

# **Evaluation of Human-Structure Interaction Effects on Footbridges Using Real-Time Hybrid Simulation**

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**DOCTORAL THESIS**

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## **Abstract**

In recent years, the literature has highlighted a global synergy within societies focused on the urban regeneration of their civil infrastructure. This new perspective has led to an enhancement of the technical and aesthetic requirements demanded in the comprehensive management of civil structures. Additionally, there has been a rapid development of construction materials that are increasingly stronger and lighter within the Construction Industry. These developments have become a focal point for civil infrastructure, such as bleachers, slabs, staircases, and especially pedestrian bridges, which are being designed to be slenderer and more flexible, making them susceptible to excessive vibrations due to dynamic loads. Among the wide range of actions affecting pedestrian bridges, human walking is identified as the most influential and recurring dynamic load impacting the structural serviceability concerning vibrations. One of the most representative cases describing this dynamic sensitivity occurred with the Millennium Bridge in London over the River Thames in 2000, where excessive lateral vibrations were observed during its inauguration due to the coupling between the dynamic behavior of the structure in this direction and the pseudo-synchronized action of pedestrian-induced loads. This situation sparked significant interest within the scientific community to determine, quantify, and understand the causes of this phenomenon, generally known as Human-Structure Interaction (HSI) effects. As a result, the study of serviceability under vibrations due to these dynamic effects has emerged as one of the key challenges to be addressed by structural engineering in the coming years.

Research advancements on the evaluation of HSI effects on pedestrian bridges have typically concentrated on two major fields: Numerical Modeling and Experimental Testing. The field of numerical modeling has focused on simulating the effects of this phenomenon, generally through the development of simplified biomechanical models of human walking and analytical representations of the structure. However, various researchers agree that, to date, no model has been correctly formulated and experimentally validated to simulate HSI effects, highlighting the need for more practical tests to feedback and calibrate these models. On the other hand, experimental tests have been oriented towards laboratory trials on instrumented surfaces or treadmills, as well as in-situ tests on pedestrian bridges. Although experimental evaluations have proven to be of great investigative value, they present challenges such as short data collection periods, limited documented information on the kinetic and kinematic behavior of pedestrians, limited data on the dynamic behavior of the structure, and constraints on pedestrian movement.

Due to the limitations posed by numerical modeling and the challenges in the experimental evaluation of HSI effects on pedestrian bridges, this research is structured around an innovative cyber-physical evaluation technique known in the literature as Real-Time Hybrid Simulation (RTHS). This testing methodology allows for substructuring the HSI effects into two components: a Numerical Substructure (NS), corresponding to analytical models of pedestrian bridges, and an Experimental Substructure (ES), composed of active walking subjects. Both substructures were interconnected at each time step  $\Delta t$  through a transfer and instrumentation system, established by the Integral Testing Framework for HSI at the Universidad del Valle. The proposed evaluation of HSI effects through RTHS represents an innovative field of analysis that enhances the benefits of both experimental and numerical tests, mitigating the challenges associated with their individual evaluation limitations.

**Keywords:** Footbridge, Human-Structure Interaction, Real-Time Hybrid Simulation, Dynamic Substructuring.

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## Chapter No.1. Research problem

### 1.1 Description.

Currently, the hypothesis of an exponential regeneration of urban infrastructure at a global level is increasingly supported, where key sectors with high social interest in communities are interconnected or developed through the construction of civil infrastructure with notable architectural requirements and demanding engineering designs [1], [2]. In order to achieve this new urban perspective, accelerated advancements and progressive integrations of structural materials with superior physical-mechanical properties compared to those conventionally used in the construction industry have been promoted, resulting in increasingly lighter structures with optimal design strengths. However, this has triggered a condition of dynamic sensitivity in these structures, as their dynamic behavior is generally associated with low frequencies (approximately 0.5-10.0 Hz), which coincide with the natural frequencies, and some harmonics, of primary human activities (such as walking, running, among others). The complexity of this situation lies in the fact that these types of dynamic loads are the most frequent and recurrent on these structures, having the potential to affect their structural serviceability [2]-[10].

This limitation of serviceability capacity due to dynamic sensitivity under the influence of anthropic loads has been observed in structures such as: grandstands or bleachers [11]-[15], slabs [16]-[22], staircases [23]-[27], and particularly in pedestrian bridges [1], [28]-[39]. This latter type of structure has been the most affected due to its inherent function of spanning large distances, relatively light masses, and characteristically low damping. Furthermore, improvements in material properties and the increasing engineering demands in their design have led these bridges to become increasingly slender and flexible, making them more susceptible to vibrations caused by human walking loads—an anthropic activity that is more frequent on these structures [3], [40]-[43]. This pedestrian-structure interaction has been formally documented since the late 20th century in various pedestrian bridges: Toda-Park Bridge, Toda-Japan [28]; Solferino Bridge, Paris-Francia [44]; Long Span Steel bridge, Changi Airport-Singapore [45], [46]; Millennium Bridge, London-United Kingdom [33], [47], [48]; Brooklyn Bridge, New York-USA [49]; Pedro e Inês Bridge, Coimbra-Portugal [32], [50]; among others [29], [31], [51], which are illustrated in *Figure 1*.

Although Dallard et. al. [33] were the first to describe the events of dynamic coupling that occurred on June 10, 2000, during the inauguration of the Millennium Bridge, between the structure and the pedestrians crossing it, which caused excessive lateral vibrations of up to 7.0 cm and led to its complete closure the following day, it was the review work by Zivanovic et al. [3] that thoroughly compiled this phenomenon of Bridge-Pedestrian interaction observed on the Millennium Bridge, the load tests conducted, and its subsequent intervention for controlling unwanted vibrations. This set a precedent that caught the attention of the scientific community by defining this dynamic relationship between Structure and Pedestrian and established the events on the Millennium Bridge as an international reference for the problem of vibrations in civil structures [3], [27], [37], [52]-[58]. The study of pedestrian-induced vibrations on footbridges has been growing in recent years and is now entering its third decade of research, aiming to define the existing coupling between Structure and Pedestrians, to consolidate this



phenomenon as a serviceability issue, and to address the limitations in the design criteria of these structures [31], [59]-[65].

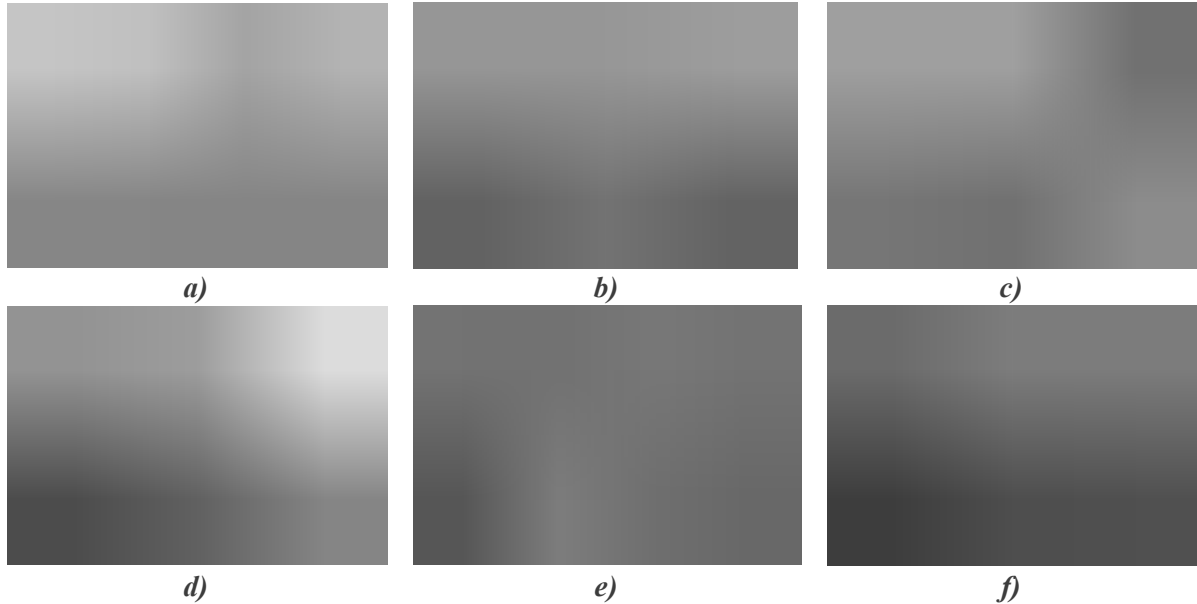


**Figure 1.** Pedestrian bridges with a dynamic Structure - Pedestrian ratio. *a) Toda - Park Bridge, b) Solferino Bridge, c) Millennium Bridge, d) Changi Airport Bridge, e) Brooklyn Bridge, f) Pedro e Inês Bridge.*

The problem of pedestrian-induced structural vibrations is commonly known in the literature as Human-Structure Interaction (HSI) effects, with studies focusing on the dynamic effects of the structure on human activity, and vice versa [8], [37], [58], [66]-[73]. In pedestrian bridges, the primary investigated and documented activity is human walking due to its influence and recurrence on these structures, although other activities such as vandal-induced loads have also been detailed [20], [62], [74], [75]. Human walking is defined as an active load on pedestrian bridges and given its phases of single and double support during movement, its frequency content is related to frequencies between 0.7-1.3 Hz and 1.7-2.3 Hz for lateral and vertical loads, respectively [59], [76], [77], [78]. This formed the basis for addressing HSI effects as a resonance problem caused by synchronization between the natural frequencies of human walking and those related to the bridge's dynamic behavior; these effects are known as “Lock-In” and describe changes in walking dynamics due to coupling with the vibration levels experienced by the structure when interacting with these anthropic loads [1], [26], [79]-[85]. However, focusing solely on HSI as a resonance problem due to frequency tuning overlooks and neglects important aspects of this phenomenon. Recent research in this field has divided HSI into two simultaneously occurring components: Human-to-Structure Interaction (H2SI) and Structure-to-Human Interaction (S2HI) effects [29], [38], [86]-[88]. H2SI describes the effects of human walking on the dynamic behavior of the pedestrian bridge, specifically the alteration of the structure's dynamic properties, such as natural frequency and the addition of damping to the system. However, human walking, being an active load, should not be considered a mere imposed load or damping, but rather as a highly nonlinear dynamic system that directly affects the behavior of these structures [6], [18], [19], [24], [29], [89]-[94]. On the other hand, S2HI details the influence of structural vibrations on the kinetic and kinematic



changes in human walking; pedestrians' reactions to structural vibrations are complex, as humans themselves are highly anisotropic beings [95]. According to Lippert et al. [96], not only do pedestrians react differently under the same conditions, but the same individual exposed to the same vibrations at different times may exhibit different behaviors [3], [97]. This is now known as inter- and intra-individual variability in humans and constitutes a crucial basis for understanding the serviceability of vibration-induced issues in pedestrian bridges. However, it is not only the randomness in walking that conditions the serviceability of these structures, but also the high levels of vibration perception by pedestrians [73], [90], [98]-[103]. This affects the technical limitations proposed for controlling vibrations in pedestrian bridges and the development of comprehensive models to evaluate HSI effects.

Various studies have focused on the numerical modeling of Human-Structure Interaction (HSI) in pedestrian bridges to simplify the evaluation of its effects. These models are characterized by representing the structure as a 1-degree-of-freedom (1-DOF) dynamic system or, in some cases, as a continuous system using partial differential equations [43], [88], [104], [105]. On the other hand, the human walking system has been represented in two main groups: individual pedestrians and crowds. Individual pedestrian models have primarily been used to simulate ground reaction forces (GRF) and, in recent years, have been incorporated as active components of HSI effects using inverted pendulum models, biomechanical models with mass-spring-damper combinations, and bipedal biomechanical models [43], [58], [64], [71], [106]-[117]. Crowd models have gained significant attention in recent years due to their ability to allow Human-Human Interaction (HHI) and include probabilistic or stochastic loading effects on pedestrian bridges, making them robust models capable of approximating the real effects of HSI [10], [35], [58], [73], [81], [93], [102]-[104], [116]-[126]. Despite the significant advances in modeling the effects of HSI with individual pedestrians and crowds, many researchers conclude that more robust models are still needed to accurately reproduce the complex dynamics present in this interaction, and these should be developed through experimental testing [29], [38], [57], [68], [121], [127]-[131].

Experimental tests aimed at determining the effects of Human-Structure Interaction (HSI) on pedestrian bridges are now formally entering their third decade of research, driven by the need for experimental data to support the calibration and development of numerical models capable of simulating these effects. The first studies characterizing human walking as an anthropic load that could impact civil infrastructure were conducted by Galbraith [132] and Andriacchi [76], and later compiled with other investigations by Zivanovic [3]. Experimental tests focused on HSI effects, like numerical models, are divided into two key groups: laboratory tests and in-situ tests. Laboratory tests have been conducted to characterize human walking in a controlled environment. The primary investigations on rigid surfaces have utilized instrumented treadmills equipped with load cells or force plates, allowing the collection of extensive, uninterrupted data on these anthropic loads [4], [79], [101], [133]-[135]. For studying "Lock-In" effects, instrumented treadmills that oscillate sinusoidally, primarily in the lateral direction, have played a crucial role in determining the coupling levels of human walking loads with induced movements, perception thresholds reached by test subjects, and the variability in walking dynamics [54], [79], [100], [101], [103], [135]-[137]. The capture of human walking kinematic data in laboratories has increased due to technological advancements in optical systems, with significant research focusing on estimating walking kinetics based on this kinematic data [35], [56], [79], [89], [121], [136], [138], [139]. The latest series of laboratory studies involves benchmark models of pedestrian bridges developed worldwide to contribute



to open research and facilitate comparisons between research groups; universities such as Warwick, Monash, and T. Darmstadt have developed such structures for studying HSI effects [29], [34], [35], [129], [139]-[145]. On the other hand, in-situ tests have been characterized by collecting precise information on the structural dynamics under HSI effects, although with reduced data collection on pedestrians; nevertheless, this information has been critical for the characterization of proposed HSI models [35], [38], [127], [130], [135], [146]-[154]. In general, operational condition tests on pedestrian bridges unfortunately involve few pedestrians, short data collection periods, and limited kinetic and kinematic data on pedestrian behavior, along with restricted information on the dynamic behavior of the structure [29]. In contrast, laboratory tests can create an environment that forces pedestrians to alter their perceptions while walking, thus affecting the results.

Due to the limitations presented by numerical models and the constrained information acquired during experimental tests of HSI effects, this research proposes the use of a novel, cost-effective physical-cyber evaluation technique that has emerged in recent years, known in the literature as Real-Time Hybrid Simulation (RTHS) [155]-[159]. This technique combines numerical and experimental analyses, leveraging the benefits of both, and significantly improves upon purely numerical or experimental approaches. RTHS divides the evaluated dynamic system through dynamic substructuring into a Numerical Substructure (NS), which corresponds to the portion of the system that can be accurately modeled, and an Experimental Substructure (ES), which contains the portion of the system with high non-linearity or of greater interest for evaluation. These two substructures are interconnected via transfer and instrumentation systems to enable constant feedback at each time step [157], [160]-[171]. For this study, the NS consisted of numerical models of reference pedestrian bridges, while the ES will involve test subjects performing human walking. Both substructures were interconnected through the Comprehensive HSI Multiaxial Testing Framework, which was used as the transfer and instrumentation system.

## 1.2 Formulation of the research problem

All of the above motivated us to conduct a comprehensive study of IHE on pedestrian bridges through the implementation of SHTR, which led us to formulate the following research problem:

What are the characteristics of Human-Structure Interaction effects on pedestrian bridges evaluated by Real-Time Hybrid Simulations?

## 1.3 Research objectives.

### 1.3.1 General Objective

Evaluate the effects of Human-Structure Interaction on pedestrian bridges using Real-Time Hybrid Simulations.



### 1.3.2 *Specific objectives*

- Develop the experimental set-up of the IHE Integrated Test Framework to implement SHTR.
- Develop reference pedestrian bridge models as the Numerical Substructure (NS)
- Conduct an experimental campaign of human walking tests on reference pedestrian bridges using RTHS.
- Evaluate the effects of HSI on reference pedestrian bridges.
- Quantify the performance of RTHS.

## 1.4 Justification of the research problem

Modern civil structures such as grandstands, staircases, slabs, and especially pedestrian bridges, have incorporated construction materials with improved mechanical properties and have faced increasing technical requirements in cities with innovative projections. This has resulted in a growing number of increasingly slender and flexible structures, heightening their dynamic sensitivity to vibrations induced by pedestrian activities such as jumping, running, swaying, and, most frequently, walking [6], [29], [59], [116], [125], [130], [136], [172]-[176]. Consequently, predicting the effects of Human-Structure Interaction (HSI) in pedestrian bridges has become an important design consideration for evaluating the serviceability of vibrations under the operational conditions of the structure. However, there is considerable uncertainty in modeling this delicate phenomenon, making it one of the current challenges in Structural Engineering, and one that is expected to be addressed in the coming decade [35], [55]. This is further supported by the fact that, despite numerous models developed to date, there is no model that faithfully captures the effects of HSI [18], [68], [177]-[179]. Even with international standards that include various types of models to simulate these effects, significant underestimations and overestimations of structural responses have been recorded, leading to unreliable evaluations of their performance under vibration [7], [59].

It is now widely accepted that structural vibrations caused by anthropic loads primarily constitute a serviceability issue, and in limited cases, a safety concern [8], [180]. This is due to the fact that human perception is sensitive to low-intensity vibrations, with studies by Bachmann et al. [181], [182] establishing that human sensitivity varies around  $1e-3$  m. As a result, pedestrians on footbridges are positioned as the primary receivers of vibrations, and they govern the serviceability capacity of the structure. This has sparked debate since the early stages of research, with authors such as Walley et al. [183] advocating for the need to minimize structural vibrations based on design criteria to control serviceability limits. However, other researchers, such as Leonard et al. [97], quickly refuted this idea, and years later Ortiz et al. [184] supported their position, asserting that it is economically unfeasible to reduce vibrations to levels imperceptible to pedestrians, as the associated costs would render such projects unviable.

The interest scientific of the community in understanding, quantifying, and modeling vibration problems due to HSI effects in pedestrian bridges, as well as establishing design guidelines to mitigate this issue, has been divided into two main approaches. The first approach focuses on calculating or experimentally obtaining the dynamic response of the pedestrian bridge in service, and based on this information,



determining whether it meets acceptable vibration limits for pedestrians. Some researchers and international organizations that have adopted this first evaluation concept include: The American Association of State Highway and Transportation Officials [185], BS 5400: Part 2 and BS 5400: BD 37/01 [186], The Ontario Highway Bridge Design Code [187], Eurocódigo 5 [188], [189], The Hong-Kong Structures Design Manual for Highways and Railways [190]. The second approach aims to prevent dynamic coupling between the frequency content of the pedestrian bridge and the incident anthropic loads; proponents of this methodology include Mason et al. [191], The American Guide Specifications [192], Yoshida et al. en [193], and Pimentel et al. en [194]. In an effort to better understand the intrinsic dynamics of HSI effects, some organizations and researchers have addressed this issue and proposed design procedures to reduce vibrations from these effects. Among them are the French SETRA guide [195], the European research project SYNPEX with the HIVOSS guide [30], the UK Highway Agency [196], and studies conducted by Heinemeyer et al. [197], [198].

The current Colombian standards that define the design, analysis, and construction criteria for structures face a research deficit regarding the dynamic effects of HSI, which can impact both new and existing structures. Only the Colombian Seismic-Resistant Design and Construction Code (NSR-10) [199], in its section B.4.7, refers to certain recommendations concerning the frequency content limits of structures that may be affected by anthropic loads, neglecting relevant aspects of HSI evaluation, such as the influence of pedestrians on changing the dynamic properties. Additionally, the Colombian Seismic-Resistant Design Code for Bridges (CCP-14) [200] does not include any specifications regarding the evaluation of anthropic loads on pedestrian bridges. This local outlook is concerning, especially considering that, despite international advancements, there is currently no analysis methodology or numerical model capable of accurately simulating HSI effects on pedestrian bridges, much less one that can be validated with experimental data [18], [68], [95], [178]. However, experimental data also present challenges, such as the lack of documented kinetic and kinematic behavior of pedestrians, limited information on the dynamic behavior of structures, and the conditioned behavior between pedestrians, which reduces the precision of numerical model development and calibration [29], [35], [128]. All these issues highlight the need for increasingly robust experimental tests that contribute to improving the development of numerical models capable of simulating HSI effects [6], [8], [34], [99], [101], [201].

In Colombia, some contributions have been made regarding the study of HSI effects, mainly in the pathological evaluation of the dynamic sensitivity of pedestrian bridges [40], [154], [184], [202]. While these contributions have been relevant to the local understanding of HSI, they represent an initial step toward aligning with the global synergy in developing more robust experimental tests and numerical models for evaluating HSI. In this regard, to contribute to the need for more robust experimentation and modeling of HSI, this research proposes the implementation of the innovative physical-cyber technique of Real-Time Hybrid Simulation (RTHS), which allows for dynamically substructuring the HSI problem in pedestrian bridges into two main components [155], [156], [157], [158], [159], [203]. The first component is the Numerical Substructure (NS), corresponding to an analytical model of the pedestrian bridge, as proposed by several researchers [43], [89], [104]. The second component is the Experimental Substructure (ES), representing the human system (test subject) characterized by its complex non-linearity and behavioral uncertainties [73], [94], [100], [102], [103]. Both components are interconnected at each time step through transfer and instrumentation systems [165], [204]-[206], as illustrated in *¡Error! No se encuentra el origen de la referencia..* This decoupling of the HSI problem in pedestrian



bridges into two substructured components has been approached numerically with positive results [41], [207], [208]. Although RTHS was initially proposed for evaluating the dynamic behavior of devices or structural elements of interest in civil structures, particularly buildings, during seismic events [156], [165], [171], [209], this technique has gradually expanded into other research fields in recent years [210], [211]. The potential impact of RTHS lies in its ability to simulate the operational conditions of structural systems affected by highly complex and nonlinear dynamic systems [212], [213]; in the development of an economical, efficient (space-time), and cost-effective methodology for evaluating highly nonlinear phenomena, yielding results comparable to large-scale testing [159], [167], [214], [215]; and in the evaluation of numerous possible experimental component scenarios or different configurations of the numerical component, a situation that is practically unfeasible with traditional experimental evaluation techniques [216]. Therefore, RTHS emerges as a versatile, efficient, and cost-effective alternative for evaluating HSI effects in pedestrian bridges, allowing full understanding of the NS (Pedestrian Bridge) behavior and acquisition of experimental kinetic and kinematic information from the ES (Human System) during walking. This physical-cyber approach aims to efficiently contribute to closing the research gap regarding the evaluation of pedestrian bridges subjected to HSI effects at a local level. It provides reliable numerical and experimental data as a first step toward establishing technical recommendations for the design and analysis of pedestrian bridges affected by anthropic loads, and consequently, their serviceability under vibrations. Finally, it seeks to lay the foundation for extending this technique to crowd loads and their human-human interaction phenomena.



*Figure 2. RTHS pedestrian bridge IHE assessment scheme*



## Capítulo 2. Marco de referencia

### 2.1 Antecedentes y Estado del arte: Interacción Humano-Estructura (IHE)

#### 2.1.1 Estudios iniciales de la Marcha Humana y acercamiento a la IHE.

Una de las primeras investigaciones en tratar formalmente el efecto de las vibraciones sobre la actividad humana fue la realizada por Lippert et. al. en [96] durante el año 1946, determinando experimentalmente la respuesta humana a los diferentes tipos de vibraciones que se encuentra comúnmente al someterse al tráfico de la calle, los ascensores, los barcos, los trenes, los automóviles y los aviones. Investigación que se complementó para los puentes peatonales con el trabajo reportado por Leonard et. al en [97], describiendo el nivel de tolerancia de los peatones a las vibraciones estructurales. Los resultados obtenidos a esta fecha ya generaban una fuerte especulación de la importancia de las cargas antrópicas como fuente fundamental de las vibraciones estructurales. Por ello, las investigaciones sobre sollicitaciones debidas a actividades humanas tomaron fuerte relevancia como la presentada por Galbraith et. al. en [132] en la cual se realizaron una serie de pruebas experimentales para medir el historial de la carga durante la marcha y los niveles de tiempo que se ejercen sobre el suelo. El propósito fue obtener información básica como parte de un estudio para mejorar detección de intrusos mediante la detección de señales sísmicas de las pisadas. Se sometieron tres sujetos de prueba con peso variable a diferentes ritmos de marcha, con y sin calzado, además de diferentes superficies; estas pruebas son mostradas en la *Figura 3*.

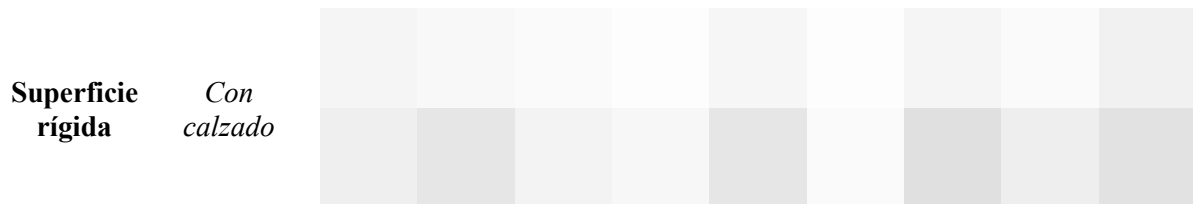




**Figura 3.** Fuerzas de reacción de suelo (GRF) debido a la marcha humana [132].

Andriacchi et. al en [76] presentaron la marcha de 17 sujetos de prueba sanos con edades entre 22-59 años, con una altura de 155-180 *cm* y peso 436-1050 *N*; y 16 sujetos con patología en su rodilla con edades entre 48-76 años, estatura de 154-178 *cm* y pesos entre 730-980 *N*, reportando las GFR témpora-espaciales con relación a la velocidad de marcha. El protocolo de ensayo incluyó por peatón hasta 20 recorridos por una pasarela de 10 metros a velocidades de marcha norma. Estos valores se establecieron como indicadores útiles para identificar anomalías en la marcha. Los resultados encontrados por este estudio se muestran en las **Figura 4** and **Figura 5**.

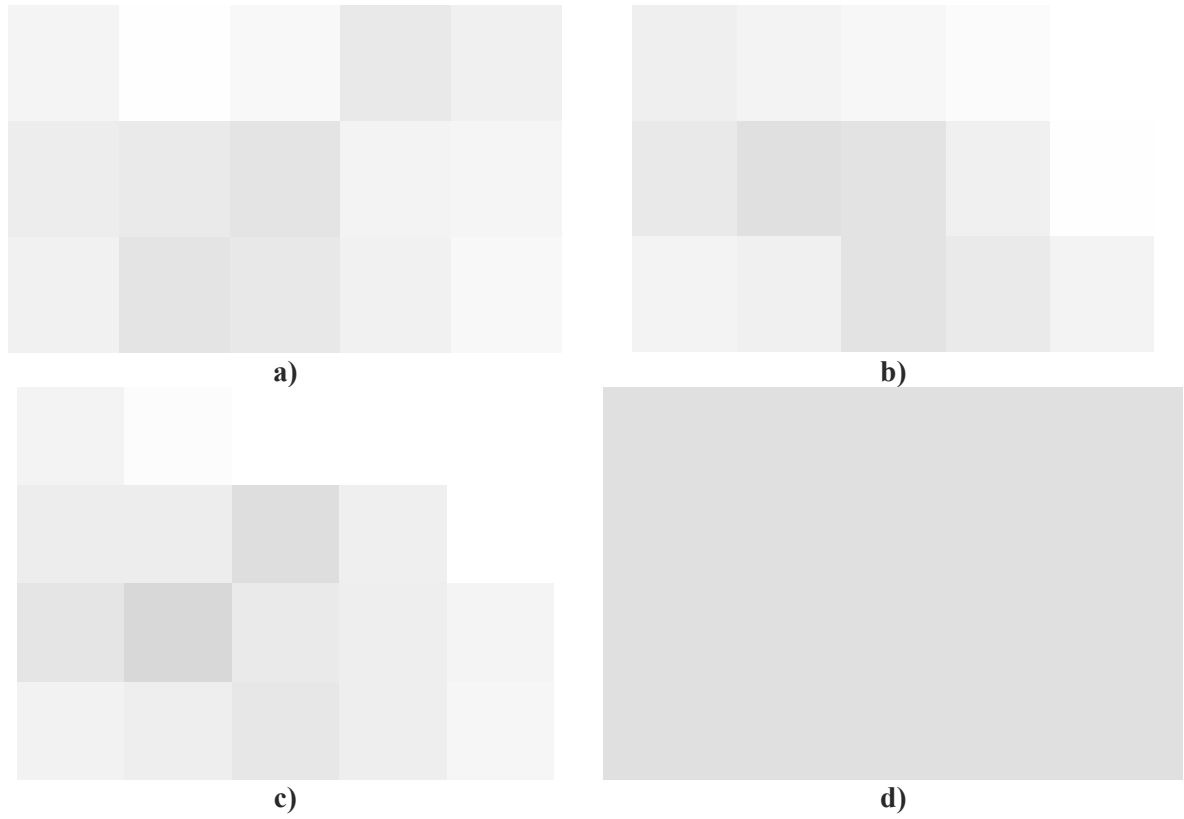


**Figura 4.** Comparación de la Marcha – Sujeto sano vs Discapacitado de rodilla [76]; **a)** Longitud de paso vs Velocidad; **b)** Tiempo de soporte vs Velocidad; **c)** Tiempo de balanceo vs Velocidad; **d)** Fuerza de amplitud de peatón vs Velocidad.

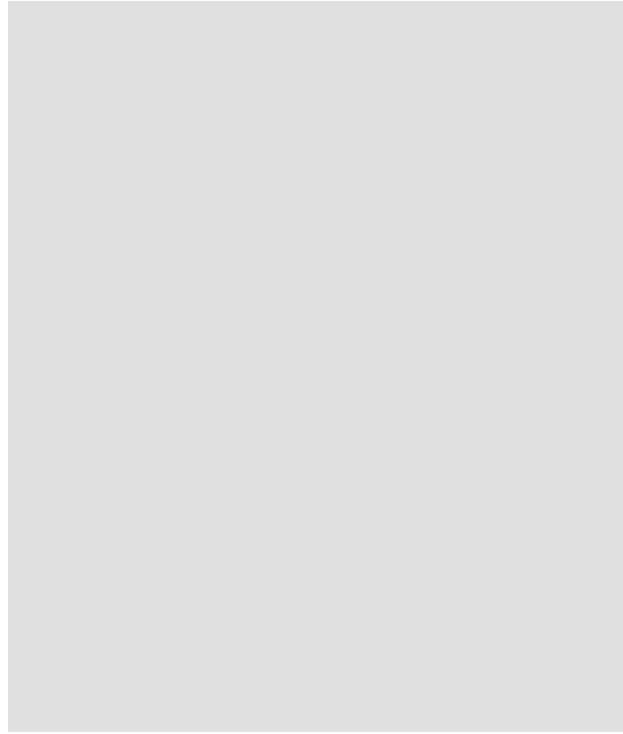


**Figura 5.** Ilustración de las tres componentes de GRF de la Marcha Humana: Lateral, Longitudinal y Vertical [76].

Bachmann and Amman en [182] y Newlands et. al. en [78] estudiaron y definieron estocásticamente el contenido frecuencial presente en la marcha y en el trote humano, en el sentido lateral y vertical. Estableciendo una base de análisis para la fundamentación de los efectos “Lock-in” en las estructuras susceptibles a vibraciones ante estas solicitaciones antrópicas, estos valores se resumieron en la **Tabla 1**.

**Tabla 1.** Frecuencias fundamentales de la Marcha, Trote y Corrida Humana [182], [78].

|                      | <i>Frecuencia de Paso (Hz)</i> | <i>Velocidad de Avance (<math>m/s</math>)</i> | <i>Longitud de paso (m)</i> | <i>Frecuencia Fundamental vertical (Hz)</i> | <i>Frecuencia Fundamental lateral (Hz)</i> |
|----------------------|--------------------------------|-----------------------------------------------|-----------------------------|---------------------------------------------|--------------------------------------------|
| <i>Marcha lenta</i>  | 1.70                           | 1.10                                          | 0.60                        | 1.70                                        | 0.85                                       |
| <i>Marcha normal</i> | 2.00                           | 1.50                                          | 0.75                        | 2.00                                        | 1.00                                       |
| <i>Marcha rápida</i> | 2.30                           | 2.20                                          | 1.00                        | 2.30                                        | 1.15                                       |
| <i>Trotar</i>        | 2.50                           | 3.30                                          | 1.30                        | 2.50                                        | 1.25                                       |
| <i>Correr</i>        | > 3.20                         | 5.50                                          | 1.75                        | > 3.20                                      | > 1.60                                     |

Las condiciones de aleatoriedad de las cargas inducidas durante la marcha humana [73], [90], [98], [101], ha llevado al entendimiento de su comportamiento como un mecanismo dinámico altamente no-lineal e influyente cuando se evalúan los efectos de la IHE en estructuras livianas y flexibles, como lo son los puentes peatonales. Debido a ello, se fundamenta la idea de que a principios de la primera década del siglo XXI el paradigma de evaluación de los efectos de la IHE sobre estas estructuras, en lo competente a la ingeniería estructural, ha cambiado a una visión de investigación integral más allá de un simple



problema de acoplamiento dinámico o a un desorientado estudio individual de marcha o estructura. Por lo cual, los estudios experimentales de la marcha humana orientados solo a su evaluación sobre superficies rígidas se han desligado del interés de la Ingeniería estructural.

### 2.1.2 Pruebas experimentales sobre puentes peatonales.

El caso primer caso registrado de problema de serviciabilidad en un puente peatonal debido a cargas inducidas por la marcha humana se menciona discretamente en [217] como un hecho ocurrido en el año 1154 d.C. cuando una de estas estructuras colapsara al paso de una multitud que pretendía acercarse a un arzobispo. Sin embargo, el trabajo reportado por Stevenson en [218] durante el año 1821 se configura como el caso reportado más antiguo en lo relacionado al problema de IHE en un puente peatonal, en este trabajo se describe el colapso del puente *Broughton* (Figura 6-a) en el reino unido debido al acoplamiento de las cargas dinámicas inducidas por 60 soldados al cruzar la estructura bajo una marcha sincronizada. Aunque en algunas investigaciones se han mencionado las condiciones de sensibilidad frente a cargas dinámicas en puentes peatonales, en especial las inducidas por la actividad humana [97], [181], [219], [220], [221], [222], fue Fujino et. al. en [28] en realizar la primera investigación orientada a los efectos de la IHE sobre una estructura moderna con materiales convencionales y en tránsitos de concurrencia típica de estas estructuras (Figura 6-b).

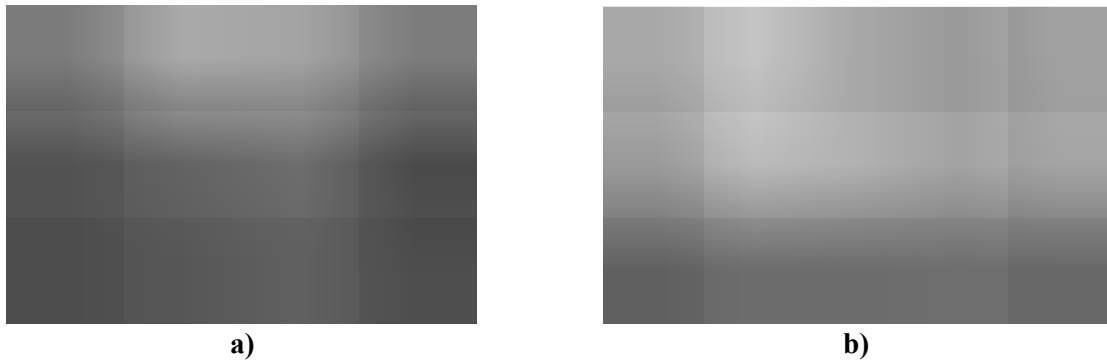


**Figura 6.** Puentes peatonales; **a)** Puente de Broughton [218], UK; **b)** Puente de Toda-Park [28], Japón.

En este último trabajo, Fujino et. al. informa la observación de vibración lateral de gran amplitud inducidas por la actividad humana en un puente peatonal con condiciones de congestión considerable. Además, se observa y analiza el movimiento de los peatones grabados por una cámara de video. Comprobando la sincronización de alrededor del 20.0% de los peatones los cuales iban incrementando paulatinamente su condición de resonancia. Cabe mencionar que su comportamiento dinámico se componía de una frecuencia natural lateral de 0.90 Hz, la cual queda dentro del rango probabilístico de contenido frecuencial de marcha lateral [78], [181], [182].

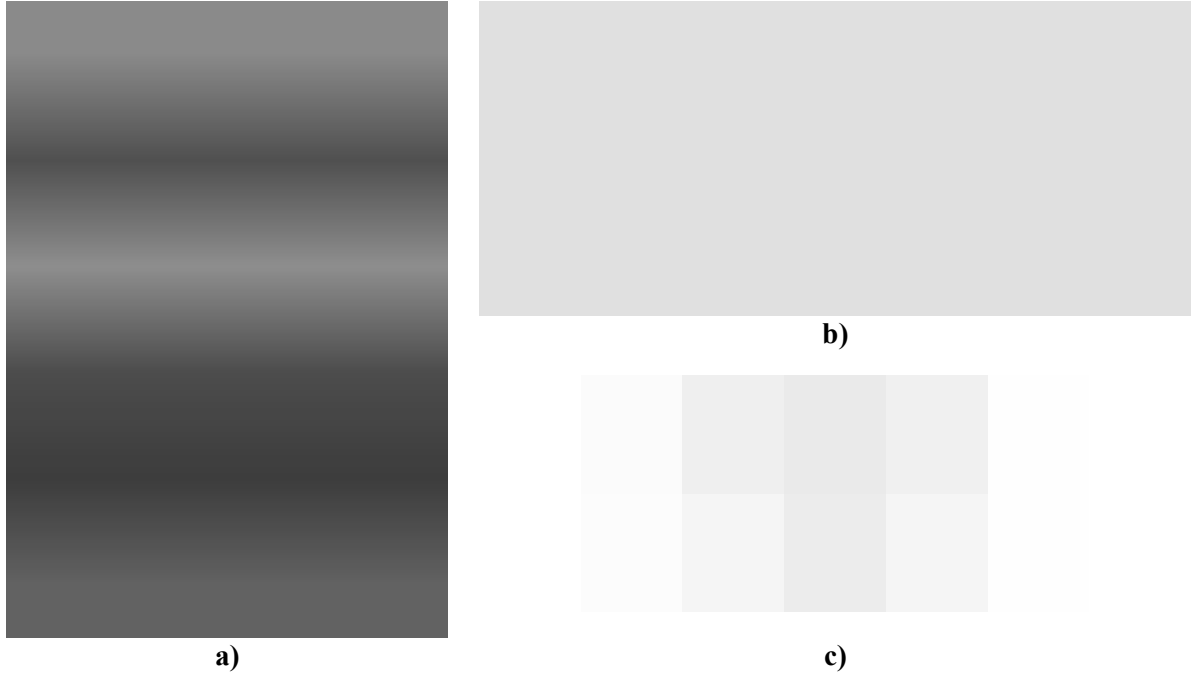


**Figura 7.** Puente de Toda Park – Japón [28]; **a)** Condición de congestión; **b)** Sección Frontal; **c)** Sección de viga-cajón

Años más tarde, sucede quizás el acontecimiento más relevante en el campo de la IHE en puentes peatonales. Los hechos registrados en Londres el 10 de junio del año 2000 en el puente Millennium, tomados por algunos como un caso simple de sincronización espontánea [3], [47], posicionaron el problema de serviciabilidad de estas estructuras como un tema relevante para la ingeniería estructural. Se registraron vibraciones laterales debido al importante flujo de peatones que asistieron a la inauguración de la estructura, alcanzando desplazamiento de hasta 7.0 cm y aceleraciones cercanas a 0.25 g; esta inapropiada dinámica de acoplamiento generó el cierre parcial y luego total de la estructura para el control de las vibraciones no deseadas. A diferencia de Strogatz et. al. [47], trabajos realizados por Dallard et. al. en [33] y Zivanovic et. al. en [3] posicionaron este fenómeno como un problema de interacción constante entre Estructura-Peatones diferente a un simple caso de sincronización. Este mismo fenómeno, se presentó unos meses antes en el puente Solferino en la ciudad de París sobre el *Sena*. Al igual que lo sucedido en el Millennium, se reportaron vibraciones laterales excesivas debido a las cargas inducidas por la actividad humana; esta situación fue reportada detalladamente por [44]. En estas dos estructuras se han realizado pruebas de carga a gran escala, las cuales consisten en inducir cargas de marcha humana de forma creciente en la estructura con el fin de determinar los cambios presentados en el amortiguamiento a partir de la respuesta dinámica y el número crítico de peatones que desencadena las vibraciones no deseadas, tal como lo describe Ingólfsson et. al en [1], **Figura 8** and **Figura 9**.

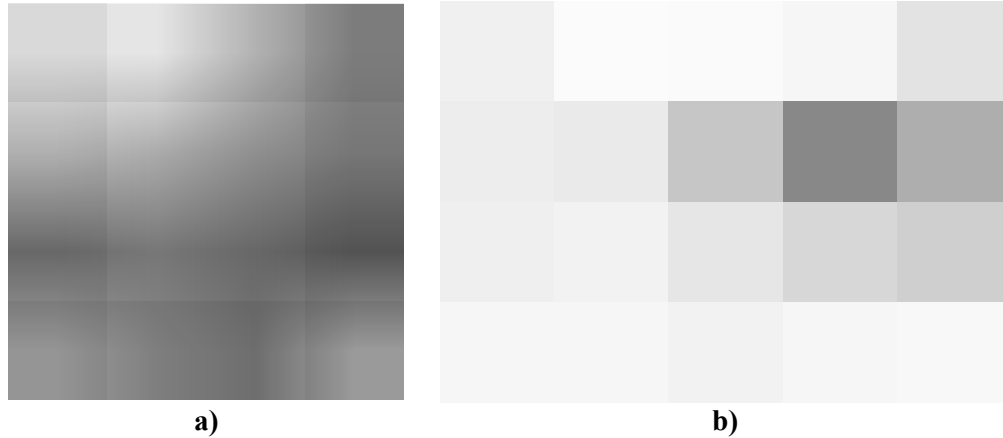


**Figura 8.** Puente Millennium [1]; **a)** Ilustración; **b)** Prueba de carga.

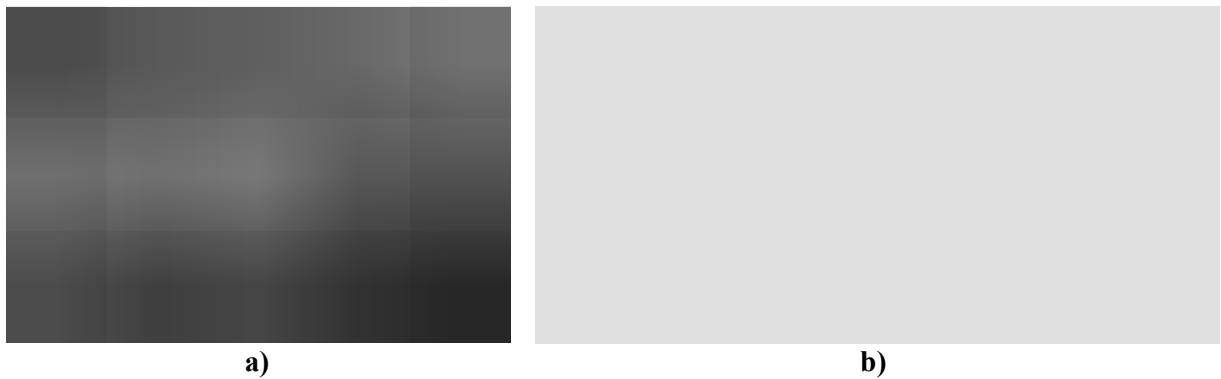


**Figura 9.** Puente Solferino [195]; **a)** Ilustración; **b)** Prueba de carga.

Otra de las estructuras ampliamente referenciada en la literatura por presentar condiciones de sensibilidad dinámica a las vibraciones debido a las solicitaciones de la marcha humana es el puente peatonal del aeropuerto de Changi en Singapur, reportado detalladamente por Brownjohn et. al. en [45], [46]. En este puente peatonal, durante su construcción, se tomaron datos experimentales que permitieron caracterizar la dinámica de la estructura posicionando al primer modo como lateral con una frecuencia de  $0.90\text{ Hz}$ , segundo modo de forma torsional con una frecuencia de  $1.64\text{ Hz}$  y un amortiguamiento característicamente bajo  $0.20\text{-}0.40\%$ . Adicionalmente, se realizaron pruebas de carga similares a las del Millennium y el Solferino con el fin de determinar la llamada *Excitación Lateral Crónica* (SLE). Posteriormente, Zivanovic et. al. en [72] investigaron los efectos de la IHE en tres puentes peatonales al cruce de un único peatón y con ello calibrar modelos numéricos de esta carga antrópica vertical. Y al siguiente año Ye et. al. en [49] examinan dinámicamente el puente de Brooklyn en Nueva York mediante vibraciones ambientales debido a reportes dados por los peatones de posibles vibraciones excesivas de la estructura durante el apagón de la Costa Este del 14 de agosto de 2003, los carriles para vehículos en dirección a Manhattan se abrieron a los peatones, mientras que los carriles en dirección a Brooklyn permanecieron bajo el tráfico de automóviles. En la **Figura 10**, se disponen algunas ilustraciones de estas investigaciones.

En el año 2010, se presenta el trabajo asociado al hito estructural en la ciudad de Coimbra, Portugal. Caetano et. al en [32], [50] expone el problema de vibraciones asociado al Puente Peatonal Pedro e Ines diseñado por Adão da Fonseca liderado por AFAssociados en supervisión de la firma Arup. En este

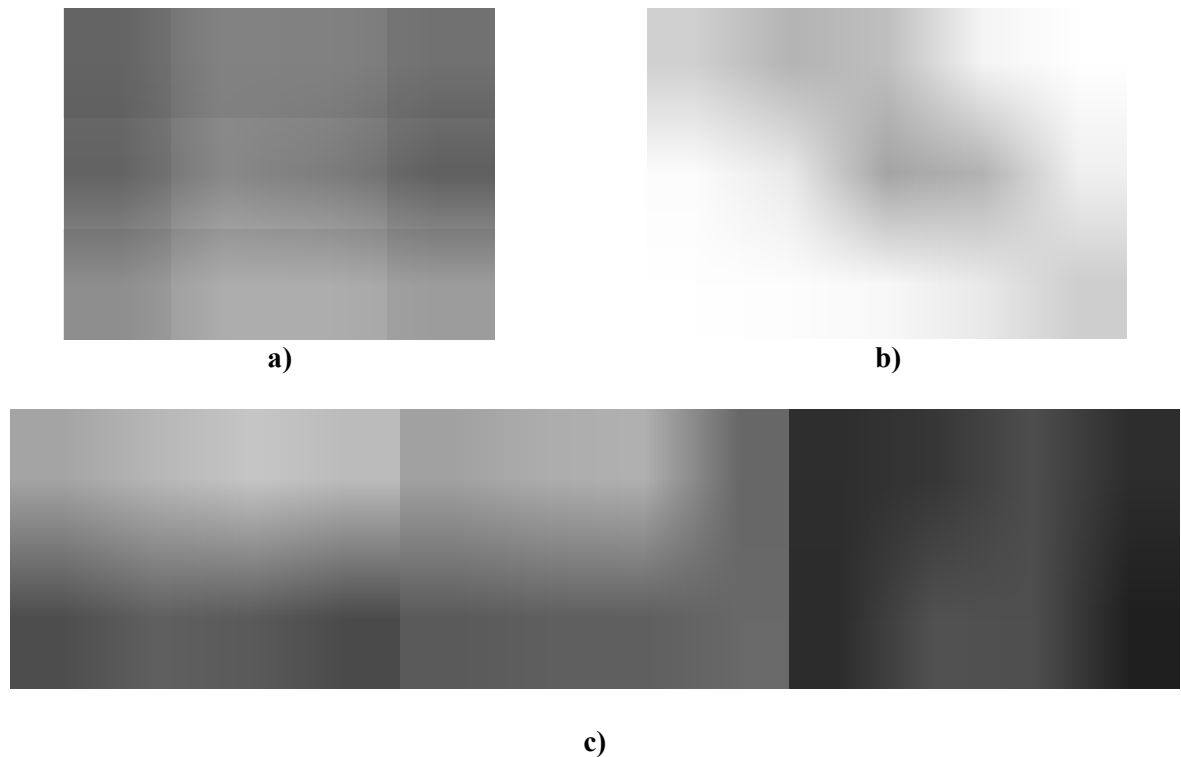
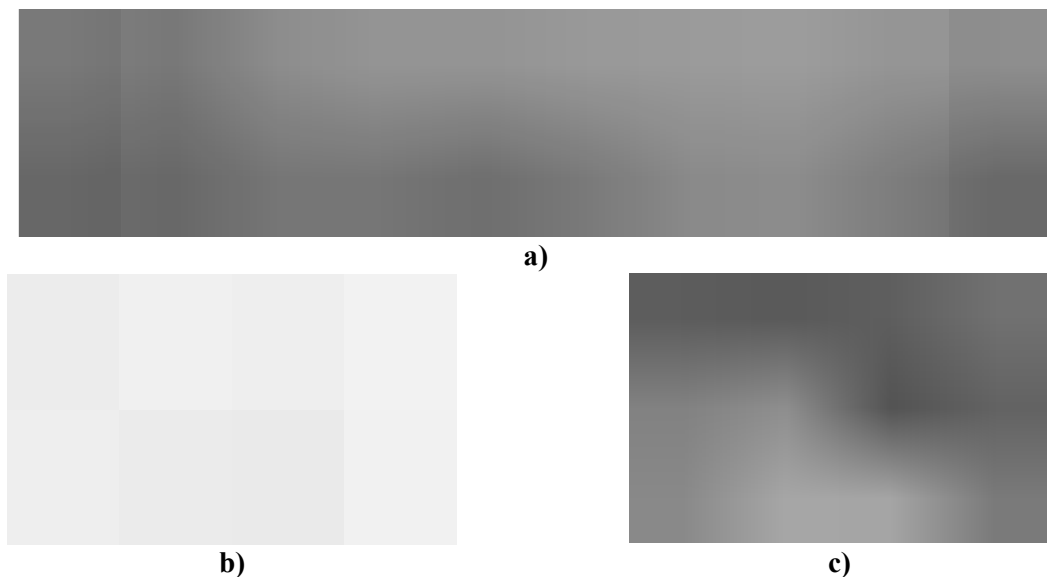


**Figura 10.** Puentes Peatonales; **a)** Puente de Aeropuerto Changi-Singapur [45], [46]; **b)** Modelo de EF en ANSYS del puente Brooklyn [49]; **c)** Puentes peatonales estudio Zivanovic [72].

estudio se centran en la descripción de los estudios realizados en la etapa de diseño y los resultados de evaluaciones experimentales minuciosas de las propiedades dinámicas de la estructura. Enfocando estos resultados en la determinación de las propiedades modales y un seguimiento detallado de los efectos “Lock-In” bajo el tránsito de peatones; adicionalmente, se cotejó de la misma forma que en el Millennium la SLE. Esta caracterización experimental permitió el diseño de un sistema de control compuesto por TMD a lo largo de la estructura, **Figura 11**.



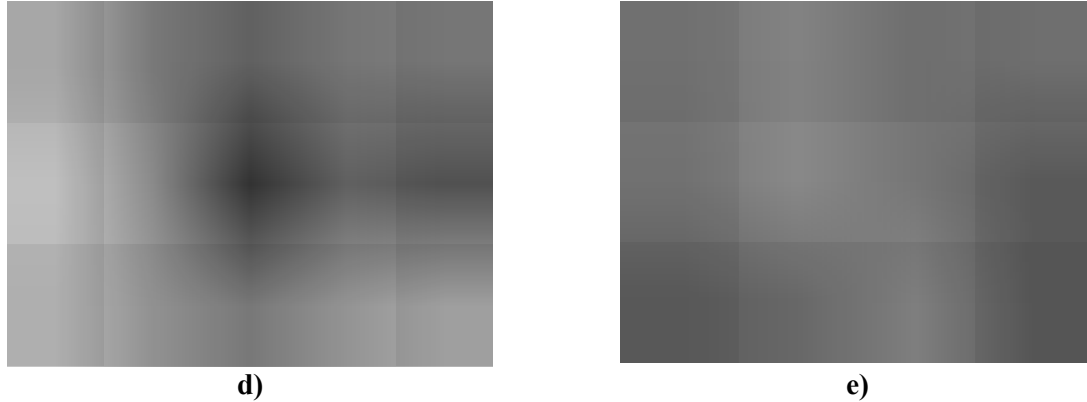


**Figura 11.** Puente Peatonal Pedro e Ines [32], [50]; **a)** Vista general del Puente peatonal Pedro e Ines; **b)** Registro de prueba SLE; **c)** Registro fotográfico prueba SLE; **d)** Conexión TMD en el centro de la luz; **e)** TMDs instalado en el centro de la luz.

En Colombia se han realizado limitados estudios sobre los efectos de la IHE en puentes peatonales y en estructuras civiles en general. A comienzos del año 2012 Sanches et. al. en [40] realizaron una clasificación de los puentes peatonales en Santiago de Cali a los cuales se le evaluaron los efectos de Interacción Humano-Estructura, aceleraciones máximas y frecuencias naturales, con los criterios de vibraciones para diseño especificado en las principales normas de la fecha. Aunque en los 19 puentes estudiados no hubo una superación de los límites de aceleración establecidas por las normas para una persona marchando, 16 de los puentes tipos presentaron vibraciones que indujeron incomodidad en los peatones. Evidenciando una carencia en las normas colombianas para el control de estos efectos en función de la serviciabilidad de la estructura, **Figura 12.**

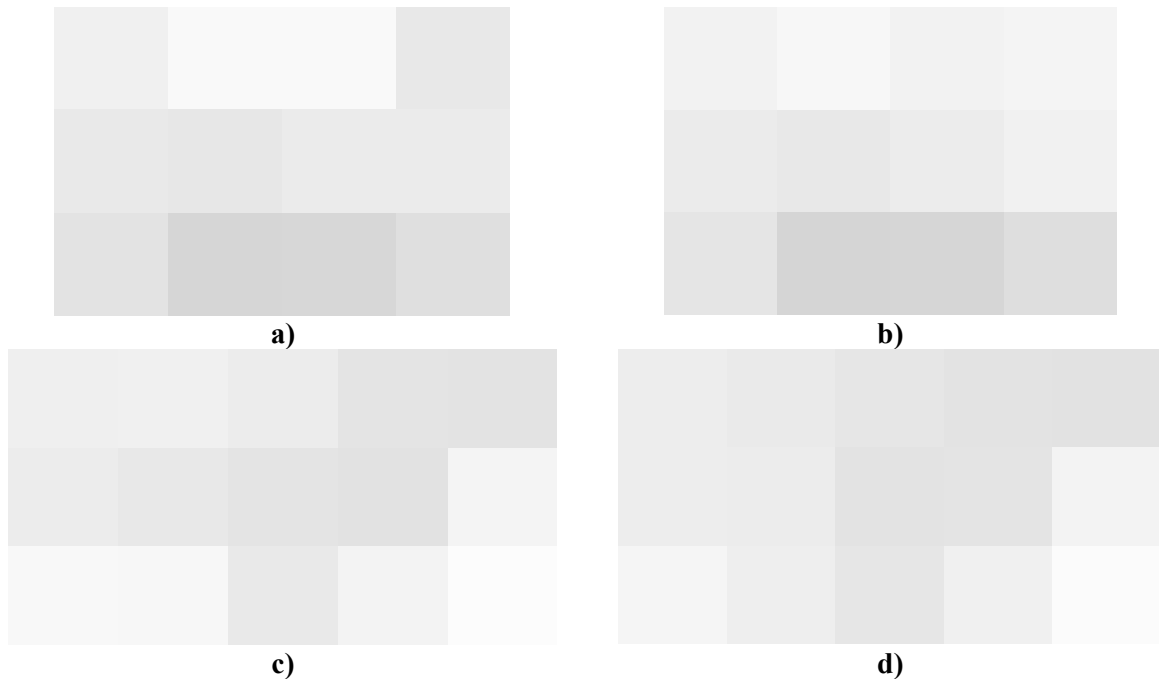


**Figura 12.** Estudio vibraciones en Santiago de Cali [40]; **a)** Luz vs Frec. natural vertical por tipo de puente; **b)** Luz vs Frec. natural vertical por tipo de material; **c)** Evaluación de servicio para vibraciones ambientales Verticales; **d)** Evaluación de servicio para vibraciones ambientales Laterales.



Otras investigaciones en Colombia han abordado indirectamente la condición de sensibilidad dinámica de los puentes peatonales frente a las cargas antrópicas. Desde el punto de vista de monitoreo estructural Castellanos et. al. en [202] evaluaron la capacidad de los teléfonos inteligentes para determinar el comportamiento dinámico de los puentes peatonales en la ciudad de Santiago de Cali, encontrando una alta susceptibilidad de estas estructuras frente al contenido frecuencial de las cargas de marcha humana, **Figura 13** (b-c). Por otro lado, desde el punto de vista patológico Rodríguez et. al. en [154] determinaron las características dinámicas y el comportamiento vibratorio de ocho puentes peatonales deteriorados de la ciudad de Tunja, evidenciando una importante condición de falta de serviciabilidad de las estructuras, **Figura 13** (a).

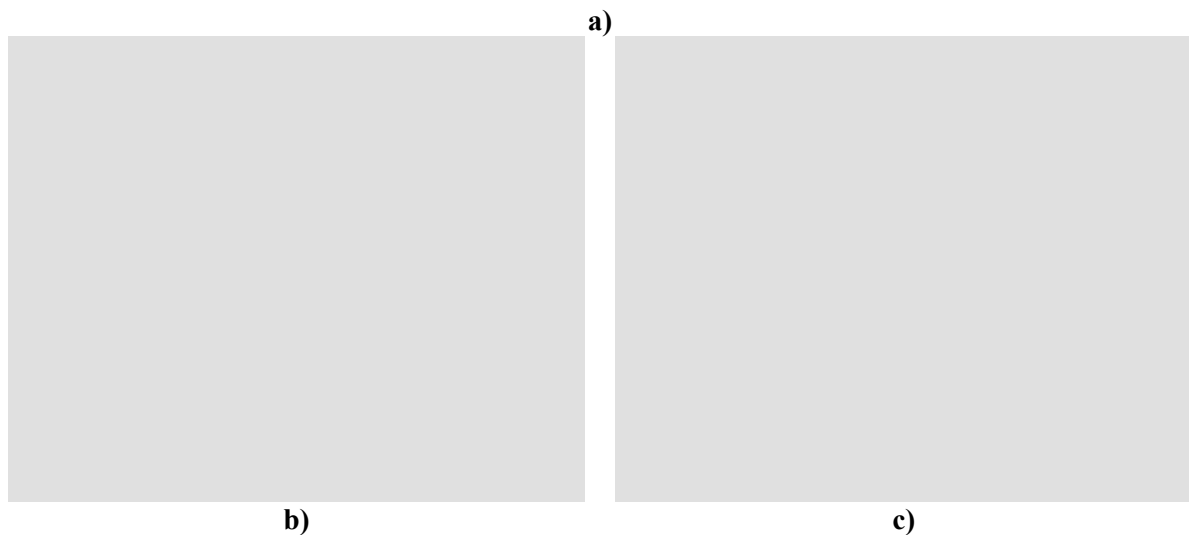


**Figura 13.** Estudios indirectos de serviciabilidad en Colombia; **a)** Ubicación de los puentes peatonales evaluados en [154]; **b)** Luz vs Frec. Natural en la dirección vertical [202]; **c)** Luz vs Frec. Natural en la dirección lateral [202].



En la literatura se pueden encontrar muchos otros casos relacionados a los efectos de IHE en puentes peatonales: El puente de Wuhan Yangtze, en China [223]; Le Ponte del Mare, en Pescara-Italia [51]; The Goodwill Bridge, en Brisbane-Australia [224]; y entre otros [38], [127], [130], [146], [149], [150], [151], [153], [178]. Sin embargo, las investigaciones más ampliamente desarrolladas a la fecha sobre un puente peatonal a escala real para evaluar su condición de sensibilidad dinámica frente a las cargas inducidas por la actividad humana son las desarrollada por Van Nimmen et. al en [29], [35] y Colmenares [31], [225] constituyendo un conjunto de datos Benchmark a escala real del Puente peatonal Eeklo and The Folke Bernadotte Bridge, de acceso público, con el fin de permitir una retroalimentación de los modelos numéricos desarrollados. Van Nimmen et. al registraron dos bloques de datos de vibración ambiental, se realizaron cuatro bloques de datos con densidades de peatones de 0.25 y 0.50  $Person./m^2$ , en **Figura 14** se muestran algunos esquemas de esta estructura y de los ensayos dinámicos.

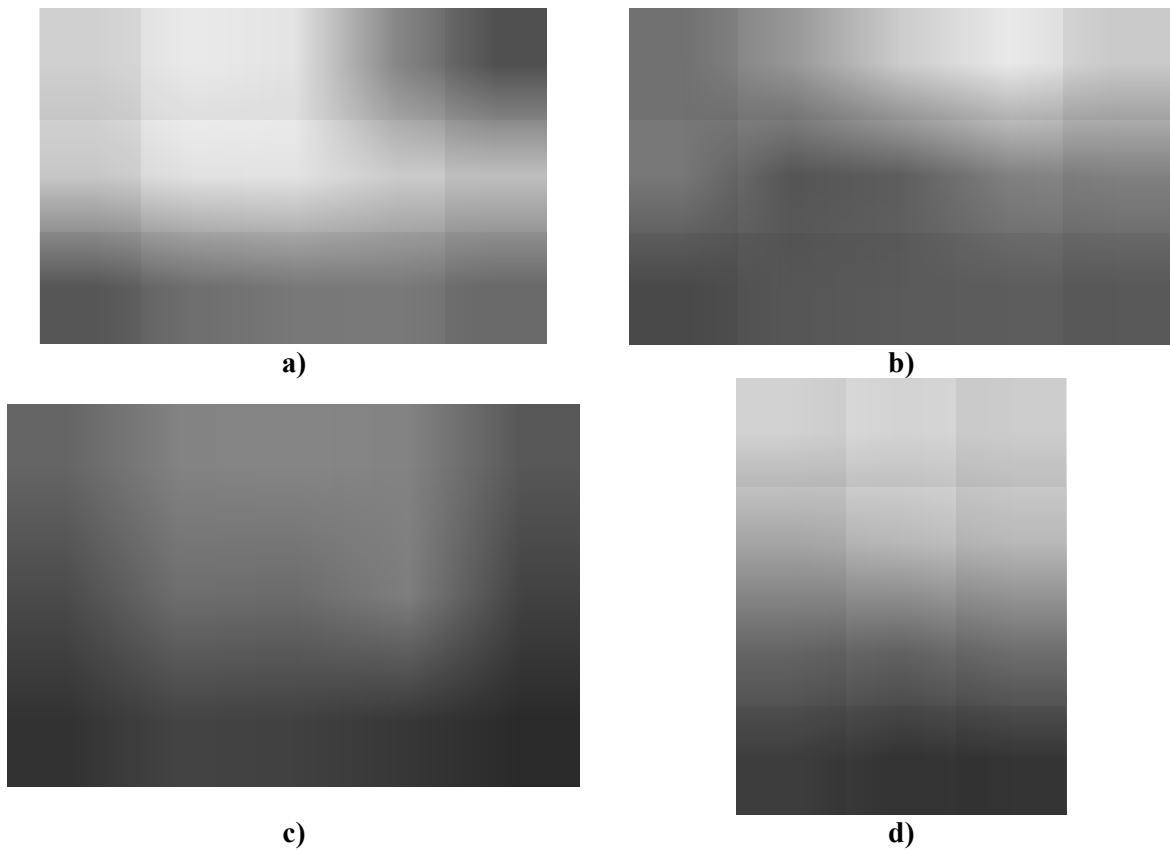


**Figura 14.** Puente peatonal Eeklo [110]; **a)** Fotografía lateral; **b)** Fotografía lateral de viga de carga principal; **c)** Sensor inercial y marcadores; **d)** Prueba de carga antrópica por multitud.



### 2.1.3 Ensayos sobre Plataformas Instrumentadas

Uno de los primeros trabajos de laboratorio y quizás uno de los más referenciados en futuros estudios fue el realizado por Reiher and Meister en 1931 [226]. Ellos investigaron los efectos de las vibraciones armónicas sobre 10 personas en diferentes posturas sobre una plataforma a diferentes amplitudes, frecuencias y dirección de amplitud. Como resultado de ello clasificaron la percepción humana en seis categorías y como una función de la amplitud de la vibración y la frecuencia. Luego, Lippert et. al. [96] desarrollaron una serie de estudios enfocados en los efectos de las vibraciones verticales, evaluando las respuestas humanas ante un movimiento vertical sinusoidal con frecuencias entre 0.10-256 cps y amplitudes entre 0.03-100 pulgadas. Posteriormente, Ebrahimpour et. al. en [227], [228] midieron las fuerzas dinámicas generadas por los ocupantes en tránsito utilizando una plataforma de fuerza instrumentada. Las pruebas consistieron en varios conjuntos de marcha de un peatón, dos peatones y cuatro peatones; utilizando una plataforma de fuerzas instrumentada con seis placas de fuerza de nido de abeja independiente.

La incursión en Europa de los efectos de la IHE en puentes peatonales, luego de los sucesos ocurridos en el puente Millennium y del trabajo realizado por Zivanovic en [3], trajo consigo el interés de la comunidad científica por posicionar estos fenómenos de vibraciones como uno de los temas más relevantes a nivel estructural. Este desencadenamiento en masa de conocimiento condujo a Europa a un proyecto interdisciplinar e intercontinental conocido como SYNPEX [30], [197], [198] el cual abordó desde diferentes puntos de vista y conocimiento avances relacionados con las vibraciones producto de los efectos de IHE en infraestructura civil. De este programa de investigación a gran escala resultaron investigaciones como la realizada por Nakamura et. al en 2008 en [133] para 5.0 sujetos de prueba, la cual consistió en desplazamientos entre 10.0-70.0 mm con frecuencias entre 0.75-1.25 Hz. El montaje utilizado puede verse esquemática y visualmente en *Figura 15*.

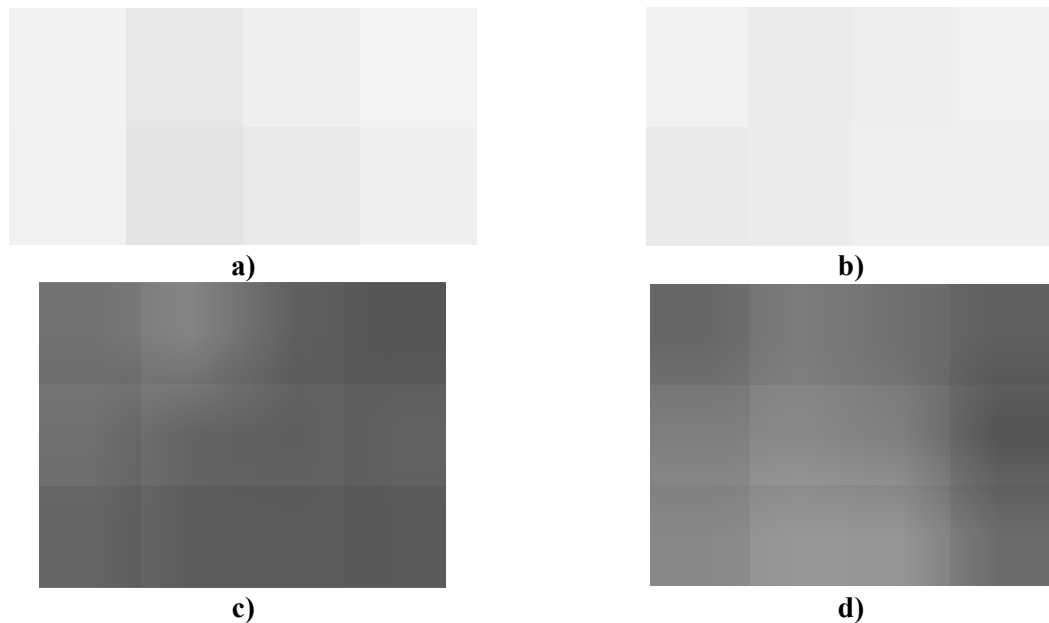


**Figura 15.** Estudio experimental de fuerzas laterales inducidas por peatones [133]; *a)* Vista superior de montaje experimental; *b)* Vista frontal de montaje experimental; *c)* Montaje experimental durante el ensayo; *d)* montaje experimental antes de ensayo.



Otra de las investigaciones relacionadas con el uso de caminadoras instrumentadas es la realizada por Ingolfsson et. al. en [100] en el 2011, basándose en el montaje experimental publicado por Pizzimenti et al. en [134], [229], mostrado en **Figura 16**. En este trabajo se analizaron experimentalmente las fuerzas laterales generadas por peatones individuales cuando marchan continuamente sobre una caminadora instrumentada. Inicialmente fue ajustada la caminadora a una estructura de soporte para ejercer un movimiento lateral sinusoidal con una combinación variable de frecuencias  $0.33 \rightarrow 1.07 \text{ Hz}$  y amplitudes  $4.50 \rightarrow 48.00 \text{ mm}$ . En este estudio experimental se involucraron 71 hombres y mujeres adultos, cubriendo aproximadamente  $55 \text{ km}$  de información de marcha distribuidos en 4954 paquetes de pruebas individuales.

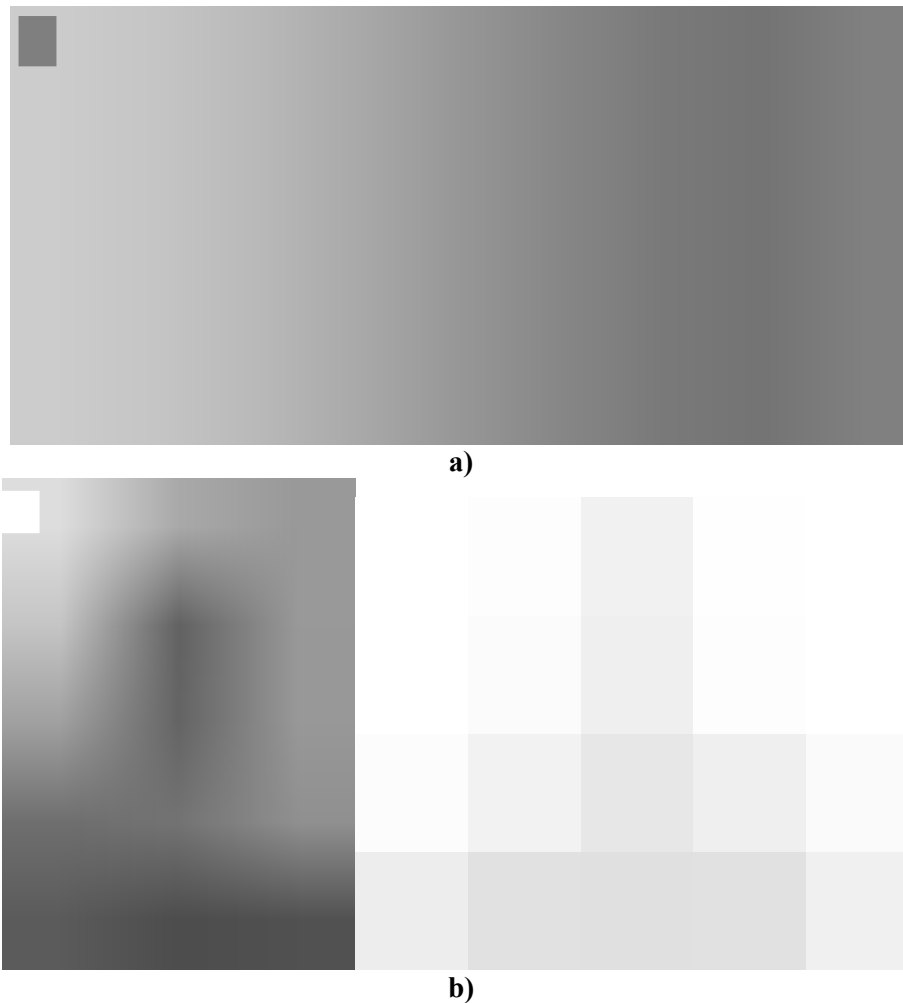


**Figura 16.** Montaje experimental Ingolfsson et al. [100]; **a)** Descripción de componentes del sistema experimental; **b)** Vista esquemática del montaje experimental y en operación.

