Magazine of Civil Engineering

ISSN 2712-8172

journal homepage: http://engstroy.spbstu.ru/

Research article

UDC 699.841

DOI: 10.34910/MCE.129.4

Tuned mass damper for reduction seismic and wind loads

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Keywords: tuned mass damper, wind load, seismic load, vibration control, optimization

Abstract. Tuned mass dampers (TMDs) are used mainly for reduction of seismic and wind oscillations in high-rise buildings. It is well known that base isolation is ineffective in tall buildings. In general, TMDs can reduce seismic loads in tall building, but it needs a large mass of TMDs. In addition, TMDs cannot reduce vertical oscillations, which can be very destructive due to P-delta effect. This paper presents an engineering solution for mitigation of structural response caused by seismic excitations. The approach of using the upper part of the building as a TMD can significantly reduce horizontal accelerations and stresses in building elements up to 50 % along the entire height. In addition, the proposed TMD can significantly reduce vertical oscillations in a primary building up to 30 % in comparison with building without TMD. This solution can be used in both existing and new buildings. This solution does not require any additional mass and its transportation to the installation site. An optimization criterion for defining optimal TMD's properties was developed. The criterion is the objective function of maximum difference in accelerations of floors with and without TMD along the entire height. For analytical studies, matrix of stiffness that takes into account bending and sliding motions and dissipation matrix that takes into account damping ratio for soil, TMD constructions and constructions of the building were developed.

Citation: Bondarev, D.E. Tuned mass damper for reduction seismic and wind loads. Magazine of Civil Engineering. 2024. 17(5). Article no. 12904. DOI: 10.34910/MCE.129.4

1. Introduction

Tuned mass damper (TMD) is a device for reduction of seismic and wind responses of buildings and structures. The first mention of the use of this technology, as far as we know, appeared in 1909 when H. Frahm received a patent for "Device for damping vibrations of bodies" [1]. This device is used in structures to prevent discomfort, damage, or structural failure caused by dynamic excitations. Vibration control of structures can be divided into four groups: active, semi-active, hybrid, and passive systems [2]. It is possible to use several types of vibration control systems in one building [3]. TMD is a passive control device that does not require any external sources of energy. There are many types of TMDs: friction TMD [4], conventional TMD [5], pendulum TMD [6], bidirectional [7], tuned liquid column damper (TLCD) [8], etc. TMDs are widely used in tall structures [9], chimneys [10], long span transmission tower-line systems [11], high-rise buildings [12, 13], flexible bridges [14], etc. Usually, TMDs are installed at the upper floors of high-rise building, under bridge's spans and/or at bridge pylons, etc.

In general, TMD consists of a mass, a spring, and a damper. A two-degree-of-freedom (-DOF) system is shown in Fig. 1 and 2 for seismic and wind excitations respectively.

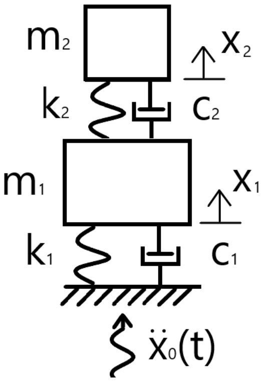


Figure 1. A two-DOF damped system subjected to seismic excitation.

Applying Newton's second law to the main mass m_1 gives:

$$m_1\ddot{x}_1 + c_1\dot{x}_1 + k_1x_1 + c_2\left(\dot{x}_1 - \dot{x}_2\right) + k_2\left(x_1 - x_2\right) = -m_1\ddot{x}_0. \quad (1)$$

Applying Newton's second law to the mass of a TMD m_2 gives:

$$m_2\ddot{x}_2 + c_2\left(\dot{x}_2 - \dot{x}_1\right) + k_2\left(x_2 - x_1\right) = -m_2\ddot{x}_0. \quad (2)$$

Transforming these equations, we receive a system of differential equations of the second order:

$$\begin{cases} \ddot{x}_1 = -\ddot{x}_0 - \dfrac{k_1}{m_1}x_1 - \gamma^2\mu\omega_1^2\left(x_1 - x_2\right) - 2\xi_2\omega_1\gamma\mu\left(\dot{x}_1 - \dot{x}_2\right) - 2\xi_1\omega_1\dot{x}_1 \\ \ddot{x}_2 = -\ddot{x}_0 - \omega_2^2\left(x_2 - x_1\right) - 2\xi_2\omega_1\gamma\left(\dot{x}_2 - \dot{x}_1\right) \end{cases}, \quad (3)$$

where $m_1,\ k_1,\ c_1$ are the mass, stiffness, and damping of a primary structure; $m_2,\ k_2,\ c_2$ are the mass, stiffness, and damping of a TMD's construction, $\ddot{x}_0$ is time history acceleration of seismic excitation. x_1, $x_2,\ \dot{x}_1,\ \dot{x}_2,\ \ddot{x}_1,\ \ddot{x}_2$ are displacements, velocities, and accelerations of structure and the TMD; μ represents the ratio of the TMD mass $\left(m_2\right)$ to structural mass $\left(m_1\right)$. $\xi_1,\ \xi_2$ are the damping ratios of the structure and the TMD; γ is the ratio of the frequency of the TMD $\left(\omega_2\right)$ to the frequency of the structure $\left(\omega_1\right)$.

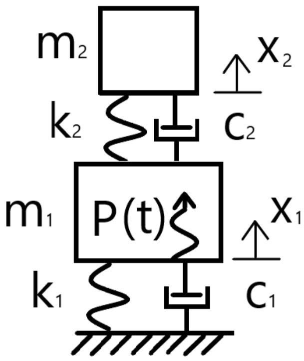


Figure 2. A two-DOF damped system subjected to wind excitation.

For the system subjected to wind excitation where $P\left(t\right)$ is a time-dependent external force:

$$\begin{cases} \ddot{x}_1 = \dfrac{P\left(t\right)}{m_1} - \dfrac{k_1}{m_1}x_1 - \gamma^2\mu\omega_1^2\left(x_1 - x_2\right) - 2\xi_2\omega_1\gamma\mu\left(\dot{x}_1 - \dot{x}_2\right) - 2\xi_1\omega_1\dot{x}_1 \\ \ddot{x}_2 = -\omega_2^2\left(x_2 - x_1\right) - 2\xi_2\omega_1\gamma\left(\dot{x}_2 - \dot{x}_1\right) \end{cases}. \quad (4)$$

These equations of motion undergoing seismic and wind excitations have an analytical solution if an external force (seismic or wind load) is roughly expressed by harmonic loading. For wind excitation, it is a sinusoidal time-dependent pressure of wind and, in turn, for seismic excitation – a sinusoidal displacement of soil motion [15].

In addition, these equations can be solved by numerical integration method. For instance, it can be solved by Runge–Kutta method where right parts in (3) and (4) are vectors of the first and the second derivatives in an explicit form and, in addition, it is necessary to use a vector of initial conditions, such as velocity and displacement in the beginning of the motion. Using numerical solution of differential equations, it is possible to take into account all frequencies of the external force expressed with accelerograms and wind pressure time histories.

It is well known that there are simple equations to define the optimum damping ratio and frequency (ξ_2, Hz) of a TMD. For minimum structural displacement amplitude, the formulae were given by Den Hartog [16]:

$$f_2 = \frac{f_1}{1+\mu}; \tag{5}$$

$$\xi_2 = \sqrt{\frac{3\mu}{8(1+\mu)^3}}. \tag{6}$$

If the acceleration amplitude of the structure is to be minimized:

$$f_2 = \frac{f_1}{\sqrt{1+\mu}}; \tag{7}$$

$$\xi_2 = \sqrt{\frac{3\mu}{4(2+\mu)(1+\mu)}}. \tag{8}$$

There are many other criteria of optimization: maximum dynamic stiffness of the main structure [17], maximum effective damping of combined structure [18], minimum travel of damper mass relative to the main structure [18], minimum force in the main structure [19], minimum velocity of the main structure [19] etc. [15, 20, 21].

The equations (3) and (4) are very simply used to demonstrate the basic principles of TMD operation, but they cannot be used in actual engineering practice because real civil engineering structures cannot be considered as single-DOF systems. External excitation does not have the only frequency and structures may undergo nonlinear deformations. In addition, it is necessary to consider the stiffness of soil in equations of motions, if a structure is located at a soft soil.

