

Article

Low-Cost Angular-Velocity Measurements for Sustainable Dynamic Identification of Pedestrian Footbridges: A Case Study of the Footbridge in Gdynia (Poland)

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Abstract

This study investigates the practical value of angular-velocity measurements in the dynamic identification of pedestrian footbridges, addressing the need for reliable yet cost-effective diagnostics for slender civil structures. A comprehensive experimental campaign on a steel footbridge in Gdynia combined ambient vibration tests, forced excitation (light and heavy shakers), and controlled pedestrian loading. Synchronous translational accelerations and rotational velocities from MEMS sensors enabled evaluation of both bending and torsional responses. Three identification techniques—Peak Picking (PP), Frequency Domain Decomposition (FDD), and Stochastic Subspace Identification (SSI)—were applied and compared with a validated beam–shell FEM developed in SOFiSTiK. The results show that rotational data improve mode-shape interpretation and classification, notably resolving a coupled torsional–vertical mode (VT₂) that was ambiguous in acceleration-only analyses. The fundamental frequency of 3.1 Hz places the bridge in a resonance-prone range; field tests confirmed predominantly vertical response, with horizontal accelerations < 0.05 m/s² and peak vertical accelerations exceeding comfort class CL3 during synchronised walking of six pedestrians (≈2.55 m/s²) and jumping (up to 3.61 m/s²). Overall, the outcomes highlight that low-cost gyroscopic sensing offers substantial benefits for structural system identification and mode-shape characterization, enriching acceleration-based diagnostics and strengthening the basis for subsequent analyses. By reducing the financial and material demands of vibration testing, the proposed approach contributes to more sustainable assessment and maintenance of pedestrian bridges, aligning with resource-efficient monitoring strategies in civil infrastructure.



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Keywords: footbridges; human-induced vibration; modal analysis; gyroscopes; rotation sensors; field tests; FEM modeling; MEMS sensors; angular velocity; rotation angle response order

1. Introduction

Dynamic loads constitute one of the most critical factors influencing the serviceability and safety of civil engineering structures. Vibrations induced by natural phenomena or human activity may lead to significant structural responses that compromise user comfort or, in extreme cases, affect structural integrity [1–3]. Among civil structures, pedestrian footbridges are particularly sensitive to dynamic excitation because of their slender geometry, low mass, and limited damping [4–11]. The growing architectural trend toward lightweight and elegant designs often places their fundamental frequencies within the

range of pedestrian-induced loading [12–18]. Consequently, vibration serviceability has become a decisive design criterion, prompting both researchers and designers to improve the accuracy of dynamic characterization and to understand the complex interaction between structures and pedestrians [19–26].

Reliable evaluation of a bridge's dynamic behaviour depends on the accurate identification of its modal parameters—natural frequencies, damping ratios, and mode shapes [27–33]. Experimental modal testing has proved indispensable for this purpose, providing an objective insight into real structural behaviour under operational conditions. Most studies and structural-health-monitoring systems, however, are based exclusively on acceleration measurements, obtained from piezoelectric or MEMS-type accelerometers [34–41]. Acceleration data, owing to their sensitivity and compactness, constitute the primary source for operational modal analysis (OMA) techniques, such as Frequency-Domain Decomposition (FDD) or Stochastic Subspace Identification (SSI), and have been successfully employed in numerous field studies on footbridges of various structural systems [42–47]. Although acceleration-based monitoring provides valuable information on the translational motion of structures, it yields limited insight into their rotational response, which may play a decisive role in distinguishing coupled bending–torsional modes or in evaluating local deck–arch interactions.

Although every vibrating structure simultaneously undergoes translational and rotational motion, the direct measurement of rotational response in bridge dynamics has received limited attention compared to acceleration-based studies. Previous investigations have demonstrated that while accelerometers are highly effective in capturing translational components of motion, they offer limited insight into torsional and coupled dynamic effects, which can play a decisive role in serviceability assessment and fatigue-related deterioration. To address this limitation, recent studies have explored the use of angular-velocity sensors and gyroscopes to directly capture the rotational component of bridge motion. For example, Sekiya et al. [48] proposed a practical methodology for determining the rotational angle response of in-service steel–concrete bridges from angular velocity measurements obtained using MEMS-based inertial measurement units (IMUs). Their full-scale field experiments demonstrated that these compact and low-cost sensors can yield rotation data consistent with reference displacement gauges under real traffic excitation, thus enabling more comprehensive deformation monitoring and fatigue assessment of bridge decks and girders. Similarly, Gan et al. [49] developed a continuous deformation measurement system (CDMS) for bridges based on a fibre optic gyro (FOG). The system integrates angular velocity and linear acceleration data to reconstruct the bridge's deformation curve in real time without installing sensors on the structure or interrupting traffic. Field tests on a large cable-stayed bridge demonstrated that FOG-based measurements achieve accuracy comparable to levelling methods, providing a practical approach for continuous deformation and rotation monitoring in bridge health assessment. Complementary to these approaches, Sung et al. [50] proposed a multi-scale bridge health monitoring system that integrates accelerometers and gyroscopes to overcome the insensitivity of acceleration-based damage indices near bridge supports. By combining modal flexibility from acceleration data with rotational modal flexibility derived from angular velocity measurements, the system enables reliable localisation of multiple damages. Laboratory experiments confirmed that gyroscope-based rotation sensing significantly improves detection sensitivity near hinged supports, providing a complementary and practical approach for comprehensive bridge diagnostics. Among alternative non-contact approaches, the image-based system proposed by Park et al. [51] demonstrated the capability to measure minute rotational angles at bridge supports with sub-millidegree accuracy, emphasising the increasing recognition of rotational kinematics as a valuable complementary diagnostic indicator. Ha et al. [52]

developed a wireless MEMS-based inclinometer node for structural health monitoring applications. The system measures tilt angles using an accelerometer-based MEMS sensor and transmits data through a hybrid short- and long-range wireless network (RF 2.4 GHz and CDMA). Laboratory verification showed that the wireless system achieved millidegree-level accuracy ($\approx 0.003^\circ$ deviation from wired measurements), demonstrating that MEMS-based inclinometers can reliably capture structural tilt and serve as practical tools for deformation and rotation monitoring in SHM.

Nevertheless, the systematic use of gyroscope data in dynamic identification and modal analysis of footbridges remains scarce. Most bridge monitoring systems continue to rely on translational accelerations or displacements, while rotational quantities are neglected. Particularly in wide or asymmetric pedestrian bridges where bending–torsional coupling significantly influences vibration serviceability, the absence of direct angular-velocity measurements constrains a complete understanding of the structural response. Therefore, it becomes essential to examine whether the inclusion of low-cost gyroscopes can enhance the reliability of experimental modal identification and provide additional information inaccessible from acceleration data alone.

The rapid progress of micro-electro-mechanical systems (MEMS) technology has recently transformed the field of vibration monitoring. Modern MEMS gyroscopes are increasingly accurate, lightweight, and energy-efficient, with noise levels and bandwidths suitable for the low frequency range characteristic of footbridge vibrations [48,52]. Their cost-effectiveness and ease of integration with existing accelerometric systems make them attractive candidates for field measurements [49,50]. Moreover, the ability to capture both translational and rotational responses synchronously enables a more comprehensive description of the dynamic behaviour of complex structures [50]. These advances open promising perspectives for the application of angular-velocity data in vibration-based structural identification, model updating, and long-term health monitoring of bridges.

In practical engineering applications, translational modal parameters remain the primary basis for evaluating vibration serviceability, validating numerical models and designing mitigation measures. However, in many lightweight or asymmetric pedestrian bridges, rotational and coupled bending–torsional behaviours can influence user comfort and dynamic performance yet remain difficult to capture using acceleration measurements alone. In such cases, the integration of rotational sensing offers a meaningful practical benefit by improving the identification of torsion-dominated or closely spaced modes and by supporting more reliable updating of numerical models. Owing to their low cost and ease of deployment, MEMS gyroscopes provide a scalable and field-ready tool that can complement conventional acceleration-based testing not only during full-scale modal identification but also within structural health monitoring frameworks for the long-term observation of bridge dynamics. The practical outcomes of this study are directly applicable to full-scale modal identification, FEM model updating and vibration-based serviceability or SHM monitoring of lightweight pedestrian bridges, where combined translational–rotational measurements can improve the reliability of dynamic assessment.

The present study aims to investigate the usefulness of angular-velocity measurements in the dynamic identification of pedestrian footbridges. Using a comprehensive experimental campaign performed on a steel footbridge in Gdynia (Poland), this research combines translational accelerations and rotational velocities recorded by MEMS sensors under ambient, forced, and pedestrian-induced excitations. The objective is to evaluate how gyroscopic data can complement conventional acceleration-based analyses, improve the interpretation of coupled bending–torsional modes, and enhance the accuracy of numerical model validation. The potential benefits and limitations of this approach are discussed with respect to its applicability in practical vibration-based monitoring and serviceability

assessment of slender bridge structures. Furthermore, it should be noted that most of the studies on the dynamic identification of pedestrian footbridges rely exclusively on acceleration measurements, typically obtained from piezoelectric or MEMS accelerometers [34–41]. Compared with the existing literature, the novelty of the present work stems from its focus on the dynamic identification of a lightweight pedestrian footbridge rather than on deformation monitoring or damage detection, which dominate current rotation-based studies. While Sekiya et al. [48], Gan et al. [49] and Sung et al. [50] demonstrated the potential of rotational sensing for deformation reconstruction or multi-scale SHM, none of these studies investigated the identification of coupled vertical–torsional modes in slender footbridges, nor validated the benefits of rotational data against a numerical model under multiple types of excitation. To the best of the authors’ knowledge, this work is among the first full-scale applications of low-cost MEMS gyroscopes for modal identification of a pedestrian footbridge. The results show that rotational measurements substantially improve the recognition of torsion-dominated modes—particularly mode VT_2 —and enhance the agreement between experimental and numerical mode shapes. This establishes MEMS-based angular-velocity sensing as a practical, accurate and sustainable tool for vibration-based diagnostics of pedestrian bridges, extending current approaches reported in the literature.

2. Materials and Methods

2.1. Footbridge Descriptions

The analysed structure is a steel pedestrian footbridge located in Gdynia, Poland, spanning Chwarznieńska Street. It was designed to ensure a safe and convenient crossing of an expanded urban roadway, featuring both staircases and elevator access for individuals with reduced mobility. The structure was selected for its relatively slender geometry, low damping and lightweight deck, which make it susceptible to dynamic excitation.

The superstructure consists of a single-span steel girder system with an orthotropic deck (Figure 1). The span measures 21.00 m, with an overall deck width of 4.97 m and a clear width between balustrades of 3.75 m (Figure 2). The structural depth varies from 0.530 m at midspan to 0.555 m at the supports. The deck comprises a steel plate inclined longitudinally at 1.5% and transversely at 2.5%, forming a gable profile. Open longitudinal ribs (130×10 mm) are spaced at 300 mm intervals for stiffening, and transverse beams are provided every 2.0 m. Two primary steel I-girders, spaced 900 mm apart, support the deck. Web and top plate thicknesses are 14 mm, while the bottom flange is 16 mm thick and 200 mm wide.



Figure 1. Side and bottom view of the footbridge in Gdynia city.

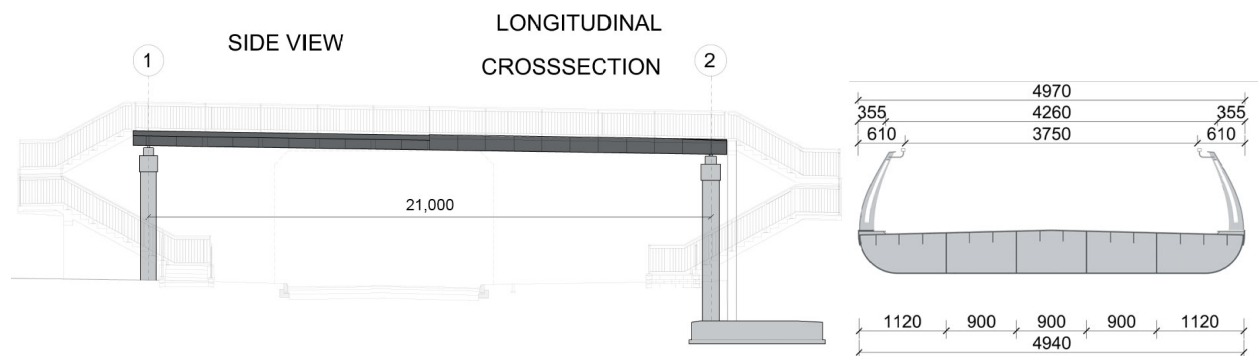


Figure 2. Structure layout of the footbridge in Gdynia city.

