Experimental qualitative identification of damping and stiffness characteristics of lattice towers

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Abstract. The results of full-scale test and numerical simulation of the dynamic behaviour of the latticed sightseeing tower are presented in the article. It is noted that for effective control of the structural vibrations, it is necessary to study the mechanisms of energy dissipation in real objects. The phase trajectories in the extended phase space were applied in order to identify sources of non-linearity.

1. Introduction

Structures of lattice towers and sightseeing platforms are sensitive to dynamic loads. Such loading can be caused not only by the action of wind, ice, earthquakes, blows, explosions and pedestrians but also by mechanical failures of some bearing elements. The vibrations that are excited by these effects cover a wide range of frequencies. They can cause both feelings of discomfort for people, and fatigue and destruction of structures. In a number of cases, different damping devices are installed to control such oscillations in self-supporting towers and masts. Understanding and controlling vibration damping mechanisms is important for control of vibrations. Therefore, in order to assess the damping capacity of lattice towers, it is necessary to identify and evaluate the dominant sources of damping.

In modern analysis of the dynamic behaviour of mechanical systems, special attention is directed to the development of complicated computer models. These models are usually used to solve two basic types of tasks. The first problem is to estimate the response of the mechanical system under investigation to external excitements, while, the second task is to predict the effect of changes or modifications, the very mechanical system on its dynamic characteristics.

Unfortunately, the existing database of full-scale dynamic testing of lattice towers is very limited [1 - 7]. In the work of Kitipornchai et al. [2] it was found that the behaviour of lattice towers is non-linear. It can be explained by the presence of gaps, and the possibility of slippage of a joint. The behaviour of bolted connections is linear with loads not exceeding the value of frictional force in the joint. However, for large values of loads, slippage is observed. This leads to the manifestation of a local nonlinearity. The load on the slippage of the bolt is related to its pre-tension [3], which is a

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parameter that is not always well controlled in practice. Slippage of the connection leads to loss of flexibility and an increase in damping.

This underlines the possible dependence of the behaviour of lattice towers on the level of the load. Glanville et al. [4] tested towers and estimated damping factors from 0.5% to 1% of the critical value damping were measured on the response of free oscillations after forced low-level vibration. Ostendorp [5] tested a lattice tower with a height of 30 m. The damping coefficient was measured by time processes of free oscillations after unloading and ranged from 0 to 40% with an average value of 17%. The relationship between the damping coefficient and the loading level was established in that study.

2. Methods for refinement of finite element models

Digital computers generate predictions of the dynamic behaviour of mechanical systems based on discrete models. However, the comparison of the results of numerical simulation and the data of dynamic testing does not always show they are satisfactorily correlated. Therefore, the use of discrete models meets a number of limitations.

Inaccuracies of discrete numerical models give rise to both quantitative and qualitative discrepancies in the results of numerical simulation and experimental data. It should be noted that experimental measurements also inevitably contain noise and errors, but they more fully reflect the behaviour of real objects. In most cases, the improvement of the model consists of changing mass parameters, stiffness and damping to obtain the best correlation between the numerical results and the experimental data.

When constructing finite element models, we usually introduce a number of simplifying assumptions about the mechanical features of materials, boundary conditions, and operating loads. In order to more accurately represent the geometry of elements of the mechanical system, in many cases an excessively fine grid of finite elements is used. In view of this, it becomes necessary to estimate the sensitivity of the results of numerical simulation to the change in the size of the finite element grid.

In simple terms, the lattice observation tower can be regarded as a system that consists solely of bar finite elements. The solid observation deck is located at the top of the tower and several platforms for tourists are provided at different levels along the tower height. These elements possess significant flexural stiffness and therefore**,** they were simulated as plates. As it has been stressed in a number of investigations [8-9], the simulation of joints in plate-bar connections is a very challenging task that requires special experience.

Figure 1. Sightseeing tower in Dzintari.