In general, TMD shall be tuned close to a dominant response frequency of the structure. TMD usually requires an essential mass related to a mass of a primary structure and a large space for its installation at high elevations. Usually ratio μ of a mass of TMD's construction and a mass of a primary construction of building ranges between 0.02 and 0.08 [22] for buildings subjected to seismic excitations and 0.0005 and 0.02 [23] for buildings subjected to wind excitations to achieve demanded TMD efficiency.

TMDs are rather complex and expensive devices that are limited in mass and damping with levels far away from optimal parameters and usually are tuned to only one dominant frequency of the structure providing protection from wind loads only and being ineffective in case of seismic excitation. Inefficiency of the TMD in case of seismic excitation makes researchers find other technological and engineering solutions. It is necessary to search other criterion of optimization for increasing efficiency of the TMD system.

TMD approach described in this paper allows significantly improve TMD efficiency against seismic excitation and create a three-dimensional TMD system with optimal mass, stiffness and damping parameters for structures' protection, while significantly reducing the cost of TMD itself.

2. Methods

2.1. The Motion Equation of a Multi-Degree-Of-Freedom System of Shear-Wall Building with the TMD

The motion equation of a multi-DOF system for the high-rise building subjected to a seismic excitation can be written as follows:

$$[M]\ddot{u}+[C]\dot{u}+[K]u=-[M]\left(\{I_x\}\ddot{x}_0+\{I_y\}\ddot{y}_0+\{I_z\}\ddot{z}_0\right), \quad (9)$$

where $[M]$, $[C]$ and $[K]$ represent the mass, damping, and stiffness matrices, respectively. u, $\dot{u}$, $\ddot{u}$ are the relative displacement, velocity, acceleration vectors with respect to the base. $\{I_x\}$, $\{I_y\}$, $\{I_z\}$ are the vectors, which consist of cosines between vector of displacements and vector of excitation. $\ddot{x}_0$, $\ddot{y}_0$, $\ddot{z}_0$ are time history acceleration of a seismic excitation in X, Y, Z directions.

Considering only shear stiffness of the floors and X-direction of seismic excitation, we may use the following equation:

$$[M]\ddot{x}+[C]\dot{x}+[K]x=-[M]\left(\{I_x\}\ddot{x}_0\right). \quad (10)$$

Multi-DOF system with TMD is shown in Fig. 3:

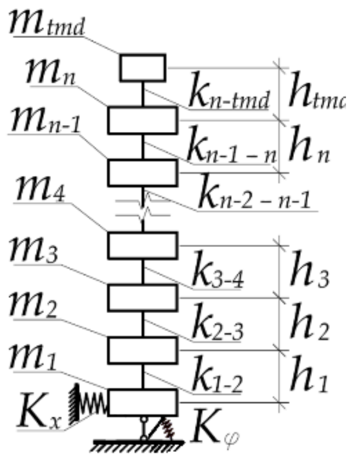


Figure 3. Multi-DOF system of shear-wall building subjected to a seismic excitation in X-direction with TMD located on the top floor.

Stiffness matrix can be written as follows [24]:

$$[K]=$$
$$=\begin{bmatrix} K_x+K_{1-2} & -K_{1-2} & 0 & \dots & 0 & K_{1-2}h_1 \\ -K_{1-2} & K_{1-2}+K_{2-3} & -K_{2-3} & 0\dots & 0 & \left(K_{2-3}h_2-K_{1-2}h_1\right) \\ 0 & -K_{2-3} & \ddots & -K_{n-1-n} & 0\dots 0 & \vdots \\ \vdots & \vdots & -K_{n-1-n} & K_{n-1-n}+K_{TMD} & -K_{TMD} & \left(K_{TMD}h_{TMD}-K_{n-1-n}h_n\right) \\ 0 & 0 & 0 & -K_{TMD} & K_{TMD} & -K_{TMD}h_{TMD} \\ K_{1-2}h_1 & \left(K_{2-3}h_2-K_{1-2}h_1\right) & \dots & \left(K_{TMD}h_{TMD}-K_{n-1-n}h_n\right) & -K_{TMD}h_{TMD} & \sum_{i=1}^{n}\left(K_{i,i+1}h_i^2+K_\varphi\right) \end{bmatrix} \quad (11)$$

Mass matrix [24]:

$$[M]=\begin{bmatrix} m_1 & 0 & \dots & 0 \\ 0 & \ddots & 0 & \vdots \\ \vdots & 0 & m_n & 0 \\ 0 & \dots & 0 & I \end{bmatrix}; \quad I=I_c+\sum_{i=1}^{n} m_i h_i^2, \quad (12)$$

where I_c is a moment of inertia of the construction relative to horizontal axes passing through the center of gravity (CG); h_i is dimension between i-floor and CG; K_x, K_φ are translational and rocking stiffnesses of soil; K_{i-i+1} is shear stiffness of the floor; K_{TMD} is stiffness of the TMD.

Vector $\{I_x\}$ will be:

$$\{I_x\}^T = \{1\ldots1\,0\}. \qquad (13)$$

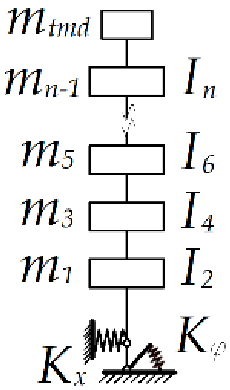


Figure 4. Multi-DOF system of building subjected to a seismic excitation in X-direction with TMD located on the top floor.

Model in Fig. 3 can be used only in case of shear-wall buildings. It is necessary to develop stiffness matrix for considering shear and bending stiffnesses of a building.

2.2. Multi-DOF System for Building with TMD. Shear and Bending Stiffnesses

In general, to consider shear and bending stiffnesses of floors (Table 1), stiffness matrix can be written as follows (Fig.4):

$$[K] = \\ = \begin{bmatrix} A\frac{12i}{l^2}+\frac{12i}{l^2} & \frac{6i}{l}-B\frac{6i}{l} & -\frac{12i}{l^2} & \frac{6i}{l} & 0 & 0 & 0 & \cdots & 0 \\ \frac{6i}{l}-B\frac{6i}{l} & C4i+4i & -\frac{6i}{l} & 2i & 0 & 0 & 0 & \cdots & 0 \\ -\frac{12i}{l^2} & -\frac{6i}{l} & \frac{12i}{l^2}+\frac{12i}{l^2} & 0 & -\frac{12i}{l^2} & \frac{6i}{l} & 0 & \cdots & 0 \\ \frac{6i}{l} & 2i & 0 & 4i+4i & -\frac{6i}{l} & 2i & 0 & \cdots & \vdots \\ 0 & 0 & -\frac{12i}{l^2} & -\frac{6i}{l} & \frac{12i}{l^2}+\frac{12i}{l^2} & 0 & \ddots & \frac{6i}{l} & 0 \\ 0 & 0 & \frac{6i}{l} & 2i & 0 & \ddots & -\frac{6i}{l} & 2i & 0 \\ \vdots & \vdots & 0 & 0 & \ddots & -\frac{6i}{l} & \frac{12i}{l^2}+K_{TMD} & -\frac{6i}{l} & -K_{TMD} \\ 0 & 0 & \vdots & \vdots & \frac{6i}{l} & 2i & -\frac{6i}{l} & 4i & 0 \\ 0 & 0 & 0 & 0\cdots & 0 & 0 & -K_{TMD} & 0 & K_{TMD} \end{bmatrix} \qquad (14)$$

where $A = \dfrac{l^2 K_x \left(K_\varphi + i\right)}{l^2 K_x \left(K_\varphi + 4i\right) + 12i\left(K_\varphi + i\right)}$; $B = \dfrac{l^2 K_x \left(K_\varphi + 2i\right)}{l^2 K_x \left(K_\varphi + 4i\right) + 12i\left(K_\varphi + i\right)}$;

$C = \dfrac{3i\left(K_\varphi + l^2 K_x\right) + l^2 K_x K_\varphi}{l^2 K_x \left(K_\varphi + 4i\right) + 12i\left(K_\varphi + i\right)}$; $i = \dfrac{EI}{l}$; l is the height of the floor; EI is bending stiffness of the floor.

