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Multi-dataset OMA and Finite Element Model Updating of Steel Observation Tower

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Abstract. Operational modal analysis for identifying dynamic parameters, together with finite element model updating algorithms, is a promising and powerful tool for detailed analysis of complex civil engineering structures. It is also an integral part of vibration-based methods in structural health monitoring. In this work, the identification of the dynamic parameters was performed based on output-only vibration data recorded in a testing campaign on a 36 m high steel structure used as an observation tower for tourists in Latvia. To correctly interpret the experimental results, a finite element model of the tower has been developed within the Ansys environment. The model updating is performed by adopting an artificial intelligence algorithm called Particle Swarm Optimization. The calibration is performed with the aim of obtaining a numerical model that simulates the real dynamic behaviour of the case study with high accuracy. The calibrated model can be used as a base for the development and design of the structural health monitoring system of the tower.

1. Introduction

The maintenance of structures during their life is fundamental to guarantee their full and safety usage. A recommendable maintenance programme is that based on the condition-based philosophy instead of the much common time-based maintenance [1] since a more rational use of economic resources is desirable, especially for public entities. However, the development of a condition-based maintenance programme requires the structure to be periodically inspected and assessed, sometimes performing tests with the aim of identifying possible damage on structural members.

In recent years, advanced techniques for assessing the dynamic behavior of structures have gained significant attention among researchers and practitioners alike. One such method, which combines Operational Modal Analysis (OMA) and Finite Element Model (FEM) updating, has emerged as a powerful tool for structural health monitoring and detailed analysis of complex structures [2].

The requirement for damage inspection techniques that can give global information on the structure led to the development of the so-called model-based methods that numerically examine changes in the modal parameters of the structure [3]. The growing popularity of vibration-based methods (VBM) for structural health monitoring (SHM) also necessitates highly accurate FEMs to enable early detection and monitoring of structural damage, ensuring prompt intervention and maintenance [4].

Steel sightseeing towers serve as excellent candidates for the application of VBMs for SHM and damage detection [5]. Their flexibility makes them sensitive to global vibrations, while the material's isotropy reduces uncertainty in modelling mechanical properties, such as stiffness and strength.
But on the other hand, possible complex geometry, eccentricity, and structure-ground coupling effects can introduce additional challenges to the predictability of structures' actual behavior under service loads. Therefore, dynamic properties obtained from the FEM often are different from the results obtained from experimental tests. In this case, it is desirable to adjust or update the FEM so that the obtained numerical results are close as possible to the experimental data [6,7], enhancing the trustworthy of the numerical model in predicting the real structural behaviour. The process that tries to solve this issue is called model updating. Classically, model updating includes trial-and-error-based updating and optimization-based updating. While trial-and-error-based updating leads to a satisfactory solution when there is little prior information, optimization-based updating is generally more efficient and accurate for more complex situations. Model updating procedure are usually time-consuming and sometimes may not be feasible [8]. Artificial intelligence algorithms are frequently used for FE model updating. One of the well-known is the Particle Swarm Optimization (PSO) algorithm, which belongs to the swarm intelligence algorithms [9].
This paper deals with the experimental and numerical dynamic investigation of a 36 m high steel structure used as an observation tower for tourists in Latvia. The tower is dynamically tested through ambient vibration tests (AVTs) and OMAs to identify its dynamic behaviour. Then, a FEM of the structure was developed using the Ansys commercial software. This numerical model is updated adopting the PSO algorithm and the model updating is performed comparing the experimental dynamic behaviour (in terms of natural frequencies and mode shapes) with the relevant numerical one. The calibrated model is used to further support the experimental dynamic identification and, most of all, it may be used in the future for the design of a SHM system to put in place on the tower.