There are also doubts about the extent to which the accepted boundary conditions reflect the real behaviour of bolted and welded joints of structures. It is not always possible to state with certainty that the boundary conditions and connections between some elements adopted in the simulation reflect the actual operation of the structures. For example, bolted joints and wildings. In such cases, the researcher can test the sensitivity of the numerical simulation results to changes in grid configuration or boundary constraints. Inaccuracies and mistakes will inevitably accumulate in the model. Therefore, the main goal of engineering calculations is to ensure acceptable results [7].

3. Study and methods

The object of the study is a sightseeing tower with a height of 36.48 m, which is located in Dzintari city of Jurmala, Latvia, see figure 1. It was erected in 2010. The construction of the tower is shown in figure 2. The structure consists of a strengthened inner core with a size of 1500x1500 mm made of tubes with a cross-section of 200x200x8 and an outer core of 4240x4240 mm, made of tubes with a cross-section of 140x140x5. All supporting elements of the tower, namely the inner and outer core, platforms and stairs are made of steel. Elements, facade cladding and facing of the core were made of wood. There are no vertical tie elements.

To evaluate the dynamic properties of the tower, a three-dimensional model of the finite element method was developed. The modal analysis of this structure was carried out in the software package STRAP 12.5 [10].

Let us assume that the external forces and damping are zero. The equation of free vibrations of the structure is expressed in matrix form as

$$[\mathbf{M}]\{\ddot{u}\} + [\mathbf{K}]\{u\} = 0 \ , \tag{1}$$

where $\mathbf{M}$ is the mass matrix, $\mathbf{K}$ is the stiffness matrix, $\ddot{u}$ is the acceleration vector, and u is the displacement vector.

Suppose, the free oscillations are harmonic. Then, the displacements of the dynamical system can be represented in the following form:

$$\{u\} = \{\phi\}_i \cos \omega_i t, \tag{2}$$

where $\{\phi\}_i$ is the i^{th} eigenvector representing the form of oscillations at the i^{th} frequency, ω_i is i^{th} eigen circular frequency, t is a time.
Substituting (2) into (1), we obtain:

$$([K] - \omega_i^2 [M])\{\phi\}_i = \{0\}. \tag{3}$$

This equality is satisfied, providing either $\{\phi\}_i = 0$ or the determinant of the matrix obtained from $[K] - \omega_i^2 [M]$ is zero. The first case gives a trivial solution and therefore, it is not analyzed. The second case leads to the generalized eigen value problem (4). The solution of the generalized eigen value problem consists in finding the number n of pairs of the eigen frequencies and the eigenvectors, and the value n depends on the order of the system, i.e. the set of degrees of freedom of the structure.
In these full-scale experimental studies, in addition to the circular eigen frequency ω_i, the eigen frequency f_i was also used, which represented the number of oscillations per unit time expressed as $f_i = \omega_i / 2\pi$.

To evaluate the effect of individual structural elements on the dynamic behaviour of structures as a whole, three finite element model models [11] were constructed, presented in Table 1. The results of the modal analysis for all three models are presented in Table 1. A peculiar feature of the structure under study is the symmetry of elements of its outer circular shell structure and central inner core with respect to their central axes (figure 2). For this reason, the frequencies of the first and second modes of bending vibrations in the first model coincide with the frequencies of the same modes in the second model. It is also illustrated in Table 1. However, the same pattern is not observed in model 3 because the additional elements (stairs and platforms) introduced into the simulation perverted the symmetry of the model.

Table 1. Results of numerical simulation.

Model Nr	Description	Inner core dimensions (mm)	Outer core dimensions (mm)	1st mode (Hz)	2nd mode (Hz)	3rd mode (Hz) Torsional mode
1	Only inner core	1500x1500	-	1.521	1.521	6.051
2	Inner and outer core	1500x1500	4240x4240	0.874	0.874	2.031
3	Inner and outer core, stairs	1500x1500	4240x4240	1.506	1.511	2.010

As expected, the fundamental frequencies of the shapes of the first and second modes mainly depend on the flexibility of the inner core. The outer core has no vertical bonds and its flexibility is not significant. Thus, it acts as an attached mass, reducing the oscillation frequency of the entire tower. Analyzing the model number 3, we note that the stairs increase the flexibility of the tower in the horizontal direction. Because of this, the frequencies of oscillations in modes 1 and 2 increase.

