JOURNAL OF GREEN SCIENCE AND TECHNOLOGY

IDENTIFICATION OF TRAFFIC-INDUCED SOIL–STRUCTURE RESONANCE IN HERITAGE BUILDINGS AND MITIGATION USING OPERATIONAL MODAL ANALYSIS

Heri Khoeri[1*], Dini Sofiana[2]
[1*])Universitas Tama Jagakarsa, Jakarta.
Corresponding Author's Email : hkhoeri@hesa.co.id
[2]) Postgraduate Program, Faculty of Engineering, University of Indonesia.

ABSTRACT

Excessive vibrations in heritage buildings located near heavy traffic often cause occupant discomfort despite adequate structural capacity. This study investigates the vibration mechanism and proposes a mitigation strategy with minimal intervention using Operational Modal Analysis (OMA) and Stochastic Subspace Identification (SSI). The identified fundamental frequency of the building is 3.09 Hz, which closely matches the dominant traffic-induced ground frequency (3.10 Hz), yielding a frequency ratio $r \approx 1.0$ and confirming soil–structure resonance as the governing mechanism. This resonance amplifies responses, particularly at intermediate floors. To mitigate the effect, a Multiple Tuned Mass Damper (MTMD) system with a total mass of 4 tons ($\approx 0.54\%$ of structural mass) is proposed. The MTMD effectively reduces dynamic amplification while remaining non-invasive, making it suitable for heritage structures. The results demonstrate that small-mass MTMD provides a practical and efficient vibration mitigation solution.

Keyword: damper, heritage, modal, resonance, vibration.

1. INTRODUCTION

Buildings located near roads with heavy vehicular traffic often experience vibrations that are perceived by occupants, particularly on certain floors. In many cases, these vibrations are not caused by insufficient structural strength, but rather by the dynamic response of the building to repeated external excitations. [1], [2]. This phenomenon often leads to a perception of insecurity and reduced comfort, even though the building remains structurally safe. One of the primary mechanisms responsible for excessive vibrations is resonance, which occurs when the excitation frequency approaches or matches the natural frequency of the structure. Resonance can result in significant amplification of structural responses, even when the excitation amplitude is relatively small. [1]. This condition becomes increasingly critical in existing buildings and heritage structures, where conventional structural interventions are often constrained by conservation requirements. Therefore, this study focuses on identifying resonance as the primary cause of excessive vibrations and on developing effective mitigation recommendations that are technically sound and require minimal intervention to the main structural elements.

Excessive vibrations are commonly associated with resonance, which occurs when the excitation frequency approaches the structural natural frequency, resulting in significant dynamic amplification [1], [2]. In buildings founded on soil, traffic-induced ground motions may interact with structural modes through soil–structure interaction effects [3]. For existing and heritage buildings, non-invasive identification techniques such as Operational Modal Analysis (OMA) are preferred, as controlled excitation is often impractical [4], [5]. Soil, as a wave propagation medium, exhibits a dominant frequency that depends on its dynamic properties. When the dominant soil frequency f_s induced by traffic approaches the natural frequency of the building f_n, soil–structure resonance occurs, leading to

amplification of the building response even when the amplitude of ground vibrations is relatively small This phenomenon is part of soil–structure interaction and is often a primary cause of excessive vibrations in buildings located near roadways or railway lines [2], [3].

This study aims to identify traffic-induced soil–structure resonance as the primary cause of excessive vibrations using Operational Modal Analysis and to develop an effective mitigation strategy with minimal structural intervention. The main contribution of this work lies in integrating field-based modal identification with a quantitative resonance assessment and small-mass Multiple Tuned Mass Damper (MTMD) design, providing a practical and conservation-compatible solution for heritage buildings.

2. RESEARCH METHOD

The overall research workflow consists of five main stages: field vibration measurement, data acquisition and pre-processing, operational modal identification, resonance assessment, and mitigation design using MTMD. A simplified flowchart of the adopted methodology is presented in Figure X to clarify the sequential procedure.
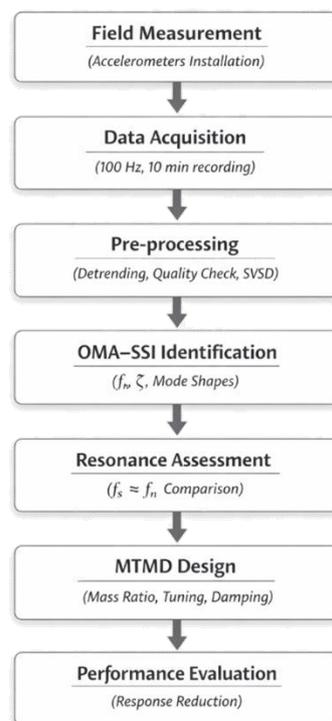

Figure 1. Flowchart of the research methodology, illustrating the sequence from field measurement and modal identification to resonance assessment and MTMD mitigation design

2.1. Instrumentation, Measurement Setup and Data Acquisition

Vibration measurements were carried out using three-dimensional accelerometers installed at the first, second, and third floors of the building (Figure 1). The measurements were conducted over a 10-minute period under normal operational conditions, without any artificial excitation.

