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21 November 2026

Structural Health Monitoring of the Tower of Moletta: A Detailed Analysis of Structural Responses to Concert Activities

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Abstract.

The preservation of historical structures presents a multitude of challenges for those dedicated to maintaining our architectural heritage. Therefore, a meticulous study is essential for obtaining realistic outcomes. In this paper an 1145's masonry tower is analyzed. A site visit was instrumental in gathering point cloud data and geometric properties, which were subsequently utilized for drafting in REVIT software and exported to MIDAS GEN using BIM features. The Finite Element (FE) model was then calibrated by minimizing the discrepancy between its eigenfrequency and the results obtained from experimental tests using accelerograms. To verify the model's accuracy, the Modal Assurance Criterion (MAC) number was computed. We then calculated the acoustic pressure generated by a concert held near the tower and its impact on the tower in terms of stress and displacement for static loads, and acceleration, velocity, and displacement for dynamic models. By reversing the process, we determined the minimum acoustic pressure that could potentially damage the building. Furthermore, accelerometers placed at the base of the tower during the concert were used to observe the combined effects of jumping people and acoustic pressure.

The comparison is performed between the experimental methods of investigation: by ambient noise to obtain an equivalent global elastic modulus E of the tower. the loads came from concert (acoustic and ground motion) may cause damage on the tower because it may exceed the threshold allowed in Italian code.

Keywords: Moletta Tower, Historical Buildings, Masonry building, health monitoring, acoustic structure interaction, Model assurance criterion.

1 Introduction

Preservation of the architectural heritage is a fundamental issue in the cultural life of modern societies and the analysis of the historical masonry constructions is a challenging field of study because of lake of data about properties of its components and impossibility to do a destructive test, and especially for irregular texture of the masonry walls. Specially in Italy, A large portion of the historical cities are provided with masonry towers. Therefore, various numerical studies like linear and nonlinear analysis had been done on this type of building (masonry towers) [1-6], a lot of studies dynamic characterization to identify the young's modulus [4,6]. Numerous experimental tests for determining the mechanical properties of masonry walls [7-10].
The tower of moletta has been considered here as a case-study. Based on the data provided, different detailed and representative three-dimensional finite-element models of the tower are developed in this study (by using point cloud obtained by 3D laser scanner and autocad plans), to identify the best fitting geometrical and mechanical properties of the system. On the other hand, it also highlights some criticisms that may affect the identification process when ancient masonry towers are concerned. This matter is addressed in the conference. Fig.1 shows the location of the tower and concert's place. This study of the towers and historical building is not the first, there was a lot of studies had been done on the towers around Italy [11-13].



Figure 1. satellite pic taken from google map shows the location of the tower, and the concert's place.

Given the tower's location in a bustling area, we were interested in examining the impact of various human physical activities such as jumping, running, and walking. The vicinity of the tower (Circo Massimo) frequently hosts concerts, which occur periodically. This has raised concerns within the government about potential damage to the tower during these events.

1.1 Objectives of this study

In this work, the aim was to address the following objectives:

1. Modeling and Calibration: To accurately model the Tower of Moletta using Midas Gen and calibrate the finite element model based on experimental tests to get an accurate result in the following work.
2. Acoustic-structure interaction: To investigate the effects of acoustic pressure, produced by loudspeakers during a concert, on the structural integrity of the tower.
3. Physical Activity Impact Analysis: To assess the impact of physical activities of concert attendees, such as jumping, running, and walking, on the tower's stability by analyzing the acceleration at its base.
4. Structural Health Monitoring: To use the findings from the acoustic pressure and physical activity impact analyses to monitor and ensure the structural health of the Tower of Moletta.
5. Preservation and Safety: To provide insights and recommendations for the preservation of the tower and the safety of its visitors during large events like concerts.

1.2 Methodology

To achieve this purpose, a various 3D Finite element models created. some of them being more approximate and easier to apply, some others more rigorous but time-consuming.
Our study on the focused on the detailed comprehensive analysis of the Tower of Moletta. The process was multifaceted and involved several key steps:

1. First, constructing a model of the tower using Revit, which was then corrected in ROBOT Structural Analysis. The final model was exported to MIDAS GEN for further calculations.
2. computing the mechanical properties based on the characteristics of the masonry.
3. The model was calibrated in accordance with the results derived from experimental tests.
4. The Modal Assurance Criterion (MAC) value was calculated to verify the accuracy of the finite element model.
5. Assessment of the acoustic pressure and modeled it as both static and dynamic loads to understand its impact, particularly during concerts at Circo Massimo.
6. Accelerograms measured at the base of the tower were used to evaluate the effects of various concerts and physical activities such as jumping, running, and walking by the attendees.

