$

Let's make another substitution:

$$\omega^2 = z. \qquad (2.24)$$

Using Fermat's theorem, we get:

$$\frac{d\varphi}{dz} = 0, \qquad (2.25)$$

$$\frac{d\varphi}{dz} = \frac{3{,}165 \cdot 10^9 \cdot z - 1{,}621 \cdot 10^9}{4 \cdot \sqrt{1{,}583 \cdot 10^9 \cdot z^2 - 1{,}681 \cdot 10^9 \cdot z + 4{,}189 \cdot 10^8}} = 0$$

Assuming that the denominator cannot be zero, consider the linear equation with respect to z:

$$3{,}165 \cdot 10^9 \cdot z - 1{,}621 \cdot 10^9 = 0, \qquad (2.26)$$

From here

$$z = \frac{1{,}621 \cdot 10^9}{3{,}165 \cdot 10^9} = 0{,}531.$$

In accordance:

$$\omega = \sqrt{z} = \sqrt{0{,}531} = 0{,}729 \left[\frac{\text{рад}}{\text{с}}\right]. \qquad (2.27)$$

Thus, the critical flow velocity will be determined by

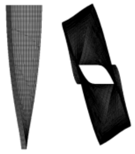
$$V_{1,2,3,4} = \pm\sqrt{\varphi_{1,2}},$$

$$V_{1,2,3,4} = \pm\sqrt{\pm 0{,}5 \cdot \sqrt{1{,}583 \cdot 10^9 \cdot \omega^4 - 1{,}681 \cdot 10^9 \cdot \omega^2 + 4{,}189 \cdot 10^8} - 29{,}885}. \qquad (2.28)$$

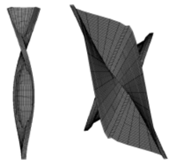
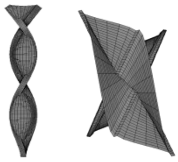
Analysis of (2.28) shows that for a given circular frequency, the values of the critical flow velocities are expressed as complex numbers.

However, this conclusion cannot be considered final, because the adopted flow fluctuations in the form of a single harmonic oscillation do not reflect the whole variety of situations. To draw a more complete picture, we will determine the natural frequencies of the building. The solution is built numerically in the PC "Lira" environment based on the finite element model shown in Figure 2.3[5].

Table 2.4 - Forms of natural oscillations

Form No.	Oscillation shape	Type of oscillation	Frequencies		V_{kr}, m/s
			Round frequency (s^{-1})	Frequency (Hz)	
1	2	3	4	5	6
1		Bending vibrations along the Y axis	0.840	0.134	47.0
2		Bending vibrations along the X axis	1,417	0.225	171.0
3		Bending vibrations by overtones along the Y axis	5,097	0.811	711.0
4		Bending vibrations by overtones along the X axis	8,577	1,365	1205.0
5		Torsional vibrations	12,758	2,031	1796.0

Continuation of table 2.4

1	2	3	4	5	6
6		Bending vibrations by overtones along the Y axis	13,302	2,117	1873.0
7		Bending vibrations along overtones	19,714	3,138	2778.0
8		Bending vibrations along overtones	22,221	3,537	3132.0
9		Bending vibrations by overtones along the Y axis	22,603	3,597	3186.0
10		Bending vibrations by overtones along the X axis	22,772	3,624	3209.0

At the same time, for modeling the walls of the stiffening core, the elements "FE-41 - universal rectangular element of the shell of zero Gaussian curvature" were used. The dimensions of the finite elements were established based on the solution of test problems and compiled for each linear meter of the element.

The dead weight of the structure was calculated automatically, taking into account the standardized volumetric weight of reinforced concrete of 25kN/m^3.

The calculation was performed for dynamic action ("modal analysis") for the first 10 forms of natural oscillations (Table 2.4).

By substituting the circular frequency of each of the vibration forms into formula (2.28), we obtain the value of the critical speed (Fig. 2.4).

Analysis shows that the solution (2.28) corresponds to two complex polar conjugate roots and two real roots. Moreover, there is only one positive real root.

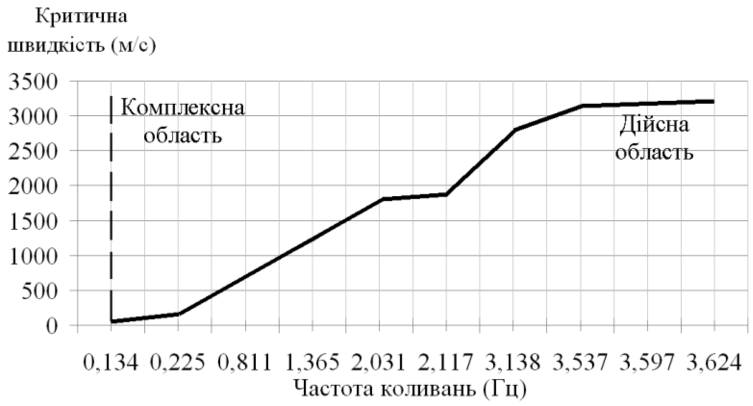


Figure 2.4 – Dependence of critical speed on oscillation frequency

Thus, it is obvious that dangerous (resonant) modes of building vibrations occur, corresponding to certain critical wind flow speeds (Fig. 2.4). As can be seen from Table 2.4, vibrations in these cases can be of both bending and torsional nature.