2. Description of the Steel Observation Tower

The steel observation tower is located in Jurmala city Dzintaru Forest park, Latvia. It was built in 2010 and serves as an observation tower for visitors to Jurmala city. This observation tower provides a view of Jurmala city, the Baltic Sea, and even the capital city of Latvia - Riga.
This slender steel structure is constructed on a shallow foundation consisting of a concrete slab freely supported on the ground. The observation tower is 36.480 m tall with 9 main floors and a top floor on a 33.495 m level. The main outer columns cross-section is a square hollow section (SHS) 140x140x5 mm and the internal columns section is SHS 200x200x8 mm. Diagonal braces between internal columns cross-section is SHS 80x80x3, the cross-section of beams for staircases are UPE 120, and stair threads flat bars are 10x200 mm. The upper-level platform beams are HEA 200. The material of the main structures is steel with structural grade S235. The exterior wooden beam decoration cross section is 45x45 mm. Stair landings and steps are made of welded platform grid (Figure 1).
Previous researches [10] investigated this tower's response to human-induced vibrations. The tower's natural frequency in this study was found to be close to that of human pacing, resulting in significant longitudinal, transverse, and torsional vibrations caused by visitors' movements on the stairs. In this study, short measurement campaigns were done by simple unsynchronized MEMS accelerometers. Nevertheless, it gives us the possibility to compare frequency identification results with a 10-year difference.

3. Ambient Vibration tests

Identification of the dynamic parameters of the tower was performed based on vibration data recorded during a testing campaign performed from 01.11.2022 to 08.11.2022.
The sensors used for the measurements are 12 Dytran IEPE (mo00. del DY3191A1) piezoelectric accelerometers (Figure 2) with a built-in Faraday shield for electrostatic noise immunity. The sensors are hermetically sealed with a sensitivity of 10 V/g and operation range -51°C to 121°C. Dynamic signal analyzer DT9857E (Figure 3) with an output range of ±10 V to the sensors is connected by 15 m and 30

m polyurethane, 2-pin(F) signal cables. Measurements were performed continuously for 40 minutes in each set initially with a sampling frequency of fs=400 Hz but afterwards decimated to 10Hz.
Sensors are positioned at the corners of stair landings on each level, in both horizontal and vertical directions (x, y, and z), with a reference level at 33.49 m. Vibration measurements were collected in 13 different measurement setups, with each subsequent setup skipping 2 floors. 6 reference sensor locations and directions remained constant throughout all measurements. From SET1 to SET5 and from SET11 to SET14, 6 sensors were placed at the outer column corners, starting from the top of the tower and progressing downward in two-floor increments. For SET8 to SET9 reference sensors were placed combined with inner column corners. For SET10 reference sensors were placed and combined with outer column corner sensors (Figure 4, Figure 5).

Figure 1. Steel observation tower.

Figure 2. Dytran IEPE piezoelectric accelerometers.

Figure 3. Dynamic signal analyzer DT9857E connected by signal cables.