The basement at an elevation of -3.37 m is assumed to be stationary (approaching base/foundation motion = 0) for reference and comparison of relative responses between floors [12]. Such a vertically distributed sensor layout is commonly used to capture global translational modes (in the X and Y directions) and to indicate possible torsional behavior. With one measurement point per floor, torsional modes may be inferred from non-collinear lateral response components; however, clear separation of pure torsional modes generally requires multiple measurement points on the same floor. The use of

three-dimensional accelerometers allows evaluation of modal behavior in two horizontal directions as well as the vertical direction, although vertical modes in buildings typically occur at higher frequencies and exhibit smaller amplitudes under ambient excitation [13].
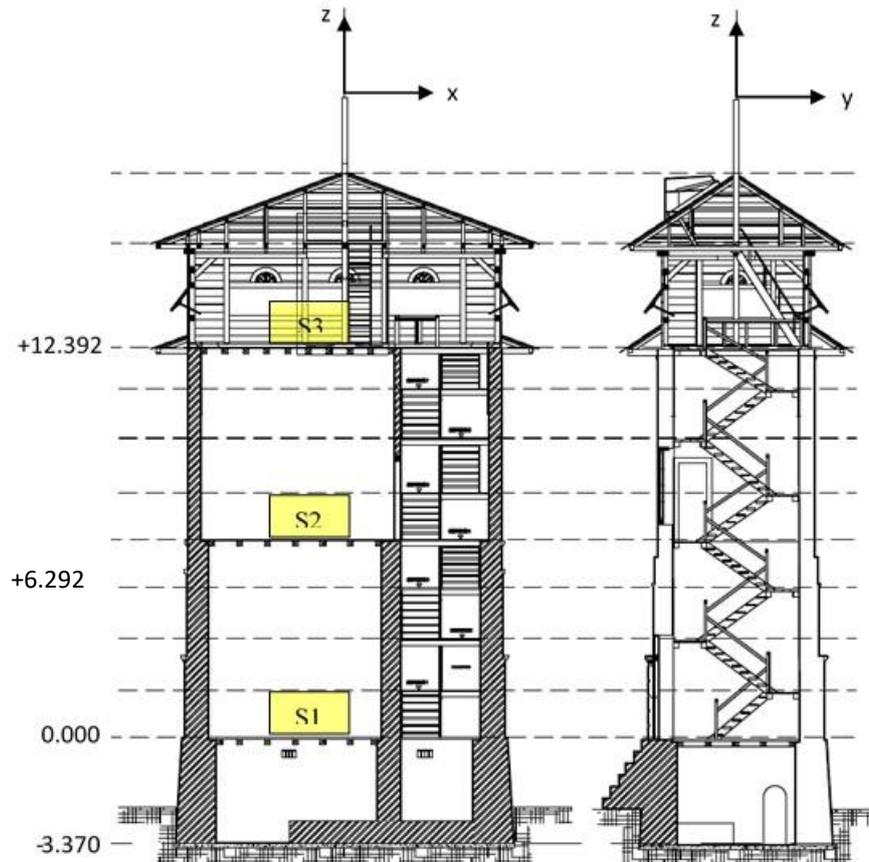
Figure 2. Accelerometer Placement

The sampling rate used was:
- Sampling frequency f_s = 100 Hz, resulting in a Nyquist frequency f_N = 50 Hz (which corresponds to a theoretical analysis range of 0–50 Hz).
- Measurement duration T =10 minutes = 600 seconds, yielding a total number of samples N = fs·T = 100·600 =60,000 samples per channel.
- The fundamental frequency resolution (first-order approximation) is given by $\Delta f \approx 1/T$ = 1/600 = 0,00167 Hz

The specifications of the sensors and data logger used in this study are presented in **Table 1**.

