



Structural response of half-scale pumice concrete masonry building: shake table/ambient vibration tests and FE analysis

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Received: 15 December 2023 / Accepted: 1 July 2024 / Published online: 8 July 2024
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Abstract Seismic performance evaluation of masonry structures is of paramount importance for ensuring the safety and resilience of buildings in earthquake-prone regions. There are limited number of studies on pumice elements in the literature. In addition, there are almost no studies investigating the earthquake behavior of pumice masonry building as a whole structure. In this context, a comprehensive understanding of their seismic response and dynamic characteristics has been lacking. To address this knowledge gap, a shake-table experimental campaign was undertaken, wherein half-scale pumice masonry building was exposed to simulated seismic forces. To enhance the experimental findings, numerical simulations were performed to confirm and expand our comprehension of how the pumice masonry structure responds to dynamic forces. Integrating both

experimental and numerical outcomes provides a holistic understanding of how pumice masonry buildings behave during seismic events. At the end of the experimental study, the frequency values of the pumice model were observed to decrease up to 23.5% in the modes compared to the undamaged state. In the numerical model, this value decreases up to 19.85%. For the undamaged and damaged model, the first three experimental mode shapes were similar to the numerical mode shapes. Both experimental and numerical results show that the expected damages occur in the same regions. These results show that nonlinear FE models can be helpful in determining potential damage model locations. The findings have implications for the seismic design and retrofitting of similar traditional masonry buildings, facilitating the

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development of resilient and sustainable engineering solutions in seismic-prone regions.

Keywords Ambient vibration test · Pumice concrete · Finite element analysis · Shake table test · Dynamic characteristics · Damage

List of symbols

TBEC	The Turkish building earthquake code
E	Modulus of elasticity
a	Acceleration
v	Velocity
F	Force
σ	Stress
ε	Strain
A	Area
V	Volume
ρ	Density
m	Mass
e	Energy
t	Time
ω	Frequency
ξ	Damping
ERZ	Erzincan earthquake
PGA	Peak ground acceleration
PGD	Peak ground displacement
OMA	Operational modal analysis
URM	Unreinforced masonry
DS	Damage state
GM	Ground motion
EFDD	Enhanced frequency domain decomposition
DI	Damage index
IDR	Inter-story drift ratio
CDP	Concrete damage plasticity

1 Introduction

Masonry structures have been an integral part of the building stock for centuries, providing enduring architectural masterpieces and functional buildings around the world. As construction materials and methods evolve, researchers are constantly working to improve the ability of masonry structures to withstand seismic events, especially in earthquake-prone areas. Pumice concrete, known for its favorable strength-to-weight ratio, low bulk density, and exceptional thermal insulation qualities, is one of the materials receiving growing interest. Research

in earthquake engineering is particularly focused on understanding the seismic performance of masonry structures, including those built with pumice concrete. The tensile strengths of masonry units, including materials like brick, pumice concrete, and gas concrete, are observed to be relatively low despite their high compressive strengths [1]. This characteristic presents a significant challenge in the design and structural integrity of masonry constructions, particularly in resisting lateral forces such as those induced by seismic activity. While these materials excel in withstanding compressive loads, their limited ability to resist tension can lead to vulnerabilities, especially during dynamic events like earthquakes. Therefore, understanding and mitigating the effects of this inherent weakness is essential in ensuring the overall stability and safety of masonry structures. Various techniques, including repair/strengthening methods and seismic retrofitting, are employed to enhance the tensile capacity of masonry elements and improve their performance under seismic loading conditions [2–8].

Pumice concrete is a lightweight concrete variant made by incorporating pumice aggregate, a type of volcanic rock characterized by its porous and lightweight nature, into the concrete mix. This material offers several advantages, including reduced weight, improved insulation properties, and enhanced workability compared to traditional concrete mixes. Pumice concrete finds applications in various construction projects where lightweight and insulating properties are desired, such as in building envelopes, insulation layers, and lightweight fill materials. The use of pumice concrete has been extensively studied and documented in research literature. For example, research by Karthika et al. [9] explores the effects of pumice aggregate on the properties of lightweight concrete, highlighting its potential for sustainable construction practices. Additionally, studies by Tran and Ghosh [10] investigate the mechanical properties and performance of pumice concrete in structural applications, providing valuable insights into its structural behavior and suitability for different construction contexts. Overall, pumice concrete represents a promising material option for lightweight and sustainable construction practices, with ongoing research contributing to its further development and application in the construction industry. Several investigations, encompassing both state-of-the-art analyses and comprehensive reviews [11–14], have thoroughly examined the

incorporation of pumice in concrete, building upon earlier research and exploring its diverse applications. These studies offer valuable insights into the utilization of pumice in various concrete formulations, considering its wide-ranging benefits and potential applications. Through a meticulous examination of previous literature, researchers have elucidated the role of pumice in enhancing concrete properties and its effectiveness in fulfilling specific construction requirements.

The masonry buildings have disadvantages as well as advantages. There is a tendency for such buildings to be damaged in earthquakes. Under earthquake loads, pumice structures may face cracks and deformations due to its low strength. Especially in high intensity earthquakes, the structural integrity of pumice buildings can be severely affected and may even be at risk of collapse. The seismic performance of masonry buildings constructed with pumice can be influenced by factors such as local structural conditions, the design and construction practices employed, as well as the seismic building codes applicable in the earthquake-prone region. Therefore, appropriate engineering methods and retrofitting solutions are important to improve earthquake resilience. In particular, retrofitting of existing pumice structures to minimize earthquake damage is an important step in terms of long-term structural stability.

Shaking table tests have emerged as a powerful experimental tool for investigating the dynamic behavior and seismic behavior of masonry structures under controlled conditions. By subjecting scaled-down models to simulated earthquake excitations, shaking table tests enable researchers to observe and analyze the structural response, identify failure modes, and assess the effectiveness of seismic retrofitting and strengthening measures. In addition to experimental studies such as shaking table tests, numerical analyses are frequently used research methods on masonry structures. After the 1976 Friuli earthquake in Italy, non-linear analysis methods gained prominence for assessing the seismic resilience of masonry structures [15–19]. Dynamic analyses were subsequently introduced in the literature and quickly became the standard method for assessing existing buildings, as mandated by several Codes [20–23], now widely adopted by professionals.

Recently, non-linear dynamic analyses have emerged as an alternative tool in both research and

practical applications, especially for more intricate buildings. To assess structural resilience, this approach entails exposing the building to a sufficient number of earthquake records, whether they are natural or artificially created, with alignment to the local seismic spectrum [24]. In these analyses, Finite Element (FE) Models and Discrete Element Models (DEM) are utilized. FE models are well-suited for representing buildings with what is often referred to as “box behavior,” where the building behaves predominantly as a rigid block. On the other hand, DEM is employed to capture phenomena such as out-of-plane rocking and the collapse of masonry piers, offering a more detailed representation of structural behavior [25–27]. Post-earthquake surveys have provided insights into the principal collapse mechanisms of masonry buildings [28, 29]. Scholars have undertaken extensive research, employing a combination of numerical simulations and experimental research, to assess the seismic behavior of diverse masonry materials, including Unreinforced Masonry (URM) and pumice concrete structures [30–34].

The integration of experimental and numerical approaches has proven to be highly valuable in seismic studies of masonry buildings. Experimental data serves as a means to validate and update numerical models, while numerical simulations enable extrapolation and analysis that go beyond the constraints of physical experiments. The combination of these approaches has enabled researchers to improve design guidelines, propose innovative retrofitting techniques and increase the dynamic resistance of masonry buildings [35, 36].