Carroll et al. en 2013 [79], realizó un montaje experimental similar al realizado por Pizzimenti et al. [134], [229], como se muestra en la **Figura 17**. El objetivo de esta investigación fue realizar una campaña de pruebas experimentales para la observación de sujetos marchando sobre una caminadora instrumentada que oscilaba lateralmente. Para ello, se utilizó un sistema de cámaras de captura de movimiento tridimensional para localizar la posición de 31 marcadores activos, colocados en los puntos



estratégicos del cuerpo de los sujetos de prueba; con el fin de emplear la teoría de dinámica inversa para reproducir las GFR y validar con las adquiridas del montaje experimental.

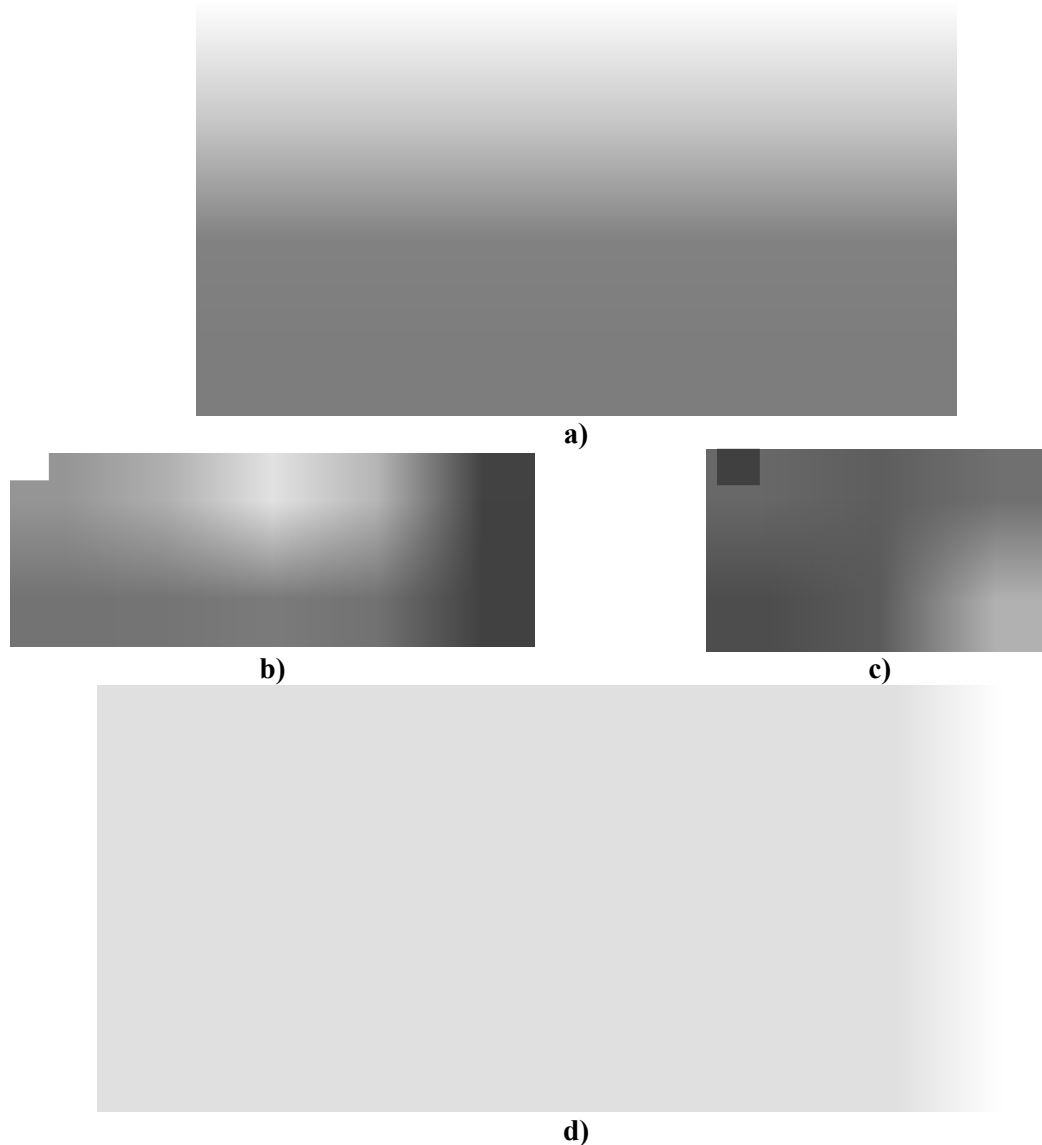


**Figura 17.** Montaje experimental Carroll et. al. [79]; **a)** Vista tridimensional del montaje experimental; **b)** Sistema de movimiento sinusoidal lateral; **c)** sistema de medición de carga vertical y lateral; **d)** descripción de los componentes del sistema experimental lateral.

Algunos investigadores exponen su preocupación por el condicionamiento que se puede presentar en la forma de marcha humana debido a las condiciones mismas de los laboratorios, entre ellos se encuentra Bocian et. al [101], [230], quienes alrededor de 2015 presentaron un novedoso montaje experimental desarrollado para evaluar la incidencia en el comportamiento de la marcha de los peatones cuando estos son sometidos a movimientos oscilatorios laterales y a su vez cuantificar su acción sobre la estructura en un entorno de laboratorio, evitando las implicaciones de la artificialidad y permitiendo una marcha sin



restricciones. Lo anterior se realizó, mediante la implementación de tecnologías de visión artificial, con lo que se adoptó un enfoque de inspiración biológica, basado en la apreciación de las complejidades operativas de los sistemas biológicos, en particular su adaptabilidad y requisitos de control natural. En la Fig. 18 se expone un esquema del montaje utilizado y una ilustración del montaje experimental en operación en la Universidad de Bristol.



**a)**



**b)**

**Figura 18.** Montaje experimental Bocian et al. [101], [230]; **a)** Esquema de montaje experimental; **b)** Ilustración de montaje experimental en operación.



Uno de los estudios más recientes enfocados al uso de caminadoras instrumentadas fue realizado por Haowen et. al. en [71]. En esta investigación se realizaron test experimentales con sujetos marchando sobre una caminadora excitada lateralmente mediante una mesa vibratoria, con un montaje basado en el propuesto por Pizzimenti et a. en [229], **Figura 19**. Mostrando en los resultados la influencia de los efectos de IHE en el comportamiento de la marcha de los sujetos, lo que permitió una retroalimentación y validación de los modelos de péndulo invertido propuestos en esta misma investigación.



**a)**

**b)**

**Figura 19.** Montaje experimental Haowen et. al. [71]; **a)** Descripción montaje experimental a. Lateral, b. Longitudinal, c. sistema de celdas de carga vertical, d. sistema de celdas de carga Lateral, e. plantillas de adquisición de carga antrópica marca: Novel Loadsol; **b)** Resultados experimentales de marcha de sujetos de prueba.



#### 2.1.4 Proyectos tipo *Benchmark* para evaluación de los efectos de IHE

Los problemas o planteamientos de investigación tipo *Benchmark* han sido especialmente utilizados para que una comunidad de investigadores pueda avanzar en un tema determinado, promover lecciones aprendidas, e identificar las capacidades y limitaciones de los métodos competentes [231]. En el estudio de los efectos de IHE en puentes peatonales este concepto se ha abordado desde hace ya más de una década. Una de las primeras investigaciones basada en este concepto fue desarrollada por Zivanovic et. al. en [34] donde se describe el diseño, construcción y caracterización dinámica de un puente peatonal experimental de 19.90 m de longitud, de baja frecuencia y ligeramente amortiguado, con el fin de evaluar los efectos de IHE en este tipo de estructuras. Este puente peatonal se encuentra ubicado en el Laboratorio de Estructuras de la Universidad de Warwick en UK, con una combinación estructural tradicional de acero-hormigón. Su comportamiento dinámico fue evaluado durante los primeros meses de construcción, encontrando una frecuencia natural vertical de 2.40 Hz y una razón de amortiguamiento del 0.5%. Su diseño es mostrado en la **Figura 20**, y algunas imágenes de su proceso constructivo en la **Figura 21**.



**Figura 20.** Diseño Puente peatonal Zivanovic et. al [34]; **a)** Vista frontal; **b)** Vista en planta; **c)** Sección transversal; **d)** Contenido frecuencial de la estructura; **e)** Modos de vibración.

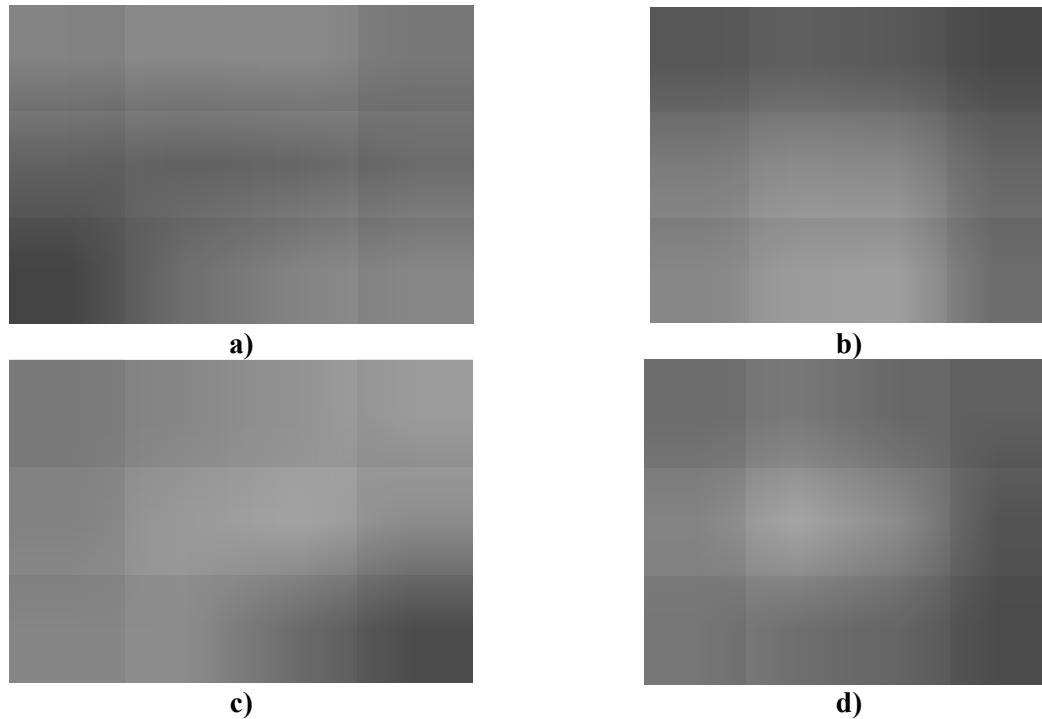
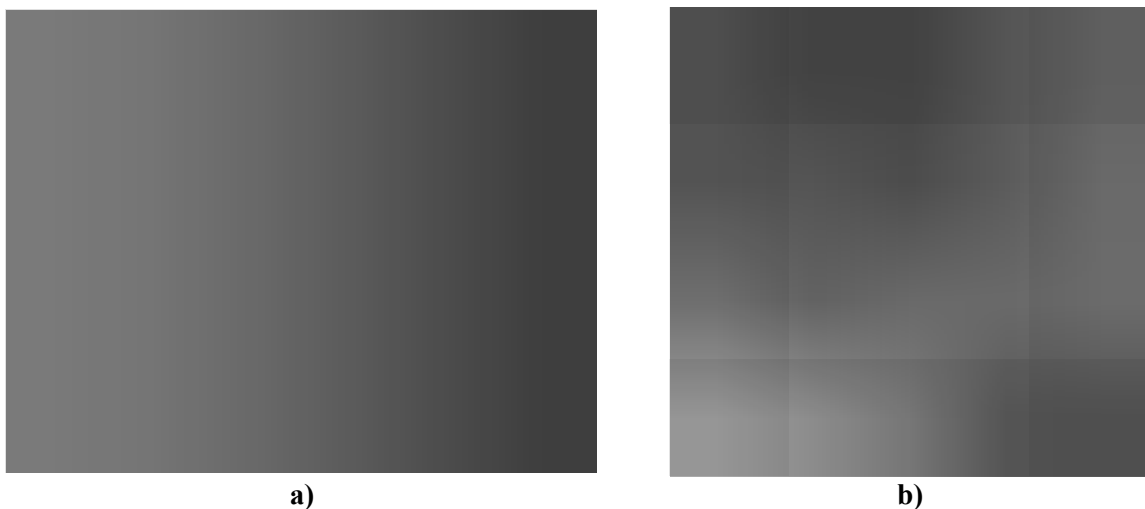


**Figura 21.** Construcción puente peatonal Zivanovic et. al. [34]; **a)** Vigas con conectores de cortante para sección compuesta; **b)** Disposición de acero de losa; **c)** Fundición de concreto; **d)** Puente peatonal operativo.

En el 2016, Dey et. al. en [140] evaluaron las disposiciones de diseño utilizadas a la fecha en Norteamérica y Europa. Se ensayaron en condiciones de laboratorio dos puentes peatonales (12.20 m y 22.90 m) de aluminio a escala real, y se determinó su rendimiento en términos de cumplimiento de los límites de servicio bajo las solicitaciones de los peatones marchando, de acuerdo con las normas y directrices vigentes en esos días. Se realizaron marchas de diferentes densidades de sujetos marchando en el puente encontrando una discrepancia entre los datos adquiridos experimentalmente y los esperados de los modelos empleados en las normas, lo que podría ser atribuido con gran firmeza a los efectos de IHE. Lo anterior se evidencia en la **Figura 22**.



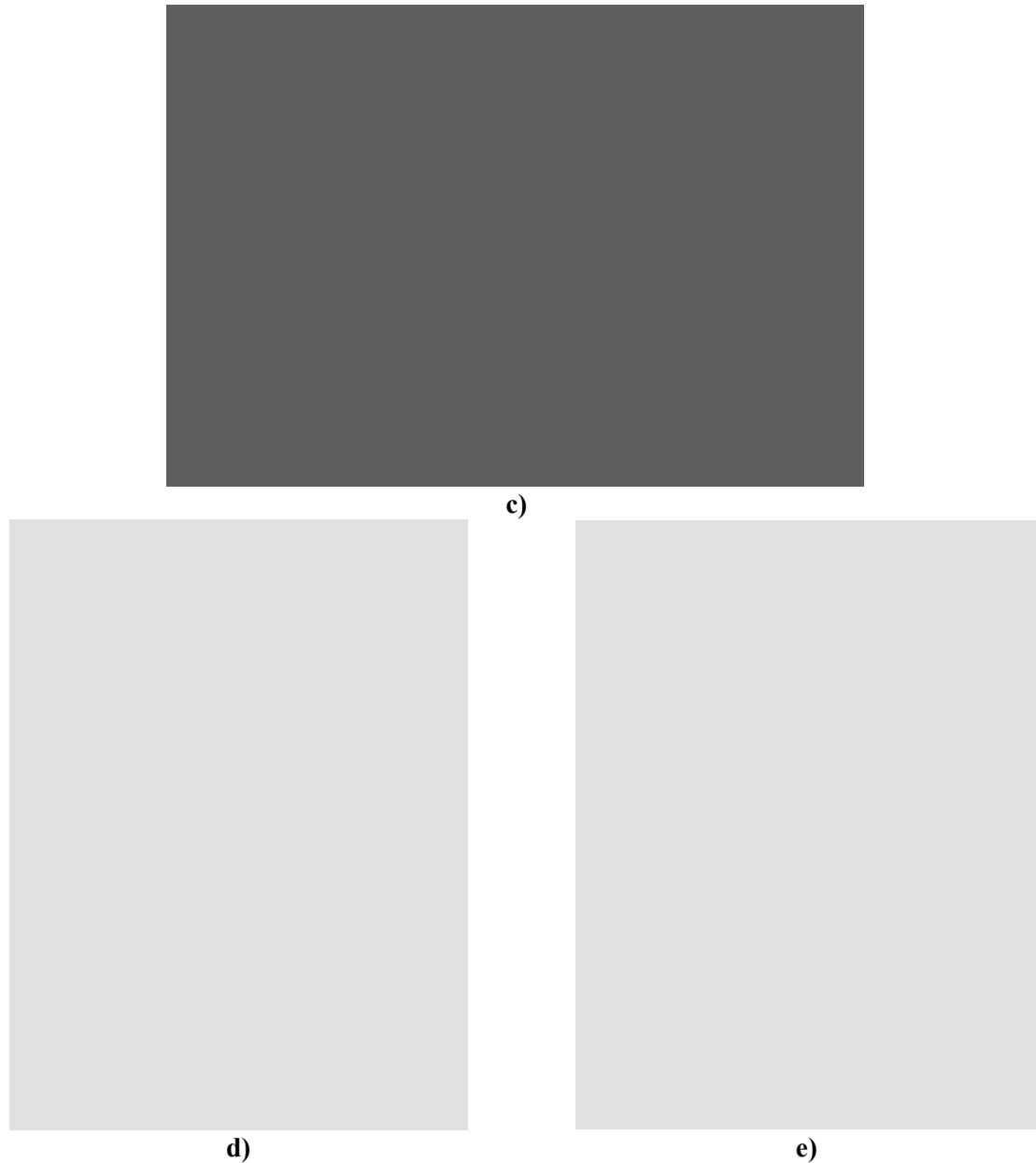


**Figura 22.** Puente en Aluminio Dey et. al. [140]; **a)** Vista estructural; **b)** Acelerómetro biaxial; **c)** Pruebas de marcha en multitudes; **d)** Resultados en aceleración del puente peatonal de  $L=12.20\text{ m}$ ; **e)** Resultados en aceleración del puente peatonal de  $L=22.90\text{ m}$

Se han desarrollado otros puentes peatonales en con la finalidad de evaluar sus respuestas de forma controlada. Autores como Hudson et. al. [141] Claff et. al. [139], Wei et al. [175], Firus et. al. [232] y Pfeil et. al.[233] se destacan en la literatura por sus investigaciones que, aunque no plantean un problema innato tipo *Benchmark*, tienen el potencial de plantearse como uno. Además, lo más importante de sus investigaciones ha sido el hecho de aprovechamiento de estos datos experimentales como una fuente de retroalimentación, calibración y validación de modelos numéricos más robustos con el fin de abordar de una mejor manera los efectos relacionados con la IHE.



En Colombia, solo se reporta en la literatura el trabajo realizado por Gómez et. al. [224] como un problema tipo *Benchmark* para la evaluación de puentes peatonales. En esta investigación la Escuela de Ingeniería Civil de la Universidad del Valle, en colaboración con la Universidad de Purdue y la Universidad de Carolina del Sur, construyó un banco de pruebas para investigar los efectos de la IHE en Puentes peatonales. Esta estructura tiene una longitud total de 12.00 m y un ancho de 1.20 m, se conforma de una celosía de acero simplemente apoyada con una plataforma de acero. Además, se soporta sobre cuatro celdas de carga, dos por cada extremo del puente peatonal. Por otra parte, su contenido frecuencia se encuentra entre 3.86→25.54 Hz; situación que no lo hace llamativo para el estudio de los efectos de la IHE en este tipo de estructuras bajo cargas de marcha humana. Sobre esta estructura se realizaron más de 100 pruebas experimentales de marcha con peatones caminando a diferentes frecuencias de paso. Los sujetos de prueba fueron constituidos por participantes femeninos y masculinos con pesos entre 50.00→105.00 kg, y alturas entre 1.45→1.90 m. En la **Figura 23** se muestra el puente peatonal desarrollado en esta investigación.

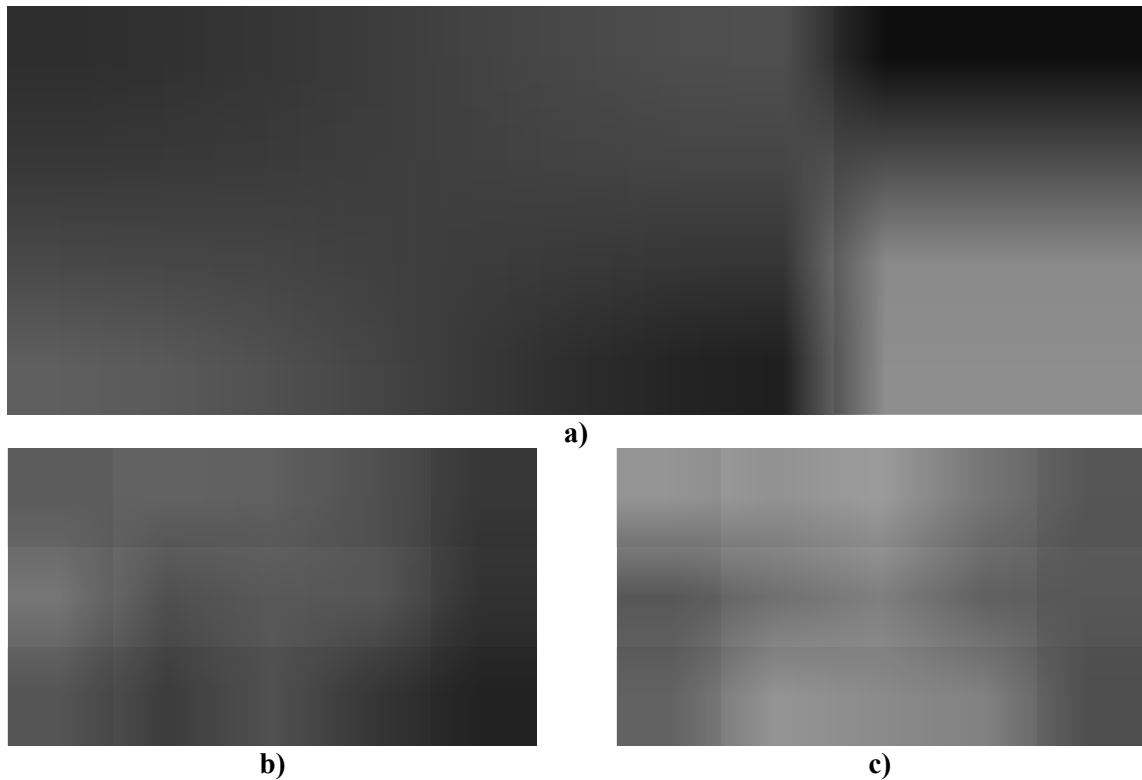


**Figura 23.** Puente Benchmark UV Gómez et. al. [224]; **a)** Vista general del puente peatonal; **b)** Celosía de acero; **c)** Soporte con celda de carga.

Finalmente, uno de los últimos puentes peatonales tomado como un problema tipo *Benchmark* fue el desarrollado por Gallegos et. al. en [36], [145] quienes desarrollaron una estructura ultraligera de polímeros reforzados con fibras (Fiber Reinforced Polymer, FRP) sensible dinámicamente a vibraciones verticales bajo carga inducidas por la marcha humana. Se realizó su caracterización dinámica y estática de la estructura. Además, esta investigación permitió la calibración de un modelo básico de IHE tomando al peatón como un sistema Masa-Amortiguador-Rigidez. En la **Figura 24**, se observa la estructura construida.

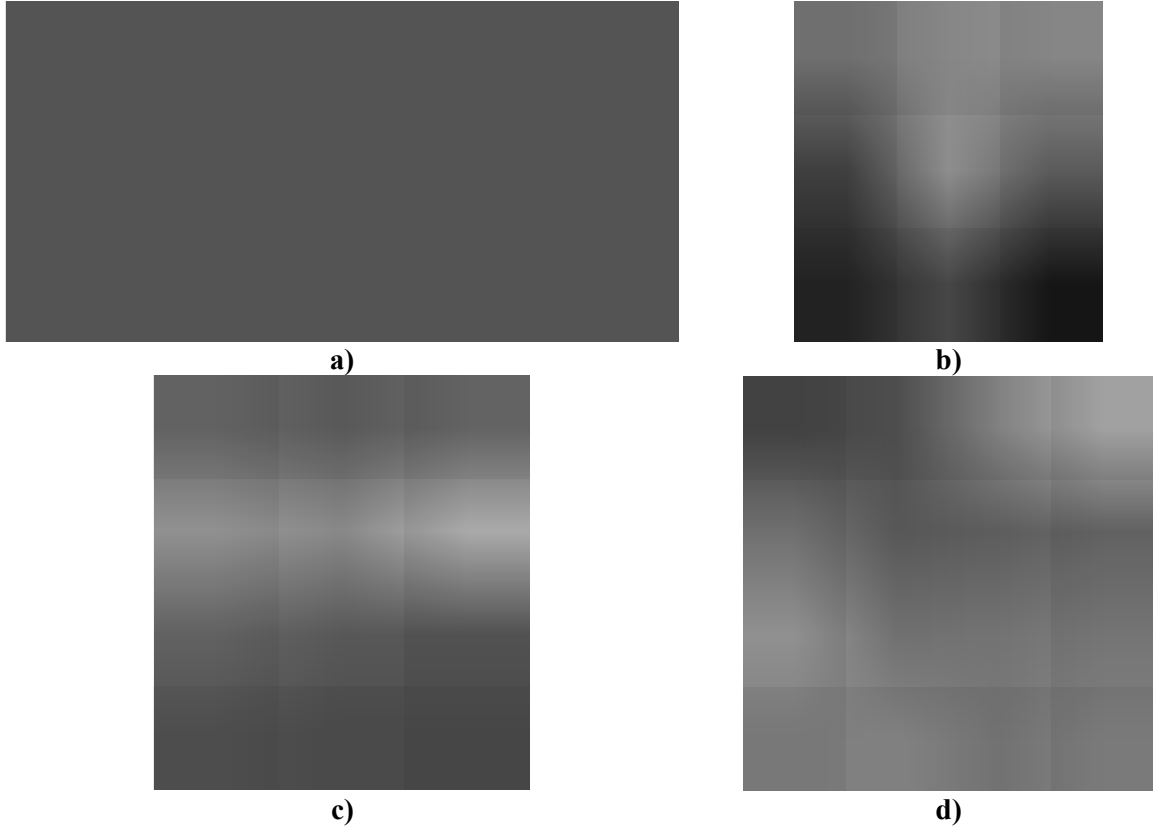


**Figura 24.** Puente peatonal de FRP Gallegos et. al. [145]; **a)** Vista general del puente; **b)** Prueba estática; **c)** Puente peatonal operativo; **d)** Adquisición de datos en aceleración.

### 2.1.5 Modelos Analíticos de Marcha e Interacción Humano-Estructura

Hasta los primeros años del siglo XX los modelos de caminata existentes para la evaluación de la capacidad de serviciabilidad de las estructuras debido a vibraciones producto del tránsito de peatones eran típicamente basados en réplicas de pisadas en intervalos determinados. Este concepto de representar las solicitaciones de marcha como cargas periódicas mediante series de Fourier, basadas en su contenido frecuencial, fue rápidamente objetado por los resultados obtenidos en las pruebas sobre caminadoras instrumentadas que permitían una adquisición de más pasos de marcha continuos, en comparación con los obtenidos sobre placas de fuerza, que evidenciaron la aleatoriedad intrínseca de las mismas. Investigaciones de este tipo fueron desarrolladas por Brownjohn et. al. [234] en el año 2004. Sin embargo, los modelos de marcha humana continuaron planteándose como cargas periódicas, especialmente en modelos biomecánicos que su objetivo no eran emplearse como cargas en estructuras civiles. Estos modelos biomecánicos utilizan las bondades de la dinámica inversa para el cálculo de la cinética, a partir de la optimización de la cinemática, asociada a la marcha humana. Ren et. al. [106], [107], [108] utilizó estos conceptos para el desarrollo de modelos 2-D y 3-D de la marcha humana sobre superficie rígida con el objetivo de minimización de la energía mecánica en el sistema humano. Un trabajo similar, aunque más simple, fue desarrollado por Font-Llagunes et. al. en [109] en el cual se estudió la dinámica de la marcha enfocada al momento de impacto de cada pie (*Heel Strike*) donde existe una mayor pérdida de energía del sistema humano.



**Figura 25.** Modelos biomecánicos; **a)** Modelo Ren et. al. [108]; **b)** Modelo Font-Llangunes et.al. [109].

Estos modelos biomecánicos segmentados, aunque tienen un buen desempeño para simular condiciones de carga de la marcha humana, son limitados en el sentido de la periodicidad de la cinemática de los mismo y de la necesidad misma de información antropométrica, como los estudios realizados por Leva et. al. [235], lo que los hace impráctico en el momento de evaluarlos en condiciones de diseño técnico. Por otra parte, los modelos de Fuerza Móvil (FM) aplicados al estudio de los efectos de la IHE han sido descartados en los últimos años debido a su carencia de información dinámica propia de la marcha humana, lo que conduce a condiciones de subestimación y sobrestimación de las vibraciones en la estructura. Por lo cual, con el fin de involucrar las propiedades dinámicas de la marcha con las del puente peatonal, diferentes investigadores han estudiado el sistema humano como un sistema biodinámico de Masa-Resorte-Amortiguador (MRA) de 1-GDL o MGD. Jiménez et. al. [85], [122], Pfeil et. al. [236], Ahmadi et. al. [129], Pena et. al. [83], Wang et. al. [105] en sus investigaciones han desarrollado modelos biomecánicos asociando sistemas de 1-GDL de MRA para el estudio de los efectos de la IHE en puentes peatonales, algunos esquemas de modelos se muestran en **Figura 26**.

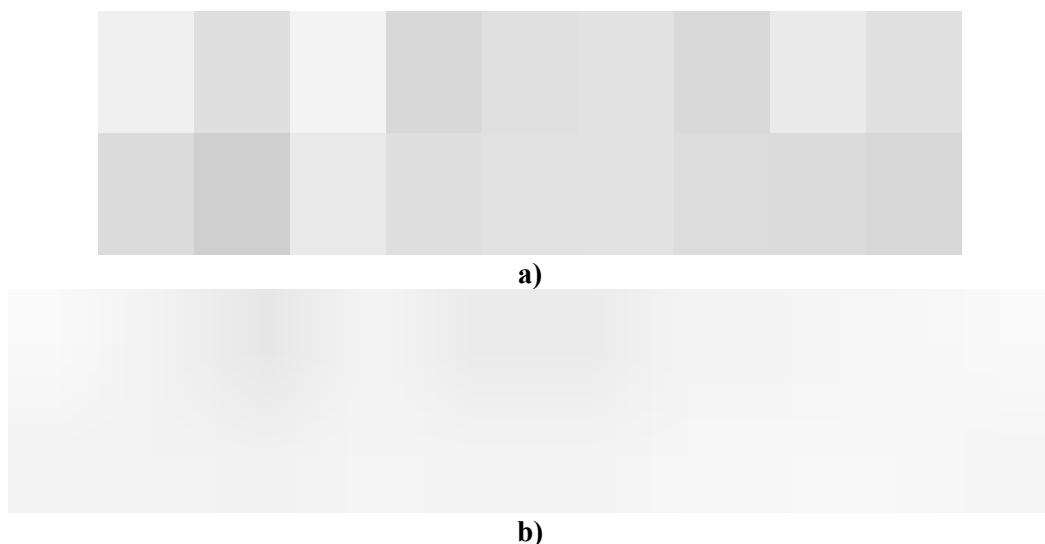


**Figura 26.** Modelos biodinámicos 1-GDL: **a)** Modelo Ahmadi et. al. [104]; **b)** Modelo Wang et. al. [105].



Con el mismo objetivo de las investigaciones anteriores, pero con el propósito de modelar condiciones más realistas de la marcha humana, diferentes autores optaron por estudiar estos sistemas biomecánicos mediante modelos dinámicos de MRA bípedos con el fin de simular las condiciones de balanceo simple y doble soporte, típicos de esta actividad antrópica [55], [71]. Investigadores como Qingdu. Et. al. [110], Keun et. al. [112], McDonald et. al. [113], Yang et. al. [114] y Vega et. al. [131] realizaron estudios de modelo dinámico MRA bípedos sobre superficie rígida, **Figura 27**.

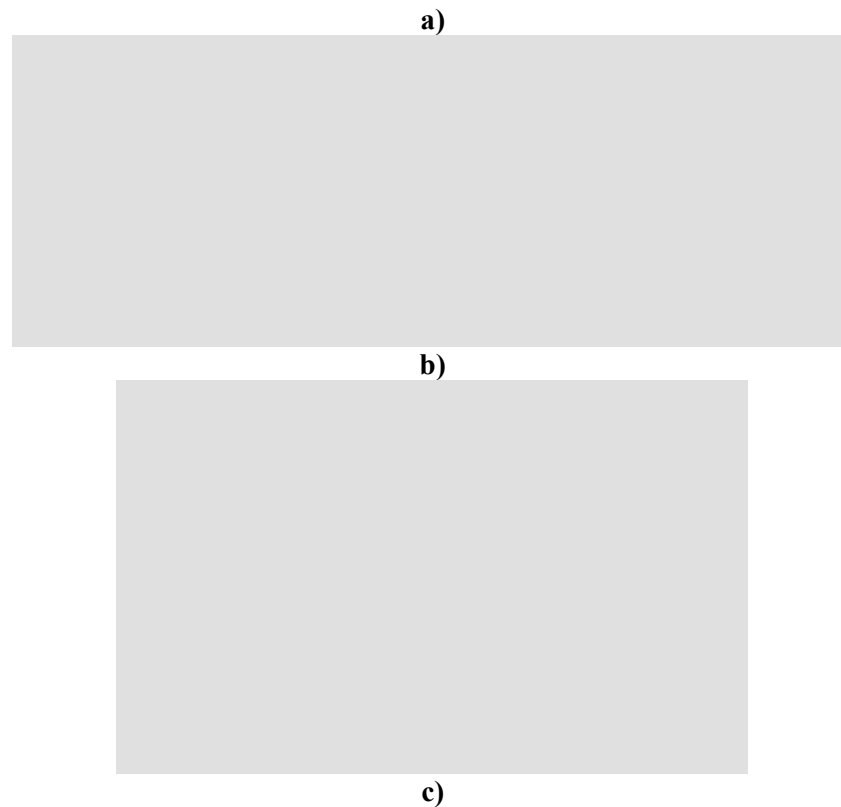


**Figura 27.** Modelos biomecánicos MRA bípedos sobre superficie rígida; **a)** Modelo Yang et. al. [114]; **b)** Modelo Vega et. al. [131]; **c)** Modelo McDonald et. al. [113].

De la misma manera, autores como Qin et. al. [111], Gao et. al. [115], Shahabpoor et. al. [68], Carroll et. al. [79] y Haowen et. al. [71] desarrollaron modelos biomecánicos MRA bípedos incluyendo efectos de la IHE sobre puentes peatonales, permitiendo el estudio analítico de la influencia del peatón en las propiedades dinámicas de la estructura, y a su vez, la influencia de las vibraciones sobre las solicitaciones de marcha.

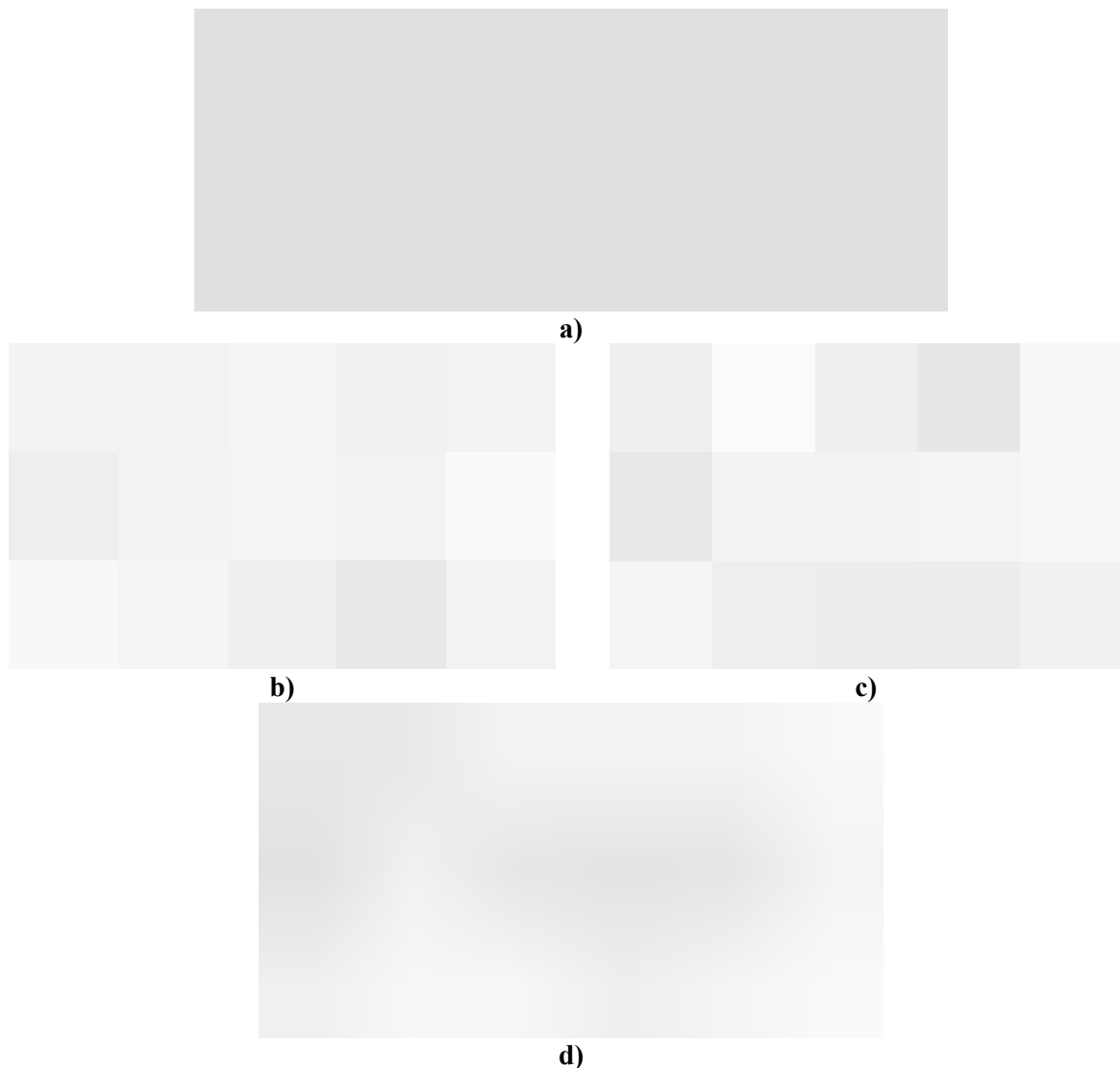


**Figura 28.** Modelo biomecánico MRA con IHE bípedo Qin et. al. [111]; **a)** Esquema de modelo planteado; **b)** Cambios en la Frecuencia del sistema Humano-Estructura; **c)** Cambios en el amortiguamiento del sistema Humano-Estructura; **d)** Respuesta dinámica de estructura con y sin efectos de la IHE.



b)

c)

d)

**Figura 29.** Modelo biomecánico MRA con IHE bípodo Gao et. al. [115]; **a)** Esquema de modelo planteado; **b)** Cambios en la Frecuencia del sistema Humano-Estructura; **c)** Cambios en el amortiguamiento del sistema Humano-Estructura; **d)** Respuesta dinámica de estructura con y sin efectos de la IHE.

Aunque en la literatura se encuentran algunos modelos de tipo espectral [18], [92], [174]; han sido los modelos de multitudes los que han predominado en el interés de la comunidad internacional, esto se debe principalmente a que es en estas condiciones donde los puentes peatonales son más propensos a experimentar efectos de la IHE con mayor magnitud, además de estar presentes los efectos IHH los cuales condicionan aún más la dinámica de la marcha de los peatones. Los primeros estudios de cargas inducidas por multitudes en puentes peatonales trataron estas solicitaciones bajo teorías hidrodinámicas de los fluidos, teniendo como objetivo representar la complejidad no-lineal multifacética del sistema dinámico para dar una descripción detallada de la multitud a lo largo de la estructura [4], [5]. La implementación de modelos biomecánicos de 1-GDL o MGDG singulares y bípodos para el estudio de la Interacción Multitud-Estructura (IME), junto con análisis dinámicos de flujo, han sido ampliamente investigado. Bruno et. al. [99] presenta una modelización física para describir el comportamiento inteligente, no-local y anisótropo de los transeúntes; Venuti et. al. [10] desarrollaron un modelo de interacción dinámica entre el peatón y la estructura en la dirección vertical con el fin de abordar la interacción inter-sujeto y el comportamiento inteligente en grupos de personas; Tubino et. al. [69] implementan sobre un modelo de 1-GDL peatones representados por sistemas biomecánicos tipo MRA simples; y Zhu et. al. [9] estudiaron un método híbrido basado en el control de flujo de la multitud. Las anteriores investigaciones, en torno al desarrollo de modelos numéricos de la marcha de multitud de peatones, busca simular y atender los efectos de la IME en estructuras como puentes peatonales. En la **Figura 30** se muestran algunos de estos modelos mencionados.

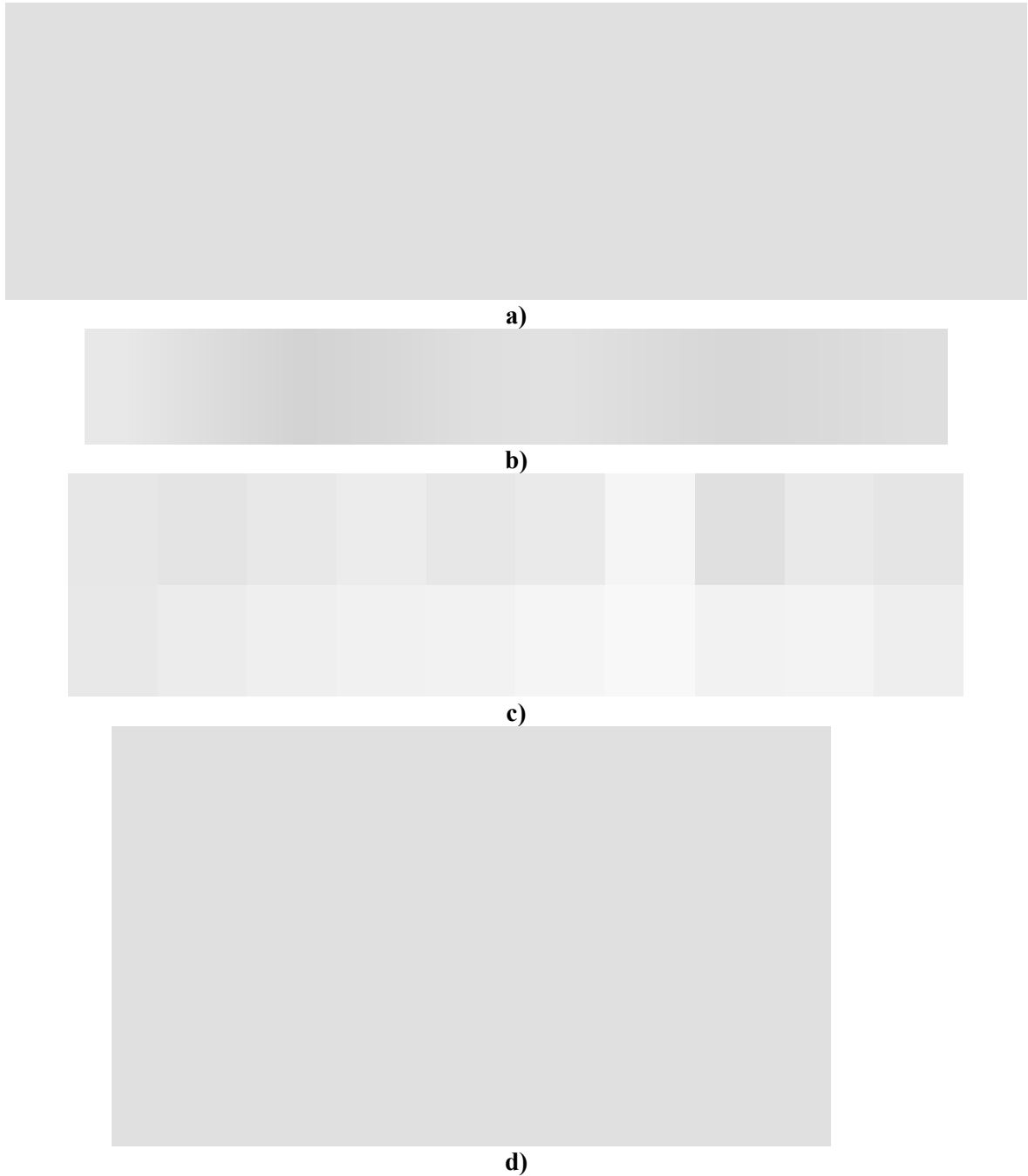


**Figura 30.** Modelos de IHE; **a)** Modelo Bruno et. al. [99]; **b)** Modelo Venuti et. al. [10]; **c)** Modelo Tubino et. al. [69]; **d)** Modelo Zhu et. al. [9].

En los últimos años, debido al entendimiento de la aleatoriedad presente en la marcha humana y bajo nuestro concepto del fenómeno de IHE en general, se ha optado por implementar en los procesos de modelación incertidumbres a los parámetros que constituyen los sistemas humanos y en menor medida al sistema estructural. Con este enfoque se pretende abordar el problema de IHE e IME desde modelos robustos capaces de contemplar diversos escenarios de carga, contribuyendo con los diseños estructurales desde un punto de vista de estados límites. En estas investigaciones se han incluido en gran



medida Simulaciones de Montecarlo con el fin de recrear situaciones posibles bajo la variabilidad de los datos estocásticos de los modelos [7], [73], [98]. Las principales investigaciones recientes que han desarrollado este concepto han sido Piccardo et. al. [124], Venuti et. al. [95], [177], Tubino et. al. [102], Gómez et. al. [41]; Wei et. al. [73]; sin embargo, sus estudios se han concentrado principalmente a los parámetros estocásticos del sistema biomecánico de marcha y no en conjunto con la variabilidad del sistema estructural, como se muestra en el modelo propuesto por Gómez et al. [41] en la **Figura 31**.

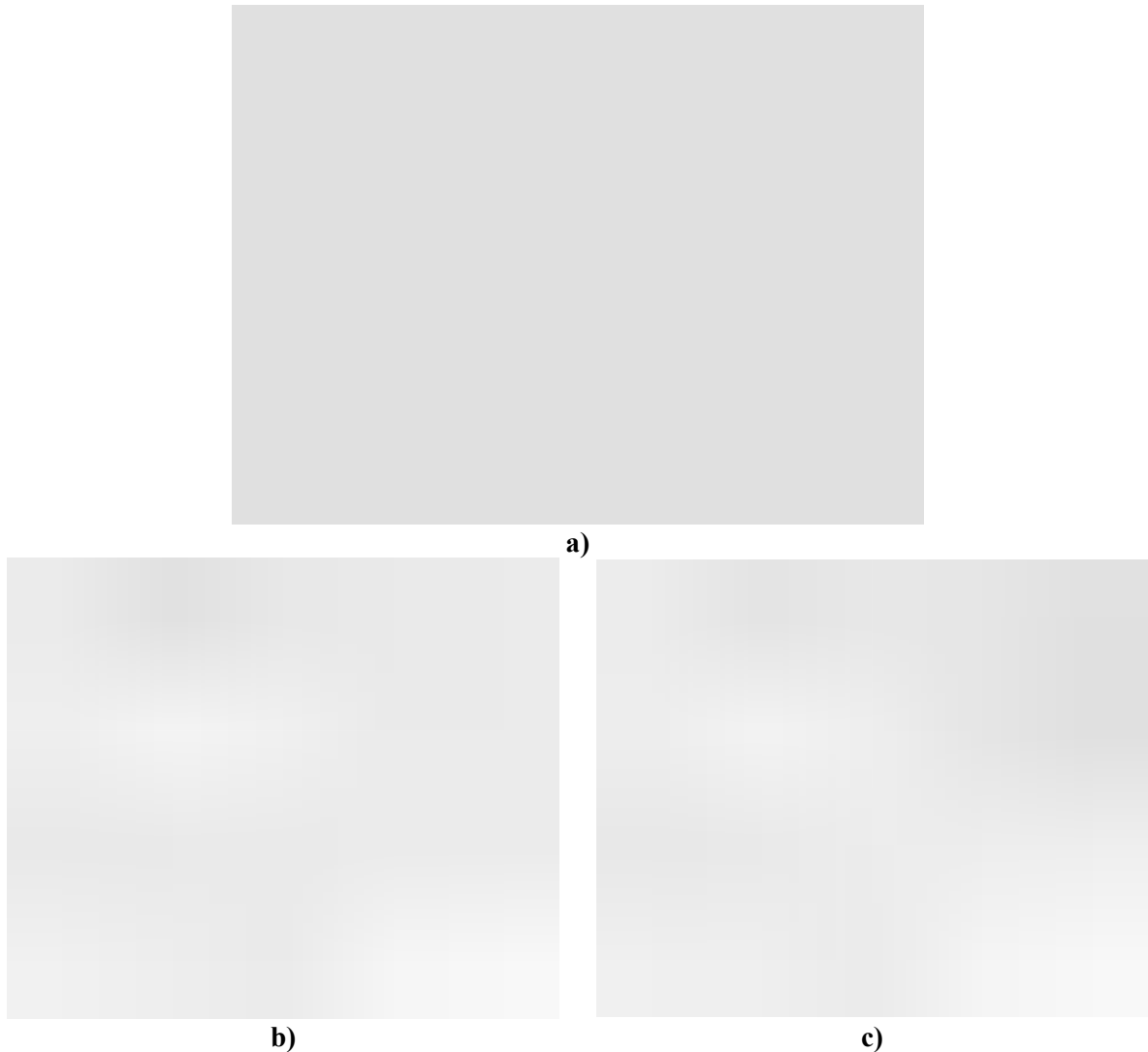


**Figura 31.** Modelo de la IHE Gómez et. al. [41]; **a)** Modelo esquemático; **b)** Respuesta sujeto S-02 y modelo de incertidumbre uPSI; **c)** Respuesta sujeto S-09 y modelo de incertidumbre uPSI.

Muchos modelos numéricos han sido evaluados y calibrados a partir de ensayos experimentales de puentes peatonales ya identificados sometidos a excitaciones por la marcha humana, entre las muchas investigaciones se pueden resaltar las realizadas por Lai et. al. [42], [116], Mulas et. al. [43] y Jiménez et. al. [237]. Sin embargo, a la fecha de hoy existe una fuerte necesidad por el desarrollo de modelos numéricos robustos capaces de simular con precisión los efectos de IHE e IME [38], [70], [127], ya que no se ha desarrollado en la actualidad ningún modelo correctamente formulado y además verificado con pruebas experimentales [18], [179]. Por lo que, en los próximos años estos tipos de modelo tienen tres



cuestiones importantes por tratar: 1. La interacción auto-exitada entre peatones o efecto IHH, 2. El efecto de los sistemas humanos sobre las propiedades dinámicas de la estructura civil no ocupada, 3. La variabilidad intersujeto.

### **2.1.6 Mediciones Indirectas de la Marcha Humana**

Durante las pruebas convencionales sobre placas de fuerza o caminadoras instrumentadas con celdas de carga, los sujetos de prueba tienen a condicionar su forma de marcha normal [101]. Además, realizar mediciones de la cinética o cinemática de la marcha en puentes peatonales operativos de forma directa representa un desafío económico y logístico importante que, hoy en día, ninguna institución ha realizado. Con el fin de abordar esta situación se ha transferido y adaptado paulatinamente, de las investigaciones biodinámicas, la tecnología de captura de movimiento. Los resultados en la última década muestran que este tipo de datos pueden ser satisfactoriamente utilizados en estudios de la IHE como una alternativa viable de adquirir información indirecta de la cinética y cinemática de la marcha humana.

Una de las primeras revisiones en la literatura realizadas sobre la medición indirecta de fuerzas de marcha humana fue realizada por Racic et. al. [55], posicionando al sistema de adquisición y procesamiento de imagen como una de las metodologías más funcionales para tal objetivo. En esta investigación se resumen los avances realizados hasta el año 2009 y posiciona esta técnica de procesamiento de imágenes como una tecnología prometedora y con potencial de desarrollo; se concentran muy especialmente en el estudio de seguimiento de marcadores corporales que permitan reconstruir la marcha a partir de antropometrías conocidas, **Figura 32**, donde los estudios como el de Leva et. al. [235] toman gran relevancia.



**Figura 32.** Configuración típica de experimento 3D para el análisis cinético y cinemático de la marcha por seguimiento de marcadores, Racic et. al. [55].

Otras investigaciones que se han concentrado en evaluar esta técnica de medición indirecta a partir de procesamiento de imágenes han sido McRobie et al. [84], Racic et. al. [74], Bocian et. al. [238], Gómez et. al. [128] y Firus et. al. [232], con el objetivo de adquirir la cinética de la marcha y mediante las bondades de la dinámica inversa determinar las variables cinéticas. Existen otros métodos indirectos de cálculo como el desarrollado por Oh et. al. en [239] aplicando redes neuronales artificiales, con el fin de estimar la cinética de la marcha humana.



### 2.1.7 Artículos de revisión en literatura.

El interés en las últimas dos décadas por el estudio de los fenómenos asociados con la serviciabilidad de vibraciones de los puentes peatonales debido a las solicitaciones antrópicas ha despertado un desarrollo de investigaciones a gran escala alrededor del mundo. Posicionando esta problemática como uno de los temas de mayor relevancia en la ingeniería estructural en los próximos años. Debido a este gran flujo de proyectos de investigación diversos autores se encuentran activamente reorganizar los estudios de mayor relevancia, de uno o varios ámbitos de esta problemática, con el fin de realizar trabajos de revisión los cuales se conocen comúnmente como: *A literatura review*. En esta sección, se espera recolectar las revisiones literarias de mayor impacto relacionados con los efectos de IHE en puentes peatonales y ordenarlas cronológicamente, describiendo los ámbitos de la problemática en que centraron su estudio. Esta reorganización es mostrada en la **Tabla 2**.

**Tabla 2.** Revisiones en la literatura de los efectos de IHE en Puentes peatonales.

| Autor                   | Título                                                                                                      | Referencia | Descripción                                                                                                                                                                                                                                                                                                                                                                         |
|-------------------------|-------------------------------------------------------------------------------------------------------------|------------|-------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|
| Sachse R.<br>et. al.    | Human-Structure<br>Dynamic Interaction<br>in Civil Engineering<br>Dynamics.<br>2003                         | [240]      | <ul style="list-style-type: none"> <li>Efectos de la dinámica estructural sobre la marcha humana.</li> <li>Influencia de los peatones sobre propiedades dinámicas de la estructura.</li> </ul>                                                                                                                                                                                      |
| Živanović<br>S. et. al. | Vibration<br>serviceability of<br>footbridges under<br>human-induced<br>excitation.<br>2005                 | [3]        | <ul style="list-style-type: none"> <li>Caracterización de la fuente de vibración.</li> <li>Identificación del medio por el cual se transmite la vibración.</li> <li>Efectos sobre el receptor de la vibración.</li> <li>Revisión de más de 200 referencias a la fecha de publicación.</li> </ul>                                                                                    |
| Racic V. et.<br>al.     | Experimental<br>identification and<br>analytical modelling<br>of human walking<br>force.<br>2009            | [55]       | <ul style="list-style-type: none"> <li>Caracterizaciones experimentales y analíticas de marcha humana.</li> <li>Revisión de 270 referencias a la fecha de publicación.</li> </ul>                                                                                                                                                                                                   |
| Venuti F.<br>et. al.    | Crowd-Structure<br>Interaction in lively<br>footbridges under<br>synchronous lateral<br>excitation.<br>2009 | [54]       | <ul style="list-style-type: none"> <li>Problemas de vibración lateral inducido por sincronización de peatones, Sincronización de Excitación Lateral (SEL).</li> <li>Segregación de los diferentes componentes del sistema dinámico global acoplado (Estructura-Multitud).</li> <li>Modelación explícita de la interacción entre sistemas mecánicos y los sistemas vivos.</li> </ul> |



|                              |                                                                                            |      |                                                                                                                                                                                                                                                                                                                                                                                                                                                                            |
|------------------------------|--------------------------------------------------------------------------------------------|------|----------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|
| <i>Ingólfsson E. et. al.</i> | Pedestrian-induced lateral vibrations of footbridges.<br>2012                              | [1]  | <ul style="list-style-type: none"> <li>Estudios relacionados con las vibraciones laterales inducidas, especialmente en la última década a la fecha de publicación.</li> <li>Pruebas a escala real de puentes peatonales sometidos a cargas de multitudes.</li> <li>Estudios en laboratorio sobre IHE entre peatones y plataformas instrumentadas.</li> <li>Modelación numérica de las cargas inducidas por peatones.</li> </ul>                                            |
| <i>Racic V. et. al.</i>      | Modern facilities for experimental measurement of dynamic loads induced by humans.<br>2012 | [56] | <ul style="list-style-type: none"> <li>Visión general de las tecnologías e instalaciones disponibles para determinar fuerzas dinámicas inducidas por peatones en las estructuras civiles.</li> <li>Revisión de los equipos tradicionales para la medición directa de las fuerzas dinámicas de marcha.</li> <li>Identificación del potencial de sistemas ópticos y no ópticos de seguimiento de movimiento para la identificación de la cinemática de la marcha.</li> </ul> |
| <i>Maraveas C. et. al.</i>   | Human Induced vibrations on footbridges.<br>2015                                           | [53] | <ul style="list-style-type: none"> <li>Vibraciones inducidas por el hombre que experimentan los puentes peatonales flexibles.</li> <li>Información cualitativa de forma exhaustiva para proporcionar métodos comunes y recomendaciones de códigos.</li> <li>Se aborda la Sincronización de Excitación Lateral (SEL).</li> </ul>                                                                                                                                            |
| <i>Shahabpoor E. et. al.</i> | Interaction between Walking Humans and Structure in Vertical Direction.<br>2016            | [52] | <ul style="list-style-type: none"> <li>Simulación de los efectos de la IHE debido a la marcha humana en sentido Vertical.</li> <li>Se destacan ámbitos con necesidad de más investigación.</li> </ul>                                                                                                                                                                                                                                                                      |

## 2.2 Antecedentes y Estado del arte: Simulación Híbrida en Tiempo-Real (SHTR)

El aumento de la capacidad de resiliencia de la Infraestructura Civil ante solicitudes dinámicas se encuentra directamente relacionado con el desarrollo y aplicación de las tecnologías emergentes para la mitigación de la sensibilidad dinámica en las estructuras. Las evaluaciones sobre el desempeño dinámico de los sistemas estructurales con estas novedosas tecnologías pueden ser clasificados en dos grupos importantes: investigaciones analíticas y experimentales [156], [157], [158], [160], [203], [241], [242].



El vertiginoso desarrollo computacional de los últimos años hace tentativo contemplar la posibilidad de reemplazar los ensayos experimentales por modelaciones analíticas en diferentes disciplinas de la ingeniería; algunos ejemplos de esta situación es posible encontrarlos en campos de estudio como: nuclear y termodinámico [243], reología del concreto [244], mecánica de fluidos [166] y bioingeniería [1]. Sin embargo, los ensayos experimentales son considerados por muchos investigadores como esenciales para determinar el comportamiento dinámico de las estructuras civiles a baja, mediana y gran escala. Estas campañas de experimentación han sido los pilares para el desarrollo de normativas de infraestructura y para el estudio de elementos estructurales de interés, puesto que proporcionan información relevante del comportamiento lineal e inelástico de su respuesta dinámica, y del desempeño de las tecnologías emergentes propuestas para la mitigación de la vulnerabilidad estructural. Además, esta información experimental permite el desarrollo y la calibración de los modelos analíticos para análisis computacionales más precisos y confiables [164], [165], [167], [170], [171], [216], [245], [246], [247].

Aunque los ensayos experimentales son una parte importante en casi la totalidad de los estudios del desempeño dinámico de la Infraestructura Civil, muy a menudo estas pruebas se encuentran limitadas por condiciones físicas como el tamaño del laboratorio y las cargas de servicio de los equipos, así como por el presupuesto y el calendario de ejecución de los proyectos. Los ensayos dinámicos a gran escala de estructuras civiles (edificaciones, puentes peatonales, puentes ferroviarios, etc.) son generalmente costosos y demandan una cantidad importante de tiempo. Además, requieren de una gran capacidad en los equipos hidráulicos como mesas sísmicas y/o muros de reacción; los cuales en muchos ocasiones se ven superados en su capacidad [170], [171]. Las pruebas sobre mesas sísmicas (*Shake Table Test*, STT) han sido históricamente los principales métodos de ensayo para la evaluación del desempeño dinámico de sistemas estructurales debido a que desarrollan adecuadamente condiciones muy realistas [161], [162], [203], [248], [249]. Sin embargo, en contraste con su frecuente uso esta técnica de ensayo presenta fuertes desventajas debido a sus altos costos de operación, restricciones de tamaño y peso en los especímenes de prueba; dadas las limitantes de capacidad de la mayoría de estos dispositivos alrededor del mundo. Lo que ha generado la realización de STT con sistemas estructurales a menor escala, donde la evaluación del desempeño de estas estructuras es extrapolada bajo un análisis detallado de reglas de similitud de los materiales, geometría y propiedades intrínsecas; las cuales son una fuente de inexactitud y propagación de errores en las respuestas dinámicas [159], [244], [246], [250]. Con esta gama de limitaciones cada vez más complejas, producto de la constante evolución de la experimentación dinámica, se hace evidente la necesidad de técnicas de ensayo a la vanguardia que permitan realizar evaluaciones lo suficientemente detalladas del comportamiento dinámico de los sistemas de Infraestructura Civil y/o componentes bajo cargas dinámicas considerables, como en un evento sísmico, cargas eólicas o cargas antrópicas [251], [252].

Una innovadora y versátil técnica de investigación para la evaluación del comportamiento dinámico de sistemas estructurales, conocida en la literatura como Simulación Híbrida (SH), se ha desarrollado activamente en las últimas décadas [160], [161], [162], [203], [247], [253], [254], [255]. Esta técnica de ensayo, en el campo estructural, fue propuesta inicialmente por Hakuno et. al. [253] en el año 1969 para el ensayo de una viga Cantilever como componente experimental de un sistema dinámico analítico; y fue renombrada a mediados de los años 70's por Takanashi como *On-line Test* [155], [160], [256], [257], [258]. Este método de prueba consisten en combinar los análisis numéricos y experimentales, empleando las ventajas que pueden proporcionar ambos conceptos. Las SH son una metodología experimental en la cual la porción más crítica o de interés en el sistema, que generalmente expone un comportamiento no-



lineal, es testeada experimentalmente. Mientras que el resto del sistema, que presenta un comportamiento predecible numéricamente con alto grado de precisión, es simultáneamente analizado en un modelo numérico. Ambos componentes se denominan subestructura experimental y numérica, respectivamente. Las subestructuras son conectadas mediante la implementación de actuadores, los cuales componen los sistemas de transferencia, y sensores que proveen los medios para la interacción entre la interfaz de ambas subestructuras, formando un bucle para resolver la respuesta dinámica del sistema en cada paso de tiempo [156], [158], [159], [162], [163], [164], [170], [171], [245], [254], [259], [260], [261]. Esta novedosa técnica de investigación tiene el potencial, dada su versatilidad, de simular la respuesta dinámica de especímenes a gran escala lo que la hace una metodología aplicable a los sistemas estructurales civiles en general [165], [216]. Además, la principal ventaja de las SH radica en el hecho de concentrar los recursos en la parte crítica o de interés del sistema teniendo como resultado menores costos, mejores rendimientos de tiempo en ejecución y mayor desempeño frente a las limitaciones de capacidad comparado con experimentación dinámica a gran escala tradicional (STT, ensayos cuasi-estáticos, etc.). Y lo más imprescindible, sin perder la fiabilidad y precisión en las respuestas dinámicas del sistema estructural evaluado.

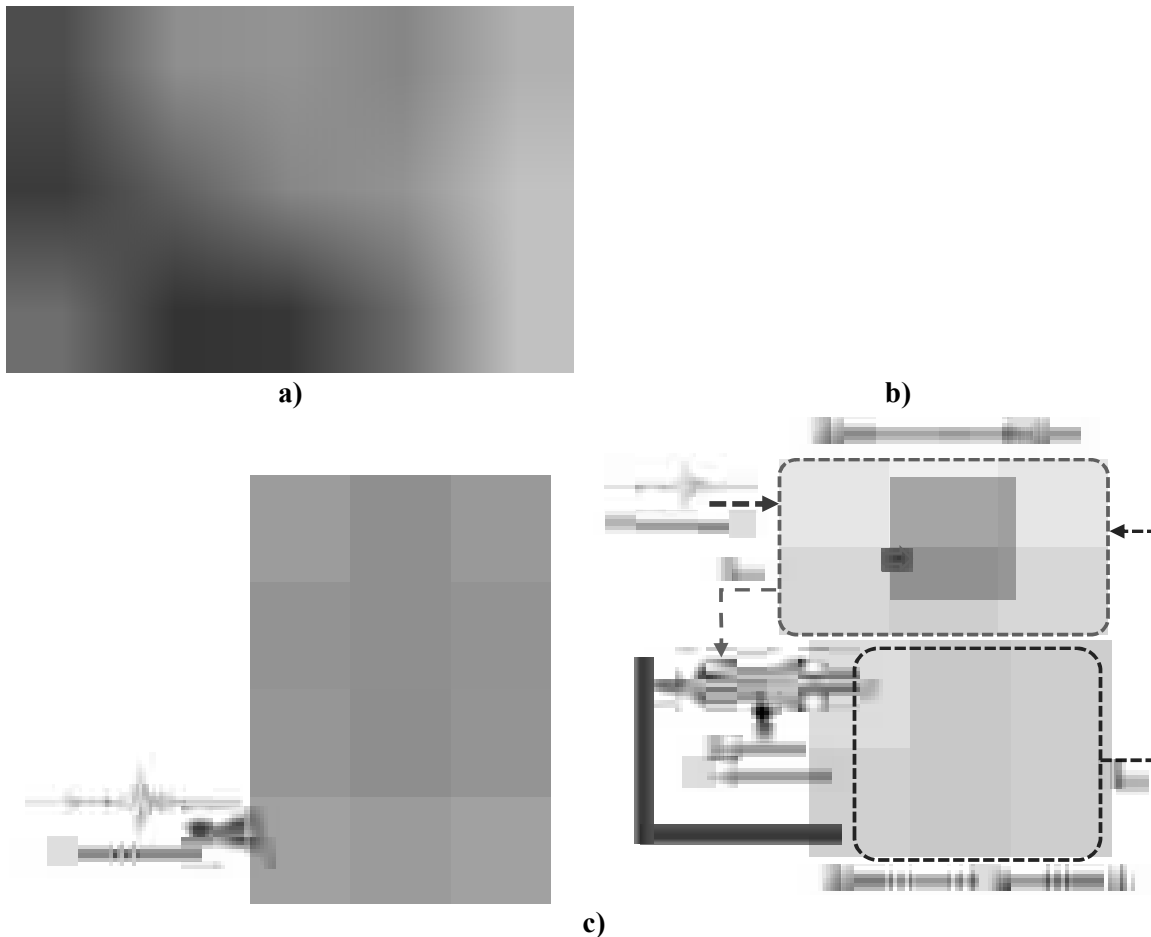


**Figura 33.** Simulaciones Híbridas; **a)** Montaje experimental Hakuno et. al. [253]; **b)** Diagrama de bloques esquemático del funcionamiento de Online Test, Takanashi et. al. [155], [258]; **c)** Esquema del funcionamiento de las Simulaciones Híbrida.



Las pruebas experimentales que incluyen analíticamente los efectos inerciales, implementando modelos numéricos del sistema, son conocidas comúnmente como ensayos Pseudo-dinámicos (PSD) [155], [161], [254]. Estas pruebas PSD tienen como objetivo la evaluación del componente experimental de rigidez de la probeta experimental para su calibración o retroalimentación de modelos numéricos como histéresis. En el momento de implementar el concepto de subestructuración dinámica se da como resultado los ensayos de Simulación Híbrida Pseudo-Dinámicos (Pseudo-Dynamic Hybrid Simulation, PSDHS) en los cuales la subestructura experimental conforma la rigidez y amortiguamiento, y la parte numérica lo restante [165]. La ventaja de las pruebas PSD o PSDHS radica en que no requieren cargas dinámicas para su ejecución, aún para simular la respuesta dinámica de un elemento estructural durante una excitación dinámica, esto se debe a la formulación numérica de los efectos inerciales. Lo anterior permite la realización de ensayos a gran escala, dado que la capacidad de carga de los actuadores cuasi-estáticos es generalmente mayor que en el caso dinámico; permitiendo adquirir detalladamente el desempeño del elemento estructural [156], [161], [163], [254]. La implementación de esta técnica se considera válida para una gran variedad de estructuras construidas en materiales convencionales como concreto reforzado o acero, siempre que los deltas de carga se mantengan en niveles producidos por las excitaciones dinámicas. Este hecho es una de las razones por las cuales esta metodología experimental ha sido acogida en gran medida [161], [231]. Sin embargo, una serie de diversas tecnologías se han acoplado a los sistemas estructurales para el control y mitigación de la respuesta dinámica, en los cuales su desempeño depende directamente de la velocidad [216], [250]. Desafortunadamente, las pruebas PSD o PSDHS no son técnicas aplicables en esta situación debido a que las cargas generalmente son aplicadas en amplias escalas de tiempo, por lo que los efectos dinámicos dependientes del tiempo son suprimidos en estas tecnologías implementadas [156], [161], [163], [164], [165].

Como una forma de abordar las dificultades presentadas por las pruebas PSD y/o PSDHS, una serie de investigadores en las últimas dos décadas plantean la implementación de pruebas de Simulación Híbrida en Tiempo-Real (SHTR). Esta técnica de ensayo mantiene las bases de la subestructuración dinámica tradicional, como en las SH, pero incorpora unas condiciones de interacción en la interfaz de las subestructuras debido a su ejecución en Tiempo-Real [162], [231], [252]. El principal avance de la técnica de SHTR respecto a sus antecesoras radica en su capacidad de reproducir con altos niveles de fidelidad la velocidad, lo que permite desarrollar investigaciones en la ingeniería estructura con excitaciones dinámicas de la respuesta local del elemento/dispositivo o global del sistema estructural en estudio [165], [246]. De igual manera, los potenciales impactos de las SHTR se reflejan en: la observación y seguimiento de las condiciones operacionales de sistemas experimentales sin la utilización de modelos numéricos limitados en los elementos con condiciones altamente no-lineales[231]; el desarrollo de una metodología de ensayo económica, eficiente tanto en tiempo como en espacio y relativamente rentable para la evaluación de sistemas estructurales con dispositivos emergente, en los cuales su comportamiento es directamente dependiente de la velocidad, obteniendo resultados comparables con técnicas de experimentales tradicionales [159], [167], [214], [262]; la evaluación de numerosos escenarios posibles de prueba de una tecnología, situación que no es viable con técnicas tradicionales [216], [263]; y por último la transferencia de tecnología desde otras disciplinas ingenieriles como la aeroespacial o Automovilística [166].

Las ventajas presentadas por la técnica de SHTR tienen el potencial para desarrollar una perspectiva de ensayos sistemáticos enfocada a la evaluación de la respuesta dinámica global, permitiendo el monitoreo del desempeño del componente de interés del sistema dinámico en condiciones prácticas y operacionales, y no de forma aislada e independiente [166]. No obstante, las pruebas de SHTR aún no han alcanzado



un estado de madurez total puesto que, generalmente, los análisis donde se han implementado son enfocados principalmente a la evaluación del desempeño dinámico de edificaciones con tecnologías emergentes para el control de la respuesta sísmica [170]; y son pocas las investigaciones que se orientan a la implementación de SHTR para la evaluación de la Infraestructura Civil bajo fuentes de excitación dinámica como cargas antrópicas, eólicas e/o hidráulicas. Aunque este panorama, a nivel internacional, está cambiando paulatinamente en los últimos años. En Colombia, esta técnica se encuentra en sus primeras fases de desarrollo y promete ser una alternativa viable técnica y económicamente para la evaluación del comportamiento de Infraestructura Civil, no solo bajo cargas sísmicas, sino bajo diversas cargas dinámicas. En particular, la implementación de SHTR en las investigaciones de la serviciabilidad de vibraciones de los puentes peatonales debido a la excitación dinámica de cargas de marcha humana se dislumbra como un camino prometedor e incluso ya se pueden encontrar algunos avances relacionados a la subestructuración del sistema dinámico acoplado en los efectos de IHE. Aunque los avances de Mulas et. al. [43], Chen et. al. [264] y Gómez et. al. [41] se han desarrollado completamente analíticos presentan viable la subestructuración del sistema estructural (Puente peatonal) como un componente numérico y al sistema humano como un componente experimental, dado el comportamiento no-lineal intrínseco de su marcha.