Table 1. The specifications of the sensors and data logger

No	Spesifications	Photo alat
1	**Accelerometer Sensor 3-Axis** - Sensor MPU 9025 Gyro + *Accelerometer* - MEMS 3-*axis accelerometer* and 3-*axis* gyroscope values combined - Power Supply = 3-5V - Communication = I2C protocol - Built-in 16-bit ADC provides high accuracy - Communication using NRF24l0 N (2, 4 ghz), Rs485	

No	Spesifications	Photo alat
2	**Data Logger RS485** - Clock speed = 16 MHz - Memory = up to 64 gb - Komunikasi data = NRF24l0 N 2,4 Ghz, Rs485, USB 3.0 - Pipe = up to 6 reciever	

2.2. Data Pre-processing

Prior to OMA, the raw acceleration time histories were pre-processed to improve data quality and ensure the assumptions required by SSI. First, the mean value was removed and detrending was applied to eliminate low-frequency drift. Second, the signals were visually inspected to exclude segments with obvious non-stationary disturbances unrelated to traffic-induced excitation. Third, the channels were checked for clipping and abnormal spikes; if present, short segments were removed or replaced using standard outlier handling to avoid bias in spectral estimates. Finally, the pre-processed signals were used to compute Singular Values of Spectral Densities (SVSD) for preliminary frequency-range screening, followed by modal identification using SSI-UPC. In the SSI implementation, channel projection was enabled as permitted in the software to reduce noise sensitivity and improve the numerical conditioning of the identification process.

2.3. Operational Modal Analysis (OMA)

Operational Modal Analysis is a method for identifying modal parameters based on structural response data (*output-only*), without requiring measurements of the excitation forces [6], [7]. Modal parameters, including natural frequencies, damping ratios, and mode shapes, can be identified from ambient vibration data, making this method particularly suitable for existing structures where controlled excitation testing is not feasible [7], [8].

The identified modal parameters include:
- Natural frequency f_n,
- Damping ratio ζ_n,
- Mode shape ϕ_n.

In the Stochastic Subspace Identification (SSI) method, the system model is expressed in discrete state-space form [7], [9]:

$$\mathbf{x}_{k+1} = \mathbf{A}\mathbf{x}_k + \mathbf{w}_k$$
$$\mathbf{y}_k = \mathbf{C}\mathbf{x}_k + \mathbf{v}_k$$

(1)

Where **A** and **C** are the system matrices, and $\mathbf{w}_k$ dan $\mathbf{v}_k$ represent process disturbances and measurement noise, respectively.

Modal parameter identification was performed using modal analysis software implementing the Stochastic Subspace Identification (SSI) method. The analysis included evaluation of Singular Values of Spectral Densities (SVSD), stabilization diagrams, and estimation of natural frequencies, damping ratios, and mode shapes [6], [7], [9]. The distribution of vibration responses across floors was analyzed using RMS acceleration values to identify floors exhibiting dominant responses.

2.4. Structural Dynamic and Resonance Governing Equations

The dynamic response of a structure to excitation can be modeled as a multi-degree-of-freedom system. When the excitation frequency approaches the natural frequency of the structure, the response amplitude increases significantly, depending on the system damping ratio. This condition is known as resonance

and is one of the primary causes of excessive vibrations in civil structures. The dynamic behavior of a structure can generally be represented by the equation of motion of a multi-degree-of-freedom system as follows [1], [2]:

$$\mathbf{M}\ddot{\mathbf{x}}(t) + \mathbf{C}\dot{\mathbf{x}}(t) + \mathbf{K}\mathbf{x}(t) = \mathbf{f}(t) \qquad (2)$$

where:
$\mathbf{M}$ = mass matrix
$\mathbf{C}$ = damping matrix
$\mathbf{K}$ = stiffness matrix
$\mathbf{x}(t)$ = displacement vector
$\mathbf{f}(t)$ = dynamic force vector

Resonance occurs when the excitation frequency fff approaches the natural frequency of the structure fnf_nfn, leading to a significant increase in the amplitude of the structural response. For a single-degree-of-freedom (SDOF) system, the dynamic amplification factor is given by [1]:

$$DAF = \frac{1}{\sqrt{(1 - r^2)^2 + (2\zeta r)^2}} \qquad (3)$$

where:
$r = \dfrac{f}{f_n}$ = frequency ratio,
ζ = damping ratio.

2.5. TMD/MTMD Design Formulation

A Tuned Mass Damper is a secondary oscillator system tuned to a specific frequency to absorb and dissipate vibration energy from the primary structure. Theoretically, the installation of a TMD transforms the structural system from a single-degree-of-freedom system into a two-degree-of-freedom system, thereby splitting a single resonance peak into two peaks with lower amplitudes [6]. For heritage buildings, TMDs are non-invasive and reversible, making them more suitable than conventional structural strengthening measures [10], [11].