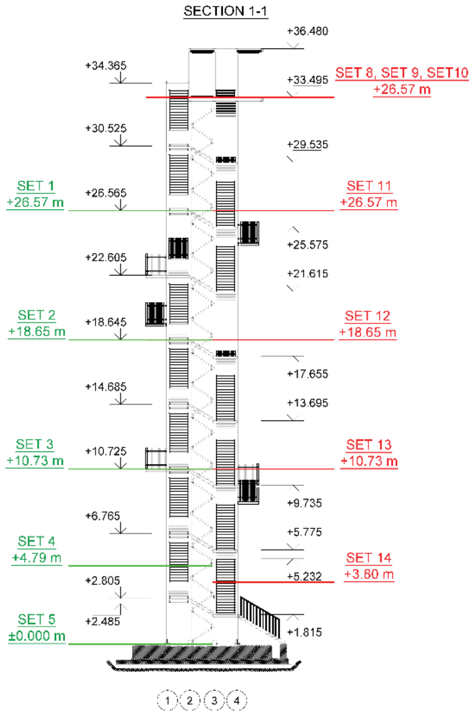
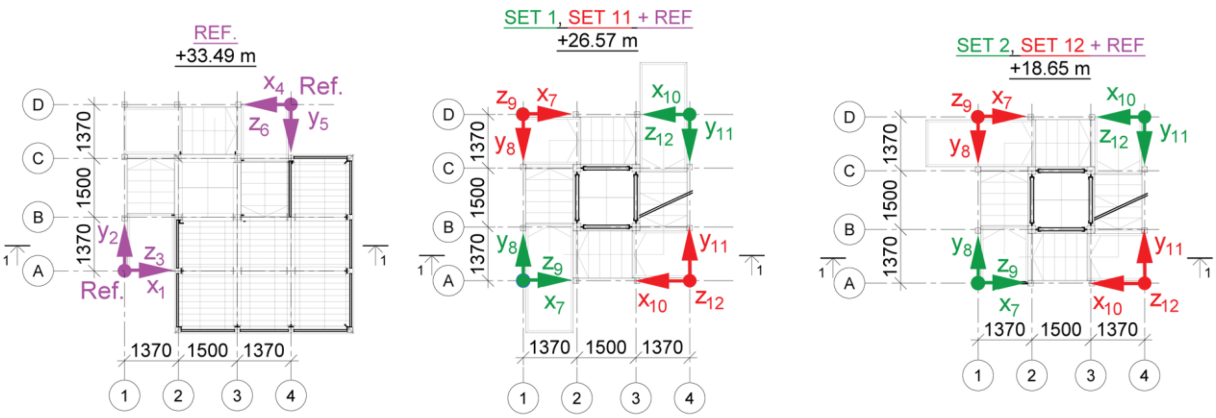


Figure 4. Steel observation tower section with measurement setup. For section 1-1 notation see plans in Figure 5.