Apart from shake table testing, ambient vibration experiments are carried out to analyze and understand the dynamic characteristics of the pumice concrete masonry structure. These tests are performed both before and after exposing the building to seismic loading, providing valuable insights into its undamaged and damaged states. Operational modal analysis (OMA) [37] is a valuable technique employed to ascertain fundamental characteristics of structures, including natural frequencies, mode shapes, and damping ratios. These parameters provide essential insights into a dynamic behavior of structure. In numerous experimental studies, researchers have applied the non-destructive OMA method to extract the dynamic properties of masonry buildings [38–42]. OMA

has become increasingly popular due to its non-destructive nature, allowing for the assessment of a dynamic behavior of structure without causing any harm to the building under investigation. OMA has been widely applied to various types of structures, including buildings, bridges, mosques, churches and minarets, which are often constructed using masonry materials [43–46]. In this approach, precise accelerometers are strategically positioned on the structure in question, and a data acquisition system is employed to record the vibration signals. Subsequently, specialized software is utilized to analyze the collected data and extract the dynamic characteristics of the building.

It is essential to highlight that there has been a growing trend towards validating numerical models by comparing them with dynamic characteristics obtained from experimental seismic tests. The field of computer graphics has witnessed significant advancements in the past decade, greatly contributing to the structural analysis of various structures, including masonry constructions. In the available literature, computational modeling techniques, specifically categorized into continuum and discontinuity models, have been utilized to analyze the response of masonry structures, providing separate approaches for evaluating masonry behavior [47–49]. The analysis of dynamic performance in masonry buildings commonly involves micro and macro modeling approaches. Micro modeling approaches generate large amounts of computational data, resulting in complex and resource-intensive analyses. Furthermore, macro modeling methods presume a linear elastic material behavior, which provides a more practical framework for assessing extensive and intricate masonry structures. In macro modeling, masonry components such as masonry units, mortar, and the interfaces between masonry units and mortar are treated as a homogenized continuum, combining their individual characteristics into a unified representation [50].

The principal aim of this study is to investigate the response of a pumice concrete building constructed with lightweight masonry units to seismic forces and to analyze the mechanisms of damage caused by these forces. Additionally, the study validates the precision of numerical models by comparing them with the results obtained from these experimental investigations.

2 Experimental program

Conducting tests on full-scale models to explore the seismic response of masonry buildings is not a common or feasible practice, mainly due to constraints related to laboratory resources, construction challenges, and safety precautions. Instead, experimental investigations often opt for scaled-down models or individual structural components to study how masonry buildings respond to seismic forces. The primary goal of this study is to examine the seismic behavior of a lightweight masonry pumice concrete building, focusing on its performance under seismic forces and the mechanisms of damage that may occur. The accuracy of numerical models will also be verified using the results of these experimental investigations. To achieve this, a laboratory model was built using pumice concrete, and shake table tests were conducted with various ground motions to simulate scenarios of earthquake damage. This chapter provides comprehensive details of the test model, the experimental setup, the measurement systems used, and the sequence of tests performed. The FE model verification process and the experimental procedure are outlined in Fig. 1.

In this study, the shaking table in the Earthquake and Structural Health Monitoring Laboratory of Karadeniz Technical University was utilized. This 4.00×4.00 m shaking table can achieve a maximum acceleration of ± 2.00 g. It is capable of displacements up to ± 400.00 mm and operates within a frequency range of 0.10–50.00 Hz. Its acceleration capacity of 628.00 mm/sec enables the use of various ground motions from past earthquakes in experiments. The shaking table, which moves in a single direction, has a dynamic horizontal force capacity of 500.00 kN and a vertical load capacity of 350.00 kN. Test specimens are fixed to the table using threaded holes in the table surface.

2.1 Materials and test model

The URM building model was built using filled pumice concrete, a common choice for low-rise structures. The filled pumice units, each measuring $100 \text{ mm} \times 195 \text{ mm} \times 90 \text{ mm}$, were chosen in compliance with the Turkish Building Earthquake Code (TBEC-2018) [51]. The 200 mm wide pumice units were halved to produce half-scale 100 mm wide

Fig. 1 Flowchart of the study

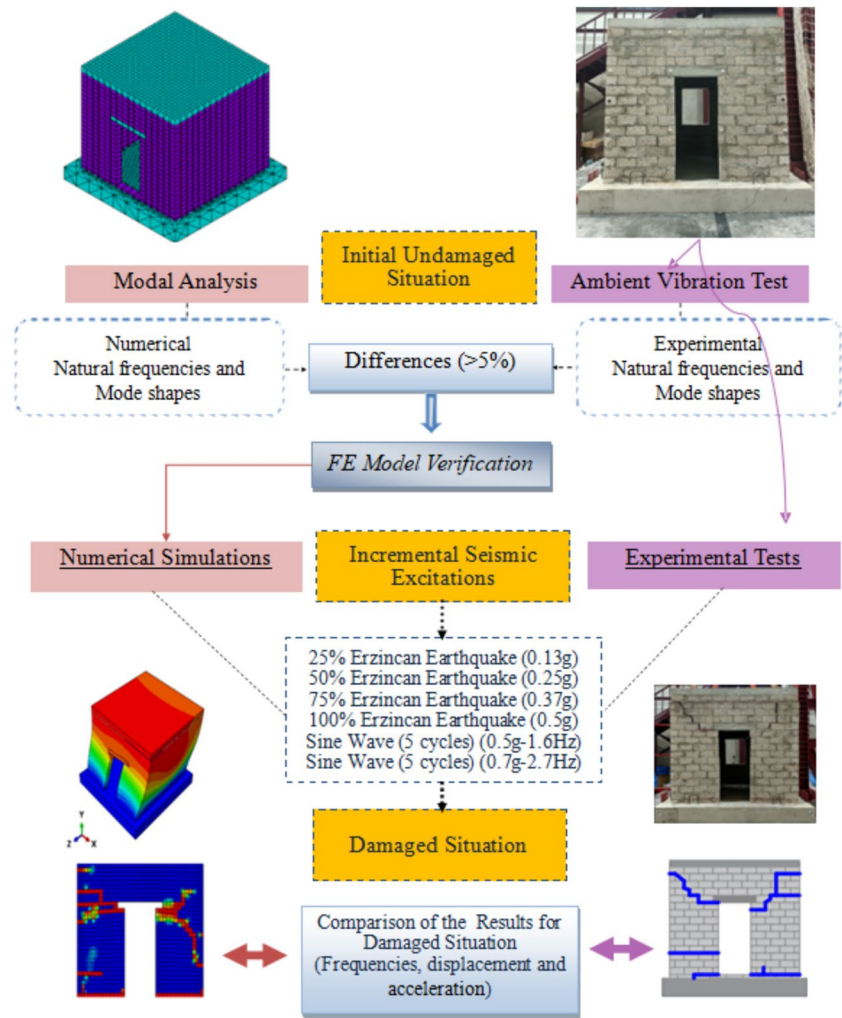


Table 1 Mechanical characteristics of materials

<i>Pumice concrete unit</i>	
Compressive strength (MPa)	5.4
Density (kg/m ³)	1290
Size (mm)	100 × 195 × 90
<i>Mortar</i>	
Compressive strength (MPa)	14.22
Tensile strength MPa	1.77

units. A 5 mm thick mortar layer was utilized, with a cement-to-sand ratio of 1:3 to adhere the masonry units together. The material properties of both the masonry wall units and the mortar can be found in Table 1.