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## **Chapter No.3. Evaluation of Dynamic Human-Structure Interaction Effects on Footbridges using Real-Time Hybrid Simulation.**

### ***3.1 Human-Structure Interaction Multiaxial Test Framework (HSI-MTF): A Cutting-Edge System for Evaluating Multiaxial Gait-Induced Loads.***

The development of modern civil infrastructure demands solutions that address dynamic interactions between human activity and structural behavior. The Human-Structure Interaction Multiaxial Test Framework (HSI-MTF) represents a pioneering approach in evaluating multiaxial loads induced by human gait on vibrating surfaces, a critical factor in ensuring structural serviceability and safety, especially in pedestrian bridges. By simulating lateral and vertical harmonic motions, the HSI-MTF enables comprehensive analysis of human-induced loads across multiple axes, bridging the gap between theoretical models and practical application.

This innovative system integrates advanced biomechanical and structural dynamic analysis, allowing for precise measurement of gait-induced loads under realistic, variable frequency conditions. The HSI-MTF incorporates both lateral and vertical motion components, offering unparalleled insight into how human movement interacts with flexible structures, such as footbridges. It combines state-of-the-art sensor technology with a robust hydraulic actuation system, ensuring accurate data collection even in complex, multiaxial environments.

Through controlled experimental protocols, the HSI-MTF provides engineers and researchers with critical data, helping to refine predictive models and inform the design of safer, more resilient infrastructure. With its potential applications in urban environments, this framework is set to become a cornerstone in the ongoing evolution of civil infrastructure management and design, meeting the increasing demands for both aesthetic appeal and structural integrity.

Each article plays a crucial role in describing the development and functionality of the Human-Structure Interaction Multiaxial Test Framework (HSI-MTF). The first article, *"A Multiaxial Test Framework for the Evaluation of Human Gait-Induced Loads on Lateral Harmonic Surfaces"*, provides a foundational overview of the HSI-MTF, focusing on its design and experimental capabilities for capturing three-dimensional gait-induced loads under lateral harmonic motion. It highlights the technical specifications and testing protocols, demonstrating how the framework effectively simulates human-structure interaction in real-world conditions. On the other hand, the second article, *"Experimental Evaluation of Pedestrian-Induced Multiaxial Gait Loads on Footbridges"*, expands the application of the HSI-MTF by exploring how human gait changes in response to different structural vibrations, particularly under the effects of lateral displacement. This study contributes valuable experimental data on how human gait dynamics interact with flexible surfaces, providing insight into Structure-to-Human Interaction (S2HI) effects. Together, both articles establish the HSI-MTF as a versatile tool for analyzing multiaxial gait-induced loads, combining theoretical modeling with practical experimentation to enhance the understanding of dynamic human-structure interactions.

## Article

# Experimental Evaluation of Pedestrian-Induced Multiaxial Gait Loads on Footbridges: Effects of the Structure-to-Human Interaction by Lateral Vibrating Platforms

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**Abstract:** The introduction of resistant and lightweight materials in the construction industry has led to civil structures being vulnerable to excessive vibrations, particularly in footbridges exposed to human-induced gait loads. This interaction, known as Human–Structure Interaction (HSI), involves a complex interplay between structural vibrations and gait loads. Despite extensive research on HSI, the simultaneous effects of lateral structural vibrations with fundamental frequencies close to human gait frequency (around 1.0 Hz) and wide amplitudes (over 30.0 mm) remain inadequately understood, posing a contemporary structural challenge highlighted by incidents in iconic bridges like the Millennium Bridge in London, Solferino Bridge in Paris, and Premier Bridge in Cali, Colombia. This paper focuses on the experimental exploration of Structure-to-Human Interaction (S2HI) effects using the Human–Structure Interaction Multi-Axial Test Framework (HSI-MTF). The framework enables the simultaneous measurement of vertical and lateral loads induced by human gait on surfaces with diverse frequency ranges and wide-amplitude lateral harmonic motions. The study involved seven test subjects, evaluating gait loads on rigid and harmonic lateral surfaces with displacements ranging from 5.0 to 50.0 mm and frequency content from 0.70 to 1.30 Hz. A low-cost vision-based motion capture system with smartphones analyzed the support ( $T_{su}$ ) and swing ( $T_{sw}$ ) periods of human gait. Results indicated substantial differences in  $T_{su}$  and  $T_{sw}$  on lateral harmonic protocols, reaching up to 96.53% and 58.15%, respectively, compared to rigid surfaces. Normalized lateral loads ( $L_L$ ) relative to the subject's weight ( $W_0$ ) exhibited a linear growth proportional to lateral excitation frequency, with increased proportionality constants linked to higher vibration amplitudes. Linear regressions yielded an average  $R^2$  of 0.815. Regarding normalized vertical load ( $L_V$ ) with respect to  $W_0$ , a consistent behavior was observed for amplitudes up to 30.0 mm, beyond which a linear increase, directly proportional to frequency, resulted in a 28.3% increment compared to rigid surfaces. Correlation analyses using Pearson linear coefficients determined relationships between structural surface vibration and pedestrian lateral motion, providing valuable insights into Structure-to-Human Interaction dynamics.

**Keywords:** human–structure interaction; anthropic loads; human gait; synchronization; structural vibrations



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## 1. Introduction

The limitations of serviceability due to vibrations in civil structures, reported on various occasions in structural engineering, are often related to the dynamic coupling between anthropic loads and the dynamic response of these structures. The dynamic relationships between pedestrians and structures have been extensively documented in the literature for civil structures, such as stairs [1,2], grandstands [3,4], slabs [5,6], and footbridges [7–16]. Footbridges have become one of the most affected structures in terms of vibration serviceability limit state owing to their natural condition of spanning large distances and the progressive incorporation of lighter and more resistant materials in the

construction industry, which generates relatively low masses and damping and increases technical and aesthetic demands [17–19]. Consequently, these conditions generate structures that are increasingly slender and flexible, increasing their dynamic sensitivity to excessive vibrations owing to the action of loads induced by human activity [20,21]. The events that occurred on the Millennium Bridge in London during its inauguration support the previously mentioned idea, as excessive vibrations occurred in this structure due to the lateral synchronization of forces induced by pedestrians crossing the bridge and the dynamic response of the structure in this direction. This coupling resulted in accelerations close to 0.25 g with displacements of up to 7.0 cm [22,23].

Dallard et al. [13] initially described the dynamic effects in the Millennium Bridge, and the review conducted by Zivanovic et al. [17] positioned the effects of dynamic Human–Structure Interaction (HSI) as one of the topics of increasing interest in the scientific community during the last two decades [24–26]. However, this event has not been configured as an isolated case because there are historical records of similar situations [27]. The oldest cases for which there is technical reference were the collapse of the Broughton Bridge in the UK due to the dynamic loads induced by the harmonic march of 60 infantry soldiers crossing the structure [28], the lateral vibration affectations on the Toda-Park Bridge in Japan [10], and the Solferino Bridge in France [29]. Additionally, more recent incidents have been reported in this type of structure caused by anthropic forces induced, such as the cases of The Wuhan Yangtze Bridge, China [30]; Pedro and Inês Bridge, Coimbra-Portugal [14,15]; Long Span Steel Bridge, Changi Airport, Singapore [11,12]; Le Ponte del Mare, Pescara, Italy [31]; Goodwill Bridge, Brisbane, Australia [32]; Eeklo Bridge, Belgium [8,9]; and the Folke Bernadotte Bridge, Sweden [33], among others [34–38].

These footbridges, which have been affected by HSI effects, promote the hypothesis of great urban regeneration on a global level, where isolated segments are interconnected with higher technical, aesthetic, architectural, and engineering requirements [39]. This innovative perspective led to a remarkable revolution in the design, construction, and management of these structures. However, their sensitivity to anthropic forces is noteworthy, considering that the dynamic behavior of footbridges is generally associated with frequencies lower than 10.0 Hz. The vertical and lateral frequency content characteristics of these structures, typically between 1.6 and 2.4 Hz, establish a focus of sensitivity to the coupling between their structural vibrations and the pedestrian gait dynamics, which is classified as the most recurrent source of excitation in the footbridges, with a typical frequency content between 0.8 and 1.2 Hz [40]. This implies a latent risk that footbridges and pedestrian gaits may be affected by a synchronization phenomenon.

The coupling effects of anthropic loads with the vibrations of pedestrian bridges are due to the adaptive [41], random [42,43], and nonstationary conditions of the human gait [17,44]. HSI effects in flexible surfaces are configured as an advanced case of synchronization and are composed of three main important aspects: changes in the dynamic properties of the structure due to pedestrians, Human-to-Structure Interaction (H2SI) [45–47]; interactions between pedestrians in crowds, Human-to-Human Interaction (HHI) [48–50]; and changes in the pedestrian gait due to the structure's vibration, Structure-to-Human Interaction (S2HI) [18,19,43,51,52]. Although many researchers have concluded that studies and interest in these topics have increased considerably in recent decades, aspects related to the behavior of the loads induced by the human gait in response to structural vibrations have not been fully detailed or understood [9,53]; for example, the effects of structural vibrations on pedestrian-induced gait loads in vertical and lateral directions simultaneously.

Human-induced gait loads produce a total ground reaction force in each direction composed of two principal contributions: passive and active components of the HSI effects. The passive component is related to the dynamic human response to the vibration of the footbridge; in contrast, the active component is associated with the active motion of the pedestrian on the structure [42,54–57]. The structural vibrations control the dynamic coupling with human-induced loads in a vertical direction. For lateral vibrations, additional

effects have been observed, including a synchronization phenomenon of pedestrians with the vibration of the structure and interaction between pedestrians in the case of densely occupied structures [57,58].

In an effort to push the boundaries of research in HSI, this paper presents a comprehensive study on the assessment of experimental multiaxial S2HI effects. Although researchers such as Nakamura et al. [59], Pizzimenti and Ricciardelli [60], Ingólfsson et al. [61], Bocian et al. [57,62,63], and, more recently, Zhang et al. [64], have made significant strides in this field of study, developing experimental campaigns to evaluate the human gait on vibrating lateral surfaces using instrumented treadmills or force plates, none of these studies have tackled the experimental evaluation of simultaneous vertical and lateral human loads as was performed in this investigation. This paper is structured as follows. Section 2 describes the experimental methods used in this study; five subsections were set up: general description, anthropometric information of the STs, experimental setup, data processing, and verification of the experimental setup. In Section 3, the results of the experimental gait tests on rigid and vibrating surfaces are presented and discussed. Finally, Section 4 presents the conclusions of this investigation.

## 2. Experimental Methods

### 2.1. Investigation General Design

This study experimentally assessed the spatiotemporal and kinetic behavior of human gait using an experimental test framework called the Human–Structure Interaction Multi-axial Test Framework (HSI-MTF) and a low-cost marker-based motion capture (MOCAP) system with smartphone devices. From prior data on the study population [65], seven test subjects (TS) were selected for inclusion in this research. This number was determined considering practical limits while ensuring that the chosen subjects reliably represented the variability of the population. Initially, STs were equipped with passive retroreflective markers. Then, the STs had an adaptive period of walking at a regular pace for 2.0 min with constant lateral vibration. Later, they executed gait protocols of 30.0 s each with different frequency and displacement for the lateral movement of the platform, while body kinematics, videos, and kinetic data were collected. The data collected during the tests were used to assess the vertical and lateral behavioral kinetics of human gait on surfaces with harmonic lateral motion in comparison with its behavior on a rigid surface.

### 2.2. Subjects of the Test

Seven STs (7 males,  $23.0 \pm 3.4$  years,  $172.9 \pm 4.6$  cm, and  $71.9 \pm 5.2$  kg) volunteered to participate in this investigation. Their anthropometric information is shown in Table 1, according to the scheme shown in Figure 1. All STs were free of injuries for at least 1.0 month, had a body mass index (BMI) of  $<25.5$  (normal weight level), and were aged 20–30 years. All tests involving human test subjects were carried out in accordance with the Code of Ethics of the World Medical Association (Declaration of Helsinki) for experiments involving humans and the Universidad del Valle Ethics Committee.

**Table 1.** Anthropometric information.

|                |    | Test Subject |       |       |       |       |       |       |
|----------------|----|--------------|-------|-------|-------|-------|-------|-------|
|                |    | No. 1        | No. 2 | No. 3 | No. 4 | No. 5 | No. 6 | No. 7 |
| Length<br>(cm) | D1 | 27.2         | 24.7  | 23.6  | 27.0  | 28.0  | 31.0  | 35.0  |
|                | D2 | 47.5         | 47.5  | 47.2  | 42.0  | 57.0  | 55.0  | 54.0  |
|                | D3 | 22.3         | 19.8  | 21.7  | 20.0  | 22.0  | 20.5  | 40.0  |
|                | D4 | 18.7         | 16.7  | 15.8  | 8.0   | 9.0   | 6.0   | 13.0  |
|                | D5 | 31.4         | 30.6  | 30.2  | 28.0  | 30.0  | 23.0  | 27.0  |
|                | D6 | 26.7         | 27.4  | 26.7  | 24.0  | 23.0  | 25.0  | 27.0  |

Table 1. Cont.

|                     |             | Test Subject |       |       |       |       |       |       |
|---------------------|-------------|--------------|-------|-------|-------|-------|-------|-------|
|                     |             | No. 1        | No. 2 | No. 3 | No. 4 | No. 5 | No. 6 | No. 7 |
| Length<br>(cm)      | D7          | 17.7         | 19.2  | 19.8  | 19.0  | 22.0  | 17.0  | 33.0  |
|                     | D8          | 47.5         | 43.0  | 44.2  | 48.0  | 48.0  | 48.0  | 45.0  |
|                     | D9          | 46.7         | 46.5  | 45.8  | 43.0  | 43.0  | 42.0  | 41.0  |
|                     | D10         | 9.8          | 10.4  | 10.1  | 9.0   | 9.0   | 11.0  | 12.0  |
|                     | D11         | 17.5         | 15.5  | 15.2  | 15.0  | 15.0  | 15.0  | 14.0  |
|                     | D12         | 19.4         | 18.6  | 17.9  | 17.0  | 18.0  | 18.0  | 18.0  |
| Diameter<br>(cm)    | Head        | 57.0         | 58.0  | 54.0  | 56.0  | 56.0  | 53.0  | 58.0  |
|                     | Shoulder    | 37.0         | 47.0  | 52.0  | 35.0  | 36.0  | 37.0  | 38.0  |
|                     | Elbow       | 25.0         | 27.5  | 28.5  | 25.5  | 27.0  | 26.5  | 27.0  |
|                     | Wrist       | 17.0         | 16.0  | 17.0  | 16.0  | 18.0  | 16.5  | 16.0  |
|                     | Hip         | 90.0         | 85.5  | 88.5  | 91.0  | 104.0 | 91.0  | 92.0  |
|                     | Knee        | 38.9         | 38.5  | 41.0  | 34.5  | 43.0  | 38.5  | 36.0  |
|                     | Ankle       | 25.0         | 24.5  | 25.0  | 26.0  | 26.0  | 23.0  | 21.0  |
|                     | Heel        | 32.0         | 23.0  | 23.0  | 30.5  | 33.0  | 34.0  | 30.0  |
|                     | Metatarsus  | 24.0         | 24.0  | 54.0  | 22.0  | 23.5  | 27.5  | 58.0  |
| General<br>Informa. | Age         | 30           | 21    | 23    | 23    | 24    | 22    | 29    |
|                     | Height (cm) | 178          | 179   | 174   | 173   | 170   | 166   | 170   |
|                     | Weight (kg) | 70.0         | 74.0  | 76.0  | 70.0  | 80.0  | 69.0  | 64.0  |

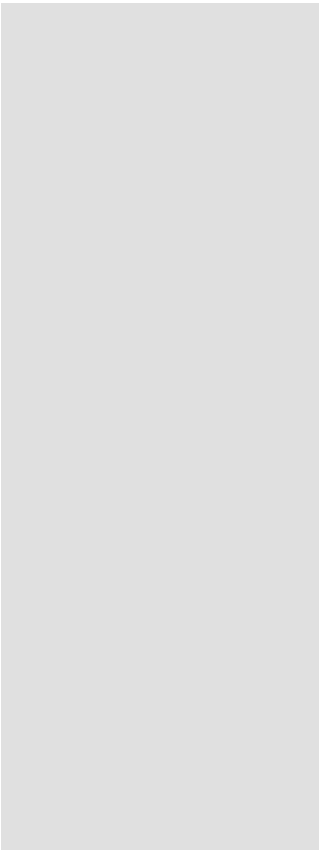


Figure 1. Schematic anthropometric information.

### 2.3. Assembly Setup

An experimental test framework called the Human–Structure Interaction Multiaxial Test Framework (HSI-MTF), positioned in the Laboratory of Seismic Engineering and Structural Dynamics (LINSE) at the Universidad del Valle, Cali, Colombia, was developed to evaluate the kinetic effects on human gait produced by surfaces with lateral harmonic motion. This framework allowed the generation of three main conditions: (i) the repetitiveness of human gait for the evaluation of its stochastic and random nature; (ii) the incidence of net lateral motion on test subjects; and (iii) the acquisition of the generated anthropic loads. One or two of these characteristics are present in the setups performed by Nakamura et al. [59], Ingólfsson et al. [61], Pizzimenti et al. [66], Carroll et al. [46], Bocian et al. [57], and more recently, Haowen et al. [67]. However, none of these studies performed a biaxial configuration for the lateral and vertical evaluation of the loads induced by the human gait under the effects of a surface with lateral motion, which is the condition addressed in this research. The HSI-MTF, schematically shown in Figure 2, consists of four main components, as listed in Table 2.

**Table 2.** Main components HSI-MTF.

| Item No. | Components Name            | Description                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                         |
|----------|----------------------------|-------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|
| 1        | Unidirectional shake table | The custom unidirectional shake table of the School of Civil and Geomatic Engineering, consists of an aluminum plate of 1.10 m × 1.50 m directed by low-friction linear rails and coupled to a Shore Western hydraulic actuator of 45.0 kN (Shore Western, Monrovia, CA, USA). This actuator has two DyVal servo-valves that are connected to a hydraulic potential of up to 977.9 cm <sup>3</sup> ·s <sup>−1</sup> at 21 MPa. In addition, it has an internal linear variable differential transducer (LVDT) that measures the position of the actuator rod. Recent work has allowed the identification of this system through frequency sweep tests from 0.10 to 20.0 Hz, with a duration of 250.0 s and constant amplitudes of 0.50, 1.0, 3.0, and 5.0 mm. A representation of the GHA plant of the seismic simulator was found to be a continuous system with a pole at the origin, a nominal gain of 14.0, and a time delay of 33.0 ms. In addition, this device was implemented with a robust H <sub>∞</sub> displacement tracking controller conditioned by a Kalman filter. |
| 2        | Lateral support system     | The lateral support system initially consisted of four custom ASTM A36 steel plates with a thickness of 1.27 cm, which were intended to increase the working area of component 1. Four lateral supports with a height of 74.0 cm were placed on top of the plates, which connected the guides of the vertical linear axes of the HSI-MTF. In addition, it supports a human gait loading acquisition system.                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                         |
| 3        | Treadmill                  | A SOLE brand treadmill model F65 was used (Sole Fitness, Salt Lake City, UT, USA), which has an effective action area of 72.0 cm × 150.0 cm and operating speeds ranging from 0.50 to 10.0 cm <sup>3</sup> ·s <sup>−1</sup> . The forward motion is generated by a 120.0 V commercial motor controlled by the factory hardware. The treadmill was compacted and coupled with the HSI-MTF.                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                           |
| 4        | Load acquisition system    | This system was developed to quantify the anthropic loads induced by human gait and consisted of a Ø1.59 mm steel wire with an ultimate load of 4.0 kN, attached to the lateral support system on the linear axes and to the bottom of the treadmill through four Omega DYLY-103 load cells (Omega Engineering, Norwalk, CT, USA) with an individual capacity of 200.0 kg and a sensitivity of 0.15 mV/kg. The vertical acquisition component was formed by a Ø3.18 mm steel wire with an ultimate load of 10.0 kN, attached to the lateral support system on the linear axes and the vertical suspension system through four Omega LC-703 load cells with a capacity of 90.70 kg each and a sensitivity of 0.20 mV/kg. Finally, the longitudinal acquisition component had the same steel wire attached to the most extreme parts of the vertical suspension system, using four Omega DYLY-102 load cells with a capacity of 50.0 kg each.                                                                                                                                         |

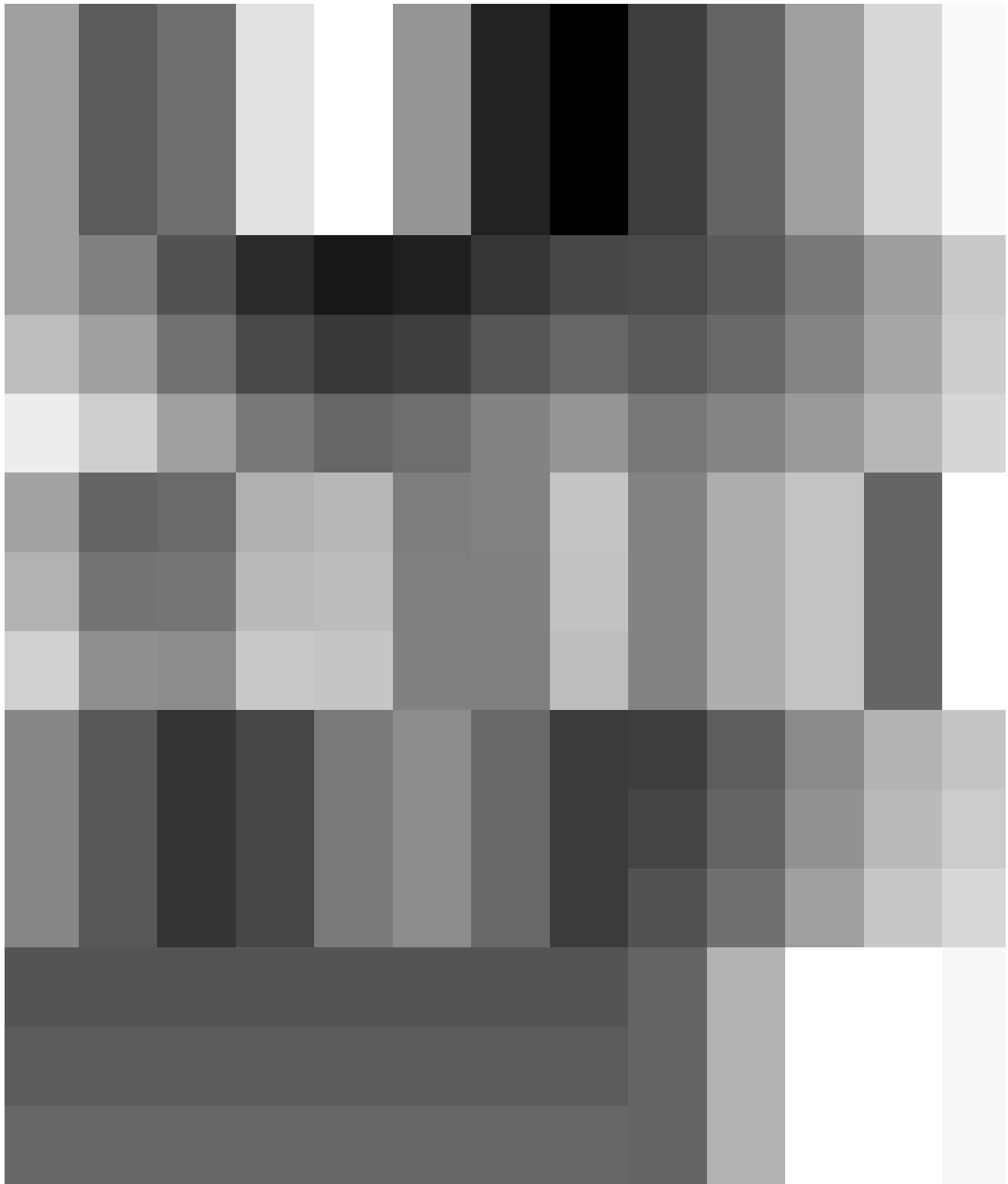


**Figure 2.** Human–Structure Interaction Multiaxial Test Framework: isometric view (**top**), frontal view (**middle**), real frontal view (**bottom**).

#### 2.4. Data Collection Procedures

Experimental tests were conducted on the HSI-MTF for the STs, as shown in Figure 3. These tests involved acquiring the lateral and vertical forces induced by human gait on rigid and flexible surfaces with a sample time of 500 Hz. The human gait was performed at a constant velocity of  $1.10 \text{ m}\cdot\text{s}^{-1}$  under lateral sinusoidal movements induced for the

HSI-MTF of 5.0, 10.0, 20.0, 30.0, and 50.0 mm, with frequencies for each displacement ranging from 0.70 to 1.30 Hz in steps of 0.10 Hz, and a duration of 30.0 s for each frequency step. In addition, a low-cost MOCAP system was implemented using smartphone devices (Xiaomi Redmi 9T (Xiaomi, Beijing, China) with 48.0 Mpx and a sampling rate of 30.0 fps) and four passive retroreflective adhesive markers with a diameter of 12.0 mm that were attached at specific points of interest: the center of mass, ankle, heel, and metatarsus (Figure 3, middle). This technology was implemented with the aim of identifying the gait cycle times and displacements in relation to the behavior of lateral loads. Before the tests, each ST carried out 2.0 min gait pre-testing at their typical gait velocity (approx.  $0.90 \text{ m}\cdot\text{s}^{-1}$ ). Seven static tests and 245 dynamic tests were performed.

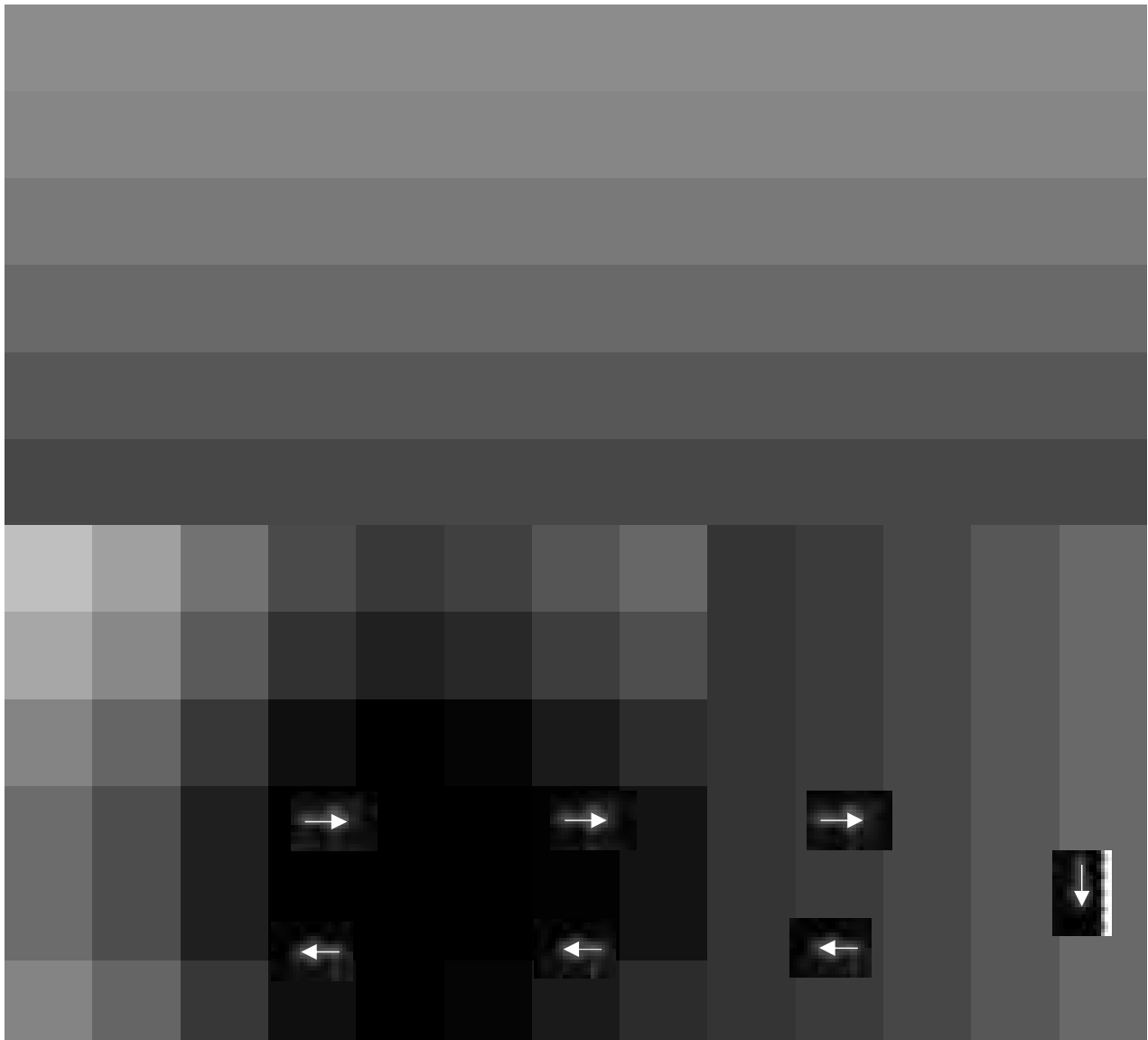


**Figure 3.** Experimental setup of the human gait test for ST No. 1. Isometric view (**top-left**), lateral view (**top-middle**), and top view (**top-right**). Additionally, a gait cycle (**bottom**).

### 2.5. Data Post-Processing

The multiaxial kinetics of human gait were obtained using the HSI-MTF, where the gait loads were measured in three dimensions. Lateral and vertical data were processed using custom algorithms in MATLAB (R2023a). The signals were re-sampled from the original 500 Hz to 20 Hz by applying a digital anti-alias in a glow pass filter. Points of interest were labeled in the sagittal plane (ankle, heel, and metatarsus) and coronal plane (center of mass—hips and HSI-MTF reference point), as shown in Figure 3. Initially, the vertical and lateral loads were acquired on rigid and flexible surfaces for each ST, as described in Section 2.4. Then, these load data were processed for a period of 30.0 s, approximately 25–30 complete gait cycles. Simultaneously, motion data of the feet and the center of mass were acquired for 30.0 s in order to calculate the average periods of the gait cycles and the behavior of the center of mass with respect to sinusoidal motions induced by the HSI-MTF.

### 2.6. HSI-MTF Lateral-Load Verification System

The lateral gait loads were measured during the tests, including the anthropic and inertial forces of the HSI-MTF. The lateral human gait load can be calculated using the equilibrium of forces according to D'Alembert's principle [68].

$$F_{Lateral\ Gait} = \sum_{i=1}^4 F_{ML,i} - m_{HSI-MTF} \cdot a_{HSI-MTF} \quad (1)$$

where  $F_{ML,i}$  denotes the measured load from the  $i$ th load cell and  $m_{HSI-MTF}$  and  $a_{HSI-MTF}$  are the mass and measured acceleration of the HSI-MTF, respectively. First, two sine waves of 7.50 mm at 1.0 Hz and 2.50 mm at 1.30 Hz were used as inputs in the system, and the lateral load and acceleration of the HIS-MTF were recorded without the action of any human gait. In this case,  $F_{LateralGait}$  should be zero. The measured inertial force, which is the first right portion of Equation (1), coincided with the one calculated with  $m_{HSI-MTF} \cdot a_{HSI-MTF}$ , as shown in Figure 4 for the first proposed sine-wave signal, which indicates a zero for  $F_{LateralGait}$ , according to Equation (1). The goodness of fit (FIT, Equation (2)) and Pearson linear correlation coefficient ( $\rho$ , Equation (3)) were calculated to assess the accuracy of the HIS-MTF in acquiring the induced force in human gait and inertial force.

$$FIT = \left( 1 - \frac{\|F_M - \hat{F}_C\|}{\|F_M - \text{mean}(F_M)\|} \right) \quad (2)$$

$$\rho = \frac{1}{N-1} \sum_{i=1}^N \left( \frac{F_{M,i} - \mu_{F_M}}{\sigma_{F_M}} \right) \left( \frac{\hat{F}_{C,i} - \mu_{\hat{F}_C}}{\sigma_{\hat{F}_C}} \right) \quad (3)$$

where  $F_M$  and  $\hat{F}_C$  are the measured loads from the load cells and the calculated using the acceleration and mass of the HSI-MTF, respectively.  $N$  is the data number, and  $\mu_{F_M/C}$  and  $\sigma_{F_M/C}$  are the mean and standard deviation of  $F_M$  or  $\hat{F}_C$ , respectively. For the proposed sine-wave signals, FITs of 0.852 and 0.844, and  $\rho$  of 0.989 and 0.988, respectively, were obtained for each signal. These evaluations demonstrate the accuracy and sensitivity of the proposed system.

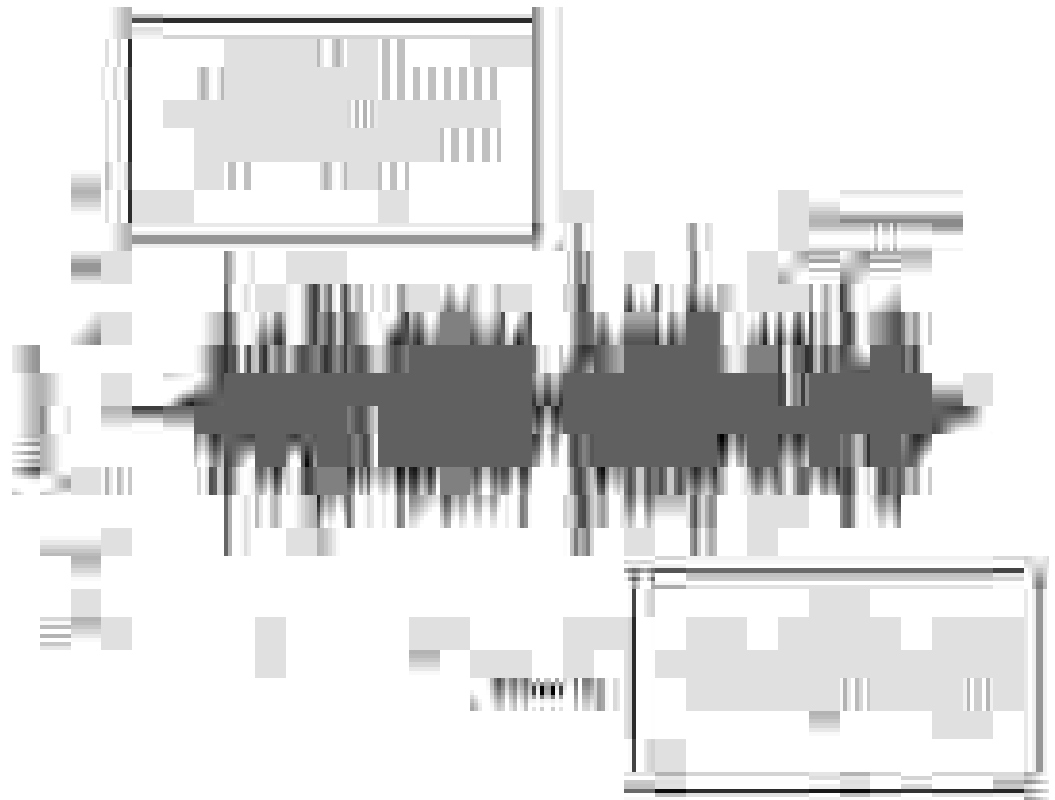


**Figure 4.** Inertial force calculated and measured by HSI-MFT.

### 3. Results and Discussion

#### 3.1. Support and Swing Phase Periods of the Human Gait

This section focuses on the existing variability of the step frequency ( $f_s$ ) in the human gait.  $f_s$  is one of the primary parameters that provides information regarding the randomness of the loads induced during human gait. According to the research conducted by Nakamura et al. [59], Pizzimenti [66], and Ingólfsson et al. [61], the  $f_s$  parameter is susceptible to variations due to changes occurring in human gait, particularly when STs are exposed to dynamic effects on surfaces with harmonic motion close to their fundamental gait frequency. Therefore, in this study, motion data of the left foot were acquired as described in Sections 2.4 and 2.5, to obtain the periods of the support ( $T_{su}$ ) and swing ( $T_{sw}$ ) phases of the STs, both on a rigid surface and in the flexible surface protocols described in Section 2.4. The frame sequence shown in Figure 3 (bottom) corresponds to a test on the rigid surface of ST 1.

$$\text{PDF}_{T_i} = f(T_i | \mu_{T_i}, \sigma_{T_i}) = \frac{1}{\sigma \sqrt{2\pi}} e^{-\frac{(T_i - \mu_{T_i})^2}{2\sigma_{T_i}^2}}, \text{ for } \forall T_{si} \in \mathbb{R} \quad (4)$$

The frame sequences had a duration of 30.0 s. This time window allowed the acquisition of approximately 30 complete gait cycles, enabling a stochastic analysis of the  $T_{su}$  and  $T_{sw}$  gait periods of the STs. Using violin diagrams, in Figure 5, the normal probability density function (PDF) obtained for these periods on a rigid surface, according to Equation (4), using the average ( $\mu_{T_i}$ ) and standard deviation ( $\sigma_{T_i}$ ) of these swings ( $i = 1, T_1 = T_{sw}$ ) and support ( $i = 2, T_2 = T_{su}$ ) periods acquired at all STs ( $1 \rightarrow 7$ ) are presented (shown as a white circle), along with the first quantile of the standard deviation, in comparison to the obtained data.



**Figure 5.** Statistical analysis of human gait support and swing periods; violin plot of the support phase duration and its probability density function for each subject test (**top**); and violin plot of the swing phase duration and its probability density function for each subject test (**bottom**); [51]—Grieve et al.; [52]—Andriacchi et al.; [53]—Stenum et al.; [54]—Stacy et al.; [55]—Cao et al.

The  $T_{su}$  and  $T_{sw}$  periods evaluated during human gait in the STs on the rigid surface were consistent with those presented in previous studies. Grieve et al. [69] conducted an analysis and parameterization of human gait on fifty test subjects, including both women and men. They determined that the swing time during gait was less than half of the apparent individual period, as is the case for over 70.0% of the STs studied. The parameterizations of swing time conducted on individuals with a gait speed of 1.10 m/s reported average periods of 0.43 s, deviating by 10.0% from the mean values reported in this study. In the late 1970s, Andriacchi et al. [70] conducted an experimental campaign with 70 test subjects ranging from 22 to 59 years of age, with a height of  $173 \pm 70$  cm and weights of  $757 \pm 307$  N. They determined support periods of 664 ms and swing periods of 521 ms for a gait speed of 1.10 m/s. According to the mean values of this research, there were overestimated swing periods by 8.0% and underestimated support periods by 22.0%. Similarly, Stenum et al. [71] reported an analysis of human gait in 32 individuals (22 males and 10 females) using a Motion Capture (MOCAP) system with pose estimation, indirectly determining support and swing periods of  $0.75 \pm 0.10$  s and  $0.46 \pm 0.025$  s,

respectively. These values have differences of less than 12.0% compared with the  $T_{su}$  and  $T_{sw}$  values evaluated here using the MOCAP system with smartphone devices. Because of the importance of the support time during gait, as this is the period when loads are transmitted to the surfaces, Stacy et al. [72] used a wireless sensor system integrated into footwear to establish a correlation between gait periods ( $1/f_s$ ) and average support periods, determining an average support value of  $0.70 \pm 0.026$  s, which differs by under 18.0% from those reported in this study. Cao et al. [73] recently conducted a gait analysis on lightweight and large-span structures to evaluate the influence of gait loads on the development of HSI effect phenomena. For this purpose, tests were conducted on a force plate with 25 test subjects, and although the research focused on determining a parameterization of the induced loads relative to the  $f_s$  of pedestrian, an average support period of  $0.68 \pm 0.10$  s was determined, with a difference of 9.0% from the  $T_{su}$  evaluated in this investigation.

Despite the low percentage differences compared with other investigations, as shown in Figure 5 (top: support time; bottom: swing time), the results of human gait on a rigid surface suggest significant intra- and inter-personal spatiotemporal variability (ST 3, 4, and 6; shown in Figure 5) between the participants, emphasizing the need for additional experimental assessment. This condition of relative variability within everyone becomes even more critical when comparing walking on a rigid surface with walking on surfaces with harmonic lateral movements because of noticeable changes in the support and swing phases generated by the coupling of the walking of the STs with the harmonic dynamics of the excitations induced by the HSI-MTF. Figures 6 and 7 present the variations in the periods  $T_{su}$  and  $T_{sw}$ , respectively, for displacements (5.0 to 50.0 mm) and lateral frequencies (0.7 to 1.3 Hz) induced in the gait of all evaluated STs, according to the proposal in Section 2.4. In these figures, a forest plot diagram is used, where white dots represent the mean values obtained for the indicated displacements and frequencies. In addition, the size of the blue boxes varied inversely proportional to the standard deviation of the respective test, and the horizontal red lines indicate the first calculated standard deviation. Although in the case of ST 1 the variations for the period  $T_{su}$  are minimal compared to the average of all STs on a rigid surface, significant changes occur in the period  $T_{sw}$  for displacements of 20.0 and 30.0 mm, indicating a brief implication of the coupling effects that alter gait dynamics. This is further supported by the remaining STs, which show decreases of up to 96.53% and 58.15% for the periods  $T_{su}$  and  $T_{sw}$ , respectively, for some induced displacements and excitation frequencies compared with the overall average obtained on a rigid surface for each ST. These results align with those of studies on laterally instrumented treadmills reported by Nakamura et al. [59], Sun and Yuan [30], Pizzimenti et al. [66], Ingólfsson et al. [61], Ricciardelli et al. [74], Bocian et al. [63], and Haowen et al. [67]. Furthermore, they are consistent with investigations on some pedestrian bridges in a similar analytical context, such as those conducted by Butz et al. [75] and Rönnquist [76]. In both cases, an alteration in the  $f_s$  of the STs was reported as the frequency and magnitude of the induced lateral displacement increase or an increase in anthropic loads. These reports indicate an alteration of the natural gait kinematics and, consequently, of  $f_s$ , consistent with the behavior presented in this assessment of the human gait in this study.



**Figure 6.** Support period ( $T_{su}$ ) of the subject tests on multiplex harmonic surfaces.



**Figure 7.** Swing period ( $T_{sw}$ ) of the subject tests on multiplex harmonic surfaces.

### 3.2. Assessment of Human Gait-Induced Loads

Human gait is described as a specific form of locomotion in which the weight of the human body is alternately transferred and supported by both lower limbs [77]. These displacement conditions generate dynamic loads known technically as Ground Reaction Forces (GRF), which, when interacting with civil structures with dynamic sensitivity conditions, such as pedestrian bridges, can induce dynamic coupling phenomena (Lock-In Effects) that increase the probability of structural vibrations, conditioning their serviceability. The study of these coupling phenomena has significantly increased in recent years, focusing especially on the nature and behavior of lateral and vertical loads induced by human gait in direct interaction with structures. In this study, the assessment of loads induced by human gait on vibrating flexible surfaces was divided into two main components: lateral and vertical loads.

#### 3.2.1. Lateral Load

In accordance with Section 2.5, a 30.0 s evaluation of lateral loading induced by walking on a rigid surface was conducted for the seven STs. The induced lateral load ( $L_L$ ) was normalized with respect to the weight ( $W_0$ ) of each ST, yielding an average normalized lateral load of 2.0% and a maximum of 3.7%. These values are depicted in Figure 8 (top) for all evaluated STs within a 5.0 s range. Furthermore, these  $L_L$  loads were processed using a cross-power spectral density (CPSD, Equation (5)) as proposed in [78–80].

$$\text{CPSD}_{xy}(\omega) = \sum_{m=-\infty}^{\infty} R_{xy}(m) \cdot e^{-j\omega m}; R_{xy} = E\{x_n y_n^*\} \forall \{-\infty < n < \infty\} \quad (5)$$

In the implementation of CPSD, both  $x_n$  and  $y_n^*$  correspond to the  $L_L$  load of the  $i$ -th ST. Additionally,  $E\{\cdot\}$  denotes the expected value operator. Based on the aforementioned, the average fundamental lateral gait period was identified as  $f_s = 0.73 \pm 0.10$  Hz, the second gait harmonic as  $f_2 = 1.45 \pm 0.15$  Hz, and the third gait harmonic as  $f_3 = 2.20 \pm 0.15$  Hz for all the STs, as illustrated in Figure 8 (middle). Conversely, in Figure 8 (bottom), the normalized  $L_L$  load induced by ST 1 on the lateral harmonic surface is considered, featuring a frequency of 1.0 Hz and an amplitude of 20.0 mm. The objective was to showcase the sinusoidal behavior of  $L_L$ , its phase in relation to the harmonic lateral movement, and the anisotropic behavior of its magnitude compared to that recorded on a rigid surface [46]. This aligns with the characteristics of S2HI effects elucidated by Butz et al. [75], Rönquist et al. [76], and Heinemeyer et al. [81]. These findings provide a theoretical foundation for implementing an  $L_L$  lateral load model based on a Fourier sine series with dynamic load factors (DLF), pedestrian weight ( $W_0$ ), and gait frequency ( $f_m$ ), as proposed in Equation (6).

This load model was employed in one of the initial pedestrian bridge design guidelines, the French SETRA [82], with a recommended constant DLF ( $n = 1$ ) of 0.05 for a pedestrian with weight  $W_0 = 700$  N and gait frequency  $f_m = 2.0$  Hz.

$$L_L(t) = \text{DLF} \cdot W_0 \cdot \sum_{i=1}^n \sin(2\pi \cdot i \cdot \left(\frac{f_m}{2}\right) \cdot t) \quad (6)$$

Literature reviews conducted by Zivanovic et al. [17] and Racic et al. [45] presented two pioneering investigations concerning the lateral modeling of human gait based on DLFs. Initially, reference is made to the work by Schulze (later expanded upon by Bachmann and Ammann [67]), which reported the values of  $\alpha_1 = 0.039$ ,  $\alpha_2 = 0.010$ ,  $\alpha_3 = 0.043$ ,  $\alpha_4 = 0.012$ , and  $\alpha_5 = 0.015$  as lateral DLFs for human gait interacting with civil structures at a  $f_m = 2.0$  Hz. Similarly, Bachmann et al. [83,84] reported values of  $\alpha_1 = 0.012$  and  $\alpha_3 = 0.015$  for  $f_m = 2.0$  Hz. However, these values are conditioned by the acquisition on rigid surfaces or the effects of S2HI relative to the studied civil structure, as in [85]. Nakamura et al. [59] developed a device consisting of a shake table and a force plate and reported the following maximum DLFs for five test subjects: {0.08, 0.11, 0.12, 0.13}, {0.09, 0.11, 0.13, 0.17}, {0.10, 0.13, 0.16, 0.18}, and {0.11, 0.15, 0.18, 0.21}. These values were obtained in an experimental campaign

combined with displacements of {10.0, 30.0, 50.0, and 70.0} mm and lateral oscillation frequencies of {0.75, 0.87, 1.0, and 1.25} Hz, respectively. However, these reported DLF values are associated with static gait, limiting their applicability in representing continuous gait in civil structures.



**Figure 8.** Lateral load induced by human gait on rigid and harmonic surfaces. Lateral loads ( $L_L$ ) normalized with respect to weight  $W_0$  induced by STs on rigid surfaces (**top**), frequency content of lateral loads induced by STs on rigid surfaces (**middle**), and lateral loads induced by ST 1 on harmonic surfaces with a frequency of 1.0 Hz and a displacement of 30.0 mm (**bottom**).

The utilization of instrumented treadmills marked a significant advancement for the continuous measurement of forces induced by human gait over time, unencumbered by restrictions on stride length owing to test area dimensions. Sun and Yuan [30] integrated a treadmill, a force plate, and a shake table, and seven STs were assessed using this framework at two walking speeds ( $0.83$  and  $1.00 \text{ m}\cdot\text{s}^{-1}$ ), subjected to combinations of lateral vibrations with frequencies of  $0.65$ – $1.2$  Hz and amplitudes of  $4.0$ – $50.0$  mm. The expression  $\text{DLF}(U_0) = 1.18 \times U_0 + 0.05 \forall U_0 < 0.05 \text{ m}$  was introduced in this study; however, this formulation solely depends on the lateral displacement  $U_0$ , disregarding the significant

effects of frequency changes, even for the same displacement value. This phenomenon, as demonstrated in this study and depicted in Figure 9, was observed.

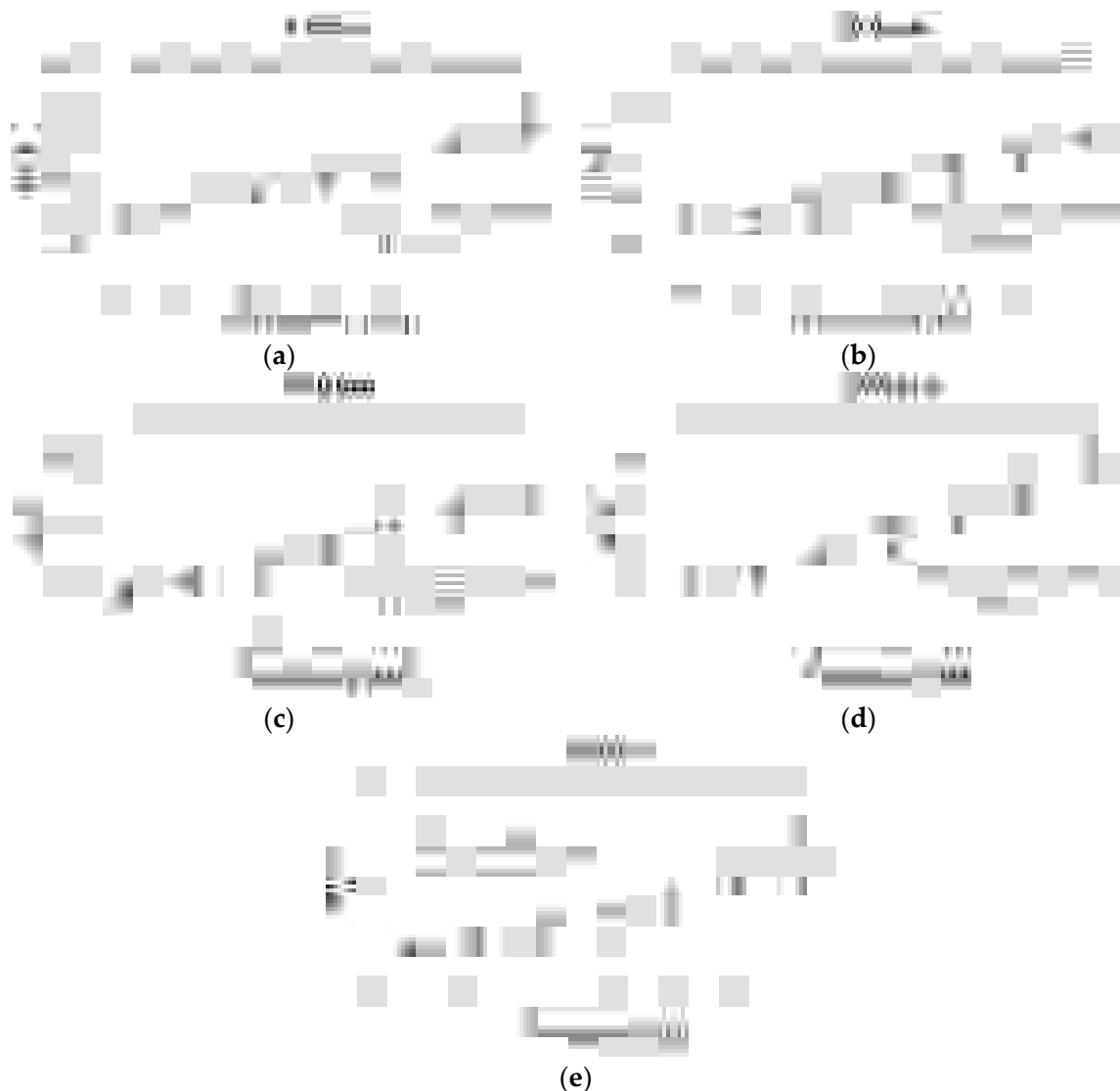


**Figure 9.** Experimental evaluation of pedestrian-induced lateral load on harmonic lateral surfaces with displacements of (a) 5.0 mm, (b) 10.0 mm, (c) 20.0 mm, (d) 30.0 mm, and (e) 50.0 mm.

A similar study was conducted by Pizzimenti [66], who instrumented a treadmill capable of executing predetermined combinations of amplitudes and frequencies. Five STs were evaluated in this research under three amplitudes (15.0, 30.0, and 45.0 mm) and five lateral vibration frequencies (0.60–0.92 Hz). Based on [60], the existence of two DLFs, intrinsic to gait behavior and self-excited lateral movement dynamics, was concluded. However, these results were not assessed around the critical lateral frequency ( $\approx 1.0$  Hz) outlined in the literature for pedestrian bridges subjected to HSI, thereby limiting their application under real service conditions. Later, Ingólfsson et al. [61] performed the most extensive experimental pedestrian campaign to induce lateral forces by employing the experimental setup of Pizzimenti. In this study, measurements were taken from 71 individuals for various combinations of vibration frequencies (0.33–1.07 Hz) and lateral amplitudes (4.5–48.0 mm). However, their findings were presented in terms of pedestrian load coefficients proportional to the velocity and acceleration, denoted as  $C_p$  and  $Q_p$ , respectively. Similarly, Carroll et al. [46] and Bocian et al. [63] employed instrumented treadmills to scrutinize the behavior of loads laterally induced by human gait. The former focused on indirectly estimating lateral loads through visual marker data, whereas the latter examined differ-

ences in lateral loads when developed with and without a virtual reality environment. In recent years, Haowen et al. [67] conducted a multi-axial load analysis on five test subjects by employing a treadmill, shake table, and pressure insoles to evaluate lateral excitations with variable frequencies (1.2–1.7 Hz) and amplitudes (10.0 and 20.0 mm). Although their analysis was centered on stride rates exceeding 2.0 Hz (running conditions), rendering it incomparable to human walking conditions, this research establishes a precedent for multi-axial anthropic load assessment.

Considering the aforementioned aspects, the present study emerges as a contribution for experimental campaigns of simultaneous lateral and vertical gait loads acquisition (utilizing the HSI-MTF) and assessment of lateral and vertical human-induced DLFs. The focal point of this investigation is the variation of these factors due to their interdependence with the lateral displacement amplitudes and frequencies. As evidenced by the outcomes depicted in Figure 9, these results show a linear growth in DLF values proportional to the escalation in the lateral excitation frequency. Additionally, an augmentation of this proportionality constant was identified in relation to the increase in the amplitude of the lateral vibrations. Moreover, parametric expressions capturing the interdependence between the lateral excitation frequencies and DLFs were established for each lateral amplitude value, yielding an average  $R^2$  value of 0.815 for linear regressions of the form  $DLF = M * f_n + b$ . In addition, lateral excitation amplitudes equal to or exceeding 50.0 mm induce alterations in pedestrian gait kinematics as individuals strive to maintain the center of mass stability during vibrations. This adjustment substantially heightens the laterally induced force from pedestrians (an 82.6% increase compared to lateral excitations with 40.0 mm amplitudes).

### 3.2.2. Vertical Load

The total vertical load ( $L_V$ ) induced by the gait of the STs on a rigid surface under the influence of lateral vibrations was acquired and assessed in this study. The acquisition was conducted using the HSI-MTF, which constitutes a novel comprehensive development for the multiaxial acquisition of anthropic loads that has not been previously documented in the literature. As outlined in Section 2.5, a 30.0 s evaluation of the  $L_V$  induced by gait on a rigid surface was performed for the seven STs. The  $L_V$  load was normalized with respect to the weight  $W_0$  of each ST, resulting in an average normalized vertical load of 110.0% and a maximum of 123.0%. These values are presented in Figure 10 (top) for all STs within a 3.0 s range. Furthermore, these  $L_V$  loads were processed using cross-power spectral density in accordance with Equation (5), as elaborated in Section 3.2.1. Then, the average fundamental vertical gait period was identified as  $f_{sv} = 1.45 \pm 0.25$  Hz, the average second vertical harmonic as  $f_{v2} = 2.90 \pm 0.30$  Hz, and the average third vertical harmonic as  $f_{v3} = 4.35 \pm 0.45$  Hz for all STs, as illustrated in Figure 10 (middle). Additionally, a noticeably lower fundamental lateral gait frequency was discernible when compared to those presented in the vertical direction. This observation arises from the analysis of the total vertical force. Moreover, in Figure 10 (bottom), the normalized  $L_V$  load induced by ST 1 on the lateral harmonic surface is considered, featuring a frequency of 1.0 Hz and an amplitude of 20.0 mm. The objective was to demonstrate the typical behavior of a sinusoidal  $L_V$  series and its anisotropic behavior, both in phase, with respect to lateral vibrations, and in magnitude.

During lateral vibrations, encompassing combinations of amplitudes and frequencies, the maximum normalized  $L_V$  loads induced for the STs were identified over 30.0 s intervals, as detailed in Section 2.4. These assessments revealed that for vibration amplitudes below 20.0 mm across the entire evaluated frequency range, the normalized  $L_V$  loads remained relatively constant (Figure 11), akin to those recorded on a rigid surface. However, for lateral amplitudes equal to or exceeding 30.0 mm, a linear increasing trend in normalized  $L_V$  proportional to the rise in lateral excitation frequency was observed. Additionally, the augmentation of this proportionality constant with respect to the increase in the lateral vibration amplitude was determined beyond this critical amplitude value. Furthermore,

parametric expressions capturing the interdependence between lateral excitation frequencies and the maximum normalized  $L_V$  values for lateral amplitudes of 30.0 mm and 50.0 mm are established; these expressions demonstrate maximum increases of 26.5% and 28.3%, respectively, compared to a rigid surface.

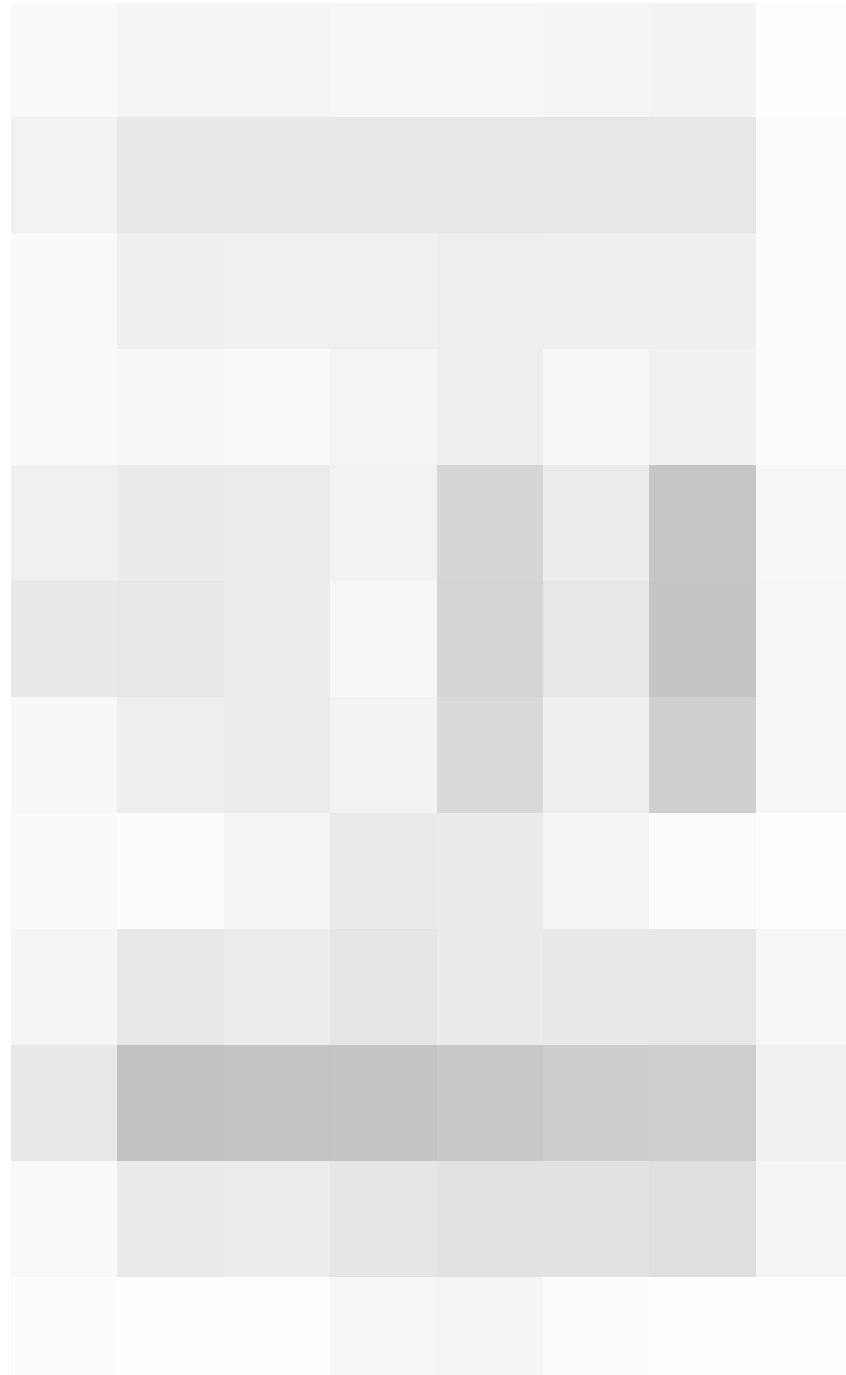


**Figure 10.** Vertical load induced by human gait on rigid and harmonic surfaces. Vertical loads normalized with respect to weight induced by STs on rigid surfaces (**top**), frequency content of vertical loads induced by STs on rigid surfaces (**middle**), and vertical loads induced by ST 1 on harmonic surfaces with a frequency of 1.0 Hz and a displacement of 30.0 mm (**bottom**).

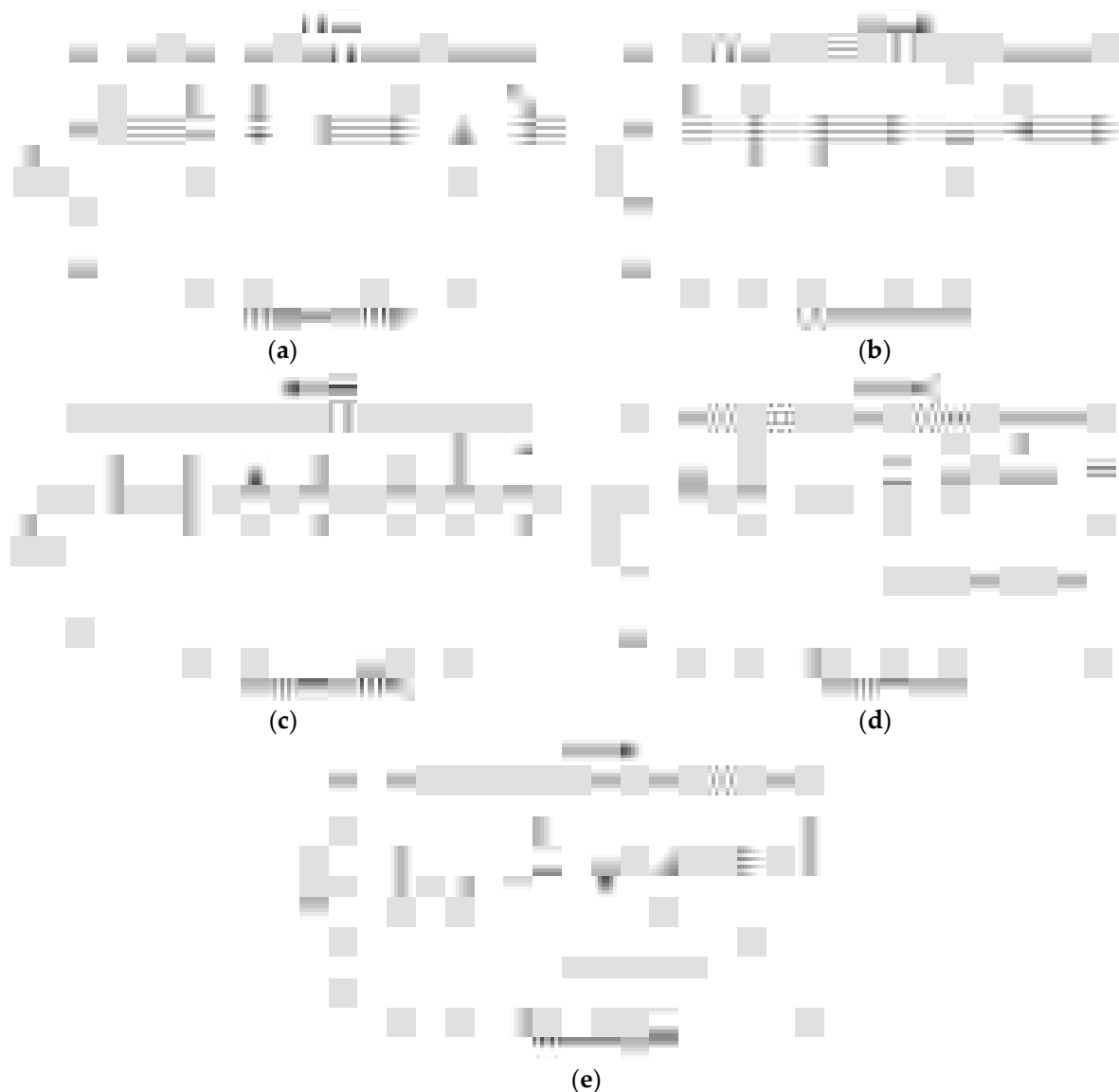


**Figure 11.** Experimental evaluation of pedestrian-induced vertical load on harmonic lateral surfaces with displacements of (a) 5.0 mm, (b) 10.0 mm, (c) 20.0 mm, (d) 30.0 mm, and (e) 50.0 mm.

### 3.3. Correlation between CM and Treadmill Movements

The impact of flexible surface dynamics on the behavior and development of anthropic activities has been a topic of widespread interest in structural engineering and one of the most promising research avenues in the coming years. The work undertaken by Fujino et al. [10] set a precedent in analyzing the dynamic coupling phenomena between pedestrian bridges and pedestrians traversing them. Through image processing, they determined that approximately 20.0% of pedestrians exhibited interaction effects with the lateral vibrations of the Toda Bridge in Japan, which exhibited a frequency behavior close to that of human gait in that direction. In subsequent years, image processing from motion capture systems employed in the study of interaction phenomena has focused on indirectly determining loads induced by anthropic activities [10,46,86,87]. Similarly, motion capture systems have been utilized in various studies with instrumented treadmills [63,64,66]. However, the objectives of these investigations were primarily focused on analyzing the kinematic behavior of the ST.

In this context, the present research determined existing correlation levels between the induced lateral vibrations (HSI-MTF reference control point) and the estimated center of mass (CM) displacements for each ST. The tracking points are shown in Figure 3

(top, right). The marker-based motion capture system used and implemented is rated in Sections 2.4 and 2.5. The correlation values were calculated for the seven STs based on Equation (3), considering different combinations of HSI-MTF lateral vibration frequencies and amplitudes, as illustrated in Figure 12. Within these correlation surfaces, a distinct and specific behavior is established for each ST, which is an important detail to note given that even under the same lateral vibration conditions, STs experience varying S2HI effects. Maximum correlation levels of  $\rho = 12.0\%$  were obtained for ST 1 during lateral vibrations with an amplitude of 10.0 mm and a frequency of 0.8 Hz. By contrast, ST 5 exhibited a maximum correlation of  $\rho = 60.0\%$  for vibrations with amplitudes of 30.0 mm and frequencies exceeding 1.0 Hz. These experimental findings establish a foundation for understanding the kinematics of each pedestrian under the effects of S2HI.

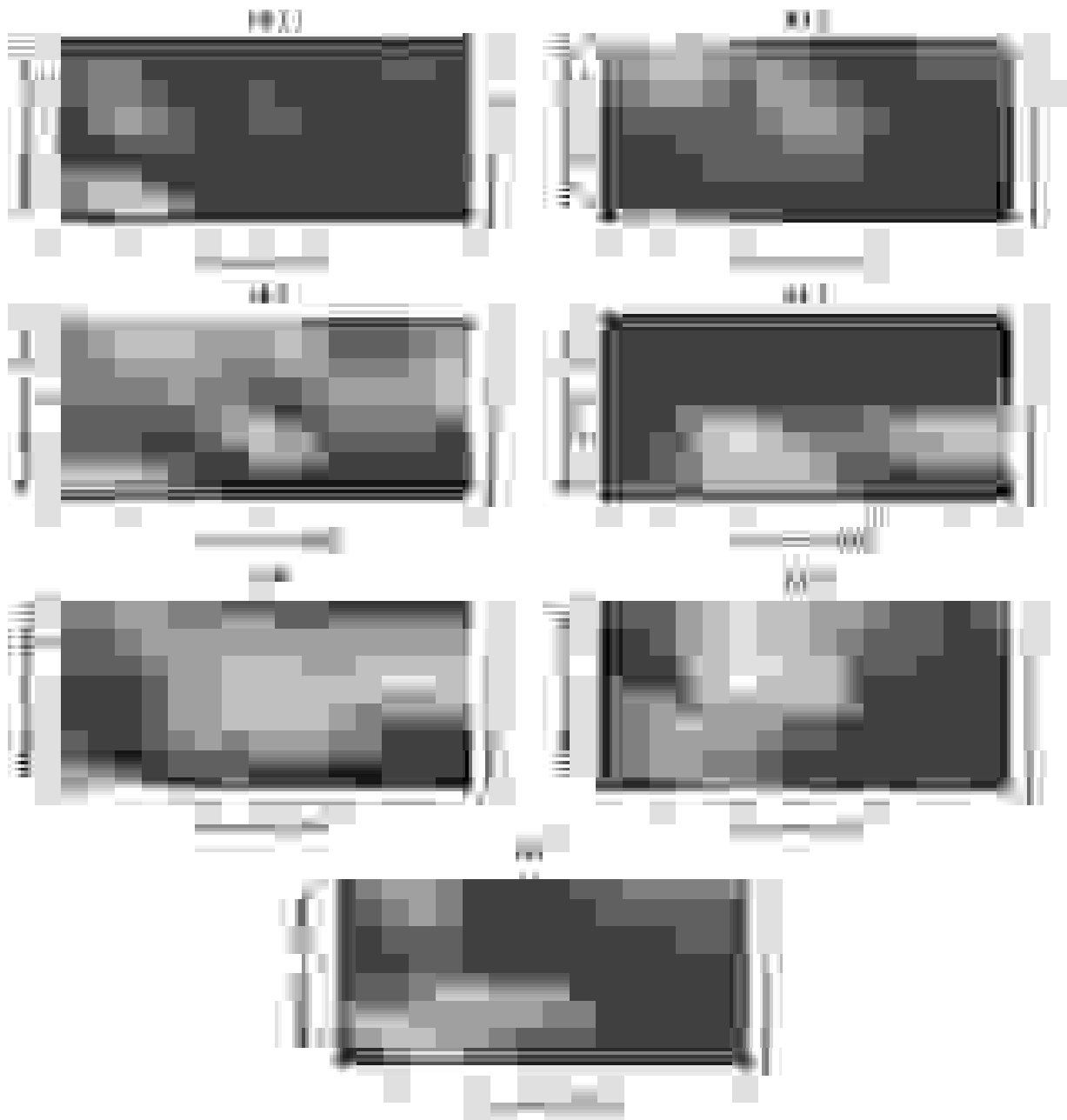


**Figure 12.** Correlation between the CM of the STs and the reference control point in the HSI-MTF.

#### 4. Conclusions

A Human–Structure Interaction Multiaxial Test Framework (HSI-MTF) was developed and used to investigate the effects of Structure-to-Human Interaction (S2HI). The framework consisted of an instrumented treadmill placed on a shake table. The HSI-MTF allowed for the measurement of the pedestrian-induced gait loads in vertical and lateral directions simultaneously while the walking surface was subject to lateral sinusoidal motion. During the experimental program, the subjects walked at a constant pace and were exposed to a combination of lateral vibrations with varying amplitudes and frequencies. The primary conclusions are summarized as follows:

The support ( $T_{su}$ ) and swing ( $T_{sw}$ ) times of human gait exhibited differences of up to 96.53% and 58.15%, respectively, when comparing them between pedestrians on lateral harmonic moving surfaces to those on rigid surfaces. These outcomes suggest significant alterations in gait kinematics owing to lateral vibration effects, resulting in decreased support times on the dynamic surface to maintain walking stability. Furthermore, correlation coefficients between the center of mass of the STs and a reference control point in the HSI-MTF were computed, revealing that S2HI effects are intrinsic to each pedestrian, even under identical lateral vibration conditions.