This approach allowed us to gain a deeper understanding of the various factors influencing the structural integrity of the Tower of Moletta.

2 Case study

In this research, we will delve into the study of the Moletta Tower, a historical monument nestled in the border of Circo Massimo, Rome, Italy. This masonry edifice, constructed in the 12th century around 1145, stands tall at 13.8 meters. Interestingly, its walls extend a continuous 5 meters beneath the soil surface [14]. More details are shown in the Fig. 2, longitudinal and transversal cross sections in Fig 2b and Fig 2e. Fig 2c represents the 3D model on Revit. While Figs 2a and 2d represent the north-east facade in AutoCAD and ReCap respectively.

The tower occupies a surface area of 17 square meters (4.25m x 4m). The wall thickness is not uniform throughout the structure. The subterranean walls boast a thickness of 0.98 meters, while the above-ground walls vary between 0.5 and 0.75 meters. A noteworthy feature of the Moletta Tower is the upper part's inclination, which is tilted by 1.5° (it appears clearly in Fig 2a). in addition to non-existence of intermediate slabs.

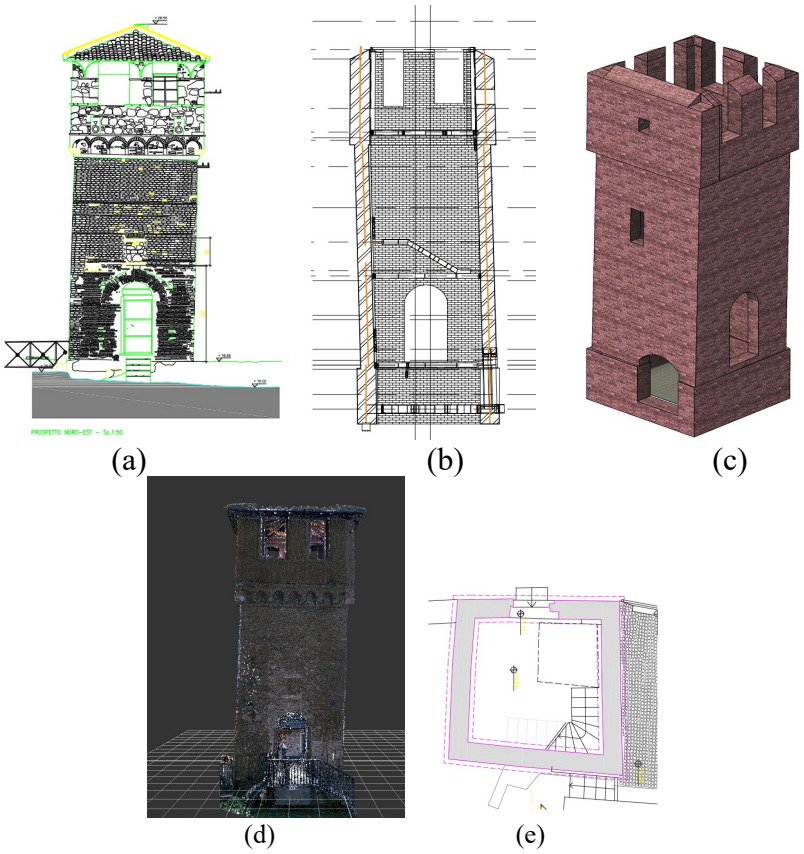


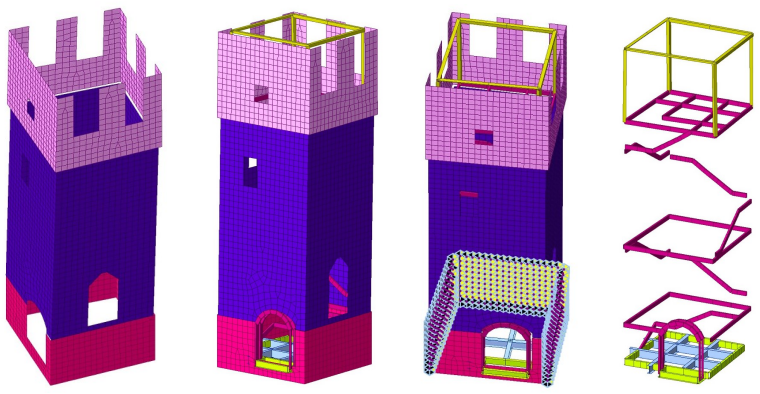
Figure 2. Drawings of the tower: (a) and (e) in AutoCAD, (b) and (c) in Revit and d represents point could in ReCap.