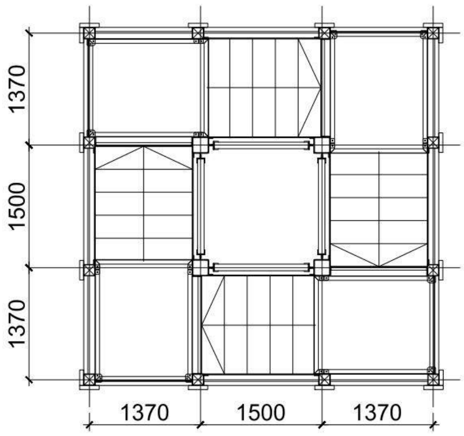

Figure 2. Plan of the Sightseeing tower in Dzintari.

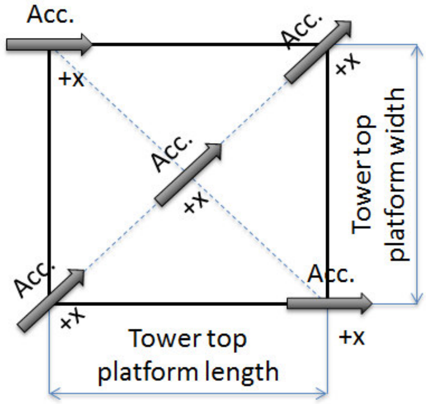

Figure 3. Scheme of accelerometers arrangement.

4. Technique of full-scale experiment

The forced dynamic test of the lattice tower in Dzintari was carried out for each vibration mode to investigate its elastic and damping characteristics. External excitement was created by the movement of two people in time with a metronome at different frequencies of the tower. External excitement stopped when resonance modes of oscillations were established, then free oscillations of the tower were observed until it stopped completely. Tests for forced vibration were conducted in quiet atmospheric conditions to eliminate any aerodynamic effects [10]. It should be noted that the external excitement from the movement of two people along the height of the tower is small in comparison with the eigen mass and flexibility of the sightseeing tower. That is why the peaks in the output frequency spectrum are the reactions of the design itself.

During the experiments, the vibroaccelerations of the points of the upper platform of the tower were measured and recorded. The accelerations were recorded by means of five accelerometers triaxial USB accelerometer model X6-1A. Each accelerometer simultaneously recorded vibration accelerations in three directions. The sampling frequency of the sensor signals was 160 Hz. This value exceeded the largest value of the modal frequency obtained in numerical simulation and ensured the correctness of signal processing. The weight of the sensors was small (55 g), and therefore could not interfere with the inertial characteristics of the object. The arrangement of the accelerometers is shown in figure 3.

5. Phase trajectories of oscillations of non-linear systems in the expanded phase space

Dynamic behavior of mechanical systems is usually presented as an oscillating processes in various graphic forms such as time processes, the Lissajous patterns and hodograph. Such patterns of presentations enable to determine the type of a process and to perform numerical estimations of its characteristics, but do not disclose any properties of the governing system. Unlike these patterns, classic phase trajectories have a row of advantages.

A phase space in classic mechanics is represented as a multidimensional space. The number of measured values for a phase space is equal to the doubled number of degrees of freedom of the system being investigated. The state of the system is presented as a point in the phase space, and any change in the system state in time is depicted as the displacement of the point along a line called a phase trajectory. The image on phase plane $(y,\ \dot{y})$ is a more vivid presentation because it depicts inharmonious oscillations particularly well. Each phase trajectory represents only one definite clearly defined motion. As is has been shown by the investigations [12-14] , the expansion of a phase space by taking into account the phase planes $(y, \ddot{y})$ and $(\dot{y}, \ddot{y})$ substantially promotes the efficiency in analyzing a dynamic system behaviour. Thereby we pass on to a three-dimensional phase space confined with three co-ordinate axes, i.e. displacement, velocity and acceleration. An interest taken into accelerations in dynamic systems is conditioned by the fact that these accelerations are more sensitive to high-frequency components in oscillating processes. Phase plane $(y, \ddot{y})$ is of particular interest in the analysis of dynamic system behaviour, because it allows a more evident interpretation of power relations in the dynamic system under investigation. Namely, the area confined by curve $\ddot{y}(y)$ and axis $(0\,\ddot{y})$ is equal to work, and the anticlockwise motion around its contour corresponds to the energy spent by the system for one cycle of oscillating. Another important characteristic of phase trajectories on plane $(y, \ddot{y})$ is the fact that dependence $\ddot{y}(y)$ for autonomous non-conservative systems is a mirror symmetric image in relation to axis $(0\,\ddot{y})$ to the graph of changes in elastic force characteristics.