The superstructure is supported on elastomeric bearings placed atop two reinforced concrete piers with “T”-shaped caps. Each pier consists of a 600 mm diameter shaft and a 600 × 700 mm cap beam. Both the elevator shafts and the piers are founded on common deep foundations consisting of bored CFA piles (Continuous Flight Auger), 600 mm in diameter and 8.0 m in length. The orthotropic deck surface is protected with a 5 mm thick epoxy-polyurethane resin layer, ensuring waterproofing and slip resistance. All steel elements were corrosion-protected using a combined system of hot-dip galvanisation, zinc metallization, and multilayer epoxy coating. The primary load-bearing elements, including the orthotropic deck, main girders, transverse beams and longitudinal ribs, are made of S355J2 steel. The reinforced concrete elements, such as the piers and in situ staircases were constructed using C35/45 concrete, while the elevator shafts and abutments were made of C30/37 concrete. Reinforcement bars throughout the substructure conform to class B500SP. Elastomeric bearings are made of natural rubber and comply with the requirements of EN 1337-3 [53].

2.2. Experimental Procedures

In the experimental campaign, which included ambient vibration tests, forced excitation using actuators, and pedestrian-induced loading, the same measurement system was employed. It enabled simultaneous acquisition of 24 measurement channels, ensuring consistent data quality across all test scenarios. The setup included 12 vertical acceleration channels, 1 horizontal acceleration channel, and 11 angular velocity channels, distributed along the footbridge as shown in Figure 3.

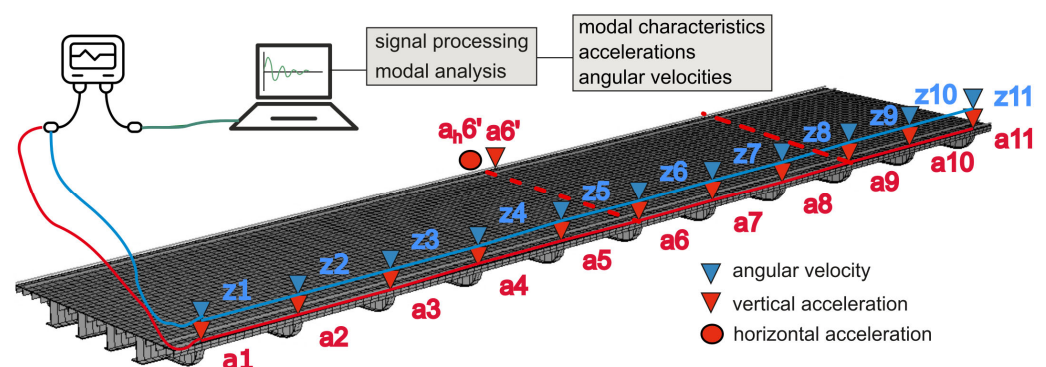


Figure 3. Layout of accelerometers (a1–a11, a6', a_h6') and gyroscopes (z1–z11) used for experimental in situ tests.

Three types of sensors were used during the experimental campaign. Angular velocity measurements were performed using MEMS-based gyroscopic sensors LPY403AL (STMicroelectronics, Kirkop, Malta). These sensors operate based on the Coriolis effect

with a measurement range of $\pm 30^\circ/\text{s}$ and a maximum effective frequency response of 140 Hz. Their unit cost is approximately 20 EUR, which makes them a particularly attractive option for resource-efficient and sustainable vibration monitoring of pedestrian structures. Structural acceleration measurements were performed using capacitive MEMS accelerometers (TE 4332M3-002 and TE 4312M3-002, Measurement Specialties, Hampton, VA, USA), selected for their high sensitivity in the low-frequency range typical of pedestrian-induced vibrations. The signals from both sensor types were recorded using three 8-channel QUANTUM HBM measurement amplifiers (Darmstadt, Germany), which provided signal conditioning and analogue-to-digital conversion. The amplifiers were coupled and synchronised to operate as a single acquisition unit, enabling fully synchronous sampling of all measurement channels. All sensors were mounted on the steel bridge's deck using neodymium magnets. This configuration ensured that acceleration and angular velocity signals were recorded simultaneously under identical environmental conditions (such as wind and temperature) and identical dynamic excitations from the shakers. Although the analyses of translational and rotational responses were carried out independently, the data remained fully comparable as they originated from the same measurement conditions and time intervals.

In selected forced vibration tests, the measurement setup was extended with a RIGOL DG1022 signal generator (Suzhou, China) used to produce controlled excitation signals. A dedicated laptop served as the data acquisition station, handling real-time signal visualisation, storage, and preliminary processing of the measurement results.

All tests were conducted at a sampling rate of 600 Hz and were repeated twice to ensure the repeatability and reliability of the measured responses.

To provide a clear overview of the measurement campaign, a concise workflow diagram (Figure 4) has been included to illustrate the sequential steps of the procedure, from numerical preparation and sensor calibration through field testing to signal processing and modal identification.

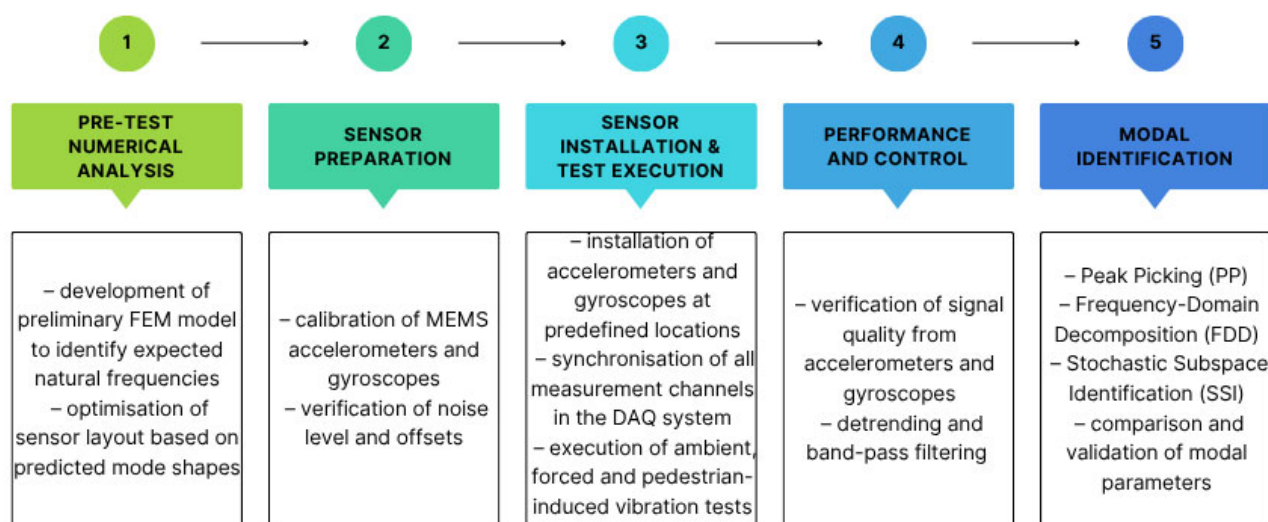


Figure 4. Workflow of the experimental campaign.

2.3. Excitation and Ambient Vibration In Situ Measurements

In order to assess the dynamic characteristics of the footbridge, both ambient vibration tests and forced vibration tests using mechanical exciters were conducted. Ambient vibration measurements were performed to capture the structure's natural dynamic response under operational conditions, including wind, distant traffic, and other environmental excitations. Each test lasted 20 min. The acquired acceleration data served as the basis for

operational modal analysis, enabling the identification of modal frequencies, modal shapes and damping ratios without requiring external excitation sources.

To complement these measurements and extend the frequency content of the excitation, controlled vibration tests were also carried out using two electromechanical shakers, shown in Figure 5. The first device was a light electrodynamic shaker with an excitation mass of 13 kg, while the second was a heavy, custom-designed mechanical shaker with a total excitation mass of approximately 300 kg. The use of shakers enabled the application of dynamic forces in a controlled and repeatable manner, thereby allowing for the excitation of a broader range of vibration modes than is typically possible with ambient excitations or vehicle passages alone.

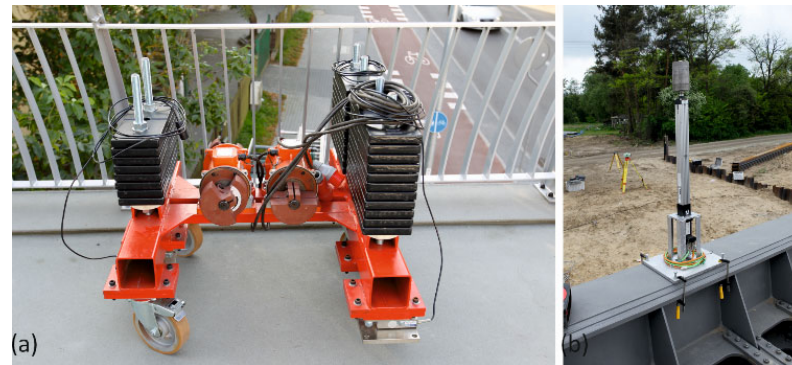


Figure 5. Vibration exciters used in the experimental campaign: (a) heavy mechanical shaker with an excitation mass of 300 kg, (b) lightweight electrodynamic shaker with an excitation mass of 13 kg.

In the case of the larger shaker, the dynamic force transmitted to the structure was measured directly using force sensors (load cells) mounted at the base of the exciter, which ensured accurate quantification of the applied load. For the lighter shaker, a different approach was adopted: an additional accelerometer was mounted on the moving mass of the shaker itself, allowing the excitation force to be estimated indirectly based on the measured acceleration and known mass. To evaluate the effect of excitation intensity on the possibility of identification of modal parameters, the heavy exciter used in this study was specifically designed not only to overcome the limitations related to the low mass of a small device but also to examine how the magnitude of the applied dynamic force influences the detectability of mode shapes and natural frequencies.

Both the light and heavy exciters were installed at two distinct positions on the structure: at cross-section 6 (CS6), located at midspan of the footbridge, and at cross-section 8 (CS8), positioned approximately at one-quarter of the total span (Figure 3). While the lightweight shaker had negligible influence on the structure's dynamic properties due to its small mass, the heavy exciter introduced a significant inertial load to the system. Therefore, in each case of its use, the effect of the added mass on the identified dynamic characteristics was carefully examined to avoid distortions in the interpretation of modal parameters. In all forced vibration tests, the input excitation was generated using a sweep sine (sine-sweep) function, allowing for a broad frequency range to be systematically excited. Each measurement lasted 180 s and was conducted with the excitation frequency ranged from 1 Hz to 100 Hz.

2.4. Tests with Walking Pedestrians

Pedestrian-induced vibrations are one of the most critical serviceability concerns for slender footbridges, often leading to discomfort or resonance phenomena when walking frequencies approach the structure's natural frequencies. To investigate the dynamic response of the footbridge under realistic loading scenarios, a series of tests was conducted

using controlled pedestrian activity. The excitation was introduced by individuals or groups of participants performing walking, running, and jumping at frequencies intentionally aligned with the structure's natural frequencies. This approach allowed for the evaluation of resonance phenomena and user comfort in real operating conditions.

The measured response parameters, including accelerations and angular velocities, were recorded during the following test scenarios:

- free walking of 1, 3, and 6 pedestrians without synchronisation,
- fast synchronised marching of 1, 3, 6, and 9 pedestrians at resonance frequency,
- synchronised running of 1, 3, and 6 pedestrians at resonance frequency,
- synchronised jumping of a group of 6 pedestrians at the resonance frequency.

In all tests related to free walking, each pedestrian was moving at their own pace. However, the synchronous excitations were harmonized using a metronome. Although the present study does not analyse bicycle or scooter loading explicitly, these lightweight forms of traffic typically generate vibration levels that are small compared with those induced by pedestrians and therefore do not meaningfully excite global modal behaviour of slender footbridges. In the case of the investigated structure, their impact is expected to be even more limited: although access via the elevator technically allows wheeled users, the actual presence of such traffic is sporadic due to the bridge's location, function and predominant pedestrian usage.

2.5. Modal Parameters Identification Techniques

The identification of the dynamic properties of the tested footbridge was carried out using three well-established techniques: Peak Picking (PP), Frequency Domain Decomposition (FDD), and Stochastic Subspace Identification (SSI) [54–56]. Each of these methods has specific advantages and applicability depending on the type of excitation and measurement quality.

The Peak Picking (PP) method is one of the most popular classical techniques due to its simplicity and efficiency [57,58]. It is based on the identification of resonance peaks in the Frequency Response Functions (FRFs), which are estimated from input-output data under known excitation conditions. The method can use receptance, mobility, or accelerance transfer functions, depending on the type of measured signals (displacement, velocity, or acceleration). In this study, accelerance and angular velocity functions were primarily used. The PP method is best suited for lightly damped structures, as it tends to lose accuracy for systems with high or very low damping. In the forced vibration tests, the excitation was applied in selected locations, with the input defined as the dynamic force generated by the shaker, while the output consisted of structural responses measured as vertical accelerations (a_1 – a_{11}) and angular velocities (z_1 – z_{11}) recorded at multiple points along the footbridge. Natural frequencies and mode shapes were extracted from the peaks of the measured FRFs. Damping was estimated using the half-power bandwidth method.