Mass matrix can be written as follows:

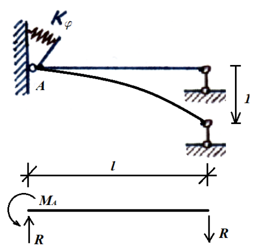
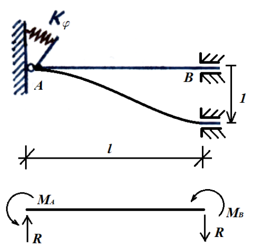
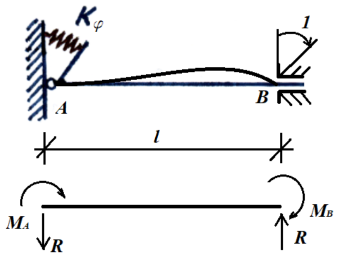
$$\left[M\right] = \begin{bmatrix} m_1 & 0 & \dots & \dots & \dots & \dots & \dots & 0 \\ 0 & I_2 & 0 & \dots & \dots & \dots & \dots & 0 \\ \vdots & 0 & m_3 & 0 & \dots & \dots & \dots & 0 \\ \vdots & \vdots & 0 & I_4 & 0 & \dots & \dots & 0 \\ \vdots & \vdots & \vdots & 0 & \ddots & 0 & \dots & 0 \\ \vdots & \vdots & \vdots & \vdots & 0 & m_{n-1} & 0 & 0 \\ \vdots & \vdots & \vdots & \vdots & \vdots & 0 & I_n & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 & 0 & m_{TMD} \end{bmatrix}; \quad I_i = \frac{m_i}{12}\left(l^2 + b^2\right), \tag{15}$$

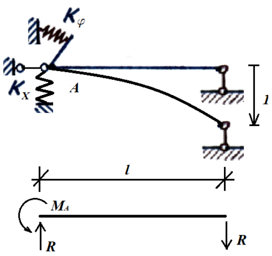
where I_i is the moment of inertia of the floor; l is the height of the floor; b is the width of the building.

Vector $\left\{I_x\right\}$ will be:

$$\left\{I_x\right\}^T = \left\{1 \;\; 0 \;\; 1 \;\; 0 \;\; \cdots \;\; 1 \;\; 0 \;\; 1\right\}. \tag{16}$$

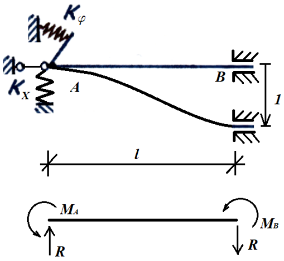
Table 1. Equations for support reactions for beams supported by springs.

Beam subjected to single displacement	Equations for support reactions
	$M_A = \dfrac{3i}{l}\left[\dfrac{K_\varphi}{K_\varphi + 3i}\right]; R = \dfrac{3i}{l^2}\left[\dfrac{K_\varphi}{K_\varphi + 3i}\right]$
	$M_A = \dfrac{6i}{l}\left[\dfrac{K_\varphi}{K_\varphi + 4i}\right]; M_B = \dfrac{6i}{l}\left[\dfrac{K_\varphi + 2i}{K_\varphi + 4i}\right]$ $R = \dfrac{12i}{l^2}\left[\dfrac{K_\varphi + i}{K_\varphi + 4i}\right]$
	$M_A = 2i\left[\dfrac{K_\varphi}{K_\varphi + 4i}\right]; M_B = 4i\left[\dfrac{K_\varphi + 3i}{K_\varphi + 4i}\right]$ $R = \dfrac{6i}{l}\left[\dfrac{K_\varphi + 2i}{K_\varphi + 4i}\right]$



$$M_A = \frac{3i}{l}\left[\frac{lK_xK_\varphi}{lK_xK_\varphi + \frac{3i}{l}K_\varphi + 3liK_x}\right]$$

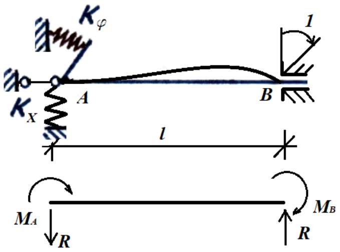
$$R = \frac{3i}{l^2}\left[\frac{lK_xK_\varphi}{lK_xK_\varphi + \frac{3i}{l}K_\varphi + 3liK_x}\right]$$



$$M_A = \frac{6i}{l}\left[\frac{l^2K_xK_\varphi}{l^2K_x(K_\varphi+4i)+12i(K_\varphi+i)}\right]$$

$$M_B = \frac{6i}{l}\left[\frac{l^2K_x(K_\varphi+2i)}{l^2K_x(K_\varphi+4i)+12i(K_\varphi+i)}\right]$$

$$R = \frac{12i}{l^2}\left[\frac{l^2K_x(K_\varphi+i)}{l^2K_x(K_\varphi+4i)+12i(K_\varphi+i)}\right]$$



$$M_A = 2i\left[\frac{K_\varphi(l^2K_x-6i)}{l^2K_x(K_\varphi+4i)+12i(K_\varphi+i)}\right]$$

$$M_B = 4i\left[\frac{3i(K_\varphi+l^2K_x)+l^2K_xK_\varphi}{l^2K_x(K_\varphi+4i)+12i(K_\varphi+i)}\right]$$

$$R = \frac{6i}{l}\left[\frac{l^2K_x(K_\varphi+2i)}{l^2K_x(K_\varphi+4i)+12i(K_\varphi+i)}\right]$$

To use a superposition of modal responses we must have a diagonal damping matrix. We cannot use Rayleigh damping because our system (Fig. 4) consists of three parts with significantly different levels of damping: soil, TMD, and the rest part of the building. In general, the modal damping ratio for the soil system would be much different from the structure's one, for example 15 to 20 % for the soil conditions compared to 3 to 5 % for the structure [25] and 2 to 15 % for TMD (following (6)). This operation can be used:

$$[C] = \begin{bmatrix} \xi_1\omega_1M_1 & 0 & \dots & \dots & \dots & \dots & \dots & 0 \\ 0 & \xi_2\omega_2M_2 & 0 & \dots & \dots & \dots & \dots & 0 \\ \vdots & 0 & \xi_3\omega_3M_3 & 0 & \dots & \dots & \dots & 0 \\ \vdots & \vdots & 0 & \xi_4\omega_4M_4 & 0 & \dots & \dots & 0 \\ \vdots & \vdots & \vdots & 0 & \ddots & 0 & \dots & 0 \\ \vdots & \vdots & \vdots & \vdots & 0 & \xi_{n-2}\omega_{n-2}M_{n-2} & 0 & 0 \\ \vdots & \vdots & \vdots & \vdots & \vdots & 0 & \xi_{n-1}\omega_{n-1}M_{n-1} & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 & 0 & \xi_n\omega_nM_n \end{bmatrix} \quad (17)$$

where $\xi_1 \dots \xi_n$ are damping ratios; $M_1 \dots M_n$ are generalized modal masses; $\omega_1 \dots \omega_n$ are modal frequencies.

If rocking motion of the building on soil and TMD motion have different modes, it is very simple to use damping ratio for each mode on its own according to the matrix above. Fig. 5 shows mode shapes. In case of using TMD for reduction of wind loads, it is necessary to use TMD tuned to the 1st frequency. In case of using TMD for reduction of seismic loads, it sometimes necessary to use TMD tuned to the 2nd frequency. It depends on frequency composition of seismic excitation. If high frequencies prevail over low frequencies, it is necessary to tune TMD to the 2nd or (sometimes) to the 3rd eigenfrequency.

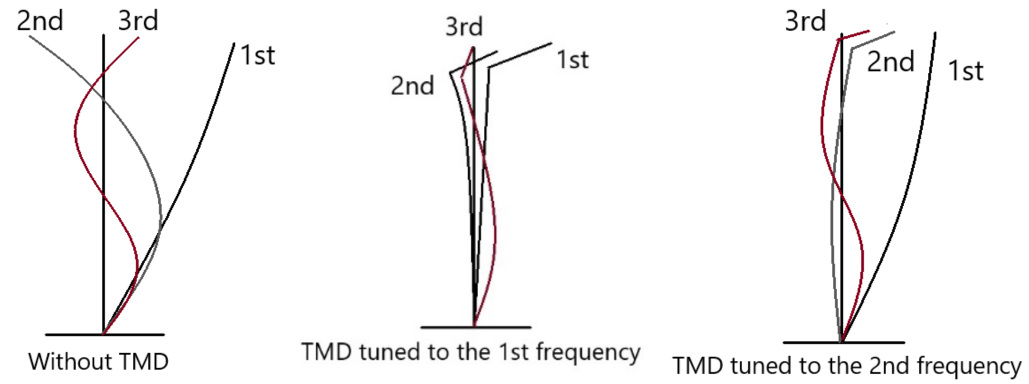


Figure 5. Mode shapes.