A Tuned Mass Damper (TMD) is a secondary oscillator system consisting of a mass m_d, a spring k_d, and a viscous damper c_d, which is installed on the primary structure to reduce resonant responses. [10].

The mass ratio of the TMD is defined as:

$$\mu = \frac{m_d}{M} \qquad (4)$$

where MMM denotes the equivalent mass of the structure. The circular frequency of the TMD is given by:

$$\omega_d = \sqrt{\frac{k_d}{m_d}} \text{ dengan } \omega_d = 2\pi f_d \qquad (5)$$

The viscous damping coefficient of the TMD is expressed as [6]:

$$c_d = 2\zeta_d m_d \omega_d \qquad (6)$$

The installation of a TMD transforms the structural system from a single-degree-of-freedom system into a two-degree-of-freedom system, thereby splitting a single resonance peak into two peaks with lower amplitudes [10], [11].
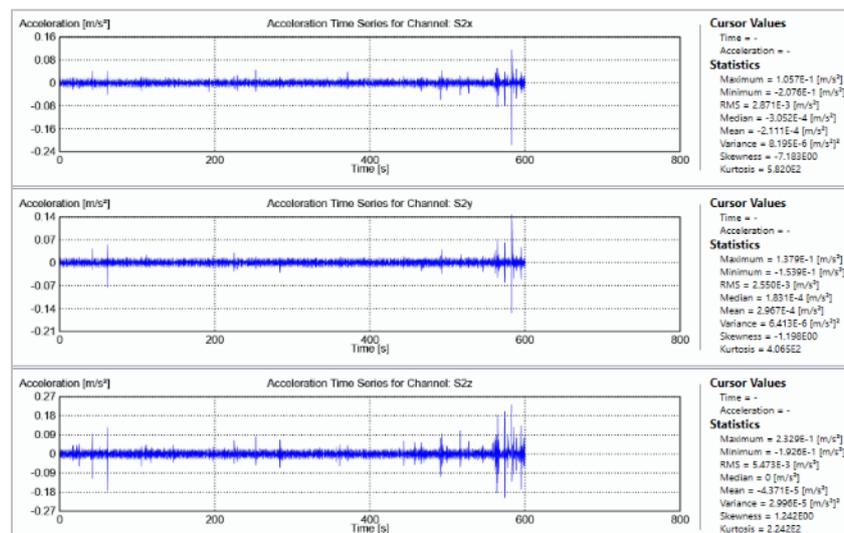
3. RESULT AND DISCUSSION

3.1. Acceleration Data Records

The acceleration data recorded during the dynamic testing for sensors S1, S2, and S3 are presented sequentially in **Figure 3**, **Figure 4**, and **Figure 5**.
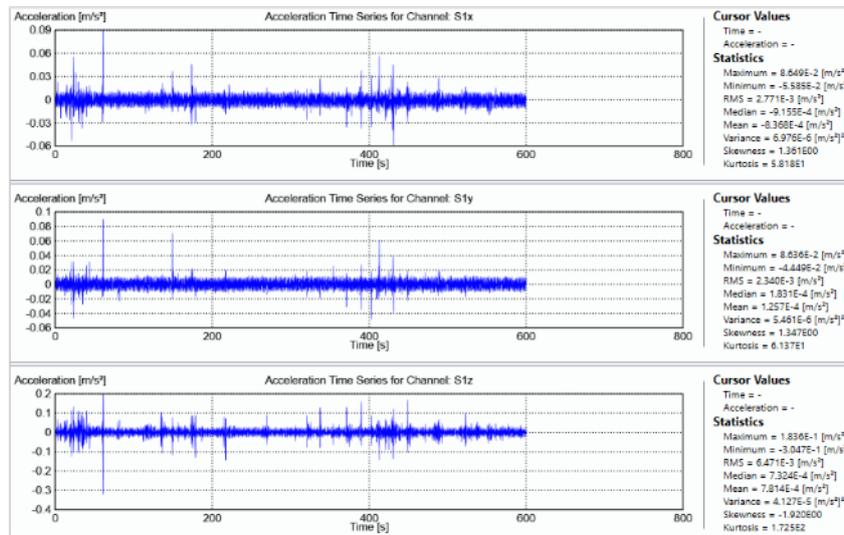
Figure 3. Acceleration Time History Recorded by Sensor S1