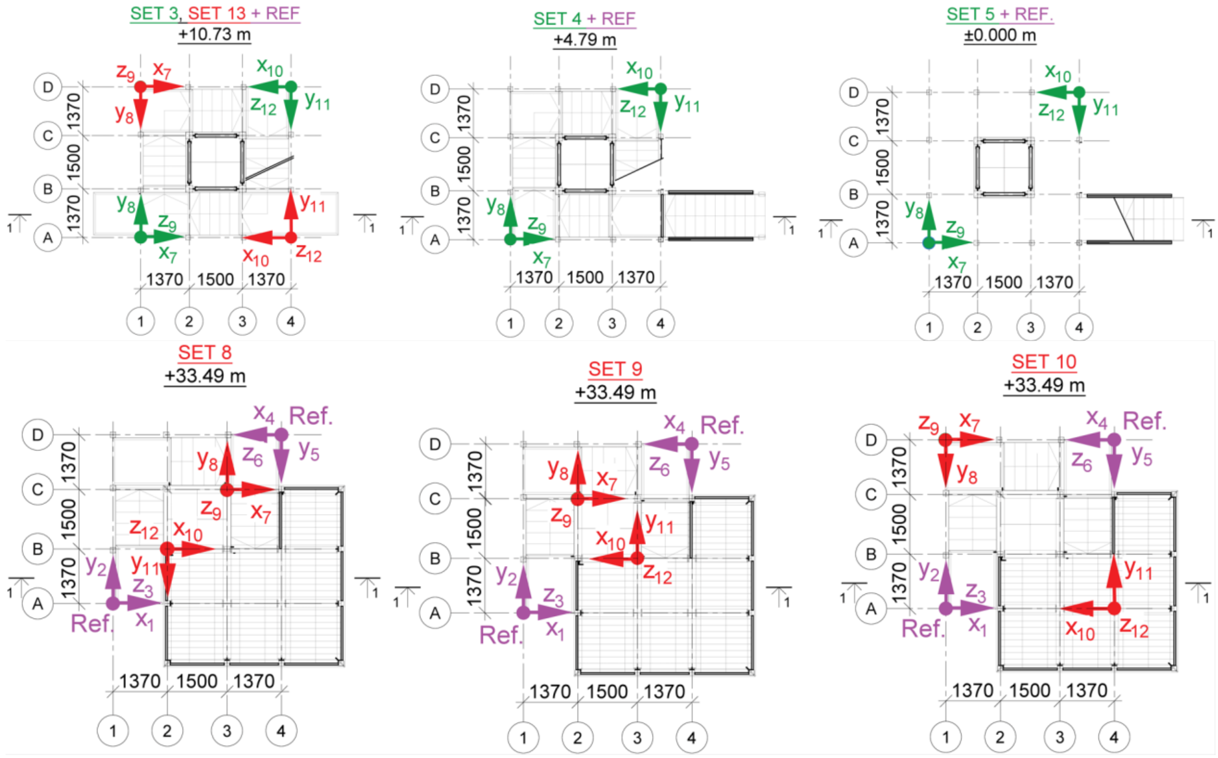


Figure 5. Vibration measurements location of sensors for each setup.

4. Case Study Application of Operational Modal Analysis

The commercial computer program ARTeMIS was used to perform modal identification of the structure. To simulate modal configurations, a 3D model of the structure with test points was created using structural geometry. The model includes the discretized locations of 26 measurement points of the observation tower.

Two different techniques were used for modal identification: the Frequency Domain Decomposition (FDD) and the Stochastic Subspace Identification (SSI) technique. The SSI method is a time-domain method that involves fitting a parametric model to time series recorded by sensors. It takes a time-history data matrix and performs a series of mathematical transformations, resulting in a set of mathematical models representing the system that generates the data. All modal parameters are exposed using a specific representation of the transfer function [11].

The FDD method roughly decomposes the system response density spectral matrix into an ensemble of SDOF systems using singular value decomposition (SVD) in the frequency range to exclude values that do not have a very specific relationship. The singular values are estimates of the spectral density of the systems, SDOF, and the singular vectors are mode shape estimates. The mode configuration is estimated from singular vectors by selecting the highest peaks of the responses [12].

The results of FDD peak collection method represent (Figure 6) structural modes and natural frequencies of the observation tower. In this case, the first 6 vibration modes are clearly identified as global modes due to the analysis of the experimental data. The singular values of spectral densities for all test setups are displayed in Figure 6, while the corresponding mode shapes can be found in Figure 7. In the latter figure, the blue arrows represent location and orientation of reference sensors and green arrows represent location and orientation of roving sensors.

A minimum signal-to-noise ratio (SNR) of 40 dB is often considered an adequate threshold for OMA. The singular value decomposition (SVD) diagram indicates that the experimental results were obtained with an SNR of 60 dB, which suggests that the measurements are of high quality and should yield reliable and accurate modal parameter estimates. The frequency of the 7th mode was also clearly identified; however, the preliminary FEM indicated that one measured level was missing to accurately capture the entire structural mode. Therefore, the further analyses focused only on the first 6 modes.