The maximum lateral loads ( $L_L$ ) normalized to the weight  $W_0$  of the STs exhibited a linear growth proportional to the increase in the lateral vibration frequency. Additionally, an increase in this proportionality constant was identified in relation to the elevation of vibration amplitudes. Furthermore, parametric expressions capturing the interdependence between the lateral excitation frequencies and DLFs are established for each lateral amplitude value. These expressions yielded an average  $R^2$  value of 0.815 for linear regressions of the form  $DLF = Mf_n + b$ . In addition, the maximum vertical loads ( $L_V$ ) normalized to the weight  $W_0$  of the STs remained constant in relation to those recorded on a rigid surface for lateral vibration amplitudes equal to or less than 30.0 mm. However, beyond this value, a linear growth is observed, which is proportional to the increase in the lateral vibration frequency. Furthermore, an increase in the proportionality constant was identified with respect to the elevation of lateral vibration amplitudes.

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**Institutional Review Board Statement:** This study was approved by the Ethics Committee of the Universidad del Valle (Acta de Aval No. 011-2023) and all participants signed an informed consent form before the measurements.

**Informed Consent Statement:** Informed consent was obtained from all subjects involved in the study.

**Data Availability Statement:** The raw and time-normalized marker-based with Opti-Track and smart-phone kinematic data, as well as the test video, are available from Mendeley Data at <https://data.mendeley.com/datasets/6r53j3576r/1> (accessed on 18 January 2024).

**Conflicts of Interest:** The authors declare no conflicts of interest.

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### ***3.2 Evaluation of Human Gait on Rigid and Harmonic Surfaces: Direct and Indirect Measurements Using the HSI-MTF and Low-cost Motion Capture System with smartphones.***

The accurate evaluation of human gait on both rigid and moving surfaces is critical for understanding dynamic interactions in civil structures such as footbridges. The Human-Structure Interaction Multiaxial Test Framework (HSI-MTF) has emerged as a robust system for directly capturing multiaxial loads induced by human gait, offering precise data on vertical and lateral forces. Simultaneously, low-cost motion capture (MOCAP) systems using smartphones present a cost-effective alternative for indirect gait analysis, providing spatial and kinematic data through video tracking of body movements.

Direct measurements from the HSI-MTF allow for the detailed assessment of gait-induced loads on moving surfaces with varying amplitudes and frequencies, simulating real-world conditions of pedestrian-structure interaction. This system captures the biomechanical response of pedestrians through advanced instrumentation, enabling the study of gait variability under dynamic lateral motions. In parallel, the low-cost MOCAP system captures spatiotemporal characteristics of human gait using smartphone cameras and retroreflective markers, offering reliable joint angle tracking and kinematic insights with minimal resources.

The combination of these direct and indirect approaches allows for a comprehensive evaluation of human gait, where the strengths of each system complement the other. While the HSI-MTF provides high-fidelity measurements of force dynamics, the smartphone-based MOCAP system adds flexibility and accessibility to gait analysis, making it feasible for broader applications in structural monitoring and gait research. Together, these technologies advance the study of human gait under various structural conditions, contributing to safer and more resilient infrastructure design.

Each article contributes uniquely to the evaluation of human gait on rigid and moving surfaces through both direct and indirect measurements. The article *"Experimental Evaluation of Pedestrian-Induced Multiaxial Gait Loads on Footbridges: Effects of the Structure-to-Human Interaction by Lateral Vibrating Platforms"* focuses on direct measurements using the HSI-MTF framework, which captures multiaxial loads induced by human gait on vibrating surfaces, providing detailed insights into the dynamic interaction between pedestrians and flexible structures. It emphasizes how direct force measurements on both rigid and laterally oscillating surfaces contribute to understanding human-induced structural vibrations. In contrast, the article *"Assessing Spatiotemporal Behavior of Human Gait: A Comparative Study Between Low-Cost Smartphone-Based MOCAP and OptiTrack Systems"* explores the potential of indirect gait evaluation through low-cost smartphone-based motion capture systems. It highlights the accuracy of kinematic data collected through video tracking and spatiotemporal analysis, offering a cost-effective alternative for gait assessment. Together, these articles showcase the complementary use of direct (HSI-MTF) and indirect (MOCAP) methods, combining high-resolution load measurements with flexible, accessible motion capture techniques to advance the study of human gait dynamics on both rigid and mobile surfaces.



# Assessing Spatiotemporal Behavior of Human Gait: A Comparative Study Between Low-Cost Smartphone-Based Mocap and OptiTrack Systems

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## Abstract

This study assessed the accuracy of a low-cost marker-based motion capture system with smartphone devices to estimate the spatiotemporal behavior of human gait in comparison with the performance of the commercial OptiTrack system. Initially, three test subjects were selected for the study, and after equipping them with passive retroreflective markers, they were recorded for gait velocities of 1.50, 1.90, and 2.30  $m \cdot s^{-1}$  while collecting kinematic data and videos. The results showed that the smartphone motion capture system exhibited significant spatiotemporal tracking and accuracy in the x-y trajectories and estimation of joint relative angles of the hip, knee, and ankle joints ( $\theta_1$ ,  $\theta_2$ , and  $\theta_3$ , respectively) compared to the commercial OptiTrack system. In this comparison, an average goodness-of-FIT and normalized root mean square error of over 88.93% and 2.71% were obtained, respectively, for the joint relative angles of the hip and knee ( $\theta_1$  and  $\theta_2$ ) in all tests performed. However, the accuracy of the joint relative angle of the ankle ( $\theta_3$ , average FIT: 71.04% and nRMSE: 4.26%) was lower because of the low capture rate of the retroreflective markers in the smartphone system and the higher relative velocity in the lower extremities of the test subjects, which generated noise in the calculation of x-y trajectories. This decrease in accuracy has been reported in other studies. However, both motion capture systems experienced marker data loss at the hip, highlighting the need for improvement in the spatial distribution of the optical devices. The OptiTrack system demonstrated better optical redundancy but still required improvements. In contrast, the smartphone system, with its inherent limitations in terms of optical redundancy and spatial distribution, can be enhanced by incorporating multiple cameras for a three-dimensional view. Despite these limitations, the low-cost smartphone system showed optimal performance with minimal errors compared with the commercial system, making it a cost-effective option with potential for further development. The rapid advancement of smartphone technology and its accessibility make it an attractive choice for motion capture applications.

**Keywords** Motion capture · Smartphones · Gait Analysis · OptiTrack

## Introduction

In recent years, the use of optical motion capture systems (MOCAPs) has focused on digital animation of human activity of characters for the film and video game industry. Therefore, this technology has made rapid inroads in the field of

human motion analysis and measurement because it provides accurate and reliable spatiotemporal measurements, versatile testing schemes, high spatial measurement resolution, and relative ease of implementation [1–4]. MOCAPs employ a single camera (measurement 2-D) or more cameras (measurement 3-D) and sophisticated computer algorithms, which allow for a detailed assessment of anthropic activities [5–9]. In the case of human gait, compared to traditional methods such as wearable sensors or force plates, optical systems offer non-invasive tracking, allowing for a more natural and unconstrained physical motion analysis [10–12]. This non-intrusive approach is particularly beneficial in the analysis of the dynamic interaction of human gait with civil structures [13–18], where it is important to capture and analyze motion

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patterns in a manner that does not disrupt the natural gait of the test subject (TSs) while transiting structures.

The accuracy of MOCAP systems is outstanding because of advanced algorithms and calibration techniques, which achieve millimeter accuracy in tracking the position and orientation of body segments, enabling a detailed understanding of the kinematics and kinetics of human gait. This level of accuracy is important for acquiring subtle movements and biomechanical parameters during gait analysis. Accurate measurements are essential for detecting abnormalities, assessing rehabilitation progress, and designing customized interventions. By providing highly reliable data, MOCAP systems offer researchers and clinicians valuable tools for studying and evaluating human gait dynamics. Numerous MOCAP technologies have been developed and are widely used today [19–22]. Marker-based systems, often considered the gold-standard method to quantify human activities, involve the attachment of retroreflective markers to specific body landmarks, allowing the precise tracking of their positions. These markers reflect the light emitted by the environment, thereby enabling accurate position and orientation calculations [23–25]. On the other hand, Markerless systems use computer algorithms to identify and track anatomical features without the need for markers. These markerless approaches leverage pattern recognition and machine-learning techniques to analyze captured video data and estimate joint positions and movements [9, 19, 21, 26]. Examples of popular marker-based systems used for the assessment of human gait include *Vicon* [15, 27–29], *CODA* [15, 18, 30, 31], and *OptiTrack* [32–39], whereas markerless systems such as *Kinect* [24, 40–43] and *OpenPose* [19, 44–49] have gained prominence in recent years.

The market prices of MOCAP systems are often very expensive [27]. These systems are frequently tailored for specific applications and offer advanced features and capabilities. However, the high costs associated with these systems make them inaccessible in many research and clinical settings, thereby limiting their widespread adoption and implementation. To address this problem, there is an increasing need to develop low-cost optical technologies that can provide accurate motion capture capabilities, while remaining affordable. By developing cost-effective and reliable solutions, researchers and clinicians can access motion analysis tools and promote their integration into various human gait analysis fields [11, 12, 19, 41, 42]. Smartphones are potential devices for integrating the MOCAP systems. Smartphones are equipped with high-resolution cameras, substantial computational power, and portability, making them attractive platforms for implementing motion capture technologies [50–53]. In addition, the general accessibility of smartphones to society allows researchers to develop low-cost solutions that utilize built-in cameras and combine them with computer vision algorithms to capture and analyze the

human gait. The versatility of smartphones allows for real-time data processing [54–56], immediate feedback [51, 57, 58], and seamless integration with other applications for data visualization and analysis [52, 53, 57, 59].

Although continuous advancements in smartphone technology have enhanced their potential as MOCAP devices, the evaluation of the accuracy, performance, and cost-effectiveness of motion capture using this type of low-cost device in comparison with gold-standard methods, such as marker-based systems, for assessing human gait is still in its early stages. This study compared the *x-y* trajectory in sagittal plane gait measured using two marker-based MOCAP systems: a custom low-cost MOCAP system using a smartphone device and a commercial MOCAP system known as OptiTrack. In order to evaluate human gait using a low-cost MOCAP system, cameras of Huawei Ascend G7 smartphones as devices within a Marker-based system were integrated to evaluate three TSs ( $65.00 \pm 8.00$  kg, and  $1.65 \pm 0.10$  m) in the sagittal plane, subjected to three walking speeds ( $1.50$ ,  $1.90$ , and  $2.30$  m  $\cdot$  s $^{-1}$ ) on a treadmill. The obtained results were compared with those of tests using the OptiTrack system. The remainder of this paper is organized as follows. Section "Experimental Methods" provides details of the experimental methods. In Section "Results", the results obtained for both the MOCAP systems are presented and evaluated. Finally, section "Discussion" presents a discussion of the results.

## Experimental Methods

### Investigation General Design

This investigation assessed the spatiotemporal behavior of human gait through two MOCAP systems: a Marker-based OptiTrack system and Low-cost Smartphone systems. Three TSs were selected for these gait tests and their general anthropometric information was recorded. Initially, TSs were equipped with passive-retro-reflective markers. The TSs then walked on the treadmill for at least 2.0 minutes to allow them to adapt to the testing environment. Subsequently, the TSs performed gait tests for 3.0 minutes, whereas body kinematics and smartphone videos were collected for both MOCAP systems. The data collected during the human gait tests were used in this investigation to compare the tracking performance of the OptiTrack system and marker-based motion capture with a smartphone camera for assessing gait kinematics.

### Test Subjects (TSs)

Three TSs (2 female, 1 male,  $24.6 \pm 5.5$  years,  $172.3 \pm 11.2$  cm, and  $71.6 \pm 9.6$  kg) were volunteers in this investigation.

Basic anthropometric information on the TSs was obtained by physical measurements using a tape measure to provide relevant reference data for similar investigations. The anthropometric measurements of the TSs are shown in Table 1 according to the scheme shown in Fig. 1. All TSs were free of injuries for at least one month, had a body mass index (BMI) of less than 25.5 (Normal weight level), and were in the 20–30-year age range.

## Assembly, Acquisition, and Calibration Setup

### Assembly setup

The experimental setup, as shown in Fig. 2, was developed in the Laboratory of Seismic engineering and Structural at Universidad del Valle, Cali, Colombia, and consists of a SOLE F65 treadmill (belt length and width  $1.40 \times 0.55$  m, 60 Hz belt speed update frequency, and  $0.5\text{--}10 \text{ m} \cdot \text{s}^{-1}$  speed range) on which the human gait of the three TSs in this study was performed. A rigid frame, set up at a horizontal distance of 2.25 m parallel to the sagittal plane of the treadmill, was used as a support for the six interconnected 3-D motion capture system with their acquisition integrated circuit (OptiTrack system) and a Huawei Ascend G7 smartphone system with a 13-megapixel main camera. Both video and motion capture data were recorded using both MOCAP

**Fig. 1** Schematic anthropometric information

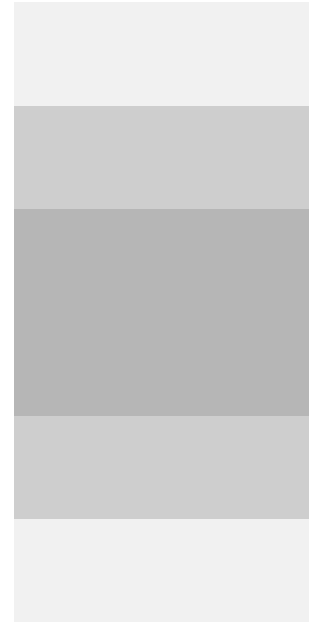


**Table 1** Anthropometric information

|               |            | Test subject |       |       |
|---------------|------------|--------------|-------|-------|
|               |            | No. 1        | No. 2 | No. 3 |
| Length (cm)   | $L_1$      | 27.2         | 30.0  | 27.0  |
|               | $L_2$      | 47.5         | 47.5  | 43.0  |
|               | $L_3$      | 22.3         | 20.0  | 23.0  |
|               | $L_4$      | 18.7         | 11.5  | 10.5  |
|               | $L_5$      | 31.4         | 29.3  | 31.0  |
|               | $L_6$      | 26.7         | 25.2  | 27.0  |
|               | $L_7$      | 17.7         | 22.5  | 17.0  |
|               | $L_8$      | 47.5         | 49.0  | 51.0  |
|               | $L_9$      | 46.7         | 37.2  | 44.0  |
|               | $L_{10}$   | 9.8          | 7.9   | 10.0  |
|               | $L_{11}$   | 17.5         | 14.2  | 17.0  |
|               | $L_{12}$   | 19.4         | 19.0  | 20.0  |
| Diameter (cm) | Head       | 57.0         | 54.0  | 55.0  |
|               | Shoulder   | 37.0         | 40.0  | 45.0  |
|               | Elbow      | 25.0         | 27.0  | 25.0  |
|               | Wrist      | 17.0         | 18.0  | 17.0  |
|               | Hip        | 90.0         | 97.0  | 87.0  |
|               | Knee       | 38.9         | 38.0  | 35.0  |
|               | Ankle      | 25.0         | 25.0  | 23.0  |
|               | Heel       | 32.0         | 30.0  | 32.0  |
|               | Metatarsus | 24.0         | 22.0  | 22.0  |

systems. Each optical system transferred the information to laptops, which processed the data captured during human gait tests. A flash of light and an audible beep were used at the beginning and end of each human gait test, respectively, to synchronize the data from both systems.

### Pinhole camera model calibration

The pinhole camera model is based on the model proposed by J. Bouguet [60], as presented in equation (1), where  $Depth_u$  corresponds to approximated depth value to the sagittal plane,  $(x_u, y_u)$  are undistorted depth pixel coordinates, and  $(f_x, f_y, c_x, c_y)$  are intrinsic parameters of the camera. This model requires camera calibration to calculate the intrinsic and extrinsic parameters of a smartphone camera. The extrinsic parameters represent the location of the camera in the 3-D scene and the intrinsic parameters represent the optical center and focal length of the camera. The world points are transformed into camera coordinates using extrinsic parameters, and the camera

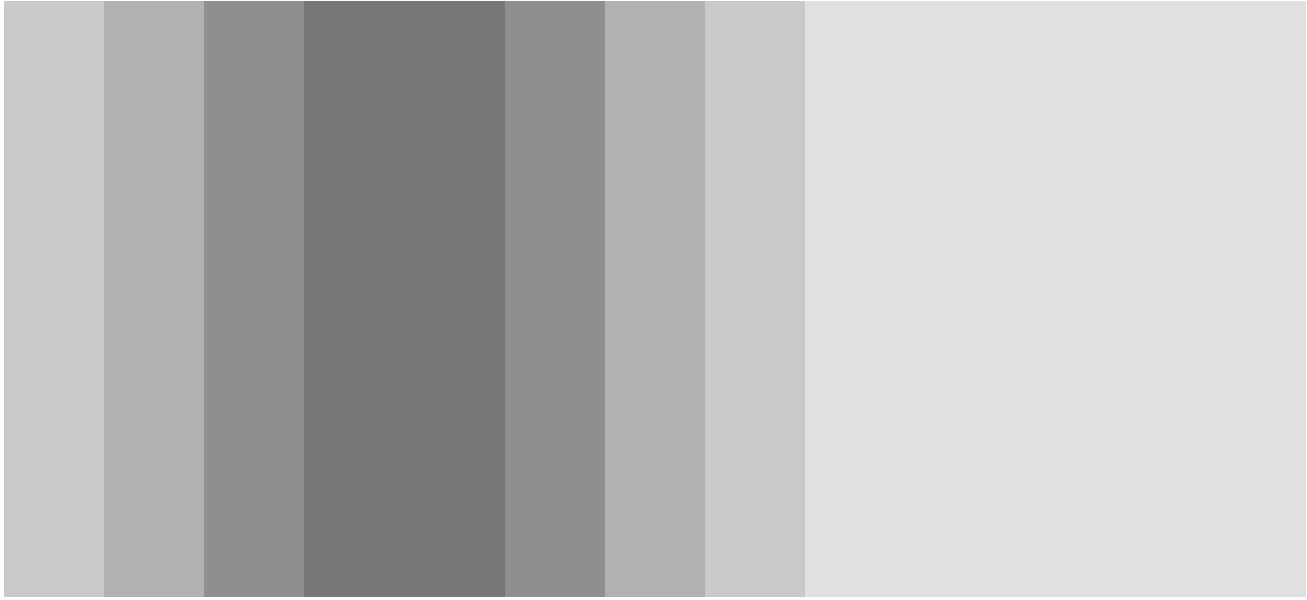


**Fig. 2** Scheme setup: experimental assembly (left), and Schematic assembly (right)

**Table 2** Smartphone Imaging Sensor Calibration Parameters

|                       | $c_x(\text{pxl})$ | $c_y(\text{pxl})$ | $f_x(\text{mm})$ | $f_y(\text{mm})$ |
|-----------------------|-------------------|-------------------|------------------|------------------|
| Smartphone RGB sensor | 968.21            | 563.95            | 1580.96          | 1583.26          |

coordinates are mapped onto the image plane using intrinsic parameters. A special calibration pattern-type checkerboard of 9 squares on the  $x$ -axis and 13 squares on the  $y$ -axis, with a square size of 7.0 cm each, was used as the reference object for calibration. In this study, 66 images of the checkerboard at different positions with randomness and the highest resolution were selected, close to the sagittal plane of the gait zone, to estimate the pinhole camera model parameters of the camera smartphone with a mean reprojection error per image of less than 0.25 pixels.

$$X = (x_u - c_x) \cdot \frac{\text{Depth}(x_u, y_u)_u}{f_x} \quad (1)$$

$$Y = (y_u - c_y) \cdot \frac{\text{Depth}(x_u, y_u)_u}{f_y}$$

The intrinsic and extrinsic pinhole camera model parameters of the camera smartphone were calculated using the MATLAB function *estimateCameraParameters*, which uses images of the checkerboard at different positions. The intrinsic parameters are presented in Table 2.

## Data Collection Procedures

Initially, the MOCAP systems were calibrated using the procedure outlined in Section "Experimental Methods" for low-cost smartphone and the procedure recommended by the manufacturer for OptiTrack. Nine passive retroreflective adhesive markers, with a diameter of 12.0 mm were attached to specific joints of the human body, as shown in Fig. 3. Prior to each gait test, the TSs were adapted to the gait velocity by walking on the treadmill for two minutes. Tests were conducted at three velocities: 1.50, 1.90, and 2.30  $\text{m} \cdot \text{s}^{-1}$ , which were chosen as they are representative of the slow, normal, and fast velocities of human gait [61]. Marker-based MOCAP system data were simultaneously collected during all human gait tests and used for assessment in this investigation. In addition, the mean sample rate of the smartphone camera was determined in  $29.85 \pm 0.89$  fps. Similarly, an average capture rate of 100.0 fps was reported for the OptiTrack system.

## Marker-based OptiTrack and Low-Cost Smartphone System Data Processing

Gait kinematics were acquired using a middle-body sagittal model (human middle body, compound of seven rigid body segments generated by nine passive retroreflective markers), and the output was processed using custom algorithms in MATLAB to extract the joint coordinates. The anatomical joints were then labelled as the head, shoulder, elbow, wrist, hip, knee, ankle, heel, and metatarsus, as shown in Fig. 3. Although the OptiTrack system tracks

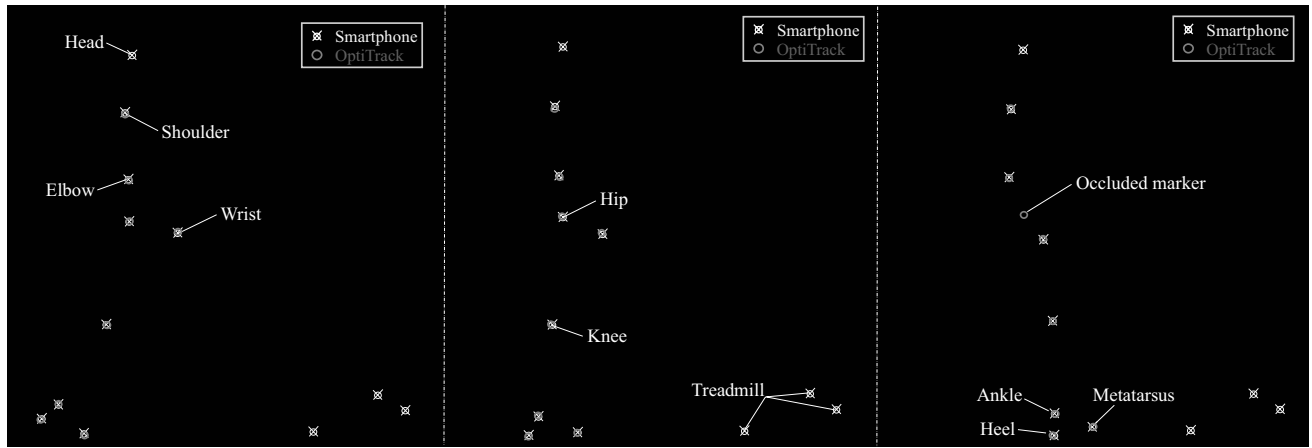


**Fig. 3** Example images from one test subject in the test set showing the identified landmarks with OptiTrack and low-cost smartphone systems

and computes gait kinematics using its own hardware and software, the low-cost MOCAP system tracks each of the nine passive retroreflective markers in each video frame. The spatial position of each marker in the gait-plane was determined using the calibration parameters and the custom algorithms developed in MATLAB. The  $x$ - $y$  positions of the nine reflective markers of the TSs were acquired and processed for all the test times. On the other hand, when a measurement contained incorrect data or empty data owing to non-redundancy capture or occluded marker (**Figure**, right), the gaps were filled with a combination of splines and estimated from previous data. The biomechanical results of marker-based motion capture were compared based on their potential relevance for acquiring kinematic data for human gait [6, 7, 13, 24, 25, 48]. Only the right side of the body was used for assessment as it was closest to the sagittal view in the MOCAP system. In addition, the length of the gait period acquired from the tests will be able to support the statistical analysis of performance between both MOCAP systems in subsequent studies, positioning these records as reference data.

### Motion Capture Systems Performance Evaluation

The tracking accuracy of the low-cost smartphone system in comparison with the tracking realized by the gold standard system (OptiTrack) was performed using the coefficients of Normalized Root Mean Square Error (nRMSE) and goodness-of-FIT (FIT), as described in equations (2) and (3), for the assessed joint angles, which were generated by a passive retroreflective marker across all temporal assessments and all TSs. Both acquired data systems were resampled at a rate of 100.0 Hz for their comparison.

$$nRMSE = \frac{100}{x} \sqrt{\frac{1}{n} \sum_{j=1}^n (y_{OptiTrack} - \hat{y}_{Smartphone})^2} \quad (2)$$

$$FIT = (1 - \|y_{OptiTrack} - \hat{y}_{Smartphone}\| / \|y_{OptiTrack} - \text{promedio}(y_{OptiTrack})\|) \quad (3)$$

Where  $y_{OptiTrack}$  and  $\hat{y}_{Smartphone}$  are the measurement signals ( $x$ - $y$  trajectory) with the OptiTrack and low-cost smartphone systems,  $x$  is the normalization factor, which is equal to the difference between the maximum and minimum values in the required range of the reference signal and  $n$  is the data number.

### Results

The  $x$ - $y$  positions of the head, shoulder, elbow, wrist, hip, knee, ankle, heel, and metatarsus were acquired, according to the arrangement of the retroreflective markers in Fig. 4. The low-cost smartphone system performed an acceptable spatiotemporal tracking compared to that of the commercial OptiTrack system, as shown in **Figure**. In this figure, three representative gait cycles at a velocity of  $1.90 \text{ m} \cdot \text{s}^{-1}$  are shown for illustrative purposes and all other gait cycle data is included in the shaded bands. Likewise, hip, knee, and ankle relative angle in the sagittal plane were calculated with low-cost smartphone system data show joint extensions with minimal differences from those calculated with the OptiTrack system data for the gait velocities of 1.50, 1.90 and  $2.30 \text{ m} \cdot \text{s}^{-1}$ , are shown in Fig. 5. The relative angles were selected in this comparative study because of their major contribution to the mechanical energy expenditure during

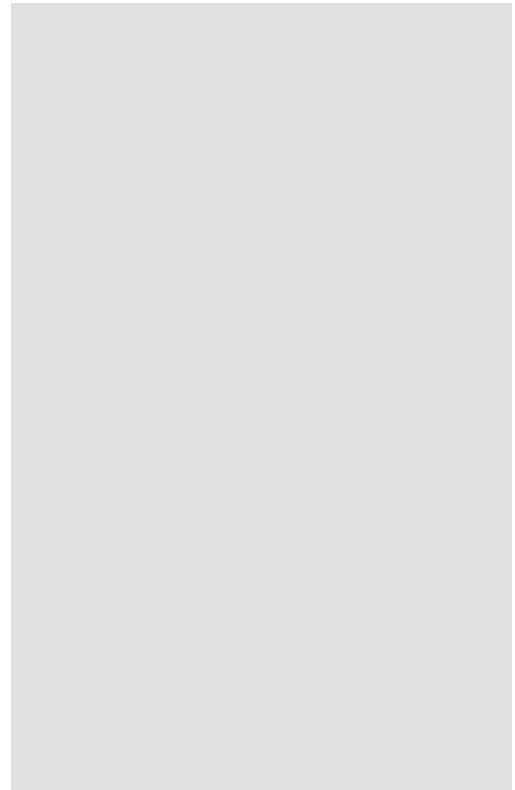


**Fig. 4** Measured x-y trajectory of Subject No.1 at a gait velocity of  $1.90 \text{ m} \cdot \text{s}^{-1}$  with OptiTrack and low-cost smartphone systems. The blue area is the size of the retroreflective marker calculated by the low-cost smartphone system

gait tests, relevant to human gait models [62]. In these figures, the blue area is the size of the retroreflective marker calculated by the low-cost smartphone system, which for the x-y positions cover the trajectory determined by the OptiTrack system, evidencing the high accuracy in the tracking of this low-cost marker-based system.

The nRMSE and FIT for the assessed joint relative angles, across all temporal assessments and all TSs for the low-cost smartphone system and OptiTrack system are reported in Table 3 at  $1.50$ ,  $1.90$  and  $2.30 \text{ m} \cdot \text{s}^{-1}$ .

## Discussion

The relative hip ( $\theta_1$ ) and knee ( $\theta_2$ ) angles, calculated from OptiTrack and low-cost smartphone system data, demonstrate a high level of accuracy in estimating human gait kinematics across all tested velocities. The goodness-of-fit averages above 88.93%, with an nRMSE average below 2.71%. This accuracy surpasses rates reported in previous studies for gait velocities between  $2.5$ - $3.4 \text{ m} \cdot \text{s}^{-1}$  [63–66], as illustrated in Fig. 6. According to the resources to post-process data collected using the MOCAP systems described in Section "Experimental Methods" and the results in Section "Results", the

low-cost smartphone system was more cost-effective than OptiTrack because of the simple calibration requirement and open-access algorithms. However, in the case of angle ankle ( $\theta_3$ ), a lower performance was obtained, with an average goodness-of-FIT of 72.07% and nRMSE of 2.86%. This decrease in accuracy was due to the speed of angular movement of the ankle and the low capture rate of the low-cost smartphone system ( $\approx 30.0 \text{ fps}$ ) compared to the OptiTrack system ( $\approx 100 \text{ fps}$ ). These factors produced a deformation of the marker in the acquired frame-propagating error in the calculation of the marker centroid, as shown in Figure (right). This causes a decrease in the estimation of the ankle angle ( $\theta_3$ ) and an increase in the noise of the calculated data; this situation has been previously reported in other studies [67]. In addition, the maximum peaks of the relative angles evaluated by both MOCAP systems were determined for all the TSs and velocities in this study, as shown in Table 4. These analyses demonstrated differences between the maximum relative angles calculated by both systems of less than 3.40% for speeds of  $1.50$ - $1.90 \text{ m} \cdot \text{s}^{-1}$  and differences of less than 12.20% for speeds of  $2.50 \text{ m} \cdot \text{s}^{-1}$ , which demonstrating the robustness and accuracy of the low-cost smartphone system for calculating kinematics for typical gait speeds.



**Fig. 5** Relative angle calculated hip ( $\theta_1$ , left column), knee ( $\theta_2$ , middle column), and ankle ( $\theta_3$ , right column) with OptiTrack (red) and low-cost smartphone (blue) systems at gait velocities of 1.50 (top), 1.90 (middle), and 2.30  $\text{m} \cdot \text{s}^{-1}$  (bottom). The blue area is the size of the retroreflective marker calculated by the low-cost smartphone system

Despite the optical redundancy demonstrated by the OptiTrack system (3D view) in the experimental setup (Fig. 2), there is a clear need for improvement in the spatial distribution of the optical devices around the TS to prevent the loss of kinematic data, even when only data in the sagittal plane of the gait are desired. In contrast, low-cost smartphone system uses a single optical device (2D view) and exhibits the inherent limitations of optical redundancy and spatial distribution, which can be overcome by combining the vision from multiple cameras to approach a three-dimensional view, as demonstrated in other studies [45, 53, 67–69]. However, based on the obtained results, the MOCAP system with smartphones achieves optimal performance with minimal errors compared with the commercial OptiTrack system, positioning it as a low-cost motion capture system with the potential for further development because of the rapid advancement

of technical specifications in smartphones and their accessibility to the general population.

### Limitations and Future Work

Overall, the results and discussion show that the low-cost smartphone system used as MOCAP systems exhibit a difference less than 3.40% in the maximum peak relative angles of the joints in the sagittal plane during treadmill gait compared to the commercial OptiTrack system for velocities less or equal than 1.90  $\text{m} \cdot \text{s}^{-1}$ . However, the low capture rate of the smartphone device, combined with the lack of inherent redundancy, resulted in markers that moved at a higher relative speed during the gait cycles, experienced deformation during optical capture, generated noise in the estimation of the  $x$ - $y$  trajectories, as shown in **Figure** (right), and consequently in the calculation of the relative angle of the ankle ( $\theta_3$ ).

**Table 3** Goodness of FIT and nRMSE for relative angles of the test subjects at 1.50, 1.90, and 2.30 m • s<sup>-1</sup>

|                 |                          |                   |           | Relative angles    |                     |                      |
|-----------------|--------------------------|-------------------|-----------|--------------------|---------------------|----------------------|
|                 |                          |                   |           | Hip [ $\theta_1$ ] | Knee [ $\theta_2$ ] | Ankle [ $\theta_3$ ] |
| Gait velocities | 1.50 m • s <sup>-1</sup> | Test Subject No.1 | FIT (%)   | 87.20              | 89.61               | 65.00                |
|                 |                          |                   | nRMSE (%) | 4.28               | 2.74                | 7.21                 |
|                 |                          | Test Subject No.2 | FIT (%)   | 93.94              | 91.62               | 69.39                |
|                 |                          |                   | nRMSE (%) | 0.48               | 1.34                | 1.53                 |
|                 |                          | Test Subject No.3 | FIT (%)   | 93.12              | 93.44               | 90.30                |
|                 |                          |                   | nRMSE (%) | 0.02               | 0.02                | 0.03                 |
|                 | 1.90 m • s <sup>-1</sup> | Test Subject No.1 | FIT (%)   | 89.58              | 91.37               | 72.61                |
|                 |                          |                   | nRMSE (%) | 2.97               | 2.63                | 5.66                 |
|                 |                          | Test Subject No.2 | FIT (%)   | 92.62              | 88.05               | 74.02                |
|                 |                          |                   | nRMSE (%) | 0.55               | 2.12                | 1.50                 |
|                 |                          | Test Subject No.3 | FIT (%)   | 88.46              | 87.84               | 81.65                |
|                 |                          |                   | nRMSE (%) | 0.03               | 0.04                | 0.04                 |
|                 | 2.30 m • s <sup>-1</sup> | Test Subject No.1 | FIT (%)   | 82.64              | 88.65               | 71.90                |
|                 |                          |                   | nRMSE (%) | 4.74               | 3.49                | 5.87                 |
|                 |                          | Test Subject No.2 | FIT (%)   | 85.44              | 86.43               | 73.30                |
|                 |                          |                   | nRMSE (%) | 1.63               | 5.58                | 3.77                 |
|                 |                          | Test Subject No.3 | FIT (%)   | 91.63              | 85.85               | 50.49                |
|                 |                          |                   | nRMSE (%) | 0.03               | 0.05                | 0.11                 |

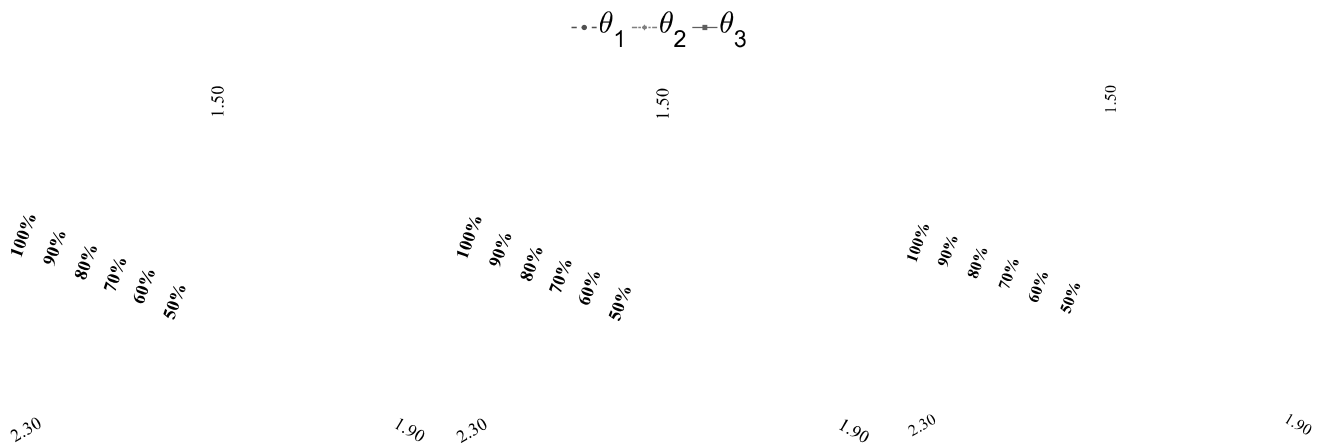


**Fig. 6** Goodness of FIT calculated for Relative angle acquired with OptiTrack and low-cost smartphone systems at gait velocities of 1.50, 1.90, and 2.30 m • s<sup>-1</sup> for TS<sub>1</sub> (left), TS<sub>2</sub> (middle), and TS<sub>3</sub> (right)

To overcome these limitations, smartphones with higher capture rates and a greater number of strategically placed devices can be used to provide optical redundancy, thereby enabling more precise and reliable estimations. Additionally, this low-cost smartphone-based motion capture system is expected to be accurate and cost-effective, which can easily be achieved owing to the increasing technical specifications of this technology and its easy accessibility. The findings

of this study establish a smartphone-based motion capture system as a precise, reliable, and cost-effective technology for the study and assessment of human gait. In addition, although these marker-based systems utilize paid software (e.g., MATLAB) for some of the data processing, further steps are necessary to implement these processing steps in free software (e.g., Python) without the need for coding to facilitate usage by practitioners. In this regard, the integration

**Table 4** Maximum of relative angles of the test subjects at 1.50, 1.90, and 2.30 m • s<sup>-1</sup>

|                 |                        |                   |            | Hip $ \theta_1 $ | Maximum peak relative angles |                   |              |                    | $\Delta$ [%] |
|-----------------|------------------------|-------------------|------------|------------------|------------------------------|-------------------|--------------|--------------------|--------------|
|                 |                        |                   |            |                  | $\Delta$ [%]                 | Knee $ \theta_2 $ | $\Delta$ [%] | Ankle $ \theta_3 $ |              |
| Gait velocities | $1.50m \bullet s^{-1}$ | Test Subject No.1 | OptiTrack  | 15.20            | 1.97                         | 55.30             | 1.80         | -57.50             | 1.39         |
|                 |                        |                   | Smartphone | 14.90            |                              | 54.70             |              | -58.30             |              |
|                 |                        | Test Subject No.2 | OptiTrack  | 13.80            | 2.22                         | 54.60             | 1.49         | -55.20             | 2.36         |
|                 |                        |                   | Smartphone | 13.50            |                              | 53.80             |              | -56.50             |              |
|                 |                        | Test Subject No.3 | OptiTrack  | 14.70            | 3.40                         | 54.90             | 1.09         | -58.10             | 2.23         |
|                 |                        |                   | Smartphone | 14.20            |                              | 54.30             |              | -59.40             |              |
|                 | $1.90m \bullet s^{-1}$ | Test Subject No.1 | OptiTrack  | 16.10            | 1.89                         | 54.10             | 1.50         | -55.60             | 2.58         |
|                 |                        |                   | Smartphone | 15.80            |                              | 53.30             |              | -54.20             |              |
|                 |                        | Test Subject No.2 | OptiTrack  | 14.20            | 0.70                         | 55.10             | 0.54         | -53.60             | 2.98         |
|                 |                        |                   | Smartphone | 14.10            |                              | 54.80             |              | -55.20             |              |
|                 |                        | Test Subject No.3 | OptiTrack  | 14.80            | 1.36                         | 54.30             | 1.11         | -56.90             | 1.41         |
|                 |                        |                   | Smartphone | 14.60            |                              | 53.70             |              | -57.70             |              |
|                 | $2.30m \bullet s^{-1}$ | Test Subject No.1 | OptiTrack  | 22.50            | 12.20                        | 58.80             | 1.20         | -51.20             | 6.25         |
|                 |                        |                   | Smartphone | 19.80            |                              | 58.10             |              | -54.40             |              |
|                 |                        | Test Subject No.2 | OptiTrack  | 14.80            | 8.82                         | 57.30             | 0.88         | -52.30             | 2.68         |
|                 |                        |                   | Smartphone | 13.60            |                              | 56.80             |              | -50.90             |              |
|                 |                        | Test Subject No.3 | OptiTrack  | 19.80            | 5.32                         | 58.60             | 2.09         | -54.50             | 3.22         |
|                 |                        |                   | Smartphone | 18.80            |                              | 57.40             |              | -52.80             |              |

of real-time analysis is expected to provide an important tool for professionals and researchers, which can be achieved using recent tools focused on real-time pose estimation [58].

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**Data Availability** Informed consent was obtained from all study participants according to the guidelines of the ethics committee of the Universidad del Valle. The raw and time-normalized marker-based with OptiTrack and Smartphone kinematic data, as well as the test video, are available from the Mendeley Data at DOI <https://doi.org/10.17632/hnecbssmdb.1>

## Declarations

**Competing Interest** The authors declare that they have no know competing financial interest or personal relationships that could have appeared to influence the work reported in this paper.

**Institutional Review Board** This study was approved by the ethics committee of the Universidad del Valle (Acta de Aval No.011-2023); in

addition, all participants signed an informed consent form before the measurements.

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### ***3.3 Modeling of Human-Structure Interaction: Predictive Biomechanical Approaches and Bridge Response Analysis.***

The interaction between pedestrians and flexible pedestrian bridges has emerged as a significant research area due to the increasing utilization of lightweight and slender structures in modern urban infrastructures. These bridges are highly susceptible to vibrations caused by pedestrian traffic, which can lead to discomfort, reduce serviceability, and even risk structural integrity. To accurately predict and mitigate such effects, robust models that couple the dynamic responses of both the pedestrian and the structure are essential.

Recent developments in Human-Structure Interaction (HSI) modeling have integrated advanced biomechanical models to predict pedestrian gait on flexible bridges. Two-dimensional and three-dimensional predictive biomechanical models have been employed to capture the complexity of human gait dynamics under varying structural conditions. These models, grounded in inverse dynamics and optimization techniques, aim to minimize mechanical energy expenditure and simulate the gait cycle with high accuracy. The two-dimensional models typically represent the human body as a system of rigid segments, while the three-dimensional models incorporate more complex interactions, providing a comprehensive understanding of the human gait as it responds to structural vibrations.

For the structural component, pedestrian bridges are often modeled as simply supported beams following the Euler-Bernoulli beam theory, which allows for an analytical description of the dynamic response of footbridges to pedestrian loads. The coupling of human gait models with bridge vibration models enables the prediction of ground reaction forces (GRFs), pedestrian-induced accelerations, and structural displacements, facilitating a more accurate representation of the human-bridge dynamic behavior of the system.

The first article, “*Evaluation of Vertical Human-Structure Interaction on a Pedestrian Bridge Using a Predictive Human Gait Model*”, provides significant contributions to the understanding of vertical human-structure interaction (HSI) through a two-dimensional biomechanical approach. By focusing on the sagittal plane, this study employs inverse dynamics and optimization techniques to predict human gait on flexible pedestrian bridges, emphasizing the minimization of mechanical energy expenditure. This work demonstrates the effects of human-induced vibrations on bridge dynamics, offering a robust model for assessing vertical forces and bridge displacements under pedestrian loads. The second article, “*Modelling Human-Structure Interaction in Pedestrian Bridges Using a Three-Dimensional Biomechanical Approach*”, extends the evaluation of HSI by incorporating a three-dimensional biomechanical model. This model captures the complexities of human gait and its interaction with structural vibrations in all three dimensions, leading to a more comprehensive analysis of pedestrian-induced effects. By integrating this 3D human model with a pedestrian bridge represented as a Euler-Bernoulli beam, the study provides more precise predictions of gait kinematics and kinetics, as well as bridge responses under dynamic loading conditions. This approach enables a deeper understanding of how structural vibrations influence pedestrian movement and vice versa.

## Research Article

# Evaluation of Vertical Human-Structure Interaction on a Pedestrian Bridge Using a Predictive Human Gait Model

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Many modern pedestrian bridges exhibit flexibility and susceptibility to vibrations due to the use of lightweight and high-strength materials, which can cause discomfort for pedestrians and affect their serviceability. Although gait biomechanics have been extensively studied and optimisation techniques for gait prediction on rigid surfaces have been previously employed, there is a paucity of studies investigating the effects of human-structure interaction on pedestrian crossings over flexible structures. In this study, inverse dynamics and optimisation techniques were employed to predict human gait on a flexible structure in the sagittal plane. Gait was formulated as an optimal motor task subject to multiple constraints, with the performance criterion being the minimization of mechanical energy expenditure throughout a complete gait cycle. Segmental movements, pedestrian-applied forces, and bridge vibrations were predicted based on parameters describing gait (such as gait speed, gait frequency, and double support duration), as well as physical and dynamic parameters characterizing the pedestrian bridge (including natural frequency, damping coefficient, and bridge length).

## 1. Introduction

The main source of excitation of pedestrian bridges corresponds to the forces induced by the traffic of people occupying them. However, with the increasing progress in the mechanics of materials linked to current construction trends, architectural designs have focused on the development of light and slender structures with large spans between support points [1]. These physical characteristics cause their dynamic properties, particularly the modal frequencies, to present values below 10 Hz, which makes them highly susceptible to dynamic loads in this frequency range [2, 3]. However, it has been shown that the normal walking frequency of a pedestrian corresponds to approximately 2 Hz [1], thereby demonstrating the possible occurrence of excessive vibrations resulting from the close coincidence of bridge frequencies and anthropometric loads. This was the case for the Millennium Bridge [4] and the Solferino Bridge [5], which were closed days after their

opening because of excessive lateral vibrations caused by pedestrian traffic.

Following these events, standards were developed to govern the design of these types of structures [6–9]. However, none of these fully consider Human Structure Interaction (HSI) and mostly only suggest a minimum modal frequency value and an assessment using a moving ground reaction force (GRF) model [10]. Therefore, it is possible to expect an underestimation of the vibrations produced by human activity [11], triggering rehabilitation and maintenance cost overruns to the initial budget. In this context, the study of HSI plays an important role in the design of pedestrian structures, which is why the development of robust models and methodologies that adequately represent HSI characteristics is of great relevance.

To reliably represent the dynamic behavior of pedestrian bridges under anthropometric loads, it is necessary to analyze the effect of human loads on the dynamic response of the structure and the effect of vibrations on pedestrian gait.

However, this problem is scaled depending on the number of pedestrians, affecting walking conditions such as frequency and step length in individual people [12]; this phenomenon is known as Human-Human Interaction (HHI), and on the other hand, changing the dynamic properties of the coupled human-structure system, in contrast to the empty structure or with low occupancy levels [13–15]. In this paper, we will focus on HSI and consider a single pedestrian in the structure; in that sense, we will disregard the effects of HHI.

To simulate the HSI, several models have been proposed, the simplest of which corresponds to describing the human gait as a time varying equivalent GRF, represented by a Fourier series [16, 17]. However, this basic model does not consider HSI effects. This neglects the influence of the human body on structural dynamics and the impact of structural vibrations on anthropometric loads [18, 19]. Therefore, researchers introduced a model that represents pedestrians as a single-degree-of-freedom (SDoF) dynamic system consisting of a spring, mass, and damper (SMD) in the vertical direction. Many other researchers have developed similar models, known as biodynamic models [14, 20–23], and one or more degrees of freedom have been added with parameters defined by experimental tests [14, 24–29].

The second method used by the authors represents human gait using a mechanical inverted pendulum model (IPM) [30, 31], which consists of two rigid legs connected to a concentrated mass. However, in this model, the representation of GRFs is limited by simply considering single-stance phases during gait [31]. Therefore, the IPM has become sophisticated by incorporating springs into each limb and defining the double stance phase, resulting in a spring-loaded inverted pendulum (SLIP) [32]. Its main limitation is that it overestimates the GRFs in the double stance phases. To improve the SLIP model, the damped bipedal inverted pendulum model (DSLIP) [33, 34] has been proposed as the main option because it has been shown to provide a good approximation in the representation of vertical GRFs. IP, SLIP, and DSLIP models have been incorporated into HSI studies [33, 35, 36]; however, the reliability of GRFs in vibrating structures has not yet been demonstrated [32].

Although these investigations represent an important advance, the main problem lies in the low accuracy of GRFs, due to the simplifications assumed by this type of model, especially the transient, nonstationary condition and the biological variability of human gait [37]. On the other hand, it has been shown that a realistic representation of human gait based on anthropometric characteristics improves the prediction of GRFs and therefore of HSI [38]. In this context, the authors have made advances in the development of biomechanical gait models that consider anthropometric characteristics based on optimisation techniques [39, 40]. Although the previous methodology has, among its advantages, the possibility of including more specific characteristics of people, the problem lies in its high computational requirements. Therefore, the inverse dynamics method has become a very attractive alternative for the development of human gait models based on optimisation techniques

[39–41] because it does not require the solution of differential equations, which are responsible for the greatest processing load. However, these models do not consider the support surface motion. In this sense, there is still a need to develop robust models capable of accurately and efficiently predicting HSI characteristics, which will serve as a starting point for the formulation of appropriate standards in the design of pedestrian structures.

In this study, a coupled HSI model involving the dynamics of human and structural systems is proposed. This model is based on the principle that natural processes use as little energy as possible to execute [37, 39]. Therefore, the mechanical energy required to cross a pedestrian bridge is proposed as an objective function. In addition, it includes a set of kinematic, kinetic, and forward constraints that allow it to predict human walking.

## 2. Human-Structure Interaction Model

The HSI model implemented in this research combines the representation of human gait in the sagittal plane proposed by [39] and the analytical description of a pedestrian bridge (see Figure 1).

**2.1. Human System.** The movement of the human body on the sagittal plane is represented through a model of 7 rigid elements connected by joints, as proposed by [39]. The upper limbs, head, and torso are combined into a single element called the head-arm trunk (HAT), and each leg contains three elements representing the thigh, shank, and feet. The kinematics of this model are described by a Fourier series representing the angular displacement ( $\theta_i$ ) of  $i$ th body segment as a function of the walking frequency ( $\omega_c$ ) due to the periodic nature of the human body movements. The angular displacement is represented by a fifth-order Fourier series as shown in the following equation:

$$\theta_i = a_o + \sum_{n=1}^5 \left[ a_n^{(i)} \cos(n\omega_c t) + b_n^{(i)} \sin(n\omega_c t) \right]. \quad (1)$$

In this equation,  $n$  is the order of the truncated Fourier series and its respective coefficients  $a_o$ ,  $a_n^{(i)}$ , and  $b_n^{(i)}$  are the optimisation parameters as described in Section 2.4. The seven-element model was used in a minimization process, where the objective function was the mechanical energy of the human gait ( $E_m$ ). This energy is expressed as the integral of the power over the time, and it takes a person to cross a pedestrian bridge or platform, as shown in equation (2). It is important to note that this methodology does not require experimental kinematic information to determine the motion of a human body during a gait cycle because all necessary information is derived from the optimisation process. The anthropometric data used for this model correspond to the anthropometric information presented in [42] and after modified by [39].

$$E_m = \int_0^{T_c} \sum_{i=1}^7 |T_i(\omega_p^{(i)} - \omega_d^{(i)})| dt. \quad (2)$$

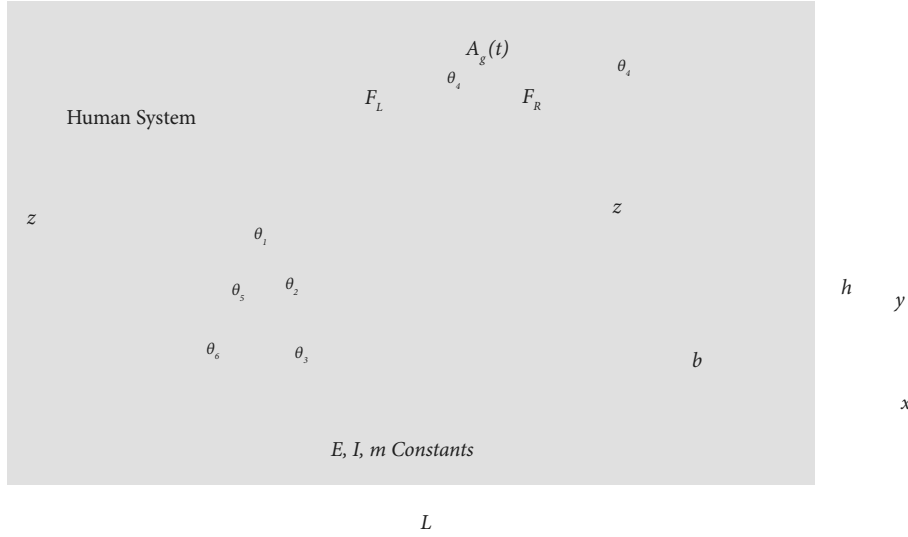


FIGURE 1: Human structure interaction system.

In equation (2),  $T_i$  represents the net torque of element  $i$ th and  $\omega_p^{(i)}, \omega_d^{(i)}$  represents the angular velocity of each joint proximal and distal element. From the movements of each body segment, the inverse dynamics methodology was used to determine the merge kinetics of the human system using the expressions formulated by [39] (equations (2)–(4)). These equations were used to calculate the reaction forces ( $\vec{F}_i$ ) and moments ( $M_i$ ), which were then used to quantify the mechanical energy used during the transit over the structure.

$$\sum \vec{F}_i = \sum_{i=1}^7 m_i (\vec{a}_i - \vec{g} + \vec{a}_g(t)), \quad (3)$$

where  $m_i$  and  $\vec{a}_i$  represent the mass and acceleration in the centre mass of  $i$ th body element and  $\vec{g}$  is the gravity. Equation (3) was modified to include the action of vibrations generated in the structure ( $\vec{a}_g(t)$ ) produced by anthropometric loads, thus coupling the merge kinetics of the HSI.

$$\sum M_i = \sum_{i=1}^7 I_i \alpha_i - \sum_{i=1}^7 \sum_{k=1}^{n_i} M_{ki}. \quad (4)$$

Likewise,  $M_{ki}$  is the moment of the resultant joint force acting at the  $k$ th joint acting on the  $i$ th segment,  $I_i$  represents the inertia, and  $\alpha_i$  corresponds to angular acceleration on the  $i$ th body element.

In the literature, there are models such as the SMD or IP, which have been recently validated on rigid surfaces, showing a well behaviour to reproduce the GRFs [32]. However, the IP models present limitations owing to the own simplifications considered, for example, the assumptions of the location of the center of mass for the whole body and the value for the stiffness and damping in the legs.

Therefore, the proposed model in this study presents the possibility of including anthropometric data to reduce the uncertainty associated with biological variability, managing to predict the body movements and consequently the GRFs by inverse dynamics, which in turn can be validated by experimental trials. In summary, based on this model, experimental validations could be performed on the two main components of the HSI, such as first, to the effect of anthropic loads on structural vibrations and the second one corresponds to the effect of these vibrations on the human walking behaviour and GRFs.

**2.2. Structural System.** The flexible structure proposed to carry out the HSI simulation consisted of a simply supported beam with a prismatic section made of normal-weight concrete. The mechanical and dynamic properties of the structure are presented in Table 1. The frequencies for each vertical vibration mode ( $f_n$ ) were calculated using equation (5). This represents the conditions of a flexible structure susceptible to dynamic loads, with a fundamental frequency close to the pedestrian walking frequency.

$$f_n = \frac{(\pi \bar{m})^2}{2\pi} \sqrt{\frac{EI}{mL^4}}, \quad (5)$$

where  $E$ ,  $I$ ,  $L$ , and  $\bar{m}$  represent the modulus of elasticity of the concrete, the moment of inertia of the section, length, and mass per unit length of the structure, respectively.

The partial differential equation (PDE) is a fourth order that describes the deflection  $u(x, t)$  of any point located on the  $x$  axis of the beam at a given instant  $t$  (equation (6)). Here,  $p(x, t)$  represents the load generated by a person walking on a structure,  $m$  represents the mass, and  $c$  is the coefficient viscous damper.

TABLE 1: Properties structural system.

| Mechanical properties |         |         |                                |              | Dynamics properties |            |            |             |
|-----------------------|---------|---------|--------------------------------|--------------|---------------------|------------|------------|-------------|
| $b$ (m)               | $h$ (m) | $L$ (m) | $\bar{m}$ (kg/m <sup>3</sup> ) | $f'_c$ (MPa) | $f_1$ (Hz)          | $f_2$ (Hz) | $f_3$ (Hz) | $\zeta$ (%) |
| 2                     | 0.031   | 12      | 2300                           | 28           | 2.017               | 8.076      | 18.152     | 5           |

$$\frac{\partial^2}{\partial x^2} \left[ EI(x) \frac{\partial^2 u(x, t)}{\partial x^2} \right] + m(x) \frac{\partial^2 u(x, t)}{\partial x^2} + c(x) \frac{\partial u(x, t)}{\partial t} = p(x, t). \quad (6)$$

The PDE was solved using the separation of variables method, which involves modal decomposition, as shown in equation (7). For this case, the vibration modes are described by sinusoidal functions (equation (8)). Equation (9) represents the motion ( $Y_{\hat{n}}$ ) of the  $\hat{n}$ th vibration mode ( $\phi_{\hat{n}}$ ) in modal coordinates, where the natural frequency ( $\omega_{\hat{n}} = 2\pi f_{\hat{n}}$ ) and the damping ratio ( $\xi$ ) are described using the values from Table 2. The mass ( $M_{\hat{n}}$ ) and modal loads ( $P_{\hat{n}}$ ) are characterised by equations (10) and (11), respectively.

$$u(x, t) = \sum_{\hat{n}=1}^{\infty} \phi_{\hat{n}}(x) Y_{\hat{n}}(t), \quad (7)$$

$$\phi_{\hat{n}}(x) = \sin\left(\frac{\hat{n}\pi}{L} x\right), \quad (8)$$

$$\ddot{Y}_{\hat{n}}(t) + 2\xi\omega_{\hat{n}}\dot{Y}_{\hat{n}}(t) + \omega_{\hat{n}}^2 Y_{\hat{n}}(t) = \frac{P_{\hat{n}}(t)}{M_{\hat{n}}}, \quad (9)$$

$$M_{\hat{n}} = \int_0^L [\phi_{\hat{n}}(x)]^2 m(x) dx, \quad (10)$$

$$P_{\hat{n}}(t) = \int_0^L \phi_{\hat{n}}(x) p(x, t) dx. \quad (11)$$

**2.3. Formulation of HSI.** Starting from the equations that describe the kinematics and kinetics of the human gait, as presented in Section 2.1, the dynamic systems of human and

structure are coupled by incorporating the force induced by walking as a function of time and position in the human system, as shown in equation (12). The left and right foot force transition  $D(t)$ , Figure 2, is defined as function normalised into 0 to 1, where in single-stance zones, that is, when only one foot is in contact, begin (BGN) to heel contact right (HCR) and toe off left (TOL) to heel contact left (HCL), the function  $D(t) = 1$ . However, in double stance zones, in the shaded area from HCR to TOL and HCL to toe off right (TOR),  $D(t)$  changes linearly from 0 to 1 and from 1 to 0, respectively.

$$p(x, t) = \left[ (1 - D(t)) \sum_{i=1}^7 m_i (a_i(t) - g + a_g(t)) \right] \delta(x - x_L(t)) + \left[ D(t) \sum_{i=1}^7 m_i (a_i(t) - g + a_g(t)) \right] \delta(x - x_R(t)), \quad (12)$$

$$x_L(t) = x'_L(t) + CoP_l(t), \quad (13)$$

$$x_R(t) = x'_R(t) + CoP_r(t),$$

here,  $x'_L(t)$  and  $x'_R(t)$  in equation (13) represent the positions of the left and right feet, respectively, where  $CoP_l(t)$  and  $CoP_r(t)$  are the positions of the pressure centres for the left and right feet, respectively. The accelerations induced by the structure owing to the human gait are represented by equation (14), where  $a_g(t)$  is calculated based on the positions of the feet and vertical modes of vibration.

$$a_g(t) = \sum_{\hat{n}=1}^3 \left[ (1 - D(t)) \sin\left(\frac{\hat{n}\pi x_L(t)}{L}\right) + D(t) \sin\left(\frac{\hat{n}\pi x_R(t)}{L}\right) \right] \ddot{Y}_{\hat{n}}(t). \quad (14)$$

The applied force of human walking in modal coordinates was obtained by combining equations (10) and (12), and we obtain equation (15)

TABLE 2: Simulation constraints. Adapted from [39].

| Constrain              | Lineal                                                                                                                                                                         | Nonlinear                                                                                                   |
|------------------------|--------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|-------------------------------------------------------------------------------------------------------------|
| Range angular motion   | $\theta_{\min}^{(i)} \leq \theta_i(t) \leq \theta_{\max}^{(i)}, t \in [0, T_c] \quad (i = 1, 2, \dots, 7)$                                                                     |                                                                                                             |
| Relative knee rotation | $\theta_{\min}^{(1)} \leq \theta_2(t) - \theta_3(t) \leq \theta_{\max}^{(1)}$<br>$\theta_{\min}^{(2)} \leq \theta_4(t) - \theta_3(t) \leq \theta_{\max}^{(2)}, t \in [0, T_c]$ |                                                                                                             |
| Foot contact           |                                                                                                                                                                                | $y_{\text{up}}(t) > 0$ , during the swing phase<br>$y_{\text{up}}(t) = 0$ , when in contact with the ground |
| Kinetics               |                                                                                                                                                                                | $F_y > 0$ and $-\mu < ([F_x/F_y]) < \mu$ , when in contact with the ground                                  |
| Stride length          |                                                                                                                                                                                | $x_{\text{ant}}(T_c) - x_{\text{ant}}(0) = V_a T_c$                                                         |

$T_c = 1/\omega_c$  is the walking period,  $\mu$  is the coefficient of static friction,  $y_{\text{up}}$  is the lowest point of the foot measured from the ground surface,  $x_{\text{ant}}$  is the position of the ankle in the x coordinate, and  $V_a$  is the walking speed. The maximum values  $\theta_{\max}^{(i)}$  and minimum angular values  $\theta_{\min}^{(i)}$  were defined with respect to the limits of the human body, which were defined in [43].

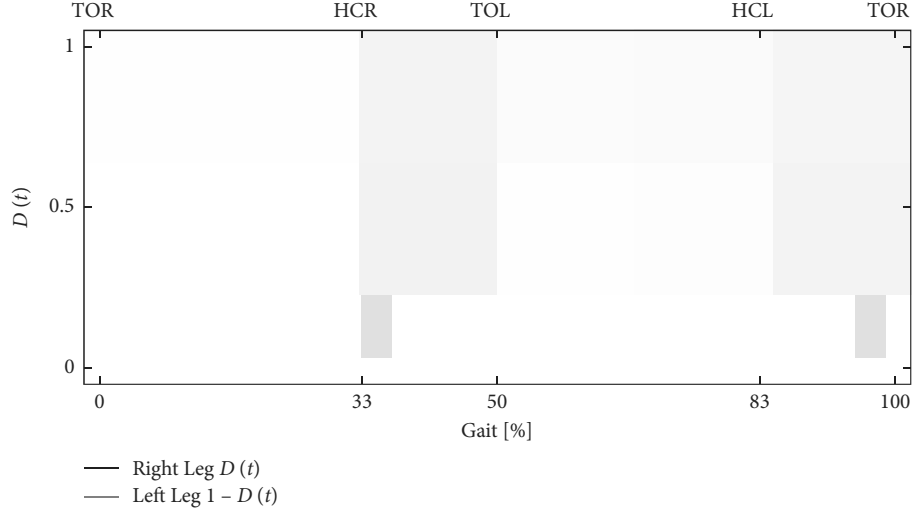


FIGURE 2: The normalised left and right foot force transition functions during a gait cycle.

$$P_{\hat{n}}(t) = - \left[ F_L(t) \sin\left(\frac{\hat{n}\pi x_L(t)}{L}\right) + F_R(t) \sin\left(\frac{\hat{n}\pi x_R(t)}{L}\right) \right] - \left[ (1 - D(t))m_p \sin\left(\frac{\hat{n}\pi x_L(t)}{L}\right) + D(t)m_p \sin\left(\frac{\hat{n}\pi x_R(t)}{L}\right) \right] a_g(t), \quad (15)$$

$$\ddot{Y}_{\hat{n}}(t) + \frac{m_p}{M_{\hat{n}}} S_{\hat{n}}(t) \left[ \sum_{m=1}^3 S_m(t) \ddot{Y}_{\hat{n}}(t) \right] + 2\zeta_{\hat{n}} \omega_{\hat{n}} \dot{Y}_{\hat{n}}(t) + \omega_{\hat{n}}^2 Y_{\hat{n}}(t) = -\frac{1}{M_{\hat{n}}} \left[ F_L(t) \sin\left(\frac{\hat{n}\pi x_L(t)}{L}\right) + F_R(t) \sin\left(\frac{\hat{n}\pi x_R(t)}{L}\right) \right], \quad (16)$$

$$S_{\hat{n}}(t) = (1 - D(t)) \sin\left(\frac{\hat{n}\pi x_L(t)}{L}\right) + D(t) \sin\left(\frac{\hat{n}\pi x_R(t)}{L}\right), \quad (17)$$

where  $F_L(t)$  and  $F_R(t)$  correspond to the forces induced by walking on the left and right feet, respectively, without considering the influence of acceleration on the structure. The total mass of the human system is denoted by  $m_p = \sum_{i=1}^7 m_i$ .

Substituting equation (15) into equation (9), the modal coordinate equation for solving the HSI model, given by equation (16), can be expressed in matrix form, as shown in equation.

$$\left[ M + m_p S^T(t) S(t) \right] \ddot{Y}(t) + C \dot{Y}(t) + K Y(t) = -\Phi(x_L(t)) F_L(t) - \Phi(x_R(t)) F_R(t). \quad (18)$$

In equation (18),  $M$  represents the modal mass,  $C$  represents the damping, and  $K$  represents the stiffness of the human-structure system, as given by equations (19)–(21), respectively.

$$M = \begin{bmatrix} M_1 & & & \\ & M_2 & & \\ & & \ddots & \\ & & & M_n \end{bmatrix}, \quad (19)$$

$$C = \begin{bmatrix} 2\zeta_1\omega_1M_1 & & & \\ & 2\zeta_2\omega_2M_2 & & \\ & & \ddots & \\ & & & 2\zeta_n\omega_nM_n \end{bmatrix}, \quad (20)$$

$$K = \begin{bmatrix} \omega_1^2M_1 & & & \\ & \omega_2^2M_2 & & \\ & & \ddots & \\ & & & \omega_n^2M_n \end{bmatrix}. \quad (21)$$

**2.4. Optimization.** The optimisation process employed an interior-point optimisation algorithm implemented in MATLAB's `fmincon` function. Although `fmincon` interior-point is a local optimizer, the large number of linear and nonlinear constraints means that the area of feasible solutions is consequently limited. In this sense, it can be expected that a local solution is close to the global minimum and that the angles and forces will be similar to those corresponding to the global minimum. This has been discussed by other authors including Ren et al. [39]. Moreover, as the initial coefficients ( $x_0$ ) used are those corresponding to gait on a rigid surface, it can be expected that the solution obtained, although possibly suboptimal, will be representative of gait motion patterns that are subject to HSI effects. The computations were carried out on a computer with the following specifications: Windows 11 Pro 64bit operating system, AMD Ryzen 3900X processor running at 3.97 GHz and 32.0 GB RAM.

The objective of the optimisation was to predict the angular kinematics of each body segment by determining the coefficients of a Fourier series of order five, as described in equation (1). The primary goal was to minimise the mechanical energy expended by the human system during the traversal of the structure, as indicated in equation (2). To ensure the feasibility of the solution, various constraints were considered, as listed in Table 2. These constraints include limitations on joint angles to prevent hyperextensions, requirements for positive vertical contact forces at the feet, adherence to friction coefficients for horizontal components, and the need to maintain at least one foot in contact with the structure throughout the gait cycle. Meanwhile, equation (18), which describes the merge HSI, was solved using the fourth and fifth-order Runge–Kutta method in `Ode45` function of MATLAB because, among its options, it allows the solution of ordinary differential equations with non-constant coefficients.

Throughout the optimisation process, the initial values for the gait parameters, such as average walking speed, walking frequency, and double support duration, were

specified. These initial values served as starting points for the optimisation algorithm, which iteratively refined the parameters to determine optimal gait characteristics. This optimisation approach aims to develop a biomechanical model that accurately predicts human gait on a flexible structure, while minimizing energy expenditure and satisfying defined constraints.

### 3. Results and Discussion

In this section, the results obtained from a simulation of the interaction between the human system and a flexible structure are presented. Detailed analyses were conducted to assess the impact of the HSI on various aspects, including the kinematics of the human body, vertical walking forces, and dynamic response of the reference structure.

An HSI simulation was performed with an 80 kg test subject walking along the reference structure at a walking frequency of  $\omega_c = 2\text{ Hz}$  and stride length of 1.5 m (two steps). Regarding foot stance times during a gait cycle, the authors agree that the duration of the dual stance phases (HCR to TOL and HCL to TOR) are in a range between 32% and 42% of the total gait cycle [44–47]. In this order, following the way of Ren et al. [39], the double support duration was defined in the ranges between 33 to 50 and 83 to 100% (see Figure 2). The simulation processing time was approximately two minutes, with 162 iterations, resulting in a minimum value of  $E_m = 201\text{ J}$  for the cost function. The optimal coefficients of the Fourier series, which describe the variation in the absolute angles of each body segment during pedestrian passage, are presented in Table 3. These coefficients allow for a precise and realistic representation of the kinematics of the human body in response to the vibration of the structure ( $a_g(t)$ ).

In Figures 3 and 4, the dotted black line and the solid blue line correspond to the results of the gait model proposed in this research for rigid surface and the results considering HSI, respectively. On the other side, the shaded areas refer to the statistical analysis of the experimental data presented in [48, 49], where the upper and lower limits are given by the mean  $\pm$  two standard deviations.