2.1 Numerical model

Various numerical models were considered to evaluate the structural behavior of the tower. These models were developed using different data. On one hand, point cloud data was utilized (Fig 2d), while on the other hand, computer-aided design (CAD) plans (as shown in Fig 2a and Fif 2c) were used to obtain geometrical properties.

The detailed geometrical models were created as a 3D model in building information modeling software (REVIT). Subsequently, the numerical models were exported to the commercial structural analysis software MIDAS GEN. Initially, the finite element model (FE model) was developed without considering steel beam stairs as shown in Fig 3a, but later it was included (see Fig 3b). In one case, the interaction with the adjacent soil was neglected, while in another case, soil-structure interaction was accounted for by introducing elastic support (referred to as a 'spring') (Fig 3c).

The initial material properties were computed by using the Masonry Quality Index method (MQI) after that those values compared with NTC 2018 values, homogeneous isotropic continuous materials with both linear elastic and non-linear behavior were assumed. Modal analysis, as well as linear and non-linear seismic analyses, were conducted to understand the tower's response to physical human activities (such as walking, jumping, and running around the tower). The masonry walls were modeled using triangular and rectangular mesh in 2D. At the base of the tower, all six degrees of freedom were fixed because the walls extend continuously until five meters below the ground surface [14]. Due to the importance of wall thickness, rigid links were added to connect the walls and beams. Additionally, rigid links were used to connect the walls to each other, as they were not in the same plane. Figure 3 provides further details of the finite element model.



(a) (b) (c) (d)

Figure 3. FE models in Midas Gen, walls and opening without stair beams in a, after including the stair beams b, c the soil structure interaction, distribution of the beams inside the tower in d.

2.2 Mechanical properties of elements

To get an accurate FE model, the correct inputs should be incorporated. In this section, we explain the criteria and approaches used to define the mechanical properties required for the analysis.

Mechanical properties of soil. According to previous works were done about soil in Circo Massimo [14-18], the mechanical properties of soil are presented in the Table 1:

Table 1. mechanical properties of soil around the tower.

γ[KG/m3]	ν	E [MPa]	G[MPa]	Vp[m/s]	Vs[m/s]
1800	0.42	418	147	835	286

Mechanical properties of soil and type of foundations allow us to compute the stiffness of the soil. FEMA 356 [19], explained how to calculate the soil stiffness for each translation and rotation directions [20]. After calculation of specific stiffness[N/m3], it should be distributed on all the area of the walls (by dividing the stiffness in all the nodes in FE model, which represent the wall).

Mechanical properties of steel staircase. The beams used in the tower, for the last retrofitting few years ago, have the following properties (see Tab 2):

Table 2. mechanical properties of beams.

γ (Kg/m3)	f_m (MPa)	E(GPa)	ν
7860	235	210	0.3

Mechanical properties of masonry. Due to the absence of precise data on the mechanical properties of masonry walls, an identification procedure was carried out using the Masonry Quality Index (MQI)[21]. The MQI assesses the structural behavior of historic masonry walls based on visual surveys of wall panels. By verifying compliance with construction standards, the MQI provides valuable information about mechanical characteristics and overall structural response. Notably, it considers parameters such as compression strength (fm), Young's modulus (E), shear modulus (G), and mass density (γ). Importantly, destructive testing is not feasible for historic masonry structures. The Eurocode 6 and NTC2018 are used to check that the values of these properties used in the updating model is in the range (Tab 3,4):

Table 3. mechanical properties of the masonry walls computed by MQI method.

γ (Kg/m3)	fm (MPa)	fs (KPa)	E(GPa)	G(GPa)
1900	3.54	73.4	1452.3	546

while the value of ν was directly obtained though the well-known relationship $G = E/(2(1 + \nu))$; the value of the Poisson ν ratio being calculated is approximately $\nu = 0.33$. A damping ratio $\xi = 5\%$ was assumed for all modes, which is a realistic value for masonry.