3 SPECIAL CASES

When designing and calculating buildings in which a certain number of pylons do not reach the foundations, the following tasks should be solved separately [6]:

– assessment of sufficient load-bearing capacity of the remaining elements (usually columns) reaching the foundations;

– assessment of the shear force of the corresponding resource of the column cross-section in the place of collapse of the pylons;

– assessment of the strength of the overlap disk at the point of pylon breakage;

– assessment of the bearing capacity (in terms of transverse force) of other additionally loaded pylons reaching the foundations.

When designing and calculating buildings in which some of the pylons do not reach the top, the following should be considered:

– assess the strength of the overlap disk located at the point of the pylon break;

– assess the bearing capacity of the remaining pylons and columns for shear at the location of the pylon break.

The inaccuracy of erecting vertical structures should be taken into account:

– when checking the strength of columns, by introducing a random eccentricity [7];

– when the strength of the floor disks is checked, taking into account the calculated fracture angle of vertical structures $\varepsilon = 0.0075$.

For the numerous implementation of the listed tasks, appropriate local calculation models should be formed. At the same time, final conclusions about the reliability of acceptable structural solutions should be made on the basis of a joint consideration of the results of calculations obtained according to the general scheme of the model "the building itself - foundation structure - base" and the local models mentioned.

The bearing capacity of the vertical section of the pylon at shear T is taken equal to the sum of the shear resistances of all connecting elements included in the section under consideration. If the vertical section consists of several sections of height hj, differing in shear resistance per unit height tj, then

$$T = \sum t_j h_j \ . \qquad (3.1)$$

The bearing capacity tj of the vertical section at the considered displacement is taken as:

for solid sections of walls - equal to their bearing capacity for transverse force without taking into account reinforcement, equal to the multiplication of the calculated resistance of the concrete of the pylons in tension fctk by their thickness;

for lintels over openings – equal to the lesser of their load-bearing capacities under shear force or bending moment divided by the floor height.

Strength testing of normal cross-sections of stiffening cores is performed as for open-profile pylons.

The shear strength test of the stiffness cores differs from the shear strength test of open-profile pylons: any surface with a vertical generatrix intersects a closed-profile pylon simultaneously in two vertical sections (Fig. 3.1, a). In addition, the torques, which can be quite large in closed-profile pylons, cause shear forces Tcr (Fig. 3.1, b). These forces consist of shear forces from vertical loads and forces from transverse bending of the pylons.

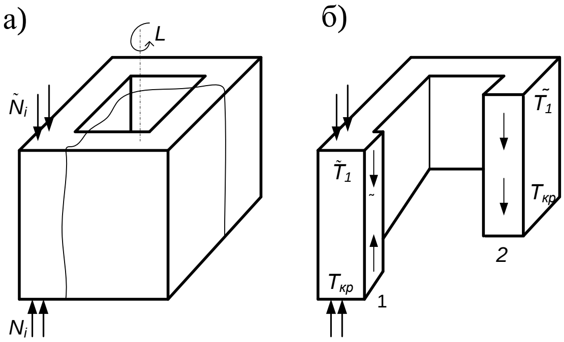


Figure 3.1 – Before checking the shear strength of the stiffener cores

The lintels above the openings in the pylons should be designed in such a way that they are of equal strength in terms of transverse force and bending moment. In this case, the bending moment in the lintel is determined by:

- if the width of the smaller branch of the pylon adjacent to the opening is greater than or equal to the height of the lintel above the opening

$$M_n = 0{,}5V_n b_n; \tag{3.2}$$

- if the width of the smaller branch is less than the height of the lintel above the opening

$$\grave{I}_{\ddot{\imath}} = 0{,}5V_{\ddot{\imath}} b_{\ddot{\imath}} (1 - 0{,}5\frac{b_c}{h_{\ddot{\imath}}}). \tag{3.3}$$

Here all the notations are according to Figure 3.2.

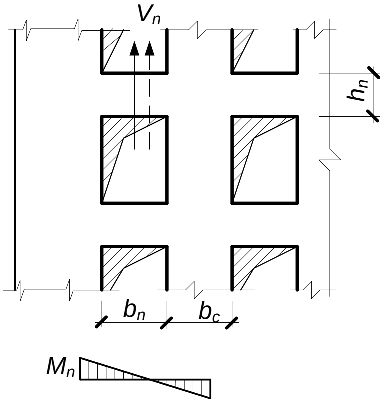


Figure 3.2 – Before calculating the lintel above the opening

The transverse force in the lintels V_n is taken on the basis of static calculation. The force from the local load should be added to the forces V_n and M_n.

For an approximate assessment of the level of weakening of the pylon in its plane by openings and to simplify the calculation of the building, it is permissible to use the reduction coefficient of O. R. Rzhanitsyn [6]:

$$\beta = \sqrt{\frac{3J_{\ddot{\imath}} H^2}{2a^3 h}\left(\frac{F_1 + F_2}{F_1 F_2} + \frac{v^2}{J_1 + J_2}\right)} \tag{3.4}$$

where J_n – moment of inertia of the jumper;

F_1, F_2 – cross-sectional areas of the pylon branches;

J_1 and J_2 – moments of inertia of the pylon branches;

H – diaphragm height;

h – floor height;

a – half the width of the opening;

v – distance between the centers of gravity of the pylon branches.

At $\beta \geq 15$, the compliance of the jumpers is small and the diaphragm works with a single cross-section and is calculated according to the beam-cantilever scheme. At $\beta \leq 0.5$, the compliance of the jumpers is large and their influence on the operation of the pylons is insignificant; the branches of the pylons (diaphragms) work separately and are calculated as separate independent beam consoles. At intermediate values $15 > \beta > 0.5$, the nature of the operation of the diaphragm with openings resembles the operation of the frame. In this case, the forces in the diaphragm elements can be determined as in frame systems.

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