The Frequency Domain Decomposition (FDD) method belongs to the category of Operational Modal Analysis (OMA) techniques, which allow for identification using only output signals (structure response) without knowledge of the input excitation [59,60]. FDD is based on the decomposition of the Power Spectral Density (PSD) matrix of the measured responses. By performing Singular Value Decomposition (SVD) of the PSD matrix at each frequency, the dominant singular values corresponding to structural modes can be isolated. Mode shapes are extracted from the corresponding singular vectors. FDD is well-suited for lightly damped structures and provides reliable estimates of natural frequencies and mode shapes, especially under ambient excitation. In this study, vertical accelerations (a_1 – a_{11}) and angular velocities (z_1 – z_{11}) measured along the footbridge were used as output signals for the FDD analysis.

The Stochastic Subspace Identification (SSI) method is a time-domain OMA technique that allows the estimation of natural frequencies, damping ratios, and mode shapes based purely on measured output signals [42,61]. Unlike FDD, which operates in the frequency domain, SSI models the system as a stochastic state-space model and identifies its parameters by projecting measured signals into orthogonal subspaces. It is particularly effective for systems with closely spaced modes and higher damping. The SSI method provides robust estimates even in the presence of measurement noise, making it a powerful tool for operational identification in civil engineering applications. In the current study, the SSI method was applied to the measured structural responses, including vertical accelerations (a1–a11) and angular velocities (z1–z11), recorded at multiple locations along the footbridge. In addition to controlled excitation tests, the use of Operational Modal Analysis (OMA) methods—including the SSI technique—enables fully non-invasive and resource-efficient measurements based on ambient-induced vibrations, contributing to more sustainable diagnostic practices.

This multi-method approach increased the reliability of the identified modal parameters and allowed for deeper insights into the structural dynamics under both ambient and controlled excitations. The consistency and quality of the identified mode shapes were additionally verified using the Modal Assurance Criterion (MAC) [62,63], which quantifies the correlation between experimental and numerical mode shapes. A MAC value above 0.90 is generally considered to indicate a high level of agreement between mode shapes.

The agreement between experimental and numerical frequency results was quantified using the Relative Difference (RD), defined as:

$$RD[\%] = \frac{f_{FEM} - f_{EXP}}{f_{EXP}} \times 100, \quad (1)$$

where f_{FEM} is the natural frequency from the FEM model and f_{EXP} is the experimentally identified frequency.

Additionally, the Mean Absolute Relative Error (MARE) was calculated as the average of the absolute RD values for all considered modes.

All analyses were conducted in MATLAB R2024a [64] using validated proprietary algorithms developed by the authors, dedicated to structural modal identification using Peak Picking (PP), Frequency Domain Decomposition (FDD), and Stochastic Subspace Identification (SSI) techniques.

2.6. The Finite Element Model of the Footbridge and the Numerical Analysis

It is recognised that each in situ test should be preceded by an appropriate numerical analysis. A reliable FEM model helps determine sensor layout and expected natural frequencies that are most relevant to the structural response. Moreover, the availability of a verified model supports the interpretation and validation of test results [1].

In this study, a beam-shell model of the footbridge was developed using SOFiSTiK 2024 software (Figure 6) [65]. The geometry and material properties were adopted from the technical documentation. The footbridge slab was modeled using four-node quadrilateral shell elements (QUAD) of the Timoshenko-Reissner type. These elements have an enriched state of deformation in the surface, reduction in the blocking effect; they also take into account the shear effect, and the different position of the coating reference surface (eccentricity). The main girders, crossbeams, and longitudinal ribs were modeled with two-node spatial beam elements (BEAM), also of the Timoshenko type, including shear effects and eccentric axes. Support conditions were represented by linear elastic spring elements (SPRI). One abutment was equipped with longitudinally fixed and cross-locked bearings, while the other allowed lateral movement. The opposite abutment included one

longitudinal sliding and one multi-directional bearing. The final numerical model consisted of 5551 nodes, 5632 shell elements, 3190 beam elements, and 8 spring supports.

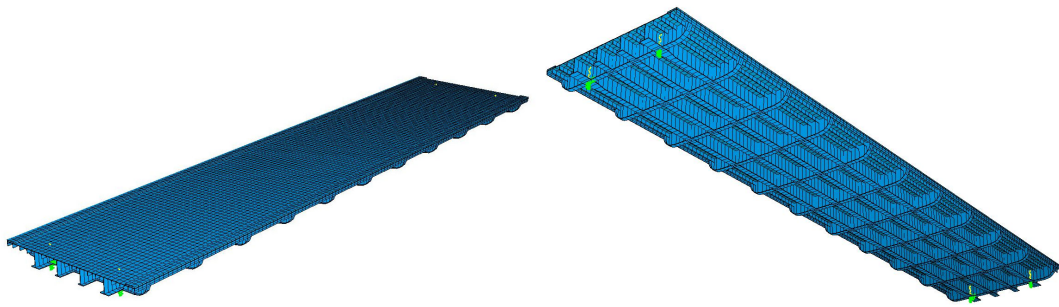


Figure 6. FEM model of the footbridge in Gdynia.

The footbridge was modelled using S355 structural steel according to EN 1993-1-1 [66]. The following material properties were adopted: Young's modulus $E = 205,000$ MPa, Poisson's ratio $\nu = 0.30$, shear modulus $G = 78,846$ MPa, yield stress $f_y = 355$ MPa, and ultimate tensile strength $f_t = 490$ MPa. In order to accurately reproduce the dynamic characteristics of the structure, a validation procedure was carried out. The total mass of the FEM model was adjusted to match the value provided in the workshop documentation, taking into account the weight of elements omitted in the numerical model, such as welds and ribs of the girders. The stiffness of non-structural components, including barriers and pavement, was not modelled; however, their masses were included. In addition, the stiffness of the elastic springs representing the bearings was determined based on the characteristics of the installed elastomeric bearings.

In the numerical analyses, two FEM model variants were considered to reflect the conditions of the experimental tests. The first variant, corresponding to the PP method with the small exciter as well as the FDD and SSI methods, represented the unloaded structure without additional external mass. The second variant included the mass of the large modal exciter, as used in the PP tests, with the exciter positioned in cross-sections 6 and 8. In this case, the added mass was accounted for in both the natural frequency calculations and the corresponding mode shapes.

2.7. Vibration Serviceability of Footbridges According to the Current Codes

Only a few design guidelines and standards explicitly address the influence of pedestrian-induced vibrations on walking comfort. The most commonly referenced documents are the French S  tra guidelines [67], the European HiVoss recommendations [68], and the DoLFfHIV guide for lightweight footbridges [69]. Eurocode 1 [70] also includes relevant provisions, although they are more general in scope.

Despite differing in their assumptions and load modelling approaches, these guidelines follow a similar two-stage process for dynamic assessment. The first step is to evaluate the natural frequencies of the structure. If the fundamental frequencies lie outside the defined critical ranges, it is assumed that the dynamic response will remain within acceptable limits, and further analysis is not required.

Figure 7 and Table 1 summarise the frequency ranges considered critical, acceptable, or negligible for both vertical and horizontal vibration modes. According to S  tra, HiVoss, and DoLFfHIV, the vertical frequency range between 1.7 and 2.1 Hz carries the highest risk of resonance, especially during synchronised walking. Frequencies below 1.0 Hz and above 5.0 Hz are generally considered safe. Eurocode 1 adopts a more conservative rule, requiring dynamic analysis for any structure with a fundamental vertical frequency below 5 Hz.

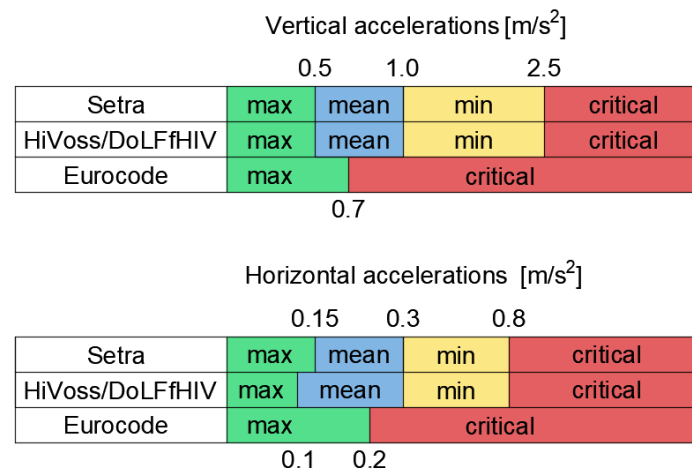


Figure 7. Comfort levels and corresponding acceleration ranges according to Sétra, HiVoSS, DoLFfHIV and Eurocode.

Table 1. Critical frequency ranges according to Sétra and HiVoss/DoLFfHIV guidelines and Eurocode 1.

Guideline	Resonance Risk Level			
	High	Medium	Low	Negligible
Vertical frequencies [Hz]				
Sétra	1.7–2.1	1.0–1.7; 2.1–2.6	2.6–5.0	0–1.0; >5
HiVoss/DoLFfHIV	1.7–2.1	1.25–1.7; 2.1–2.3	2.3–4.6	0–1.25; >4.6
Eurocode	0.0–5.0	–	–	>5
Horizontal frequencies [Hz]				
Sétra	0.5–1.1	0.3–0.5; 1.1–1.3	1.3–2.5	0.0–0.3 >2.5
HiVoss/DoLFfHIV	0.7–1.0	0.5–0.7; 1.0–1.2	–	0.0–0.5 >1.2
Eurocode	0.0–2.5	–	–	>2.5

For horizontal modes, the most resonance-sensitive frequency range is between 0.5 and 1.1 Hz according to Sétra, and 0.7 to 1.0 Hz according to HiVoss and DoLFfHIV. Eurocode sets 2.5 Hz as the lower limit for acceptable horizontal modes.

If the structure's natural frequencies fall within the critical ranges, further dynamic analysis is required to estimate the acceleration levels induced by pedestrian traffic. These accelerations are then evaluated against comfort thresholds.

Perception of vibrations is inherently subjective, depending on factors such as direction, duration, and the activity or posture of pedestrians. Eurocode 1 defines comfort limits at 0.7 m/s^2 for vertical acceleration and 0.2 m/s^2 for horizontal acceleration. Below these values, vibrations are considered imperceptible or acceptable; above them, the comfort of pedestrians may be compromised.

The Sétra, HiVoss, and DoLFfHIV guidelines propose more nuanced comfort classifications with four defined levels. For vertical accelerations, thresholds are set at 0.5 m/s^2 , 1.0 m/s^2 , and 2.5 m/s^2 , with values above 2.5 m/s^2 considered unacceptable. In the horizontal direction, phenomena of forced synchronisation ('lock-in') may occur if the structural acceleration exceeds 0.15 m/s^2 (Sétra) or 0.1 m/s^2 (HiVoss/DoLFfHIV). Acceleration values above 0.8 m/s^2 are classified as unacceptable due to the high risk of resonance or pedestrian synchronisation.

3. Results

3.1. The Experimental and Numerical Identification Results

The experimental modal analysis of the footbridge was carried out using accelerations signals, which provided the translational degrees of freedom (DoFs), and rotational velocity measurements obtained from gyroscopes, which captured the rotational DoFs. In the modal identification analyses, acceleration and angular velocity data were processed independently using the PP, FDD, and SSI techniques to determine translational and rotational mode shapes, respectively. As both datasets were acquired simultaneously, their frequency and phase correspondence allowed coherent interpretation of the coupled vertical–torsional modes. In the forced tests, three excitation configurations were applied: a small modal shaker and a large shaker positioned at two deck cross-sections (cross-sections 6 and 8). The analyses of both translational and rotational responses were carried out using three independent methods: Peak Picking (PP), Frequency Domain Decomposition (FDD), and Stochastic Subspace Identification (SSI). The analysis of the recorded signals allowed for the identification of ten well-separated global vibration modes of the footbridge in the range below 40 Hz. Figures 8 and 9 present representative imaginary parts of the frequency response functions (FRFs) obtained using the PP method for translational and rotational responses, while Figures 10 and 11 show representative results from the AVT tests based on translational and rotational measurements, expressed as the singular value spectrum from the FDD method and the stabilisation diagram from the SSI method.