2.3. The Target Building. Optimization Criterion

The target building of this study is a residential high-rise building. It is a 103-m-high reinforced concrete building. Building parameters are: L (length) = 36 m, B (width) = 20 m, N_{floors} (number of floors) = 33 (superstructure) and 2 (substructure), H_{floor} (height of the floor) = 3.1 m, M_{total} (total mass) = 45800 t, $M_{structure}$ (mass of structure) = 36650 t. The typical floor is shown in Fig. 6. Equivalent bending stiffness of a floor is: EI_y = 2123292340 $t \cdot m^2$, EI_x = 2550000000 $t \cdot m^2$, translational and rocking stiffnesses of soil: K_x = 1576390 $\frac{t}{m}$, K_{φ} = 517761250 $\frac{t}{m}$, ξ_{soil} = 0.15.

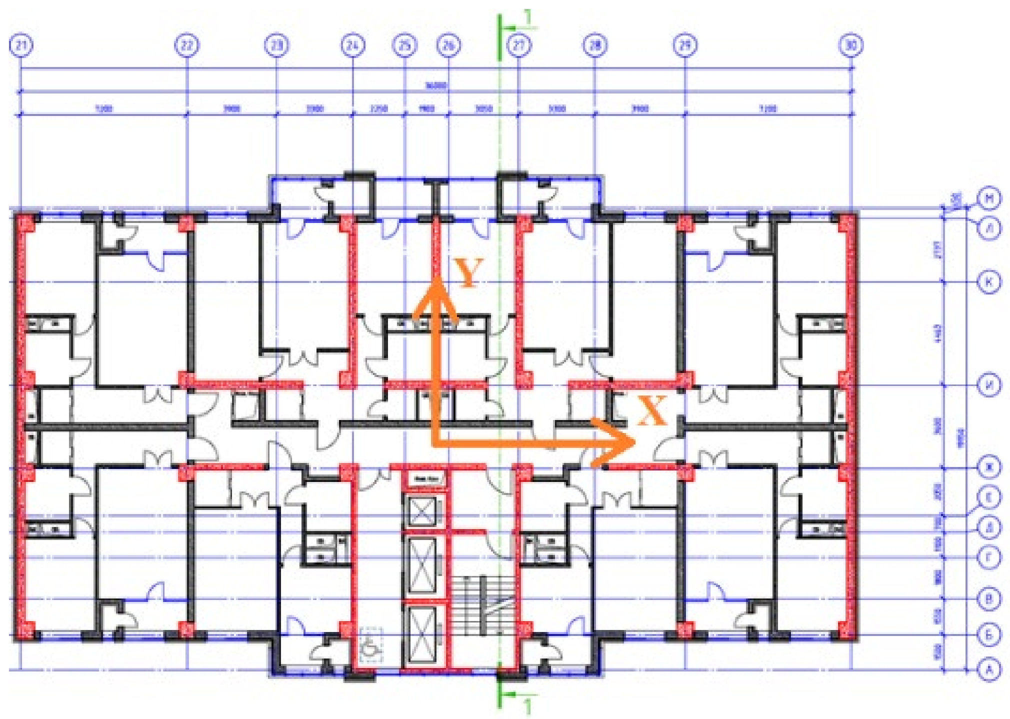
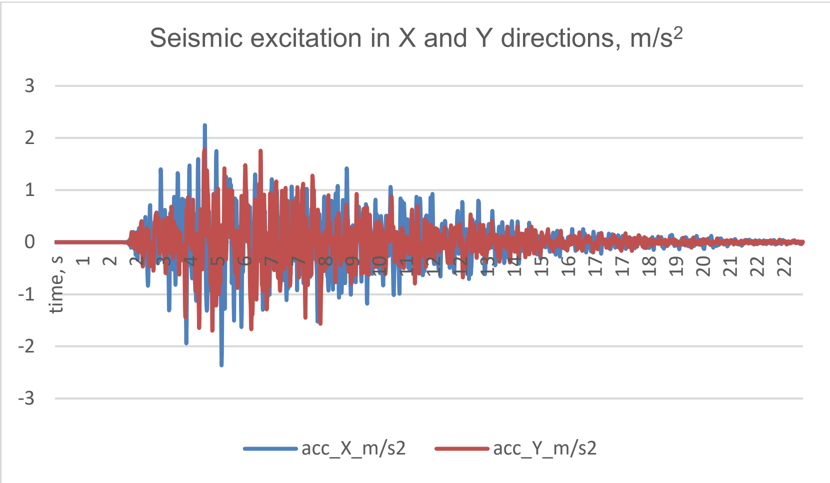
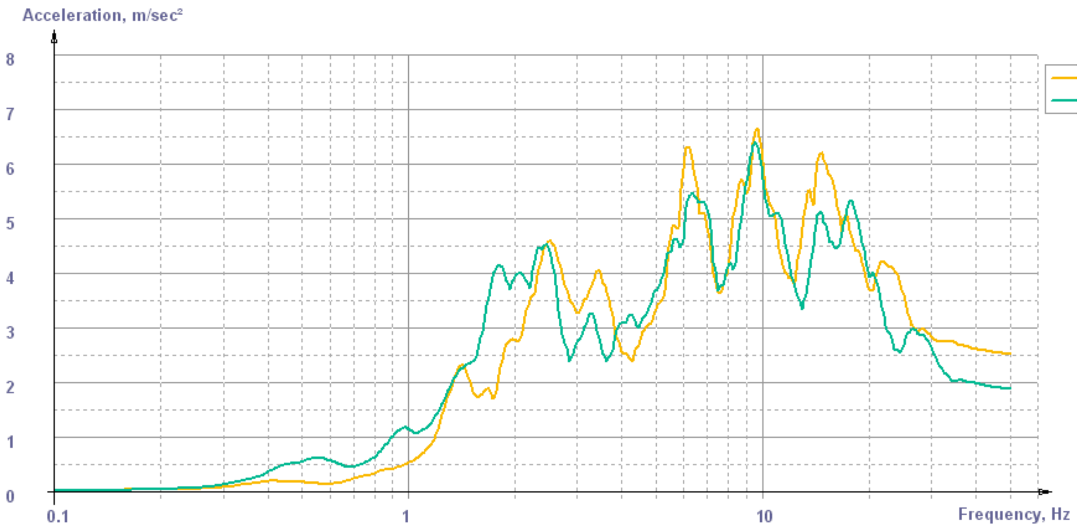


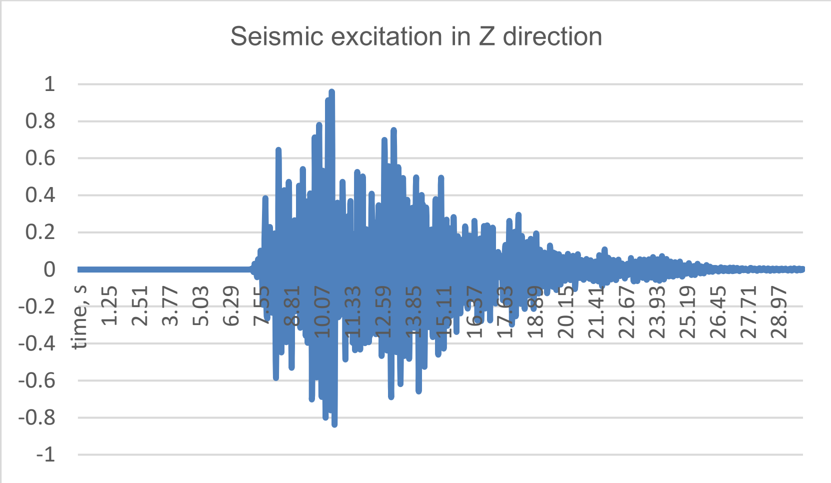
Figure 6. Typical floor plan.

Table 2. Eigenfrequencies of dynamically uncontrolled building without TMD.