Mechanical properties according to Italian code NTC 2018. According to the visual investigations (see Fig 4), the classification of masonry walls led us to adopt two types mentioned in the Table 4 below from NTC 2018 [22]. The purpose of using this standard is to check that the mechanical properties are in the range of values proposed, after calibration of the FE model.

Table 4. mechanical properties of the masonry walls that taken from NTC 2018.

Type of masonry	Fm (N/cm2)	T (N/cm2)	E (N/mm2)	G (N/mm2)	W (kN/m3)
	Min-Max	Min-Max	Min-Max	Min-Max	
Masonry with rough-hewn ashlars, with facing of limited thickness and internal core	200 300	3.5 5.1	1020 1440	340 480	20
Solid brick and lime mortar masonry	240 400	6.0 9.2	1200 1800	400 600	18



Figure 4. Picture taken during the visual investigation, which provides an information about the texture of masonry walls

2.3 Calibration method and MAC number

The identification process was carried out by comparing the three first natural frequencies of the numerical models with the experimental ones, the latter being taken from the results of the experimental tests carried out by Politecnico di Torino [23] in 2022. The natural frequencies obtained with the output-only identification technique which is Frequency Domain Decomposition (FDD).

A step-by-step iterative procedure was performed to minimize the discrepancy between the experimental first three frequencies and the numerical ones. The pro-

cedure was carried by varying the values of the elastic modulus E within the ranges given in NTC2018.

Identification of the best-fitting numerical model. During the calibration, Modal dynamic analyses were carried out assuming linear elastic behavior of materials, because it was expected that the experimental excitation provided to the structure during the tests would have to be well in the elastic range of the response.

The identification process was carried out by comparing the first natural frequencies of the numerical models with the experimental ones [24]. This operation may significantly help to obtain a reliable numerical model of an existing structure when lack of data and incoherence between sources occur.

After numerous of analysis that mentioned previously, the most realistic model was shown in Fig 3c. This model was more complicated; hence, it is too close to the real building. Therefore, we adopt this model to do the following work.

MAC number. The Modal Assurance Criterion (MAC) is a general criterion for assessing the degree of consistency (correlation) between two vectors, both real and complex [25]. MAC can be used to compare two modes shape vectors Ur and Uq. The numerical model eigenvectors are taken from the nodes (in FEM) which represent the sensor places with respecting the order of its components.

The primary method that has historically been used to validate an experimental model is the weighted orthogonality check comparing measured modal vectors and numerical ones [24]. The correlation indicator for the eigenvector of modes q and r is mathematically expressed as “Eq. (1)”:

$$MAC\ (q,r) = \frac{\left|\{\varphi_r\}^T\{\varphi_q\}\right|^2}{(\{\varphi_r\}^T\{\varphi_r\})(\{\varphi_q\}^T\{\varphi_q\})} \tag{1}$$

modal assurance criterion is used to check the identification of experimental (r) and numerical (q) for each mode shape. The table 5 represents the values of MAC number:

Table 5. Model assurance criterion for each mode and each direction.

Modes	First	Second	Third
MAC values	0.87	0.85	0.76

Concerning the obtained mode shapes, a modal assurance criterion analysis was carried out between the obtained numerical mode shapes and the correspondent experimental ones. MAC values were calculated among the modal residues extracted from the experimental analysis and the ones given by the numerical model in the same points where measurements were taken.

As evident in Table 10, the MAC coefficient is very good for the first three modes, indicating a high degree of correlation between the experimental and numerical mode shapes

Some insights about the model identification of masonry tower. The comparison between the first two numerical and experimental frequencies in the main

directions. The corresponding fundamental periods are (0.34 s and 0.31 respectively. As far as the 1st bending modes and the torsion are concerned, however, a discrepancy between numerical and experimental frequencies was found 1st torsional mode. Indeed, the model appears to be stiffer than the actual structure for this last mode. This discrepancy might be due to several factors and overcome in different ways. Some insights are drawn below:
1. Among the possible causes of the lack of the perfect identification, the heterogeneity of the materials along the tower's height and the lack of integrity of parts of the walls (due to widespread presence of cracks).
2. Accounting for anisotropy and inhomogeneity of the material along the height could lead to a better identification of the frequencies higher than the first three, also because of different textures in the tower.
3. A full tower + soil model would be even implemented to account for the dynamic interaction between them.