The model is a 1/2 scale, single-span, single-story structure erected on a uniaxial shaking table. Figure 2 illustrates the layout and measurements of the test specimen. Thorough experimental investigations were conducted using the shake table to evaluate the performance of the model, built with local materials and workmanship. This URM model measures 1800 × 1800 mm and stands 1600 mm high, representing a full-scale URM structure with dimensions of 3600 × 3600 × 3200 mm. URM walls with a thickness of 100 mm were constructed with running bond masonry, a masonry technique commonly used in construction and wall building. Openings for doors and windows are present in the north and south walls of the model, while the east and west walls are solid without any openings. To support the dead load above the door and window openings, reinforced concrete

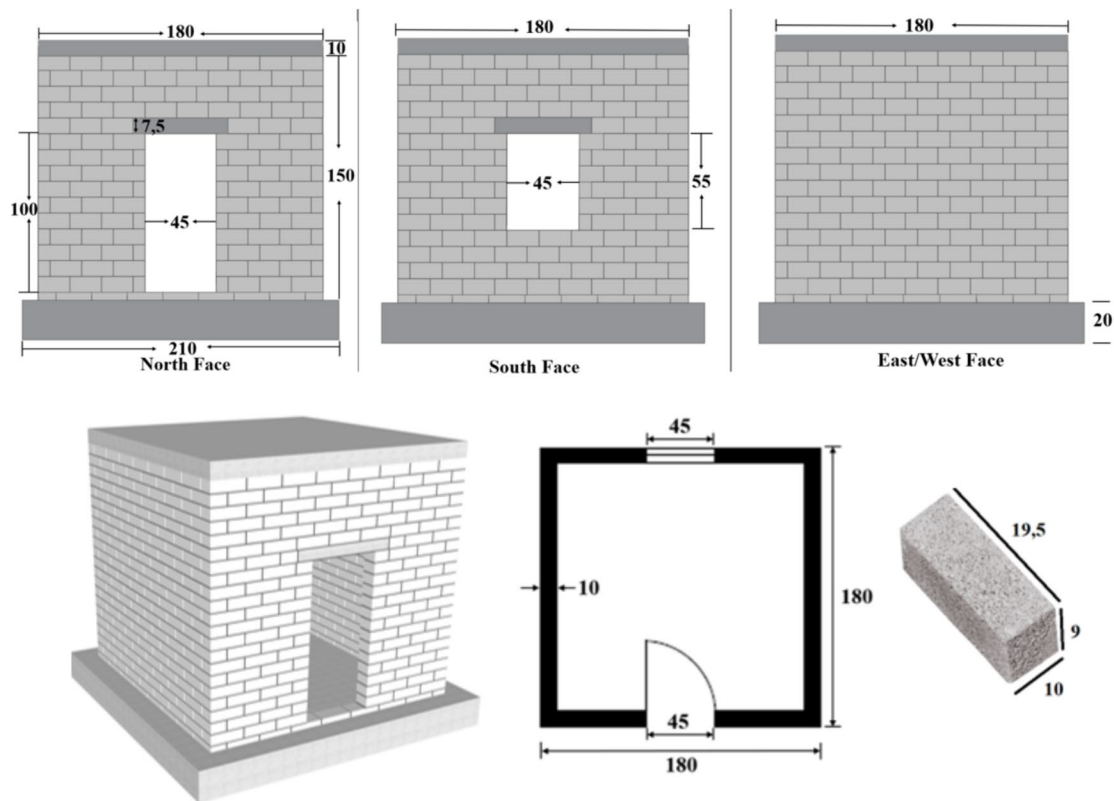


Fig. 2 Dimensions of the specimen and units (all dimensions in centimeters)

lintels were employed. The entire model has a total weight of 3750 kg. Figure 3 illustrates the various construction stages of the laboratory model, and a complete view of the model can be seen in the same figure. Due to the intricate impact of earthquake forces on structures, it was imperative to securely fasten the model to the shaking table. To achieve this, 16 rod holes were drilled into the foundation. Furthermore, to replicate a fixed boundary condition, the walls were securely integrated into the foundation of the model, ensuring no shearing, or sliding took place at the junction between the walls and the foundation during the entire testing process.

2.2 Scaling and designing of URM model

Shake table tests are a dynamic test method that allows a realistic investigation of the structural behavior of engineering structures under artificial or real ground motions. However, the characteristics of the shaking table and laboratory conditions often

necessitate the use of scaled specimens. Therefore, it is important to choose the suitable scaling approach to accomplish the desired objectives and the desired structural response in the experiments conducted on the shaking table. Scaling methods and scaling coefficient are generally preferred (If you're not using a 1/1 scale model) in shake table testing, depending on the structural behavior and parameters under investigation [33, 52, 53]. Harris and Sabnis [54] defined the scaling methods that can be used in such experiments as true replica model, artificial mass simulation model, prototype material model in which gravity effects are neglected."

The laboratory model was constructed following similarity laws, specifically the widely used Cauchy and Froude similarity laws [33]. Froude similarity laws establish the relationship between inertia and gravitational forces, while Cauchy laws relate inertia to elastic restoring forces [52]. Due to the Froude similarity laws, additional masses are necessary. For this research, Froude similarity



Fig. 3 Images of pumice wall construction stages

laws, commonly preferred in most studies, were employed [55, 56]. Since the model dimensions were based on the dimensions and capacity of the shaking table, 2 scale factors were used. The geometric dimensions and scale factors of the scaled masonry model in this study were adapted from the measurements used in the research by Kaya et al. [33]. An artificial mass simulation approach was used to determine the masses to be added to the slab in the shake table tests [57]. The additional mass to be added to the structure is calculated by Eq. 1. M_{m1} is the story mass to be added to the specimen, M_p is the unscaled building weight, S_L is the scale factor, and M_{m0} is the mass of the specimen. As a result of the calculations, the additional masses to be placed in the slab were determined to be approximately 2300 kg. These weights were connected to the experimental specimen slab with steel plates with dimensions of 250 mm × 1000 mm × 16 mm with 14 M14 rods in 6 sections (see Fig. 4).

$$M_{m1} = \frac{M_p}{S_L^2} - M_{m0} \quad (1)$$

In addition, the dimensions of the pumice units in the test specimen and the horizontal and vertical thickness of the mortar layer were also reduced by half scale. The scaling coefficients given in Table 2 were used to detail the test specimens from the full-scale building model. In this approach, the stress scale factor for concrete, mortar and masonry units was taken as 1. Therefore, unlike other scaling models, the artificial mass model allows the use of the same materials in the real building and the scaled building model. For this reason, no scaling was made in the material strengths used for the whole URM model. As shown in Table 2, the acceleration scale of the ground motion data utilized in the experiments is denoted as “1.0”. However, the time axis of the ground motion data is scaled by the square root of the scale coefficient [55, 58]. This results in lower displacement and velocity values in the scaled ground motion data. This makes it suitable for use on shaking tables where many ground motion capacities are limited.



Fig. 4 Additional floor masses on the slabs

Table 2 Scale factors

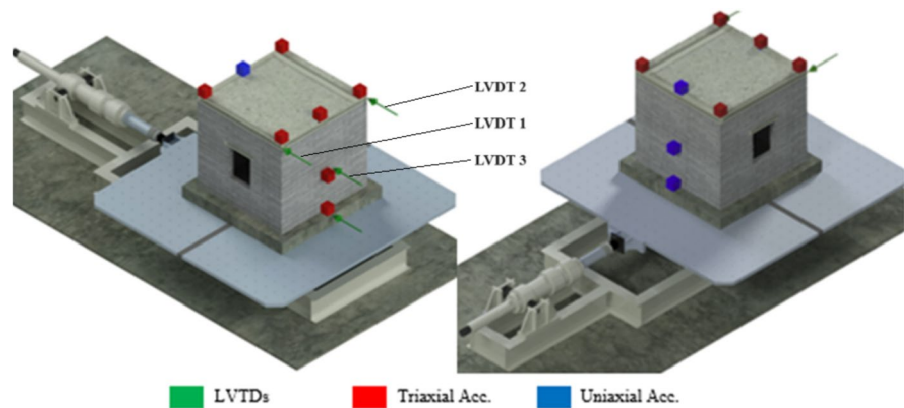
Parameters	Symbol	Scale factor
Length	l	$\lambda_l = \lambda = 2$
Modulus of elasticity	E	$\lambda_E = 1$
Acceleration	a	$\lambda_a = 1$
Velocity	v	$\lambda_v = (\lambda_l \lambda_a)^{1/2} = \lambda^{0.5} = \sqrt{2}$
Force	F	$\lambda_f = \lambda_E \lambda_l^2 = \lambda^2 = 4$
Stress	σ	$\lambda_\sigma = \lambda_E \lambda_l = \lambda = 1$
Strain	ε	$\lambda_\varepsilon = 1.0$
Area	A	$\lambda_A = \lambda_l^2 = \lambda^2 = 4$
Volume	V	$\lambda_V = \lambda_l^3 = \lambda^3 = 8$
Density	ρ	$\lambda_\rho = \lambda_E / (\lambda_l \lambda_a) = \lambda^{-1} = 0.5$
Mass	m	$\lambda_m = \lambda_\rho \lambda_l^3 = \lambda^2 = 4$
Energy	e	$\lambda_e = \lambda_E \lambda_l^3 = \lambda^3 = 8$
Time	t	$\lambda_t = (\lambda_l \lambda_a)^{1/2} = \lambda^{0.5} = \sqrt{2}$
Frequency	ω	$\lambda_\omega = 1 / \lambda_t / (\lambda_E \lambda_\rho)^{1/2} = \lambda^{-0.5} = 1 / \sqrt{2}$
Damping	ξ	$\lambda_\xi = 1.0$