6. Data acquisition and processing

Taking into account that the exact moment of the end of external excitement was not recorded, then the response was fixed throughout the experiment, up to the transition of the structure to an equilibrium state. Using the FFT function in the Matlab software, a fast Fourier transformation of the acceleration records was performed. Figure 4 shows the records of time histories for the top observation deck of the tower. As follows from figure 4, the signal consists of several frequencies: the response of the structure at its first natural frequency, which is about 0.736 Hz; response strictures at a frequency of 3.27 Hz; high-frequency components with frequency more than 9 Hz, which can be attributed to noise or the contribution of higher modes. It was noted that the experimental value of the frequency of the first form of oscillations is lower than it was numerically estimated, see Table 1. This can be explained either by the influence of the inertia forces of the wooden cladding elements of the structure exterior surfaces, the effect that was neglected in the simulation, or by the action of the positional frictional forces. In the future, a bandpass filter was used to eliminate noise caused by interference in the operation of the measuring equipment. It was assumed that such data processing will not affect the results of estimating the logarithmic decrement.

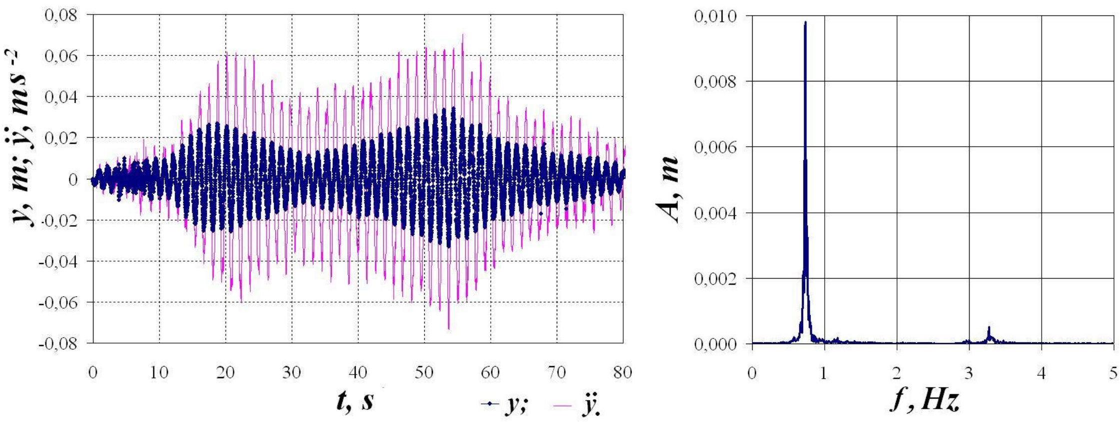

Figure 4. Typical trace of tower response to outer excitation and spectral characteristics.

The damping coefficient was determined from the value of the logarithmic decrement. Two approaches have been used. In the first approach, the logarithmic decrement was evaluated as a function of the maximum values of the accelerations for the entire record. In the second, the value of the logarithmic decrement was determined for individual cycles. The value of the logarithmic decrement, in general form, can be determined from the expression

$$\delta = \frac{1}{T}\ln\left|\frac{y_{i-1}}{y_i}\right|, \quad i = 1, 2 \ldots n. \tag{4}$$

where T - period ; $y_{i\text{-}1}, y_i$ - consecutive values of the maximum deviations.

The value of the decrement of oscillations for a given level of loading varied in the range from 0.034 to 0.055.