2.4 Acoustic pressure

Mainly there are two methods to measure the acoustic pressure. The first is to measure the intensity of acoustic power during applying the sound, the second method is the easiest and the most recognized. This last is based on the calculation of sound pressure level (SPL) and its attenuation with distance.
according to data received from organizers of the concert in Circo Massimo, the characteristics and potions of the loudspeakers are represented in Tab 6 and Fig 5:

Table 6. Technical data of loudspeakers used in concerts organized in Circo Massimo

Range of frequency	Maximum continuous SPL (dB)	Maximum Peak SPL (dB)
40 Hz to 180 Hz	138	144

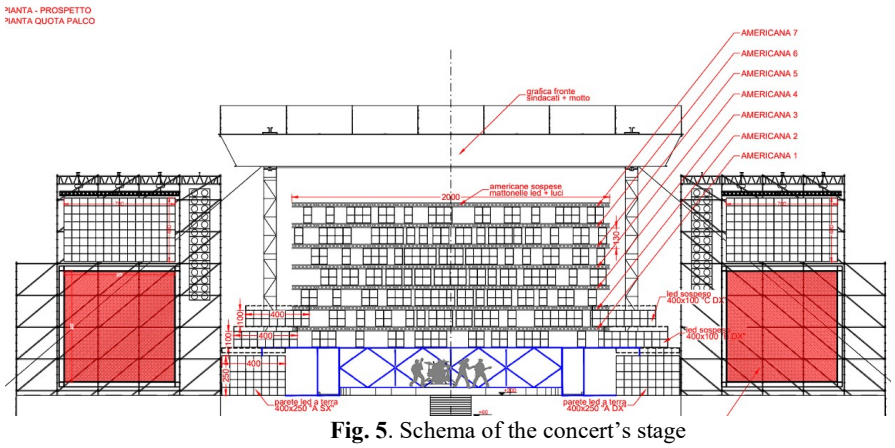


Fig. 5. Schema of the concert's stage

Calculation of the acoustic pressure. The procedure to calculate the acoustic pressure is as following [26]:

It's recognized that the reference pressure in the quiet places is: P0 = 20μPa.

1. Measuring the sound pressure level (SPL1) in the source: by using the sonometer and calculate the peak pressure by the following formula “Eq. (2)”:

$$P_{Peak} = P_0 * 10^{SPL1/20} \quad (2)$$

2. Using this relation of attenuation between two sound pressure levels (in the source and new position 2“see Eq. (3)”:

$$SPL_2 - SPL_1 = 20 \text{ Log } (r1/r2) \quad (3)$$

while r1 and r2 are the distances in meters from source.

3. calculate the pressure in our position:

After calculating the sound pressure level after attenuation, we use the following formula “Eq. (4)” to compute the pressure:

$$P_2 = P_0 * 10^{SPL2/20} \quad (4)$$

During this study, there was two approaches adopted to model acoustic structure interaction. each approach is based on some criteria.

Static uniform pressure. while the intensity of sound is constant (for the constant sound pressure level).

Dynamic impulsive uniform pressure. when the DJ increases and decreases the volume of sound (the unfavorable case was assumed which is resonance phenomenal). This load was considered as semi-sine model.

According to data obtained from the commission responsible on the organization of the concert, the peak value of the sound pressure level was around 150 dB in the concert (the peak of SPL is 144 dB in addition of the sound realized by people) for our case we took 155 dB as peak sound pressure level.

Tab 7 represents the acoustic pressure in dB and Pa in the source (concert) and after attenuation (on the tower), the Fig 6 represents the measurement places of the acoustic pressure:

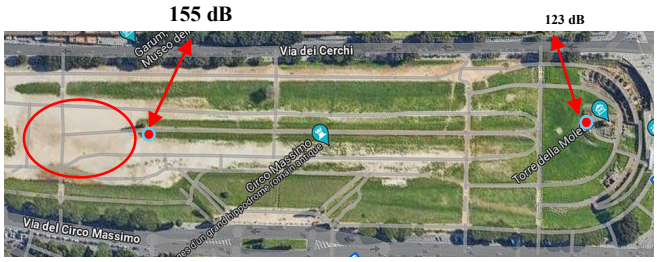


Fig. 6. Values of the acoustic pressure computed in the concert and the tower

Table 7. Peak pressure in the concert's place and the tower

	Pressure in source	Pressure on tower's wall
Pressure in dB	155	123
Pressure in Pa	1125	30