The relative angles for the hip, knee, and ankle joints, considering the effects of HSI, are shown in Figure 3. These angles were contrasted with experimental data from an optical motion capture OptiTrack system recorded by [48], showing good similarity in magnitude and shape and the differences are attributed to the stiffness of the elements of the human model, which is a characteristic of this type of model [50], confirming that the solution of the optimisation process corresponds to a normal gait pattern. The percentage comparisons with their counterparts on a rigid surface showed differences of 8, 7, and 5%, respectively. Similarly, comparisons were made for the vertical forces with and without considering the effects of the HSI. These were compared with force tests during gait recorded with a pair of insole pressure sensors (Stridalyzer Insight developed by ReTiSense) [49]. The results show that the forces predicted by the model are mostly within the range established by the experimental data. Figure 4 displays the vertical force along

TABLE 3: Fourier series coefficients.

| Element | $\theta_i$ | 0    | 1       | 2       | 3      | 4      | 5      |
|---------|------------|------|---------|---------|--------|--------|--------|
| HAT     | $a_n$      | 1.51 | 54.04   | -42.89  | 2.67   | -3.54  | 1.44   |
|         | $b_n$      |      | -20.34  | -12.1   | 5.87   | 23.93  | 0.7    |
| Thigh   | $a_n$      | 1.81 | 156.86  | -83.14  | 17.14  | 15.1   | 10.52  |
|         | $b_n$      |      | -139.29 | -20.91  | -16.06 | -0.69  | 6.73   |
| Shank   | $a_n$      | 1.31 | 453.1   | -92.95  | -9.66  | 14.76  | -6.54  |
|         | $b_n$      |      | -307.1  | -36.66  | 75.73  | -14.44 | -12.49 |
| Foot    | $a_n$      | 2.38 | 614.96  | -261.64 | 39.6   | -12.79 | 0.69   |
|         | $b_n$      |      | -386.47 | -102.11 | 138.16 | -35.6  | 5.69   |

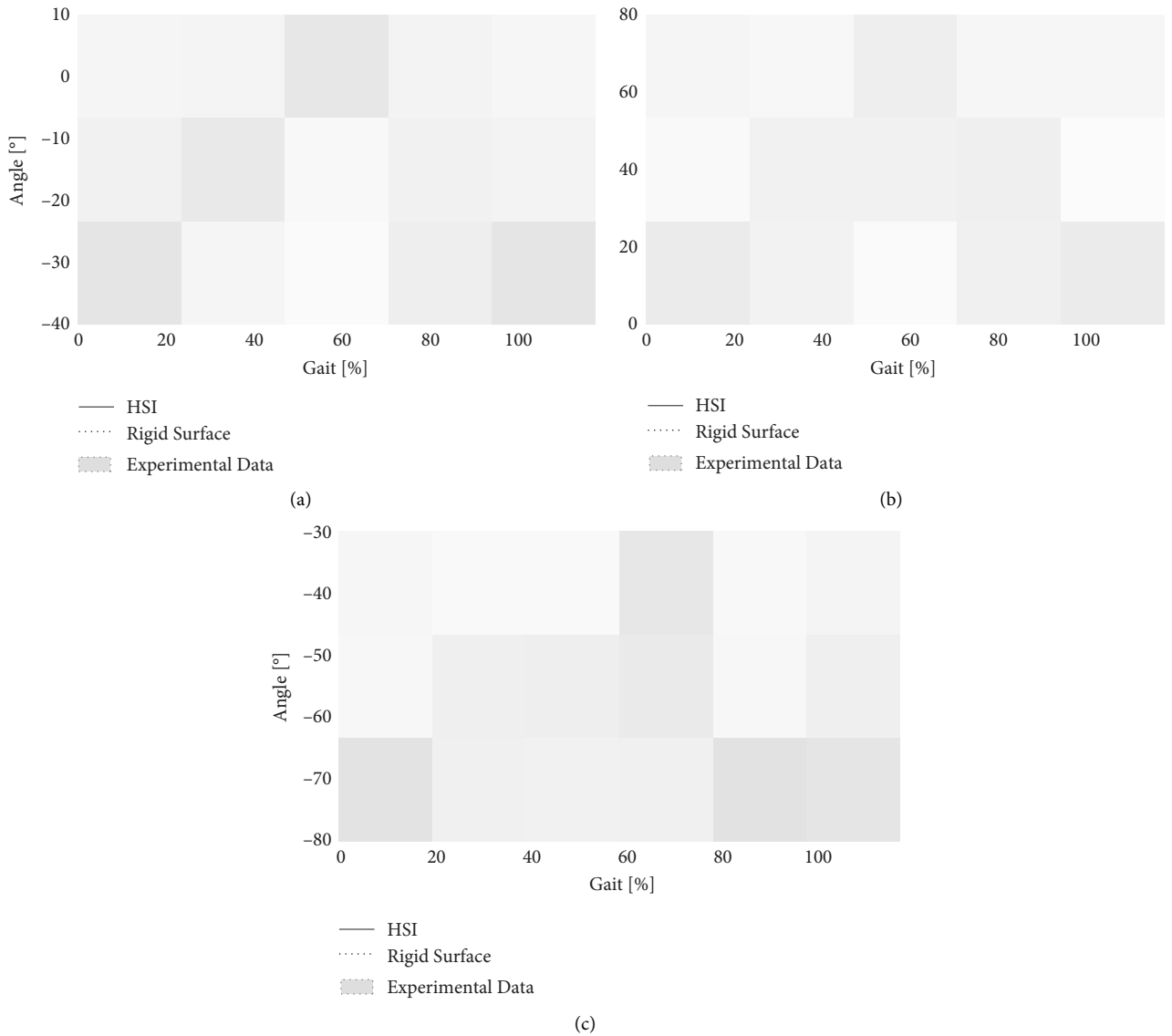


FIGURE 3: Relative angles during human gait. (a) Hip, (b) knee, and (c) ankle.

the pedestrian bridge, highlighting the gait cycle near the centre span, where the influence of the HSI on the GRFs is observed with a difference of 10%. These changes were significant, with an increase in magnitude of approximately 30 N, demonstrating the effects of HSI on the human system.

This can be primarily attributed to the flexibility of the structure and its frequency coinciding with the walking frequency.

In Figure 3, although the general shape of the angles coincides with the experimental measurements during the

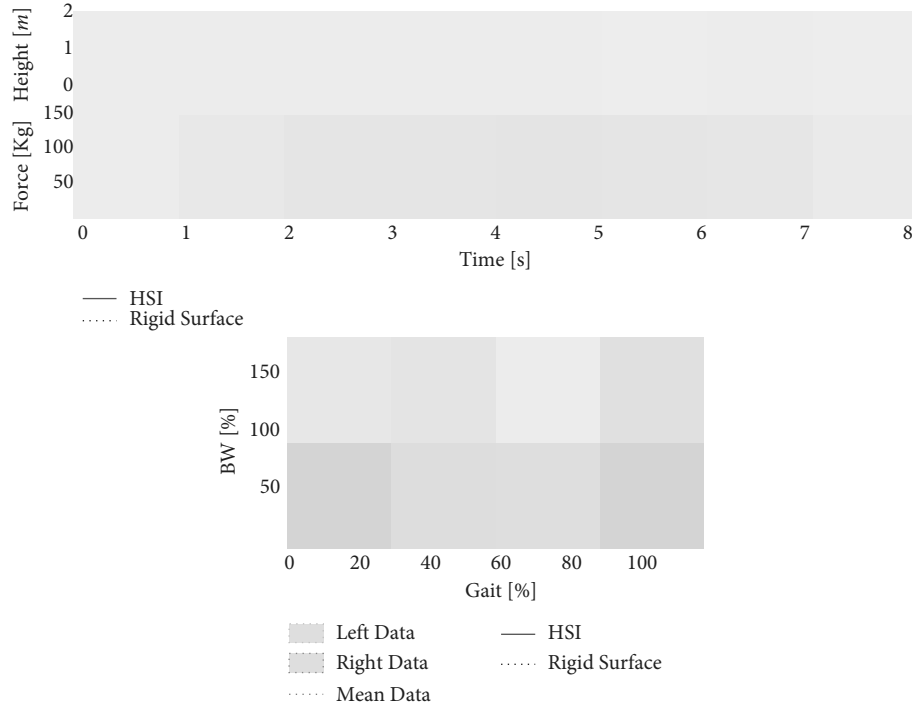
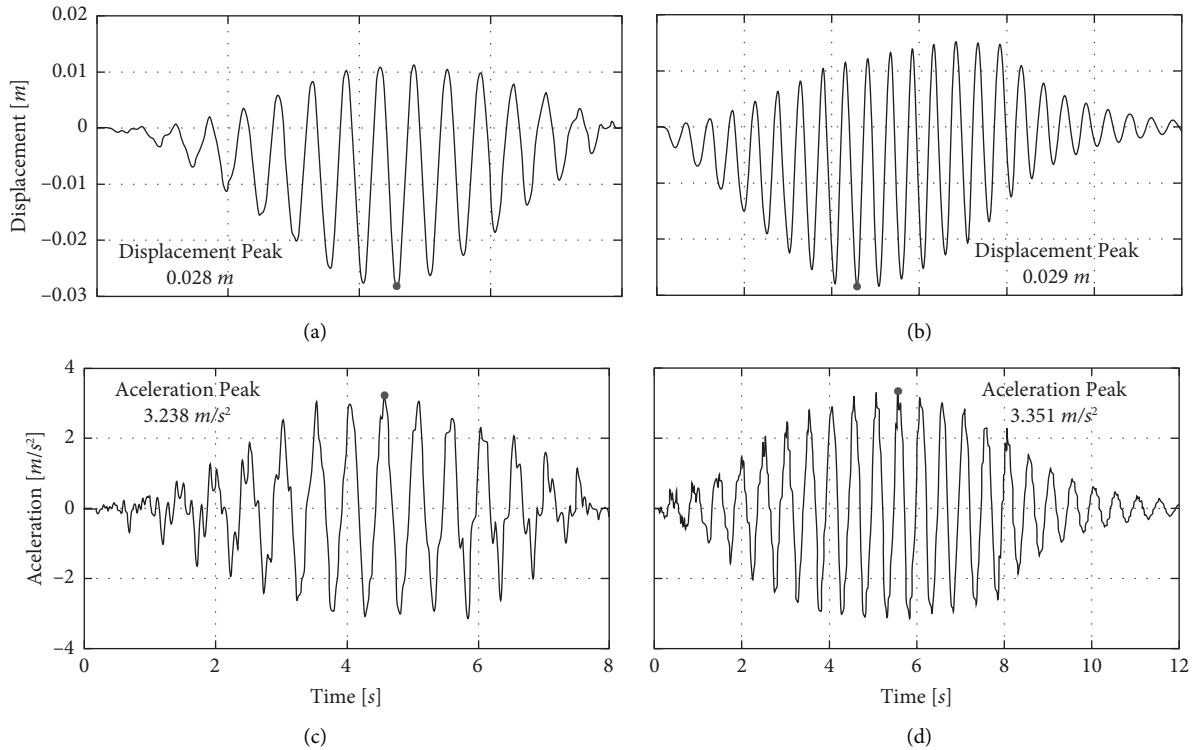


FIGURE 4: Ground reaction vertical forces during human gait.

FIGURE 5: Response dynamics structural system. (a) Displacement contact point  $D_g(t)$ , (b) displacement middle point center length bridge, (c) acceleration contact point  $a_g(t)$ , and (d) acceleration middle center.

gait cycle [48] being within the experimental reliability zone at 95.5, 82, and 81% for the pelvis, knee, and ankle angles, respectively. The differences are attributed to the

simplification of the upper body elements into a HAT element and the lack of transverse rotation of the pelvis in the model as mentioned by Ren et al. [39]. Similarly, for the

GRFs in Figure 4, the differences with the experimental data [49] are attributed to the simplification of the model and, as mentioned in the literature, to the stiffness of the elements that compose the human model [50].

The acceleration at the contact point ( $a_g(t)$ ) during HSI is shown in Figure 5(c). Acceleration levels of  $3 \text{ m/s}^2$  were recorded, with a peak of  $3.238 \text{ m/s}^2$ . These accelerations are significant enough to considerably alter the walking pattern and, consequently, result in a significant increase in the GRFs of the human system and response of the structure. This pattern is repeated until a pedestrian crosses the structure.

Finally, Figures 5(a) and 5(d) show the overall response of the structural system to anthropometric loads, considering the effects of HSI. Figures 5(b) and 5(d) show the displacements and accelerations at the centre of the span, reaching a maximum of  $0.029 \text{ m}$  and  $3.351 \text{ m/s}^2$ , respectively, highlighting the amplification in the dynamic response of the structure owing to its resonance with the anthropic loads. These results validate the capability of the developed models to represent HSI.

#### 4. Conclusions

In this study, a vertical HSI model for pedestrian bridges was developed, which consisted of a simplified biomechanical model of human gait with seven rigid elements and a structural system represented by a simply supported beam. The primary objective of this model is to minimise the average mechanical energy required for a gait cycle to cross a pedestrian bridge. The HSI effects were effectively captured by employing a combination of inverse dynamics to calculate the kinematics and kinetics of human gait and direct dynamics to solve the differential equations governing the structural response under anthropometric loads, and the HSI effects were effectively captured.

The estimation of the body segment movements and ground reaction forces was based on three essential gait characteristics: average walking speed, frequency, and stride length. Natural body movements were allowed without imposing any artificial constraints except for physically unrealistic hyperextensions. The resulting body segment movements, as depicted in Figure 3, closely resemble normal patterns and remain within the joint limits reported in the relevant literature, such as [43]. Moreover, the minimum mechanical energy obtained in this model is lower than that reported by [39], which can be attributed to the inclusion of the HSI effects.

The utilisation of inverse dynamics, as opposed to direct dynamics, offers several advantages, including computational efficiency, which is crucial for optimisation-based predictive models. The computational cost was mainly associated with solving the differential equations governing the structural system. However, by combining the direct and inverse dynamics, the overall execution time for each optimisation iteration was significantly reduced.

The proposed model effectively simulates the kinematics and kinetics of human gait and accurately captures the dynamic response of structures subjected to HSI effects. It is

important to note that the vertical force pattern (see Figure 4) exhibits a different number of peaks compared to the existing force patterns reported in the literature. However, this discrepancy is commonly observed in models with high leg stiffness at low walking speeds, where a greater number of force peaks are generated as the walking speed decreases [50].

The simulation results of the HSI demonstrate the potential of this modelling approach to represent HSI effects on pedestrian bridges, making it a viable tool for evaluating and understanding the impact of HSI.

Comparing the simulation results between walking on a rigid surface and walking on a bridge governed by partial differential equations, notable changes in the contact forces induced by pedestrians traversing the structure were observed. Specifically, an increase of 12% in the vertical force highlights the influence of the HSI effects on the contact forces and their implications for the structural response.

#### Data Availability

The data that support the findings of this study are available from the corresponding author upon reasonable request.

#### Conflicts of Interest

The authors declare that they have no conflicts of interest.

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## Article

# Modelling Human-Structure Interaction in Pedestrian Bridges Using a Three-Dimensional Biomechanical Approach

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**Abstract:** Pedestrian bridges, which are essential in urban and rural infrastructures, are vulnerable to vibrations induced by pedestrian traffic owing to their low mass, stiffness, and damping. This paper presents a novel predictive model of Human-Structure Interaction (HSI) that integrates a three-dimensional biomechanical model of the human body, and a pedestrian bridge represented as a simply supported Euler-Bernoulli beam. Using inverse dynamics, the human model accurately captures three-dimensional gait and its interaction with structural vibrations. The results show that this approach provides precise estimates of human gait kinematics and kinetics, as well as the bridge response under pedestrian loads. The incorporation of a three-dimensional human gait model reflects the changes induced by bridge vibrations, providing a robust tool for evaluating and improving the effect of structural vibrations on the properties and gait patterns.

**Keywords:** Human-Structure Interaction; three-dimensional biomechanical model; human gait; pedestrian bridges; inverse dynamics; structural vibrations; resonance



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## 1. Introduction

Pedestrian bridges are critical structures in urban and rural infrastructure, allowing the safe transit of pedestrians over obstacles, such as rivers or vehicular roads. However, these bridges exhibit high dynamic sensitivity owing to their intrinsic characteristics such as low mass, stiffness, and damping [1]. These characteristics result in low natural frequencies, generally below 10 Hz, which can coincide with the anthropic excitation frequencies generated by pedestrian traffic [2,3]. This coincidence of frequencies is known as resonance, which generates excessive vibrations that compromise the serviceability, safety, and comfort of pedestrians [4]. The most relevant cases of this problem have been observed in Millennium and Solferino pedestrian bridges, where vibrations induced by the synchronised movement of pedestrians generated significant concerns [5,6].

Following these events, research on Human-Structure Interaction (HSI) has grown significantly, and models have been proposed to evaluate the serviceability of these structures. The most widely used method is to represent a human as a periodic force that traverses the structure at a constant velocity [7]. In this case, it is convenient to model the anthropic loads by means of Fourier Series as a function of the frequency of passage, weight of the person, and Dynamic Load Factors (DLFs), which have been defined by experimental tests [8–10]. However, studies have shown that this approach generates an overestimation of up to 4 four times in the structural response compared to the experimental tests in the resonance condition [11]. This estimation error is attributed to the assumption of periodicity in anthropic loading and the omission of pedestrian interaction effects with structural vibrations [10,11].

The Mass Spring Damper (MSD) model has been proposed as a better representation of the HSI. In this model, dynamic loads are represented by modelling the pedestrian

as a single-degree-of-freedom (SDoF) system moving with constant velocity along the structure and taking a single point of contact [12]. This type of model has been widely studied and refined by adding degrees of freedom [13–16] and subsequently calibrated with experimental tests [14,17]. Nevertheless, one of its limitations is associated with the poor accuracy of the mass, stiffness, and damping parameters assumed for pedestrian systems. Several models for predicting these parameters have been proposed [13,18]; however, the high inter- and intra-subject variability makes the reliability of the results unfeasible [7]. On the other hand, the simplification of the MSD model into an sDoF means that the study of the effect of structural vibrations on the properties and gait patterns, known as structure to human interaction (S2HI), cannot be evaluated and are ignored in other studies [19]. The second method used by the authors consists of representing human gait by means of a mechanical inverted pendulum (IP) model coupled to the structural system [20], which has been sophisticated by incorporating the lateral swing condition and MSD systems for each component limb [11,21–24]. However, the limitation of this model is associated with the large number of assumptions required for its simulation, such as the control force regulating the stability for constant running added to the initial energy input which reduces the reliability of the prediction of GRFs [7].

In contrast to traditional gait models, in recent years, the authors have developed complete biomechanical models that include more Degrees of Freedom (DoF) based on specific anthropometric characteristics [25,26] shown that a realistic representation of human gait based on anthropometric characteristics improves the prediction of GRFs [27]. Anderson and Pandy [28] developed a biomechanical model of human gait based on optimization using the direct dynamics method. However, its limitation is the high CPU processing requirement with a simulation time of approximately 10,000 h. For this reason, the inverse dynamics method has become the main option for the development of human gait models based on optimization techniques added to the progress of novel numerical algorithms and robust hardware, positioning this method as a reliable, affordable, and cost-effective alternative for the simulation of human gait [26,29].

Many authors have restricted their studies on gait prediction to models in the sagittal plane and simplified the human body into fewer segments, owing to the complexity and high computational demands [25]. Some authors have utilised three-dimensional models, such as those in [25,26], which predict certain body movements based on experimentally known ones. Subsequently, [30] limited his model to the assumption of alternating leg contact and restriction of foot rotation in the coronal plane.

Although increasingly robust biomechanical models of human gait have been developed, such as [26], research has been limited to modeling human gait on static surfaces. For that reason, Aux et al. (2024) [31] developed a predictive model of vertical HSI composed of a seven-element biomechanical model and a simplified footbridge, analyzing, on the one hand, the kinematics and kinetics of gait including HSI phenomena and on the other hand the dynamic response of the footbridge. Nevertheless, this model was limited to gait in the sagittal plane and therefore the HSI analysis was only developed in the vertical direction. Consequently, it is evident that there is still a need to develop robust three-dimensional models capable of estimating the kinematics and kinetics of human gait under the effects of S2HI that allow further evaluation of anthropic loads and their implementation in HSI studies in lateral and vertical directions, which is a topic that is little addressed in the literature.

To address these gaps, this paper proposes a novel coupled HSI predictive model composed of a complete three-dimensional biomechanical model of the human body and a pedestrian bridge represented as a simply supported Euler-Bernoulli beam. The main novelties and contributions of this work are as follows: (1) The incorporation of anthropic loads defined by a realistic three-dimensional human gait model as a source of excitation for HSI, based on inverse dynamics, and able to represent the changes in gait resulting from surface vibrations (S2HI). (2) A simplified method is proposed to evaluate the three-dimensional HSI that considers the changes in the dynamic properties of the structure under anthropic

Table 1. Cont.

| ID * | Segment Body | Distal Joint | Proximal Joint |
|------|--------------|--------------|----------------|
| 11   | Left Thigh   | Knee         | Hip            |
| 12   | Left Shank   | Ankle        | Knee           |
| 13   | Left Foot    | Rollover     | Ankle          |

\* ID is the identification Segment body in Figure 1a.

### 2.1. Kinematics

A spherical coordinate system was used to determine the positions of the body segments throughout the gait cycle. The orientation of each element was defined by the angles  $\theta_i$  and  $\varphi_i$  with respect to the global reference system, as shown in Figure 1b. The Inverse Dynamics methodology was used to determine the displacements of each joint center as a function of the angles  $\theta_i$  and  $\varphi_i$  (Equation (2)), where  $x_{ank}$ ,  $y_{ank}$ , and  $z_{ank}$  correspond to the positions of the ankle on the  $x, y$ , and  $z$  reference axes, respectively, Figure 2. However,  $L_i$  corresponds to the length of the  $i$ th element ( $i = 1, 2, 3, \dots, 13$ ), and  $I(j)$  corresponds to a sign function, which is equal to one when the body segment corresponds to the leg in contact and zero when it is in swing. By deriving Equation (1) twice, the accelerations at the joint centers are obtained (Equation (3)), where  $\dot{\theta}_j$ ,  $\dot{\varphi}_j$  and  $\ddot{\theta}_j$ ,  $\ddot{\varphi}_j$  are the angular velocities and accelerations of the  $j$ th element ( $j = 13, 12, 11, \dots, 1$ ), respectively.

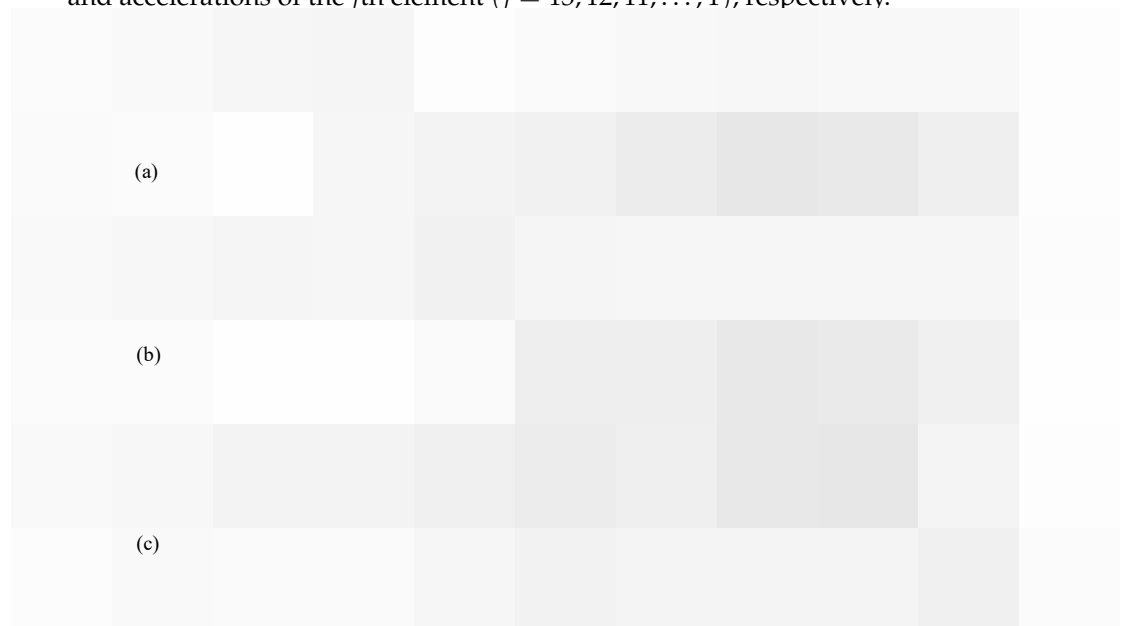


**Figure 2.** Ankle–foot kinematics during stance phase. (a) x coordinate of ankle joint, (b) z coordinate of ankle joint and (c) y coordinate of ankle joint.

$$\begin{aligned}
 x_i &= x_{ank} + \sum_{j=1}^i (I(j) L_j \cos \theta_j \cos \varphi_j) \\
 z_i &= y_{ank} + \sum_{j=1}^i (I(j) L_j \sin \theta_j \cos \varphi_j) \\
 y_i &= z_{ank} + \sum_{j=1}^i (I(j) L_j \sin \varphi_j)
 \end{aligned}
 \tag{2}$$

excitation under HSI effects. A description and formulation of the three-dimensional human gait model are presented in Section 2, followed by the proposed structural system in Section 3. Section 4 describes the simplified HSI formulation, and Section 5 presents the optimization scheme for the coupled HSI based on the minimization of the total mechanical energy of the pedestrian moving through the structure. Finally, Section 6 presents and discusses the results.

## 2. 3D Human Model

The human body was modeled as a system composed of thirteen rigid elements (body segments), connected by joints that allow free rotation in the three body planes (sagittal, frontal and coronal), Figure 1a (Table 1). A set of fifth-order Fourier series was used to represent the segmental rotations (Equation (1)), where  $\omega_w$  is the walking frequency,  $t$  is the time, and  $a_0$ ,  $a_n^{(i)}$ , and  $b_n^{(i)}$  are the coefficients associated with the series.

$$[\theta_i, \varphi_i] = a_0 + \sum_{n=1}^5 a_n^{(i)} \cos(n\omega_w t) + b_n^{(i)} \sin(n\omega_w t) \quad (1)$$

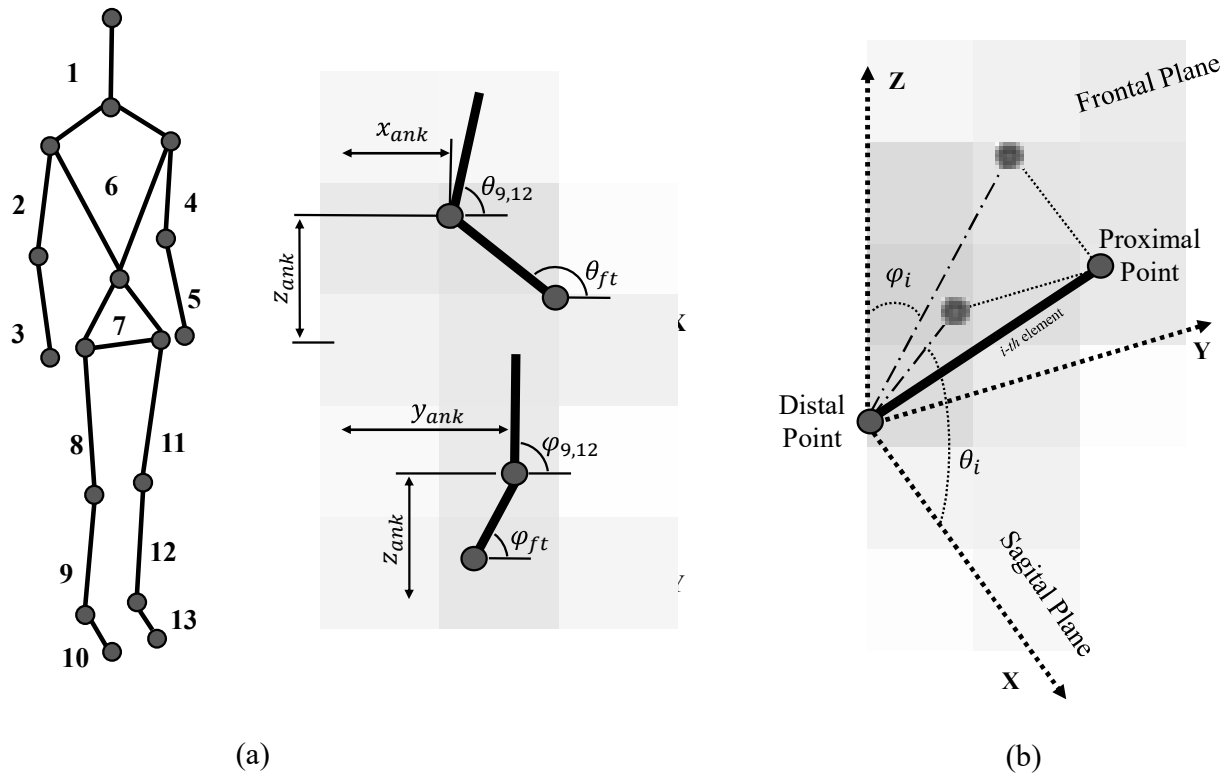


**Figure 1.** Human body description. (a) Segments body ID, (b) Spherical coordinate  $i$ th body segment.

**Table 1.** Segments body and joints included in the three human model.

| ID * | Segment Body    | Distal Joint | Proximal Joint |
|------|-----------------|--------------|----------------|
| 1    | Head            | Neck         | Skull          |
| 2    | Right Upper arm | Elbow        | Shoulder       |
| 3    | Right Forearm   | Wrist        | Elbow          |
| 4    | Left Upper arm  | Elbow        | Shoulder       |
| 5    | Left Forearm    | Wrist        | Elbow          |
| 6    | Trunk           | Lumbosacral  | Neck           |
| 7    | Pelvis          | Hip          | Lumbosacral    |
| 8    | Right Thigh     | Knee         | Hip            |
| 9    | Right Shank     | Ankle        | Knee           |
| 10   | Right Foot      | Rollover     | Ankle          |

$$\begin{aligned}
\ddot{x}_i &= \ddot{x}_{ank} + \sum_{j=1}^i \left\{ -I(j)I_j \left[ \cos\theta_j \left( \ddot{\phi}_j \sin\varphi_j + \dot{\phi}_j^2 \cos\varphi_j \right) - 2\dot{\phi}_j \sin\varphi_j \dot{\theta}_j \sin\theta_j + \cos\varphi_j \left( \ddot{\theta}_j \sin\theta_j + \dot{\theta}_j^2 \cos\theta_j \right) \right] \right\} \\
\ddot{y}_i &= \ddot{y}_{ank} + \sum_{j=1}^i \left\{ -I(j)I_j \left[ \sin\theta_j \left( \ddot{\phi}_j \sin\varphi_j + \dot{\phi}_j^2 \cos\varphi_j \right) + 2\dot{\phi}_j \sin\varphi_j \dot{\theta}_j \cos\theta_j + \cos\varphi_j \left( \ddot{\theta}_j \cos\theta_j + \dot{\theta}_j^2 \sin\theta_j \right) \right] \right\} \\
\ddot{z}_i &= \ddot{z}_{ank} + \sum_{j=1}^i \left\{ I(j)I_j \left[ \varphi_j \cos\varphi_j - \dot{\phi}_j^2 \sin\varphi_j \right] \right\}
\end{aligned} \quad (3)$$

### 2.1.1. Ankle-Foot Rollover Angle

The kinematics of the foot during gait has been well studied by several authors [29,32–34]. However, they are rather complex and limited to the sagittal plane. In that sense, for foot kinematics, ankle joint displacement functions  $(x_{ank}, z_{ank})$  defined by Ren et al. (2007) [29] are proposed (Figure 2). On the other hand, in the case of ankle kinematics in the  $y$ -direction, it was determined under the assumption that the lateral acceleration of the ankle should have a behavior similar to the total body acceleration in this direction during the contact phase. The acceleration was numerically integrated to determine the position and then approximated to a Fourier series showing the trajectory of the ankle joint center as a function of the foot Rollover angle  $\theta_{ft}$  (Figure 2c) (Table 2). The positions  $(x_i, y_i, z_i)$  and accelerations  $(\ddot{x}_i, \ddot{y}_i, \ddot{z}_i)$  of the joint centres were determined starting at the ankle  $(x_{ank}, y_{ank}, z_{ank})$  up to the  $i$ th element (Equation (2)).

**Table 2.** Fourier Series Coefficients of Ankle-foot Kinematics.

|           | $a_0$  | $a_1$ [m] | $b_1$ [m] | $a_2$ [m] | $b_2$ [m] | $a_3$ [m] | $b_3$ [m] | $\omega_{ft}$ [1/rad] |
|-----------|--------|-----------|-----------|-----------|-----------|-----------|-----------|-----------------------|
| $x_{ank}$ | −7.896 | −3.083    | 4.669     | 0.805     | 12.09     | 2.848     | −0.696    | 0.800                 |
| $z_{ank}$ | 0.206  | 0.005     | −0.012    | −0.002    | 0.090     | 0.026     | −0.006    | 1.814                 |
| $y_{ank}$ | −0.007 | 0.034     | −0.026    | 0.000     | 0.020     | −0.004    | −0.005    | 0.938                 |

### 2.2. Kinetics

The inverse dynamics method was used to calculate the joint kinetics and mechanical energy expenditure during a gait cycle based solely on body kinematics and surface vibration  $\vec{A}_g(t)$ , and the equations of motion of the  $i$ th segment of the body are presented in Equation (4), where  $m_i$  is the mass of each body segment,  $\vec{a}_i$  is the acceleration in center of mass of  $i$ th element and  $\vec{g}$  is gravity. Here, the use of the upper right arrow ( $\vec{\phantom{x}}$ ) for the variables  $\vec{A}_g(t)$ ,  $\vec{g}$  and  $\vec{a}_i$  indicates that each of these has components in the three dimensions of the  $x$ ,  $y$ , and  $z$  Cartesian planes. Specifically,  $\vec{F}_i$  contains the ground reaction forces GRFs in the lateral, vertical and longitudinal directions.

$$\sum \vec{F}_i = \sum_{i=1}^{13} m_i \left( \vec{a}_i - \vec{g} + \vec{A}_g(t) \right) \quad (4)$$

During the swing phase, only one foot is in contact with the ground, either right or left; therefore, the ground reactions (forces, moments, and Center of Pressures (COPs)) acting only on the supporting foot can be calculated directly from Equation (4). However, in the double-support phase, the ground reactions are indeterminate. To solve this problem, the linear transfer assumption  $D(t)$  proposed by [29] was used to model the transfer of forces reactions from one foot to another during the double support phase. Finally, the mechanical energy is defined as the power integral during the time taken by the pedestrian to cross the structure, as shown in Equation (5), where  $T_i$  represents the net

torque of the  $i$ th element and  $\omega_p^{(i)}, \omega_d^{(i)}$  represents the angular velocity of each joint's proximal and distal elements.

$$E_m = \int_0^L \sum_{i=1}^{13} |T_i (\omega_p^{(i)} - \omega_d^{(i)})| dt \quad (5)$$

### 3. Structural System

Walking forces are applied in the vertical and lateral directions to the surface. The frequencies of these loads are directly related to the step frequency  $f_w$ , which is approximately 2 Hz [35]. The vertical and lateral loading frequencies were 2 Hz and 1 Hz, respectively [4]. In this sense, the implemented pedestrian bridge was selected such that during the HSI evaluation, a critical resonance condition was generated in the first vibration modes in the lateral and vertical directions, which consisted of a supported standard IPE beam with a constant section of length ( $L_{br}$ ) 30m, Figure 3. The physical and mechanical properties of these structures are listed in Table 3. Three vertical and lateral vibration modes were considered, as shown in Figure 3. The frequencies ( $f_{\hat{n}}$ ) for each  $\hat{n}th$  vibration mode was calculated using the Equation (6) and presented in Table 4.

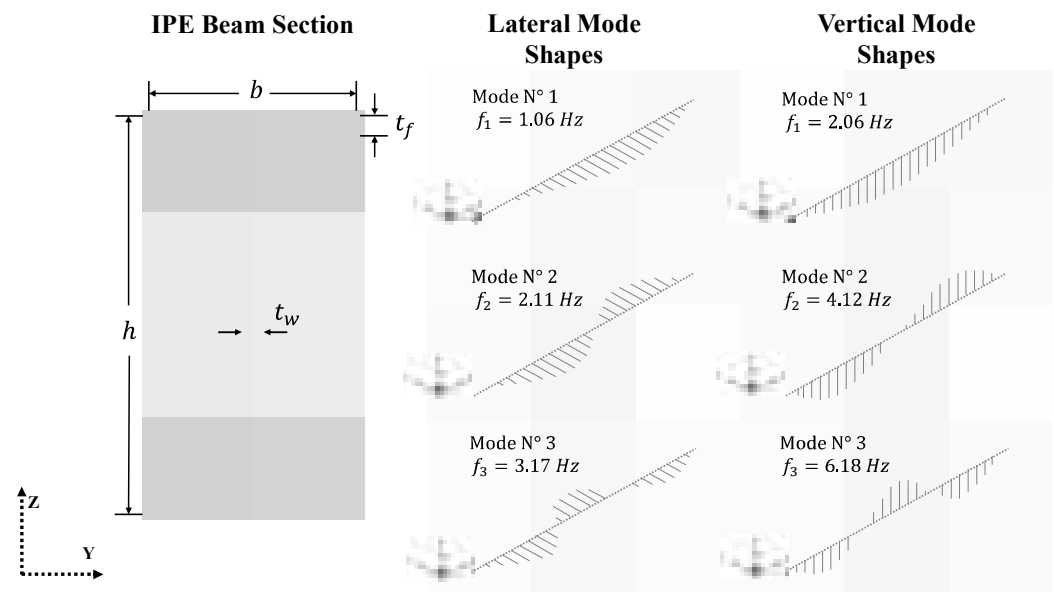


**Figure 3.** Pedestrian Bridge Structure. IPE Cross Section (left) and Mode shapes (right).

**Table 3.** Mechanical Properties of Structural System. Flange ( $b$ ), Depth ( $h$ ), Thickness of Web ( $t_w$ ), Thickness of Flange ( $t_f$ ), Length beam ( $L_{br}$ ), mass per unit length ( $\bar{m}$ ), Elasticity Modulus ( $E$ ), Inertia in local axis 1–1 ( $I_y$ ) and Inertia in local axis 2–2 ( $I_z$ ).

| Section | $b$ [m] | $h$ [m] | $t_w$ [m] | $t_f$ [m] | $L_{br}$ [m] | $\bar{m}$ [kg/m] | $E$ [GPa] | $I_y$ [cm <sup>4</sup> ] | $I_z$ [cm <sup>4</sup> ] |
|---------|---------|---------|-----------|-----------|--------------|------------------|-----------|--------------------------|--------------------------|
| IPE     | 0.3     | 0.58    | 0.01      | 0.01      | 30           | 91.06            | 200       | 633.75                   | 166.85                   |

**Table 4.** Dynamic Properties of Structural System.

|          | $f_1$ [Hz] | $f_2$ [Hz] | $f_3$ [Hz] | $\zeta$ [%] |
|----------|------------|------------|------------|-------------|
| Vertical | 2.06       | 4.12       | 6.18       | 1.00        |
| Lateral  | 1.06       | 2.11       | 3.17       | 1.00        |

$$f_{\hat{n}} = \frac{(\pi \hat{n})^2}{2\pi} \sqrt{\frac{EI}{\bar{m} L_{br}^4}} \quad (6)$$

where  $E$ ,  $I$ ,  $L_{br}$ , and  $\bar{m}$  represent the modulus of elasticity of steel, moment of inertia of the section, and length and mass per unit length of the structure, respectively. The modal shapes are described by the Equation (6) and presented in Figure 3.

$$\phi_{\hat{n}}(x) = \sin\left(\frac{\hat{n}\pi}{L_{br}}x\right) \quad (7)$$

#### 4. HSI Formulation

A fourth-order partial differential equation (PDE) describes the deflection  $u(x, t)$  at a point on the  $x$  axis of the beam at a given instant  $t$  (Equation (8)), where  $p(x, t)$  represents the load generated by a pedestrian traversing the structure,  $M_{br}$  represents the mass of the structure, and  $c$  represents the coefficient of the viscous damper.

$$\frac{\partial^2}{\partial x^2} \left[ EI(x) \frac{\partial^2 u(x, t)}{\partial x^2} \right] + m(x) \frac{\partial^2 u(x, t)}{\partial t^2} + c(x) \frac{\partial u(x, t)}{\partial t} = p(x, t) \quad (8)$$

Equation (8) was solved using the separation-of-variables method, which incorporates modal decomposition, as shown in Equation (9). In this context, the vibration modes are described by sinusoidal functions, as expressed in Equation (7). Equation (10) describes the motion ( $Y_{\hat{n}}$ ) of the  $\hat{n}th$  vibration mode ( $\phi_{\hat{n}}$ ) in modal coordinates, where the natural frequency ( $\omega_{\hat{n}} = 2\pi f_{\hat{n}}$ ) and damping ratio ( $\zeta$ ) are defined using the parameters listed in Table 4. The mass ( $M_{\hat{n}}$ ) and modal loads ( $P_{\hat{n}}$ ) were determined using Equations (11) and (12), respectively.

$$u(x, t) = \sum_{\hat{n}=1}^{\infty} \phi_{\hat{n}}(x) Y_{\hat{n}}(t) \quad (9)$$

$$\ddot{Y}_{\hat{n}}(t) + 2\zeta\omega_{\hat{n}}\dot{Y}_{\hat{n}}(t) + \omega_{\hat{n}}^2 Y_{\hat{n}}(t) = \frac{P_{\hat{n}}(t)}{M_{\hat{n}}} \quad (10)$$

$$M_{\hat{n}} = \int_0^{L_{br}} [\phi_{\hat{n}}(x)]^2 m(x) dx \quad (11)$$

$$P_{\hat{n}}(t) = \int_0^{L_{br}} \phi_{\hat{n}}(x) p(x, t) dx \quad (12)$$

Drawing from the equations that detail the kinematics and kinetics of human gait, as outlined in Section 2, the human body and the structure interact and hence are dynamically connected. This connection was established by incorporating the force generated by walking as a function of time and position in the human system, as shown in Equation (13). The transition of the left and right foot forces,  $D(t)$ , is defined as a function that is normalized to a range of 0 to 1. In single-stance zones, where only one foot is in contact with the ground, function  $D(t)$  is equal to 1 during the Begin (BGN) to Heel Contact Right (HCR) and Toe Off Left (TOL) to Heel Contact Left (HCL) periods. However, in the double-stance zones, specifically in the shaded region between the HCR to TOL and HCL to TOR,  $D(t)$  changes linearly from 0 to 1 and from 1 to 0, respectively.

$$p(x, y, t) = \left[ (1 - D(t)) \sum_{i=1}^{13} m_i \left( \vec{a}_i(t) - \vec{g} + \vec{A}_g(t) \right) \right] \delta(x - x_L(t)) + \left[ D(t) \sum_{i=1}^{13} m_i \left( \vec{a}_i(t) - \vec{g} + \vec{A}_g(t) \right) \right] \delta(x - x_R(t)) \quad (13)$$

$$\begin{aligned} x_L(t) &= x'_L(t) + CoP_l(t) \\ x_R(t) &= x'_R(t) + CoP_r(t) \end{aligned} \quad (14)$$

In Equation (13), the left and right feet are represented by  $x'_L(t)$  and  $x'_R(t)$ , respectively, and the pressure centers for the left and right feet are represented by  $CoP_l(t)$  and  $CoP_r(t)$ , respectively, in Equation (14). The accelerations induced by the structure owing to the human gait are represented by Equation (15), where  $\vec{A}_g(t)$  is calculated based on the positions of the feet and the vertical modes of vibration. The applied force of human

walking in modal coordinates was obtained by combining Equations (12) and (13), resulting the Equation (16).

$$A_g(t) = \sum_{\hat{n}=1}^3 \left[ (1 - D(t)) \sin\left(\frac{\hat{n}\pi x_L(t)}{L_{br}}\right) + D(t) \sin\left(\frac{\hat{n}\pi x_R(t)}{L_{br}}\right) \right] \ddot{Y}_{\hat{n}}(t) \quad (15)$$

$$P_{\hat{n}}(t) = - \left[ F_L(t) \sin\left(\frac{\hat{n}\pi x_L(t)}{L_{br}}\right) + F_R(t) \sin\left(\frac{\hat{n}\pi x_R(t)}{L_{br}}\right) \right] - \left[ (1 - D(t)) m_p \sin\left(\frac{\hat{n}\pi x_L(t)}{L_{br}}\right) + D(t) m_p \sin\left(\frac{\hat{n}\pi x_R(t)}{L_{br}}\right) \right] \ddot{A}_g(t) \quad (16)$$

$$\ddot{Y}_{\hat{n}}(t) + \frac{m_p}{M_{\hat{n}}} S_{\hat{n}}(t) \left[ \sum_{m=1}^3 S_m(t) \ddot{Y}_{\hat{n}}(t) \right] + 2\zeta_{\hat{n}} \omega_{\hat{n}} \dot{Y}_{\hat{n}}(t) + \omega_{\hat{n}}^2 Y_{\hat{n}}(t) = - \frac{1}{M_{\hat{n}}} \left[ F_L(t) \sin\left(\frac{\hat{n}\pi x_L(t)}{L_{br}}\right) + F_R(t) \sin\left(\frac{\hat{n}\pi x_R(t)}{L_{br}}\right) \right] \quad (17)$$

where,

$$S_{\hat{n}}(t) = (1 - D(t)) \sin\left(\frac{\hat{n}\pi x_L(t)}{L_{br}}\right) + D(t) \sin\left(\frac{\hat{n}\pi x_R(t)}{L_{br}}\right)$$

$$F_L(t) = (1 - D(t)) \sum_{i=1}^{13} m_i (a_i(t) - g)$$

$$F_R(t) = D(t) \sum_{i=1}^{13} m_i (a_i(t) - g)$$

Here,  $F_L(t)$  and  $F_R(t)$  represent the forces generated by walking on the left and right feet, respectively, without considering the influence of acceleration on the structure. The total mass of the human system is denoted as  $m_p = \sum_{i=1}^{13} m_i$ . By substituting Equation (16) into Equation (10), the modal coordinate equation for solving the HSI model, given by Equation (17), can be expressed in matrix form, as shown in Equation (18). In this equation,  $M_{br}$  represents the modal mass,  $C$  represents damping, and  $K$  represents the stiffness of the human structural system.

$$\left[ M_{br} + m_p S^T(t) S(t) \right] \ddot{Y}(t) + C \dot{Y}(t) + K Y(t) = -\Phi(x_L(t)) F_L(t) - \Phi(x_R(t)) F_R(t) \quad (18)$$

## 5. Optimization Scheme

In the HSI optimization and simulation process, initial gait characteristics such as the average walking speed  $V_w$ , walking frequency  $f_w$ , and double support duration. The interior-point optimization algorithm with the Matlab R2024a *fmincon* function were implemented on a computer with the following characteristics: 64-bit Windows 11 Pro operating system version 22H2, Intel Core i9 13900 K processor @3.0 GHz and 128.0 GB RAM (Dell Technologies, Cali, Colombia). The objective function is used to find the coefficients ( $a_0, a_n^{(i)}$ , and  $b_n^{(i)}$ ) that describe the trajectories of the body segments that achieve specified gait parameters, while minimizing the mechanical energy expenditure ( $E_m$ ) and satisfying the constraints presented in Section 5.1.

### 5.1. Constrains

Linear constraints are associated with defining joint limits, hyper-extensions, and unrealistic movements for a normal gait, Table 5. The angular range of motion is defined in Equation (19), where the limits  $\theta_{min}^{(i)}$ ,  $\theta_{max}^{(i)}$ ,  $\varphi_{min}^{(i)}$ , and  $\varphi_{max}^{(i)}$  are defined as 70% of the human body maxima defined in [36]. Similarly, the relative angles for the knee and elbow joints centers are presented in Equation (20).

$$\theta_{min}^{(i)} \leq \theta_i(t) \leq \theta_{max}^{(i)} \text{ and } \varphi_{min}^{(i)} \leq \varphi_i(t) \leq \varphi_{max}^{(i)}, \quad t \in [0, T_w] (i = 1, 2, \dots, 13) \quad (19)$$

$$\theta_{min}^{(1)} \leq \theta_2(t) - \theta_3(t) \leq \theta_{max}^{(1)}, \theta_{min}^{(2)} \leq \theta_9(t) - \theta_8(t) \leq \theta_{max}^{(2)}, \quad t \in [0, T_w] \quad (20)$$

**Table 5.** Limit range of absolute angular displacements at joints.

| Joint       | Flexion                       |                               | Abduction                      |                                |
|-------------|-------------------------------|-------------------------------|--------------------------------|--------------------------------|
|             | $\theta_{min}^{(i)} [^\circ]$ | $\theta_{max}^{(i)} [^\circ]$ | $\varphi_{min}^{(i)} [^\circ]$ | $\varphi_{max}^{(i)} [^\circ]$ |
| Neck        | 60                            | 120                           | 50                             | 130                            |
| Elbow       | 45                            | 135                           | 50                             | 180                            |
| Wrist       | 45                            | 135                           | 50                             | 180                            |
| Lumbosacral | 60                            | 120                           | 75                             | 105                            |
| Hip         | 60                            | 120                           | 75                             | 105                            |
| Knee        | 30                            | 180                           | 75                             | 105                            |
| Ankle       | 30                            | 135                           | 40                             | 120                            |
| Metatarsal  | 45                            | 200                           | 65                             | 125                            |
| Elbow *     | 35                            | 180                           | -                              | -                              |
| Knee *      | 55                            | 180                           | -                              | -                              |

\* Relative angular displacement at joints.

For the nonlinear constraints it is established that the foot contact ( $y_{tip}(t)$ ) when there is contact with the ground must be equal to zero ( $y_{tip}(t) = 0$ ). In contrast, during the swing phase, the contact must be greater than zero ( $y_{tip}(t) > 0$ ). The second nonlinear constraint states that vertical force  $F_z(t)$  is always greater than zero ( $F_z(t) > 0$ ). The third constraint defines the relationship based on the coefficient of friction ( $\mu$ ), as shown in Equation (21). Finally, the last constraint defines the pitch length as  $x_{ank}(T_w) - x_{ank}(0) = V_w T_w$ , where  $T_w = 2\pi/\omega_w$  is the walking period,  $x_{ank}$  is the position of the ankle in the  $x$  coordinate, and  $V_w$  is the walking speed.

$$\frac{\sqrt{F_x^2 + F_y^2}}{F_z} < \mu \quad (21)$$

## 5.2. Simulation

Human gait is considered as a process of minimum mechanical energy. In this regard, researchers such as Ren et. al. (2007) [29] determined normal gait patterns by optimization, starting from an upright position without walking. This approach was implemented in this study, for which the coefficients  $X_0$  correspond to the result of an optimization in which the human starts from a static position.

Figure 4 presents the optimization scheme implemented in the HSI optimization, which starts by defining anthropic parameters and gait conditions. Then, the initial coefficients  $X_0$  ( $a_0^{(i)}$ ,  $a_n^{(i)}$  and  $b_n^{(i)}$ ) used as the starting point of the optimization are defined. These initial coefficients are adjusted in successive iterations ( $X$ ) by the optimization function to reduce the mechanical energy expenditure (**Step 1**).

In **Step 2**, the  $X$  coefficients are evaluated using Equation (1), which determines the angular movements of each body segment ( $\theta_i$  and  $\varphi_i$ ). In **Step 2.1**, the Ankle-Foot Rollover was calculated using the process detailed in Section 2.1.1. The kinematics of the human body are then calculated using the methodology presented in Section 2.1, obtaining the displacements ( $x_i$ ,  $y_i$ ,  $z_i$ ) and accelerations ( $\ddot{x}_i$ ,  $\ddot{y}_i$ ,  $\ddot{z}_i$ ) for each body segment.

**Step 3** involves the calculation of gait kinetics starting from the previously calculated kinematics and using the inverse dynamics methodology, as described in Section 2.2, to get the ground reaction forces GRFs ( $F_x$ ,  $F_y$  and  $F_z$ ). After, in **Step 4**, the bridge properties are defined and used to calculate the structural system (**Step 4.1**) as described in Section 3. Subsequently, taking as input (1) the structural system, (2) the kinematics (contact position), and (3) the kinetics (GRFs) of the pedestrian, the dynamic response of the coupled HSI structural system is calculated following the methodology presented in Section 4.

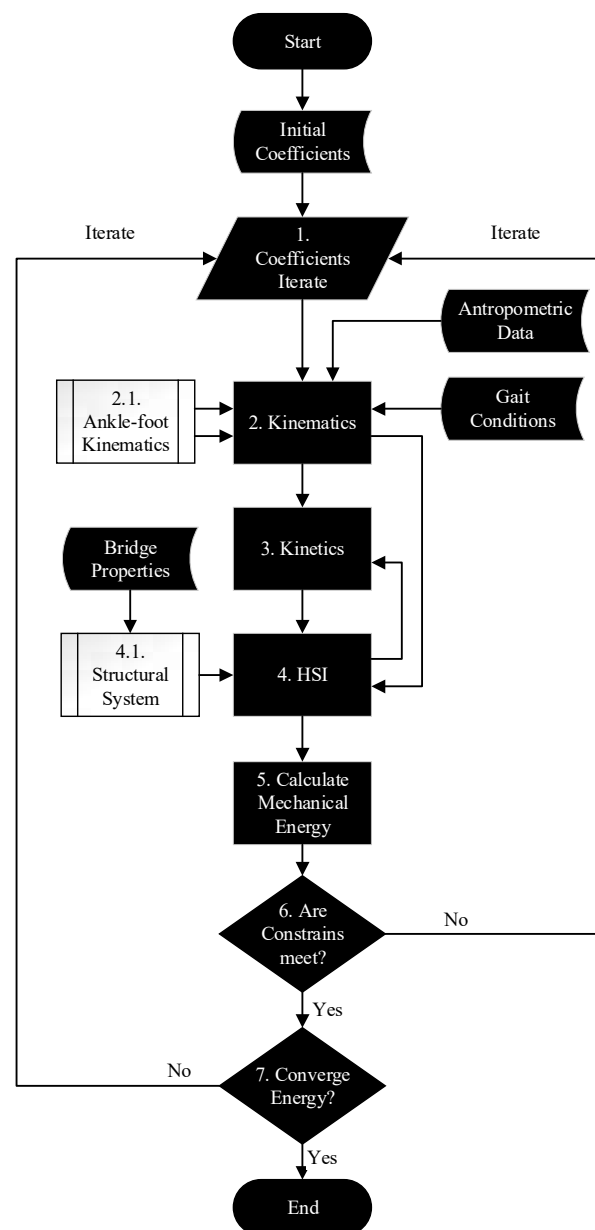


**Figure 4.** Optimization Scheme.

The mechanical energy during the entire time interval was determined and checked in **Step 5** (Equation (5)) to determine if the predefined constraints were met. If the constraints are not met, the process is repeated by changing coefficient  $X$  (**Step 6**). However, if the constraints are met, the process proceeds to verify if the solution converges, and if convergence has not been achieved, the process iterates again from the definition of the  $X_0$  coefficients, adjusting them as necessary, and repeating the entire analysis. Finally, in **Step 7**, the iterative cycle continues until the constraints are met and convergence is achieved, at which point the optimization process is terminated, thus achieving an optimal solution for the system under analysis.

## 6. Results and Discussion

### 6.1. Single Pedestrian HSI Analysis

An 80 kg male subject walking along the reference structure with the anthropometric characteristics described in Table 6 was implemented for all simulation cases. The subject moves with a step length of 1.5 m at step frequency  $f_w = 2$  Hz and double support duration between 33 and 50% for each gait cycle.

**Table 6.** Anthropometric data. Adapted from [37].

|           | Mass [%] | Length [mm] | * CoM [%] | Ratio Sagittal | Ratio Transverse | Ratio Frontal |
|-----------|----------|-------------|-----------|----------------|------------------|---------------|
| Head      | 6.94     | 203.3       | 50.02     | 30.3           | 26.1             | 31.5          |
| Upper arm | 2.71     | 281.7       | 57.72     | 28.5           | 15.8             | 26.9          |
| Forearm   | 1.62     | 268.9       | 67.51     | 43.88          | 12.31            | 42.93         |
| Trunk     | 32.29    | 386.2       | 49.85     | 33.74          | 23.4             | 29.1          |
| Hip       | 11.17    | 145.7       | 38.85     | 61.5           | 58.7             | 55.1          |
| Thigh     | 14.16    | 422.2       | 40.95     | 32.9           | 14.9             | 32.9          |
| Shank     | 4.33     | 434         | 43.95     | 25.1           | 10.2             | 24.6          |
| Foot      | 1.37     | 190         | 44.15     | 25.7           | 24.5             | 12.4          |

\* CoM is the Center of Mass of the element from distal to proximal point.

The first simulation step was to determine a normal gait pattern without considering ground vibration (rigid surface), that is,  $A_g(t) = 0$ . Here, the coefficients found were implemented as the starting point  $X_0$  in the model of this study. An HSI simulation was performed considering a single pedestrian, for which, after 67 iterations and 712 s of optimization, the model found the minimum at  $E_m = 196.06$  J. The coefficients  $a_n$  and  $b_n$  solutions of the optimization problem are presented in Table 7. The coefficients represent the adjusted parameters of the biomechanical model for each body segment during gait on the structure considering the HSI effects. These coefficients, derived from the Fourier series in Equation (1), describe the segmental rotations throughout the gait cycles and was used to calculate the forces on each joint using inverse dynamics.

**Table 7.** Optimized Coefficients for HSI.

| Element   | $a_0$  | $a_1$  | $b_1$  | $a_2$  | $b_2$  | $a_3$  | $b_3$  | $a_4$  | $b_4$  | $a_5$  | $b_5$  |
|-----------|--------|--------|--------|--------|--------|--------|--------|--------|--------|--------|--------|
| Head      | 1.503  | −0.022 | −0.069 | −0.005 | 0.022  | −0.001 | 0.011  | −0.063 | 0.005  | 0.017  | 0.002  |
| Upper arm | 1.709  | 0.196  | −0.058 | 0.036  | 0.029  | −0.002 | −0.265 | −0.027 | −0.013 | 0.001  | 0.018  |
| Forearm   | 1.81   | 0.195  | −0.058 | 0.036  | 0.029  | −0.002 | −0.264 | −0.027 | −0.013 | 0.001  | 0.018  |
| Trunk     | 1.492  | −0.022 | −0.069 | −0.005 | 0.022  | −0.001 | 0.011  | −0.063 | 0.005  | 0.017  | 0.002  |
| Hip       | 1.513  | 0.005  | −0.048 | 0.007  | 0.03   | 0.002  | −0.014 | −0.08  | −0.003 | 0.008  | −0.002 |
| Thigh     | 1.727  | 0.223  | −0.058 | 0.016  | −0.008 | 0.002  | −0.234 | 0.013  | 0.005  | −0.004 | −0.002 |
| Shank     | 1.282  | 0.381  | −0.108 | −0.04  | 0.027  | −0.004 | −0.232 | −0.082 | 0.069  | −0.006 | −0.005 |
| Foot      | 2.478  | 0.59   | −0.273 | 0.041  | 0.007  | −0.007 | −0.228 | −0.128 | 0.117  | −0.038 | 0.007  |
| Head      | 1.275  | 0.01   | 0.007  | 0      | 0.000  | 0.000  | −0.026 | −0.012 | 0.006  | 0.002  | 0.000  |
| Upper arm | −4.353 | 0.03   | 0.008  | 0.003  | −0.001 | −0.001 | −0.005 | −0.012 | 0.001  | 0.001  | −0.001 |
| Forearm   | −4.353 | 0.032  | 0.008  | 0.003  | −0.001 | −0.001 | −0.005 | −0.012 | 0.002  | 0.001  | 0.001  |
| Trunk     | 0.069  | 0.327  | 0.061  | 0.076  | 0.038  | −0.004 | −0.924 | −0.001 | 0.091  | 0.019  | 0.04   |
| Hip       | 0.074  | 0.333  | 0.096  | 0.098  | 0.041  | −0.008 | −0.994 | −0.043 | 0.098  | 0.034  | 0.045  |
| Thigh     | 1.738  | −0.001 | −0.016 | 0.005  | −0.025 | −0.003 | 0.084  | 0.037  | −0.016 | −0.011 | 0.001  |
| Shank     | 1.675  | 0.016  | −0.026 | 0.008  | −0.022 | −0.003 | 0.043  | 0.029  | −0.012 | −0.017 | −0.002 |
| Foot      | −0.278 | 0.011  | −0.026 | 0.01   | −0.023 | −0.004 | 0.036  | 0.035  | −0.016 | −0.016 | −0.002 |

The ground reaction forces (GRFs) in the vertical and lateral directions as the pedestrian traverses the bridge are shown in Figures 5 and 6, respectively. The percentage differences for the GRFs, comparing the HSI with respect to those on a rigid surface, were calculated as the Root Mean Square Error (RMSE) over 10% intervals of the bridge span, as shown in Figure 7. As expected, the highest RMSE are found around the mid-span of the bridge, where the accelerations  $\vec{A}_g(t)$  are greater by 63.7% in the vertical direction and 23.42% in the lateral direction, with respect to the GRFs on rigid surfaces.

The movements of the body segments are presented in Figures 8 and 9 and display typical patterns as reported in investigations [36]. The percentage differences for the angles corresponding to HSI with respect to those corresponding to the rigid surface were calculated and are very small when the pedestrian is near the bridge ends. Hence, the RMSE was calculated over the 40–60% span of the bridge, where the accelerations  $\vec{A}_g(t)$

are influential, and are given in Table 8. In the case of angular movements in the frontal plane (Figure 9), the differences between the HSI and rigid surface are much greater than the differences in the sagittal plane. We believe that this is due to the lock-in phenomenon, for which the human model increases its lateral sway [24].



**Figure 5.** Vertical Ground Reaction Forces, HSI (Green solid line) and rigid surface (Black dotted line). (a) Forces over the entirety of the bridge, (b) Forces corresponding to four strides at mid-span.

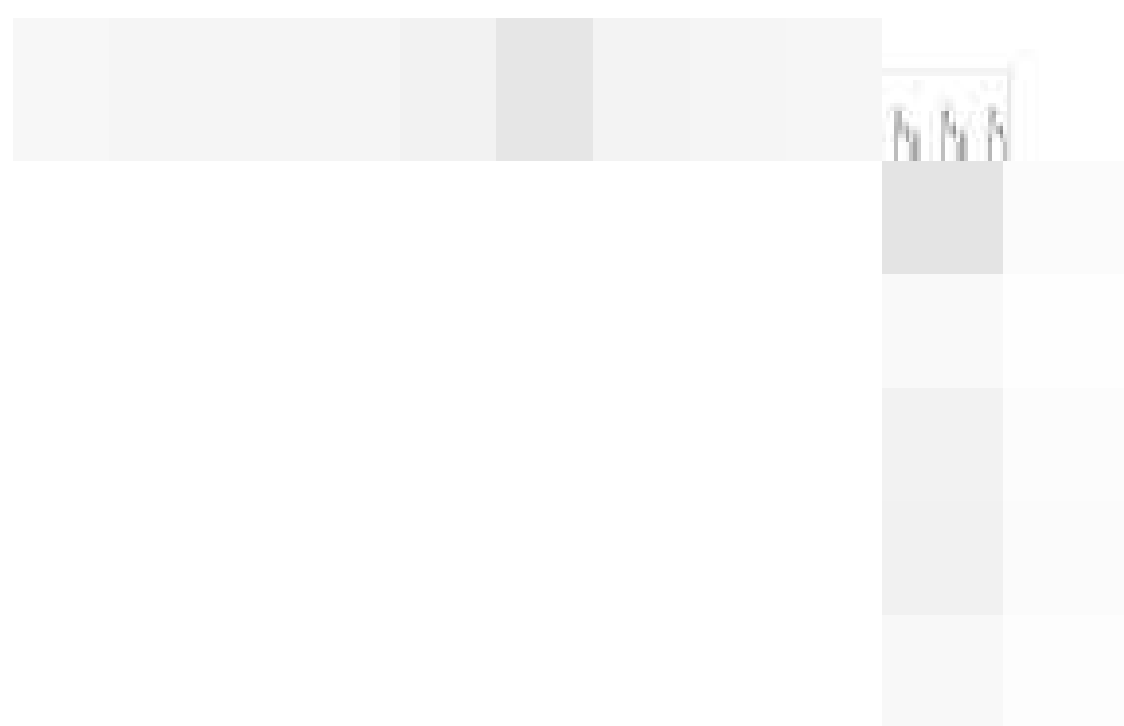


**Figure 6.** Lateral Ground Reaction Forces, HSI (Green solid line) and rigid surface (Black dotted line). (a) Forces over the entirety of the bridge, (b) Forces corresponding to four strides at mid-span.

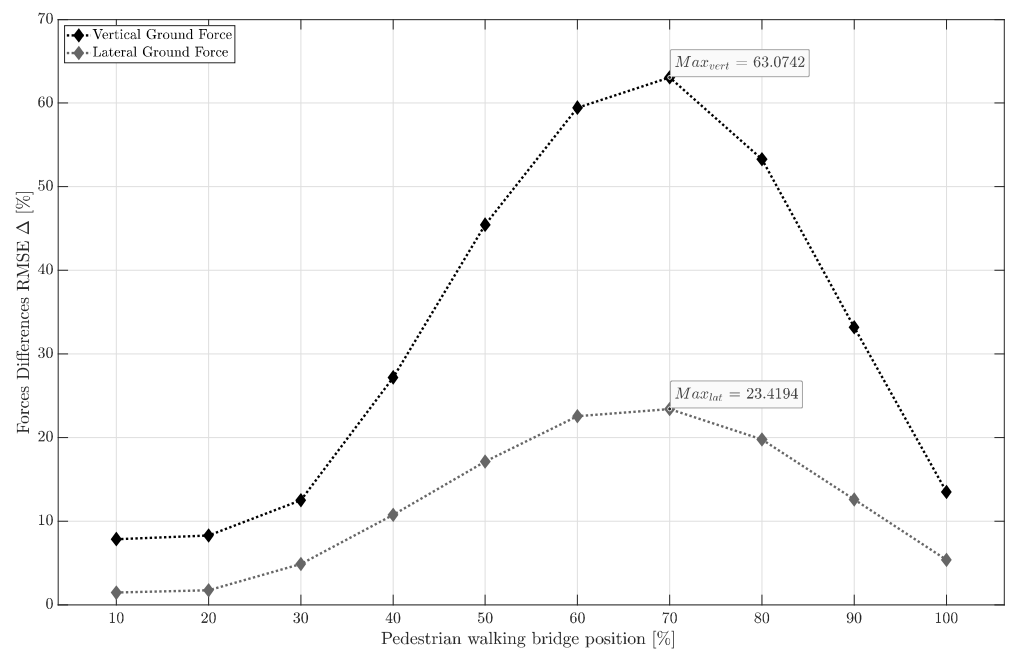


**Figure 7.** Forces Differences (RMSE) between HSI and Rigid Surface.

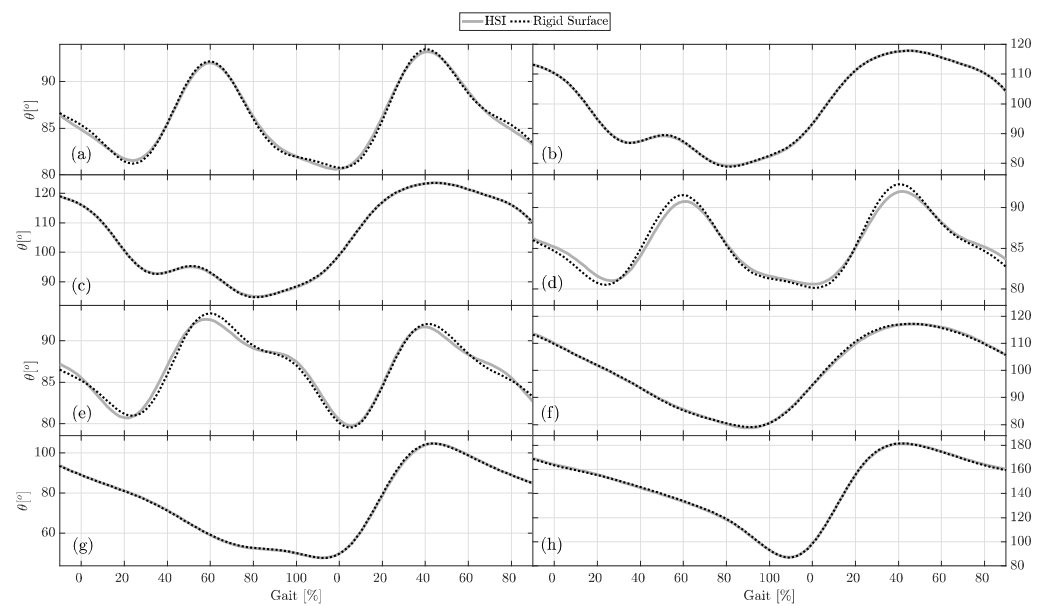


**Figure 8.** Sagittal Plane Angular Displacements at Mid-Span. HSI (Green solid line) and rigid surface (Black dotted line). (a) Head, (b) Upperarm, (c) Forearm, (d) Trunk, (e) Hip, (f) Thigh, (g) Shank and (h) Foot.

**Table 8.** Differences angular displacements during human gait between 9.5 and 10.5 s.

|                      | Head  | Upper Arm | Forearm | Trunk | Hip   | Thigh  | Shank | Foot  |
|----------------------|-------|-----------|---------|-------|-------|--------|-------|-------|
| RMSE $\theta_i$ [%]  | 6.27  | 0.99      | 0.97    | 14.37 | 12.74 | 2.58   | 1.32  | 1.48  |
| RMSE $\varphi_i$ [%] | 24.93 | 36.87     | 37.09   | 3.13  | 4.18  | 129.19 | 54.64 | 50.51 |

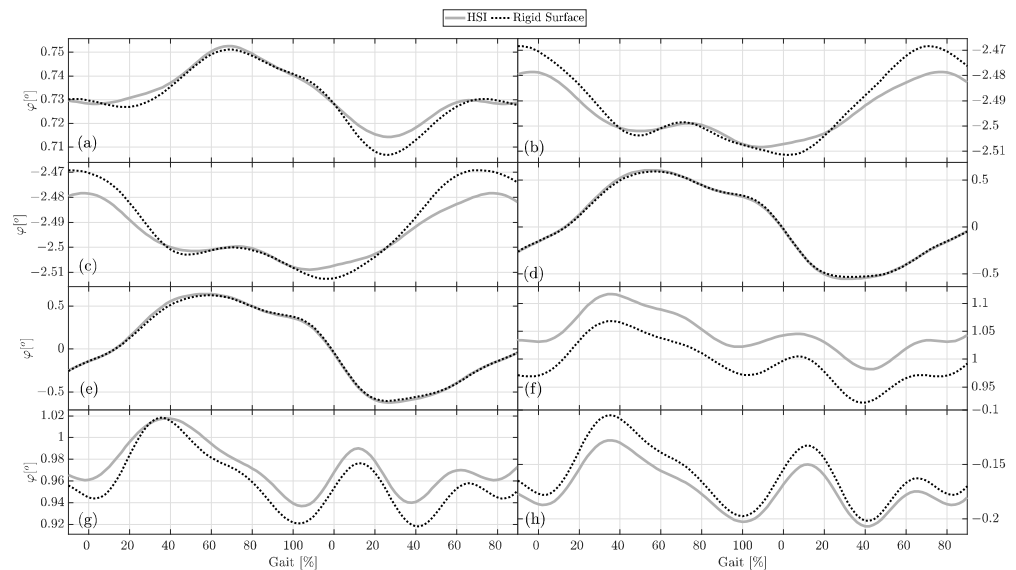


**Figure 9.** Frontal Plane Angular Displacements at Mid-Span. HSI (Green solid line) and rigid surface (Black dotted line). (a) Head, (b) Upperarm, (c) Forearm, (d) Trunk, (e) Hip, (f) Thigh, (g) Shank and (h) Foot.

The differences in the angular displacements presented in Table 8 indicate significant variations in the gait kinematics owing to the bridge vibrations. Specifically, the RMSE  $\varphi_i$  differences are greater in the lower and upper extremities (Figure 9f–h). It was observed that the abduction motion of the lower limbs (Thigh, Shank, and foot) and arms (upper and forearm) increased, that is, the lateral body sway was increased to ensure stability. This is consistent with observations made by Brady et. al. (2009) [38] in human experimental trials on an oscillating treadmill, where they recorded that the lateral step width generally increased as a consequence of vibration [39].

Figure 10 shows the instantaneous natural frequency and damping ratio of the coupled system during pedestrian transit over the structure. It can be seen that, as suggested by several studies, the effect of anthropic loads generates a change in the dynamic properties of the coupled system [40], decreasing the modal frequency and increasing the damping of the pedestrian structure system under the effects of HSI. The instantaneous frequency decreases as the pedestrian advances along the structure, reaching a minimum value when the pedestrian is at mid-span of the bridge, with a difference of 1.98% and 2.42% for the vertical and lateral directions, respectively. Similarly, the damping increases by 3.5% in the vertical direction and by 1.93% in the lateral direction.

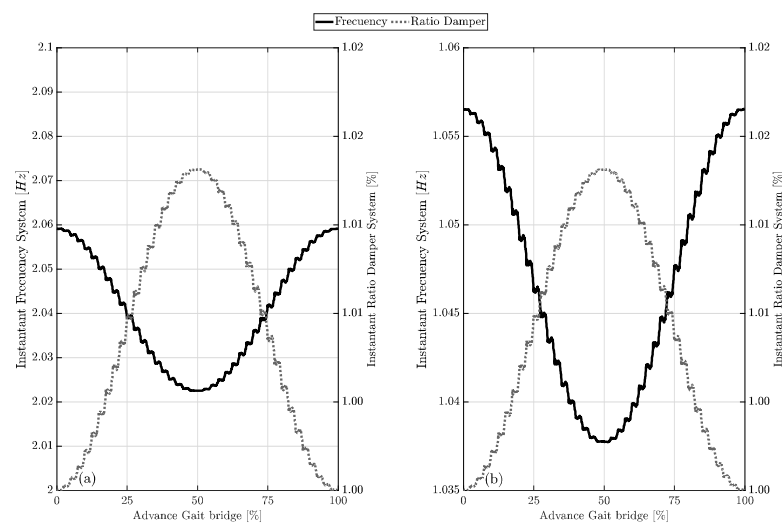


**Figure 10.** Instantaneous Natural Frequency and Damping Ratio of Coupled System. (a) First vertical mode, (b) First lateral mode.