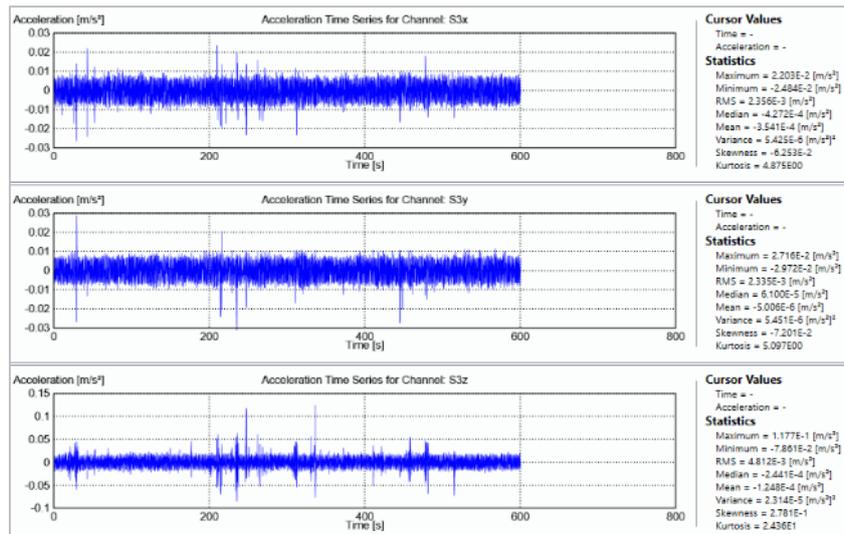**Figure 4.** Acceleration Time History Recorded by Sensor S2

Figure 5. Acceleration Time History Recorded by Sensor S3

The response distribution analysis indicates that the maximum vibration amplitude does not occur at the top floor but rather at an intermediate floor. This suggests that the operational mode activated by traffic is not a pure cantilever mode, but instead a mode with an antinode located at a specific floor level or a combination of global and local modes [1], [7].

3.2. Spectral Analysis of Response Data (Singular Values of Spectral Densities – SVSD)

Prior to modal identification using the Stochastic Subspace Identification (SSI) method, the spectral characteristics of the response data were first evaluated through Singular Values of Spectral Densities (SVSD) analysis.

The SVSD analysis aims to identify frequency ranges dominated by structural responses and to indicate the presence of natural frequencies, while also assessing data quality and the level of modal energy separation prior to further modal estimation.
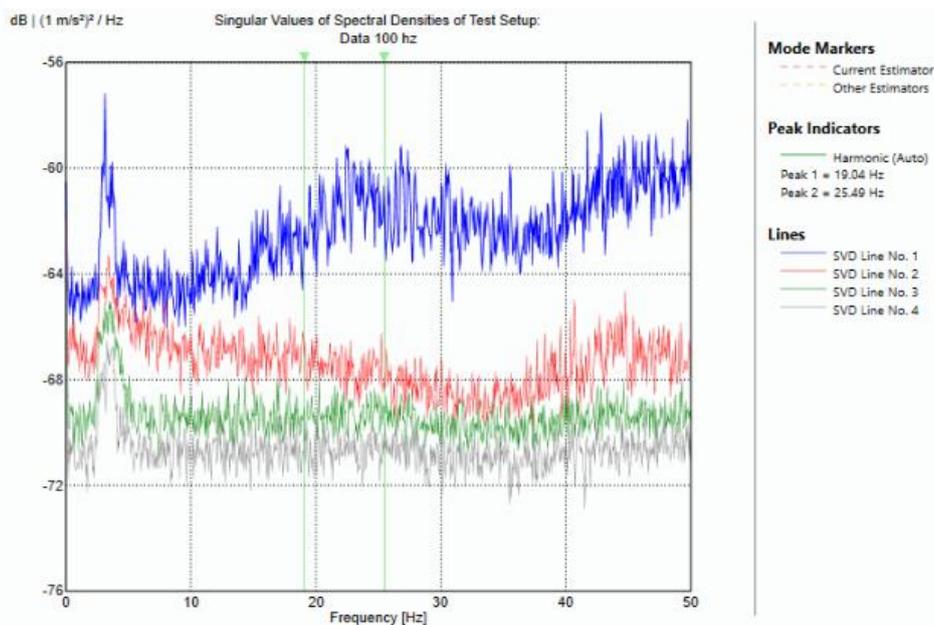

Figure 6. Singular Value Spectral Density Analysis of Ambient Vibration Data

Consistent peaks observed in the singular value curves across several principal components indicate the presence of structural modes, whereas frequency ranges without distinct peaks are generally associated with measurement noise.