The characteristics of dynamic acoustic pressure are represented in Tab 8 as following:

Table 8. Dynamic characteristics for acoustic pressure

Intensity (Pa)	Impact time (s)	Duration of application(s)	Frequency (Hz)
100	0.2	30	3

2.5 Accelerograms registered in the bottom of the tower

For the accelerograms registered in the bottom of the tower we choose three different times, which are as following:

- In a normal day, for ambient vibrations.
- During the acoustic tests (when the loudspeakers were activated at the same time)
- During the concert (when there are 4500 people jumping, moving, dancing, walking and running)

In the beginning, a comparison has been done to see if the vibrations due to acoustic wave and physical activities reach to the tower or not. The peak ground acceleration of the registration in the bottom of the tower are shown in the Tab 9:

Table 9. Peak ground acceleration registered in the bottom of the tower

Acceleration [cm/s²]	During Regular day	During Acoustic tests	During the concert
X direction	0.16	3.47	7.78
Y direction	0.08	1.16	1.91

The results showed that the biggest PGA is in the concert, when there was a combination between human activities and acoustic wave, these vibration propagations transfer throughout the earth. By comparing the PGA in both directions, we observe that in the X-direction is bigger because it's the direction of propagation of wave in soil.

2.6 Limits allowable in different codes for historical buildings

Various standards and codes [27-31] have already put the limits on velocity of vibrations, the threshold of these limits are summarized in Tab 10:

Table 10. Limits velocity of particles allowable in different standards

Standards and codes	Structure type	Limits values (mm/s)
UNI 9916	Historical buildings	2.5
Germany DIN4150-3:1999 (Germany 1999)	Ancient architecture	3–10
UK BS7385-2 (UK 1993)	Ancient architecture	7.5
Switzerland 6.10SN640312:1992(Swiss 1992)	Old and poorly maintained buildings	2

3 Results and Discussion

3.1 Modal analysis

Dynamic testing is increasingly important as a valuable tool that provides a validation to refine and eventually update numerical simulation models and gives information about the response to an external stimulus (e.g., ground motion).

Numerical and experimental frequencies are comparatively presented in Tab 11. The calibrated FE model of the tower is attained by altering in elasticity modulus of masonry walls. Before calibrating, differences between numerical and experimental frequencies of first, second and third modes were 12.2%,19.6 % and 18.7 %, respectively. Modulus of elasticity of brick masonry wall decreased from 1452 MPa to 1120 MPa. After updating, differences between numerical and experimental frequencies of first, second and third modes were 0.3%, 3% and 5.2 %, respectively. Calibrated and non-calibrated first three numerical and experimental natural frequency values of the tower are presented in Tab 11. As results of calibrating process, it was observed that numerical and experimental results were quite close to each other's.

Table 11. The dynamic characteristics obtained before and after the FEM calibration compared with experiments frequencies.

Modes	Numerical frequencies FEM (Hz)		Experimental frequencies (Hz)	Maximum difference between frequencies (%)	
	Before calibration	After calibration		Before calibration	After calibration
1	3.31	2.96	2.95	12.2	0.3
2	3.85	3.12	3.22	19.6	3.0
3	6.99	6.2	5.89	18.7	5.2

Mode shapes of the tower. the figure 7 represents the first five mode shapes of the updated FE model:

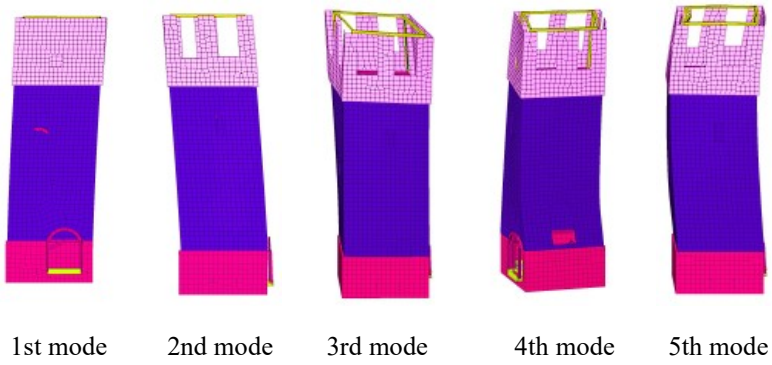


Fig. 7. First five mode shapes of the tower.