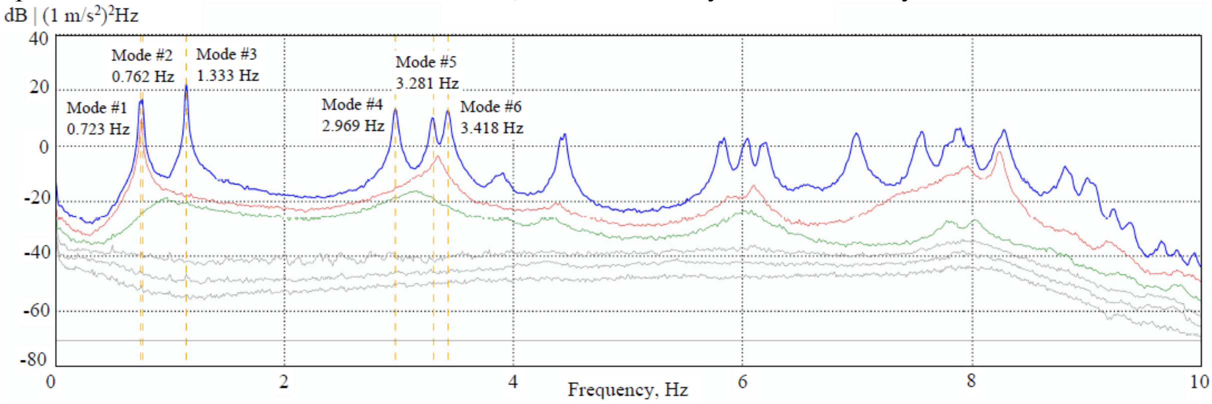
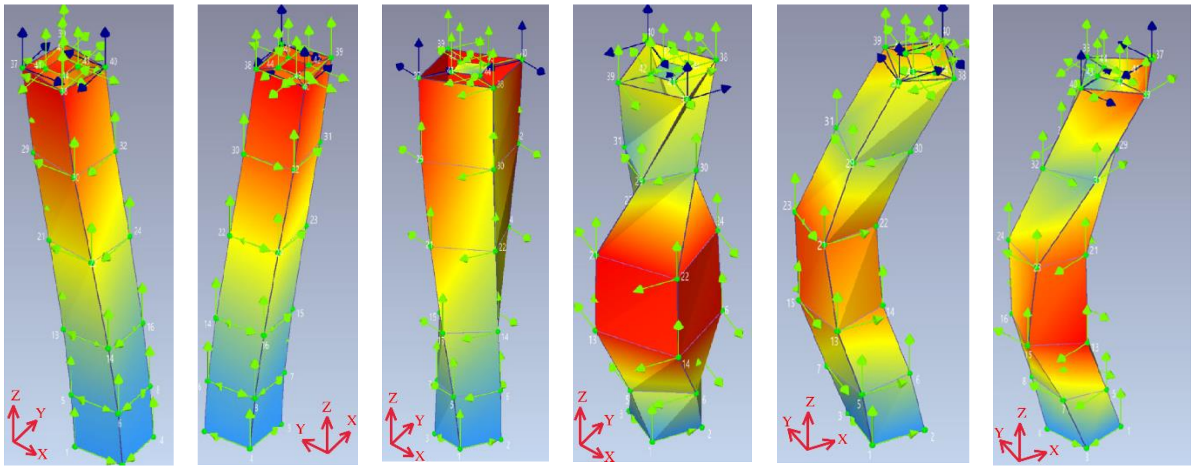


Figure 6. Singular Values of Spectral Densities of all test setups.



Mode #1	Mode #2	Mode #3	Mode#4	Mode #5	Mode#6
0.723 Hz	0.762 Hz	1.333 Hz	2.969 Hz	3.281 Hz	3.418 Hz

Figure 7. Mode shapes obtained with OMA.

Owing to the structure's geometry and the eccentric top platform, the first two identified modes are closely spaced bending modes with a torsional component. The third mode is a torsional mode, but it is interesting to note that the second bending modes (mode #5 and #6) follow after the second torsional mode (mode #4). Additionally, those bending modes are now more separated than the first two.
Upon comparing the frequency results from the simple tests conducted in 2012 [10], it is evident that the first and second frequencies have decreased over time, or possibly due to different environmental and operational conditions (EOV). The reduction is 4.9% for the first mode and 3.5% for the second mode.

5. Numerical model of the tower and model updating

The FEM of the tower is developed within the ANSYS commercial software implementing a 3D solid model created considering the real geometry of all structural and some non-structural elements. More in detail, columns, stair and landing beams, and roof beams are modelled with beam elements (BEAM188), whilst the bracing system of the inner lateral resisting core with link elements (LINK180) that simulate a pinned beam at both ends. Also, the steel grids used for landings, floors, and for the roof are modelled with shell elements (SHELL181 for the first two and SURF156 for the latter). Each single step of stairs is not modelled, but considered only in terms of mass since its contribution to the total stiffness of the