2.3 Instrumentation

During the shake table tests, the test building was equipped with a range of sensors and instruments to capture various aspects of its response. This instrumentation included three uniaxial accelerometers (B&K 4507 B 005) with ± 7 g capacity

and a frequency range of 0.3–6000 Hz, seven tri-axial accelerometers (B&K 4506 B 003) with ± 14 g capacity and a frequency range of 0.3–2000 Hz, and four LVDTs (linear variable differential transformer) (RTL400) with a stroke length of 400 mm (± 200 mm), which were strategically placed to measure displacement and acceleration responses. Before instrumentation, the test model foundation was securely fixed to the rigid surface of the shaking table using four L-steel elements and 16 steel rods. Raw displacement (mm) and acceleration values (g) were obtained by multiplying the raw values measured as voltage with the calibration coefficients of the devices. Linear baseline correction was applied to raw displacement and acceleration data in order to smooth out fluctuation in data. In addition, the raw data obtained from the accelerometer was filtered with the range of 0.1–25 Hz by butterworth bandpass filter method. The occurrence of phase shifts, which could indicate structural weakening and damage, was monitored. These phase shifts were identified by analyzing the signals recorded by the accelerometers. Specifically, uniaxial accelerometers were affixed to the west wall to capture out-of-plane responses. Triaxial accelerometers were carefully positioned, as illustrated in Fig. 5, to capture responses both in-plane and out-of-plane. Additionally, to track alterations in relative displacement values and drift ratios

Fig. 5 Instrumentation setup



along the height of the model during the shaking, four LVDTs were utilized. Additionally, three video cameras were installed at different positions to observe and record any damage or torsional behavior of the structure throughout the shake table tests.

2.4 Seismic data

The earthquake data or sinusoidal waves are used as acceleration records in shake table tests [52, 53, 59]. The ground motion used during the experiments was the acceleration record of the East–West component of the 1992 Erzincan earthquake. The Pacific Earthquake Engineering Research Center (PEER, 2022) [60] data bank was used for acceleration records obtained from earthquakes that occurred in previous years. Since scaled test buildings were used during the experiments, this acceleration record should also be scaled according to the scaling principles as shown in Sect. 2.2. During the experiments, 0.25, 0.50, 0.75 and 1.00 times of the Erzincan earthquake record were applied incrementally. In this study, the intended damage condition was not achieved in the test models using historical earthquake acceleration records. In situations like this, an alternative approach involves introducing sine wave excitations into the test procedures. This is particularly useful when the chosen historical earthquake acceleration records push the capabilities of the shaking table to their limits [33, 61]. In this study, sinusoidal waves were used as input for incremental shake table tests together with historical earthquake data. Following the incremental shake table tests using the Erzincan earthquake ground motion, two sine waves with amplitudes of 0.5 g and 0.7 g, each consisting of 5 cycles,

were used to further increase the damage levels in the building. Table 3 provides a summary of the experimental sequence and essential features of the chosen ground motions. Figure 6 illustrates the response spectra and accelerations of the selected records. The ERZ earthquake records were applied for 15 s. 0.5 g sine wave 0.7 g sine wave were applied during 7.5 and 5 s respectively. The frequencies of 1.60 Hz and 2.70 Hz were chosen because the limits of the shaking table.

2.5 Ambient vibration tests

Ambient vibration tests were carried out at various stages of the experiment to determine the natural frequencies, mode shapes, and damping ratios of the structural system. OMA was utilized to extract these parameters. These tests were conducted to uncover the dynamic characteristics of the structure along two axes: the in-plane North–South direction and the out-of-plane East–West direction. The setup

Table 3 Applied test sequence

No	Earthquake	Code	PGA g	PGD cm	Frequency Hz
GM-1	25% Erzincan	ERZ25	0.13	3.50	–
GM-2	50% Erzincan	ERZ50	0.25	7.00	–
GM-3	75% Erzincan	ERZ75	0.37	10.50	–
GM-4	100% Erzincan	ERZ100	0.50	14.00	–
GM-5	Sine Wave (5 cycles)	–	0.50	5.00	1.60
GM-6	Sine Wave (5 cycles)	–	0.70	2.50	2.70

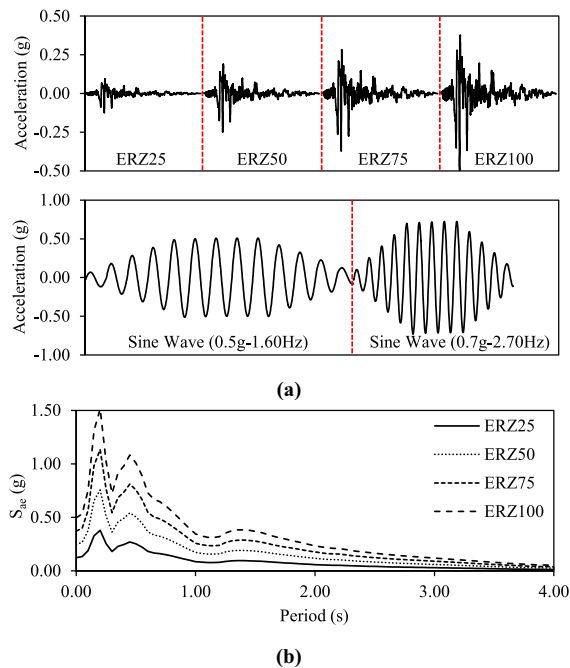


Fig. 6 Time history ground motion of selected seismic excitations: **a** acceleration and **b** response spectra

for the ambient vibration test is depicted in Fig. 7. Four triaxial accelerometers with a frequency range of 0.3–2000 Hz are placed at the top corners of the model where the vibrations are concentrated. The signals from the accelerometers are transferred to a 17-channel B&K 3560 data acquisition system. In parallel with studies in the literature, each OMA test was taken for 15 min. The signals were then processed with OMA [37] and PULSE [62] software. The mode shapes exhibited by the building type structures are longitudinal translation, transverse

Fig. 7 The setup and location of the accelerometers for the ambient vibration test



translation, and torsion. These mode shapes were taken into consideration in this study.

3 Experimental results

3.1 Assessment of post-earthquake damage

In the shake table test, the changes in the test building were examined experimentally and visually before and after each ground motion. Visual observations include crack propagation and crack widths. The experimental investigation involved the assessment of the dynamic characteristics of the structure. Ambient vibration tests were conducted both prior to and after the seismic excitations, employing the OMA method. By comparing the dynamic properties obtained before and after the tests, alterations in the structural behavior and damage to the model were ascertained.

Grünthal [63] developed the damage classification system specified in the European Macroseismic Scale (EMS-98). In this study, this classification system was used to determine the damage conditions of the model. EMS-98, the damage cases for masonry buildings are more limited than for reinforced concrete buildings. Five damage states (DS) are used for masonry buildings as shown in Table 4.