3.2 Response of the tower to acoustic pressure

Response to static acoustic pressure. The Table 12 represents the stresses induced in the bottom, after applying the static model of acoustic pressure:

Table 12. Internal loads induced after applying static acoustic load.

Max shear stress [KN/m^2]	Max compression stress [KN/m^2]
5.23	7.78

Response to dynamic acoustic pressure. The Tables 13,14 summarize the results like the maximum displacement, velocity and acceleration of particles in the top, in addition to shear and compression stresses that obtained in FE model after applying the dynamic acoustic pressure, respectively:

Table 13. Response of the tower due to dynamic acoustic load.

Max displacement [mm]	Max velocity [mm/s]	Max acceleration[cm/s2]
0.12	5.3	8.71

Table 14. Internal loads induced after applying dynamic acoustic load.

Max shear stress [KN/m^2]	Max compression stress [KN/m^2]
27.8	52.2

According to UNI 9916 [27], the maximum velocity of particles for historical buildings is 2.5[mm/s], this value was exceeded when we applied SPL of 155 dB during the concert this may cause damage in the walls. Fig 8 and Tab 15 represent the limit acoustic pressure in dB and Pa in the source (concert) and after attenuation (on the tower), which cause this vibration in the top.

After comparison between the stresses induced by static and dynamic acoustic pressure, it's noticed that the effect of the second is much more than the first.

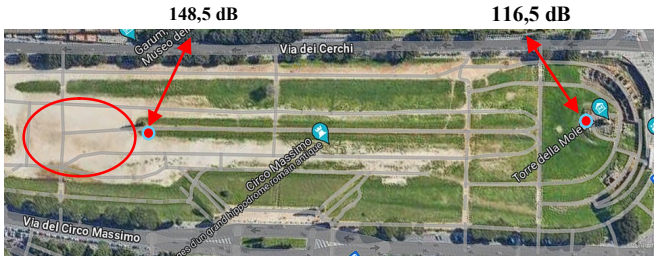


Fig. 8. The maximum values of SPL that should not be exceeded.

Table 15. Maximum acoustic pressure and sound pressure level should be allowed in the concerts organized in Circo Massimo.

	Pressure in source	Pressure on tower's wall
Pressure in dB	148.5	116.5
Pressure in Pa	540	13.4

3.3 Vibrations induced by ground motion registered by accelerometers in the bottom of the tower

Ambient vibrations. Tabs 16 and 17 represent the response of tower in a normal day:

Table 16. Response of the tower in a regular day.

Max displacement [mm]	Max velocity [mm/s]	Max acceleration[cm/s2]
0.015	0.43	2.16

Table 17. Internal loads induced in a regular day.

Max shear stress [KN/m^2]	Max compression stress [KN/m^2]
1.45	2.55

During the acoustic tests (when the loudspeakers were activated at same time). Dyring the acoustic test that done on April 30th ,2024, the response of the tower is shown in Tabs 18 and 19:

Table 18. Response of the tower due to dynamic acoustic load.

Max displacement [mm]	Max velocity [mm/s]	Max acceleration[cm/s2]
0.35	5.9	13.5

Table 19. Internal loads induced after applying dynamic acoustic load.

Max shear stress [KN/m^2]	Max compression stress [KN/m^2]
31.9	56.4

During the concerts. The assessment of response of the tower was performed for the July 17th,2023 and May 1st,2024 concerts.

Response of the tower during the May 1st,2024 concert. Tabs 20 and Tab 221 represent the peak vibrations in the top of the tower and maximum stresses in the button of the tower in concert of May 1st,2024.

Table 20. Response of the tower during the May 1st,2024 concert.

Max displacement [mm]	Max velocity [mm/s]	Max acceleration[cm/s2]
0.64	11.5	25.74

Table 21. Internal loads induced during the May 1st,2024 concert.

Max shear stress [KN/m^2]	Max compression stress [KN/m^2]
49.05	90.2

Response of the tower during the July 17th,2023 concert. Tabs 22 and Tab 23 represent the peak vibrations in the top of the tower and maximum stresses in the button of the tower in concert of July 17th,2023.

Table 22. Response of the tower during the July 17th,2023 concert.