3.3. Stabilization Diagram for Modal Identification Results (OMA–SSI-UPC)

To identify the modal parameters of the structure based on ambient vibration response data, Operational Modal Analysis (OMA) was performed using specialized software implementing the Stochastic Subspace Identification, Unweighted Principal Components (SSI-UPC) method. The modal estimation results for various model orders are presented in the form of a stabilization diagram, which illustrates the distribution of estimated natural frequencies as a function of model order and indicates the stability of each mode based on frequency, damping, and mode shape criteria. This diagram serves as the basis for selecting consistent physical structural modes and distinguishing them from noise-related or unstable modes prior to extracting the final modal parameters. The stabilization diagram is shown in Figure 6.
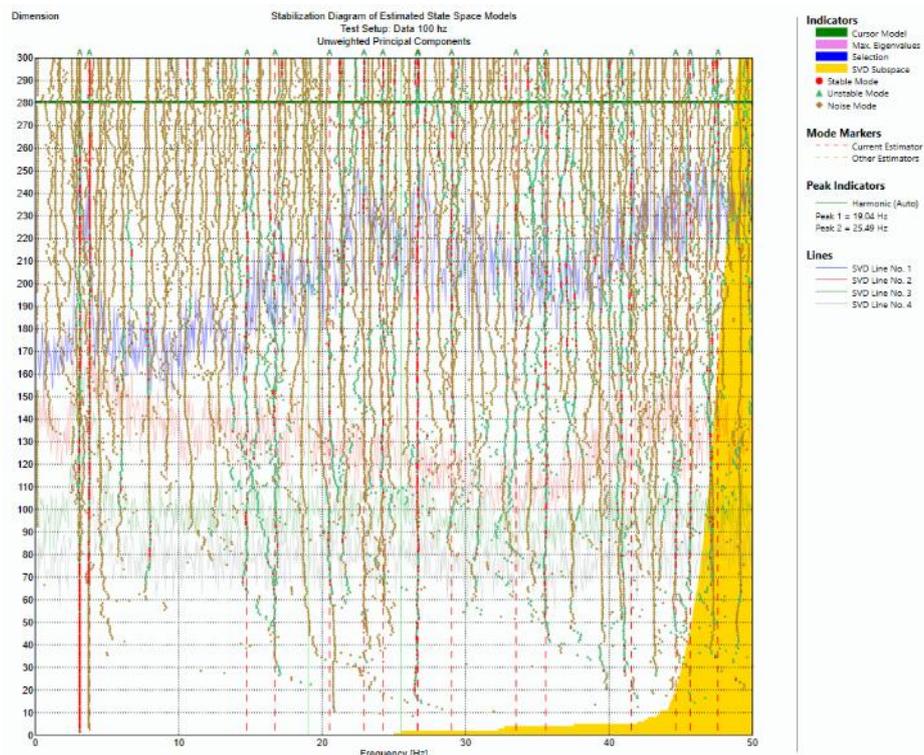
Figure 7. Stabilization diagram of estimated state-space models

The modes subsequently selected are those exhibiting stable frequencies and consistent mode shapes over a certain range of model orders, as well as realistic damping values for a multi-story building. The selected modal parameters are summarized in **Table 2**.

Table 2. Frequency and Damping

Mode	Frequency [Hz]	Damping [%]	Complexity [%]	Angle
1	3.094	7.59	30.451	0
2	3.776	6.712	39.75	0
3	14.752	1.59	35.855	0
4	16.708	1.854	19.342	0
5	20.512	0.41	34.08	0
6	22.908	0.555	38.838	0
7	24.228	1.056	32.69	0
8	26.635	0.741	24.731	0

Mode	Frequency [Hz]	Damping [%]	Complexity [%]	Angle
9	26.699	0.96	15.677	0
10	29.01	0.339	36.493	0
11	33.517	0.527	33.158	0
12	35.592	0.621	31.665	0
13	41.555	0.175	35.514	0
14	44.647	0.202	37.973	0
15	45.656	0.525	27.075	0
16	47.586	0.301	26.834	0

3.4. Identification of Resonance as the Cause of Excessive Vibrations

The OMA results indicate that the fundamental natural frequency of the building is approximately:

$$f_1 = 3.094 \text{ hz} \tag{7}$$

Meanwhile, ground vibration measurements show that the dominant frequency induced by traffic is approximately:

$$f_s \approx 3.102 \text{ hz} \tag{8}$$

The close proximity between f_1 and f_n results in a frequency ratio:

$$r = \frac{f_1}{f_n} = \frac{3.094}{3.102} = 1.003 \approx 1 \tag{9}$$

which theoretically corresponds to the maximum dynamic amplification factor, as described by the DAF expression in Eq. **Error! Reference source not found.**. Therefore, soil–structure resonance is identified as the primary cause of the excessive vibrations. The test results and subsequent analysis confirm that the dominant cause of excessive vibrations in the building is soil–structure resonance rather than insufficient structural capacity. This conclusion is further supported by the close agreement between the dominant ground frequency and the building's natural frequency, as well as the observed distribution of vibration responses across different floor levels [2], [3].