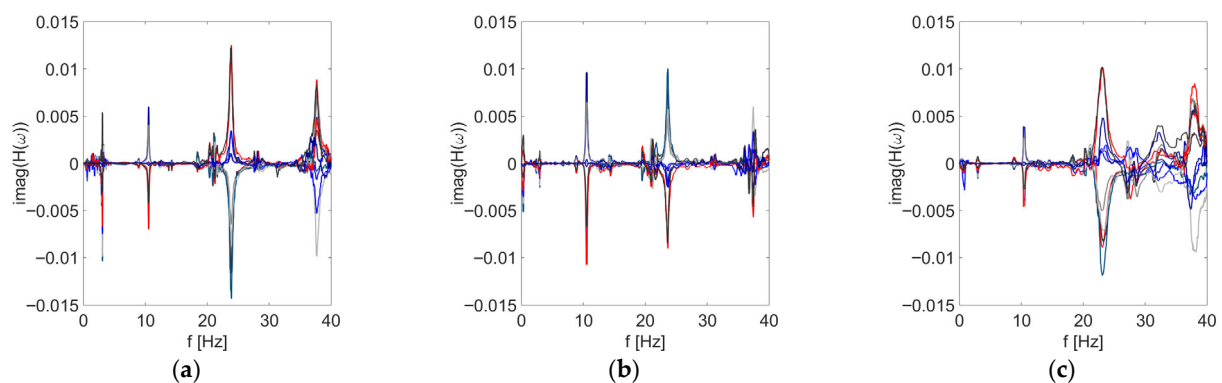


Figure 8. Representative imaginary part of the frequency response functions (FRFs) obtained using the Peak Picking (PP) method for translational responses (a) large shaker at cross-section 6, (b) large shaker at cross-section 8, (c) small shaker. Coloured lines correspond to signals recorded from different acceleration locations along the deck.

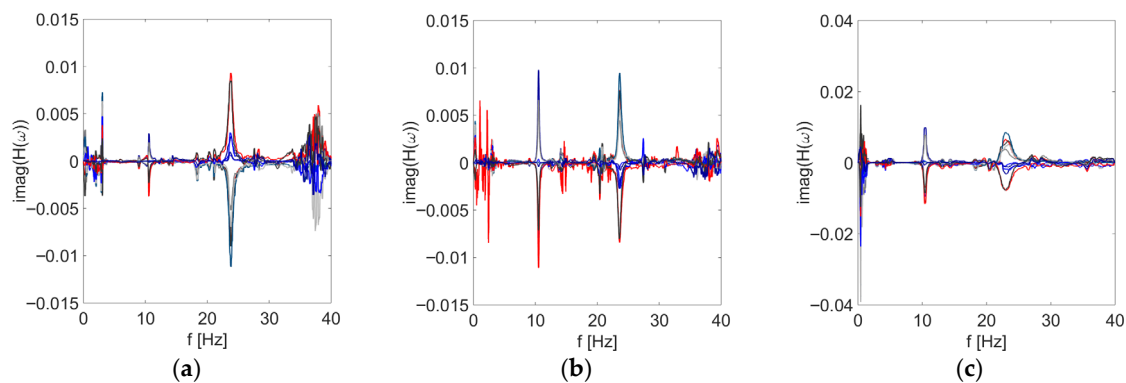


Figure 9. Representative imaginary part of the frequency response functions (FRFs) obtained using the Peak Picking (PP) method for rotational responses (a) large shaker at cross-section 6, (b) large shaker at cross-section 8, (c) small shaker. Coloured lines correspond to signals recorded from different gyroscope locations along the deck.

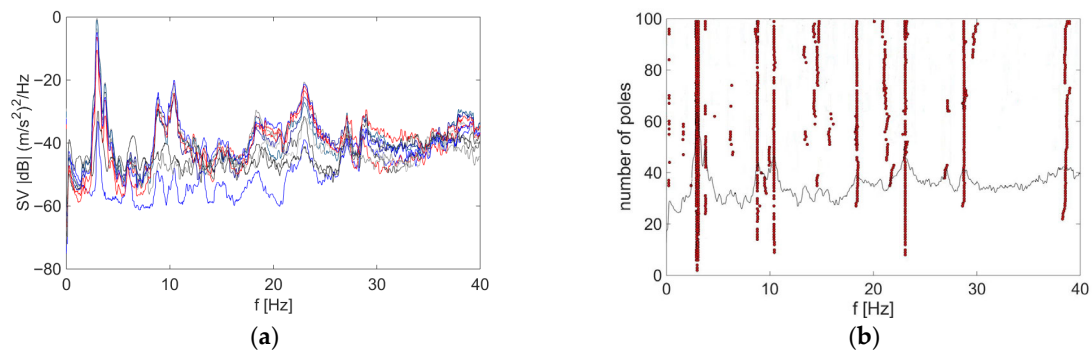


Figure 10. Representative results from the AVT test based on translational responses: (a) singular value spectrum obtained using the FDD method (The different colour lines represent consecutive singular values obtained from the singular value decomposition of the power spectral density matrix at each frequency), (b) stabilisation diagram obtained using the SSI method (The red dots represent the identified poles while the grey line shows the singular value spectrum of the output data).

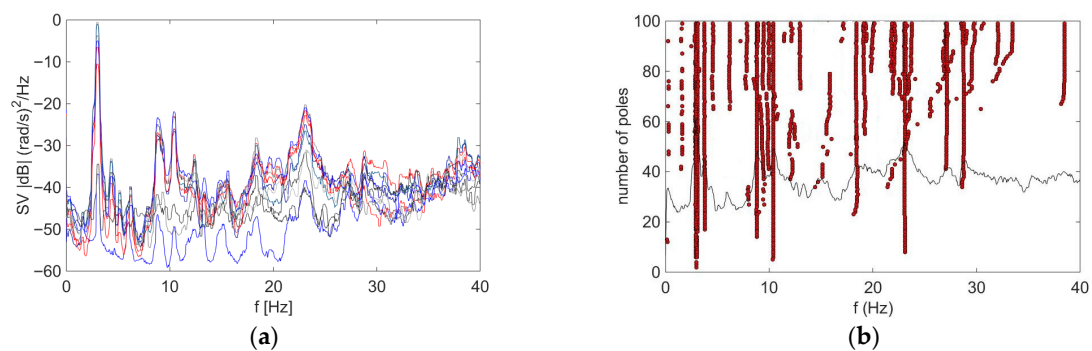


Figure 11. Representative results from the AVT test based on rotational responses: (a) singular value spectrum obtained using the FDD method (The different colour lines represent consecutive singular values obtained from the singular value decomposition of the power spectral density matrix at each frequency), (b) stabilisation diagram obtained using the SSI method. (The red dots represent the identified poles while the grey line shows the singular value spectrum of the output data).

For each mode, the final natural frequency was calculated as the arithmetic mean of the values obtained from translational (accelerometers) and rotational (gyroscopes) measurements, averaged over all repetitions of a given test. For all tests, the variation in frequencies between repetitions remained below 1%. The identified modes were classified into three categories according to their dominant deformation pattern: V—pure vertical bending modes, T—pure torsional modes, and VT—coupled vertical–torsional modes. In cases where groups of identified modes exhibited a similar number of half-waves along one edge of the deck, their classification into the respective categories was refined by analysing the relative displacements of points a1–a11 with respect to the sensor positioned on the opposite edge at location a6' (Figure 2).

The most easily identifiable modes were V_1 ($f \approx 3.10$ Hz), V_2 ($f \approx 10.5$ Hz) and V_3 ($f \approx 22$ Hz), which exhibited the most pronounced peaks in the frequency response functions or the highest stability in the SSI stabilisation diagrams. These modes provided clear reference points for correlating experimental results with the numerical model and for assessing the accuracy of the identification methods applied.

Damping ratios estimated from SSI were low to moderate, typically 0.2–0.5% for the fundamental vertical modes and up to 2–5% for selected higher-order modes. SSI yielded the most stable damping estimates, while PP half-power showed higher scatter for lightly excited modes.

The numerical modal analysis of the validated FEM model allowed for the identification of ten global vibration modes within the frequency range up to 40 Hz (Figure 12). The structure exhibits four distinct vertical bending modes (V_1, V_2, V_3, V_4), corresponding to successive half-wavelengths along the deck span, which is typical for slender beam- or plate-like systems with a uniform stiffness distribution. Notably, no pure horizontal bending modes were identified in the analysed frequency range, confirming the high in-plane stiffness of the deck. In contrast, the modal spectrum contains a considerable number of torsional and mixed torsional–vertical coupled modes ($T_1, VT_1, VT_2, VT_3, VT_4, VT_5$), with frequencies close to those of the corresponding vertical modes and with a similar number of half-waves along the span. Within each group, the torsional component produces an almost identical deformation pattern along the deck edge as the corresponding pure bending mode, differing mainly in the presence of twist. Detailed numerical results for the unloaded model are presented in Table 2, while those for the model including the large exciter are shown in Table 3.

Table 2. Correlation between experimental (PP-ex_{small}, FDD, SSI) and FE model frequencies [Hz]—model without additional exciter mass.

Mode	Frequency [Hz]				Relative Difference [%]		
	PP-ex _{small}	FDD	SSI	FE Model	PP-ex _{small}	FDD	SSI
T_1	2.95	2.91	2.90	2.88	2.37	1.03	0.69
V_1	3.10	3.07	3.09	3.10	0.00	0.98	0.32
VT_1	8.80	8.80	8.80	8.53	3.07	3.07	3.07
V_2	10.47	10.43	10.41	10.64	1.62	2.01	2.21
VT_2	14.60	14.80	14.80	16.74	14.66	13.11	13.11
VT_3	18.20	18.25	18.27	17.87	1.81	2.08	2.19
V_3	21.75	21.73	21.70	21.96	0.97	1.06	1.20
VT_4	23.10	23.13	23.10	22.19	3.94	4.06	3.94
V_4	32.20	28.70	28.79	30.46	5.40	6.13	5.80
VT_5	38.45	38.55	38.59	38.96	1.33	1.06	0.96
MARE					4.42	3.46	3.35
MEAN V_1 - V_2 - V_3					3.86	1.35	1.24

Table 3. Correlation between experimental (PP-ex_{large6}, PP-ex_{large8}) and FE model frequencies [Hz]—model with large exciter mass placed in cross-sections 6 and 8.

Mode	Frequency [Hz]				Relative Difference [%]	
	PP-ex _{large 6}	PP-ex _{large 8}	FE Model _{large 6}	FE Model _{large 8}	PP-ex _{large 6}	PP-ex _{large 8}
T_1	2.68	2.84	2.78	2.84	3.73	0.00
V_1	3.05	3.10	3.01	3.05	1.31	1.61
VT_1	8.91	8.82	8.51	8.43	4.49	4.42
V_2	10.34	10.42	10.59	10.30	2.42	1.15
VT_2	14.30	14.40	16.46	16.58	15.10	15.14
VT_3	18.03	18.50	17.46	17.62	3.16	4.76
V_3	21.13	21.25	21.48	21.75	1.66	2.35
VT_4	22.70	22.12	21.92	21.84	3.44	1.27
V_4	31.40	30.70	30.20	30.22	3.82	1.56
VT_5	37.80	37.70	38.79	38.82	2.62	2.97
MARE					4.18	3.52
MEAN V_1 - V_2 - V_3					1.80	1.70

The correlation between experimental and numerical mode shapes for translational degrees of freedom was assessed using the Modal Assurance Criterion (MAC), with results presented in Figures 13–16 and Tables 4 and 5.

Table 4. Modal Assurance Criterion (MAC) values between numerical and experimental mode shapes for translational degrees of freedom, obtained using PP-ex_{large6}, PP-ex_{large8}, PP-ex_{small}, FDD, and SSI methods.

Mode	MAC									
	PP-ex _{large6}	Nr. Group	PP-ex _{large8}	Nr. Group	PP-ex _{small}	Nr. Group	FDD	Nr. Group	SSI	Nr. Group
T ₁	0.993	1	0.995	1	0.995	1	0.995	1	0.989	1
V ₁	0.993	1	0.991	1	0.994	1	0.993	1	0.986	1
VT ₁	0.993	2	0.992	2	0.987	2	0.990	2	0.984	2
V ₂	0.997	2	0.997	2	0.994	2	0.995	2	0.958	2
VT ₂	0.879	2	0.670	2	0.883	2	0.971	2	0.956	3
VT ₃	0.981	3	0.965	3	0.938	3	0.943	3	0.927	3
V ₃	0.935	3	0.948	3	0.928	3	0.952	3	0.972	3
VT ₄	0.993	3	0.979	3	0.989	3	0.986	3	0.973	3
V ₄	0.867	4	0.812	4	0.755	4	0.737	4	0.841	4
VT ₅	0.981	4	0.980	4	0.939	4	0.917	4	0.951	4
Mean _{all}	0.961		0.933		0.940		0.948		0.954	
Mean _{V1-V2-V3}	0.975		0.979		0.972		0.980		0.972	

Table 5. Modal Assurance Criterion (MAC) values between numerical and experimental mode shapes for rotational degrees of freedom, obtained using PP-ex_{large6}, PP-ex_{large8}, PP-ex_{small}, FDD, and SSI methods.