3.1.1 Damage propagation

Masonry structures are more rigid structures than reinforced concrete structures. Therefore, they exhibit a brittle behavior and are damaged by seismic movements. Considering this model, due to the high strength of the mortar, the use of solid pumice elements and the aspect ratio of the model being 1, it was not damaged in certain seismic excitations and

Table 4 Damage ratings for masonry buildings according to EMS-98 scale

Damage state	Masonry damages	
DS-1	There are microcracking in the walls, a few small amounts of plastering and fall block elements falling from the roofs of the buildings. No structural damage.	
DS-2	A building with fissures in numerous walls, large fragments of plastering falling off and chimneys partially collapse. Slight structural damage.	
DS-3	Significant fissures on numerous walls, chimney damage at roof level and collapse of non-structural components. Moderate damage.	
DS-4	Severe structural damage and partial collapse. Very severe crack pattern, large crack expansion, partial roof, and floor collapse.	
DS-5	Complete or partial collapse of structural elements.	

showed brittle fracture in ground motion with high acceleration levels. In this study, no damage was observed on the wall surfaces after ERZ 25, ERZ 50, ERZ 75, ERZ 100 and GM-5 seismic excitations during the shake table test. The test building exhibited a rigid behavior and was suddenly damaged during the GM-6 five-cycle sine wave excitation and passed into the DS-5 type structural damage and collapse state. Figure 8 provides a visual representation of the crack propagation on the outer surfaces of the model in both the undamaged and DS-5 damage state. The blue markings indicate the cracks that were observed during the experiment. At this point, it is noteworthy that a substantial portion of the cracks in the east and west-facing walls exhibited a near-horizontal orientation. These cracks were primarily a result of shear movements occurring in the first two brick courses just below the slab level. Furthermore, in-plane

deformations led to the formation of a diagonal crack in the north-facing wall. This diagonal crack extended from the upper corners of the door to the slab, as well as from the four corners of the window to the slab and foundation. It is worth mentioning that previous studies have delved into the structural behavior of masonry structures constructed using brick units [33, 64]. The crack propagation observed in the test buildings constructed from both brick and pumice units exhibited some similarities. Diagonal cracks did not occur only along the mortar bed joints like the crack propagation of the brick model. Pumice units, which have a lower strength than bricks, were damaged by reaching their strength capacity before mortar. Additionally, shear/shear cracks were noticed in the first two brick courses situated just above the foundation level, and these cracks were also observed around the lintel beams. Toward the conclusion of the test, it was

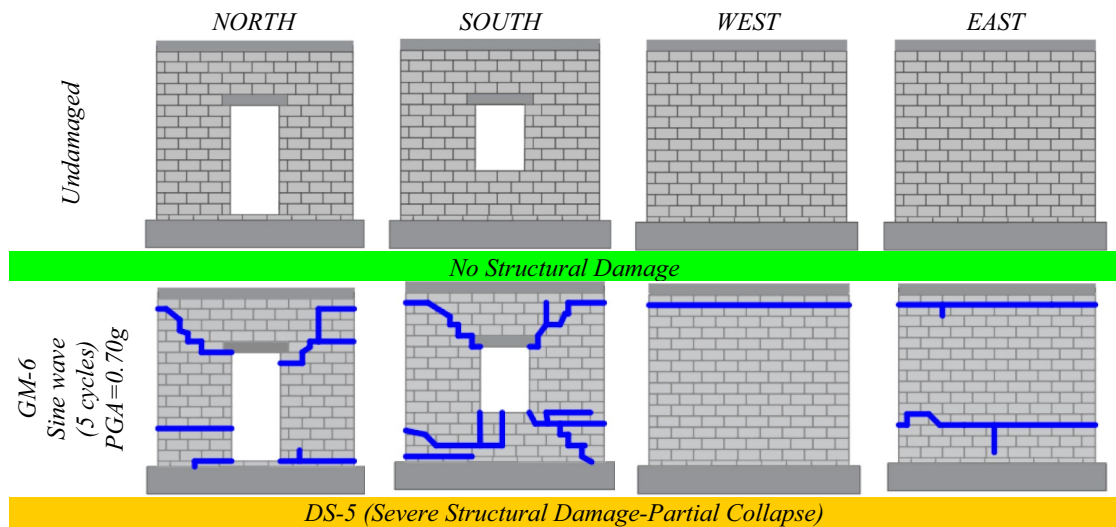


Fig. 8 Crack pattern of the test specimen

noted that the crack widths had reached dimensions in the centimeter range. Consequently, the structural damage was classified as DS-5. For visual reference, Fig. 9 presents the damage pattern of the DS-5 test building as viewed from the exterior.

3.1.2 Ambient vibration tests results

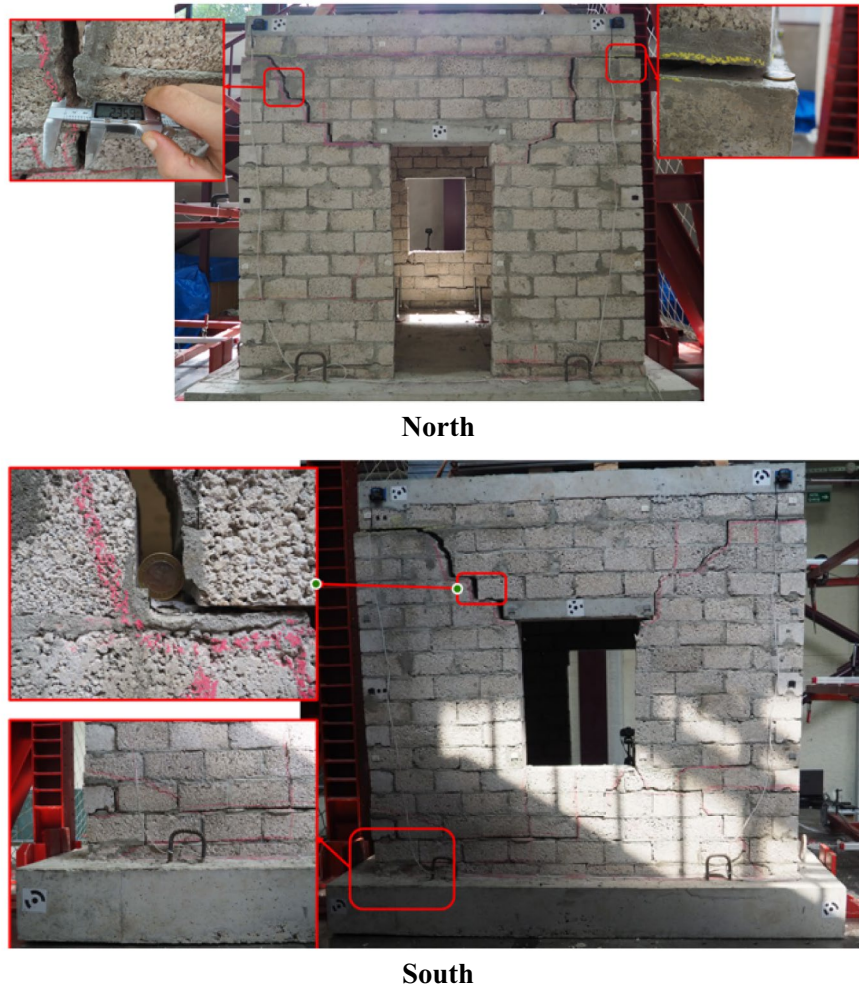
In this study, besides visual inspections, an investigation into the connection between the damage and the modal characteristics of the masonry building during the progressive shaking table tests was conducted. The accelerometers and data acquisition devices described in Sect. 2.5 were used in the OMA method used to obtain the natural frequency, mode shapes, and damping ratios of the laboratory models. The methods and measurement procedures described in the same section were applied, and dynamic characteristics before and after each seismic excitation were determined.

Following the Enhanced frequency domain decomposition (EFDD) method, the singular values of Spectral Density Matrices for the dataset were graphed, allowing us to examine the first three natural frequencies and their respective mode shapes. Figures 10a and b show the plots of the undamaged model with no load and with additional mass. Also, the graphs of the damage states for the DS-5 type are given in Fig. 10c. The three natural frequencies for the undamaged

model-no additional mass case after placement on the shaking table are 26.52 Hz, 43.48 Hz and 95.47 Hz, respectively. The pertinent mode shapes of the model were identified as longitudinal, transverse, and torsional modes. After the addition of 2.3 tons of extra mass, the dynamic characteristics of the model shifted, and the first three natural frequencies for the undamaged, additional load case were measured at 22.53 Hz, 29.8 Hz, and 76.41 Hz, respectively. A downward movement is observed in the frequencies after the additional mass is added. The additional mass affects the longitudinal, transverse, and torsional frequencies by 15%, 31% and 20%, respectively. Also, the first three natural frequencies of the damaged conditions for DS-5 are 20.55 Hz for longitudinal mode, 22.79 Hz for the transverse mode and 73.29 Hz for the torsional modes (see Fig. 10). No significant result was obtained from the OMA measurement for the unloaded condition of the heavily damaged pumice model. The heavily damaged model collapsed on the shaking table.