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ramps is deemed to be negligible. Furthermore, some non-structural elements are modelled as well; for instance, the lateral external wooden coatings and the roof wooden coatings are modelled with LINK180 elements. The wood elements placed on the outer faces of the steel columns and those located on the edges of the inner central core are considered only in terms of added messes. After a first tentative analysis, it was found that the model had to be further refined since its outcomes were significantly different from reality. For this reason, also the foundation system has been modelled. The concrete shallow foundation is modelled with solid elements (SOLID186), as well as the ground under the tower is modelled (SOLID187). The steel mechanical properties are assumed known and equal to the conventional ones (E = 210,000 MPa, γ = 78 kN/m^3), since steel members are produced under a scrupulous control. Similarly, the concrete mechanical properties of the basement are assumed constant since design details are available (E = 30,000 MPa, γ = 25 kN/m^3). Contrarily, the mass density of the wooden non-structural elements is assumed unknown, since general prescriptions are available about the adopted material, and the same for the soil stiffness, due to the high uncertainties in determining a unique value. Consequently, these parameters are considered as updating parameters in the following analyses. All the materials are assumed to be homogeneous, elastic, and isotropic. Some pictures relevant to the FEM are shown in Figure 8.

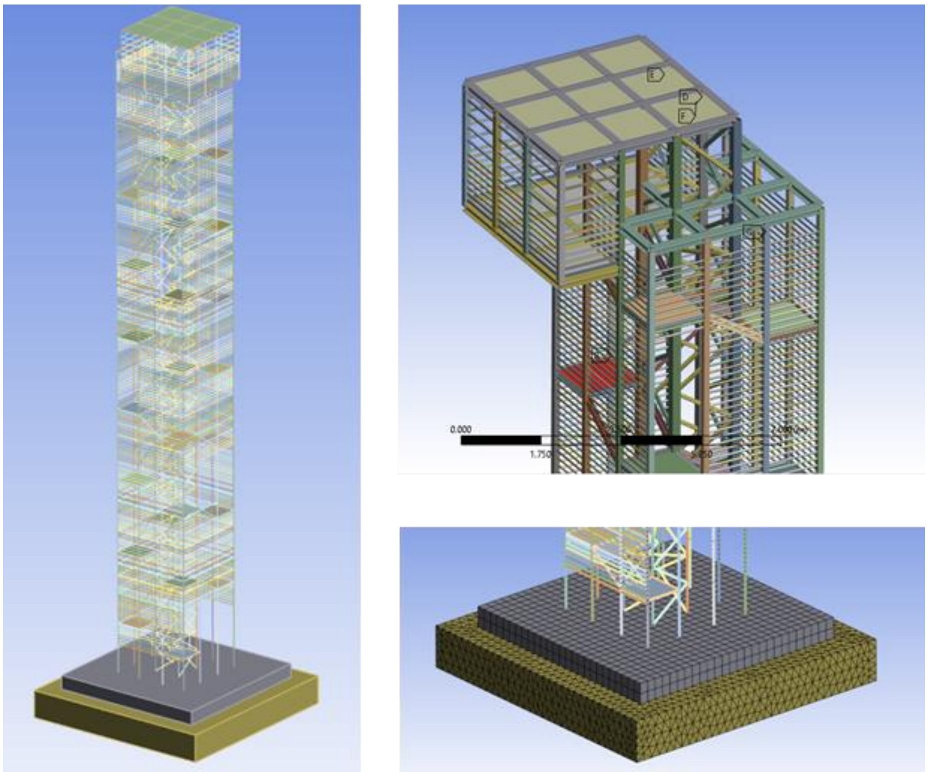

Figure 8. FEM of the tower developed in Ansys software.

The model updating process can be divided into two main procedures: the direct and indirect methods. The former requires recreating data from the real structure by small mass and stiffness matrix alterations, without accounting for changes in real physical parameters; the latter consists in altering the model's physical parameters until the FEM adequately reproduces the experimentally data and the gap between experimental and numerical modal parameters is reduced to an acceptable level. In this work, the second approach has been adopted, performing an iterative procedure based on the PSO algorithm. The basic principle of this algorithm is based on a population of simple individuals (called "particles") that evolve by interacting with each other and with the external environment. This algorithm simulates animal's social behaviour as insects, herds, birds, and fishes. The updating procedure can be considered as an optimization problem in which a set of parameters xi can be defined as optimal, and which is able to minimize (or maximize) some system characteristic. In this work, the minimization function is based on the comparison between numerical and experimental frequencies and mode shapes, the latter expressed by the MAC index. Specifically, an error function (EF) is defined as:

$$EF = \ln\left(1 + \left|\frac{f_{num}(x_i)-f_{exp}}{f_{exp}}\right| + (1 - MAC(x_i)\right) \quad (1)$$

The target of the algorithm is that of finding a set of parameters xi that allow for the minimization of the EF, namely.