Mode Order	MAC									
	PP-ex _{large6}	Nr. Group	PP-ex _{large8}	Nr. Group	PP-ex _{small}	Nr. Group	FDD	Nr. Group	SSI	Nr. Group
T ₁	0.873	1	0.955	1	0.676	1	0.898	1	0.959	1
V ₁	0.875	1	0.952	1	0.669	1	0.893	1	0.962	1
VT ₁	0.947	2	0.895	2	0.642	2	0.860	2	0.936	2
V ₂	0.955	2	0.911	2	0.627	2	0.867	2	0.946	2
VT ₂	0.854	3	0.885	3	0.667	3	0.997	3	0.917	3
VT ₃	0.860	3	0.918	3	0.735	3	0.999	3	0.925	3
V ₃	0.819	3	0.844	3	0.736	3	0.997	3	0.927	3
VT ₄	0.856	3	0.896	3	0.673	3	0.997	3	0.922	3
V ₄	0.784	4	0.743	4	0.656	4	0.386	4	0.757	4
VT ₅	0.524	4	0.644	4	0.506	4	0.764	4	0.558	4
Mean _{all}	0.835		0.864		0.659		0.886		0.881	
Mean _{V1-V2-V3}	0.883		0.902		0.677		0.919		0.945	

Overall, the agreement between the FEM model and the translational experimental results was high, with MAC values exceeding 0.93 for most modes and test configurations (Table 4). The strongest correlations ($\text{MAC} \geq 0.985$) were observed for the fundamental torsional mode T₁, the first vertical bending mode V₁, and the coupled mode VT₁, regardless of the excitation method or FEM variant.

For mid-frequency modes, such as V₂, VT₃, V₃, and VT₄, MAC values typically remained above 0.93, confirming consistent reproduction of the deformation patterns in both experimental and numerical results. The lowest correlations were recorded for the modes V₄ and VT₂. In the case of VT₂, the MAC dropped to 0.67 for PP-ex_{large8}, which coincides with the largest relative difference in frequency among the translational modes ($\text{RD} \approx 15\%$).

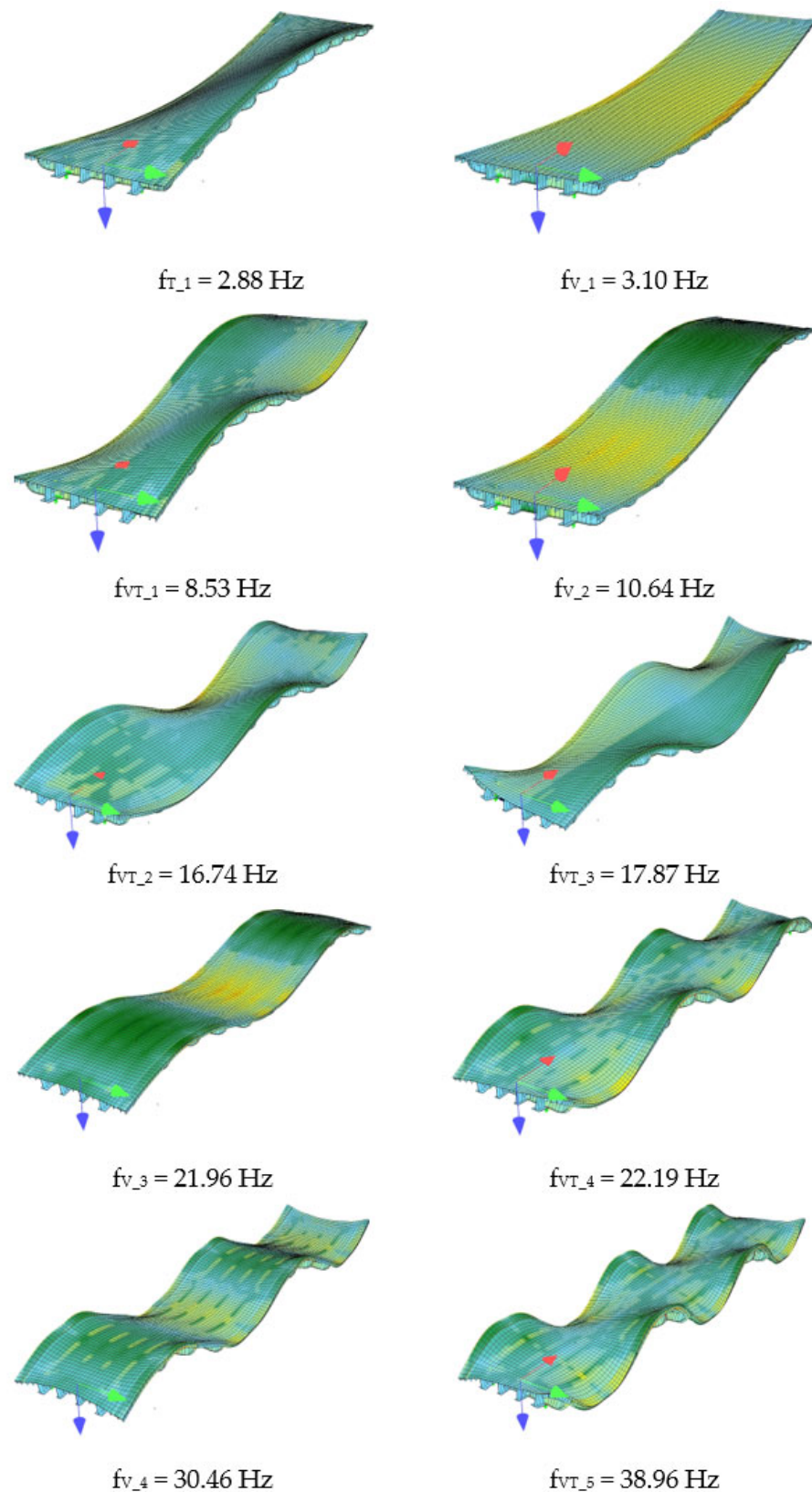


Figure 12. Numerical mode shapes of the validated beam-shell model of the Chwarznieńska Street footbridge with corresponding natural frequencies. The contours illustrate the spatial distribution of modal displacements, showing the relative deformation amplitude for each vibration mode (deformed shapes are scaled for visualisation purposes).

In the correlation between experimental and numerical rotational mode shapes the agreement was lower than for the translational measurements, with MAC values ranging from 0.51 to 0.99, depending on the mode and identification method. The strongest correlations ($\text{MAC} \geq 0.87$) were observed for the first two modes (T_1, V_1) in most cases, particularly for PP-exlarge₈ and SSI. The coupled mode VT_2 and the higher-order modes VT_3, V_3 , and VT_4 also reached high correlations ($\text{MAC} \geq 0.92$) when identified with FDD and SSI, indicating consistent reproduction of rotational deformation patterns in these cases. Despite the larger discrepancies, all methods correctly identified the fifth mode shape VT_2 .

The lowest correlations were obtained for the fourth vertical bending mode V_4 and the coupled mode VT_5 , where MAC dropped to 0.386 and 0.506. These results reflect the greater difficulty in capturing higher-order rotational shapes, where measurement noise and reduced sensitivity of rotational degrees of freedom have a stronger effect.

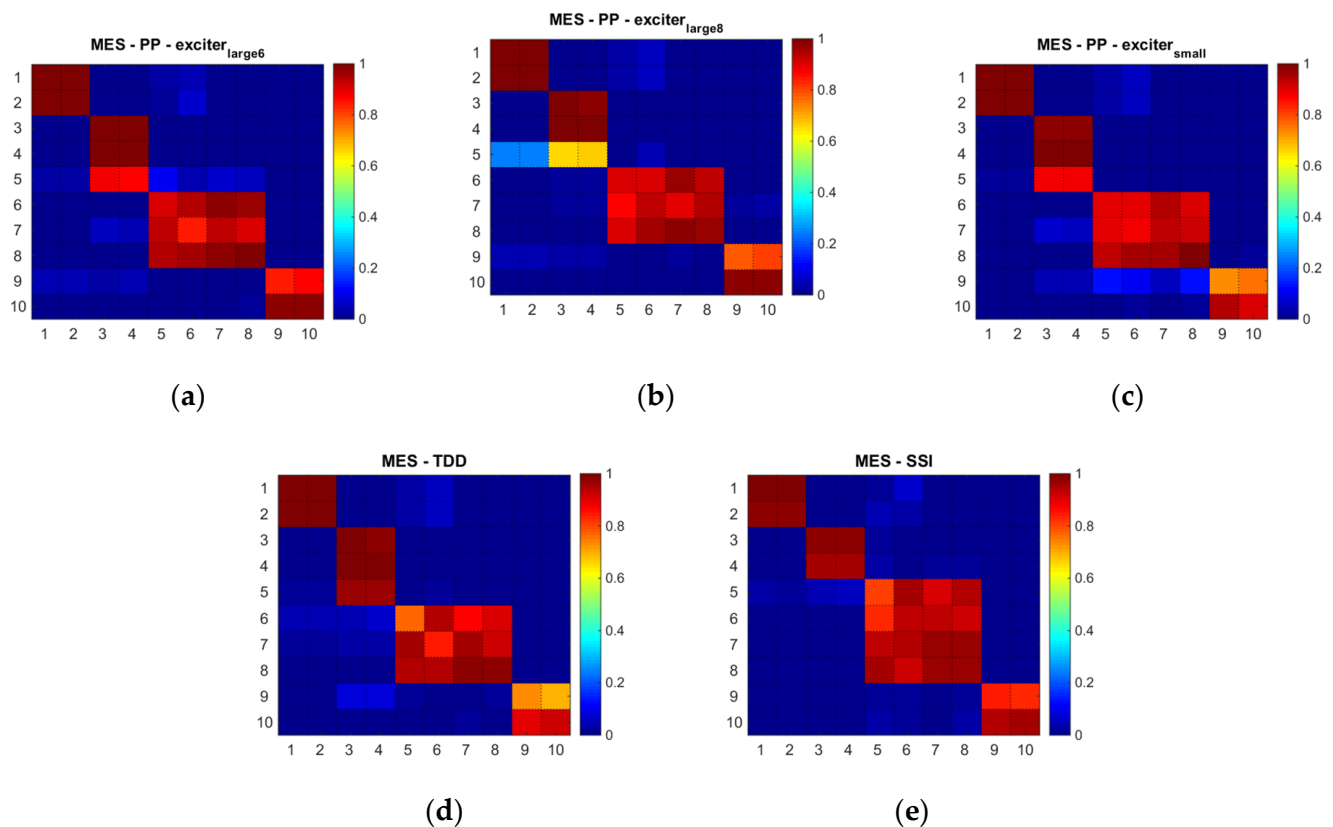


Figure 13. Modal Assurance Criterion (MAC) matrices between numerical and experimental mode shapes for translational measurements: (a) PP-exlarge₆, (b) PP-exlarge₈, (c) PP-exsmall, (d) FDD, and (e) SSI.

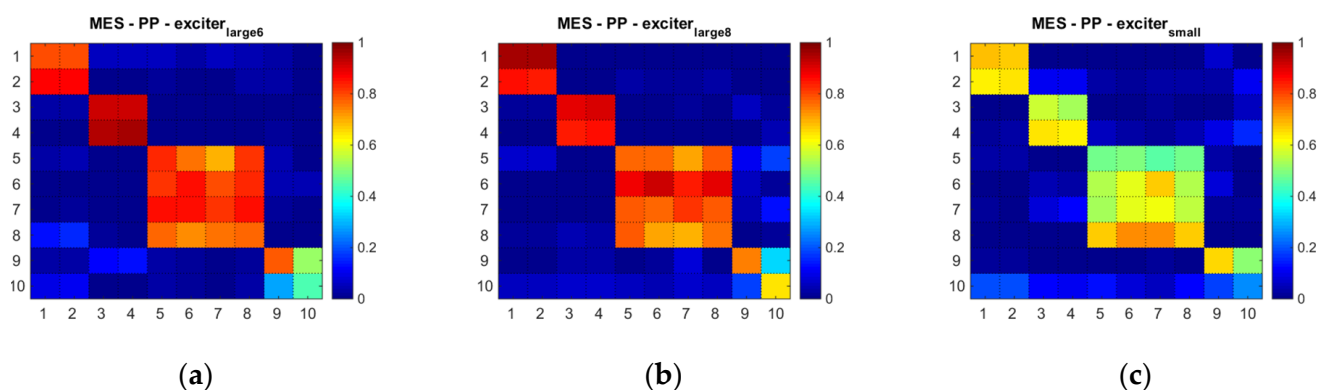


Figure 14. Cont.