Max displacement [mm]	Max velocity [mm/s]	Max acceleration[cm/s2]
0.52	9.96	20.33

Table 23. Internal loads induced during the July 17th,2023 concert.

Max shear stress [KN/m^2]	Max compression stress [KN/m^2]
53.63	80.5

In assessing the tower's response to acceleration, velocity, and displacement under various conditions - a typical day, during acoustic tests, and amidst a concert - we noted distinct influences. The concert's human activities and the acoustic tests' sound waves were found to impact the tower, transmitted via soil and air respectively. This demonstrates the propagation of waves through these mediums. It was also observed that these values surpassed the acceptable velocity limit of 2.5 mm/s in UNI 9916 for historical structures. This could potentially lead to damage to the tower if such instances occur frequently.

The tower was excited during the concerts much more than other periods because of high density of attendees in the adjacent area.

3.4 Comparison between numerical and experimental results

In this step we want to compare the peak velocities of particles in the top of the tower during those concerts computed by FE Model and Trapeze integration method of accelerograms registered by sensors in the top. On the same way of calculation of MAC number in section 2.3, we chose the node that represents the position of sensor in the top. Tabs 24,25 Represent the numerical and semi-

experimental values.it shows clearly that, in almost cases, the excitations exceed the thresholds estimated by different standards. The results obtained were too close to each other. This confirms the validity of Finite Element model performed. And the efficiency of FE software used in this research for the masonry buildings.

Table 24. maximum velocity in the top of the tower computed during the concerts in X-direction.

	July 17th,2023	May 1st,2024
velocity of FEM (mm/s)	6.53	7.22
velocity of experimental tests (mm/s)	4.16	7.75

Table 25. maximum velocity in the top of the tower computed during the concerts in Y-direction.

	July 17th,2023	May 1st,2024
velocity of FEM (mm/s)	3.21	4.9
velocity of experimental tests (mm/s)	1.5	4.53

4 Conclusion and perspectives

4.1 Conclusion and recommendation

In conclusion, this comprehensive study provides a detailed analysis of the potential risks to an 1145's masonry tower from various sources, including acoustic pressure from a nearby concert and the combined effects of jumping people and acoustic pressure. The research utilizes advanced techniques such as point cloud data, REVIT software, MIDAS GEN, Robot Structural Analysis, and BIM features to create a Finite Element (FE) model of the tower. This model is then calibrated and verified using experimental tests and the Modal Assurance Criterion (MAC) number.

The study reveals that the loads from the concert exceeded the threshold allowed in the Italian code, potentially causing damage to the tower. By reversing the process, the study also identifies the minimum acoustic pressure that could potentially cause damage to the tower.

These findings underscore the importance of meticulous study and the use of advanced technological tools in understanding and mitigating potential risks to historical structures. They highlight the need for careful management of events near such structures to prevent damage and ensure their long-term preservation.

Based on these findings, the following recommendations can be made:

- Event Management: Organizers of events near historical structures should be made aware of the potential risks and should take necessary precautions to prevent damage. This includes controlling the volume of music and limiting the crowd density.

- Structural Monitoring: Regular monitoring of the structural health of historical buildings should be carried out. This can help in early detection of any potential damage and timely intervention.
- Policy Implementation: Policies should be implemented to regulate activities near historical structures. These policies should be based on scientific research and should consider factors such as acoustic pressure and crowd density.
- Further Research: More research should be conducted to understand the impact of different types of loads on historical structures. This can help in developing more effective strategies for their preservation.
- Public Awareness: Efforts should be made to increase public awareness about the importance of preserving historical structures and the potential risks they face.

In conclusion, Operational Modal Testing can be used to safely calibrate initial finite element models of historical masonry buildings. this research offers valuable insights into the preservation of historical structures. It highlights the importance of meticulous study and the use of advanced technological tools in understanding and mitigating potential risks. The findings underscore the need for careful management of events near such structures to prevent damage and ensure their long-term preservation. The study also opens avenues for further research into the impact of different types of loads on historical structures.

4.2 **perspectives**

In the following works we propose to study the effect of jumping people during the concerts organized in Circo Massimo to see its effects on the tower and any damages probable. Therefore, the next step is studying the wave propagation in the soil from the concert area to moletta tower. in addition to nonlinear time history analysis supposed to assess the capacity of the tower, and what is the minimum seismic acceleration which could cause damages.

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