Number	Circular frequency, rad/s	Frequency, Hz	Direction	Modal mass, %
1	1.99	0.32	Y	67.46
2	2.04	0.32	X	64.85
3	12.41	1.98	X	21.94
4	12.6	2.01	Y	21.08
5	32.22	5.13	X	9.01
6	33.63	5.35	Y	7.77
7	58.02	9.23	X	3.15
8	61.88	9.85	Y	2.7







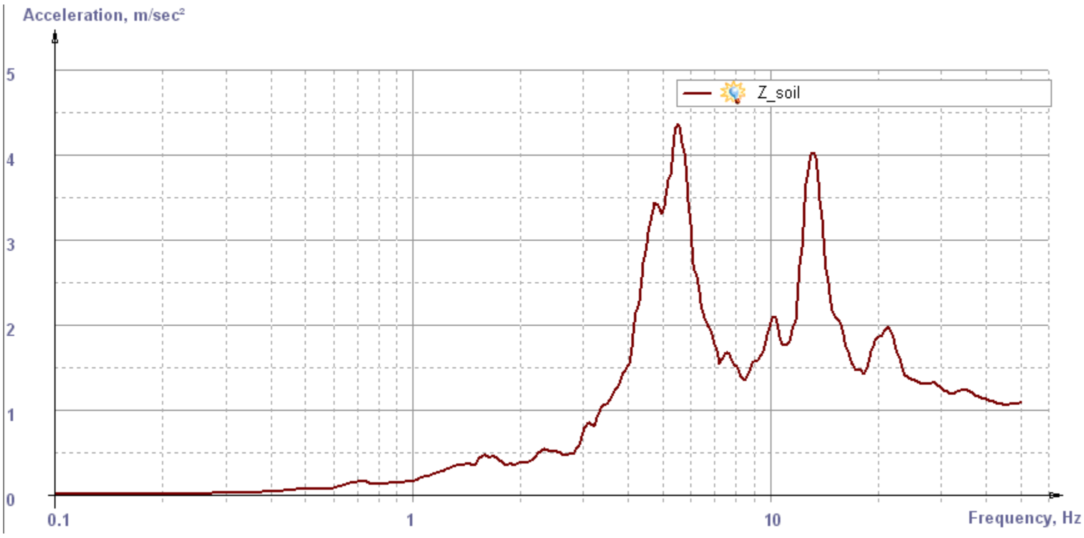


Figure 7. Accelerations on soil in X, Y, Z directions and response spectra (ξ=0.05).

In order to define stiffness, mass and damping ratio of the TMD, it is necessary to carry out optimization analysis. Criterion of optimization:

$$Cr = \max \sum_{i=1}^{n} \left(\max\left|A_i\right| - \max\left|A_i^{TMD}\right| \right), \tag{18}$$

where n is the number of a floor (substructure); $\max\left|A_i\right|$ are maximum absolute accelerations of the n th floor for the dynamically uncontrolled building (without TMD); $\max\left|A_i^{TMD}\right|$ are maximum absolute accelerations of the n th floor for the dynamically controlled building (with TMD).

3. Results and Discussion

3.1. The Optimum Parameters of the TMD

The optimum parameters were defined by taking damping ratio of the TMD $\xi_{TMD} = 0.1$ (using (8)) for the first iteration of optimization, $M_{TMD} = 1050t$ $M_{TMD} = 1050\ t$ $(\mu = 2.3\%)$ and using the model which is shown in Fig. 4, and consistently varying horizontal stiffness of the TMD. The acceleration along the entire height of the building is shown in Fig. 8, 9. Accelerations of the 13th floor in X and Y direction and response spectra are shown in Fig.10.

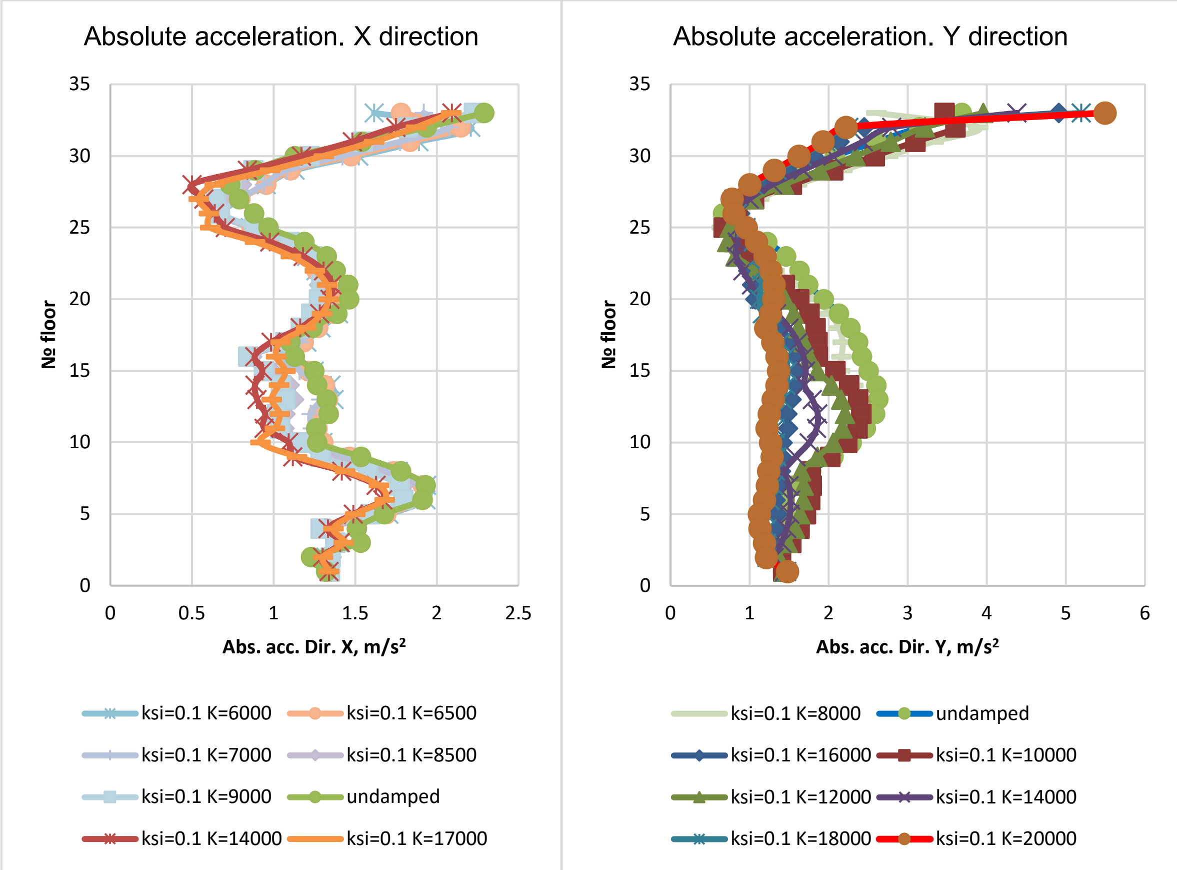


Figure 8. Maximum storey acceleration in X, Y directions.

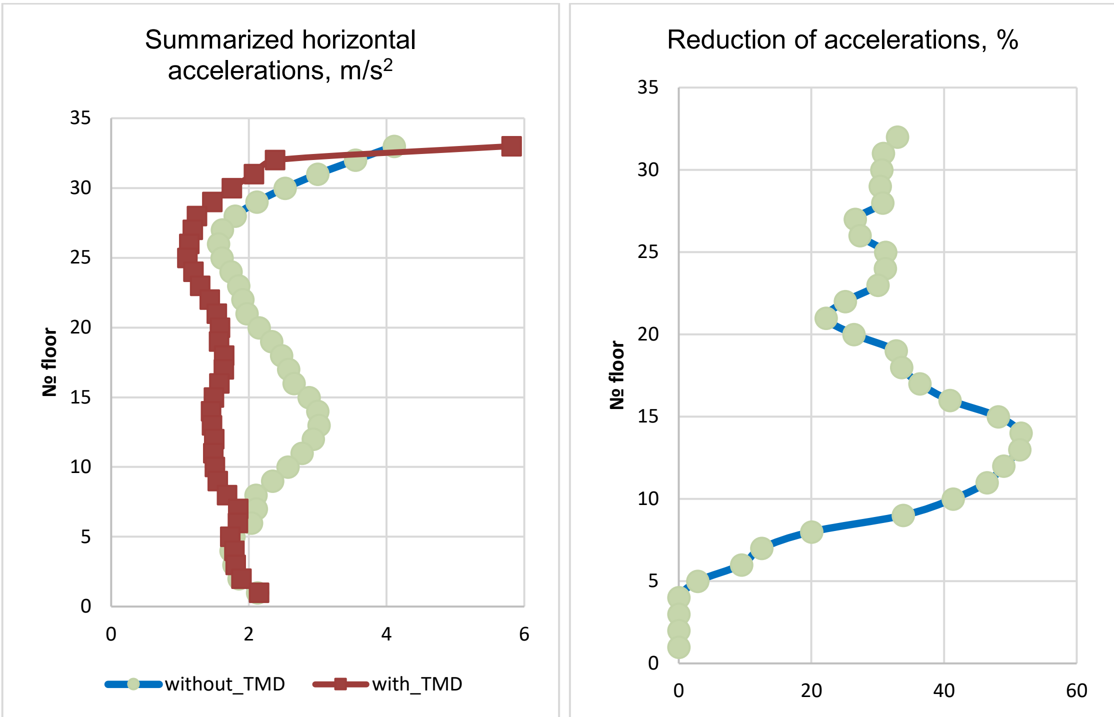


Figure 9. Maximum summarized storey acceleration and the reduction of accelerations.