Figure 11 shows a comparison of the first three mode shapes obtained from the damaged condition (DS-5 type) and the experimental measurements. The conclusion is that structural degradation changes the mode shapes. It should also be noted that the amplitude of each mode exhibits significant variation in its oscillation characteristics. This means that each mode of vibration, influenced by its natural frequency and

Fig. 9 Damage modelling of the outer surface of the test building for type DS-5

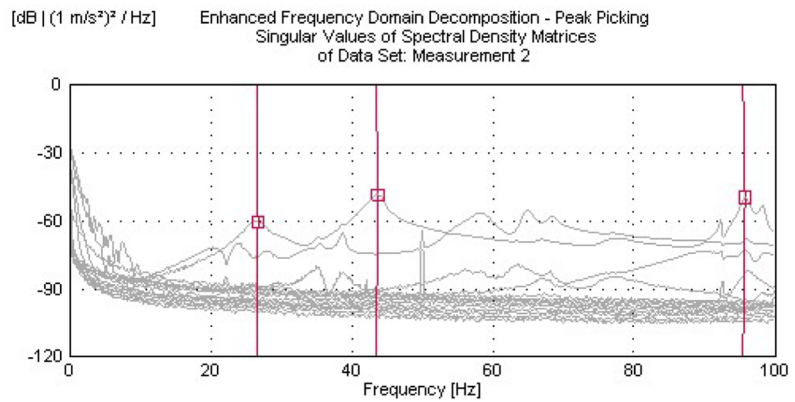


damping properties, shows distinct oscillation amplitudes. It is important to highlight that these variations in oscillation amplitude are critical for understanding the dynamic behavior of the system, as each mode responds differently to external excitations and internal structural properties. Throughout the incremental dynamic measurements, several transversal and horizontal cracks became evident, particularly at the upper and lower sections of the walls. It is shown that these damages, which reduce building stiffness, generate decreases in natural frequency values, particularly in transverse modes. When the building reached the DS-5 damage condition, the most significant frequency drop was observed. Specifically, there was an 8.8% decrease in the longitudinal mode, a 23.5% decrease in the transverse mode, and a 4.1% decrease in the torsional mode compared to the undamaged

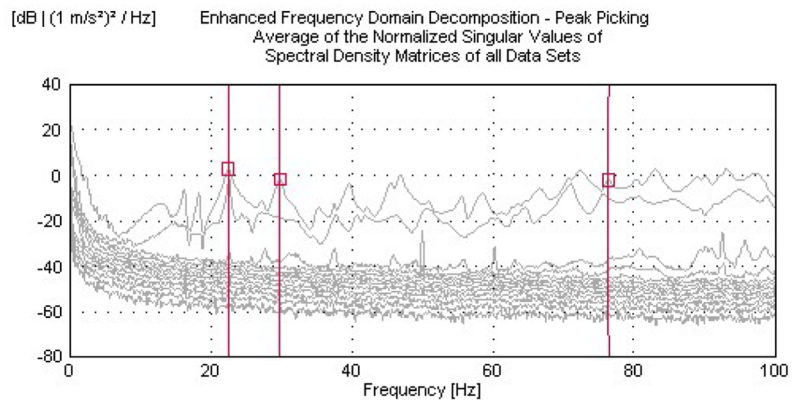
condition (cases with additional mass). Table 5 shows the dynamic characteristics for the pumice model after the shaking table test and for the undamaged cases obtained according to the EFDD method. In transverse and longitudinal modes, damping ratios decreased with additional mass. However, in torsional mode, damping ratios increased slightly with additional mass. Similar conditions were repeated with damage. A maximum 68% decrease in damping ratios was observed with damage.

Modal assurance criterion (MAC) values are calculated to analyze the similarity of the mode shapes, considering the changes and revealing the impact of damage. A MAC value close to 1.00 indicates a high similarity between mode shapes for different cases, whereas significant decreases in MAC values suggest the presence of damage, approaching

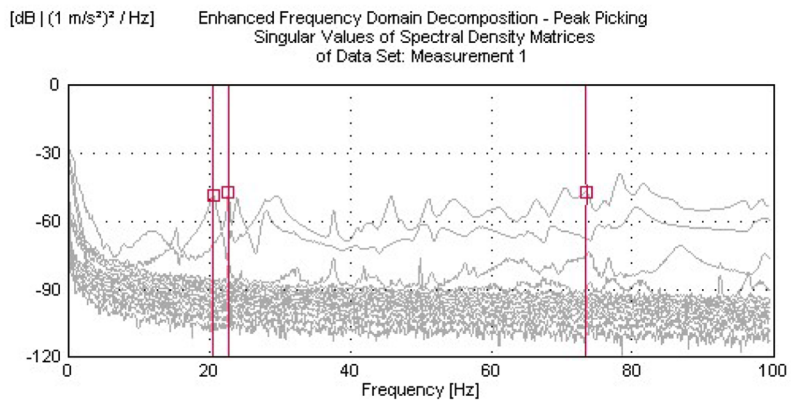
Fig. 10 The singular values of spectral density matrices



(a) For the Undamaged-No Additional Mass



(b) For the undamaged-Additional Mass



(c) For DS-5 (severe structural damage and partial collapse)

a value of 0. Figure 12 illustrates the decrease in MAC values for each masonry model by comparing the mode shapes in undamaged and damaged states after shaking table tests with increasing damage.

The stiffness of the structures decreases as soon as the damage starts to occur. This situation causes a change in dynamic characteristics. In fact, the natural frequency and damping ratios of a structure are

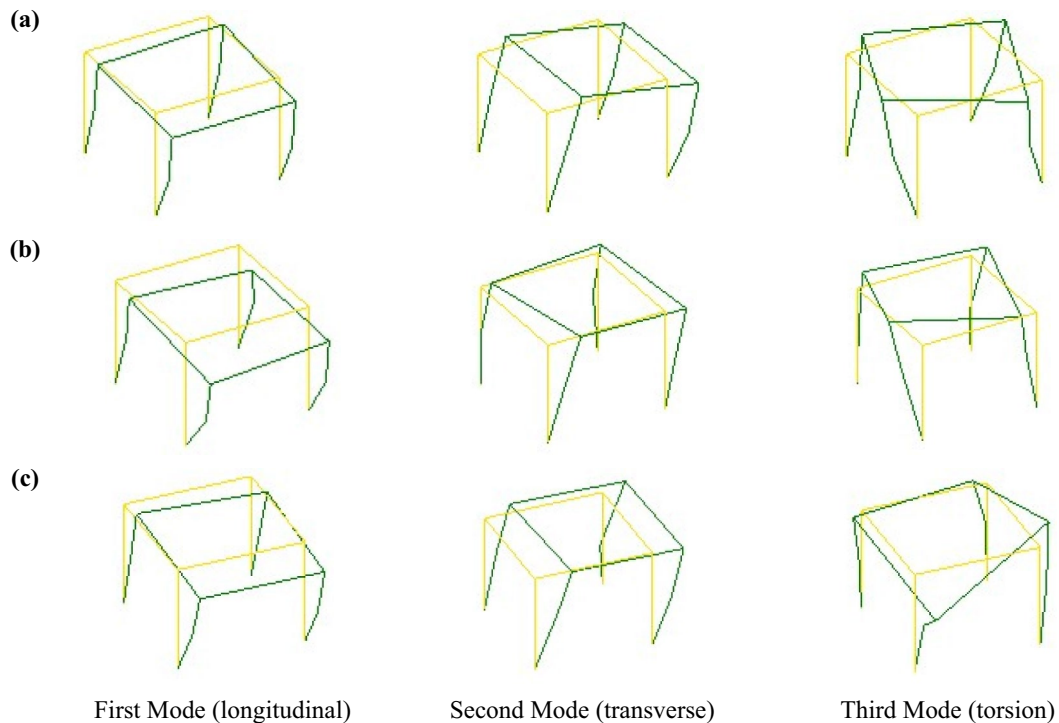


Fig. 11 Experimental mode shapes of the building **a** for the undamaged-without additional mass case **b** for the undamaged-with additional mass case and **c** for DS-5