The dynamic responses of the reference structure are presented in Figures 11–13. In addition to the HSI model proposed in this study, an analysis using a model omitting the effect of vibrations on the GRFs (without HSI) was considered. Figure 11 shows the bridge acceleration  $A_g(t)$  at contact point in the vertical and lateral directions. The displacements  $D_c(t)$  and acceleration  $A_c(t)$  in the bridge mid-span are shown in Figures 12 and 13, respectively. The peak values of the structural responses are presented in Table 9, where the lower peak values were recorded for the model without HSI in the analysed cases.

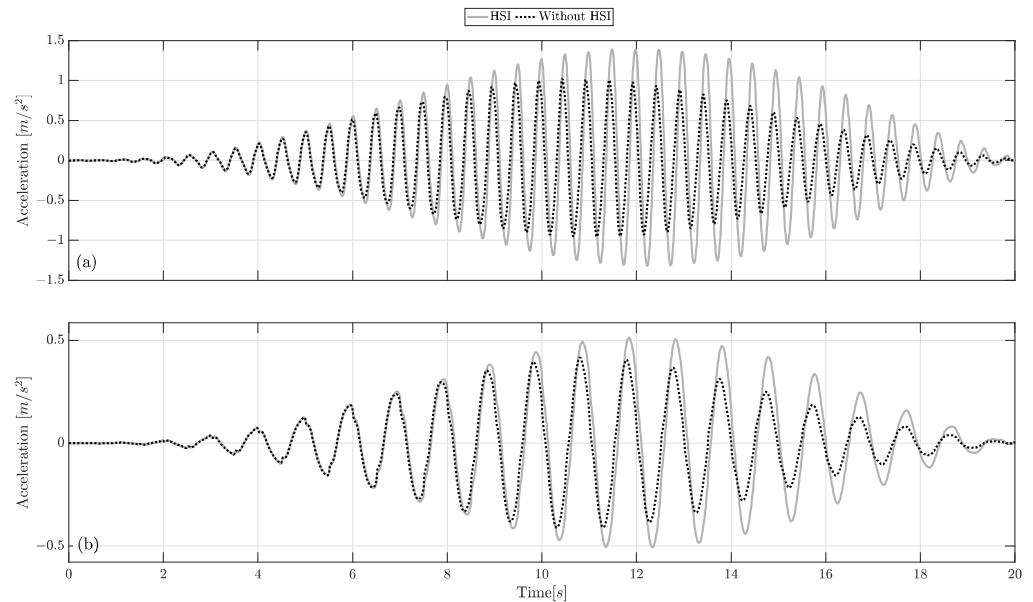


**Figure 11.** Acceleration  $A_g(t)$  at Point of Contact. (a) Vertical direction. (b) Lateral direction.

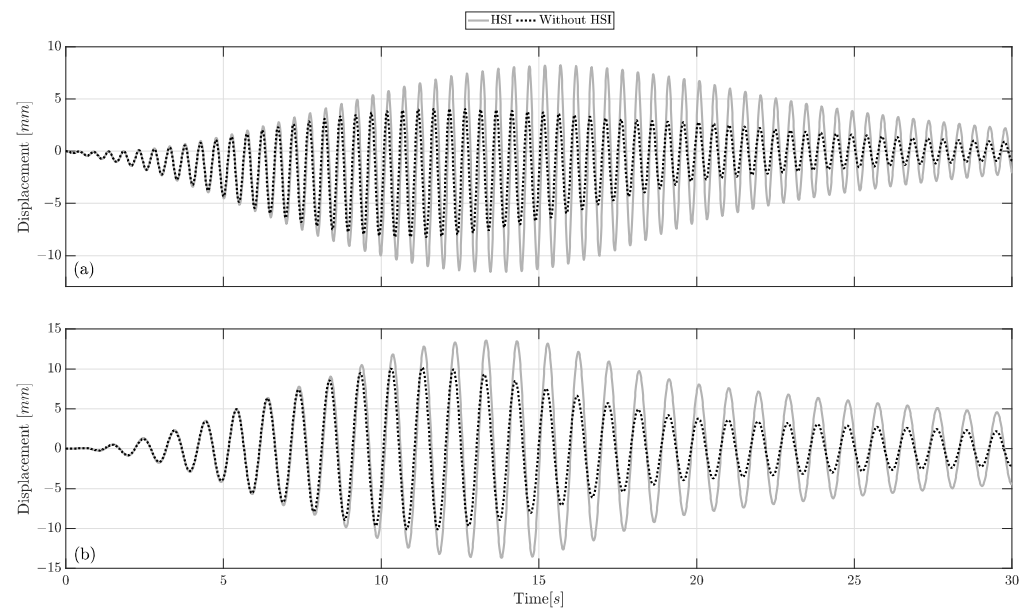


**Figure 12.** Displacement at Mid-Span  $D_c(t)$ . (a) Vertical direction. (b) Lateral direction.

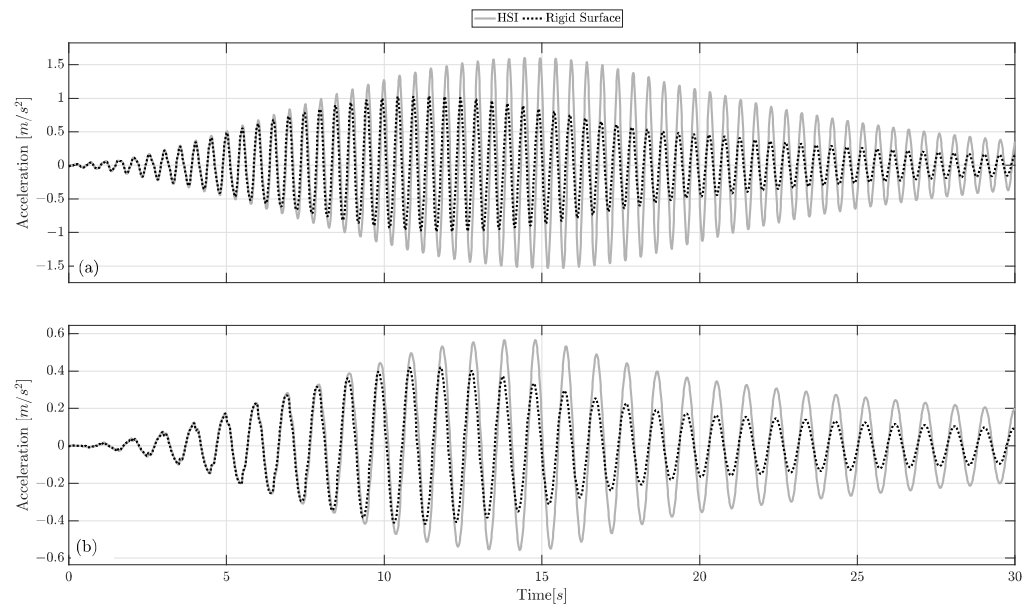


**Figure 13.** Acceleration at Mid-Span  $A_c(t)$ . (a) Vertical direction. (b) Lateral direction.

**Table 9.** Peak Response During Gait, With HSI and Without HSI.

|               | Vertical |                  |                     | Lateral  |                  |                    |
|---------------|----------|------------------|---------------------|----------|------------------|--------------------|
|               | Max. HSI | Max. without HSI | $\Delta_{Vert}[\%]$ | Max. HSI | Max. without HSI | $\Delta_{Lat}[\%]$ |
| $A_g [m/s^2]$ | 1.39     | 1.02             | 26.41               | 0.51     | 0.42             | 19.08              |
| $A_c [m/s^2]$ | 1.60     | 1.03             | 35.59               | 0.57     | 0.42             | 25.79              |
| $D_c [mm]$    | 11.57    | 8.22             | 28.94               | 13.69    | 10.17            | 25.69              |

## 6.2. Crowd HSI Analysis

In crowd HSI analysis it is commonly accepted that the anthropic pedestrian loading  $F_{crowd}$  in the human walking frequency range (1.8–2.2 Hz), following the model of Matsumoto et al. (1978) [35] is equivalent to  $\sqrt{N}F_p$  due to the small probability that if there are  $N$  pedestrians on the walkway, all of them are synchronized [41]. However, for the purpose of this study, an HSI analysis of the crowd was considered in which pedestrians are fully synchronized, defined as the worst-case loading scenario, where all pedestrians would march at the same time, applying a constant periodic load with a frequency equal to that of the bridge, for which  $F_{crowd} = NF_p$ .

Multiple simulations were performed using the methodology described in Section 5 considering different occupancy conditions (Mass Relation,  $M_{crowd}/M_{br}$ ). The peak accelerations and displacements for each occupancy case, considering the HSI model and the model without interaction, are presented in Figure 14. The results suggest that there is an overestimation of the dynamic response of the structure in the model without interaction with respect to that of the HSI model for  $M_{crowd}/M_{br}$  greater than 15%. However, for the same conditions, considering low occupancy rates, the model without interaction underestimated the response of the structure. This phenomenon is mainly due to the crowd-structure frequency detuning ( $\omega_{crowd}/\omega_{br}$ ), for which the frequency of the coupled HSI system decreases as the number of pedestrians increases, as shown in Figure 14 (top left).

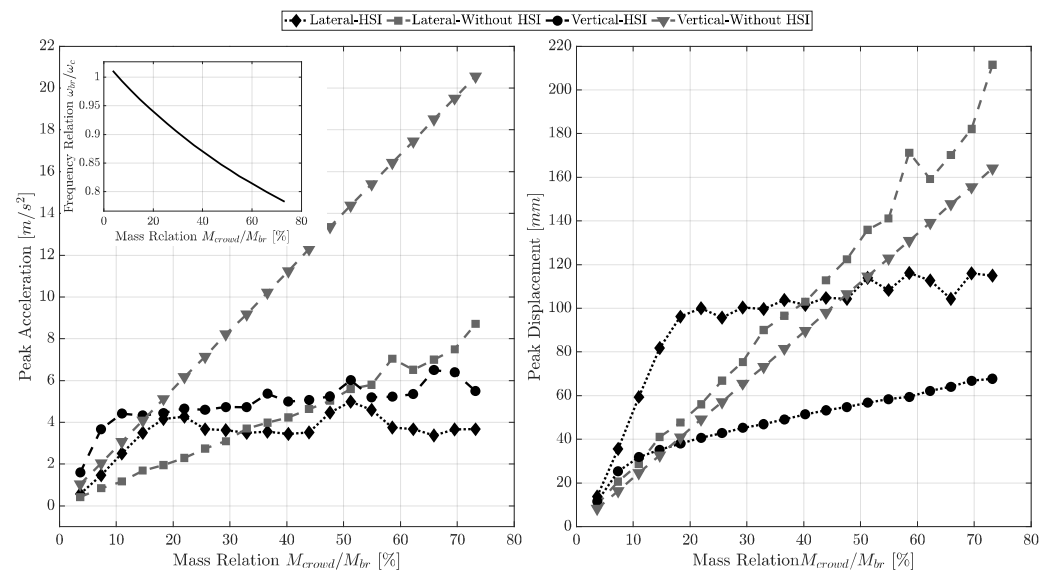


**Figure 14.** Peak Accelerations and Displacements at Mid-Span for Different Occupancy Levels.

The instantaneous natural frequency of the HSI system with the pedestrian at mid-span decreases less than 30% for occupancy condition levels  $M_{br}/M_w$  up to 75.0%, compared to those of the system Without dynamic HSI effects (Figure 14), for vertical and lateral directions. This is especially relevant as it suggests that pedestrian bridges with fundamental natural frequencies greater than walking frequencies, which in principle should not have excessive vibration problems, could display amplification of the dynamic response due the reduction in the natural frequency and variation in the structural damping of the coupled system, compromising the serviceability of these structures.

To reduce the effect of de-synchronization, simulations were performed by adjusting the step frequency  $\omega_w$  so that the ratio of frequencies  $\omega_{crowd}/\omega_{br}$  was approximately one. The simulation results are shown in Figure 15. In this case, it is observed that the peak values of acceleration and displacement are always greater than those of the model without HSI for all occupancy levels. This suggests that the traditional model underestimates the dynamic response of a structure under periodic anthropic loads.

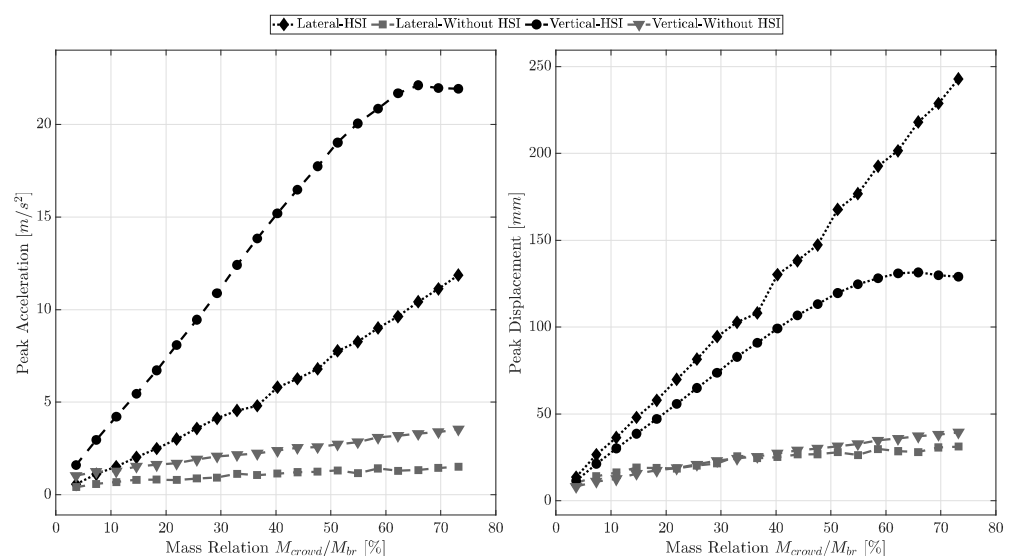


**Figure 15.** Peak Accelerations and Displacements at Mid-Span for Different Occupancy Levels. (Pace Frequency equal to Bridge Natural Frequency).

## 7. Conclusions

A three-dimensional model of the HSI was developed that consists of a whole-body biomechanical model of human gait and a structural system represented as a simple supported beam. The proposed model was used in an optimization algorithm that minimizes the mechanical energy expenditure required by a pedestrian to walk across the structure. The structural response, considering HSI, was implemented by combining inverse dynamics to predict the kinematics and kinetics of human gait and direct dynamics to solve the differential equations of the coupled system.

Multibody models such as that of Salehi et al. (2016) [26] demonstrate that predictive gait models using inverse dynamics have up to 8 times faster execution speed compared to commercial multibody models using direct dynamics. For the model presented in this research, the primary factor contributing to the computational expense is the solution of the differential equations governing the structural system and HSI. Nevertheless, the integration of direct and inverse dynamics efficiently reduces the overall time required for each optimization iteration. The computational efficiency of the model was evidenced in the simulation times obtained during the study. Each gait pattern was simulated in approximately 12 s on the computer described in Section 5, demonstrating the feasibility of the model for practical analysis compared to other approaches that require significantly longer processing times [28].

The predictions of body movements and ground reaction forces considering HSI were based solely on three simple gait characteristics: average walking speed ( $V_w$ ), frequency ( $f_w$ ), and step length, as well as the anthropometric data of the person and physical characteristics of the structure. It should be noted that none of the human body movements were predefined for a specific motion, and their behaviors were determined solely by the optimization process. The only constraints that were imposed corresponded to physically realizable gait patterns. The results suggest that human gait is a biological process of energy minimization, as other authors have suggested [42].

The proposed model effectively simulates the kinematics and kinetics of human gait and accurately captures the dynamic response of structures subjected to HSI effects. It is important to note that the vertical force pattern (see Figures 5 and 6) exhibits a different number of peaks compared to the existing force patterns reported in the literature. However, this discrepancy is commonly observed in models with high leg stiffness at low walking speeds, where a greater number of force peaks is generated as the walking speed decreases [43].

The results of the simulation of the HSI demonstrate the potential of this approach for modelling HSI effects pedestrian bridges. As such, it is a viable methodology for the evaluation and understanding of the impact of HSI. A comparison of the results of simulations of a person walking on a rigid surface and walking on a flexible bridge revealed significant differences in the contact forces induced by pedestrians traversing the structure. In particular, differences of 63% and 23% for vertical and lateral forces respectively, highlight the influence of HSI effects on contact forces and their implications for structural response.

Finally, as shown in Figure 15, pedestrian bridges that are not expected to have excessive vibrations may experience frequency synchronization problems if high occupancy levels are reached and the natural frequency of the coupled system decreases. This highlights the importance of taking this factor into account during the design and construction phases in order to prevent serviceability issues arising from HSI phenomena.

### *Limitations and Future Research*

The model presented does not enable the assessment of HSI with people running or jumping. This limitation is linked to one of the main restrictions of normal walking implemented in this model, which requires that at least one of the pedestrian's feet is in contact with the ground, contrary to the activity of running, where there may be moments where both feet are in flight. Therefore, future work will extend this study by eliminating restrictions on the duration and length of double support such that activities other than

walking can be considered. Another potential area of research is to consider other types of load associated with humans crossing pedestrian bridges, such as those due to bicycles or strollers.

Biomechanical models of human gait that use reverse dynamics have been validated in previous research by proving their feasibility in predicting the kinematics and kinetics of gait (Movements and Forces) [29,31]. However, the validation of these models considering vibrating surfaces in real environments has not been addressed. The collection of experimental data where the complete body movements of the gait are recorded under HSI conditions and simultaneously structural vibrations in real bridges are configured as the next step to validate and comprehensively evaluate the approach proposed in this research. In addition, a detailed statistical study of anthropometric characteristics is just as important. This analysis will allow defining key anthropometric parameters that can be integrated into the proposed human-structure interaction models, such as the one proposed in this study, and performing comprehensive comparisons and validations, reducing the uncertainty of the physical parameters of the human system.

Finally, it is known that anthropic loads are not the only dynamic loads to which pedestrian bridges are subjected, in that sense the integration of this model with wind or earthquake effects becomes an interesting field of research that is worth exploring.

A future key aspect to be perfected within the model presented in this research is related to the kinematic model assumed for the body segment of the foot (Section 2.1.1). This is due to the biological variability of the dynamics of the foot, where the kinematics and kinetics are different according to the anthropometric conditions of the person, such as the anatomical structure of the foot, body weight, body mass index (BMI), joint range of ankle motion, or age. In this sense, a robust model that can be implemented in this model that reduces the uncertainty gap of biological variability is of great importance.

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### ***3.4 Experimental Campaign and Real-Time Hybrid Simulation for Human-Structure Interaction Assessment on Pedestrian Bridges.***

To better understand the effects of human-structure interaction (HSI) on pedestrian bridges, a comprehensive experimental campaign was conducted on a large-scale footbridge, known as the Premier Footbridge, in Santiago de Cali, Colombia. This campaign aimed to analyze the vibration serviceability of the structure under various pedestrian-induced loads. The bridge, with its multiaxial dynamic sensitivity, served as an ideal prototype for assessing HSI effects under real operational conditions. The experimental tests involved synchronized and non-synchronized human gait scenarios at varying load densities, which provided valuable insights into how pedestrian loads influence structural vibrations, especially in the lateral and vertical directions. The results highlighted the significant effects of HSI on the dynamic behavior of the footbridge, particularly the amplification of lateral accelerations and changes in damping ratios as pedestrian load densities increased. In parallel, a reference pedestrian bridge was evaluated using real-time hybrid simulations (RTHS). This innovative approach combined the analytical formulation of the pedestrian bridge response, as previously discussed, with the use of a multiaxial transfer function (HSI-MTF) system to simulate pedestrian loads in real-time. The RTHS methodology enabled the precise simulation of human-induced vibrations and their effects on the bridge structure, considering the complexities of HSI. By incorporating real-time data and analytical models, this approach offers a robust framework for assessing the performance of pedestrian bridges under dynamic loading conditions, making it a valuable tool for enhancing the design and serviceability of such structures.

The article “*Comprehensive Experimental Analysis of the Vibration Serviceability due to Dynamic Interaction between Footbridge with Accurate Multi-Axial Dynamic Sensitivity and Human-Induced Gait Loads*” significantly contributes to the multiaxial experimental evaluation of human-structure interaction (HSI) effects on footbridges by conducting extensive field tests on the Premier Footbridge. These tests revealed critical insights into how pedestrian loads influence the dynamic behavior of the structure, particularly under varying load densities and gait conditions. The study highlighted the importance of considering both vertical and lateral vibrations in assessing the serviceability of footbridges subjected to human-induced loads, thus providing a comprehensive dataset for further model validation. In addition, building upon these experimental findings, the article “*Innovative Experimental Assessment of Human-Structure Interaction Effects on Footbridges with Accurate Multi-Axial Dynamic Sensitivity using Real-Time Hybrid Simulation*” advances the methodology by employing Real-Time Hybrid Simulation (RTHS) to evaluate HSI effects on footbridges. This innovative approach allows for the integration of real-time pedestrian load simulations with analytical models of bridge response, using a Human-Structure Interaction Multi-Axial Transfer Function (HSI-MTF) system. The RTHS methodology offers a powerful tool for assessing the performance of pedestrian bridges under dynamic loads, providing accurate indicators of performance such as structural accelerations, damping changes, and vibration serviceability limits. By capturing the complex interplay between human loads and structural response in real-time, this methodology enhances the potential for predictive modeling and design optimization in footbridge engineering.

**Comprehensive experimental analysis of the vibration serviceability due to dynamic interaction between footbridge with accurate multi-axial dynamic sensitivity and human-induced gait loads.**

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**Abstract**

Recent advances in structural engineering have facilitated the construction of large-scale structures by using lightweight and high-strength materials. However, despite innovative design techniques, urban footbridges are susceptible to excessive vibrations induced by human activity. Although vandalism human loads have the greatest dynamic impact on the response of footbridges, owing to their intensity and wide frequency range of affectation, human-induced gait loads have the highest recurrence, randomness, and capacity to limit the serviceability of this type of structure. Traditionally, the effects of the human-structure interaction (HSI) on large-scale footbridges have been evaluated for the structural direction that is most prone to present a dynamic sensitivity or dynamic coupling with pedestrian-induced loads. Nevertheless, these uniaxial analyses on urban large-scale footbridges do not consider the multidirectional effects of the HSI or are limited by the dynamics of the structure itself, which generates the need for experimental information and evaluation of these structures. In this study, an extensive and compressive investigation of the experimental assessment of HSI effects in a functional urban footbridge with accurate multiaxial dynamic sensitivity concerning the typical frequency range of human-induced gait loads was performed. Initially, the reference structure was described and identified using an operational modal analysis (OMA) with structural outputs based on ambient vibrations. Subsequently, a comprehensive and detailed experimental human gait campaign was conducted for various load densities and gait conditions. The results demonstrate that multiaxial HSI effects were concentrated on the lateral fundamental frequency of the reference footbridge, there was an increase of the damping ratios in relation to the load density level of the structure with the greatest effect when the human gait is synchronous, changes in the behavior of human-induced gait loads with HSI effects compared to the rigid surface, the reference structure has serviceability limitations even for load densities considered as moderate, and has the potential to be used as an international benchmark.

**Keywords:** Human-Structure Interaction, Footbridge, Human-Induced Loads, Human Gait, Vibration.

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**1. Introduction**

The dynamic coupling between loads induced by human activities and the response of civil structures, reported in various cases in structural engineering, is frequently a condition of limitation for serviceability, owing to the development of excessive structural vibrations. Dynamic interrelationships between pedestrians and structures have been widely discussed in the literature for civil structures, such as stairs [1]–[3], grandstands [4]–[6], slabs [7]–[11], and footbridges [12]–[20]. Footbridge have become the most affected urban civil structures in terms of vibration serviceability limits due to the increase in technical and aesthetic requirements, their natural condition spanning large distances, and the continuous incorporation of innovative materials that are more resistant and lighter in the construction process. These conditions allow for design optimization, which generates a relatively low mass and damping. Accordingly, footbridges are becoming increasingly slender and flexible, increasing their dynamic sensitivity to excessive vibrations owing to the action of loads induced by human activity [21]–[24]. Although vandalism human loads have the greatest dynamic impact on the response of footbridges [20], [25], [26], due to its intensity and wide frequency range of affectation, human-induced gait loads have the highest recurrence and capacity to limit structural serviceability due to vibrations, because human perception is sensitive to low intensity vibrations (variations around  $1e-3\text{ m}$ ) [2], [27] and to the convergence of the low frequency content of the human gait (vertical: 1.60 to 2.40 Hz; and lateral: 0.70 to 1.30 Hz) with the characteristic frequency range, or one of the harmonics of the footbridges (less than 9.0 Hz). This implies the latent risk that footbridges and pedestrian traffic may be affected by a synchronization phenomenon.

The adaptive [28]–[30], random [31]–[33], and nonstationary conditions [34], [35] of the human gait generate a coupling phenomenon with structural vibrations (Structure to Human Interaction, S2HI), while

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the dynamic properties of the footbridges are altered due to the presence of these anthropic loads, in which the participatory mass and damping provided by pedestrians are relevant for this type of dynamically sensitive structure (Human to Structure Interaction, H2SI) [21], [35]–[37]. Additionally, interrelationships between the gait dynamics of the pedestrians themselves have been developed (Human-to-Human Interaction, HHI) [38]–[40]. All these aspects of interaction are referred to in the literature as effects of Human-Structure Interaction (HSI) [20], [41]–[43]. These effects on footbridges have become an advanced case of dynamic synchronization and are one of the most interesting topics in the structural scientific community in recent years [20], [21], [41], [44].

The study of the effects of human-induced gait loads on the structural response of footbridges was initially analyzed by assessing these anthropic loads on rigid surfaces without coupling effects with the structure, underestimating dynamic interaction effects, and possible decrease in serviceability due to vibration [45]. Furthermore, the S2HI effects have been evaluated on surfaces with lateral movement using instrument treadmills, which allows unidirectional analysis of the effects of vibration on gait loads [41], [46]–[48]. However, the influence of control conditions present in the laboratory on human gait performance has been reported [46]. In the last two decades, a series of footbridge benchmarks have been reported in the literature to develop international benchmarking problems [36], [49], which are highlighted for conditioning the structural design to fundamental frequencies close to the frequency content of the human gait. Nevertheless, the constructive process and variability of the materials used have generated changes in the target fundamental frequencies, which limits its performance in the study of the effects of HSI [32], [33]. Moreover, the cost-effectiveness and obsolescence of this type of footbridge development after the investigation is completed represents another important limitation.

On the other hand, footbridges in functional conditions, belonging to the urbanisms of the cities, are an ideal prototype for assessing HSI effects owing to their geometrics, large-scale measures, actual service loads, and the environmental conditions around the structure, which affect pedestrian perception levels [20], [36], [37], [50], [51]. Fujino et. al. [13] conducted one of the first studies on a large-scale footbridge with typical pedestrian service loading conditions (Toda Park Footbridge, Japan), in which the influence of the lateral effects of HSI was quantified for more than 20.0% of pedestrians in coupling with the fundamental frequency of the structure (0.90 Hz) and the limitations on the serviceability due to structural vibrations. Similarly, in Millennium (London), Solferino (Paris), and airport Changi (Singapore), bridges reported excessive lateral vibration due to loads induced by human gait during their openings. In these structures, large-scale tests inducing degrees of anthropic load excitation gradually demonstrated dynamic alterations in the lateral structural vibration behavior caused by HSI effects, which were extensively reported by Dallard et. al., [14], Zivanovic et. al., [35], Dziuba et. al., [52], Ingólfsson et. al., [53], and Brownjohn et. al., [54], [55]. Several cases have been reported in the literature on the effects of the HSI, its modelling, and anthropic load testing in relation to footbridges: Wuhan Yangtze Bridge, China [56]; Le Ponte del Mare, Pescara-Italy [57]; and others [58]–[60]. In addition, review studies have focused on the phenomena of dynamic interaction between pedestrians and structures in the vertical [37] and lateral [53], [61] directions, respectively. Van Nimmen et. al., [19], [62] conducted the most comprehensive study of the evaluation of a large-scale urban footbridge (Eeklo Bridge, Belgium) subjected to human loads, which constituted a benchmark dataset of this structure. However, functional urban footbridges with dynamic sensitivity usually present HSI effects, human-induced gait loads, which is a limitation for the extended and comprehensive assessment of the spatial dynamic response of the structure and global serviceability due to structural vibration.

Therefore, this paper presents an extensive and compressive study on the experimental assessment of HSI effects in a functional urban footbridge with accurate multiaxial dynamic sensitivity (fundamental frequency of the structure: vertical of 1.98 Hz, and lateral of 1.07 Hz) compared to the typical frequency range of human-induced gait loads. This structure is known as the Premier Footbridge and it is located in Santiago de Cali, Colombia. The acquired Premier Footbridge dataset with these test setups is considered as one of the first assessments of serviceability performance under the effects of HSI on this type of structure. Impressions of this large-scale measurement campaign are shown in **Figure 1**. The remainder of this paper is organized as follows. *Section 2* describes the characterization and experimental dynamic behavior of the Premier Footbridge. In *Section 3*, the measurement equipment, data processing, information of the test subjects (TSs), and experimental setup used in the human gait tests are explained.



**Figure 1.** Large-scale gait test on the Premier Footbridge. View of west mast (top-left), pedestrians crossing on the footbridge center span (top-right), and aerial view of the Premier footbridge from south to north (bottom).

The results of the experimental gait tests on the rigid and Premier Bridge are presented and discussed in *Section 4*. Finally, *Section 5* presents the conclusions of the study.

## 2. Premier footbridge

The Premier footbridge has a total length of 84.0 m, with a main central span of 70.0 m and two lateral overhanging spans of 7.0 m each, as shown in **Figure 2** (bottom). The cross section of the footbridge is composed of a steel assembled box with a height of 2.0 m at a spacing of 3.50 m and a thickness of 2.50 cm, supporting a slab of 80.0 mm thickness (**Figure 2, top**). The footbridge is supported by two main



**Figure 2.** Lower view of Premier footbridge (top-left), cross section (top-right), and schematic side view from south to north (bottom).

**Figure 3.** Location of sensors during operational modal analysis and gait test, top view.

masts with  $64.0^\circ$  of inclination, which distribute ten 2.54 cm diameter tensioning strands (five on each side near the respective mast) anchored to two external concrete blocks. The total mass of the footbridge equals 614.0 ton, which is composed of 32.0 tons for the steel elements and 582.0 ton for the concrete elements, approximately. Operational modal analysis (OMA) was conducted, which provided experimental information for the development of a detailed FE model based on the as-built plans and field data. These assessments are detailed in *Section 2.1*.

### 2.1. Operational Modal Analysis (OMA) and modelling of the structure.

An extensive experimental measurement campaign was conducted to obtain the operational modal properties using OMA. The identification of the system using only the outputs was performed based on ambient vibrations, mainly due to service traffic and induced disturbances of the environment. The output-only data (structural acceleration) were processed using the following OMA algorithm: reference-based data-driven stochastic subspace identification (SSI-data/ref) [63], [64] and natural excitation technique with an eigensystem realization algorithm (NExT-ERA) [65]. Ambient vibrations of the Premier footbridge were measured between point *A* to *B* (**Figure 2**, bottom) by locating Waleker SMA-51 Sensors

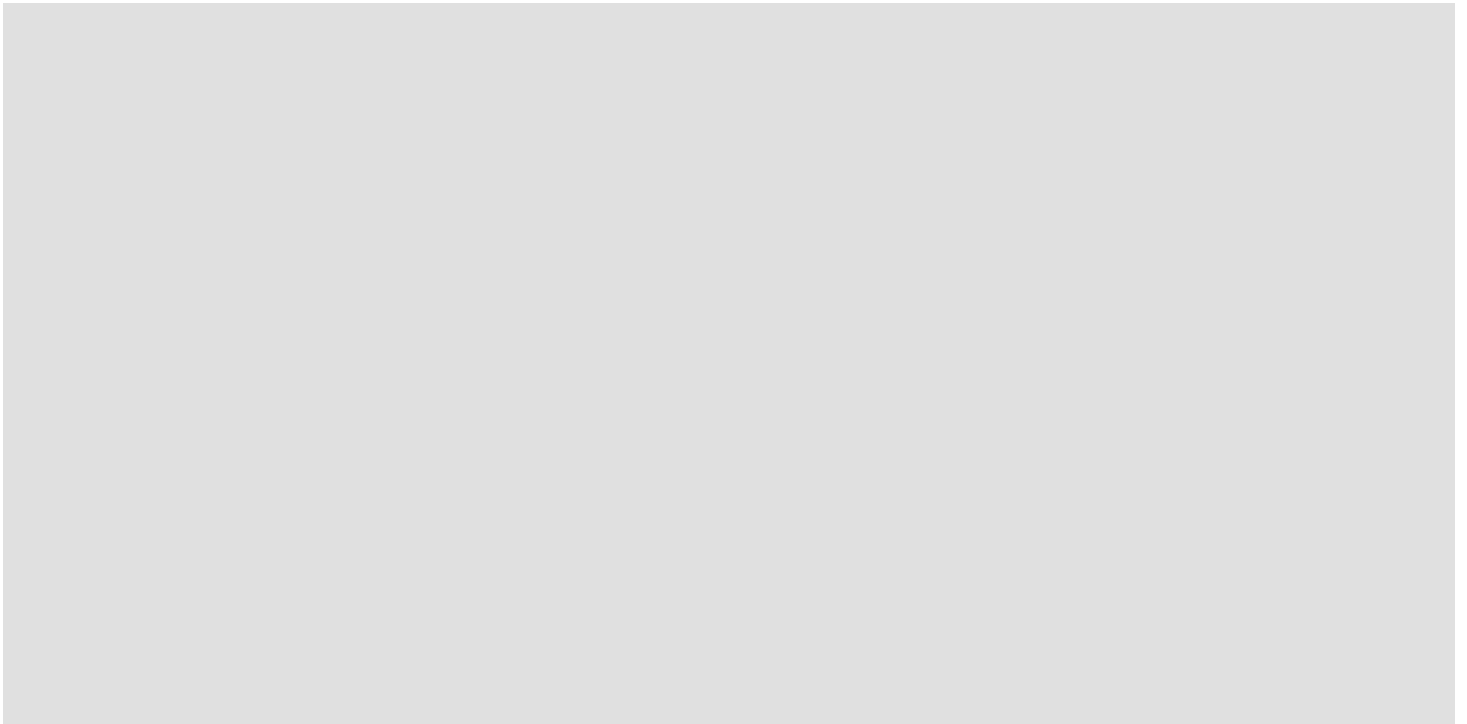


**Figure 4.** Operational Modal Analysis on Premier footbridge. Stabilization diagram of the OMA using the natural excitation technique with eigensystem realization algorithm (NExT-ERA) in comparison with Peak-Peaking location, vertical and lateral direction (top, red). Stabilization diagram of the OMA using reference-based data-driven stochastic subspace identification (SSI) in comparison with Peak-Peaking location, vertical and lateral direction (bottom, blue).

at 7.0 m intervals, as shown in **Figure 3**, the sensor used is shown in **Figure 6** (left). Twenty-one measurements were performed in the vertical and lateral directions using five experimental configurations with a sample-rate of 125.0 Hz, maintaining a fixed sensor in all configurations for data correlation. Twelve vibration modes and fundamental frequencies up to 15.0 Hz were identified by three OMA algorithms using the structural experimental data of the footbridge. A stabilization analysis of the number of poles used in the NExT-ERA and SSI analyses was performed and compared with the analysis in the frequency domain by peak peaking (**Figure 4**). The dynamic properties (natural frequencies  $\hat{f}_j$  and  $\check{f}_j$ , and damping ratios  $\hat{\xi}$  and  $\check{\xi}$ ) identified using NExT-ERA and SSI algorithms are shown in **Table 1**.

**Table 1.** Experimental identified modal properties (mode number  $\check{j}$ , natural frequencies  $\hat{f}_j$  and  $\check{f}_j$ , and damping ratios  $\hat{\xi}$  and  $\check{\xi}$ ) of all modes with a natural frequency below 20.0 Hz and corresponding mode number  $j$  and natural frequencies  $f_{m,j}$  predicted by the finite- elements (FE) model, calculated modal assurance criterion (MAC) and corresponding apparent modal mass in the vertical  $m_{\phi,V}$  and lateral  $m_{\phi,L}$  directions, calculated with the calibrated FE model of the footbridge; terms  $\check{V}$  and  $\check{L}$  indicate very large values in the respective direction.

| $\check{j}$<br>Mode | NExT-ERA       |                     | SSI              |                       |                 |                        | FE Model                                                                                               |       |              |              |
|---------------------|----------------|---------------------|------------------|-----------------------|-----------------|------------------------|--------------------------------------------------------------------------------------------------------|-------|--------------|--------------|
|                     | $\hat{f}$ [Hz] | $\hat{f}_{sd}$ [Hz] | $\check{f}$ [Hz] | $\check{f}_{sd}$ [Hz] | $\hat{\xi}$ [%] | $\check{\xi}_{sd}$ [%] | $f_m$ [Hz]                                                                                             | MAC   | $m_{\phi,V}$ | $m_{\phi,L}$ |
| 1                   | 1.07           | 0.02                | 1.06             | 0.01                  | 0.77            | 0.41                   | 1.07                                                                                                   | 0.99  | $\check{L}$  | 114          |
| 2                   | 1.98           | 0.06                | 1.95             | 0.05                  | 0.86            | 0.52                   | 1.96                                                                                                   | 0.99  | 114          | $\check{V}$  |
| 3                   | 2.56           | 0.01                | 2.56             | 0.01                  | 0.12            | 0.11                   | 2.55                                                                                                   | ----- | $\dagger^*$  | -----        |
| 4                   | 3.15           | 0.01                | 3.15             | 0.03                  | 0.47            | 0.23                   | 3.10                                                                                                   | 0.99  | 349          | $\check{V}$  |
| 5                   | 4.08           | 0.01                | 4.09             | 0.03                  | 0.18            | 0.03                   | 4.14                                                                                                   | 0.98  | $\check{L}$  | 155          |
| 6                   | 6.65           | 0.02                | 6.67             | 0.01                  | 0.48            | 0.09                   | 6.65                                                                                                   | 0.92  | $\check{L}$  | 137          |
| 7                   | 7.81           | 0.03                | 7.81             | 0.03                  | 0.65            | 0.13                   | 7.76                                                                                                   | 0.73  | 250          | $\check{V}$  |
| 8                   | 8.74           | 0.06                | 8.71             | 0.01                  | 0.28            | 0.07                   | $\dagger^* \rightarrow$ There was not sufficient experimental information to represent the modal form. |       |              |              |
| 9                   | 12.15          | 0.10                | 12.12            | 0.02                  | 0.47            | 0.12                   |                                                                                                        |       |              |              |
| 10                  | 13.36          | 0.02                | 13.37            | 0.02                  | 0.35            | 0.18                   |                                                                                                        |       |              |              |
| 11                  | 15.50          | 0.19                | 15.47            | 0.05                  | 0.94            | 0.21                   |                                                                                                        |       |              |              |
| 12                  | 18.29          | 0.16                | 18.30            | 0.01                  | 0.33            | 0.05                   |                                                                                                        |       |              |              |

A detailed finite elements model (FEM) of the Premier Footbridge was developed based on the as-built plans and field information from technical visits. This analytical model was calibrated using the experimentally identified modal properties, taking the elastic modulus of the steel, moments of inertia of the main beam and masts, the effective area of the steel tensioners, and the thickness of the bearing slab as calibration parameters. **Table 1** shows the outstanding concordance found between the experimentally identified modal characteristics and the characteristics calculated by the calibrated FE model; the relative errors in the frequencies of the first seven modes are limited to 1.59%, and the modal assurance criterion (MAC) values, calculated using equation (1), are all close to 1.0. Some of the lower modes are presented in **Figure 5**, which illustrates the alternation between the lateral and vertical modes.

$$MAC_{a-e} = \frac{(\Phi_a^T \Phi_e)^2}{\Phi_a^T \Phi_a \Phi_e^T \Phi_e} \quad (1)$$

$$m_{j,e} = \max(|\Phi_{j,e}|)^{-2} \quad (2)$$

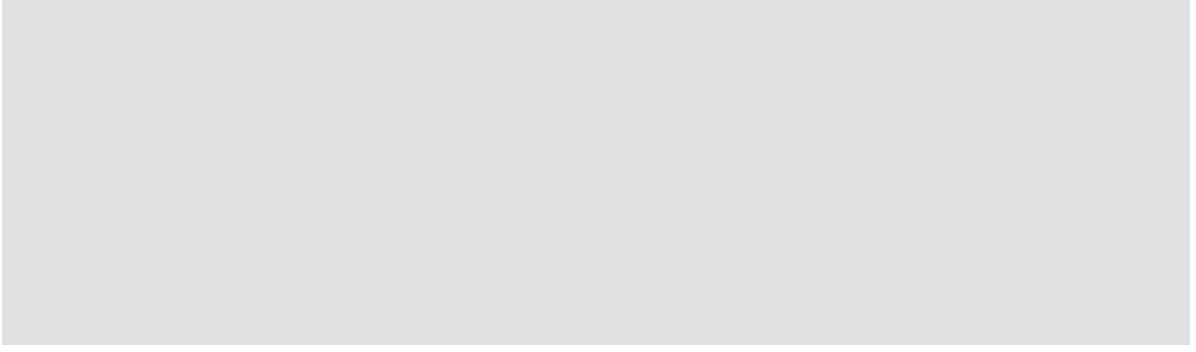


**Figure 5.** Top and lateral views of the first six modes of the empty Premier footbridge: Mode 1  $\rightarrow f_1=1.07$  Hz; Mode 2  $\rightarrow f_2=1.98$  Hz; Mode 3  $\rightarrow f_3=2.56$  Hz; Mode 4  $\rightarrow f_4=3.15$  Hz; Mode 5  $\rightarrow f_5=4.08$  Hz; and Mode 6  $\rightarrow f_6=6.65$  Hz.

To interpret the structural response, **Table 1** lists the apparent modal masses in the vertical ( $m_{\Phi,V}$ ) and lateral ( $m_{\Phi,L}$ ) directions, which were calculated using Equation (2). Where  $\Phi_{j,e}$  denotes the components in direction  $e$  (lateral  $l$  or vertical  $v$ ) of the mass-normalized mode shape  $\Phi_j$ . For a purely vertical or lateral mode,  $m_{\Phi,V}$  or  $m_{\Phi,L}$  corresponds to the proportion of physical mass of the structure employed in that mode [36]. Therefore, the smaller  $m_{\Phi-j,e}$ , the more sensitive mode  $\Phi-j$  is to the dynamic excitation in direction  $e$ . The values for  $m_{\Phi,V}$  and  $m_{\Phi,L}$  showed in **Table 1** indicate that modes 1, 5 and 6 are purely lateral bending modes (L), while modes 2, 4 and 7 are purely vertical bending modes (V); associated with a low apparent lateral ( $m_{\Phi,L}$ ) and vertical ( $m_{\Phi,V}$ ) modal mass, respectively. Moreover, mode 3 was associated with the dynamic behavior of the masts of the footbridge, which had a higher lateral performance.

### 3. Experimental tests

This section describes a large-scale experimental study involving synchronized and non-synchronized gait tests under different pedestrian-induced loading conditions. The following items were successively discussed: measurement equipment and setup, data processing, test subjects, and setup of the tests.

#### 3.1. Measurement equipment and setup

Footbridge deck accelerations were measured using wireless triaxial acceleration sensors (Waleker SMA-551, range  $\pm 2.0$  g, least significant bit (LSB) of  $3.815 \times 10^{-6}$  g, [66]) with a sample time of 125.0 Hz, which is shown in **Figure 6** (left) with its global positioning system (GPS) and power supply unit (PSU). A



**Figure 6.** Measurement Equipment and sensors for test subjects. Waleker SMA-551 triaxial acceleration sensors (left) and insole pressure sensor Stridalzyzer Insight by ReTiSense (Right).

maximum of 5.0 sensors were used under five experimental configurations, maintaining a fixed sensor in all configurations for the data correlation implemented in the OMA. In addition, a sixth experimental configuration was used for events related to pedestrian gait-induced excitation, as shown in **Figure 3**. Additionally, in each human gait-induced load test described in *Section 3.4*, two test subjects were instrumented with a pair of insole pressure sensors (Stridalyzer Insight developed) by ReTiSense [67]. This pair of sensors, shown in **Figure 6** (right), had an adjustable sample frequency between 1.0-50.0 Hz. Furthermore, they integrated a tridimensional inertial measurement unit (IMU), a three-dimensional gyroscopic sensor, and 10.0 micro load sensors distributed in the contact area of the devices.

### 3.2. Data processing

The structural acceleration signals were preprocessed in MATLAB as follows: (1) the offset was removed using function *detrend*; (2) the sample frequency remained at 150.0 Hz and a low-pass filter of 120.0 Hz; and (3) the high-pass filter was applied using a fourth-order Butterworth filter with a cut-off frequency of 0.10 Hz. Based on this preprocessing of the structural acceleration signals, an operational modal analysis was carried out with two algorithms (SSI and NExT-ERA) to identify the dynamic properties of the Premier footbridge, as described in *Section 2.1*, and to determine the dynamic effects of the human-induced gait load on the structure, as explained in *Section 4.1*.

The human-induced gait loads acquired with insole pressure sensor Stridalyzer Insight were preprocessed in Microsoft Office and MATLAB as follows: (1) the human-induced gait load data were downloaded from online database platform of ReTiSense with a sample frequency of 50.0 Hz in format Microsoft Office; (2) the data were loaded from Microsoft Office to MATLAB using function *xlsread* and reorganized in each event evaluated; (3) the sample frequency remained at 50.0 Hz and a low-pass filter of 25.0 Hz; and (4) the high-pass filter was applied using a fourth-order Butterworth filter with a cut-off frequency of 0.05 Hz. Based on this preprocessing, the human-induced gait loads were assessed to determine the changes in the dynamic behavior of these loads under the influence of structural vibration, as described in *Section 4.2*.

The synchronization of human gait-induced loads and structural vibration of the premier footbridge during the load events consisted of a random impact induced by a test subject using an insole pressure sensor close to a triaxial acceleration sensor, which was interconnected with the other sensors by the action of GPS in real time.

**Table 2.** Identification number (ID), height (H), body mass (BM), and sex (S, *f*→female or *M*→Male) of each test subject.

| ID | H<br>[m] | BM<br>[kg] | S | ID | H<br>[m] | BM<br>[kg] | S | ID | H<br>[m] | BM<br>[kg] | S | ID | H<br>[m] | BM<br>[kg] | S |
|----|----------|------------|---|----|----------|------------|---|----|----------|------------|---|----|----------|------------|---|
| 1  | 1.75     | 86.20      | M | 19 | 1.58     | 49.10      | f | 37 | 1.87     | 99.20      | M | 55 | 1.73     | 64.10      | M |
| 2  | 1.66     | 69.00      | M | 20 | 1.67     | 56.60      | f | 38 | 1.69     | 54.40      | f | 56 | 1.65     | 62.60      | M |
| 3  | 1.72     | 68.30      | M | 21 | 1.75     | 73.00      | M | 39 | 1.77     | 84.10      | M | 57 | 1.66     | 65.30      | M |
| 4  | 1.82     | 75.00      | M | 22 | 1.75     | 85.30      | M | 40 | 1.71     | 69.20      | M | 58 | 1.73     | 66.40      | M |
| 5  | 1.71     | 62.70      | M | 23 | 1.65     | 54.00      | f | 41 | 1.78     | 89.80      | M | 59 | 1.77     | 72.80      | M |
| 6  | 1.76     | 82.50      | M | 24 | 1.80     | 76.50      | M | 42 | 1.68     | 50.00      | f | 60 | 1.88     | 74.70      | M |
| 7  | 1.73     | 59.00      | f | 25 | 1.80     | 75.30      | M | 43 | 1.64     | 57.57      | M | 61 | 1.68     | 74.10      | M |
| 8  | 1.78     | 64.20      | M | 26 | 1.84     | 84.00      | M | 44 | 1.64     | 53.40      | f | 62 | 1.74     | 75.90      | M |
| 9  | 1.69     | 73.40      | M | 27 | 1.68     | 58.20      | M | 45 | 1.75     | 99.50      | M | 63 | 1.72     | 74.20      | M |
| 10 | 1.79     | 61.10      | M | 28 | 1.66     | 56.30      | f | 46 | 1.71     | 76.30      | M | 64 | 1.60     | 65.80      | M |
| 11 | 1.70     | 91.50      | f | 29 | 1.73     | 72.55      | M | 47 | 1.72     | 81.70      | M | 65 | 1.69     | 56.10      | M |
| 12 | 1.66     | 53.80      | M | 30 | 1.76     | 67.20      | M | 48 | 1.81     | 68.70      | M | 66 | 1.61     | 63.70      | M |
| 13 | 1.78     | 70.00      | M | 31 | 1.79     | 81.90      | M | 49 | 1.74     | 66.70      | M | 67 | 1.77     | 79.40      | M |
| 14 | 1.69     | 67.70      | M | 32 | 1.82     | 87.40      | M | 50 | 1.73     | 67.90      | M | 68 | 1.67     | 72.30      | M |
| 15 | 1.68     | 59.40      | f | 33 | 1.73     | 86.90      | M | 51 | 1.82     | 91.30      | M | 69 | 1.73     | 64.90      | M |
| 16 | 1.73     | 92.70      | M | 34 | 1.63     | 57.00      | f | 52 | 1.67     | 60.90      | M | 70 | 1.78     | 70.30      | M |
| 17 | 1.64     | 55.60      | f | 35 | 1.68     | 55.00      | f | 53 | 1.77     | 81.20      | M |    |          |            |   |
| 18 | 1.74     | 67.50      | M | 36 | 1.66     | 53.60      | f | 54 | 1.71     | 60.50      | f |    |          |            |   |

### 3.3. Test Subjects

The height, age, and body mass of the test participants are shown in **Table 2**. For the gait test involving a pedestrian density between 0.005 to 0.357 people\*m<sup>-2</sup>, the average height was  $1.73 \text{ m} \pm 6.40 \text{ cm}$ , and the average body mass was  $68.60 \pm 13.80 \text{ kg}$ . All pedestrians walked according to the test setup described in Section 3.4.2 on Premier Footbridge. In addition, the first twenty test subjects, highlighted in gray in **Table 2**, also participated in the rigid surface gait test, as described in Section 3.4.1.

### 3.4. Setup of the gait tests

#### 3.4.1. Rigid surface gait tests

The rigid surface gait tests scenario consisted of a runway composed of two parallel lanes, 2.60 m long and 0.50 m wide, the inner edges of which were separated by 0.75 m, as is shown in **Figure 7** (top). The runway was demarcated with yellow non-slip tape so that the test subjects could clearly identify the path. This configuration allows individuals to complete at least two consecutive linear steps. In addition, it was important that each experimental subject felt comfortable and could walk naturally.

Each participant performed two tests, the first one based on walking on the test track for 3.0 complete laps (starting and ending at the same point), with the condition of walking with a frequency close to 2.0 Hz, for which a metronome was used, configured at a rate of 120 BMP (equivalent to the frequency of 2.0 Hz) that alerted the experimental subject to take a step. The second test analogously consisted of walking for three consecutive laps; however, in this case, each person walked at their own pace. All test subjects used the insole pressure sensor Stridalyzer, as shown in **Figure 6** (right), for both types of gait tests described.

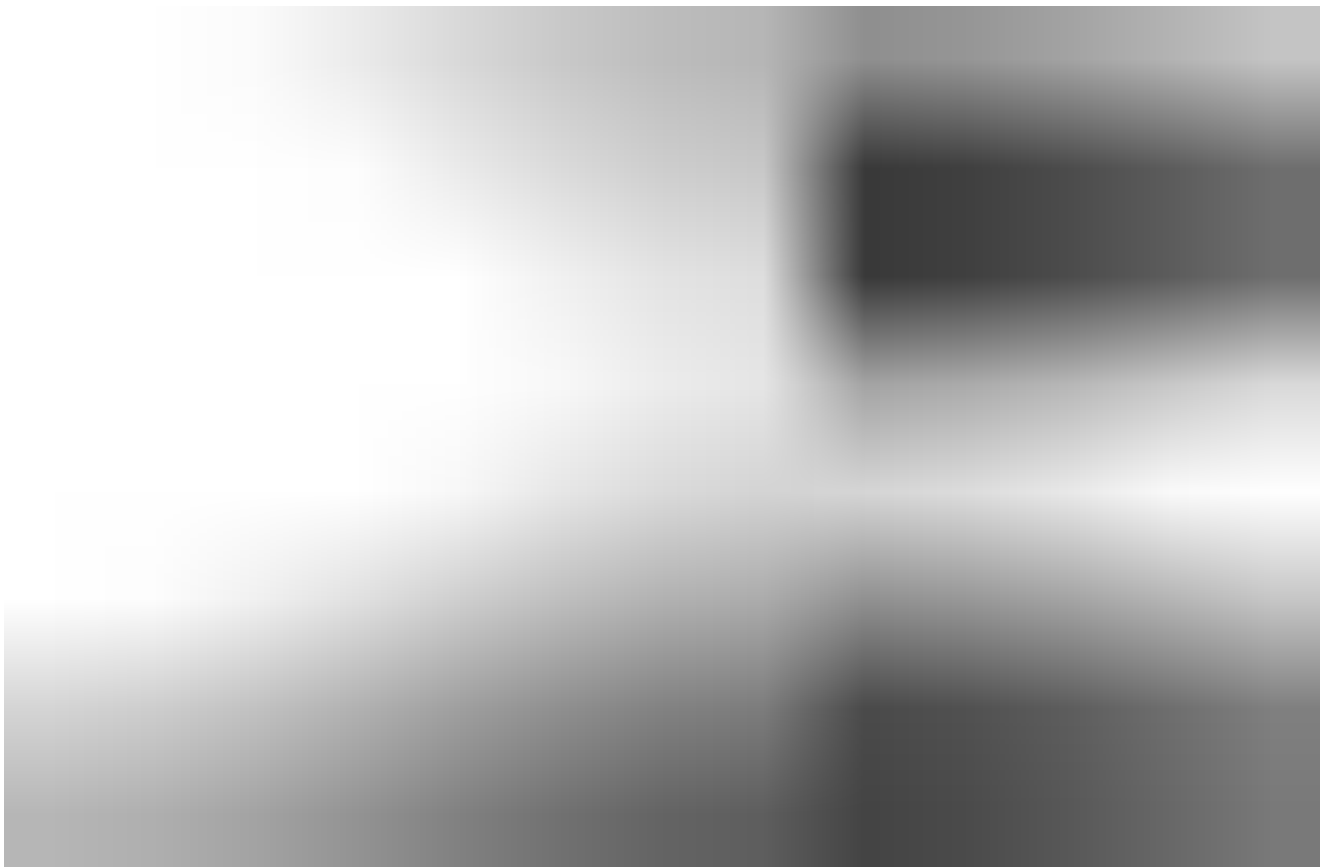


**Figure 7.** Setup of rigid surface gait tests (top) and Gait test on Premier Footbridge (bottom).

### 3.4.2. Gait tests on Premier Footbridge

The gait tests on Premier Footbridge consisted of a complete pedestrian crossing starting and finishing at points *A* or *B* (around point *B* or *A*), according to their initial location, as shown in **Figure 7** (bottom). Twenty-five gait tests were carried out on the Premier Footbridge, which were divided into synchronized tests with some metronomes in each pedestrian group with an induced gait frequency ( $f_g$ ) close to 2.0 Hz and non-synchronized tests with a free gait frequency. In addition, two random gait tests were performed with structural load densities of 10.22% and 35.77% (0.102 and 0.358  $people \cdot m^{-2}$ ). All gait tests with their gait condition, initial number of test subjects at each point (*A* or *B*), and load density of the Premier Footbridge are listed in **Table 3**. The Stridalizer insole pressure sensor was used by the same two pedestrians during the all-gait tests, except for tests 1 and 2, which had a single pedestrian. Moreover, in the tests 14, 19, and 24 there was a pedestrian in a passive state (standing) in the middle span of the structure ( $\Phi$ ) using the insole pressure devices, as is shown in **Figure 7** (bottom).

**Table 3.** Setup of the Gait test on Premier Footbridge.

| Test No. | Gait condition                       |                  |        | Number of test subjects                                                                   |          |                                                                                             | Load density of the structure [ $people \cdot m^{-2}$ ] |
|----------|--------------------------------------|------------------|--------|-------------------------------------------------------------------------------------------|----------|---------------------------------------------------------------------------------------------|---------------------------------------------------------|
|          | Synchronized [ $f_g \approx 2.0$ Hz] | Non-synchronized | Random |  $x$ $n$ | $\Phi^*$ |  $x$ $m$ |                                                         |
| 1        | $x$                                  |                  |        | 1                                                                                         | 0        | 0                                                                                           | 0.51%                                                   |
| 2        |                                      | $x$              |        | 1                                                                                         | 0        | 0                                                                                           |                                                         |
| 3        | $x$                                  |                  |        | 2                                                                                         | 0        | 1                                                                                           | 1.53%                                                   |
| 4        |                                      | $x$              |        | 2                                                                                         | 0        | 1                                                                                           |                                                         |
| 5        | $x$                                  |                  |        | 3                                                                                         | 0        | 2                                                                                           | 2.55%                                                   |
| 6        |                                      | $x$              |        | 3                                                                                         | 0        | 2                                                                                           |                                                         |
| 7        | $x$                                  |                  |        | 5                                                                                         | 0        | 5                                                                                           | 5.11%                                                   |
| 8        |                                      | $x$              |        | 5                                                                                         | 0        | 5                                                                                           |                                                         |
| 9        | $x$                                  |                  |        | 10                                                                                        | 0        | 10                                                                                          | 10.22%                                                  |
| 10       |                                      | $x$              |        | 10                                                                                        | 0        | 10                                                                                          |                                                         |
| 11       |                                      |                  | $x$    | -                                                                                         | -        | -                                                                                           | 10.22%                                                  |
| 12       | $x$                                  |                  |        | 15                                                                                        | 0        | 15                                                                                          | 15.33%                                                  |
| 13       |                                      | $x$              |        | 15                                                                                        | 0        | 15                                                                                          |                                                         |
| 14*      |                                      | $x$              |        | 15                                                                                        | 1        | 15                                                                                          | 15.33%                                                  |
| 15       | $x$                                  |                  |        | 20                                                                                        | 0        | 20                                                                                          | 20.44%                                                  |
| 16       |                                      | $x$              |        | 20                                                                                        | 0        | 20                                                                                          |                                                         |
| 17       | $x$                                  |                  |        | 25                                                                                        | 0        | 25                                                                                          | 25.55%                                                  |
| 18       |                                      | $x$              |        | 25                                                                                        | 0        | 25                                                                                          |                                                         |
| 19*      |                                      | $x$              |        | 25                                                                                        | 1        | 25                                                                                          | 25.55%                                                  |
| 20       | $x$                                  |                  |        | 30                                                                                        | 0        | 30                                                                                          | 30.66%                                                  |
| 21       |                                      | $x$              |        | 30                                                                                        | 0        | 30                                                                                          |                                                         |
| 22       | $x$                                  |                  |        | 35                                                                                        | 0        | 35                                                                                          | 35.77%                                                  |
| 23       |                                      | $x$              |        | 35                                                                                        | 0        | 35                                                                                          |                                                         |
| 24*      |                                      | $x$              |        | 35                                                                                        | 1        | 35                                                                                          | 35.77%                                                  |
| 25       |                                      |                  | $x$    | -                                                                                         | -        | -                                                                                           | 35.77%                                                  |

\* Pedestrian in a passive state (standing) in the middle span of the structure.

## 4. Results

### 4.1. Dynamic performance of the Premier Footbridge

The dynamic performance of the Premier Footbridge was assessed using the structural acceleration in the positions 30.0% and 50.0%, according to the distribution sensors (*Section 3.1*) and the setup of the gait test (*Section 3.4.2*), which presented the maximum structural response in accordance with its dynamic behavior described in *Section 2.1*. All gait tests at each assessment point were processed in the frequency domain using the *Cross-Power Spectral Density Equation* (3), as proposed in [68], [69].

**Figure 8.** Dynamic response vertical and lateral of the Premier Footbridge under gait tests. Lateral dynamic response at 30.0% length (top-left), lateral dynamic response at 50.0% length (top-right), vertical dynamic response at 30.0% length (bottom-left), and vertical dynamic response at 50.0% length (bottom-right).

$$CPSD_{xy}(\omega) = \sum_{m=-\infty}^{\infty} R_{xy}(m)e^{-j\omega m}; R_{xy} = E\{x_n y_n^*\} \forall \{-\infty < n < \infty\} \quad (3)$$

Frequency analysis using cross-power spectral density of the gait tests determined that the lateral dynamic behavior in the 30.0% length position presented a tendency to be dominated by fundamental frequency  $f_1 = 1.07 \text{ Hz}$  with mean response amplitudes higher than 88.50% compared to  $f_2 = 1.98 \text{ Hz}$ ,  $f_3 = 2.56 \text{ Hz}$ , and  $f_5 = 4.08 \text{ Hz}$ ; which also compose spectrally the dynamic response of the structure, as shown in **Figure 8** (top-left). Likewise, lateral dynamic behavior in the 50.0% length (middle span of the footbridge) presented the same dominance tendency by  $f_1 = 1.07 \text{ Hz}$ , however, only  $f_3 = 2.56 \text{ Hz}$  participated spectrally in the dynamic response of the structure at this position with a contribution of 1.30% with respect to the fundamental frequency, **Figure 8** (top-right). In turn, the vertical dynamic behavior in the 30.0% and 50.0% length position shown a dominance tendency by  $f_2 = 1.98 \text{ Hz}$  with means amplitudes of structural responses higher than 98.75% compared to lateral frequencies  $f_1 = 1.07 \text{ Hz}$  and  $f_3 = 2.56 \text{ Hz}$ , which also compose spectrally the vertical dynamic response of footbridge, **Figure 8** (bottom). The progressive increase in the load density in the Premier Footbridge, in the gait tests, generated variation of amplitude and even changes in the frequency values themselves, especially, in the frequencies higher than the fundamental frequency this behavior was significantly lower than that presented by the fundamental frequency  $f_1 = 1.07 \text{ Hz}$ , as is shown in **Figure 8**. This condition allows to infer that the fundamental frequency was the modal frequency of the structure more affected in dynamic response amplitude and frequency themselves values by the increase and interrelationship with of the test subjects, thereby the effects of HSI were concentrated in the first or fundamental frequency even though the Premier Footbridge had a dual dynamic sensitivity condition ( $f_1 = 1.07 \text{ Hz}$  and  $f_2 = 1.98 \text{ Hz}$  are close to the fundamental frequency content of human gait).

The concentration of the dynamic effects of the HSI at the structural fundamental frequency allowed the identification of an increase in the damping of the dynamic system (structure-pedestrians) and a decreased tendency of the value of  $f_1$  due to the presence of additional mass of the test subjects on the



**Figure 9.** Performance of the Premier Footbridge due to the load density. Load density  $\rho$  (x100) [people\*m<sup>-2</sup>] vs damping ratio  $\zeta$  [%] (top), Load density  $\rho$  (x100) [people\*m<sup>-2</sup>] vs lateral peak accelerations at the span center [m\*s<sup>-2</sup>] (bottom-left), and Load density  $\rho$  (x100) [people\*m<sup>-2</sup>] vs vertical peak accelerations at the span center [m\*s<sup>-2</sup>] (bottom-right).

footbridge, as shown in **Figure 8**. These dynamic effects of HSI increased with increasing load density on the Premier Footbridge. Initially, the behavior of the structural damping under different values of load density was similar for the synchronized and non-synchronized tests for load densities less than 0.256 people\*m<sup>-2</sup>. However, after this load density value, the damping of the structure for the first mode increases exponentially for synchronic tests compared with non-synchronic tests. In addition, the random test, where each test subject could walk in the desired direction and at the desired gait frequency, generated structural damping with similar behavior to that presented for the synchronous and non-synchronous tests at a load density of less than 0.256 people\*m<sup>-2</sup>. Nevertheless, a difference of 78.60% and 70.0% was identified with the synchronized and non-synchronized tests, respectively, for a load density of 0.358 people\*m<sup>-2</sup>, as shown in **Figure 9** (top) with prediction intervals of 80.0% in each case. Furthermore, it should be noted that these damping values are in accordance with the typical values expected for this type of structure according to international guidelines for the assessment of the vibrational behavior of footbridges under pedestrian loading (SETRA) [70] and human-induced vibration of steel structures (HIVOSS) [71]; In addition, this behavior of the damping ratios is according with presented by Zivanovic et. al. [72].

The maximum vertical and lateral accelerations in the middle span of the footbridge (50.0% length) for the synchronized, non-synchronized, and random tests were determined at different values of the load density of the structure, as shown in **Figure 9** (bottom). Lateral structure accelerations started to increase exponentially after a load density of 0.10 people\*m<sup>-2</sup> in the synchronized tests, by contrast, the non-synchronized and random tests followed a linear behavior, which generated a difference of 85.42% in the maximum accelerations, respectively in the related tests, at a load density of 0.358 0.10 people\*m<sup>-2</sup>, as presented in **Figure 9** (bottom-left). Likewise, vertical structure accelerations presented a linear behavior for all types of gait test, being higher for the synchronized test compared with the non-

**Figure 10.** Behavior of vertical human-induced gait load on rigid surface acquired by insole pressure sensor Stridalyzer Insight. Normalized vertical human-induced load with gait non-synchronized [%] vs gait cycle [%] (top-left), amplitude - gait non-synchronized vs frequency [Hz] (top-right), Normalized vertical human-induced load with gait synchronized [%] vs gait cycle [%] (bottom-left), amplitude - gait synchronized vs frequency [Hz] (bottom-right).

synchronized and random tests, which generated a difference of 51.01% in the maximum accelerations, respectively for a load density of 0.358 0.10 people\*m<sup>-2</sup>, as presented in **Figure 9** (bottom-right). Both analyses had prediction intervals of 80.0% for each case.

#### 4.2. Experimental behavior of human gait-induced loads

Human gait tests on rigid surfaces were performed in accordance with *Section 3.4.1* and *Section 3.3*. The human-induced vertical gait loads normalized with respect to the weight of each test subject for the synchronized and non-synchronized cases exhibited a similar performance. These loads, comparing both gait conditions, present a difference in the average values of less than 7.30%, with values of standard deviation bands lower than 27.30%, as shown in **Figure 10** (top-left and bottom-left). In addition, the average values of the gait frequency for the synchronized (red) and non-synchronized (blue) tests, advanced a difference of 11.10% with deviation bands less than 20.0%, as shown in **Figure 10** (top-right and bottom-right).

Although a similar behavior of the human-induced vertical loads was observed for low-density loads on the Premier Footbridge (approximately 0.05 people\*m<sup>-2</sup>) under synchronized and non-synchronized gait conditions in comparison with rigid surfaces, for densities loads higher than 0.15 people\*m<sup>-2</sup> amplitude changes of induced loads by pedestrians and variation of their step frequency were identified in dynamic gait tests for both gait conditions, as shown in **Figure 11**. These dynamic changes in the human-induced gait loads are attributed directly to HHI effects, which on Premier Footbridge developed two principal step frequencies (around 0.40 and 0.6 Hz) for densities loads greater than 0.15 people\*m<sup>-2</sup> in synchronized gait conditions, presenting an average difference of 50.0% compared with rigid surfaces. However, in non-synchronized gait conditions, two principal step frequencies were identified according to the increase in the load density and position on the structure, developing a step frequency difference higher than 60.0% in comparison with rigid surfaces.

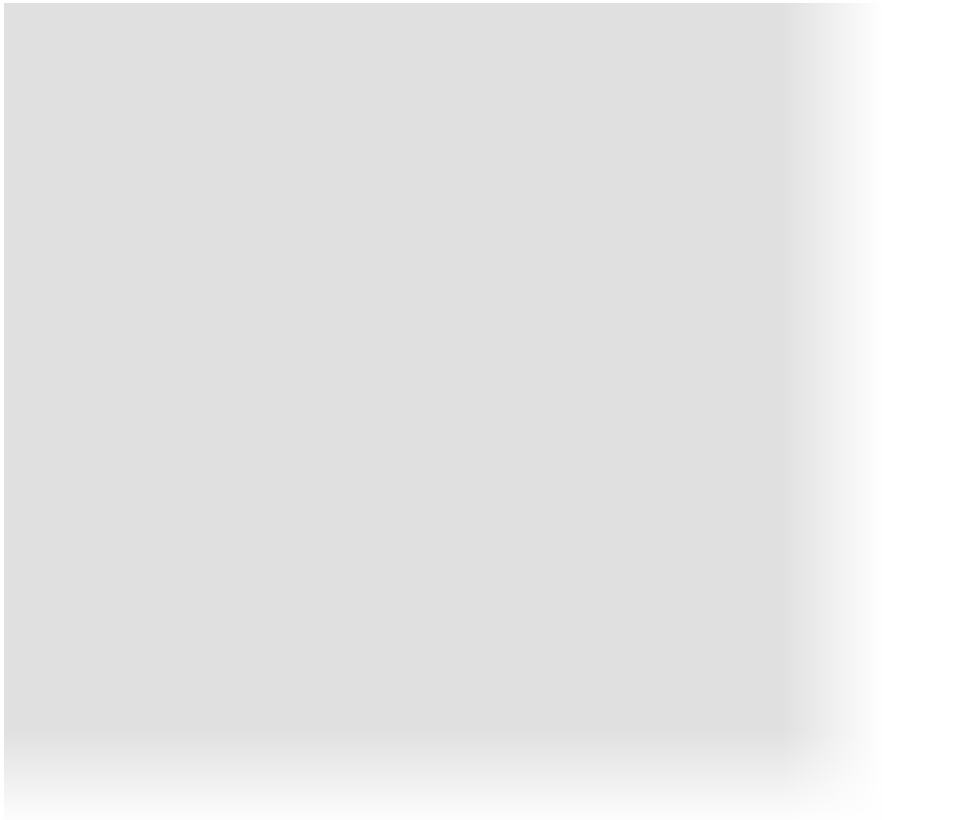


**Figure 11.** Behavior along the footbridge of the human-induced gait loads under three density loads (0.051, 0.204, and 0.358  $people * m^{-2}$ ). Synchronized (top) and non-synchronized (bottom) gait tests.

In contrast, pedestrian-induced loads were identified in a passive state (standing) in the center span of the structure during non-synchronized gait tests with load densities of 0.153, 0.256, and 0.358  $people * m^{-2}$  as described in *Section 3.4.2*. These pedestrian-induced vertical loads remained constant around the value of their weight, with a variation of less than 10.0% for all load densities evaluated on Premier Footbridge, as shown in **Figure 12**. However, Internal Human-Structure Interaction (I-HSI) effects were identified, where there was a redistribution of internal human loads without significant changes in the loads induced on the structure. The insole pressure sensor used allowed identification of the balance of the load (leg by leg) in order to maintain pedestrian stability during the dynamic response of the footbridge. This behavior can induce dynamically sensitive conditions in the perception of pedestrians, which limits the capacitive serviceability of the structure. This study made an initial breakthrough with the aim of comprehending the I-HIS effects and its performance in relation to the structural response.



**Figure 12.** Performance of human-induced loads during a passive state (standing) in the middle span of the structure under three non-synchronized human gait tests with density loads of 0.153, 0.256, and 0.358  $\text{people} * \text{m}^{-2}$ .

#### 4.3. Serviceability performance of the Premier Footbridge

In recent years, it has been accepted that structural vibrations generated by human-induced gait loads are the main serviceability problem of footbridges owing to their recurrence and dynamic coincidence. This limitation is due to the sensitivity of human perception to low vibration intensities during walking, which can be around of  $1e-3$  m [73], [74]. The sensitivity of perception makes pedestrians the main receptors of vibration and controls the serviceability of a structure. However, the intrinsic variability of human gait behavior due to weight, step frequency, age, sex, activity, and interaction with structural response, conditions the capacity for footbridge serviceability. Decreasing the serviceability of a structure generates disuse processes or, in several cases, phenomena of coupling with human-induced gait loads, such as synchronic lateral excitation (SLE) [19], [26], [73], [75], [76]. The scientific interest of the community in understanding, quantifying, and modelling vibration problems due to IHE effects in footbridges, and in turn, establishing design guidelines to reduce impairments in the serviceability of the structure, has resulted in international standards that specify allowable acceleration values to ensure adequate comfort levels for pedestrians. A brief review of permitted vertical and lateral comfort accelerations for different international standards is presented in **Table 4**.