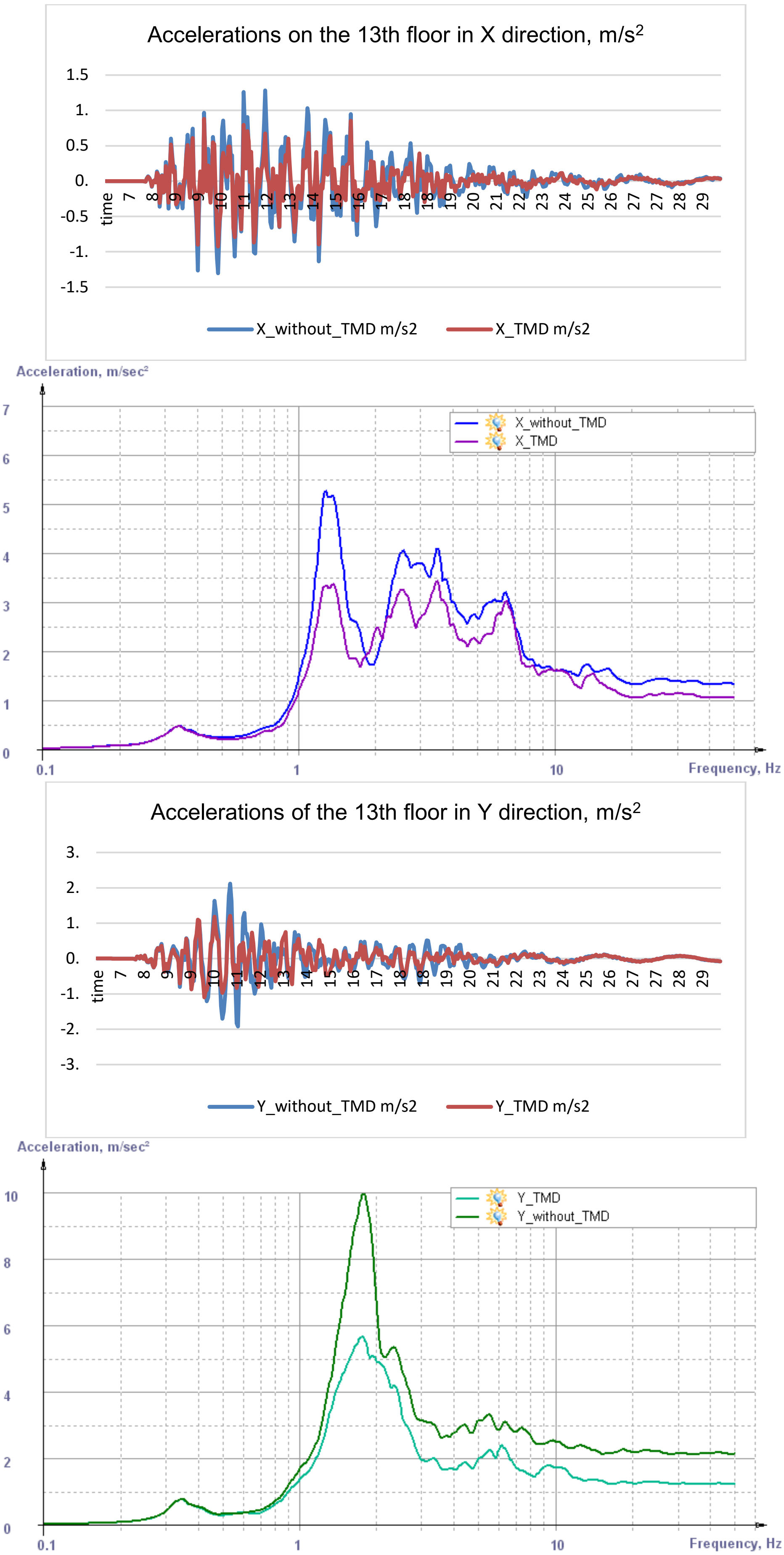


Figure 10. Accelerations of the 13th floor in X and Y direction and response spectra (ξ=0.05).

The optimum parameters for the TMD were installed as follows: K_x = 12000 $\frac{t}{m}$, K_y = 15000 $\frac{t}{m}$, ξ_{TMD} = 0.1, M_{TMD} = 1050 t. The TMD with these parameters is tuned close to the 2nd eigenfrequency of the building. During modal analysis in dissipation matrix (16) $\xi_1 = \xi_{soil}$, $\xi_2 = \xi_{TMD}$ were used. Installation of the TMD led to a minor changes in the eigenfrequencies of the dynamically controlled building (Table 3).

Table 3. Eigenfrequencies of the dynamically controlled building

Number	Circular frequency, rad/s	Frequency, Hz	Direction	Modal mass, %
1	1.98	0.32	X	67.46
2	2.03	0.32	Y	64.85
3	10.3	1.64	X	7.54
4	11.1	1.77	Y	10.63
5	13.72	2.18	X	15.22
6	14.41	2.29	Y	11.18
7	33.18	5.28	X	8.73
8	34.69	5.52	Y	7.49

3.2. Configuration of the TMD

Fig. 11 shows the configuration of the TMD developed for the target building. The construction of the TMD consists of the building' technological floor and roof that constitutes TMD's mass supported by BCS (Base Control System) 3D springs and 3D dampers [26, 27] system that provides near-optimum damping ratio ξ_{TMD} = 0.1. For the considered building according to the optimization analysis, it is necessary to use 50 spring blocks with vertical stiffness $K_v = 1021\frac{t}{m}$ and horizontal stiffness $K_h = 298\frac{t}{m}$ and 25 3D dampers VD 426/219-7 by process conditions[1]. With these near-optimal parameters (mass, stiffness, damping) TMD provides to the building efficient protection from seismic excitation with a quite reasonable relative displacements between TMD and the main structure from 20 to 45 mm only (Fig. 11). Relative displacements between TMD and primary building are shown in Fig. 12.

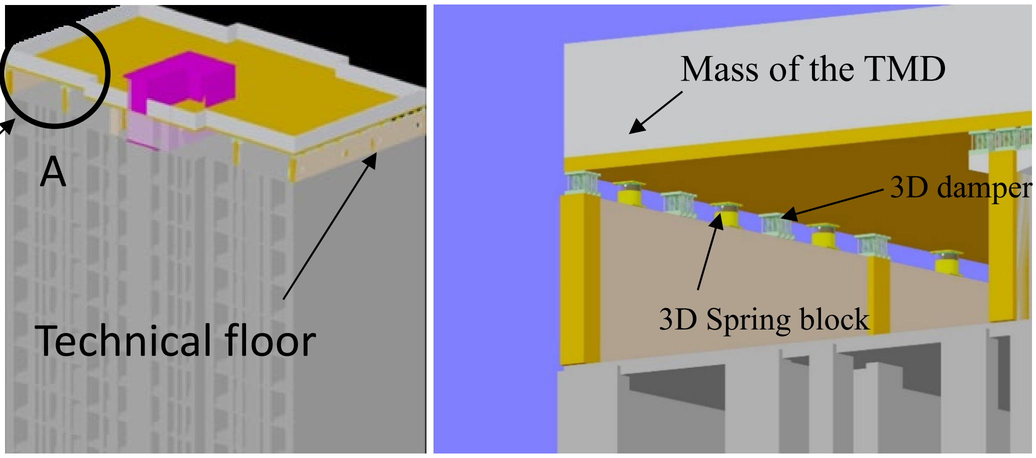


Figure 11. General and detailed view of the building.