The measured fundamental frequency of the building is 3.094 Hz, while the dominant ground frequency induced by traffic is 3.102 Hz, yielding a frequency ratio of r = 1.003, which indicates near-resonant conditions. The equivalent structural mass is 7.35×10^5 kg, and the proposed MTMD system employs a total auxiliary mass of 4,000 kg (0.54% mass ratio) with tuned frequencies of 3.03 Hz and 3.16 Hz. The corresponding stiffness and damping coefficients are 0.73–0.79 MN/m and 7.6–7.9 kN·s/m, respectively. These numerical values confirm the feasibility and effectiveness of the proposed mitigation strategy.

3.5. Reduction of the Building to an Equivalent SDOF System

For the purpose of TMD design, the structure is reduced to an equivalent single-degree-of-freedom (SDOF) system corresponding to the first vibration mode [1], [10]. The equivalent structural mass is calculated from the total seismic weight of the building as:

$$m = \frac{W}{g} \approx 1 \tag{10}$$

with $W = 7207.37$ kN, $m = 9.81$ m/s², the equivalent mass is obtained as: $m \approx 7.35 \times 10^5$ kg.

The circular natural frequency of the first mode is then calculated as:

$$\omega_1 = 2\pi f_1 = 2\pi 3.09 \approx 19.44 \text{ rad/s} \tag{11}$$

3.6. TMD Design (Multiple Tuned Mass Damper)

To limit additional mass on the heritage building, a Multiple Tuned Mass Damper (MTMD) system consisting of two identical TMD units is selected. Each unit has a mass of:

$$m_d = 2000 \text{ kg} \tag{12}$$

resulting in a total mass ratio of:

$$\mu_{total} = \frac{4000}{735000} = 0.54\% \tag{13}$$

The tuning frequencies are intentionally detuned to widen the effective damping bandwidth [14], [15]:

$$f_{d1} = 3.03 \text{ hz and } f_{d2} = 3.16 \text{ hz} \tag{14}$$

The stiffness of each TMD unit is calculated as:

$$k_d = m_d(2\pi f_1)^2 \tag{15}$$

which yields:

$$k_1 = 0.73 \text{ MN/m and } k_2 = 0.79 \text{ MN/m} \tag{16}$$

Assuming a TMD damping ratio of, $\zeta_d = 10\%$, the viscous damping coefficient is calculated as:

$$c_d = 2\zeta_d m_d \omega_d \tag{17}$$

resulting in:

$$c1 \approx 7.6 \text{ kN.s/m and } c2 \approx 7.9 \text{ kN.s/m} \tag{18}$$

It should be noted that the present MTMD design is based on analytical modeling and identified modal parameters under ambient vibration conditions; therefore, the predicted performance may vary under different traffic intensities and environmental conditions. Further validation through long-term monitoring or in-situ post-installation measurements is recommended to confirm the actual effectiveness of the proposed system.

3.7. Effectiveness Evaluation and Recommendation

Based on the identified vibration mechanism and the analytical results, several mitigation alternatives were evaluated to determine the most effective and practical solution for reducing excessive vibrations. A comparison of the proposed mitigation alternatives is presented in **Table 3**.

Based on theoretical considerations and calculation results, the installation of an MTMD at the floor level exhibiting the maximum vibration response enables a significant reduction in the dynamic amplification factor at the resonance frequency. Compared to a single TMD, the MTMD approach is considered more robust against variations in traffic-induced excitation frequencies, while simultaneously allowing the use of a relatively small total added mass [15], [14], [16].

Table 3. Comparison of Mitigation Alternatives

Alternative	Mechanism	Effectiveness	Level of Intervention
Road improvement	Reduction of excitation source	Moderate	External
Soil barrier/trench	Reduction of wave propagation	Moderate	Surrounding environment
Increase in structural stiffness	Modification of natural frequency	High	Intrusive
TMD / MTMD	Reduction of resonance amplification	**High**	**Minimal**