**Table 4.** A brief review of international standard/guideline on comfort acceleration limits.

| Standard/Guideline                                        | Direction of evaluation                                                                                                                                             |                                                                                                               |
|-----------------------------------------------------------|---------------------------------------------------------------------------------------------------------------------------------------------------------------------|---------------------------------------------------------------------------------------------------------------|
|                                                           | Vertical [ $m*s^{-2}$ ]                                                                                                                                             | Lateral [ $m*s^{-2}$ ]                                                                                        |
| BS 5400 [77]; BRO 2004 [78]                               | $0.5\sqrt{f_v}$                                                                                                                                                     | -                                                                                                             |
| ONT 83 [79]                                               | $0.25\sqrt{f_v}$                                                                                                                                                    | -                                                                                                             |
| VIDI 2057 [80]                                            | $0.6\sqrt{f_v}$                                                                                                                                                     | 0.214                                                                                                         |
| ISO 10137 [81]                                            | $\begin{cases} 1.0 - 4.0 \text{ Hz} \rightarrow 0.70 - 0.10f_v \\ 4.0 - 8.0 \text{ Hz} \rightarrow 0.70 \\ 8.0 - 80.0 \text{ Hz} \rightarrow 0.0375f_v \end{cases}$ | $\begin{cases} 1.0 - 2.0 \text{ Hz} \rightarrow 0.216 \\ > 2.0 \text{ Hz} \rightarrow 0.110f_l \end{cases}$   |
| SETRA [70]; HIVOSS [71]                                   | $\begin{cases} \text{Minimum} \rightarrow 1.0 - 2.50 \\ \text{Uncomfortable} \rightarrow > 2.50 \end{cases}$                                                        | $\begin{cases} \text{Minimum} \rightarrow 0.30 - 0.80 \\ \text{Uncomfortable} \rightarrow > 0.80 \end{cases}$ |
| Eurocode 0 [82], 1 [83], and 5 [84]; Din-Fachberitch [85] | $0.5\sqrt{f_v}$ or 0.70                                                                                                                                             | $0.14\sqrt{f_l}$ or 0.15                                                                                      |

The comfort acceleration limits of the brief review of international standards/guidelines in the vertical and lateral directions, as described in **Table 4**, were determined for Premier Footbridge. Values used for  $f_L$  and  $f_V$  (1.06 and 1.98, respectively) were in accordance with the OMA, which was developed in *Section 2.1*. The lateral accelerations of all non-synchronized tests presented a behavior that was less than the comfort limits of all international standards assessed. In contrast, the lateral accelerations of the synchronized test exceeded the comfort limits proposed by the guidelines SETRA and HIVOSS (around  $0.80 m*s^{-2}$ ) for load densities higher than  $0.30 \text{ people}*m^{-2}$ , as shown in **Figure 13** (left).

The same behavior was observed for the vertical accelerations of the non-synchronized tests, which was less than the comfort limits of all international standards assessed (less than  $0.35 m*s^{-2}$ ) for all levels of the load density in this study, only slightly superior to the ONT-83, considered as a strict standard. However, the vertical accelerations of the synchronized tests were greater than  $0.50 m*s^{-2}$ , which are

**Figure 13.** Serviceability performance of the Premier Footbridge due to the load density compared to international standards/guidelines. Load density  $\rho$  ( $\times 100$ ) [ $\text{people}*m^{-2}$ ] vs lateral peak accelerations at the span center [ $m*s^{-2}$ ] (left), and Load density  $\rho$  ( $\times 100$ ) [ $\text{people}*m^{-2}$ ] vs vertical peak accelerations at the span center [ $m*s^{-2}$ ] (right).

considered as uncomfortable for all international standards evaluated, except for the Eurocode (comfort limit of  $0.70 \text{ people} \cdot \text{m}^{-2}$ ), as shown in **Figure 13** (right).

The multi-axial dynamic sensitivity conditions in the Premier Footbridge are important because the values of the load densities used (less than  $0.358 \text{ people} \cdot \text{m}^{-2}$ ) are between low and moderate values according to the SETRA and HIVOSS guidelines. Dynamic sensitivity conditions can increase for load density levels typical for this type of structure (approximately  $0.80 \text{ people} \cdot \text{m}^{-2}$ ), which can decrease the performance of the serviceability of the footbridge.

#### 4.4. Potential use of the Premier Footbridge Benchmark Multiaxial Dataset

This section provides a brief, non-exhaustive overview of the potential use of the Premier Footbridge Dataset. As indicated in the introduction of this study, *in situ* testing is necessary to further develop and validate robust models of pedestrian-structure interaction, as this type of information is accurate, detailed, and representative of the operational performance of the structure [36], [49]. This was the primary purpose of the collected dataset under the influence of the pedestrians. The key aspects of this dataset are as follows:

- Improvement of HSI models to understand their effects on footbridges susceptible to human-induced loads. In recent years, two types of HSI effects have been described in the literature: *active* and *passive* interactions [38], [62]. The dynamic interaction between the structural system and the human body, as a nonlinear mechanical system, produces the effects of passive HSI. Therefore, the dynamic performance of a coupled human-structure system may differ significantly from that of an empty structure, particularly for flexible and lightweight structures. Many studies [20], [49], [86], including this one, have shown that the passive effects of the HSI, such as changes in the damping ratios and fundamental frequencies, are generally important factors when evaluating the vibration serviceability of footbridges. However, the lack of multiaxial full-scale validation limits the continuous improvement of HSI models and the applicability of the latest *state-of-the-art* developments in current dynamic assessment practices. The Premier Footbridge Benchmark Multiaxial Dataset comprises comprehensive, detailed, and synchronous measurements of the vertical pedestrian-induced loads in active and passive states, the multiaxial structural response, and detailed environmental measures of the empty structure. The analysis of the structural responses in *Section 2.1*, and *4.1*, evidenced clear signs of the effects of the passive HSI, especially concentrated in the fundamental modes of the structure. This focus of dynamic sensitivity shown by the collected dataset makes it suitable for calibrating and validating models for the effects of passive HSI. Innovative approaches include the development of a multi-structured inverse estimation methodology, in which human-induced loads or inputs to the structure with uncertainty values are estimated correctly from the resulting known vibration response of the structure.

As the pedestrian-induced loads were registered synchronously with the structural response, the dataset developed can be applied to research the effects of the active HSI phenomena, whereby the performance of the gait of the pedestrian is changed in relation to the levels of vibration of the structure and the type of gait developed (non-synchronized, synchronized, and random gait). Limitations in the lateral behavior of bridges have been widely reported [17], [46]. In the case of vertical behavior, studies have reported that the limitation in serviceability only occurs for vibration amplitudes that exceed the acceptable limits for vibration comfort [23], [87], [88]. Given that the multi-axial sensitivity of the Premier Footbridge lateral vibrations was high (more than  $1.20 \text{ m} \cdot \text{s}^{-2}$ ) and that the maximum vertical acceleration levels observed during the various types of tests are considerable (less than  $0.70 \text{ m} \cdot \text{s}^{-2}$ ), the data set is extensively suitable for investigations of active HSI phenomena. This is true even at low load densities.

- Improvement of models to estimate the effects of HHI on footbridges. Many gait tests were performed on the Premier Footbridge under different conditions of step frequency (synchronized close to 2.0 Hz, non-synchronized, and random) and Load density ( $0.51\%$  to  $35.77\% \text{ people} \cdot \text{m}^{-2}$ ), as are described in *Section 3.4.2*. This dataset can be used to analyze the interaction between pedestrians using structural responses as inputs to estimate the behavior of crowd-induced loads and improve microscopic and macroscopic models of pedestrian dynamics on footbridges [32], [89]. Additionally, the Premier Footbridge Benchmark Dataset aims to serve as a benchmark for future large-scale experiments involving synchronized crowd excitation and non-synchronized to maximize the use and results of experimental investigations. This study can also contribute to and accelerate the process of integrating

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the latest advances in the field of human-induced vibrations in footbridges into the design guidelines and codes.

## 5. Conclusion

In this study, an extensive experimental assessment of HSI effects in a functional urban footbridge with accurate multiaxial dynamic sensitivity, known as Premier Footbridge, was performed for load densities less than  $0.358 \text{ people} \cdot \text{m}^{-2}$  (considered as low to moderate according to SETRA and HIVOSS standards), which developed synchronized, non-synchronized, and random gait conditions. The acquired Premier Footbridge dataset, with these test setups, is considered one of the first assessments of serviceability performance under the HSI effects on this type of structure with multiaxial dynamic sensitivity. The main conclusions are as follows:

- (1) The vertical and lateral structural accelerations at the 30.0% and 50.0% positions along the Premier Footbridge in the frequency domain for all walking tests evidenced that the IHE effects were mainly concentrated at the lateral fundamental frequency of 1.07 Hz. These effects generated increases in the response amplitudes and decreases in the lateral fundamental frequency, in direct relation to the load density levels progressively present in the structure. Additionally, inter-independent behavior was identified between the vertical and lateral fundamental frequencies that compose the structural responses, which presented values of inter-independent influences lower than 2.20%.
- (2) The stability behavior of the structural damping ratios was identified for synchronized and non-synchronized tests for loading densities lower than  $0.256 \text{ people} \cdot \text{m}^{-2}$ . However, from this value of load density, the damping ratios of the structure for the first mode increased exponentially in the synchronized tests compared to the non-synchronized tests. A difference of 78.60% and 70.0% was identified between the synchronized and non-synchronized tests, respectively, for a load density of  $0.358 \text{ people} \cdot \text{m}^{-2}$ .
- (3) Lateral structure accelerations started to increase exponentially after a load density of  $0.10 \text{ people} \cdot \text{m}^{-2}$  in the synchronized tests in comparison with the non-synchronized and random tests followed a linear behavior, which generated a difference of 85.42% in the maximum accelerations for a load density of  $0.358 \text{ people} \cdot \text{m}^{-2}$ . Moreover, vertical structure accelerations presented a linear behavior for all types of gait tests, being higher for the synchronized test compared with the non-synchronized and random tests, which generated a difference of 51.01% in the maximum accelerations for a load density of  $0.358 \text{ people} \cdot \text{m}^{-2}$ . In addition, a brief review of international standards and guidelines on comfort acceleration limits for this structure was performed, which determined that there was a dynamic sensitivity in the lateral behavior of the footbridge that limited and conditioned the structural serviceability, even for low to moderate load densities.
- (4) Dynamic gait tests on the Premier Footbridge demonstrated amplitude changes in induced loads and variations in step frequency for load densities exceeding  $0.15 \text{ people} \cdot \text{m}^{-2}$ , attributed to Human-Structure Interaction (HHI) effects. In synchronized gait conditions, two principal step frequencies emerged, showing an average difference of 50.0% compared to rigid surfaces, whereas non-synchronized conditions exhibited a difference exceeding 60.0%. Pedestrian-induced loads in the passive state remained constant around the value of their weight, with variations of less than 10.0% across the load densities. Internal human-structure interaction (I-HSI) effects were identified, indicating a redistribution of internal human loads without significant changes in induced structural loads. The study provides insight into I-HSI effects and their impact on structural response, laying a foundation for further investigation into the capacitive serviceability of such structures.
- (5) The Premier Bridge dataset offers a significant opportunity to improve human-structure interaction models and study the effects of pedestrian-induced loads. The importance of these data for advancing the understanding of both active and passive human-structure interaction effects is highlighted.

Passive effects, such as changes in the damping ratios and fundamental frequencies, are key elements in the evaluation of the vibrational behavior of pedestrian bridges. In addition, the usefulness of the dataset for investigating the effects of active human-structure interaction, including variation in pedestrian gait performance in relation to structural vibration levels and the type of gait developed, is noted. These data also have the potential to improve the models for estimating the effects of crowd-induced loads on pedestrian bridges, thus contributing to improved design guidelines and guidance.

### Author contributions

All authors contributed to this research and discussed the results and reviews during all phases of the research paper. All authors read and accepted the published version of the research paper.

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### Declaration of Competing Interest

The authors declare that they have no known competing financial interests or personal relationships that could have influenced the work reported in this study.

### Data availability statement

The raw human-induced gait loads and structural response of the Premier Footbridge data, as well as the video during the test, are available from the *Mendeley Data* at doi: 10.17632/4w3ytxrxxt.1.

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# Innovative Experimental Assessment of Human-Structure Interaction Effects on Footbridges with Accurate Multi-Axial Dynamic Sensitivity using Real-Time Hybrid Simulation

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**Abstract:** This study evaluated the dynamic performance of a reference footbridge under human-structure interaction (HSI) effects using Real-Time Hybrid Simulation (RTHS). The footbridge, designed with precise multi-axial dynamic sensitivity, was tested under pedestrian gait velocities of 1.20, 1.50, and 1.80  $m \cdot s^{-1}$ . The RTHS framework involved an analytical continuous model of the footbridge as a numerical substructure and real human gait loads as the experimental substructure. The results revealed significant dynamic coupling between pedestrian-induced loads and the responses of the structure. Lateral vibrations exhibited a fundamental frequency of approximately 1.0 Hz, whereas vertical vibrations peaked near 2.0 Hz. Dynamic synchronization, particularly at higher gait velocities, amplified the structural vibrations, with lateral loading increasing by up to 300% in the middle span. Vertical loads showed substantial amplification and attenuation depending on gait velocity and footbridge location. Lateral accelerations displayed a dispersion of approximately 15.0%, whereas vertical accelerations showed higher variability, with dispersions reaching up to 20%. The RTHS technique demonstrated high fidelity and accuracy, with global errors below 2.95% and delays of less than 2.10 ms across all evaluated directions. These results emphasize the critical importance of accounting for HSI effects in the design of pedestrian footbridges, because human-induced vibrations can significantly impact structural serviceability and user comfort. This study offers important insights into optimizing footbridge design to mitigate the risks of excessive vibrations and ensure both safety and functionality under typical pedestrian loads.

**Keywords:** Human-Structure Interaction, Footbridge, Human-Induced Loads, Human Gait, Vibration, Real-Time Hybrid Simulation.

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## 1. Introduction

In contemporary urban development, there is growing emphasis on prioritizing pedestrian traffic, leading to the creation of civil structures designed specifically for walkable use. This shift aims to enhance walkability and promote sustainable urban living [1]. However, the dynamic interaction between induced-human activity loads and the response of civil structures frequently limits serviceability owing to the development of excessive structural vibrations. Dynamic coupling between gait pedestrians and structural responses has been extensively discussed in the literature concerning civil structures, such as stairs [2]–[4], grandstands [5]–[9], slabs [10]–[12], and footbridges [13]–[24]. The latter type of structure has become the most affected urban civil structure regarding vibration serviceability limits due to the increase in technical and aesthetic requirements, their condition of spanning large distances, and the ongoing integration of advanced materials that are more robust and lighter during and after the construction process. Grouping of these conditions allows for design optimization, resulting in relatively low mass and damping.

Consequently, footbridges are becoming increasingly slender and flexible, increasing its dynamic sensitivity to excessive vibrations owing to human activity [13], [17], [26]. Normally, vandalism-induced human loads have the greatest dynamic effect on the response of footbridges [18], [27]. However, owing to their intensity and wide frequency range, human-induced gait loads have the highest recurrence and potential to limit the structural serviceability due to the vibrations generated. In addition, the human gait frequency content is typically close to the dynamic properties of these types of structures, which poses a latent risk of synchronization phenomena affecting both footbridge and pedestrian traffic.

The adaptive [17], [28], [29], random [30], [31], and nonstationary conditions [17], [30], [32] of human-induced gait loads have generated an interrelation phenomenon with structural vibrations known as Structure-to-Human Interaction (S2HI) [17], [33]–[35]. Concurrently, the dynamic properties of footbridges are influenced by the presence of these anthropic loads, and the participatory mass and damping provided by human loads are significant for these dynamically sensitive structures; this relationship is known as Human-to-Structure Interaction (H2SI) [23], [36]–[38]. In addition, dynamic effects between the gait pedestrians themselves have been observed and are denoted as Human-to-Human Interaction (HHI) [39]–[41]. These interaction aspects are referred to in the literature as Human-Structure Interaction effects (HSI) [13], [18], [20], [25], [28], [42]–[44]. These effects on footbridges represent an advanced case of dynamic synchronization, and have become one of the most intriguing topics within the structural scientific community in recent years.

Traditionally, human-induced gait load effects on footbridge dynamic responses have been numerically evaluated by assuming the behavior of these anthropic loads on vibrating surfaces without complete interrelation effects with the structural response, which underestimates the dynamic coupling effects and leads to a tendency to condition serviceability due to the low estimation of structural vibrations [21], [32], [45]–[49]. Likewise, S2HI effects have been assessed on surfaces with lateral movement using instrument treadmills, which allows unidirectional or multidirectional analysis of the vibration effects on human-induced gait loads [17], [33]–[35], [50]. Although experimental tests allow better measurement of the human gait response due to controlled conditions and predefined protocols, limitations have generally been reported in the development of variability conditions under HHI effects and individual human gait disturbances [9], [14], [17], [51], [52]. In order to reduce the impact of laboratory conditions, in the last two decades, several footbridge benchmarks have been built, evaluated, and described in the state-of-the-art structural engineering to submit benchmarking problems of international reference, which have been highlighted for conditioning the structural design to fundamental frequencies close to the frequency content of the human gait [25], [26], [42], [49]. However, the construction process and variability of the materials used have led to shifts in the target fundamental frequencies, limiting their effectiveness in studying the HSI effects. In addition, the cost-effectiveness, maintenance, and obsolescence of this type of footbridge development after the research has been completed represent another important constraint.

Studies on functional urban footbridges are ideal study prototypes because of their natural geometry, large-scale dimensions, actual service loads induced by pedestrian flows, and environmental perception around the structure, which affect pedestrian gait behavior. Fujino et al. [53] conducted a pioneering study on a large-scale footbridge under typical pedestrian loading conditions (Toda Park Footbridge, Japan). They quantified the influence of HSI lateral effects for more than 20.0% of pedestrians, observing coupling with the fundamental frequency of the structure ( $\approx 0.91$  Hz) and subsequent serviceability limitations due to structural vibrations. Similarly, excessive lateral vibrations caused by

human-induced loads have been reported during the openings of the Millennium Bridge (London), Solferino Bridge (Paris), and Changi Airport Bridge (Singapore). Large-scale tests on these structures have gradually demonstrated dynamic alterations in the lateral structural vibration behavior due to HSI effects, as extensively documented by Dallard et al. [54], Zivanovic et al. [13], Dziuba et al. [55], Ingólfsson et al. [28], and Brownjohn et al. [56]. Numerous cases have been reported in the literature on HSI effects, modeling, and anthropic load testing related to footbridges, including the Wuhan Yangtze Bridge, China [57]; Le Ponte del Mare, Pescara, Italy [58], among others. Additionally, review studies have focused on the phenomena of dynamic interaction between pedestrians and structures in the vertical [20], [43], [59]–[61], and lateral [14], [17], [28], [34], [37], [43], [46], [50], [62], [63] directions. Van Nimmen et al. [24], [25] and Colmenares et al. [15] conducted the most comprehensive studies, evaluating large-scale urban footbridge: Eeklo Bridge-Belgium and The Folke Bernadotte Bridge, respectively. These structures are subjected to human loads, and have become international benchmark datasets. However, measurements of the structural responses and behavior of human-induced loads are difficult to acquire experimentally under *in-situ* conditions, which limits their widespread implementation.

In recent years, an innovative cyber-experimental technique known as real-time hybrid simulation (RTHS) has been used to assess the dynamic behavior of complex structural systems [64]–[66], in particular, to assess the influence of structural control devices on building seismic response [67]–[69]. However, this methodology has recently been implemented to evaluate non-conventional dynamic systems in structural engineering [70]–[72]. RTHSs are an experimental evaluation technique characterized by real-time scales, comprising a physical component that includes inertial effects in constant interaction with a numerical component [79]–[91], which have related communication through transfer systems (TS) that normally implement hydraulic actuators and instrumentation sensor systems. The potential, cost-effective, versatility, and relative accuracy of RTHS technique allow the concentration of resources on the assessment of the relevant components of interest within a dynamic system, which leads to reduce experimental setup and logistic cost, decreasing execution time of the tests, and increased the productivity and efficiency compared to traditional large-scale testing. Therefore, in this study, HSI effects on a footbridge with precise sensitivity in its dynamic properties affect by human-induced gait loads were evaluated using RTHS techniques. The implementation of this innovative methodology in the field of the experimental evaluation of HSI effects on footbridges allowed the division of the dynamic system into three principal components: a footbridge modeled using a fourth-order partial differential equation as numerical substructure (NS), pedestrian walking as experimental substructure (ES), and a multiaxial test framework developed and described by Castillo et. al. [29], [83], as transfer system (TS). This proposal allowed the assessment and measurement of the loads induced by human gait under laboratory conditions and simulated the complete states of the footbridge dynamic responses through a continuous model, which had an interaction in real-time to create a dynamic system with continuous feedback. The remainder of this paper is organized as follows. *Section 2* provides details of the evaluated reference footbridge, a description of the general information of the test subjects (TSs), and a test description. In *Section 3* the dynamic substructuring of the evaluation of the effects of human-structure interaction on a pedestrian bridge with precise dynamic sensitivity, the transfer systems used, and the experimental setup. A complete analysis of the dynamic performance of the reference footbridge under HSI effects of crowd-induced loads is presented and discussed in *Section 4*, in addition, the performance of RTHS technique is determinated. Finally, *Section 5* presents the conclusions of the study.

## 2. Main dynamic system

### 2.1 Footbridge with precise dynamic sensitivity

A reference footbridge (RF) with precise dynamic sensitivity in its fundamental frequency content, close to the human gait frequency (lateral and vertical frequency of 1.0 Hz and 2.0 Hz, respectively), was designed and proposed as reference structure. This structure employed structural steel with  $f_y = 210.0 \text{ GPa}$ ,  $\rho_g = 7500.0 \text{ kg} \cdot \text{m}^{-3}$ , and an equivalent cross-section. The dynamic properties of the RF were evaluated using two methods of analysis: numerical evaluation and a Finite-Elements (FE) model. Both dynamic assessments used the same cross-sectional properties, physical properties of materials, and structural damping of 1.0% for all modes. The numerical evaluation used *Eq.1* to calculate the natural frequencies of the RF based on their properties, where  $E = f_y$ ,  $I$  (specified in *Table 1* for each analysis direction),  $m_l = 400.05 \text{ kg} \cdot \text{m}^{-1}$ , and  $l = 36 \text{ m}$ ; represent the elastic module, inertial module, mass per unit length, and length of the structure, respectively. Similarly, the FE analysis modeled the RF dynamic behavior using *SAP2000 V.25.3.0*. The RF natural frequencies are reported in *Table 1* for both analysis cases, which generated a maximum frequency error of less than 2.04%, in all directions assessed.

$$f_{n,i} = (n * \pi)^2 \cdot \sqrt{\frac{f_y \cdot I}{m_l \cdot l^4}} \quad (1)$$

*Table 1. Properties of the reference footbridge.*

| Analysis Direction | Inertial Moment [m <sup>4</sup> ]     | Structural Damping [%] | Numerical Frequency [Hz] | FE Model Frequency [Hz] | Maximum Frequency Error [%] |
|--------------------|---------------------------------------|------------------------|--------------------------|-------------------------|-----------------------------|
| Lateral            | 1.00 · 10 <sup>-4</sup><br>Axis (y-y) | 1.00                   | $f_{l-N,1} = 1.04$       | $f_{l-FEM,1} = 1.06$    | 2.04 %                      |
|                    |                                       |                        | $f_{l-N,2} = 4.15$       | $f_{l-FEM,2} = 4.22$    |                             |
|                    |                                       |                        | $f_{l-N,3} = 9.35$       | $f_{l-FEM,3} = 9.48$    |                             |
| Vertical           | 3.60 · 10 <sup>-4</sup><br>Axis (x-x) | 1.00                   | $f_{v-N,1} = 2.00$       | $f_{v-FEM,1} = 2.03$    | 1.45 %                      |
|                    |                                       |                        | $f_{v-N,2} = 8.00$       | $f_{v-FEM,2} = 8.07$    |                             |
|                    |                                       |                        | $f_{v-N,3} = 18.00$      | $f_{v-FEM,3} = 17.91$   |                             |

In addition, the first three dynamic modal shapes were determined by the FE model analysis in each assessment direction, as shown in *Figure 1*. These principal modal shapes reached a cumulative modal mass of more than 95.0%, developing a representative dynamic performance of the RF in all analysis directions. The RF Dynamic behavior is closely related to the dynamic performance of the human gait, which positioned this structure with a precise dynamic sensitivity to the influence of human-induced gait loads as an international benchmark to evaluate HSI dynamic effects.

Figure 1. Modal behavior of the reference footbridge Model FE.

2.2 Human-Induced gait loads: test subjects (TS)

Fifteen volunteer test subjects (TS) (8 females and 11 males with  $24.11 \pm 4.23$  years,  $167.16 \pm 7.09$  cm of height, and  $68.10 \pm 10.51$  Kg of mass) were evaluated in this study. The general information is presented in Table 2. All TSs were free of injuries for at least 1.0 month, had a body mass index (BMI) of less than  $25.89 \text{ Kg} \cdot \text{m}^{-2}$  (average level), and were aged between 19 and 34 years. All tests involving human TSs were performed in accordance with the Code of Ethics of the World Medical Association (Declaration of Helsinki) for experiments involving humans and the Universidad del Valle Ethics Committee (Deed of Endorsement Number: 011-2023).

Table 2. General information. Identification number (ID), age, height, aproximal center mass (CM), body mass (BM), and sex (S, f→female or M→Male) for each test subject.

| ID | Age [yr] | Height [cm] | CM [cm] | BM [kg] | S |
|----|----------|-------------|---------|---------|---|
| 1  | 22.0     | 169.0       | 100.0   | 68.3    | M |
| 2  | 25.0     | 170.0       | 95.0    | 85.6    | M |
| 3  | 27.0     | 153.0       | 97.0    | 66.5    | f |
| 4  | 33.0     | 170.0       | 110.0   | 65.2    | M |
| 5  | 22.0     | 165.0       | 97.0    | 68.6    | M |
| 6  | 23.0     | 175.0       | 99.0    | 81.5    | M |
| 7  | 22.0     | 175.0       | 103.0   | 90.9    | M |
| 8  | 24.0     | 183.0       | 110.0   | 78.3    | M |
| 9  | 24.0     | 173.0       | 101.0   | 75.3    | M |
| 10 | 34.0     | 162.0       | 92.0    | 54.7    | f |
| 11 | 19.0     | 165.0       | 95.0    | 51.1    | f |
| 12 | 20.0     | 167.0       | 100.0   | 62.2    | f |
| 13 | 21.0     | 157.0       | 98.0    | 65.7    | f |
| 14 | 22.0     | 168.0       | 102.0   | 61.6    | M |
| 15 | 21.0     | 158.0       | 90.0    | 62.4    | f |

### 3. Real-Time Hybrid Simulation (RTHS)

#### 3.1. Numerical substructure (NS): Reference footbridge model

The time-history dynamic response of the reference footbridge, described in Section 2.1, under interaction with human-induced gait loads, was assessed using an analytical continuous model represented by a fourth-order partial differential equation (PDE), Eq. 2, which constitutes the numerical substructure (NS). In this equation,  $u(x, t)$  represents the deflection of any point along the  $x$ -axis of the structure for all the simple times evaluated. Likewise,  $p(x, t)$  includes human-induced gait loads as dynamic perturbations in the system that estimates the reference footbridge, where  $m(x)$  and  $c(x)$  are constant coefficients representing the mass and viscous damper of the dynamic system, respectively.

$$\frac{\partial^2}{\partial x^2} \left[ EI(x) \frac{\partial^2 u(x, t)}{\partial x^2} \right] + m(x) \frac{\partial^2 u(x, t)}{\partial t^2} + c(x) \frac{\partial u(x, t)}{\partial t} = p(x, t) \quad (2)$$

The variable separation method was used to resolve the posed equation, which describes the dynamic behavior of a reference footbridge under interaction with human gait. This solution technique incorporates modal decomposition, as shown in Eq. 3, which employs vibration models represented by sinusoidal functions as shown in Eq. 4. The dynamic motion ( $Y_{\hat{n}}(t)$ ) of the  $n$ -th in modal coordinates ( $\phi_{\hat{n}}(x)$ ) is determined using Eq. 5, where the natural frequencies ( $\omega_{\hat{n},i}$ ) and structural damping ratio ( $\xi_i$ ) correspond to the parameters described in Section 2.1 (Table 1). The mass ( $M_{\hat{n}}$ ) and modal loads ( $P_{\hat{n}}(t)$ ) were calculated using Eq. 6 and Eq. 7, respectively.

$$u(x, t) = \sum_{\hat{n}=1}^{\infty} \phi_{\hat{n}}(x) \cdot Y_{\hat{n}}(t) \quad (3)$$

$$\phi_{\hat{n}}(x) = \sin \left( \frac{\hat{n}\pi}{L} x \right) \quad (4)$$

$$\ddot{Y}_{\hat{n}}(t) + 2 \cdot \xi \cdot \omega_{\hat{n}} \cdot \dot{Y}_{\hat{n}}(t) + \omega_{\hat{n}}^2 \cdot Y_{\hat{n}}(t) = \frac{P_{\hat{n}}(t)}{M_{\hat{n}}} \quad (5)$$

$$M_{\hat{n}} = \int_0^L [\phi_{\hat{n}}(x)]^2 \cdot m(x) dx \quad (6)$$

$$P_{\hat{n}}(t) = \int_0^L \phi_{\hat{n}}(x) \cdot p(x, t) dx \quad (7)$$

$$p(x, t) = \left[ \sum_{i=1}^n m_i \cdot (\ddot{a}_i(t) - g + \ddot{a}_{s,j}(t)) \right] \cdot \delta(x - x_{fb}(t)) \quad (8)$$

Human-induced gait loads in generalized coordinates ( $p(x, t)$ ) were formulated as the sum of the inertial load of the  $n$ -th human body segment affected by the dynamic interaction of the footbridge structural acceleration effects ( $\ddot{a}_{s,j}(t)$ ), which act in the  $x_{fb}(t)$  position on the reference structure for all times evaluated, as expressed in Eq. 8. In this equation,  $g$ ,  $x$ , and  $x_{fb}$  represent the gravity acceleration (only in the  $y$ -axis), longitudinal axis of the reference structure, and current position of the anthropic loads on the footbridge, respectively. The multiaxial structural accelerations generated by the human-induced gait loads were calculated in modal coordinates using Eq. 9, which was based on the position of the anthropic load on the structure and  $n$ -th vibration modes assessed. The human-induced gait loads expressed in modal coordinates were obtained by replacing Eq. 4 and Eq. 8 in the Eq. 7 in order to obtain Eq. 10, which depends on the human-induced gait load ( $F_{human}(t)$ ), modal information, crowd modal mass, and multiaxial structural

acceleration ( $\ddot{a}_{s,j}(t)$ ). Finally, the modal coordinate equation for solving the dynamic interaction between the structural response and anthropic-induced loads, denoted and proposed by [44] as a general HSI model, is extensively described by Eq. 11, and is expressed in matrix notation by Eq. 12. In these equations,  $S_{\hat{n}}(t)$  is equivalent to dynamic modal coordinates ( $\phi_{\hat{n}}(x)$ ) proposed in Eq. 4, where  $x$  is the temporal position of the anthropic load on the reference footbridge ( $x_{fb}(t)$ ).

$$\ddot{a}_{s,j}(t) = \sum_{\hat{n}=1}^n \left[ \sin \left( \frac{\hat{n} \cdot \pi \cdot x_{fb}(t)}{L} \right) \right] \cdot \ddot{Y}_{\hat{n}}(t) \quad (9)$$

$$P_{\hat{n}}(t) = - \left[ F_{human}(t) \cdot \sin \left( \frac{\hat{n} \cdot \pi \cdot x_{fb}(t)}{L} \right) \right] - \left[ m_{crwd} \cdot \sin \left( \frac{\hat{n} \cdot \pi \cdot x_{fb}(t)}{L} \right) \right] \cdot \ddot{a}_{s,j}(t) \quad (10)$$

$$\ddot{Y}_{\hat{n}}(t) + \frac{m_{crwd}}{M_{\hat{n}}} \cdot S_{\hat{n}}(t) \cdot \left[ \sum_{m=1}^n S_m(t) \cdot \ddot{Y}_{\hat{n}}(t) \right] + 2 \cdot \zeta_{\hat{n}} \cdot \omega_{\hat{n}} \cdot \dot{Y}_{\hat{n}}(t) + \omega_{\hat{n}}^2 \cdot Y_{\hat{n}}(t) = - \frac{1}{M_{\hat{n}}} \left[ F_{human}(t) \cdot \sin \left( \frac{\hat{n} \cdot \pi \cdot x_{fb}(t)}{L} \right) \right] \quad (11)$$

$$[M + m_{crwd} \cdot S^T(t) \cdot S(t)] \cdot \ddot{Y}(t) + C \cdot \dot{Y}(t) + K \cdot Y(t) = -\Phi(x_{fb}(t)) F_{human}(t) \quad (12)$$

### 3.2. Experimental substructure (NS)

Human gait is a complex biomechanical process characterized by pseudo-cyclic movement of the human body as it progresses through space, which has a highly variable behavior. It involves the three-dimensional motion of multiple body segments, which interact through a series of joints with varying degrees-of-freedom. This dynamic system is driven by muscular forces and influenced by gravitational, external, and inertial effects. In addition, gait pseudo-cycles are typically divided into two primary phases: the stance phase, where one foot is in contact with the ground, providing support and propulsion, and the swing phase, where the opposite leg moves forward to prepare for the next step.

Human gait has been extensively modeled using different approaches in order to represent the precise and natural dynamic behavior of human activity [37], [39], [44], [84]. However, the randomness, variability, and spatiotemporal performance of the human gait on rigid surfaces is difficult because of the lack of capacity and parameterization to represent biological and biomechanical conditions computationally. Therefore, many studies have simplified the dynamic performance of human gait, which in turn reduces the reliability and accuracy of human-induced loads, particularly on vibrating surfaces. In addition, there are few models that include the dynamic interaction effects between human gait-induced loads and flexible surfaces, typically with lightweight structures, such as footbridge, thin slabs, and stairs. This dynamic feedback condition increases the requirements of analytical models, which generates greater uncertainty in the estimation of human gait-induced loads and dynamic structural responses.

In fact, human gait analytical models have limitations in simulating and evaluating the dynamic behavior of the HSI effects on footbridges. Therefore, in this study, each TS gait was assumed to be an experimental substructure with active anthropic loads ( $F_{k,j}$ ,  $k \rightarrow$  Analysis direction {vertical (blue), lateral (red), or friction (green)}, and  $j \rightarrow$  selected foot {left or right}), which were induced in the reference footbridge, as detailed in Section 3.5. Likewise, the dynamic structural response ( $\ddot{a}_{s,total}$ , total structural acceleration) feedback the experimental TS gaits, which modifies human-induced gait loads on the structure, generating a dynamic interaction in real-time, as schematically shown in Figure 2. This methodology allows the measurement and incorporation of real human gait-induced loads under dynamic HSI effects, which increases the accuracy of the structural response simulation, and acquired anthropic loads. In addition, each TS used a safety life-line to ensure the integrity and sense of security during the gait tests and was instrumented with

a pair of insole pressure sensors (Stridalyzer Insight developed) by ReTiSense [17]. Moreover, active motion markers were positioned in the body joints to measure the tridimensional displacements using two motion capture systems: sagittal and coronal planes, as experimentally shown in *Figure 3* (top). This indirect and direct measurement allows to generate a comprehensive dataset about HSI effects on reference footbridge.



*Figure 2. Experimental substructure: multiexperimental human-induced loads.*

### 3.3. Transfer system: Human-Structure Interaction Multiaxial test Framework (HSI-MTF)

Transfer systems (TrSs), as components in real-time hybrid simulations, are intended to accurately and consistently exchange information between the experimental and numerical substructures within the simulation loop. This system ensures that the dynamic interaction between the physical specimen (anthropic load induced by crowd gait,  $F_{k,j}$ ) and the computational model (dynamic response of the reference footbridge,  $\ddot{a}_{s,total}$ ) is

synchronized in real-time. In order to ensure the fidelity of the real-time synchronization requirements in the simulation, this study adopted the human–structure interaction multi-axial test framework (HSI-MTF) as the TrS prototype, which was used in previous studies of human gait on surfaces with lateral movements [17], and completely described its development by Castillo et. al. [29].



**Figure 3.** Description of the HSI-MTF

The HSI-MTF is located in the Laboratory of Seismic Engineering and Structural Dynamics at the Universidad del Valle, Cali, Colombia, and was developed to assess the multiaxial dynamic loads induced by pedestrians walking on surfaces with lateral and vertical movements. Castillo et. al. [29] decomposed the HIS-MTF, shown and detailed in *Figure 3*, in five principal components: unidirectional shake table to induce lateral movements with displacement of  $\pm 80.0\text{ mm}$ , the lateral support system to support the dynamic loads generated, a sole brand treadmill model F65 with effective action area of  $72.0\text{ cm} \times 150.0\text{ cm}$  and operating speeds ranging from  $0.50$  to  $10.0\text{ m} \cdot \text{s}^{-1}$ , a load acquisition system which was developed to measure the tridimensional anthropic loads generated by the human gait, and the vertical hydraulic system to induce vertical movements with displacement of  $\pm 75.0\text{ mm}$ . In addition, the upper section of *Figure 3* shows how the human gait is developed on HSI-MTF, and the TSs were instrumentalized to measure the vertical load wearing pressure insole and joint displacement using active motion markers of a low-cost motion capture system.

The lateral movement system implements a robust model control type  $H_\infty$  coupled with a Kalman filter, and the vertical movement system incorporates a proportional-Integral-Derivative (PID) model and an anticipatory function control. Both were integrated to ensure the fidelity, stability, and accuracy of multiaxial displacement tracking. These requirements are necessary for the proper development and stabilization of the simulation in real-time.



*Figure 4.* Closed dynamic loop to evaluate HSI effects on civil structures.

### 3.4. Experimental setup

The experimental and numerical substructures and transfer system were interrelated according to the schematic diagram shown in **Figure 4** to develop the RTHS technique. The analytical substructure was affected by the input signal, which corresponds to the human-induced measured gait loads,  $F_{human,m}(x_p, t)$ . Simultaneously, the numerical footbridge responds to this spatiotemporal load excitation according to its dynamic properties, which generates the dynamic response  $U_r(x_p, t)$ . Structural accelerations multiaxially reproduced using HSI-MTF,  $\ddot{U}_m(x_p, t)$ , were induced in the experimental test subjects, as described in *Section 3.5*. The reaction forces exerted by human gait under dynamic HSI effects were measured by an instrumentation system coupled to the HSI-MTF, which is composed of multiaxial cell loads. The human-induced gait loads,  $F_{human,r}(x_p, t)$ , represent the total feedback force,  $F_{human,m}(x_p, t)$ , of the numerical substructure, which closes the dynamic loop. This process was programmed in *Simulink* software and executed with a SpeedGoat processor using a host computer with the *Xpc* protocol of MATLAB and a sampling frequency of 1024 Hz. SpeedGoat has a high processing capacity to guarantee the real-time evaluation of the hybrid simulation.

### 3.5 Experimental input: Evaluation of HSI effects.

Gait tests were carried out in this study for each TS, which consisted in the dynamic evaluation of HSI effects due to crossing of crowd-induced gait loads over the reference footbridge to three constant typical gait velocities:  $1.20 \text{ m} \cdot \text{s}^{-1}$ ,  $1.50 \text{ m} \cdot \text{s}^{-1}$ , and  $1.80 \text{ m} \cdot \text{s}^{-1}$ . In addition, each TS induced experimental gait loads acting on the centroid of the reference footbridge, which constituted the numerical prototype, using HSI-MTF as the transfer and instrumentation system, as described in *Section 3.3*.

## 4. Results and discussion

### 4.1. Human-induced gait load performance on reference footbridge

Dynamic interactions between human activities and lightweight structural response have been reported and have significantly increased in recent years, which constitutes a risk to condition the structural serviceability due to coupling of dynamic changes in the human gait behavior and structural vibrations. Therefore, in this section, the evaluation of pedestrian-induced gait loads in direct connection with reference footbridge responses is divided into two main assessment directions: lateral and vertical loads.

#### 4.1.1 Lateral loads

The lateral gait loads induced by all TSs on a rigid surface were evaluated for 30.0 s. The induced lateral load ( $L_L$ ) was normalized with respect to the weight ( $W_0$ ) of each ST, yielding an average normalized lateral load of approximately 2.0% and a maximum of 2.9%. These human gait loads induced by TSs are presented in **Figure 5** (top-left) for a 3.0 s range and calculate the average normalized gait loads ( $\bar{L}_L/W_0$ ), with the respective standard deviation ( $\pm\sigma$ ). In addition, the lateral frequency response assessment of the anthropic loads on rigid surface determined a fundamental frequential content of  $f_1 = 1.0 \text{ Hz}$ , with relevant frequential content in  $f_2 = 2.0 \text{ Hz}$ , as shown in **Figure 5** (top-right). However, the lateral frequency response assessment of pedestrian-induced gait loads on reference footbridge showed a slight increase in the damping of the first fundamental frequency of the gait load compared to the increase in the gait velocity of the TSs (green band, **Figure 5-middle**). Moreover, a frequential content was developed by TSs close to second fundamental frequency, denoted in this study as  $f_{HSI} \cong 1.8 \text{ Hz}$  (purple band, **Figure 5-middle**), which exerts a stabilizing action on the ST during the crossing of the reference structure, proportional to the test speed. Likewise, the lateral pedestrian-induced gait

**Figure 5.** Behavior of human-induced lateral gait loads on rigid and flexible surfaces. Time history and spectral response of human-induced lateral gait loads on rigid surfaces (top); spectral response of human-induced lateral gait loads on reference footbridge for three gait velocities: 1.20 (middle-left), 1.50 (middle-center), and 1.80  $\text{m} \cdot \text{s}^{-1}$  (middle-center); and  $ST_1$ -induced lateral gait loads on reference footbridge for three gait velocities: 1.20 (bottom-center), 1.50 (bottom-left), and 1.80  $\text{m} \cdot \text{s}^{-1}$  (bottom-right).

loads on the reference structure by  $TS_1$  had a similar behavior, initiating and terminating 337  
the crossing of the structure, as shown in **Figure 5** (bottom), for all gait velocities of the 338  
 $TS$ s evaluated. However, an increase in lateral normalized pedestrian-induced gait loads 339

was identified in the middle third of the reference footbridge due to dynamic HSI effects, which increased the magnitude of the walking loads by more than 300% out of phase compared to the lateral structural response at the action point of the pedestrian. Finally, the peak lateral pedestrian-induced gait loads were determined for the three gait velocities evaluated, which reached overage-normalized gait loads and standard deviations of 9.86% and 1.33%, respectively, as presented in **Figure 6** (left).

**Figure 6.** Parametric analysis of human-induced gait loads. Peak Lateral human-induced gait loads vs gait velocity of TSs (left); and Peak vertical human-induced gait loads vs gait velocity of TSs (right).

#### 4.1.2 Vertical loads

Similarly, the vertical gait loads induced by all TSs on a rigid surface were evaluated for 30.0 s. The induced lateral load ( $L_L$ ) was normalized with respect to the weight ( $W_0$ ) of each ST, yielding an average normalized lateral load approximately of 105.0% and a maximum of 125.0%. These pedestrian-induced gait loads by TSs are presented in **Figure 7** (top-left) for a 1.5 s range, and the average normalized gait loads ( $\overline{L_L}/W_0$ ), with the respective standard deviation ( $\pm\sigma$ ). In addition, the vertical frequency response assessment of the gait loads on the rigid surface determined a fundamental frequency band around  $f_1 = 1.75$  Hz, as shown in **Figure 7** (top-right). However, the vertical frequency response assessment of pedestrian-induced gait loads on reference footbridge demonstrated that a frequential content was developed by TSs close to first fundamental frequency, called as  $f_{HSI} \cong 1.8$  Hz (purple band, **Figure 7-middle**), which allows a stabilizing action on the ST during the crossing of the reference structure, increasing its effects according to the test velocity. On the other hand, changes in vertical normalized pedestrian-induced gait load by TSs were identified at the different points of the reference footbridge due to dynamic HSI effects, which decreased ( $\approx -20.0\%$ ) or increased ( $\approx +50.0\%$ ) the anthropic loads in direct relation with the vertical structural response at the action point of the pedestrian compared to the overall gait-induced loads on the rigid surface. Finally, the peak vertical pedestrian-induced gait loads were determined for the three gait velocities evaluated, which reached overage-normalized gait loads and standard deviations of 156.56% and 15.58%, respectively.



**Figure 7.** Behavior of human-induced vertical gait loads on rigid and flexible surfaces. Time history and spectral response of human-induced vertical gait loads on rigid surfaces (top); spectral response of human-induced lateral gait loads on reference footbridge for three gait velocities: 1.20 (middle-left), 1.50 (middle-center), and 1.80  $\text{m} \cdot \text{s}^{-1}$  (middle-center); and  $ST_1$ -induced vertical gait loads on reference footbridge for three gait velocities: 1.20 (bottom-center), 1.50 (bottom-left), and 1.80  $\text{m} \cdot \text{s}^{-1}$  (bottom-right).

#### 4.2. Reference footbridge performance include HSI

The spatio-temporal dynamic behavior structural responses of the reference footbridge under HSI effects due to human-induced gait loads, described in *Section 2.1*, were experimentally evaluated using RTHS as a testing methodology, as described in *Section 3*. In order to show the complete structural response during the crossing of pedestrians (TSs), the structural displacement and acceleration under gait loads induced by TS<sub>1</sub> at the contact point between the pedestrian and footbridge and in the middle span were determined and processed, as shown in *Figure 8* and *Figure 9*. Lateral structural accelerations at the contact point for three gait velocities evaluated in time history and frequency response, which determined an inverse relation between structural response times and gait velocity of the TS. Likewise, the fundamental frequency that composes the structural response was close to 1.0 Hz, as shown in *Figure 8* (top-left). A similar behavior was observed when evaluating the lateral structural displacement at the contact point of the ST<sub>1</sub> and its frequency response (*Figure 8*, top-right). In addition, the lateral structural displacement and acceleration in the structural middle span exhibited frequential behavior similar to the fundamental frequency around to 1.0 Hz and a free vibration behavior of the footbridge was identified once the pedestrian left the structure, which allowed the identification of the structural damping, as presented in *Figure 8* (bottom).

**Figure 8.** General lateral structural response of the reference footbridge under human-induced gait loads with HSI effects by ST<sub>1</sub>. Lateral structural acceleration in the contact point of the ST<sub>1</sub> and frequential response (top-left); Lateral structural displacement in the contact point of the ST<sub>1</sub> and frequential response (top-right); Lateral structural displacement in the middle span and frequential response (bottom-left); and Lateral structural acceleration in the middle span and frequential response (bottom -right).

Similarly, vertical structural accelerations at the contact point for three gait velocities were evaluated in time history and frequency response, which determined a direct relation between structural response times and gait velocity of the TS<sub>1</sub>. Likewise, the fundamental frequency that composes these structural responses was close to 2.0 Hz, as shown in *Figure 9* (bottom -left). A similar behavior was observed when evaluating the vertical structural displacement at the contact point of ST<sub>1</sub> and its frequency response (*Figure 9*, bottom-right). Additionally, the vertical structural displacement and acceleration in the

**Figure 9.** General vertical structural response of the reference footbridge under human-induced gait loads with HSI effects by  $ST_1$ . Vertical structural acceleration in the middle span and frequential response (top-left); Vertical structural displacement in the middle span and frequential response (top-right); Vertical structural displacement in the contact point of the  $ST_1$  and frequential response (bottom-left); and Vertical structural in the contact point of the  $ST_1$  and frequential response (bottom -right).

structural middle span exhibited frequential behavior similar to the fundamental frequency around to 2.0 Hz, with some content frequency due to dynamic HSI effects (close to 1.80 Hz) and harmonic behavior (around 4.0 Hz). The free vibration behavior of the footbridge was identified once the pedestrian left the structure, which allowed identification of the structural damping assigned, as presented in **Figure 9** (top).

The assessment of the peak dynamic response of the reference footbridge using was carried out using a hybrid spider-violin plot for each gait velocity evaluated by TSs is shown in **Figure 10**. This schematic analysis allows identified, for  $n$  independents events (each branch), the normal Probability density function (PDF), using **Eq. 13**, of the peak dynamic responses ( $P_{SR}$ ) normalized by the maximum value in the respective event. This equation uses the average ( $\mu_{P_{SR}}$ ) and standard deviation ( $\sigma_{P_{SR}}$ ) of the normalized peak structural response generated during the gait tests on the reference footbridge, including the dynamic HSI effects using the RTHS technique.

$$PDF_{P_{SR}} = f(P_{SR} | \mu_{P_{SR}}, \sigma_{P_{SR}}) = \frac{1}{\sigma_{P_{SR}} \sqrt{2\pi}} e^{-\frac{(P_{SR} - \mu_{P_{SR}})^2}{2\sigma_{P_{SR}}^2}}, \text{ for } \forall P_{SR} \in \mathbb{R} \quad (13)$$

Each branch presented the respective PDF shape with punctual events that were composed, and the average structural response was identified. In addition, the colored shadow represents the first quantile of the standard deviation of the data points that compose the specific event. This approach allowed the identification of a higher dispersion in the vertical displacement of the middle span of the structure and contact point of the TSs compared to the same response in the lateral direction, with percentage differences of mean normalized values of less than 10.0% for all gait velocities, as shown in **Figure 10** (top). Likewise, a similar behavior was identified with the lateral and vertical accelerations; however, the dispersion of the vertical acceleration in the middle span of the structure and contact point of the TSs was significant because of the dynamic HSI effects calculated in the frequency response. Moreover, the percentage differences of the mean normalized values of accelerations in the middle span of the structure and contact point of the TSs were approximately of 15.0% for all gait velocities, which were always major for the lateral direction evaluated, as shown in **Figure 10** (bottom).

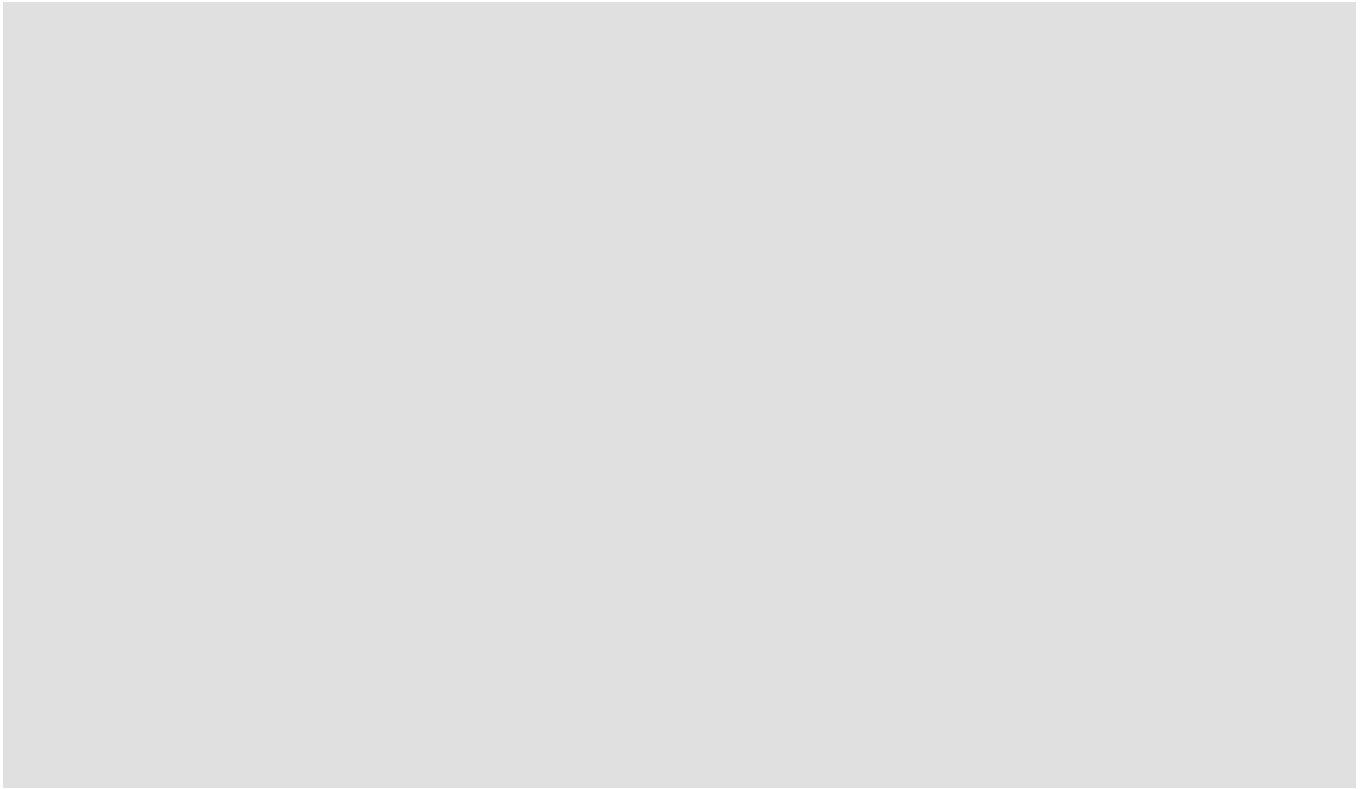


**Figure 10.** Assessment of peak dynamic response of the reference footbridge using hybrid spider-violin plot. Normalized general structural displacements for three gait velocities:  $1.20 \text{ m} \cdot \text{s}^{-2}$  (top-left),  $1.50 \text{ m} \cdot \text{s}^{-2}$  (top-middle), and  $1.80 \text{ m} \cdot \text{s}^{-2}$  (top-right); Normalized general structural acceleration for three gait velocities:  $1.20 \text{ m} \cdot \text{s}^{-2}$  (bottom-left),  $1.50 \text{ m} \cdot \text{s}^{-2}$  (bottom-middle), and  $1.80 \text{ m} \cdot \text{s}^{-2}$  (bottom-right).

#### 4.3. RTHS performance

RTHS introduces errors due to factors inherent to its methodology, such as discrepancies between the numerical simulation (NS) model and the physical structure, electronic noise in the experimental signals of the experimental substructure (ES), and the dynamics of actuators within the transfer systems, among others [85], [86]. Consequently, various indices have been proposed to assess the accuracy of RTHS, accounting for potential deviations between the reference and experimental signals. Many of these evaluation metrics emphasize the local response during hybrid simulations (HS), particularly focusing on the physical component and its interaction with its numerical counterpart. The evaluation of local performance was based on the synchronization of the boundary conditions between the numerical response ( $u_{i,d}^*$ ) sent to the actuator and the displacement achieved in the physical component ( $u_{i,m}^*$ ). The normalized root mean square in the experiment (NRMSE) can be used, according to Eq.14, to obtain a single value representing the difference between the actuator signals in the time domain. In addition, frequency domain comparison, Eq.15 to Eq.19 are used, corresponding to the frequency evaluation index (FEI), which compares the fast Fourier transform (FFT) of the desired signal  $u_{i,d}^*$  with that of the measured  $u_{i,m}^*$  at the actuator, the dominant frequency during RTHS  $f^{eq}$ , the generalized amplitude  $A_0$ , and the delay  $\delta$ , respectively [87].

$$NRMS_{error} = \sqrt{\frac{\sum_{j=1}^N [u_{i,d}^*(j) - u_{i,m}^*(j)]^2}{\sum_{j=1}^N [u_{i,d}^*(j)]^2}} \quad (14)$$

$$FEI = \sum_{j=1}^N \left( \frac{u_{i,m}^*(j)}{u_{i,d}^*(j)} \cdot \frac{\|u_{i,d}^*(j)\|^l}{\sum_{j=1}^P \|u_{i,d}^*(j)\|^l} \right) \quad (15)$$

$$f^{eq} = \frac{\sum_{j=1}^N (\|u_{i,d}^*(j)\|^l \cdot f_i)}{\sum_{j=1}^N \|u_{i,d}^*(j)\|^l} \quad (16)$$

$$A_0 = \|FEI\| \quad (17)$$

$$\emptyset = \arctan \left[ \frac{Im(FEI)}{Re(FEI)} \right] \quad (18)$$

$$\delta = -\frac{\emptyset}{2 \cdot \pi \cdot f^{eq}} \quad (19)$$

In addition, the global indices consider the interaction between the numerical and experimental substructures, and they were evaluated in the time domain, quantifying the largest difference between the calculated and measured displacements using  $e_{DM}$  and  $e_{MD}$  as shown in Equations 20 and 21 [85].

$$e_{DM} = \frac{||u_{i,d}^*|^{max} - |u_{i,m}^*|^{max}|}{|u_{i,m}^*|^{max}} \quad (20)$$

$$e_{MD} = \frac{|u_{i,d}^* - u_{i,m}^*|^{max}}{|u_{i,m}^*|^{max}} \quad (21)$$

All indexes were calculated for all test subjects and all gait velocities evaluated. The mean values per each velocity are listed in **Table 3**. All tests developed a high level of horizontal tracking, with a generalized amplitude  $A_0$  around to 1.0, and global errors lower than 2.95%, which allowed us to infer that the vertical and lateral transfer system, integrated in HSI-MTF, has high performance and allows performing RTHS with a high level of accuracy, which demonstrates its optimal performance during the RTHS to simulate general HSI effects.

**Table 3.** RTHS evaluation indexes: vertical and lateral transfer systems.

| Test                     | $NRMS_{error}$ [%] | $FEI$          | $f_{eq}$ [Hz] | $A_0$      | $e_{DM}$ [%] | $e_{MD}$ [%] | $\delta$ [ms] |
|--------------------------|--------------------|----------------|---------------|------------|--------------|--------------|---------------|
| Vertical Transfer System |                    |                |               |            |              |              |               |
| 1.20 $m \cdot s^{-1}$    | 2.910              | 0.985 - 0.012i | 2.065         | 0.985      | 0.540        | 2.930        | 2.010         |
|                          | $\pm 0.80$         | $\pm 0.01$     | $\pm 0.03$    | $\pm 0.01$ | $\pm 0.30$   | $\pm 0.30$   | $\pm 0.10$    |
| 1.50 $m \cdot s^{-1}$    | 2.900              | 0.987 - 0.013i | 2.063         | 0.987      | 0.560        | 2.940        | 2.045         |
|                          | $\pm 0.11$         | $\pm 0.01$     | $\pm 0.04$    | $\pm 0.01$ | $\pm 0.20$   | $\pm 0.30$   | $\pm 0.10$    |
| 1.80 $m \cdot s^{-1}$    | 2.850              | 0.989 - 0.012i | 2.065         | 0.989      | 0.560        | 2.880        | 2.085         |
|                          | $\pm 0.16$         | $\pm 0.01$     | $\pm 0.03$    | $\pm 0.01$ | $\pm 0.30$   | $\pm 0.30$   | $\pm 0.10$    |

| Lateral Transfer System |            |                |            |            |            |            |            |
|-------------------------|------------|----------------|------------|------------|------------|------------|------------|
| 1.20 $m \cdot s^{-1}$   | 1.450      | 1.005 - 0.019i | 1.034      | 1.005      | 0.310      | 1.860      | 1.700      |
|                         | $\pm 0.43$ | $\pm 0.01$     | $\pm 0.01$ | $\pm 0.01$ | $\pm 0.28$ | $\pm 0.40$ | $\pm 0.30$ |
| 1.50 $m \cdot s^{-1}$   | 1.380      | 1.004 - 0.011i | 1.045      | 1.004      | 0.310      | 1.770      | 1.600      |
|                         | $\pm 0.36$ | $\pm 0.01$     | $\pm 0.01$ | $\pm 0.01$ | $\pm 0.29$ | $\pm 0.37$ | $\pm 0.20$ |
| 1.80 $m \cdot s^{-1}$   | 1.350      | 1.002 - 0.011i | 1.059      | 1.002      | 0.240      | 1.670      | 1.600      |
|                         | $\pm 0.19$ | $\pm 0.01$     | $\pm 0.02$ | $\pm 0.01$ | $\pm 0.15$ | $\pm 0.18$ | $\pm 0.20$ |

5. Conclusions

In this study, a dynamic performance assessment of a reference structure with accurate multi-axial dynamic sensitivity include human-structure interaction effects was performed using the RTHS technique for three gait velocities of the pedestrian: 1.20, 1.50, and 1.80  $m \cdot s$ . The main conclusions are as follows.

- (1) The assessment of human-induced gait loads on the reference footbridge revealed significant interactions between pedestrian activities and the structural responses. Both lateral and vertical loads exhibited dynamic coupling, with frequency content closely aligned to the footbridge's natural frequencies, particularly in the range of 1.0 Hz and 1.8 Hz. This dynamic synchronization, particularly during higher gait velocities, amplified structural vibrations, especially in the middle span, leading to increased loading by up to 300.0% in lateral directions. Vertical loads showed similarly variation, with substantial amplification and attenuation depending on the gait velocity and footbridge position. These findings highlight the critical role of Human-Structure Interaction (HSI) in dynamically sensitive structures, underscoring the importance of accounting for these interactions in the design and evaluation processes to ensure serviceability and safety under regular pedestrian use.
- (2) The performance analysis of the reference footbridge under Human-Structure Interaction (HSI) effects demonstrated significant dynamic responses in both the lateral and vertical directions, particularly at higher pedestrian velocities. Lateral accelerations and displacements exhibited a fundamental frequency around 1.0 Hz, whereas vertical responses peaked near 2.0 Hz. The coupling of pedestrian-induced loads with footbridge dynamic properties led to increased vibration amplitudes, especially in the middle span. Lateral accelerations showed a dispersion of approximately 15.0%, whereas vertical accelerations exhibited higher variability, with dispersions reaching up to 20.0%. Moreover, the percentage difference between lateral and vertical displacements was less than 10.0% for all evaluated gait velocities. These findings underscore the critical importance of integrating HSI considerations into footbridge design because human activity can induce substantial variations in structural performance, potentially affecting serviceability and user comfort.
- (3) The experimental assessment was performed using the RTHS technique, taking an analytical continuous model (through fourth-order partial differential equation) of the reference structure with accurate multi-axial dynamic sensitivity as a numerical substructure, and the gait of the test subjects as an experimental substructure. All cases had high-level tracking for the horizontal and vertical transfer systems, which developed a generalized amplitude  $A_0$  around to 1.0, and global errors lower than 2.95%. Furthermore, HSI-MTF had delays of less than 2.10 ms in all directions evaluated with an NRMS of less than 2.95%, employing a robust  $H_\infty$  type controller in the lateral axis and a PID setup with a phase anticipator in

the vertical axis. The above results indicate that the RTHS exhibits high fidelity and accuracy.

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**Institutional Review Board Statement:** This study was approved by the Ethics Committee of the Universidad del Valle (Acta de Aval No. 011-2023) and all participants signed an informed consent form before the measurements and tests.

**Informed Consent Statement:** Informed consent was obtained from all subjects involved in the study.

**Data Availability Statement:** The raw data are available from Mendeley Data with DOI: <https://doi.org/10.17632/bfdtdmtwf7.1> (accessed on 08 September 2024).

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### 3.5 Conclusions.

- 1) **Develop the experimental setup of the Integral Testing Framework for HSI to implement RTHS:** The implementation of the Integral Testing Framework for Human-Structure Interaction (HSI) has proven to be a critical tool in the dynamic evaluation of pedestrian bridges, allowing for precise characterization of the interaction between pedestrian-induced loads and the structural response. The use of the experimental setup, based on the combination of real-time testing and field data, has provided a solid approach for simulating and analyzing the dynamics of pedestrian bridges under real service conditions. This experimental framework establishes a robust foundation for the implementation of Real-Time Hybrid Simulations (RTHS), enabling a detailed study of the effects of multiaxial interaction on flexible structures.
- 2) **Develop reference pedestrian bridge models as the Numerical Substructure (NS):** The development of detailed numerical models of reference pedestrian bridges has been key to accurately representing the structural response to pedestrian-induced dynamic loads. These models, calibrated using data obtained from Operational Modal Analysis (OMA) and validated through comparisons with experimental results, have allowed for the development of Numerical Substructures (NS) capable of effectively capturing structural dynamics. These numerical substructures have served as a fundamental component in RTHS, facilitating the evaluation of pedestrian bridges under various load conditions and providing an accurate representation of HSI effects
- 3) **Conduct an experimental campaign of human walking tests on reference pedestrian bridges using RTHS:** The experimental campaign of human walking tests conducted on reference pedestrian bridges using RTHS has been essential for a deep understanding of the interaction between pedestrians and structures. The experimental results have confirmed the relevance of considering the multiaxial dynamic sensitivity of pedestrian bridges, revealing that both vertical and lateral accelerations are significantly affected by pedestrian load density. This campaign has validated the ability of RTHS to accurately capture structural dynamics and pedestrian-induced responses, providing a robust experimental framework for future research on Human-Structure Interaction.
- 4) **Evaluate the effects of HSI on reference pedestrian bridges:** The evaluation of Human-Structure Interaction (HSI) effects on reference pedestrian bridges has provided a comprehensive understanding of how pedestrian loads influence structural response, especially in terms of serviceability and comfort. The results have shown that pedestrian-induced vibrations can significantly affect the serviceability of bridges, highlighting the importance of integrating multiaxial models to accurately capture these effects. The



analysis using RTHS has allowed for real-time assessment of these effects and has proposed design improvements to mitigate service limitations and enhance safety and comfort for pedestrian bridge users.

- 5) Quantify the performance of RTHS:** Real-Time Hybrid Simulations (RTHS) have proven to be a powerful and effective tool for evaluating the performance of pedestrian bridges under pedestrian-induced dynamic loads. Through the integration of accurate numerical models and real-time testing, the dynamic behavior of the structures has been quantified in detail, allowing the identification of critical effects such as increased accelerations and changes in damping. RTHS has demonstrated its potential to replicate real service conditions with high precision, validating its applicability as an advanced methodology for the design and evaluation of pedestrian bridges.



## **Chapter No.4. Seismic Performance Assessment of Reference Civil Structures and Thin Reinforced Concrete Wall Buildings with Structural Control Devices Using Real-Time Hybrid Simulations.**

### ***4.1 Seismic Performance Evaluation of a Hybrid Structural Control System Integrating Passive and Semi-Active Devices Using Real-Time Hybrid Simulation.***

In recent years, structural control systems have become essential for improving the seismic resilience of civil structures, particularly in earthquake-prone regions. Hybrid control systems, which combine passive and semi-active components, have demonstrated significant potential in enhancing structural performance during seismic events. This study presents a comprehensive evaluation of an innovative hybrid control system that integrates unconnected fiber-reinforced elastomeric isolators (U-FREIs) with variable orifice dampers (VODs). The seismic performance of this system was assessed using Real-Time Hybrid Simulation (RTHS) on a low-degree-of-freedom (DOF) structure, subjected to six different seismic events, including notable earthquakes like El Centro and Loma Prieta.

The main objective of this research was to investigate the effectiveness of the hybrid control system in reducing excessive displacements and improving the overall structural response under seismic loads. The RTHS methodology, which partitions the system into numerical and experimental substructures, enabled real-time interaction between the structure and the control system, providing a detailed and accurate performance assessment.

Key findings from the study reveal that the hybrid control system reduced the maximum structural drift by over 24.0% compared to a fixed-base structure, showcasing significant improvements in seismic performance. The combination of passive and semi-active control states in the hybrid system allowed for greater adaptability to varying seismic excitations. Specifically, the semi-active mode of the VODs enabled real-time adjustments to optimize damping forces, effectively minimizing base displacement and overall drift in the structure.

This research “*Comprehensive Assessment of the Seismic Performance of an Innovative Hybrid Semiactive and passive state control system for a Low-Degree-of-Freedom structure using Real-Time Hybrid Simulation*” offers a valuable contribution to the field of seismic engineering by providing an in-depth analysis of the performance of hybrid control systems that incorporate both passive and semi-active elements. The hybrid control system proved to be highly effective in dissipating seismic energy and reducing the dynamic response of the reference structure, highlighting its potential for mitigating seismic risks in both conventional and hybrid structural configurations. This study demonstrates the significant advantages of hybrid real-time control systems in enhancing the seismic resilience of existing and future civil structures.

## Research Article

# Comprehensive Assessment of the Seismic Performance of an Innovative Hybrid Semiactive and Passive State Control System for a Low-Degree-of-Freedom Structure Using Real-Time Hybrid Simulation

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The dynamic performance of base isolation systems provides an effective means to reduce the seismic vulnerability of buildings. However, large displacements in the base isolation during seismic events pose a major challenge for this control system. Recently, novel control systems have been implemented to control and reduce the excessive displacements. This study presents an evaluation of the seismic performance of unconnected-fiber-reinforced elastomeric isolators (U-FREI) and variable orifice damper (VOD) devices used together as a hybrid control system for a low-degree-of-freedom reference structure by employing real-time hybrid simulation (RTHS) testing technique. This physical cybernetic testing methodology allows a dynamic system of interest to be substructured into two components: numerical and experimental. In this study, the numerical substructure corresponds to a 2-degree of freedom model, representing a reference structure, and an experimental substructure was formed using the hybrid control system described. To develop an interface between the two substructures, a horizontal transfer system (HTS) was implemented and coupled with a vertical transfer system (VTS). Using RTHS, the performance of the reference structure and the hybrid control system was evaluated under six seismic events: CAM, CAT, El Centro, Loma Prieta, Pizarro, and Chihuahua, at a maximum acceleration of 0.10 g ( $0.981 \text{ m}\cdot\text{s}^{-2}$ ). The experimental behavior of the hybrid control system allows the reference structure to reduce its maximum drift by more than 24% compared to the fixed structure. Finally, the performance and accuracy of the RTHS tests were calculated.

## 1. Introduction

In recent years, structural engineering has focused on developing and implementing techniques to reduce seismic vulnerability in structural systems, while encouraging resilient designs that prioritize occupant safety [1–3]. Structural control systems have been extensively investigated and implemented in response to these performance requirements, improving the seismic response of civil structures using external devices [4], such as passive control systems [5–17], active control systems [18–22], semiactive control systems [23–27], and more commonly hybrid control systems [28–36]. Passive control devices have been

the most widely adopted in buildings because of their lower installation and maintenance costs, lack of energy requirements, and low capacity to cause structural damage. In this sense, base isolator systems have become a reliable and cost-effective alternative for improving the seismic performance of civil structures [14, 16], creating a low-rigidity horizontal level between the structure and its foundation. This concentrates the deformation generated by seismic loads and reduces the energy induced in the structure, thereby reducing structural damage, repair costs, and the risk of loss of building functionality [4, 8]. Steel-reinforced multilayer elastomeric isolators (SREIs) are the most widely used base isolation devices. However, owing to

manufacturing techniques [37], the cost of importing [5], and the complexity of the required tests, there have been significant limitations for the widespread implementation of this technology in developing countries. Unconnected fiber-reinforced elastomeric isolators (U-FREIs) have been presented as an alternative to SREI device limitations, removing the mechanical connection to the structure and foundation and replacing steel-reinforcing plates with fiber laminates with similar mechanical characteristics [38], thereby reducing the weight and cost of the devices [39]. Pauletta et al. [40–42] conducted experimental tests to assess the dynamic behavior of U-FREI devices, while Spizzuoco et al. [43] and Madera et al. [5] conducted similar tests under controlled laboratory conditions. Losanno et al. [17, 44] implemented U-FREI technology as a base isolation system in a symmetrical 2-DOF steel-concrete structure, a methodology subsequently replicated by Riascos et al. [45]. Recently, Losanno et al. [46] and Calabrese et al. [47] employed U-FREI devices as base isolation systems aimed at reducing seismic vulnerability in unreinforced masonry (URM) buildings.