[1] CKTI-Vibroseism. Viscoelastic dampers VD series. Specifications. TU 4192-001-20503039-01. 1991/2022

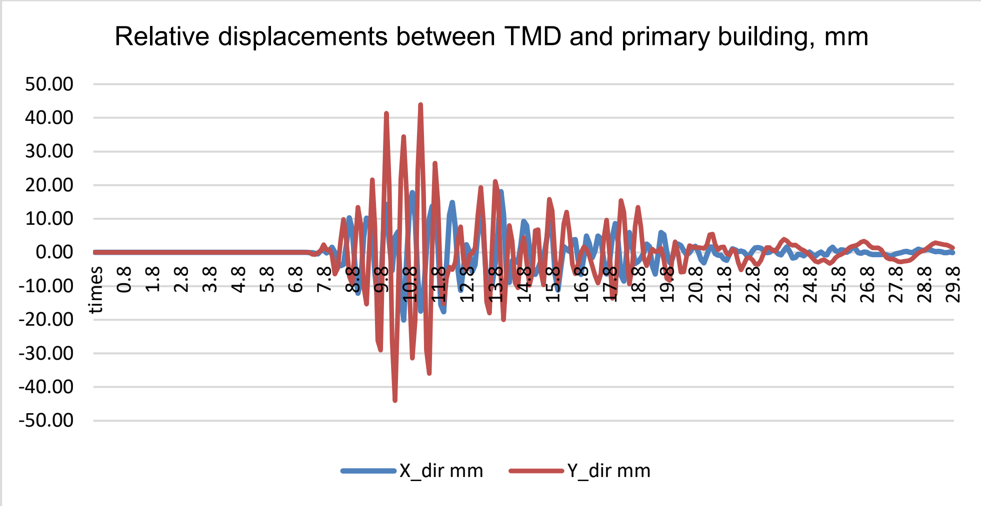


Figure 12. Relative displacements between TMD and primary building.

3.3. Comparison of an Analytical and Finite Element Models. Shear Force and Bending Moment in the Most Loaded Column

Program code of the analytical model mentioned below was written by means of MathCad's software. 3D finite element model (FEM) was used in order to assess stresses in building's elements. 3D building was created by SCad office's software. Fig. 13 demonstrates the 3D FEM of target building. It has 153000 joints and 174400 finite elements. Seismic excitation was presented by kinematic time-dependent displacements of soil. The dissipation matrix for column and beam finite elements is formed by the coefficient of internal inelastic resistance of the material [28]. For beams and columns, damping ratio was used as $\xi_{r.c.}$ = 0.05, for soil – ξ_{soil} = 0.15, and for TMD's constructions – ξ_{TMD} = 0.1.

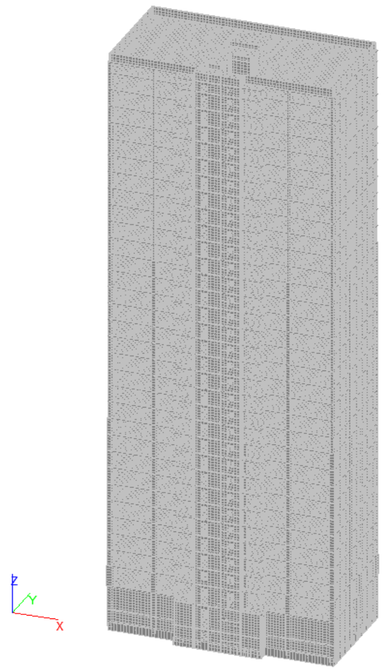


Figure 13. 3D FEM of target building.

The comparison between the analytical and finite element models is shown in Fig. 14 and 15. The displacement of the top floor has a good agreement in terms of maximum and minimum values. The accelerations, in turn, have rather large differences since the analytical model considers non-deformable slabs.

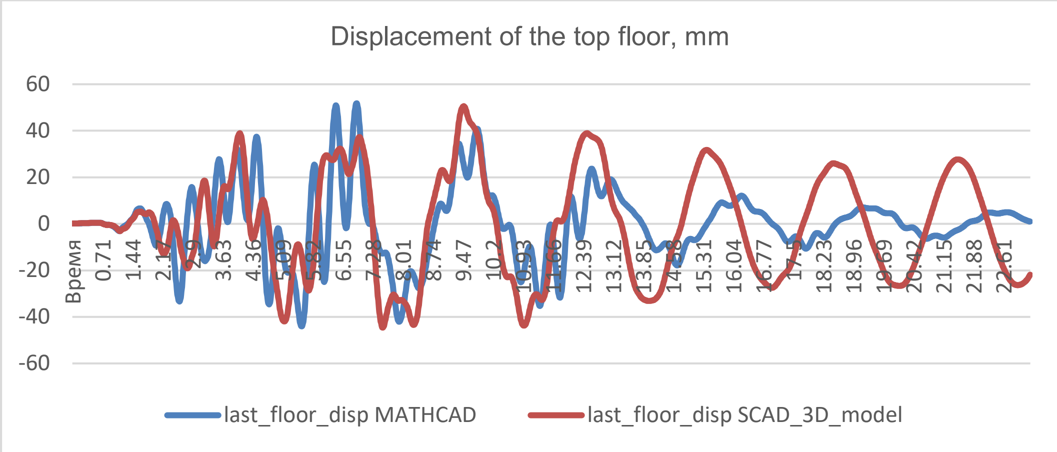


Figure 14. Comparison between two models. Displacement.

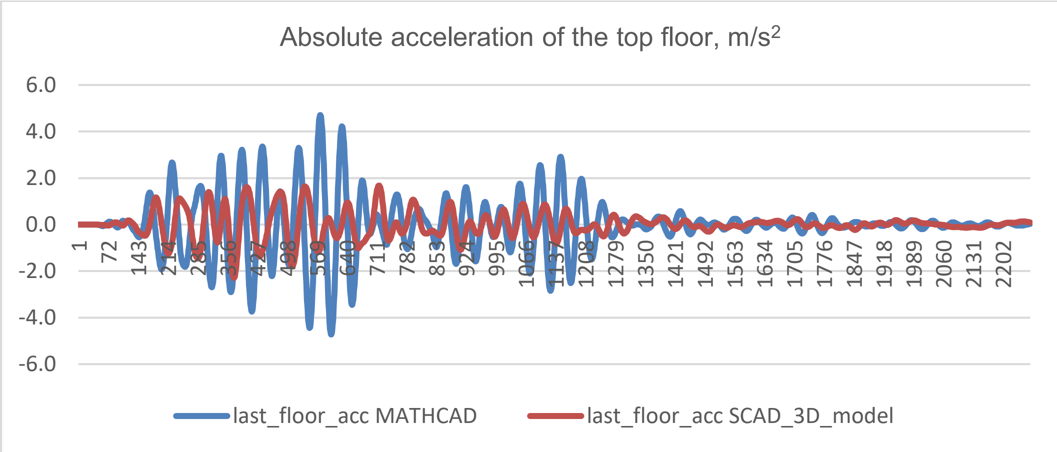


Figure 15. Comparison between two models. Acceleration.

Analytical model allows to define in short time the optimal parameters of TMD's constructions. After this analysis, it is reasonably to use FEM to assess stresses in building's elements. For example, in Fig. 16 and 17 bending model and shear force in the most loaded column are shown. Reduction of the inner forces in column has reached up to 50 %.

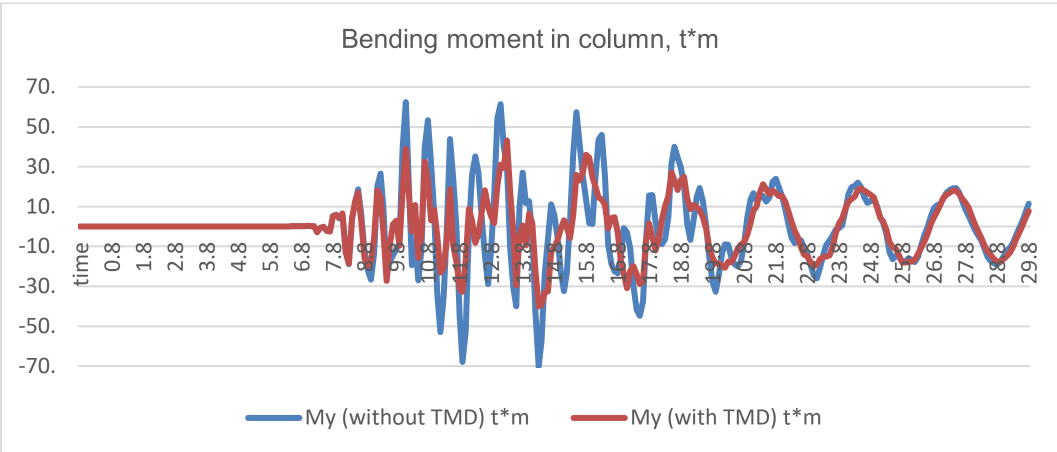


Figure 16. Bending moment in column in target building with and without TMD.