Updated models can assist in the design of SHM systems by performing numerical studies that allow for the identification of the appropriate number and position of sensors in order to detect as many vibration modes as necessary (for example, by adopting optimal sensor placement processes). Furthermore, they may be used to analyse probable damage scenarios that account for structural member damage (and occasionally even failure), and their effects on the structure's performance are investigated. In addition, the structural behaviour obtained during the damage simulations can be used as a benchmark to fix appropriate thresholds during SHM.

6. Results of the model updating and discussion

The FEM of the tower is updated by changing its mechanical characteristics (masses and stiffnesses); the model updating is performed based on the comparison between experimental and numerical modal parameters. In this work, the frequencies and mode shapes of the first 6 modes are considered, while the damping is assumed deterministic (modal analyses). As previously stated, the two considered parameters that are updated are the wood mass density of the non-structural elements and the elastic modulus of the soil under the tower. Based on the collected information, plausible ranges of these parameters are fixed, and they are listed in Table1. Then, an iterative procedure is performed adopting the PSO algorithm with the target of minimizing the EF (Eq. (1)). At the end of this procedure, the values of the aforementioned updating parameters are 13 kN/m^3 and $55.8 \cdot 10^5$ Pa for the wood density and the soil elastic modulus, respectively. The evolution of these parameters during the iterations are reported in the graphs of Figure 9: the two parameters reach the convergence rapidly and then remain constant up to the end. In addition, the EF diagram is reported in Figure 9; as can be seen, the initial EF value is around 0.75, then it quickly decreases up to a constant value of about 0.51. The updating procedure ends when a reasonable number of iterations provide a constant EF value. In this work, a number of 10 iterations that provide the same EF values has been considered.

Table 1. Value ranges for the parameters considered in the FEM updating.

Parameter	Value ranges
Wood density	0.3 – 13 kN/m^3
Soil elastic modulus	$9.8 \cdot 10^5 - 1471.0 \cdot 10^5$ Pa

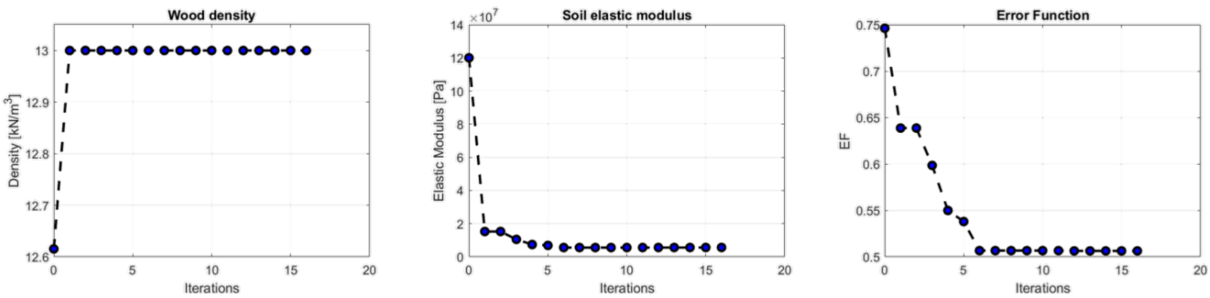

Figure 9. Evolution of the updating parameters and the EF during PSO iterations.