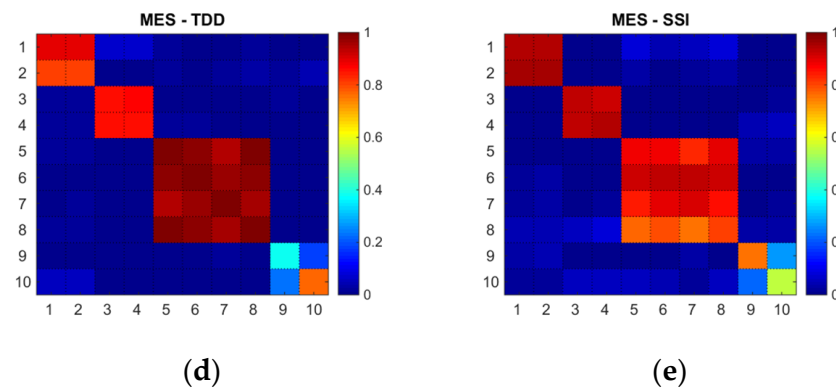


Figure 14. Modal Assurance Criterion (MAC) matrices between numerical and experimental mode shapes for rotational measurements: (a) PP-ex_{large6}, (b) PP-ex_{large8}, (c) PP-ex_{small}, (d) FDD, and (e) SSI.

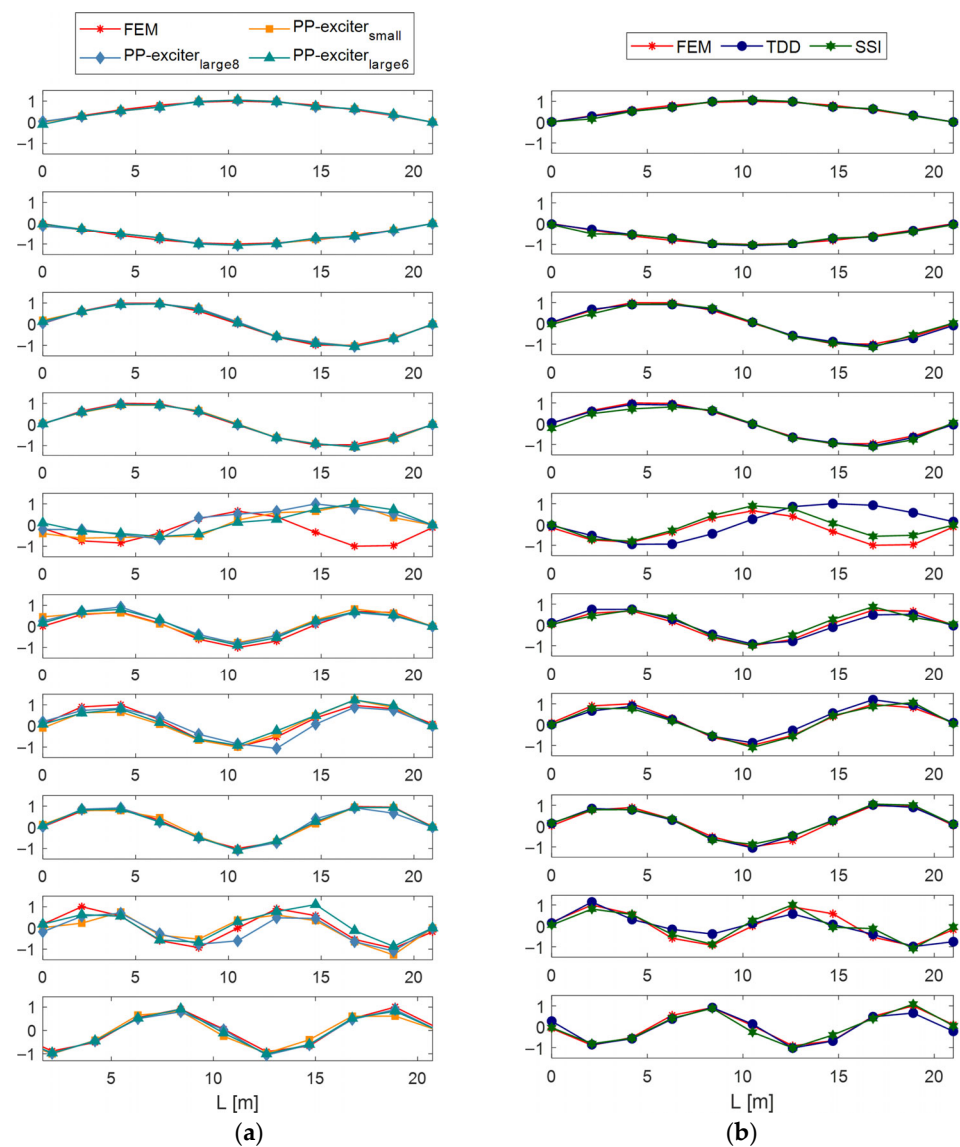


Figure 15. Experimental and numerical mode shapes for translational DoFs, obtained using: (a) Peak Picking (PP-ex_{large6}, PP-ex_{large8}, PP-ex_{small}) (Colours and markers correspond to different excitation configurations and FEM mode shapes) and (b) FDD and SSI methods, matched to corresponding FEM modes. Colours and markers correspond to different excitation configurations. (Colours represent different identification methods—FDD, SSI, and FEM model).

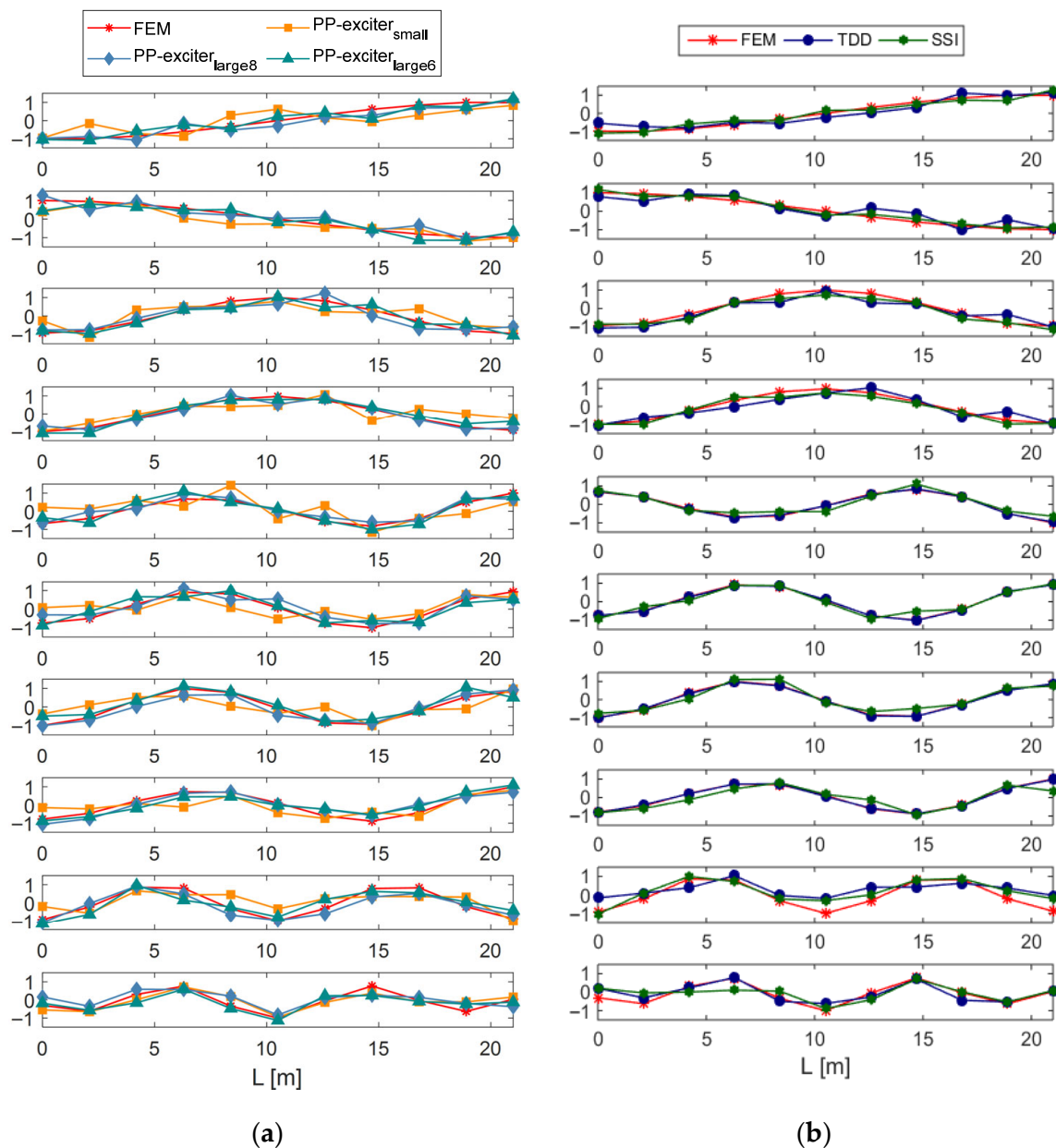


Figure 16. Experimental and numerical mode shapes for rotational DoFs, obtained using: (a) Peak Picking (PP-ex_{large6}, PP-ex_{large8}, PP-ex_{small}) (Colours and markers correspond to different excitation configurations and FEM mode shapes) and (b) FDD and SSI methods, matched to corresponding FEM modes (Colours represent different identification methods—FDD, SSI, and FEM model).

3.2. Assessment of Pedestrian Effects

All acceleration signals recorded during pedestrian loading scenarios were pre-processed by applying linear detrending and then filtered using a 5th-order Butterworth band-pass filter (0.5–20 Hz) to eliminate the irrelevant components. Representative time histories and corresponding Fourier spectra of vertical accelerations are shown in Figures 17–19 for the three loading cases: synchronous walking, running, and jumping. The records were taken at two characteristic locations: at the midspan (sensor a6) and at approximately one-fourth of the span (sensor a8). Measurements were conducted at all sensor positions a1–a11 as defined in Figure 2, but only selected results are presented here for the sake of brevity.

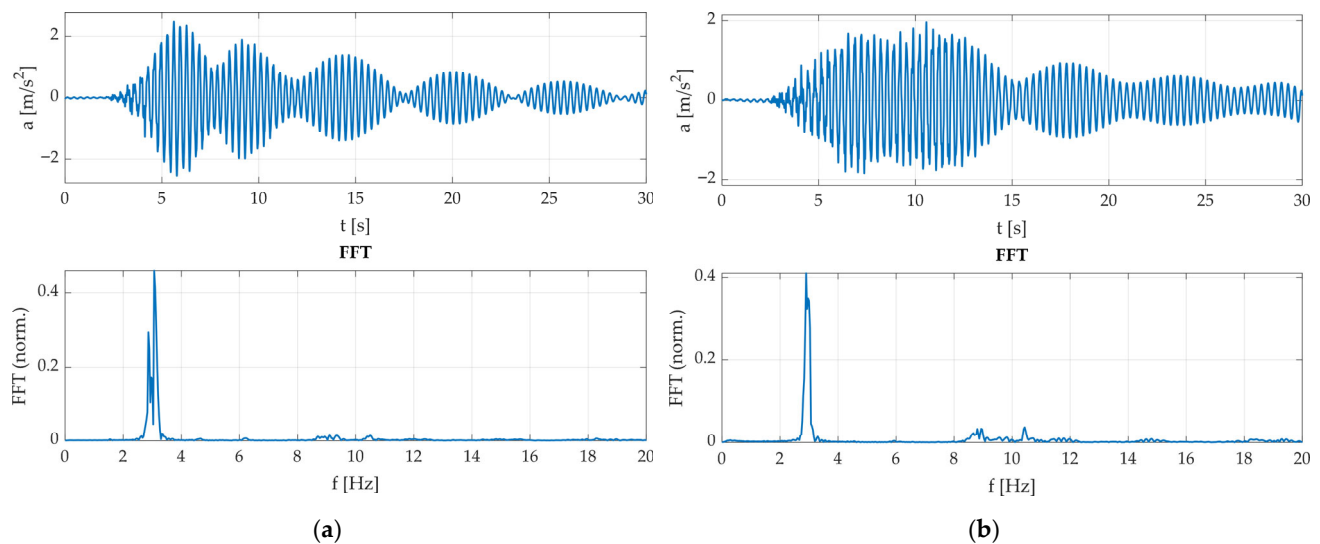


Figure 17. Time histories and corresponding Fourier spectra of vertical accelerations induced by synchronous walking of a group of six pedestrians: (a) measurement point a6—midspan, (b) measurement point a8—approximately quarter-span.