Table 5 Dynamic characteristics after shaking table tests

Mode	Undamaged (no additional mass)		Undamaged (additional mass)		DS-5 (additional mass)		Frequency difference* (%)
	Frequency (Hz)	Damping ratio (%)	Frequency (Hz)	Damping ratio (%)	Frequency (Hz)	Damping ratio (%)	
Longitudinal	26.52	6.78	22.53	1.92	20.55	1.119	8.8
Transverse	43.48	2.51	29.8	1.62	22.79	0.52	23.5
Torsion	95.47	0.527	76.41	0.55	73.29	0.79	4.1

*Frequency differences of the DS-5 type damage state compared to the undamaged state (with additional mass)

intricately tied to its mass and stiffness. Researchers have associated the change in natural frequency, mode shape and damping ratios with structural damage [65]. There is no visible change in the natural frequency values until the DS-5 type damage condition. It can be said that this steady state in the frequencies is due to the story masses added to the models, which makes the structure more stable.

Mendes et al. [66] introduced a Damage Index (DI) designed to assess the damage associated with each structural mode. A DI value of 1 indicates a complete

loss of strength, while 0 indicates no damage. The DI value is modified according to the cumulative damage resulting from the shaking tests and its determination is governed by Eq. 2.

$$(DI)_n = 1 - f_n/f_1 \quad (2)$$

where, f_n is the natural frequencies values estimated after the n th shake table, and f_1 indicates the natural frequency values determined from pre-damage tests.

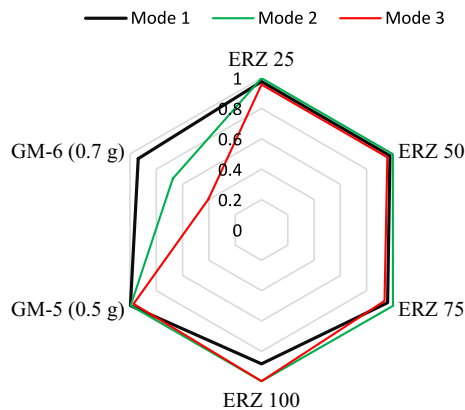
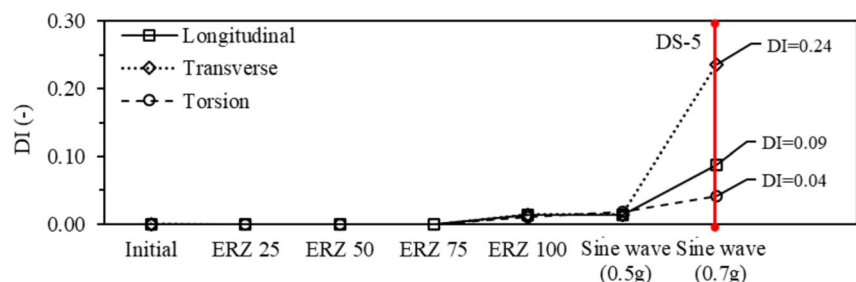


Fig. 12 Change of MAC values with increasing damage

There was no increase in DI values observed until GM-3 seismic excitation. However, after the experiment with GM-6, the DI values were determined as 0.09 for the longitudinal mode, 0.24 for the transverse mode, and 0.04 for the torsional mode at the DS-5 damage state. DI values of the first three natural frequencies of the building for the damage conditions are presented in Fig. 13. Kaya et al. [33] found similar DI values for the DS-4 damage type of the 1/2 scale model constructed with hollow brick without additional masses. Tomassetti et al. [59] investigated a single-story masonry building and calculated the DI value of the test sample as 0.15. In the shake table tests of masonry models carried out by Kallioras et al. [67] and Miglietta et al. [68], DI values of 0.51 and 0.32 were obtained for the DS-4 damage condition, respectively. It is clear that there is no consistency between the values obtained from the literature and the values obtained from this study. The variation in results can be ascribed to several factors, including distinctions in material properties, diverse simulation methods, and the utilization of distinct seismic excitations in the experiments.

Fig. 13 DI quantifies the three natural frequency values under the damage condition



3.2 Structural response assessment

In this section, the evaluation of the building responses is presented by analyzing the data collected from the measurement equipment in the test setup. The test setup includes uniaxial and triaxial accelerometers. Furthermore, LVDTs were used to determine the displacements along the model height. The experimental data were processed and analyzed for each seismic excitation. Inter-story drift ratios (IDRs), relative displacements, absolute accelerations and base shear forces were calculated.

3.2.1 Evaluation of inter-story drift ratios and relative displacements

The variations in the maximum relative displacement values associated with every seismic excitation are displayed in Fig. 14. The relative displacements identified at the slab and center height of the test building are studied comparably. As illustrated in Fig. 14, the upward relative displacement values increase with damage after the GM-5 earthquake. Figure 15 illustrates the changes in relative story displacement ratios, calculated by using the maximum relative displacements after the shake table tests. Calvi [69] has introduced drift limits for masonry structures based on their associated damage conditions. The threshold drift ratios are defined as follows: less than 0.1% for DS-1, 0.1% for DS-2, from 0.1 to 0.3% for DS-3, from 0.3 to 0.5% for DS-4, and greater than 0.5% for DS-5. In this study, the maximum IDR for seismic excitations leading to severe damages and DS-5 type cracks was determined to be 1.12%. These values partially meet the thresholds given in the literature. However, a possible reason for the partial agreement can be attributed to limitations in continuous monitoring of the relation between damage and engineering parameters (displacement, IDR, etc.) during shake

Fig. 14 Variation of maximum relative displacements measured

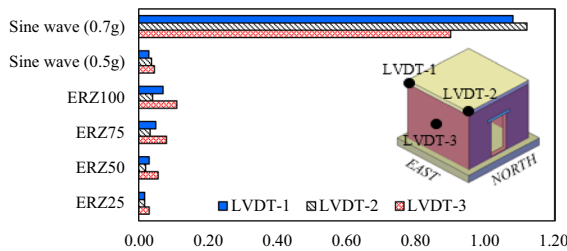
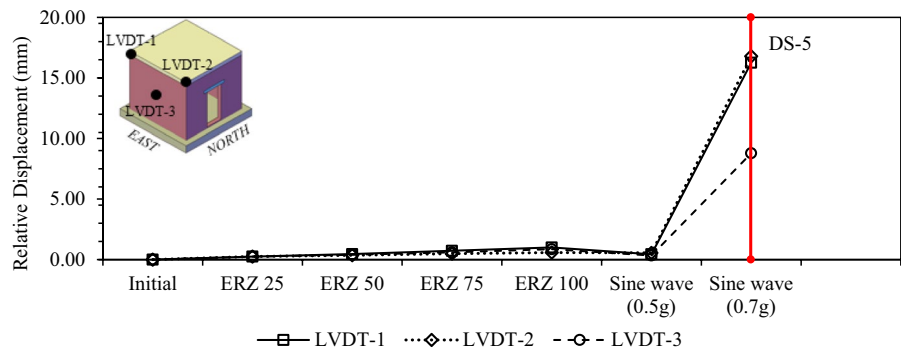


Fig. 15 Variation of maximum IDRs measured

table tests. As a result, it becomes difficult to unambiguously associate a particular damage condition with a specific threshold value and instead the damage condition is associated with a range of deformations [59].