4. CONCLUSION

Excessive vibrations in the building are primarily caused by soil–structure resonance rather than insufficient structural capacity. Quantitatively, the identified fundamental structural frequency (f_1 = 3.094 Hz) closely matches the dominant traffic-induced ground frequency (f_s = 3.102 Hz), yielding a frequency ratio of r ≈ 1.003, which confirms near-resonant soil–structure interaction as the governing mechanism of the excessive responses. Maximum vibration amplitudes may occur at intermediate floors, depending on the activated operational mode shapes. A small-mass Multiple Tuned Mass Damper (MTMD) is shown to be an effective, non-invasive mitigation solution suitable for heritage buildings. For existing structures located near vibration sources, dynamic evaluation based on Operational Modal Analysis is recommended prior to selecting mitigation strategies, and the integration of damping systems with long-term monitoring is encouraged to verify performance under varying traffic and environmental conditions. These findings provide a practical framework for civil engineers to diagnose resonance-related vibration problems and implement lightweight MTMD systems as an effective and conservation-compatible mitigation measure for existing buildings near traffic corridors.

ACKNOWLEDGEMENT

REFERENCES

[1]   A. K. Chopra, *Dynamics of Structures: Theory and Applications to Earthquake Engineering*. in Always learning. Pearson, 2017. [Online]. Available: https://books.google.co.id/books?id=R8_XjwEACAAJ

[2]   R. W. Clough and J. Penzien, *Dynamics of Structures*. Computers and Structures, Incorporated, 2003. [Online]. Available: https://books.google.co.id/books?id=TcxyAAAACAAJ

[3]   J. P. Wolf, *Dynamic Soil-structure Interaction*. in Prentice Hall Advanced Reference Series. Prentice-Hall, 1985. [Online]. Available: https://books.google.co.id/books?id=0ORRAAAAMAAJ

[4]   R. Brincker and C. Ventura, *Introduction to Operational Modal Analysis*. 2015. doi: 10.1002/9781118535141.

[5]   B. Peeters and G. De Roeck, "Stochastic System Identification for Operational Modal Analysis: A Review," *J. Dyn. Syst. Meas. Control*, vol. 123, no. 4, pp. 659–667, Dec. 2001, doi: 10.1115/1.1410370.

[6]   R. Brincker and C. E. Ventura, *Introduction to Operational Modal Analysis*. Wiley, 2015. doi: 10.1002/9781118535141.

[7] C. Rainieri and G. Fabbrocino, *Operational Modal Analysis of Civil Engineering Structures*. New York, NY: Springer New York, 2014. doi: 10.1007/978-1-4939-0767-0.

[8] H. Khoeri, S. W. Alisjahbana, J. Widjajakusuma, and N. Najid, "Estimasi Lendutan Pelat Untuk Menghitung Kapasitas Beban Dengan Akurasi Tinggi Menggunakan Uji Getar," *Konstruksia*, vol. 14, no. 2, pp. 175–188, 2023.

[9] P. Van Overschee and B. De Moor, *Subspace Identification for Linear Systems*. Boston, MA: Springer US, 1996. doi: 10.1007/978-1-4613-0465-4.

[10] J. P. Den Hartog, *Mechanical Vibrations*. McGraw-Hill, 1956. [Online]. Available: https://books.google.co.id/books?id=saNRAAAAMAAJ

[11] A. Kareem, T. Kijewski, and Y. Tamura, "Mitigation of motions of tall buildings with specific examples of recent applications," *Wind and Structures*, vol. 2, no. 3, pp. 201–251, Sep. 1999, doi: 10.12989/was.1999.2.3.201.

[12] A. K. Chopra, *Dynamics of Structures: Theory and Applications to Earthquake Engineering*. in Civil Engineering and Engineering Mechanics Series. Prentice Hall, 2012.

[13] H. Khoeri, G. Adeputra, and Z. Sembada, "Pemeriksaan Pemenuhan Standar Getaran untuk Keselamatan dan Kenyamanan Masjid Apung Ziadatul Abror dengan Smartphone," *Jurnal Konstruksi*, vol. 22, no. 1, pp. 90–100, 2024.

[14] R. Rana and T. T. Soong, "Parametric study and simplified design of tuned mass dampers," *Eng. Struct.*, vol. 20, no. 3, pp. 193–204, Mar. 1998, doi: 10.1016/S0141-0296(97)00078-3.

[15] C. Li and Y. Liu, "Further Characteristics for Multiple Tuned Mass Dampers," *Journal of Structural Engineering*, vol. 128, no. 10, pp. 1362–1365, Oct. 2002, doi: 10.1061/(ASCE)0733-9445(2002)128:10(1362).

[16] Y. L. Xu and K. C. S. Kwok, "Wind-induced response of soil-structure-damper systems," *Journal of Wind Engineering and Industrial Aerodynamics*, vol. 43, no. 1–3, pp. 2057–2068, Jan. 1992, doi: 10.1016/0167-6105(92)90632-K.