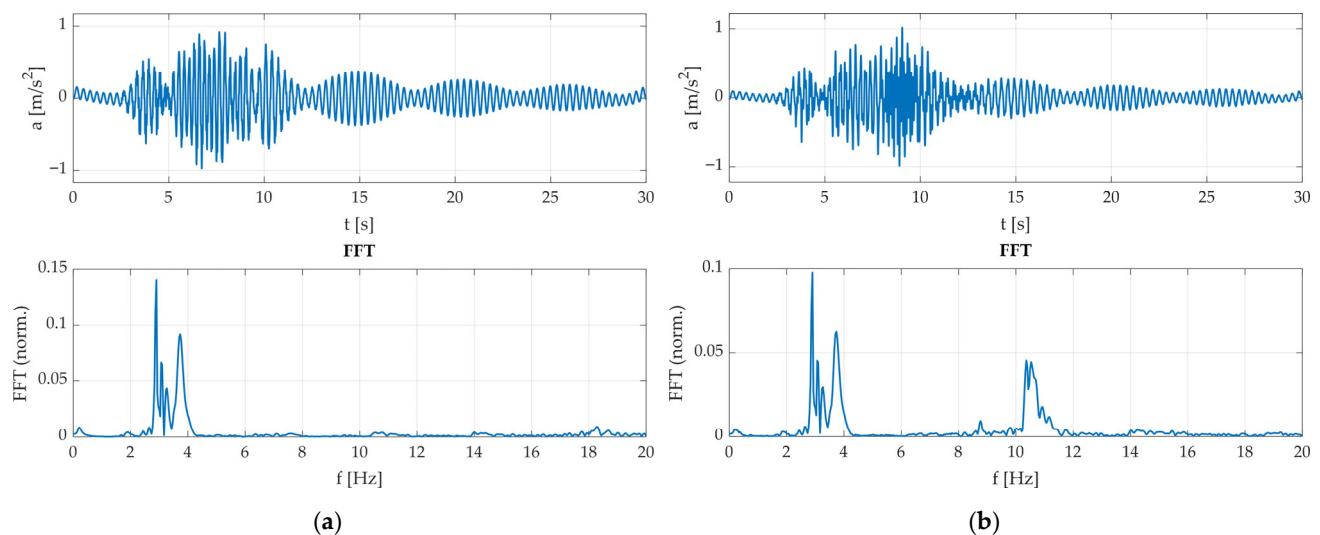


Figure 18. Time histories and corresponding Fourier spectra of vertical accelerations induced by synchronous running of a group of six pedestrians: (a) measurement point a6—midspan, (b) measurement point a8—approximately quarter-span.

Table 6 and Figures 20–23 presents the peak accelerations measured at all sensor locations (a1–a11) for different pedestrian loading scenarios. The results clearly show that synchronous pedestrian activities led to markedly higher responses compared to free walking. In particular, accelerations exceeding 1.0 m/s² were frequently observed across multiple locations, especially for midspan sensors (a5–a7). During synchronous walking of six pedestrians, vertical accelerations reached values between 1.0 and 2.6 m/s² at several sensor positions (e.g., a5–a7), while in contrast, free walking generated values typically below 1.0 m/s². Running produced a smaller response, with peak accelerations in the range below 1.0 m/s² observed at all locations. The most pronounced effects occurred during synchronous jumping, where accelerations exceeded 3.6 m/s² at the midspan (a6) and surpassed 2.5 m/s² at several other positions.

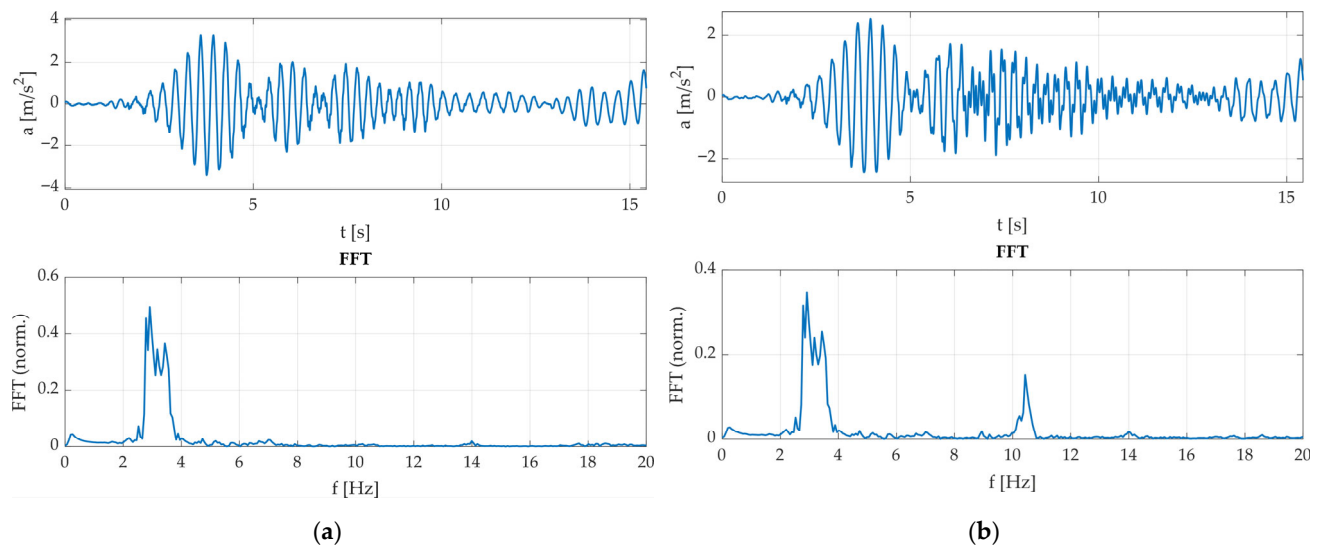


Figure 19. Time histories and corresponding Fourier spectra of vertical accelerations induced by synchronous jumping of a group of six pedestrians in the middle of the span: (a) measurement point a6—midspan, (b) measurement point a8—approximately quarter-span.

Table 6. Peak values of vertical and horizontal accelerations [m/s²] measured at sensor locations a1–a11 during pedestrian load tests.

Sensor	Walk							Run			Jump
	Free			Synchronous				Synchronous			Sync.
	1	3	6	1	3	6	9	1	3	6	6
Vertical acceleration [m/s ²]											
a1	0.02	0.03	0.06	0.03	0.12	0.11	0.09	0.02	0.02	0.03	0.05
a2	0.16	0.33	0.5	0.26	0.86	1.02	0.8	0.13	0.19	0.41	1.19
a3	0.24	0.54	0.85	0.39	1.19	1.47	1.38	0.27	0.36	0.64	1.77
a4	0.24	0.66	1.07	0.41	1.57	1.87	1.78	0.35	0.5	0.85	2.59
a5	0.29	0.86	1.22	0.38	2.09	2.44	2.3	0.42	0.57	0.88	3.4
a6	0.28	0.88	1.29	0.42	2.26	2.55	2.31	0.44	0.56	0.85	3.61
a7	0.31	0.82	1.17	0.4	2.17	2.49	2.15	0.43	0.52	0.82	3.3
a8	0.28	0.62	0.98	0.4	1.62	1.92	1.56	0.35	0.45	1.00	2.45
a9	0.34	0.55	0.91	0.44	1.43	1.74	1.39	0.32	0.34	0.67	2.01
a10	0.21	0.3	0.51	0.27	0.77	0.95	0.77	0.18	0.14	0.39	1.06
a11	0.02	0.03	0.03	0.02	0.13	0.16	0.09	0.02	0.09	0.07	0.06
Horizontal acceleration [m/s ²]											
a6y	0.02	0.02	0.01	0.01	0.02	0.05	0.01	0.01	0.03	0.04	0.05

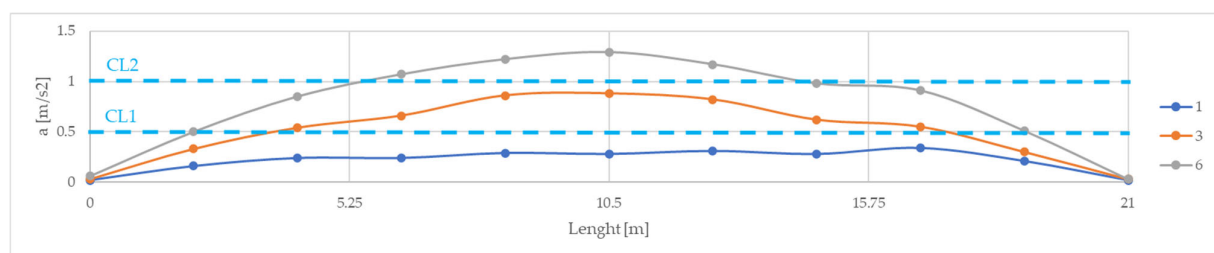


Figure 20. Distribution of peak vertical accelerations along the span during free walking of 1, 3, and 6 pedestrians compared with comfort limit levels CL1 and CL2.

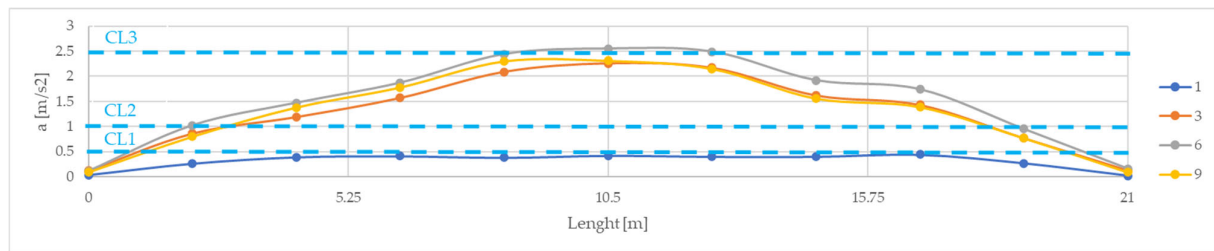


Figure 21. Distribution of peak vertical accelerations along the span during synchronous walking of 1, 3, 6 and 9 pedestrians compared with comfort limit levels CL1, CL2 and CL3.

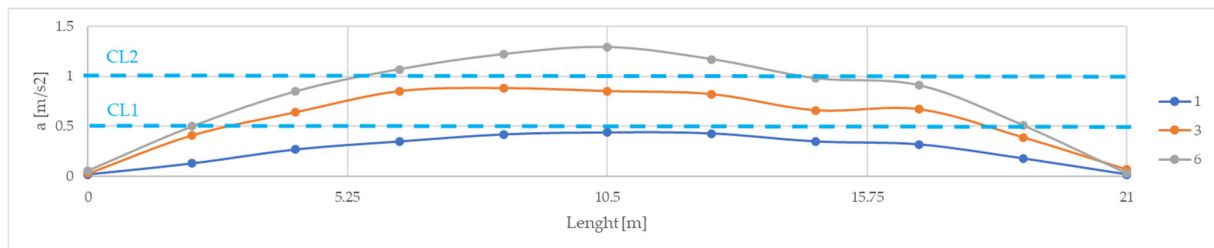


Figure 22. Distribution of peak vertical accelerations along the span during synchronous running of 1, 3, 6 and 9 pedestrians compared with comfort limit levels CL1 and CL2.

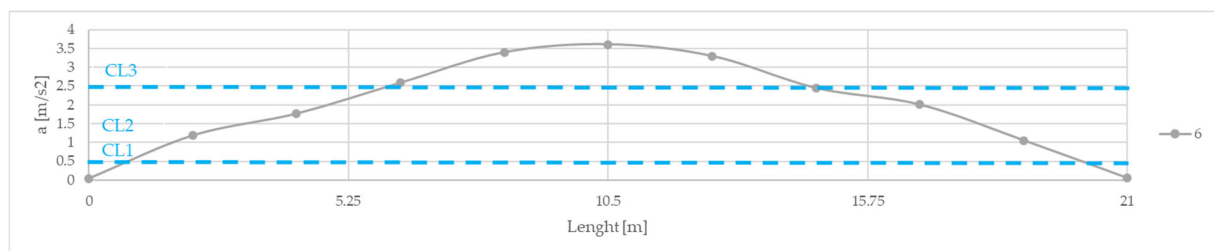


Figure 23. Distribution of peak vertical accelerations along the span during synchronous running of 1, 3, 6 and 9 pedestrians compared with comfort limit levels CL1, CL2 and CL3.

Horizontal accelerations, measured at sensor a6y, remained very low for all scenarios, not exceeding 0.05 m/s^2 , thus confirming that the pedestrian-induced response was dominated by vertical vibration modes.

The distribution of peak vertical accelerations along the span is presented in Figures 20–23 for different pedestrian loading scenarios. The results correspond to the values summarized in Table 6 and are shown together with the comfort limit levels CL1, CL2 and CL3, defined according to the Sétra and HiVoSS guidelines.

4. Discussion

4.1. Comparison Between Experimental and Numerical Identification Results

Experimental results demonstrate that rotational measurements provide valuable complementary data to conventional translational acceleration records within all dynamic identification techniques. The natural frequencies and mode shapes identified from the experimental tests (PP, FDD, SSI) were compared with the results of the two FEM model variants described in Section 2.6. The comparison included:

- (i) the unloaded model, representing the structure without additional external mass, and
- (ii) the loaded model, in which the mass of the large modal exciter was included at locations CS6 and CS8.

The analysis considered both translational and rotational degrees of freedom.