Analysis of the presented phase trajectories shows the tower is in one equilibrium position, corresponding to the initial static deviation. As can be seen from figures 5 and 6., with insignificant values of displacements, the behavior of the tower is linear, and with increasing displacements the nonlinearity of the elastic forces is manifested.

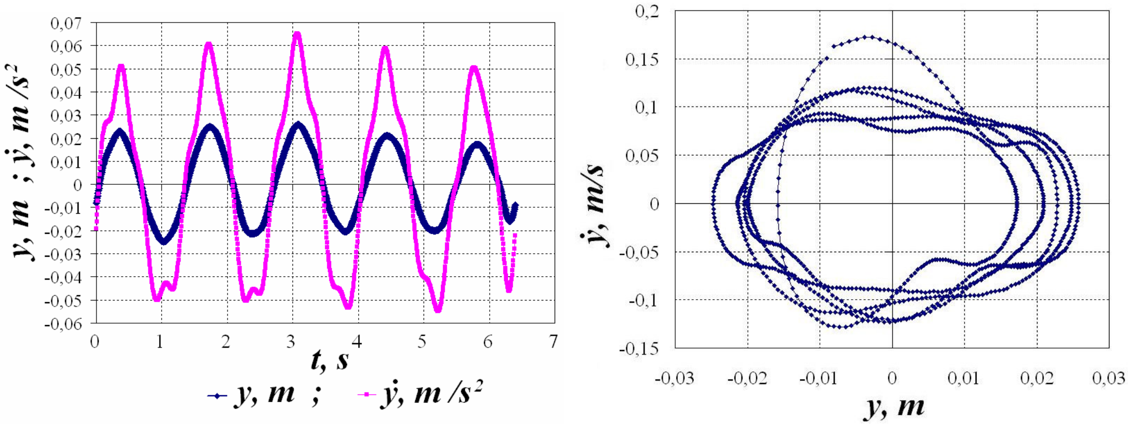

Figure 5. The experimental recordings of the time processes and the Poincare phase trajectories of free damped vibrations in the lattice tower.

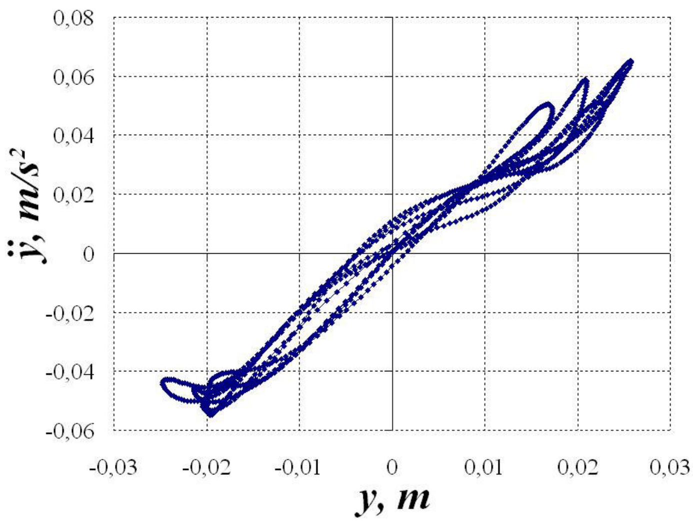

Figure 6. Phase trajectories on extended phase space of free damped vibrations in the lattice tower.

Obviously, the flexibility of the tower is variable and depends on the level of movement. For large load levels, the static flexibility of the tower decreases. There are additional loops on the end sections of the phase trajectories on the plane. This phenomenon is caused by action of subharmonics of order $\omega/5$. The nonlinearity of the elastic characteristics is also confirmed by the existence of beat modes at high frequencies.

7. Conclusion

As visible by results of the research, the scope of application of classical methods of structural analysis for lattice towers is limited by low levels of displacement and corresponding loads. Welded joints and joints on friction bolts are commonly used to connect trusses and frame elements of high-latticed towers. They lead to the appearance of elastic and dissipative properties. Since the number of mobile connections is small and their displacements are limited, the dissipation of the energy of vibrations is insignificant due to the discontinuity. The main mechanism of energy dissipation of lattice towers is positional constructive friction. The amount of energy dissipated depends on the displacements.

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