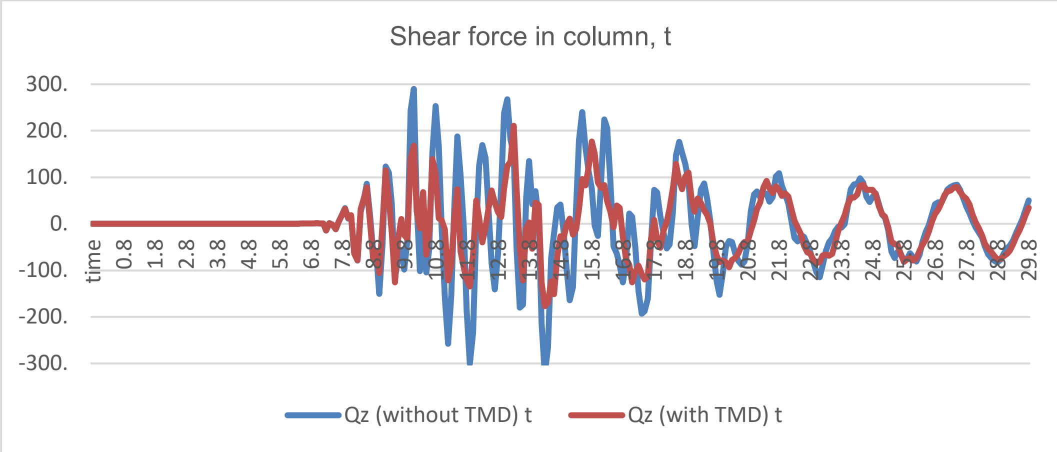


Figure 17. Shear force in column in target building with and without TMD.

3.4. Reduction of the Vertical Accelerations in Target Building

Due to vertical stiffness of BCS spring blocks and using 3D FEM, it is possible to assess the efficiency of the TMD in Z direction subjected to vertical seismic excitation (Fig. 7). In Fig. 18 and 19 time-history acceleration and response spectra of the point, which is close to shear wall of the building, is shown. The TMD can reduce vertical acceleration up to 30 % in comparison with uncontrolled building. It was achieved principally new effect of efficiency of the TMD in Z direction. This effect has not been mentioned in recent investigations in TMD's field by others researchers [4–6, 12, 20–22].

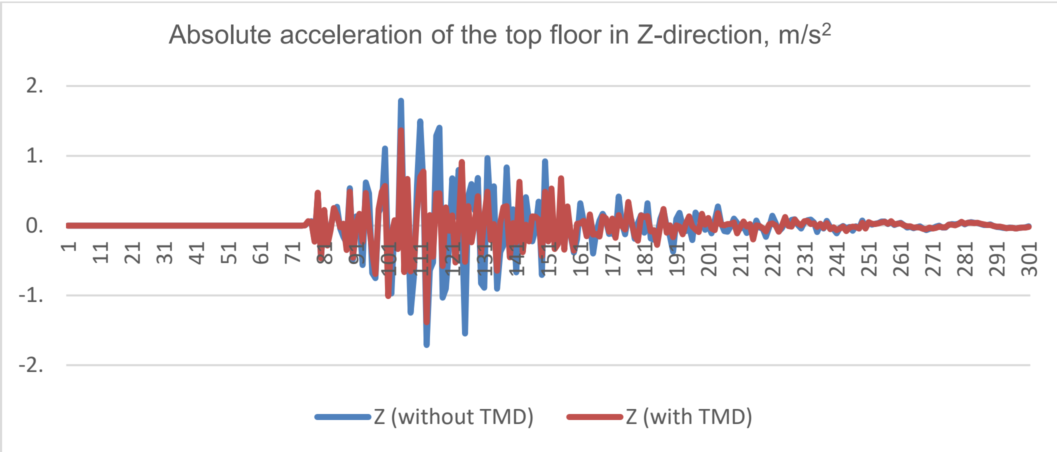


Figure 18. Vertical absolute acceleration of the top floor in target building with and without TMD.

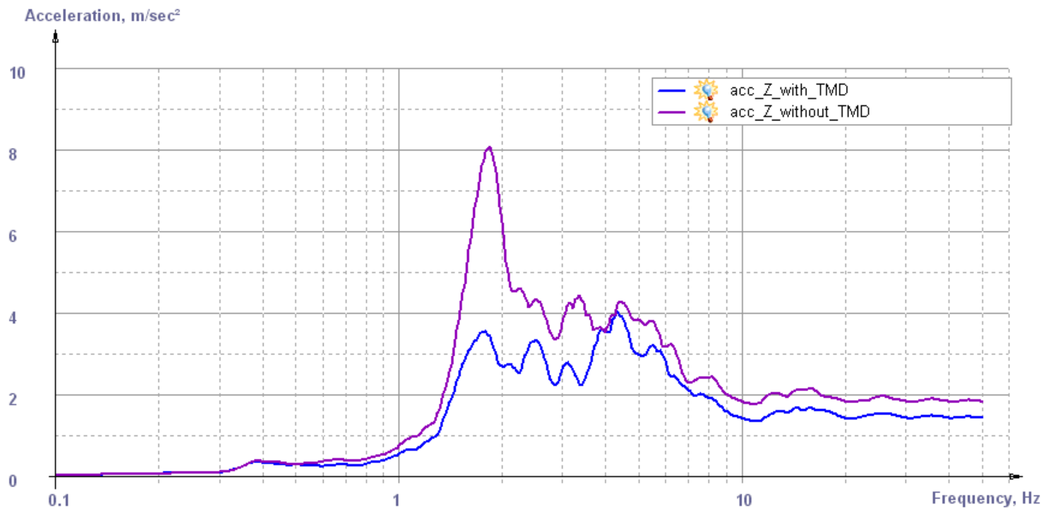


Figure 19. Response spectra (ξ=0.05) of the vertical absolute acceleration of the top floor with and without TMD.

4. Conclusions

The main idea of the proposed TMD is to use the existing upper technological part of the building, located above the residential floors, as a TMD device. This new approach can achieve significant increase in TMD's efficiency by using the optimal mass, stiffness and damping properties of the TMD.

The proposed TMD construction has an optimal mass ratio of about 2 %. It can be installed in existing buildings or in new buildings using the upper technological floor and the roof as the mass of the TMD. This solution does not require transportation of a huge TMD to the installation site on the top floor of the structure. The TMD is a passive seismic and wind control device and does not require any external energy sources or its maintenance. The relative displacements between the TMD and the building are quite acceptable, less than 5 cm, due to the optimal damping of the system.

A stiffness matrix has also been developed. It allows to consider soil stiffness, bending and shifting modes of a building. It is very easy to use the damping ratio in analysis separately for soil and for the TMD in case where the TMD is not tuned to the 1st eigenfrequency of a building.

The approach of using the top of a building as a TMD can significantly reduce response accelerations and stresses in elements of the building along the entire height subjected to seismic and wind loads. It has been shown that the accelerations was reduced by up to 50 % in comparison with a dynamically uncontrolled building.

The developed TMD construction can reduce vertical acceleration and motion in primary building due to the large mass of the roof. It has been shown that the vertical acceleration of the top floor was reduced by up to 30 % in comparison with a dynamically uncontrolled building.

Innovative TMDs were developed to reduce seismic vibrations in high-rise buildings. However, it is also possible to use this construction of TDM for reduction of wind vibrations.

In general, TMD mentioned in this paper is able to reduce rocking and torsional motions caused by earthquake and wind loads and these investigations are ongoing.

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Received: 27.12.2022. Approved after reviewing: 20.07.2024. Accepted: 21.07.2024.