For all investigated modes below 40 Hz, natural frequencies identified from both translational and rotational signals exhibited a high level of agreement across all applied methods, with relative deviations never exceeding 1%. For the unloaded structure (Table 2), the Mean Absolute Relative Error (MARE) was 4.42% for the PP-ex_{small} method, 3.46% for FDD, and 3.35% for SSI. The best agreement ($RD < 2.5\%$) was observed for modes V_1 , V_2 , VT_3 , and V_3 , while the largest discrepancies occurred for mode VT_2 ($RD \approx 13\text{--}15\%$) and V_4 ($RD \approx 5\text{--}6\%$). For the fundamental bending modes V_1 , V_2 , and V_3 , the mean RD was below 4% for PP-ex_{small} and close to 1% for FDD and SSI.

When the mass of the large exciter was included in the FEM model (Table 3), the MARE was 4.18% for PP-ex_{large6} and 3.52% for PP-ex_{large8}. The influence of the exciter's location was most evident for the torsional mode T_1 , where the placement in CS8 yielded an exact frequency match, while for most other modes the difference between the two locations was below 3%. For the fundamental bending modes V_1 , V_2 , and V_3 , the mean RD was 1.80% (CS6) and 1.70% (CS8).

Overall, both FEM variants produced natural frequencies in very good agreement with the experimental results, with the best matches for the fundamental vertical modes and moderate RD values for certain higher-order modes, especially VT_2 .

For a more structured interpretation of the identified modes and their correspondence with the numerical predictions, both the experimental and numerical results were classified into modal groups based on the number of half-waves observed along the span. Based on the translational degrees of freedom, the single half-wave group (Group 1) consists of T_1 and V_1 , representing the fundamental mode with a single half-wave along the span. The subsequent groups include the double half-wave group $VT_1\text{--}V_2$ (Group 2), the triple half-wave group $VT_2\text{--}VT_3\text{--}V_3\text{--}VT_4$ (Group 3), and the quadruple half-wave group $V_4\text{--}VT_5$ (Group 4). These higher-order modes correspond to increasingly complex deformation patterns with two, three, and four half-waves, respectively. This grouping is reflected in the block structure of the MAC matrices (Figures 13 and 14) and provides a consistent basis for the comparisons that follow.

The translational mode-shape plots (Figure 15) clearly reveal the previously defined modal groups based on the number of half-waves, with consistent grouping observed across all identification techniques. The boundaries between Groups 1–4 are distinct, and the deformation patterns within each group are generally reproduced with high accuracy. Minor deviations appear in the higher-order modes, where increased shape complexity and reduced measurement sensitivity can lead to differences between experimental and numerical results. A notable case is the VT_2 —fifth identified mode in the experimental results, which, for all methods except SSI, displayed a shape with two half-waves instead of the three predicted by the FEM model. This mismatch in the number of half-waves corresponds to the relatively high frequency error for this mode and explains the reduced MAC values observed in Table 4. The SSI method, which retained the correct three half-wave shape, produced a noticeably higher MAC for this mode (0.84) compared to other identification techniques.

When comparing the three identification techniques for translational DoFs (Table 4), all methods achieved high correlations for the fundamental and mid-frequency modes. SSI showed the highest overall agreement ($\text{Mean}_{\text{all}} = 0.954$), while FDD achieved very high agreement ($\text{Mean}_{\text{all}} = 0.948$) and provided the best match for the fundamental vertical modes ($\text{Mean}_{V_1\text{--}V_2\text{--}V_3} = 0.980$). SSI was also the only method to correctly capture the three-half-wave shape of VT_2 , yielding a noticeably higher MAC for this mode than all PP variants and FDD.

For the Peak Picking (PP) methods, the results varied slightly between the three configurations. PP-ex_{large6} achieved the highest overall MAC among the PP variants

($\text{Mean}_{\text{all}} = 0.961$), with particularly good agreement for the fundamental modes. PP-ex_{small} followed closely ($\text{Mean}_{\text{all}} = 0.940$), showing only minor degradation for some higher-order modes. PP-ex_{large8} produced slightly lower correlations ($\text{Mean}_{\text{all}} = 0.933$), mainly due to the reduced MAC for VT₂ (0.670), which coincides with the largest relative difference in frequency ($\text{RD} \approx 15\%$) among all translational modes.

The rotational mode shape plots (Figure 16) confirm the grouping of modes based on the number of half-waves, consistent with the translational case. However, for some higher-order modes, experimental results exhibited local phase differences or amplitude variations not present in the FEM predictions, contributing to lower MAC values.

When comparing the three identification techniques (Table 5), FDD achieved the highest overall agreement for rotational DoFs, with $\text{Mean}_{\text{all}} = 0.886$ and the best match for the fundamental vertical modes ($\text{Mean}_{\text{V1-V2-V3}} = 0.919$). The SSI method also performed strongly ($\text{Mean}_{\text{all}} = 0.881$), particularly for the low- and mid-frequency modes, and showed stable results across the mode range. Among the PP variants, PP-ex_{large8} performed best ($\text{Mean}_{\text{all}} = 0.864$), followed by PP-ex_{large6} (0.835), while PP-ex_{small} showed a clear drop in agreement (0.659) due to lower correlations for higher-order modes. Nonetheless, rotational data offered a unique benefit: even when MAC coefficients for VT₂ were low (≈ 0.667), all rotational methods correctly classified the mode within the three half-wave groups, whereas four of five translational approaches did not. This illustrates the complementary nature of angular velocity data, which can guide classification even when conventional correlation metrics appear weak. The results confirm that the inclusion of gyroscopic data not only enhances the interpretation of coupled vertical–torsional modes but also offers a low-cost and resource-efficient complement to conventional accelerometric measurements, promoting more sustainable practices in experimental bridge diagnostics.

From a practical perspective, methodological trade-offs must also be considered. Peak Picking is the simplest approach and can provide accurate estimates for fundamental frequencies, but it requires external excitation. In this study, the use of a lightweight exciter facilitated field deployment but resulted in lower accuracy compared to OMA methods. Conversely, a large exciter yielded excellent frequency agreement in PP analyses; however, the total system mass of approximately 300 kg imposes substantial logistical challenges, including transportation, installation, and safety constraints, which limit its applicability in typical field investigations. By contrast, OMA methods (FDD and SSI), though more demanding in terms of instrumentation quality and noise control, proved more robust and versatile, making them the preferred option for comprehensive modal identification in the presence of complex vibration patterns.

The proposed methodology, based on the combined use of MEMS gyroscopes and accelerometers, is primarily intended for experimental modal identification using PP, FDD and SSI techniques. Since these techniques rely on the dynamic response of the structure rather than on the type of excitation, the approach is transferable to a broad range of lightweight pedestrian bridges, including steel, aluminium and composite footbridges with low natural frequencies and limited damping.

The measurement parameters of the LPY403AL gyroscopes ($\pm 30^\circ/\text{s}$ range, 140 Hz bandwidth and high sensitivity) ensure that rotational velocities associated with global modal shapes remain above the noise floor for amplitudes typical of slender pedestrian bridges. This makes the sensors suitable for identifying both vertical and torsional modes, including coupled bending–torsional behaviour, in structures of different geometries and span lengths.

4.2. Assessment of Pedestrian Comfort Criteria

The field tests revealed that the structure exhibits several natural frequencies that fall within the range of typical pedestrian-induced excitations. In particular, the fundamental mode of vertical vibration was identified below the threshold of 5 Hz, which, according to Eurocode 1, requires a detailed dynamic assessment of pedestrian comfort and safety. Although the Sétra and HiVoSS guidelines classify frequencies above 2.6 Hz as associated with a low risk of resonance, the experimental results demonstrate that resonance effects may still be significant, especially when groups of pedestrians act in synchrony.

Subjective feedback obtained during testing confirmed that participants consistently reported perceivable vibrations while walking and running. Some reported mild discomfort even when standing on the bridge, while others were moving. These experimental observations were fully aligned with prior accounts from local residents and regular users of the footbridge, who had previously expressed concerns about noticeable vibrations during everyday use.

A direct comparison between the measured accelerations and the normative comfort thresholds confirms that the structure does not comply with several serviceability criteria under typical pedestrian loading conditions. According to Sétra and HiVoSS, the limit values for vertical acceleration associated with comfort classes CL1, CL2 and CL3 are 0.5 m/s^2 , 1.0 m/s^2 and 2.5 m/s^2 , respectively. While single-pedestrian walking remains within CL1, the responses induced by unsynchronised groups of 3 walkers already exceed the CL2 limit, and synchronous walking of six pedestrians produces peak accelerations slightly above the CL3 threshold. The most demanding loading case classified as a vandal-induced loading, synchronous jumping—reaches 3.61 m/s^2 , clearly exceeding all recommended comfort limits and classifying the structure as a “lively” footbridge. For horizontal modes, the measured accelerations remain very low ($<0.05 \text{ m/s}^2$), well below the perception thresholds defined in Eurocode 1 (0.2 m/s^2) and Sétra/HiVoSS ($0.1\text{--}0.15 \text{ m/s}^2$). This confirms that the dynamic discomfort is governed exclusively by vertical vibrations. The distribution of peak accelerations along the span (Figures 20–23) indicates a critical zone extending from one-quarter to mid-span, where the largest amplitudes were consistently recorded. These findings highlight that not only synchronised actions, but also the presence of multiple pedestrians moving freely, may induce vibration levels beyond the strictest comfort criteria.

Comparable levels of pedestrian-induced vibration have been reported for several lively footbridges in the literature. For a steel suspension bridge over the Serio River, peak vertical accelerations reached about 2 m/s^2 for walking groups and up to 0.48 g ($\approx 4.8 \text{ m/s}^2$) for four joggers [5]. For the fully GFRP cable-stayed Aberfeldy footbridge, single-pedestrian passages led to peak vertical accelerations of about 1.9 m/s^2 (walking at 1.64 Hz) and 3.7 m/s^2 (running at 2.71 Hz), exceeding several vertical comfort limits despite acceptable lateral performance [10]. The steel arch-girder Kolibki footbridge in Gdynia shows a similarly lively behaviour: peak vertical accelerations induced by a single pedestrian exceeded the HiVoSS CL1 and CL2 comfort limits even though the first vertical bending frequency of the deck is $f \approx 5.28 \text{ Hz}$, i.e., above the classical 5 Hz threshold [11]. Finally, for the Lambro footbridge near Milano, with a fundamental bending frequency of 1.75 Hz, full-scale tests on groups of up to 12 pedestrians yielded peak vertical accelerations at mid-span of the order of $0.9\text{--}2.3 \text{ m/s}^2$, governed by the first bending mode and maximised at mid-span [18]. These independent case studies confirm that vertical accelerations in the range of approximately $1\text{--}4 \text{ m/s}^2$ under walking or running are typical for slender footbridges and frequently lead to exceedance of serviceability criteria.

5. Conclusions

This study presented a comprehensive experimental–numerical investigation aimed at evaluating the usefulness of angular velocity measurements for the dynamic identification and serviceability assessment of a pedestrian footbridge. The results highlight several key findings concerning modal properties, vibration performance and the contribution of angular velocity measurements to structural assessment:

- Combining rotational velocities with acceleration measurements improved modal identification, particularly for coupled vertical–torsional modes. Rotational responses enabled clear recognition of mode VT_2 , which remained partly obscured when using accelerations alone.
- The applied identification methods—Peak Picking (PP), Frequency Domain Decomposition (FDD) and Stochastic Subspace Identification (SSI)—provided consistent modal parameters for translational DoF with high correlation to the FEM model. FDD and SSI gave the most accurate estimates for rotational DoF, while PP performed well under strong excitation but was less reliable for light excitation.
- MEMS-based gyroscopic sensors proved to be an effective and low-cost complement to acceleration-based monitoring.
- The serviceability assessment confirmed the resonance-prone behaviour of the footbridge: its fundamental vertical frequency of 3.1 Hz falls within the critical pedestrian excitation range. Dynamic tests recorded peak accelerations up to 3.6 m/s^2 during tests with pedestrians; while these levels do not threaten structural integrity, they may cause perceptible discomfort, especially for groups of users. The observed vibrations indicate the need for mitigation measures such as tuned mass dampers.
- Future research should advance rotational sensing within multi-scale SHM frameworks and improve hybrid identification procedures to enhance the accuracy and predictive capability of numerical models for pedestrian bridges.

Overall, this study demonstrates that angular velocity measurements significantly enhance dynamic identification. As low-cost and non-invasive tools, they also support sustainable and resource-efficient management of existing bridge infrastructure.

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Abbreviations

The following abbreviations are used in this manuscript:

CFA	Continuous Flight Auger
CS6	Cross-section 6
CS8	Cross-section 8

DoF	Degree of Freedom
FDD	Frequency Domain Decomposition
FRF	Frequency Response Functions
MARE	Mean Absolute Relative Error
MEMS	Micro-Electro-Mechanical-Systems
OMA	Operational Modal Analysis
PP	Peak Picking
PSD	Power Spectral Density
RD	Relative Difference
SSI	Stochastic Subspace Identification
SVD	Singular Value Decomposition

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