U-FREI devices, as a base isolation system, have limited seismic performance because of excessive lateral displacements, which require large areas of development to avoid damage to adjacent structures and generate additional costs to the technology [35, 36]. Viscous fluid damper (VFD) devices have been incorporated into several hybrid control systems to control excessive displacements in isolated buildings [30, 32–36, 48, 49], reducing the extensive protection zones. A notable feature of these control systems is the utilization of viscous dampers in the passive state [30, 33, 34] and magnetorheological dampers for semiactive systems [32, 35, 48, 49]. Variable orifice dampers (VODs) are a type of VFD that allow a wide range of controllability of the level of damping provided in civil structures, enabling a comprehensive range of hybrid control systems in passive or semiactive states [50]. Wongprasert and Symans [36] conducted an experimental evaluation of a semiactive hybrid control system for a 3-DOF steel building using an elastomeric base isolator system and VFD devices, which have traditionally been used as semiactive control devices in civil structures [30]. In this context, a systematic comparison between the passive and semiactive states of a hybrid control system composed of U-FREI devices as the base isolation system and VOD devices, as was carried out in this study, has not been reported in the literature.

The evaluation of hybrid control systems composed of base isolation and VFD has been primarily conducted on large-scale structures [51–53], which have limited their experimental assessment under real seismic performance conditions because of the lack of required equipment and excessive costs associated with constructing the test prototype. Therefore, the evaluation of this type of control system tends to be conducted numerically. In this sense, studies conducted by Dey et al. [32] and Losanno et al. [16, 17, 44] on low-degree-of-freedom structures permit resource optimization, structural response simplification,

and a detailed experimental assessment of the seismic performance of the structural control devices used. In recent years, an innovative experimental testing methodology known as real-time hybrid simulation (RTHS) has been implemented to evaluate the dynamic performance of complete structural systems [54]. This approach was derived from methodologies employed in aerospace engineering testing [55] and was initially introduced in the field of structural engineering as an online test or hybrid simulation (HS). The methodology involves an experimental assessment of dynamic systems characterized by extended time scales, comprising a physical component devoid of inertial effects in continuous interaction with an analytical component [56, 57], which generates a communication interface through transfer systems, typically implementing hydraulic actuators and sensors. In the RTHS, the substructuring concept proposed in the HS is adapted to real-time time-scales. This entails testing the most critical or interesting segment of the system, which typically exhibits nonlinear behavior when incorporating inertial effects [58, 59]. This testing methodology is particularly suitable for the evaluation of highly nonlinear devices such as base isolation systems or viscous fluid dampers. Experimental evaluations of base isolation systems as experimental substructures were performed by Lanese et al. [60] and Nakata et al. [61]. Recently, Riascos et al. [45] performed an RTHS test with U-FREI devices, taking the upper structure as a numerical substructure. Furthermore, viscous fluid dampers as experimental substructures have been extensively studied using RTHS [62–67]. Nevertheless, the behavior of VOD in passive and semiactive states has not been assessed to date using this innovative testing technique.

The versatility, potential, and acceptable accuracy of RTHS enable the concentration of resources on the evaluation of the specific components of interest within a system, which leads to reduced experimental setup costs, increased execution times, and the alleviation of capacity constraints often encountered in large-scale traditional dynamic experiments [68]. Therefore, in this study, the seismic performance of an innovative hybrid passive and semiactive state control system composed of U-FREI and VOD devices was evaluated for the low-degree-of-freedom structure proposed by Magliulo et al. [69] using an RTHS. This structure has become a reference structure when studying control systems, but it should be noted that a U-FREI-VOD hybrid control system has never been tested on this structure. The remainder of this paper is organized as follows. Section 2 provides details of the assessed reference structure and a description of U-FREI and VOD devices. In Section 3, the dynamic substructuring of the reference structure with a hybrid control system, transfer systems employed, experimental setup, and seismic excitations are described. Section 4 provides a detailed analysis of the seismic performance of the U-FREI and VOD used as a hybrid control system for the reference structure. In addition, it assesses the performance of the RTHS. Finally, Section 5 presents the conclusions of the study.

## 2. Main Structure

**2.1. Reference Structure.** The reference structure used, shown in Figure 1, was built at the University of Naples Federico II, Italy, with the objective of performing several seismic shake table tests on this structure to evaluate the seismic performance of base isolation systems [16, 17, 44, 45, 69–71]. This prototype had a scale factor of 1:3 for a total height of 2900 mm and a plan area ( $X$ - $Y$ ) of  $2150 \times 2650$  mm. The structural system used was a heat-formed square hollow section steel, with dimensions of  $120 \times 120 \times 12.5$  mm, and columns-welded hollow section steel with dimensions of  $150 \times 150 \times 15$  mm. The upper-level beams were bolted to the columns, whereas the lower-level was considered a rigid node. Concrete blocks were added at both levels to provide a realistic weight to the structure. The structure has a mass of  $m_1 = 3571$  kg at the lower level and a mass of  $m_2 = 4155$  kg at the upper level. The dynamic properties of the structure when it is fixed to the ground were taken from the results obtained in [69], which describe a stiffness  $K_f = 2.369 \cdot 10^6$  N·m<sup>-1</sup>, a damping coefficient  $C_f = 1.984 \cdot 10^3$  N·s·m<sup>-1</sup>, and a damping ratio  $\zeta = 1.0\%$ . The fundamental frequency of the fixed structure in the  $x$ -direction is  $f_f = 3.810$  Hz.

This reference structure has been used as an experimental test prototype and has been extensively detailed in the literature in recent years for the evaluation of structural control devices. Losanno et al. [17, 44] assessed the seismic behavior of U-FREI devices when subjected to bidirectional excitation, while the reference structure was supported by them, and concluded that these devices have significant potential in terms of reduction of the seismic vulnerability of the structure and cost-effectiveness of production using low-cost materials. Riascos et al. [45] conducted an RTHS using a numerical model of this structure with the aim of verifying the importance of considering the rocking effect in the behavior of U-FREI devices and concluded that considering this effect represents an increase in the isolator and structure displacement; however, it represents a reduction in the fundamental frequency of the structure and a significant increase in the isolation capacity. Calabrese et al. [70] performed a comparison between the results obtained in a shake table test and the results of an RTHS test, concluding that the RTHS is highly effective in validating the behavior of U-FREI devices supporting these types of structures. Magliulo et al. [69] conducted several shake table tests on this structure to obtain its dynamic properties. Subsequently, they tested the influence of gypsum board walls installed between the columns of the structure and concluded that these types of partitions do not contribute to the structural stiffness nor structural period; however, the study showed an increase in terms of damping, which has a beneficial effect on earthquake response.

**2.2. Hybrid Structural Control System.** The hybrid structural control system used in this study is composed of four U-FREI devices and two VOD devices, as shown in Figure 1. Each U-FREI device consists of 15.0 layers of rubber fiber, which are 2.0 mm thick; these layers were reinforced by 14.0

layers of 1.0 mm thick polyester fibers. The isolators had a total height of 44.0 mm and a diameter of 80.0 mm. Recently, Riascos et al. [45] conducted the RTHS tests mentioned in Section 2.1, using the same isolators. In addition, the parameters of the Bouc–Wen model that describe these devices numerically are calculated. These parameters were adopted in this study for the RTHS tests due to the testing framework for base isolators that have a capacity of only two specimens. The Bouc–Wen model is used to calculate the performance of 2 or 4 U-FREI devices, depending on the RTHS architecture. These architectures are explained later in this paper. The design, manufacturing, and validation processes of U-FREI devices can be found in [5]. In contrast, VOD devices are double-rod dampers with an external cylinder with a 210 mm stroke and an internal diameter of 25.40 mm. The rod had a diameter of 12.70 mm on each side of the piston and a total length of 700.0 mm. This damper is integrated with a needle valve that controls the flow of oil, which allows the specification of the nonlinear damping force. In addition, this device is equipped with a relief valve system that protects the seals and gate stem from excessive high pressures, which could damage the device components.

The damper has two operating modes: passive and semiactive. In passive mode, the valve opening level was selected and remained constant throughout the test. However, in semiactive mode, the valve opening changes based on the optimum force conditions required to reduce the base displacements, focusing on LQG (linear-quadratic-Gaussian) control [72]. The LQG controller implemented, shown in Figure 2, is fed with the measured displacement  $u_{1,m}(t)$  and this controller calculates the optimal total force  $F_{D\_opt}(t)$  that the two VOD devices should provide to the structure to minimize drifts. This force is divided by two in order to calculate the optimal force  $F_{opt}(t)$  that each VOD has to supply, and the absolute value of the force is calculated as  $|F_{opt}(t)|$ . The force error  $e_f(t)$  compares in real-time  $|F_{opt}(t)|$  and the absolute value of the measured force  $|F_{D,m}(t)|$  and feeds jointly with the measured force  $F_{D,m}(t)$  the controller law that calculates the motor signal  $u_{motor}(t)$  which rotates the motor that controls valve's closure. The motor controller law evaluates the force the VOD applies to be between the limits allowed. Finally,  $F_{D,m}(t)$  is multiplied by 2 to obtain the total force produced by both VOD devices  $F_D(t)$ , and this force feeds back the LQG controller and closes the loop. The device features a rotation servo motor with dimensions of  $54 \times 44 \times 20$  mm and a minimum torque of 550 N·mm, designed to control the opening of the valve. This servo motor was installed on top of the valve using a 32-hole coupling piece that allowed position control. An optocoupler encoder FC-03 was utilized to control the rotation of the servo motor. The electromechanical operation was controlled using a microprocessor.

## 3. Real-Time Hybrid Simulation (RTHS)

**3.1. Numerical Substructure (NS).** The steel frame described in Section 2.1 was considered as the numerical substructure (NS), which does not include the hybrid control system. The behavior in the  $x$ -direction was analyzed using an analytical

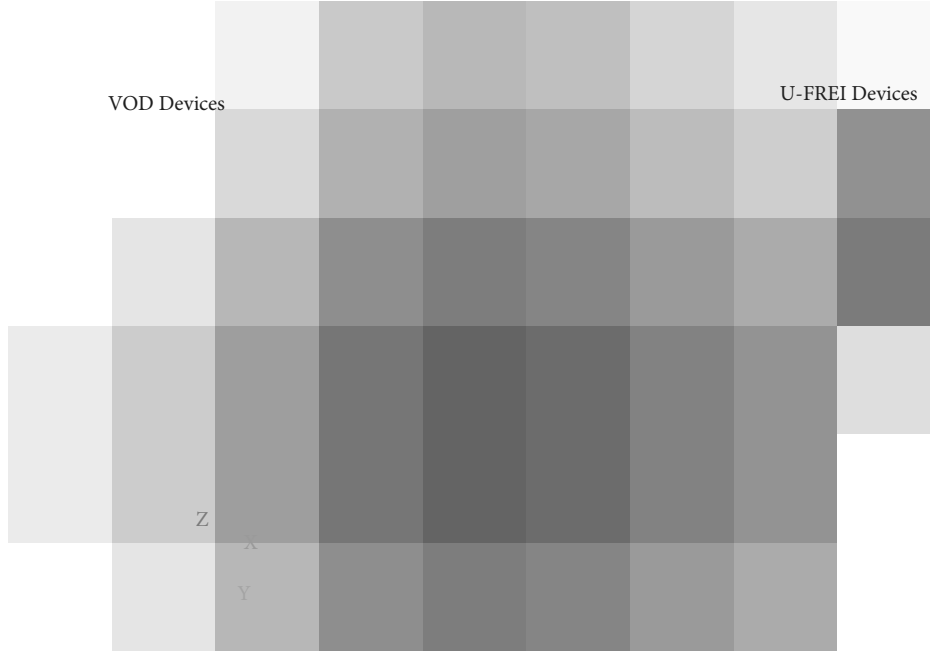


FIGURE 1: Scheme of the reference structure with a hybrid control system.

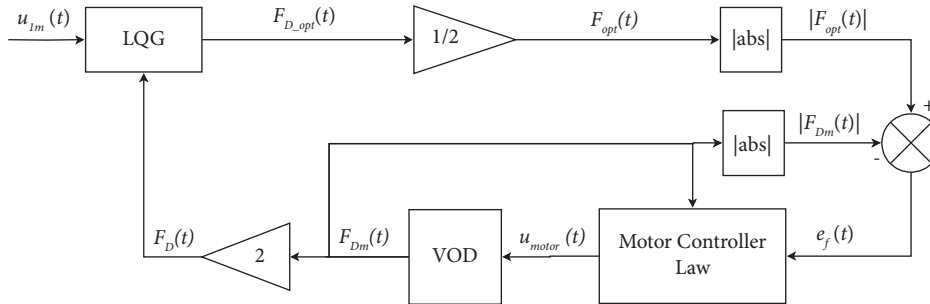


FIGURE 2: Scheme of the LQG control.

2-DOF model. The dynamic performance of this structure is described by equation (1), where  $M_r$ ,  $C_r$ , and  $K_r$  are the mass, damping, and stiffness matrices, respectively, and  $\ddot{x}$ ,  $\dot{x}$ , and  $x$  are the relative acceleration, velocity, and displacement vectors, respectively. The dynamic substructuring of the system allowed the extraction of the properties of the hybrid control system, as shown in equation (2), where the current matrices  $M_n$ ,  $C_n$ , and  $K_n$  represent only the reference structure. The term  $\Gamma_{feed} \cdot F_{feed}$  represents the restoring force of the experimental specimens (VOD and U-FREI

devices), with the influence vector  $\Gamma_{feed} = [1 \ 0]^T$  that indicates the DOF where this load was applied. Finally,  $\ddot{a}_g$  represents the ground acceleration and the influence vector  $\Gamma_g = [1 \ 1]^T$  indicates the DOF it influences. The complete system is expressed as a state-space model according to equations (3) and (4), which define a dynamic multiple-input multiple-output (MIMO) system with three inputs ( $\ddot{a}_g$  and  $F_{feed}$  of the VOD and U-FREI devices) and 2.0 outputs  $U$ , corresponding to the displacements in each DOF.

$$M_r \cdot \{\ddot{x}\} + C_r \cdot \{\dot{x}\} + K_r \cdot \{x\} = M_r \cdot \Gamma_g \cdot \ddot{a}_g, \quad (1)$$

$$M_n \cdot \{\ddot{x}\} + C_n \cdot \{\dot{x}\} + K_n \cdot \{x\} = -M_n \cdot \Gamma_g \cdot \ddot{a}_g - \Gamma_{\text{feed}} \cdot F_{\text{feed}_{\text{UFREI}}} - \Gamma_{\text{feed}} \cdot F_{\text{feed}_{\text{VOD}}}, \quad (2)$$

$$\begin{aligned} \begin{Bmatrix} \dot{x} \\ \ddot{x} \end{Bmatrix}_{[4 \times 1]} &= \begin{bmatrix} 0 & I \\ -M_n^{-1}K_n & -M_n^{-1}C_n \end{bmatrix}_{[4 \times 4]} \begin{Bmatrix} x \\ \dot{x} \end{Bmatrix}_{[4 \times 1]} \\ &+ \begin{bmatrix} 0 & 0 & 0 \\ -\Gamma_g & -M_n^{-1}\Gamma_{\text{feed}} & -M_n^{-1}\Gamma_{\text{feed}} \end{bmatrix}_{[4 \times 3]} \begin{Bmatrix} \ddot{a}_g \\ F_{\text{feed}_{\text{UFREI}}} \\ F_{\text{feed}_{\text{VOD}}} \end{Bmatrix}_{[3 \times 1]}, \end{aligned} \quad (3)$$

$$\{U\}_{[2 \times 1]} = [I \ 0]_{[2 \times 4]} \cdot \begin{Bmatrix} x \\ \dot{x} \end{Bmatrix}_{[4 \times 1]} + [0 \ 0 \ 0]_{[2 \times 3]} \cdot \begin{Bmatrix} \ddot{a}_g \\ F_{\text{feed}_{\text{UFREI}}} \\ F_{\text{feed}_{\text{VOD}}} \end{Bmatrix}_{[3 \times 1]}. \quad (4)$$

The extended Bouc–Wen model, as reported by Manzoori et al. in [73], was used to numerically model the dynamic behavior of the U-FREI devices. This model is given by the following equation:

$$F_{\text{la}}(t) = f_k(t) + f_d(t), \quad (5)$$

where  $F_{\text{la}}(t)$  denotes the total shear force generated by the deformation of a single isolator,  $f_k(t)$  denotes the hysteretic spring force, and  $f_d(t)$  denotes the damping force. These forces were calculated using equations (6) and (7), respectively. Where the pseudo-displacement  $z$  is calculated using (8). The procedure for calculating the constants shown in Table 1, which are required for this model, and their specific explain of these parameters are described in [45].

$$f_k(t) = a_1 \cdot u_1 + a_3 \cdot u_1^3 + a_5 \cdot u_1^5, \quad (6)$$

$$f_d(t) = c \cdot \dot{u}_1 + b \left( 1 - \frac{\beta}{A} |z|^n \right) z, \quad (7)$$

$$Y\dot{z}(t) = A\dot{u}_1 - \beta\dot{u}_1|z|^n - \gamma|\dot{u}_1|z|z|^{n-1}. \quad (8)$$

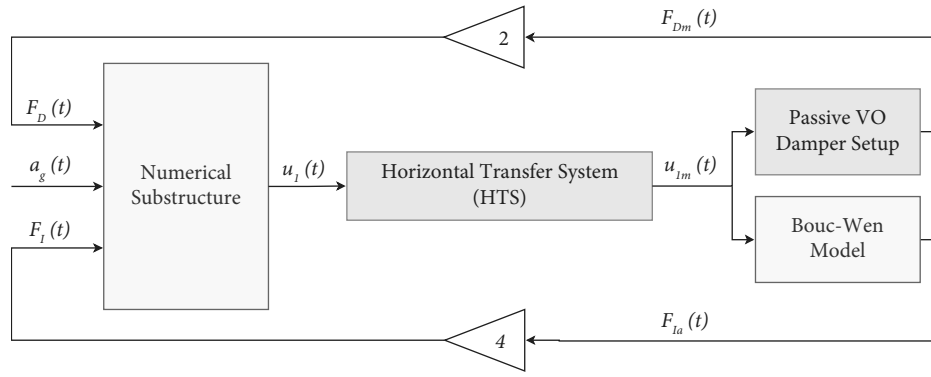
**3.2. Experimental Substructure (ES).** The experimental substructure (ES) was composed of the hybrid control system presented in Section 2.2. In addition, a framework from the LINSE Laboratory at the Universidad del Valle, Cali-Colombia, which was previously used by Riascos et al. [45], was used to test the U-FREI devices and the VOD devices. This framework was designed to simultaneously induce the boundary conditions of the experimental devices through horizontal and vertical actuator systems. Four cylindrical axes of 25.0 mm of diameter and a linear bearing allowed the vertical deformation of two U-FREI devices while they were compressed with a load constant of 19.0 kN, representing an average working gravity load of the reference structure for one of the experimental prototypes, as shown in Figure 3(a). The lateral deformation of the VOD and U-FREI devices during one of the tests was induced

relative to each prototype. The operation of the framework is schematically shown in Figure 3(b).

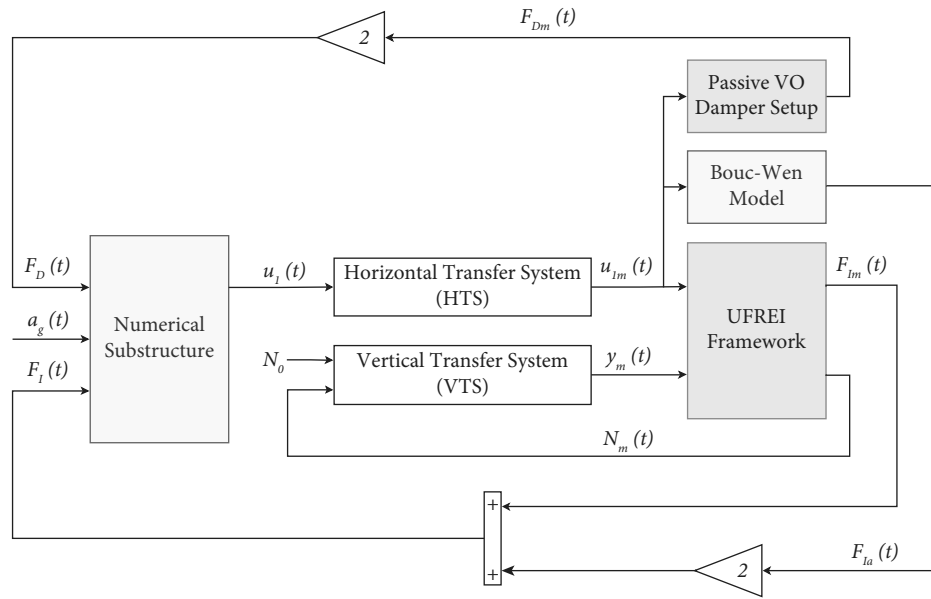
**3.3. Transfer System.** The constant interaction between the numerical structure (NS) and experimental prototypes (ES) was realized using a horizontal transfer system (HTS), which replicated the reference signal originated from the dynamic response of the DOF of interest of the reference structure with a hybrid control system. The HTS was composed of a 45.0 kN hydraulic actuator guided by two 15.0 gpm servo valves and an internal linear variable differential transformer (LVDT). These components allowed the displacement of the piston to be measured with high accuracy, which was driven by a hydraulic pump with a maximum flow rate of 6.0 gpm. The transfer system inherently exhibits delay due to electronic latency and dynamic nonlinearities arising from phenomena such as friction in the actuator. To address this, a robust  $H_\infty$  controller was implemented to generate an optimal command signal to perform accurate HTS dynamic. This controller possesses the most optimal delay compensation achievable, inherent within its own properties. In addition, a Kalman filter was used to filter the noise in the signal measured by the LVDT, thereby increasing its accuracy in an estimated manner. More details of the controller and the Kalman filter used can be found in [45, 74–76].

The load acting on the U-FREI devices was induced by a vertical transfer system (VTS) consisting of a hydraulic power unit (HPU) and a vertical actuator connected to a load cell that measured the compression force. Due to the roll-out effects exhibited by U-FREI devices [70], a control system for the vertical actuator was implemented to maintain a constant load despite the variable deformation of the specimen during different deformation states in the simulation, as described in [45]. In this study, a servo valve, EATON KBS-10, was used to control an actuator with asymmetric behavior by implementing a proportional integral (PI) controller with constants  $k_p$  and  $k_i$ , which provides high accuracy in the behavior of the VTS. The controller loop

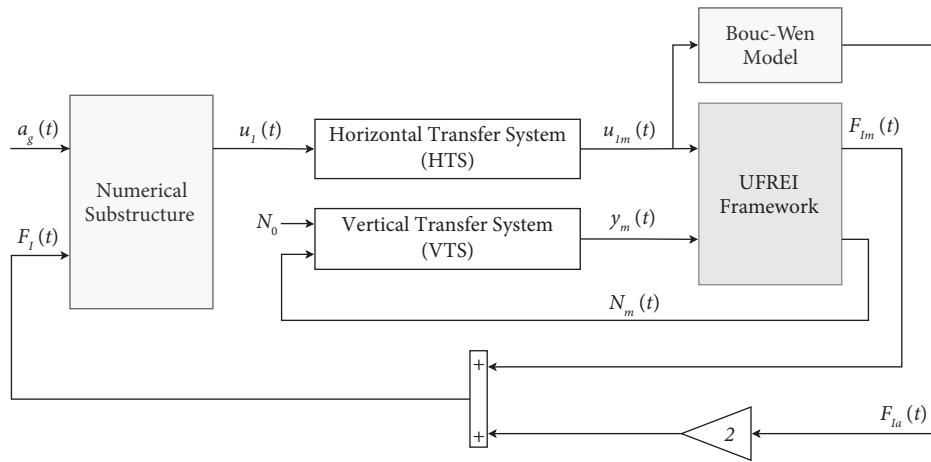




(a)



(b)



(c)

FIGURE 4: Continued.

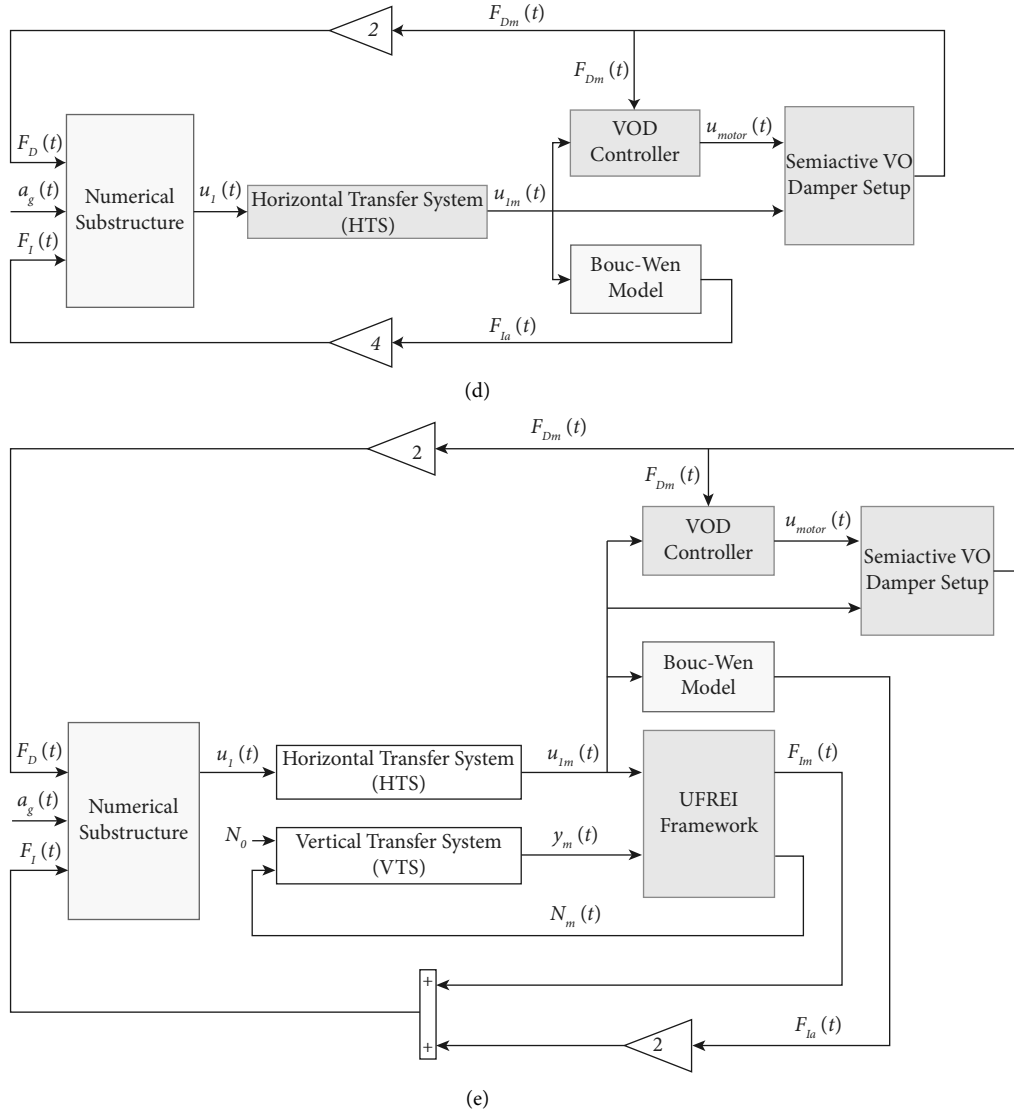


FIGURE 4: Configuration schemes. (a) Architecture no. 1, (b) architecture no. 2, (c) architecture no. 3, (d) architecture no. 4, and (e) architecture no. 5.

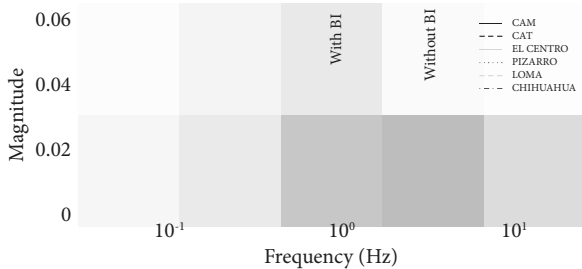


FIGURE 5: Frequency response of the assessed seismic events.

content of the evaluated earthquakes is shown in Figure 5. In addition, the graphical representation in the same figure demonstrates a reduction in the natural frequency of the entire structural system due to the influence of U-FREI devices.

## 4. Results and Discussion

**4.1. Hybrid Control System Seismic Performance.** The dynamic responses of the reference structure with the hybrid control system obtained for the five architecture setup described in Section 3.4, were compared with each other. This dynamic behavior, at level  $U_1$ , showed significant differences when architectures were used with experimental devices in comparison with numerical models for the seismic events assessed (Section 3.5), as shown in Figure 6. These discrepancies were due to the dynamics of the numerical model of the U-FREI devices (load  $F_I$ ) and changes from the passive (closing 90%) to the semiactive state of the VOD devices. The numerical model achieved an effective damping coefficient (EDC) of approximately 4.0%, whereas the experimental specimens were close to 10.0% at low strain levels [77]. For this reason, architectures nos. 2 and 5 showed

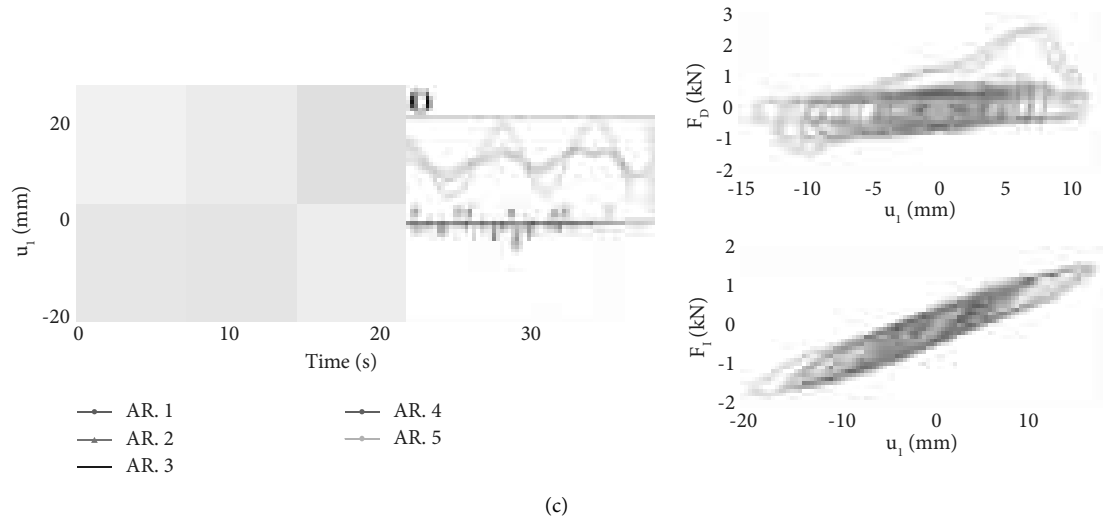
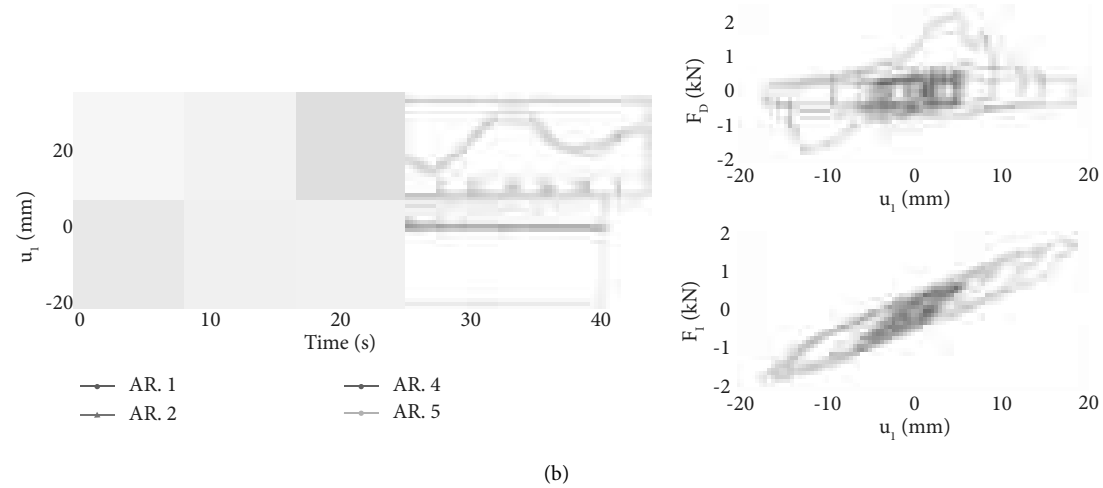
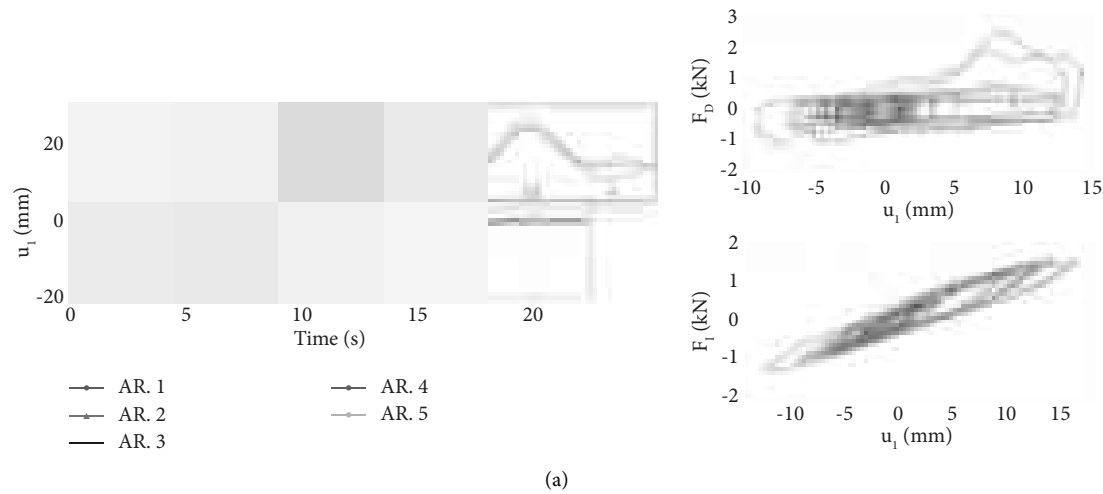


FIGURE 6: Continued.

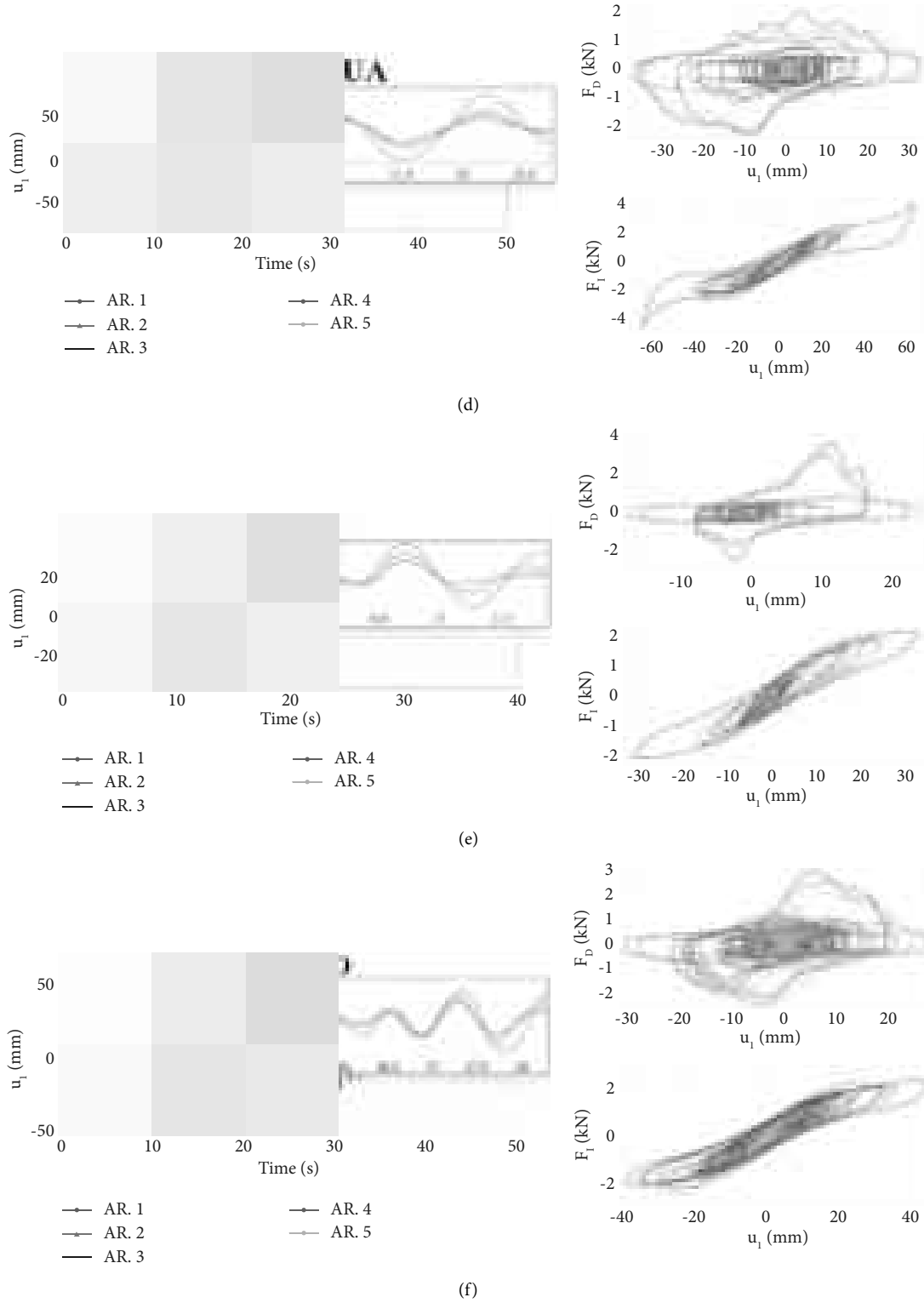


FIGURE 6: Base displacement ( $U_1$ ) and hysteresis of the hybrid control system for all seismic event assessed in each architecture used.

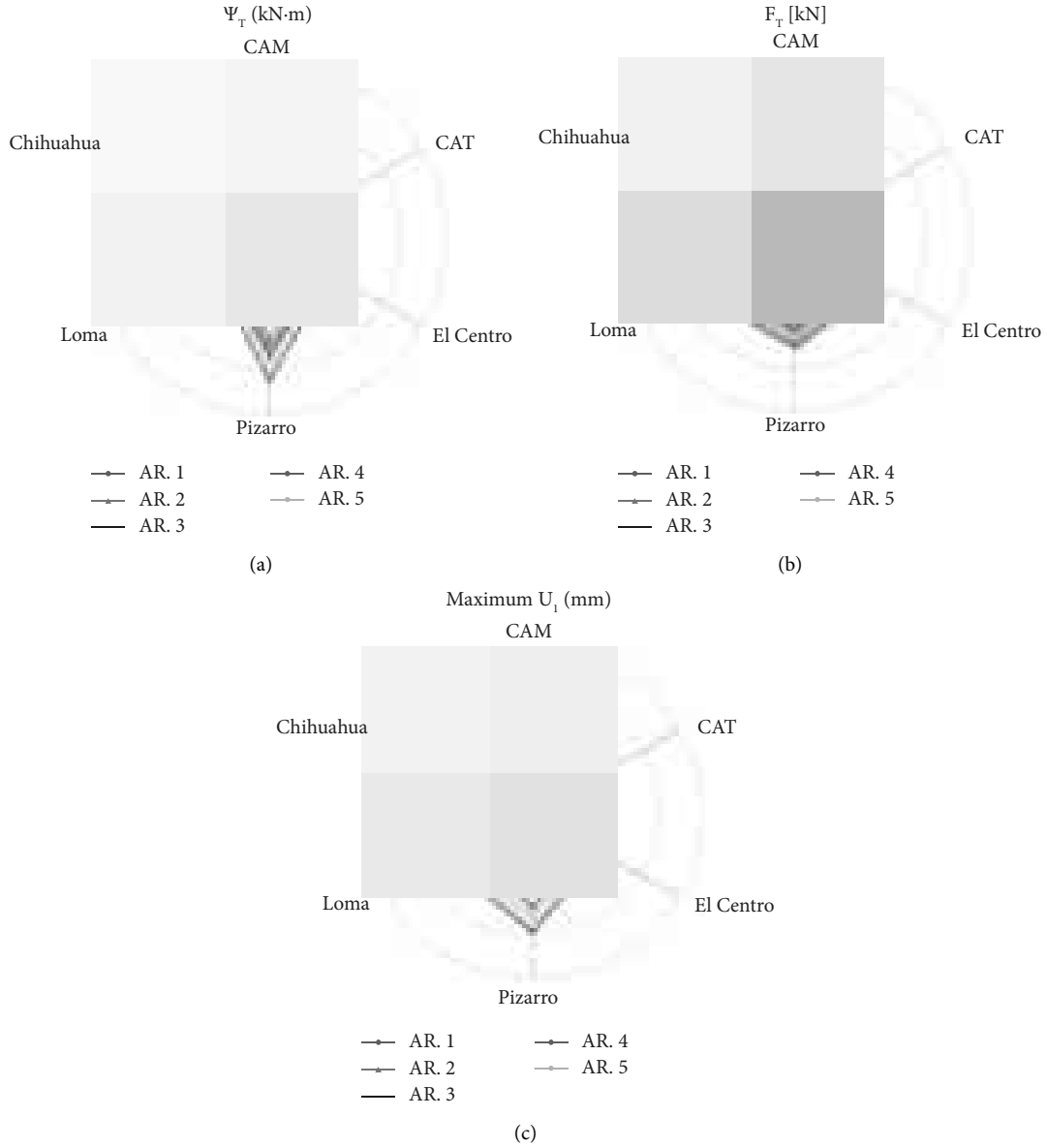


FIGURE 7: Energy dissipation capacity (a), total load of the hybrid control system (b), and maximum dynamic response  $U_1$  (c).

a reduction in the displacement of level  $U_1$  in comparison with architectures nos. 1 and 4, respectively, because experimental isolators provided more dissipation capacity to the system. Architecture no. 3 presents larger displacements and wider hysteresis curves for the U-FREI devices in comparison with the rest of architectures. In addition, an interaction was identified between the behavior of the U-FREI devices as a base isolator system and the dynamic performance of the VOD devices, which comprised the hybrid control system. The hysteresis curves of the U-FREI devices indicate consistent and reliable behavior during all tests by not reaching failure. The hysteresis curves of the VOD devices reveal that the force  $F_{D,m}$  remains constant in architectures nos. 1 and 2, while exhibiting nonlinearity in architectures nos. 4 and 5. This behavior aligns with the passive and semiactive states of the VOD.

Dissipated energy ( $\Psi$ ) is defined as the area under the hysteresis curve of a structural element, which is a crucial measure representing the ability of the system to absorb and dissipate energy during cyclic loads [78]. In this case, the total dissipated energy ( $\Psi_T$ ) of the hybrid control system represents the sum of the energy dissipated by the U-FREI devices ( $\Psi_{UFREI}$ ) combined with the energy dissipated by the VOD devices ( $\Psi_{VOD}$ ).  $\Psi_T$  exhibited similar behavior in all states of the evaluated architectures. However, Architecture No. 3 (without VOD devices) presented an average increase in  $\Psi_T$  of 43.78%. Nevertheless, the dynamic performance of the hybrid control system (load  $F_T$ ) generally induces a higher control force on the reference structure (up to 22.33% compared with Architecture No. 3), demonstrating a greater capacity for structural control, as shown in Figure 7(a). This control force allows a decrease in the dynamic

TABLE 2: Energy dissipation, total load of the hybrid control system, and maximum dynamic response  $U_1$  for each architecture evaluated and for all seismic events used.

|       | Earthquake         | CAM    | CAT    | El Centro | Pizarro | Loma   | Chihuahua |
|-------|--------------------|--------|--------|-----------|---------|--------|-----------|
| AR. 1 | $\Psi_T$ (kN·m)    | 0.326  | 0.314  | 0.532     | 1.333   | 0.273  | 0.842     |
|       | $F_T$ (kN)         | 11.464 | 8.309  | 10.166    | 12.514  | 12.728 | 13.544    |
|       | Maximum $U_1$ (mm) | 14.167 | 15.957 | 11.981    | 19.469  | 16.204 | 26.389    |
| AR. 2 | $\Psi_T$ (kN·m)    | 0.142  | 0.284  | 0.561     | 1.536   | 0.291  | 1.177     |
|       | $F_T$ (kN)         | 7.240  | 10.165 | 10.391    | 12.888  | 13.387 | 12.737    |
|       | Maximum $U_1$ (mm) | 12.509 | 14.422 | 12.495    | 21.341  | 16.618 | 35.118    |
| AR. 3 | $\Psi_T$ (kN·m)    | 0.133  | —      | 0.599     | 2.270   | 0.423  | 1.869     |
|       | $F_T$ (kN)         | 6.305  | —      | 6.856     | 9.674   | 8.419  | 18.133    |
|       | Maximum $U_1$ (mm) | 16.462 | —      | 20.220    | 44.570  | 32.248 | 65.033    |
| AR. 4 | $\Psi_T$ (kN·m)    | 0.142  | 0.321  | 0.563     | 1.706   | 0.346  | 1.078     |
|       | $F_T$ (kN)         | 6.310  | 8.195  | 7.382     | 11.588  | 9.470  | 11.856    |
|       | Maximum $U_1$ (mm) | 12.206 | 18.485 | 13.714    | 30.349  | 22.090 | 36.814    |
| AR. 5 | $\Psi_T$ (kN·m)    | 0.153  | 0.310  | 0.577     | 1.611   | 0.368  | 1.240     |
|       | $F_T$ (kN)         | 6.143  | 9.885  | 7.214     | 11.717  | 9.344  | 13.093    |
|       | Maximum $U_1$ (mm) | 12.929 | 16.773 | 14.399    | 24.728  | 24.281 | 36.256    |

response of DOF  $U_1$  of more than 35.49% compared with the structure with only U-FREI devices as base isolator system (Figure 7(b)). All these values were arranged in Table 2.

**4.2. Reference Structure Seismic Performance with Hybrid Control System.** The reference structure dynamic behavior was assessed in five architectures (Section 3.4) using VOD and U-FREI experimental devices as the hybrid control system and with the base fixed (through a numerical model). Each case was subjected to six seismic events described in Section 3.5. In the reference structure, the hybrid control systems generated a minimal reduction in the spectral response amplification factors of 78.57%, 85.33%, 40.82%, 85.12%, 63.89%, and 47.34% for the CAM, CAT, Chihuahua, El Centro, Loma Prieta, and Pizarro earthquakes, respectively, in comparison with the fixed reference structure, as shown in Figure 8. In addition, this figure also shows that the time-history response of DOF  $U_2$  with the hybrid control system decreased with respect to the structure fixed for all seismic events evaluated.

U-FREI devices, as base isolator system, allow the reference structure to reduce its maximum drift by 60.53%, -1.96%, 75.09%, 32.08%, and 56.01%, respectively, for the CAM, Chihuahua, El Centro, Loma Prieta, and Pizarro seismic events compared to the fixed structure, as shown in Figure 9(a). Similarly, the hybrid control system in the passive state (90.0% closure) allowed the reference structure to reduce its maximum drift by 14.05%, -47.21%, 39.06%, 40.35%, and 23.35%, respectively, for the earthquakes compared with the response of the reference structure with only U-FREI devices. Furthermore, the hybrid control system in the semiactive state allowed the reference structure to reduce its maximum drift by 12.43%, 12.55%, 8.07%, 39.09%, 29.90%, and 6.16%, respectively, for the CAM, CAT, Chihuahua, El Centro, Loma Prieta, and Pizarro earthquakes compared with the response of the reference structure with the hybrid control system in the passive state (90.0% closure).

In brief, the maximum drift obtained for all evaluated architectures, including the 10.0%, 50.0%, and 90.0% valve-opening conditions for architectures 1 and 2, are shown in Figure 9(b). This figure quantifies the tendency of the hybrid control system in the semiactive state (architectures 4 and 5) to equal or improve up to 36.80% of the dynamic response of the structure when using only U-FREI devices as the base isolation system, compared to the response of the fixed structure. In addition, an increase in the maximum drift was observed when the passive-state hybrid control system with 90.0% opening was implemented in architectures 1 and 2 during seismic events compared to the fixed structure and a performance improvement of up to 15.50% was observed for this condition when the passive-state hybrid control system had a 10.0% opening. This observation shows that a passive control system does not guarantee a reduction in the drift for a seismic event with a wide frequency content; even, in some cases, it may cause an increase of the drift, with potential significant damage.

**4.3. RTHS Performance.** RTHS have errors owing to factors inherent to the evaluation methodology, such as DOF condensation in the numerical substructure, intrinsic noise in electronic signals of the ES and dynamics of the transfer systems, among others [79, 80]. To evaluate the accuracy of the RTHS, some indices have been proposed that consider the differences between the signals of references and measured at the interface of the substructure. The assessment of local performance relied on achieving synchronization in boundary conditions between the numerical response ( $u_{i,d}^*$ ) transmitted to the actuator and the displacement attained in the physical component ( $u_{i,m}^*$ ). This involves deriving a singular metric that signifies the disparity between actuator signals in the time domain. To facilitate comparison in the frequency domain, equations (9)–(13) were employed, corresponding to the frequency evaluation index (FEI). FEI compares the fast Fourier transform (FFT) of the desired signal  $u_{i,d}^*$  with that of the measured signal  $u_{i,m}^*$  at the

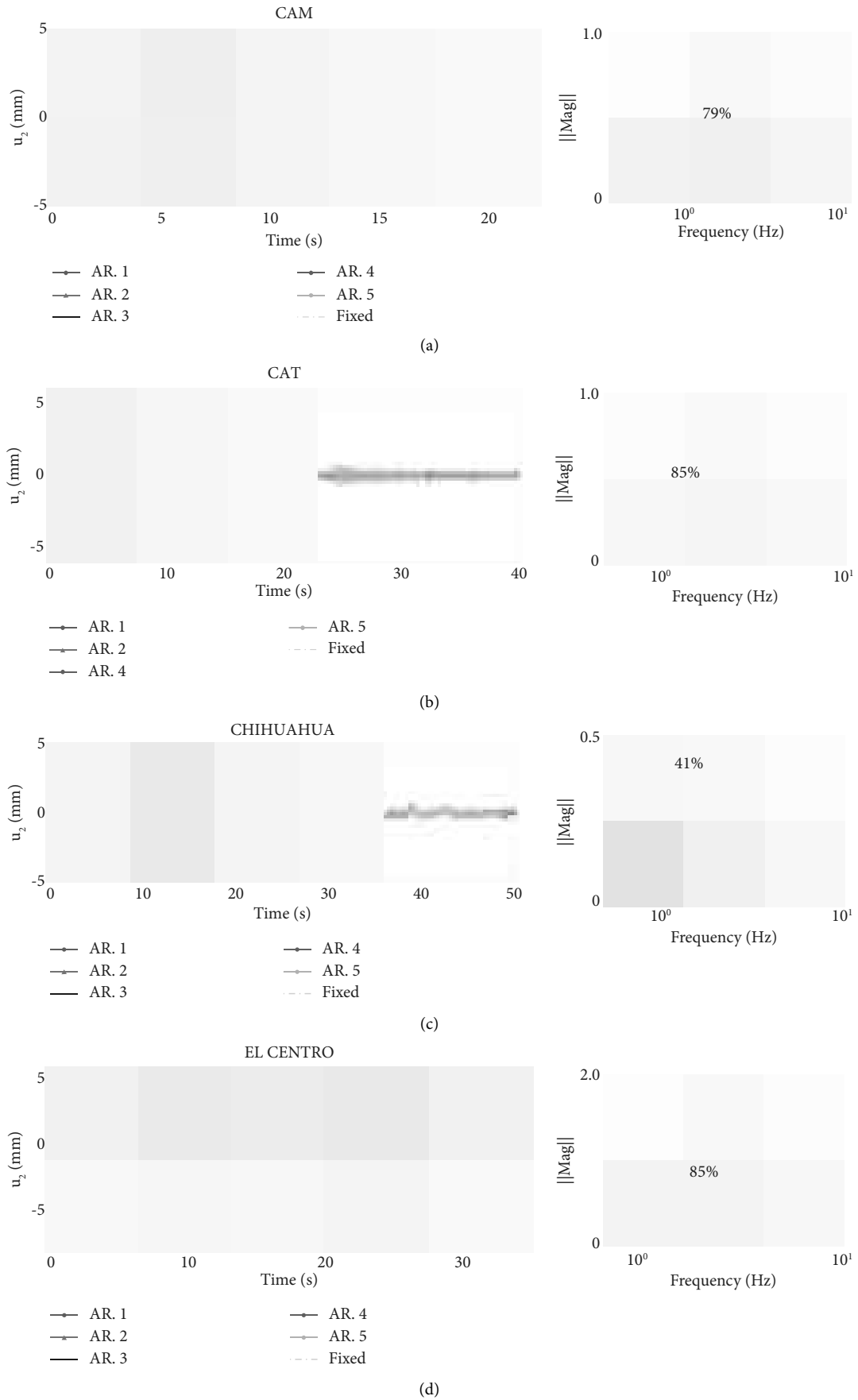
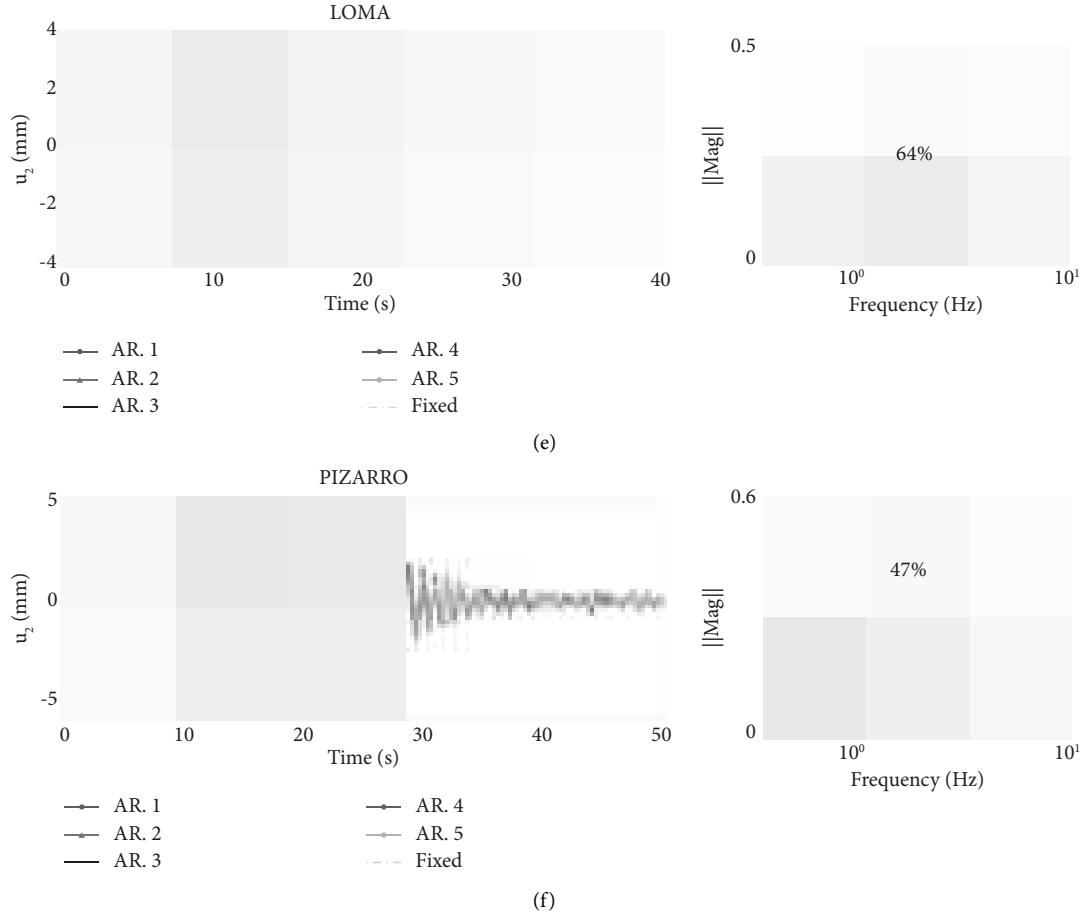


FIGURE 8: Continued.

FIGURE 8: Dynamic response of the DOF  $U_2$  for all test architectures.

actuator, considering parameters such as the dominant frequency during the RTHS denoted as  $f^{eq}$ , the generalized amplitude  $A_0$ , and the time delay  $\delta$ , as described in [81].

$$FEI = \sum_{j=1}^N \left( \frac{u_{i,m}^*(j)}{u_{i,d}^*(j)} \cdot \frac{\|u_{i,d}^*(j)\|^l}{\sum_{j=1}^P \|u_{i,d}^*(j)\|^l} \right), \quad (9)$$

$$f^{eq} = \frac{\sum_{j=1}^N \left( \|u_{i,d}^*(j)\|^l \cdot f_i \right)}{\sum_{j=1}^N \|u_{i,d}^*(j)\|^l}, \quad (10)$$

$$A_0 = \|FEI\|, \quad (11)$$

$$\emptyset = \arctan \left[ \frac{Im(FEI)}{Re(FEI)} \right], \quad (12)$$

$$\delta = -\frac{\emptyset}{2 \cdot \pi \cdot f^{eq}}. \quad (13)$$

In addition, global indices account for the interaction between the numerical and experimental substructures. These were assessed within the time domain by quantifying the most substantial difference between the computed and observed displacements using  $e_{DM}$  and  $e_{MD}$ , as delineated in (14) and (15) [59, 82].

$$e_{DM} = \frac{||u_{i,d}^*|_{\max} - |u_{i,m}^*|_{\max}|}{|u_{i,m}^*|_{\max}}, \quad (14)$$

$$e_{MD} = \frac{|u_{i,d}^* - u_{i,m}^*|_{\max}}{|u_{i,m}^*|_{\max}}. \quad (15)$$

All the indices were obtained for the five architectures used and are presented in Table 3. All cases presented a high level of horizontal tracking, with a generalized amplitude  $A_0$  close to 1.0, and global errors lower than 6.80%, which allowed us to conclude that HTS has high performance and allows performing RTHS with a high level of accuracy.

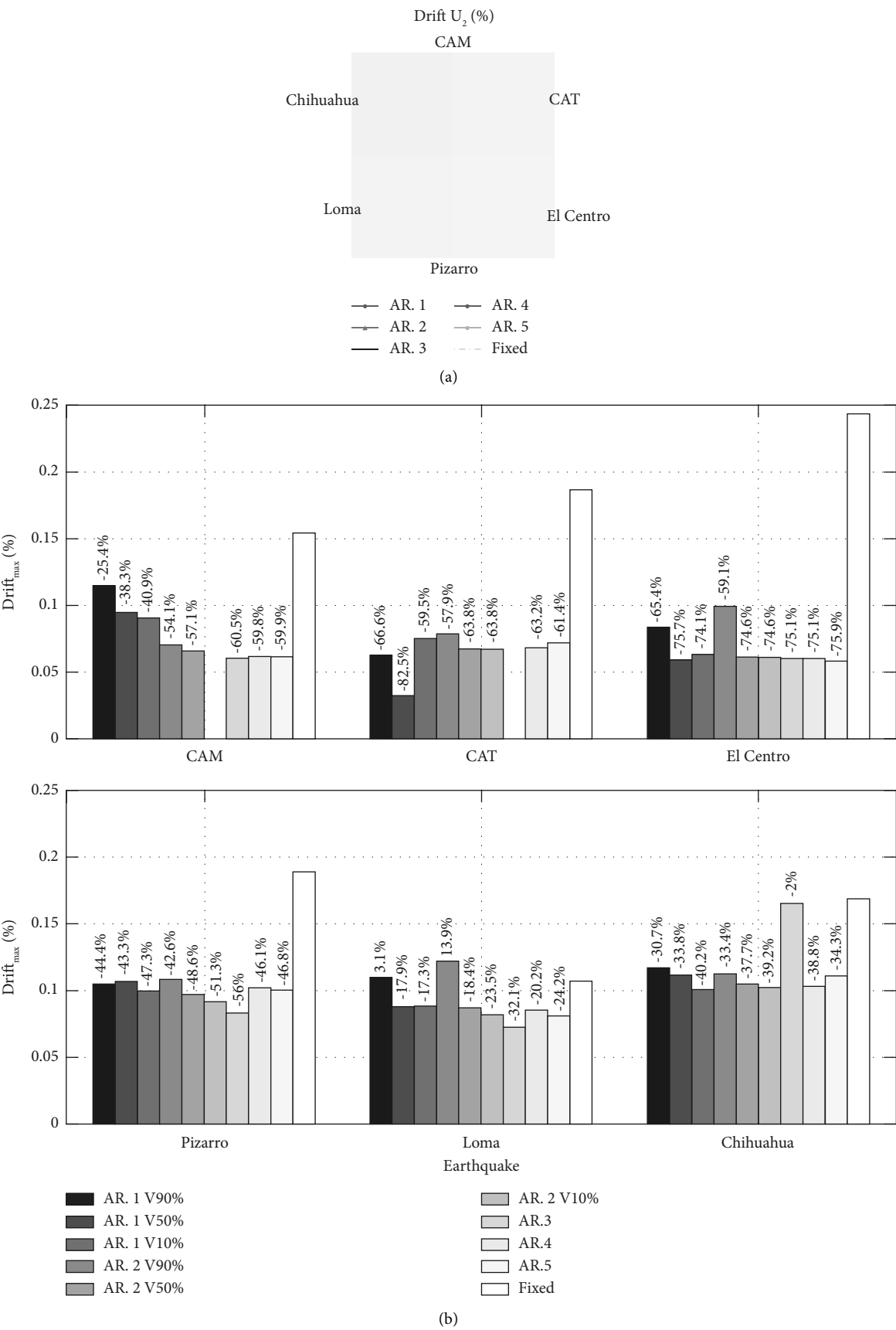


FIGURE 9: Structure drift performance for all test architectures.

TABLE 3: RTHS assessment indexes.

| Transfer system            | Earthquake | FEI         | $f_{eq}$ (Hz) | $A_0$ | $e_{DM}$ (%) | $e_{MD}$ (%) | $\delta$ (ms) |
|----------------------------|------------|-------------|---------------|-------|--------------|--------------|---------------|
| <i>Architecture no. 1*</i> |            |             |               |       |              |              |               |
| HTS                        | El Centro  | 0.988–0.03i | 1.358         | 0.988 | 5.120        | 6.791        | 3.146         |
|                            | Loma P.    | 1.018–0.02i | 1.079         | 1.018 | 1.859        | 3.182        | 2.194         |
|                            | Pizarro    | 0.980–0.02i | 1.038         | 0.980 | 1.711        | 2.903        | 2.465         |
|                            | Chihuahua  | 0.996–0.01i | 0.754         | 0.996 | 0.977        | 1.880        | 2.135         |
|                            | CAM        | 1.065–0.02i | 0.776         | 1.065 | 3.525        | 5.515        | 4.173         |
|                            | CAT        | 0.937–0.02i | 1.118         | 0.938 | 2.649        | 6.183        | 2.952         |
| <i>Architecture no. 2*</i> |            |             |               |       |              |              |               |
| HTS                        | El Centro  | 0.982–0.02i | 1.308         | 0.982 | 3.597        | 6.048        | 2.813         |
|                            | Loma P.    | 1.027–0.01i | 0.736         | 1.027 | 1.968        | 2.808        | 2.011         |
|                            | Pizarro    | 1.012–0.01i | 0.908         | 1.012 | 0.976        | 3.265        | 2.488         |
|                            | Chihuahua  | 1.009–0.01i | 0.663         | 1.009 | 0.535        | 1.652        | 1.954         |
|                            | CAM        | 0.931–0.01i | 0.633         | 0.931 | 1.954        | 3.752        | 3.889         |
|                            | CAT        | 1.062–0.01i | 0.699         | 1.062 | 1.909        | 3.782        | 2.344         |
| <i>Architecture no. 3</i>  |            |             |               |       |              |              |               |
| HTS                        | El Centro  | 0.981–0.02i | 1.118         | 0.982 | 2.549        | 5.044        | 2.959         |
|                            | Loma P.    | 0.991–0.02i | 0.934         | 0.992 | 1.381        | 3.483        | 3.487         |
|                            | Pizarro    | 0.993–0.01i | 0.934         | 0.993 | 0.849        | 2.156        | 1.628         |
|                            | Chihuahua  | 0.997–0.00i | 0.722         | 0.997 | 0.639        | 1.728        | 0.935         |
|                            | CAM        | 0.952–0.02i | 0.832         | 0.953 | 3.315        | 5.237        | 4.015         |
|                            | CAT        | —           | —             | —     | —            | —            | —             |
| <i>Architecture no. 4</i>  |            |             |               |       |              |              |               |
| HTS                        | El Centro  | 0.953–0.03i | 1.145         | 0.953 | 2.880        | 5.990        | 3.681         |
|                            | Loma P.    | 0.986–0.01i | 0.941         | 0.986 | 0.606        | 2.471        | 1.556         |
|                            | Pizarro    | 0.999–0.01i | 1.025         | 0.999 | 1.022        | 3.285        | 2.084         |
|                            | Chihuahua  | 0.998–0.01i | 0.780         | 0.998 | 0.535        | 1.759        | 1.785         |
|                            | CAM        | 1.006–0.01i | 0.854         | 1.006 | 1.340        | 2.771        | 2.743         |
|                            | CAT        | 0.982–0.01i | 1.082         | 0.982 | 1.496        | 2.857        | 1.581         |
| <i>Architecture no. 5</i>  |            |             |               |       |              |              |               |
| HTS                        | El Centro  | 0.982–0.02i | 1.177         | 0.982 | 1.402        | 5.470        | 2.613         |
|                            | Loma P.    | 0.998–0.01i | 0.995         | 0.998 | 0.793        | 2.229        | 1.784         |
|                            | Pizarro    | 0.984–0.01i | 0.859         | 0.984 | 1.133        | 2.661        | 2.223         |
|                            | Chihuahua  | 0.994–0.01i | 0.690         | 0.994 | 0.713        | 2.046        | 2.083         |
|                            | CAM        | 0.886–0.01i | 0.624         | 0.886 | 3.773        | 5.306        | 2.930         |
|                            | CAT        | 0.900–0.01i | 0.668         | 0.900 | 3.615        | 5.776        | 1.592         |

\* Valve with 90.0% opening.

## 5. Conclusions

In this study, the assessment of the seismic behavior of a low-degree-of-freedom reference structure with a hybrid control system composed of unconnected fiber-reinforced elastomeric isolator (U-FREI) and variable orifice dampers (VOD) devices was performed using the RTHS methodology for six seismic events: CAT, CAM, El Centro, Loma Prieta, Chihuahua, and Pizarro, scaled to an acceleration of 0.10 g ( $0.981 \text{ m}\cdot\text{s}^{-2}$ ). In addition, five architectures were evaluated for these seismic events, including a numerical model (Bouc–Wen model) of the U-FREI devices for two test architectures, which had an effective damping coefficient in the experimental base isolation system ( $\text{EDC} \approx 10.0\%$ ) higher than that used in the numerical model ( $\text{EDC} \approx 4.0\%$ ), presenting differences between architectures 1-2 and 4-5.

The architecture no. 3 without VOD devices presented an increase in the energy dissipation capacity ( $\Psi_T$ ) of the hybrid control system of 43.78%. In addition, the induced

control load ( $F_T$ ) was higher in the reference structure by up to 22.33% compared with architecture no. 3. Furthermore, this control load reduces the response of the DOF  $U_1$  by more than 35.49% compared to the structure with only U-FREI devices.

The hybrid control system for all architectures generated, in the reference structure, a minimal reduction in the spectral response amplification factor of 78.57%, 85.33%, 40.82%, 85.12%, 63.89%, and 47.34% for CAM, CAT, Chihuahua, El Centro, Loma Prieta, and Pizarro earthquakes, respectively, in comparison with the reference structure fixed. Similarly, U-FREI devices as base isolator system allow the reference structure to reduce its maximum drift by 60.53%,  $-1.96\%$ , 75.09%, 32.08%, and 56.01%, respectively, for the evaluated earthquakes (without CAT) compared with the fixed structure. Likewise, the hybrid control system in the passive state (90.0% closure) allowed the reference structure to reduce its maximum drift by 14.05%,  $-47.21\%$ , 39.06%, 40.35%, and 23.35%, respectively,

for the evaluated earthquakes (without CAT) compared with the response of the reference structure with only U-FREI devices. Furthermore, the hybrid control system in the semiactive state allows the reference structure to reduce its maximum drift by 12.43%, 12.55%, 8.07%, 39.09%, 29.90%, and 6.16%, respectively, for the evaluated earthquakes compared to the response of the reference structure with the hybrid control system in the passive state (90.0% closure). This demonstrates the dynamic robustness of the hybrid control system in the semiactive condition to decrease base displacements and maintain maximum floor drifts under minimum conditions. The results position this study as one of the first tests of the technical potential of a hybrid control system composed of smart base isolation with U-FREI and VOD devices to reduce the seismic vulnerability of low-degree-of-freedom reference structures.

The experimental assessment was performed using the RTHS methodology, with a simplified 2-DOF model of the reference structure as a numerical substructure and U-FREI and VOD devices as a hybrid control system as an experimental substructure. For the tracking levels of the horizontal transfer systems, all cases presented high-level tracking, with a generalized amplitude  $A_0$  close to 1.0 and global errors lower than 6.79%. Furthermore, HTS had delays of less than 4.17 ms, employing a robust  $H_\infty$  controller in all cases. The above results indicate that the RTHS exhibits high fidelity and accuracy.

## Data Availability

The data that support the findings of this study are available from the corresponding author upon reasonable request.

## Disclosure

This work is part of the research project: Real-time hybrid simulations: a reliable, fast, and economical alternative for the evaluation of resilient structures, under the program: Emerging technologies for the mitigation of seismic risk in civil infrastructure, CT 463-2020 – program code 110685270483.

## Conflicts of Interest

The authors declare that there are no conflicts of interest.

## Authors' Contributions

All authors contributed to this research and discussed the results and reviews during all phases of the research paper. All authors read and accepted the published version of the research paper.

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#### ***4.2 Seismic Performance Evaluation of Low-Rise Thin Reinforced Concrete Wall Buildings with Unconnected Fiber-Reinforced Elastomeric Isolators Using Real-Time Hybrid Simulation.***

The construction industry is increasingly adopting innovative methods to address the growing demand for housing, particularly in seismic-prone regions. Among these innovations, thin reinforced concrete walls (TRCW) have gained prominence due to their efficiency in the Industrialized Housing Construction (IHC) system, which allows for rapid and cost-effective construction. However, TRCW structures present significant challenges in seismic performance, primarily due to their limited ductility and poor energy dissipation capacity, making them vulnerable to damage during earthquakes, particularly from flexural compression and out-of-plane buckling.

The first article, titled "*Multi-experimental Seismic Analysis of Low-Rise Thin Reinforced Concrete Wall Building with Unconnected Elastomeric Isolators Using Real-Time Hybrid Simulations*", contributes to this study by introducing a novel approach using Multi-Axial Real-Time Hybrid Simulation (M-RTHS). This method combines numerical models and experimental testing to evaluate the seismic response of low-rise TRCW buildings equipped with U-FREI devices as base isolation systems. The article highlights the significant reduction in upper structure floor drifts, by more than 57.0%, and the improved energy dissipation performance provided by the isolators under various seismic excitations.

The second article, titled "*Seismic Performance Assessment of a Low-Rise Thin Reinforced Concrete Wall Building with Unconnected Fiber Reinforced Elastomeric Isolators Using Real-Time Hybrid Simulation*", focuses on the detailed seismic performance of TRCW buildings using U-FREI devices under different seismic events, such as El Centro, Loma Prieta, and Kobe. The study demonstrated that U-FREIs effectively reduced maximum structural drifts by up to 83.0%, significantly lowering shear forces and mitigating critical failure modes like flexural compression and out-of-plane buckling. Additionally, the energy dissipation capabilities of U-FREIs were evaluated, revealing their viscoelastic behavior that contributed to minimizing the dynamic response of the building during seismic events.

Both articles collectively provide a comprehensive evaluation of the seismic performance of low-rise TRCW buildings. Through RTHS, the studies demonstrated that U-FREI base isolation systems can enhance the structural resilience of TRCW buildings by effectively reducing seismic drifts, shear forces, and overall vulnerability. These findings highlight the potential of U-FREI systems as a cost-effective and efficient solution for improving seismic safety in regions at high risk of earthquakes. The application of RTHS offers a reliable method for accurately assessing the dynamic behavior of hybrid structural systems, paving the way for safer building designs.