The numerical modal parameters of the tower's FEM after the updating procedure are compared with the relevant experimental ones. Specifically, the frequencies and mode shapes of the first 6 vibration modes (the same considered in the updating procedure) are compared. Concerning the frequency values, the results are reported in Table 2. As can be seen, the numerical frequencies are in good agreement with the experimental ones, especially those of the first 2 modes (the most important ones). Considering the mode shapes, the comparison with the experimental ones is reported in Figure 10 in term of MAC matrix. In this matrix, a black box indicates a perfect matching between two mode shapes (numerical and experimental), while the white box is the opposite. As can be observed, a very good matching is reached for the first 3 and for the 5th modes, while a lower agreement (but still satisfying) is achieved for the 4th and the 6th modes. Nevertheless, this can be due to the so-called spatial aliasing phenomena, being the 4th and the 6th higher modes. Indeed, the number of employed sensors and/or their position during the experimental campaign could be not adequate to compare in a correct way these two experimental mode shapes with the numerical ones. Summarizing, the comparison between experimental and numerical modal parameters shows a good agreement between them, proving that the updated model well represents the dynamic behaviour of the real tower. However, some improvements can be done, especially for the frequency of the 3rd, 4th, and 5th modes (which are mainly torsional), and for the mode shapes of the 4th and 6th modes. Generally, it can be asserted that the 4th and 6th modes require a general improvement, while for the 3rd, only the frequency value needs to be refined. It is worth noting that the wood density reached the upper bound during the updating procedure. Hence, further insights are necessary to get more information about the adopted materials and typology with the aim of in-depth investigating its influence in the dynamic behaviour of the tower.

Table 2. Comparison between experimental and numerical natural frequencies.

Mode	Frequency [Hz]		Difference [%]
	Experimental	Numerical	
1	0.72	0.74	2
2	0.76	0.76	0
3	1.13	1.39	23
4	2.97	3.49	17
5	3.28	3.57	9
6	3.42	4.16	21

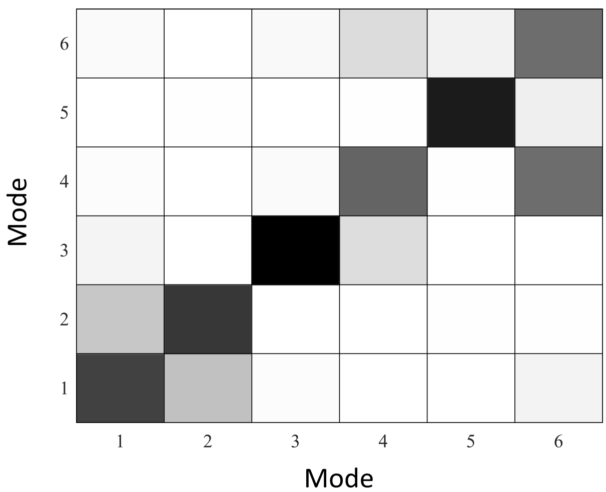

Figure 10. MAC matrix for the comparison of mode shapes.

7. Conclusions

In this paper, the experimental and numerical investigation of the steel observation Dzintaru tower located in Latvia has been presented. The experimental part dealt with AVTs recently performed on the structure and the relevant OMA that allows for the identification of the modal parameters that characterize the actual dynamic behaviour of the tower. Numerically, a detailed FEM has been developed within the Ansys environment. Then, the model was updated adopting and artificial intelligence algorithm (the so-called PSO); specifically the mechanical parameters of the wooden non-structural elements and of the soil under the tower were iteratively modified since the convergence between experimental and numerical modal parameters was reached.

Nevertheless, further numerical investigations are planned in the next future with the aim of achieving even better numerical results, in terms of good matching between experimental and numerical modal parameters. A sensitivity analysis will be performed on the actual model with the aim of understating which parameters (in addition to those considered in this work) mostly affect the dynamic behaviour of the tower. Then, the plausible ranges of the updating parameter values will be better analysed, especially that of the wood mass density, because the actual model seems require more wood mass to reach a convergence with the real dynamic behaviour, but this could be physically meaningless. Finally, the PSO algorithm will be applied again and considering all parameters obtained from the sensitivity analysis, with the aim of further reducing the EF.

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