Penna et al. [70] conducted research on the cyclic behavior of masonry walls and found that the ultimate drift ratio for the examined masonry wall was approximately 0.65%. They considered this drift ratio as the point at which the maximum load reduced by 20%. However, in the study by Arslan and Celebi [71], this drift ratio was determined to be around 3.33%. As evident from these findings, there is a wide range of drift ratio limits for this type of walls, influenced by various factors such as aspect ratio, loading mechanism, and loading protocol.

3.2.2 Torsional behavior and response

The experimental building has door and window openings in the north and south wall sides. For improved comprehension of the torsional behavior of the model, these openings were selected with different dimensions. The window opening in the south wall has an area of 0.25 m², while the door opening in the north wall has an area of 0.45 m². These

opening differences affect the in-plane behavior of the wall and cause the south and north walls to reach different displacement values. Therefore, the structure exhibited significant torsional responses, leading to differences in the relative displacements measured at the roof corners. These differences were determined to be up to 42%. (see Fig. 14). Torsional responses become more pronounced depending on the damage and crack size [33]. For a comprehensive explanation, Fig. 16 presents the time history of the relative displacements measured at the roof corners during GM-5 and GM-6 seismic excitations. It is seen that the torsion increases in the damage cases corresponding to the DS-5 type where the asymmetric collapse mechanism occurs. The results of the previous studies agree with the results of this study. Kayirga and Altun [72] and Kaya et al. [33] showed that walls with different wall openings exhibit torsional behavior during earthquakes. These studies also highlighted the substantial influence of torsional response on the structural damage of masonry buildings.

3.2.3 Acceleration results

After the shake table tests, abrupt changes and phase shifts in the signal data recorded by the accelerometers can serve as indicators of the onset of weakening and the propagation of cracks. Earlier research on masonry structures has linked the discrepancies between the data and the extent of damage incurred [73]. For this aim, the acceleration responses at the wall-foundation intersection, middle height and slab heights were plotted comparatively for the shake table tests performed for GM-1, GM-5 and GM-6 seismic excitations where the DS-5 type occurred (see Fig. 17). The acceleration-time records for GM-5, which did not cause

Fig. 16 Relative displacements and torsions obtained from slab corners (a GM-5 five-cycle sine wave (PGA=0.50 g); b GM-6 five-cycle sine wave (PGA=0.70 g))

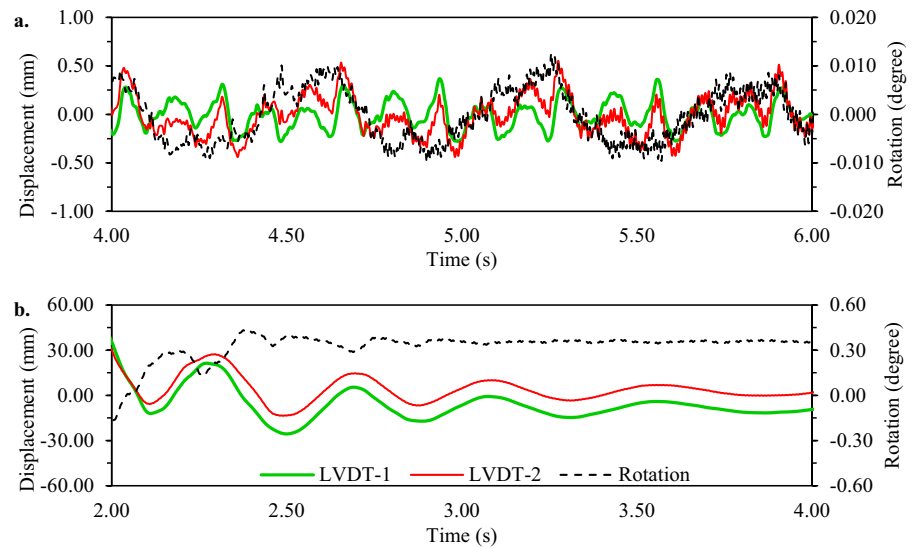
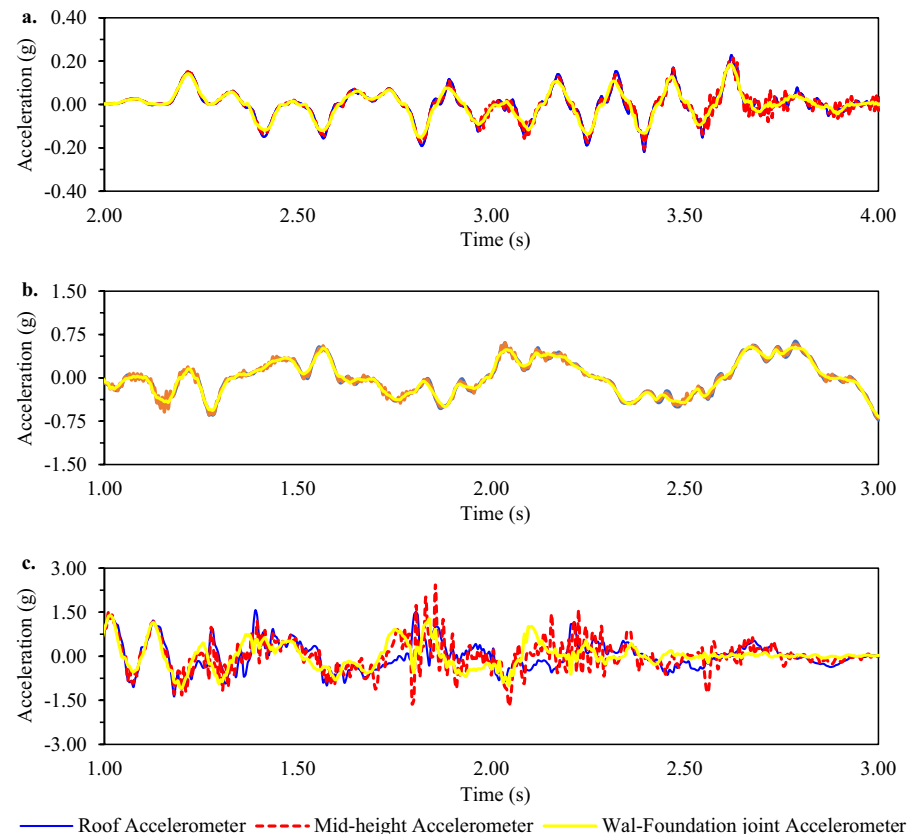


Fig. 17 Acceleration plots in time domain obtained from accelerometers at the wall-foundation junction, mid-height and slab elevation: a GM-1 for ERZ-25 earthquake (PGA=0.13 g), b GM-5 for five-cycle sine wave (PGA=0.50 g) and c GM-6 for five-cycle sine wave (PGA=0.70 g)



any damage, showed good agreement with no phase shifts or signal jumps. Damage to GM-6 was observed. Figure 17c illustrates substantial differences in acceleration responses at mid-height and

slab elevations when damage occurred. This was accompanied by the presence of cracks with widths on the order of centimeters. As a result, acceleration responses recorded from various levels might

serve as significant indications for detecting damage in masonry buildings.

In Fig. 18, a comparison of maximum acceleration values at various points of the test building was shown, including the wall-foundation intersection, middle height, and slab heights. Initially, these values showed minimal differences in the undamaged condition. However, as the damage progressed, discrepancies in maximum accelerations compared to the initial undamaged state became apparent. The acceleration values increased significantly during the last ground motion, GM-6 (0.7 g). Due to the damage pattern, at mid-height, the acceleration peaked at 2.43 g, the highest recorded during the tests.

3.2.4 Determination of base shear forces

Figure 19 shows the variations in the base shear force values along the test sequence. The base shear forces were determined by summing the mass within the area of influence of each accelerometer located

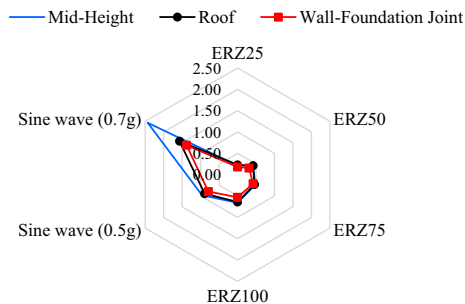
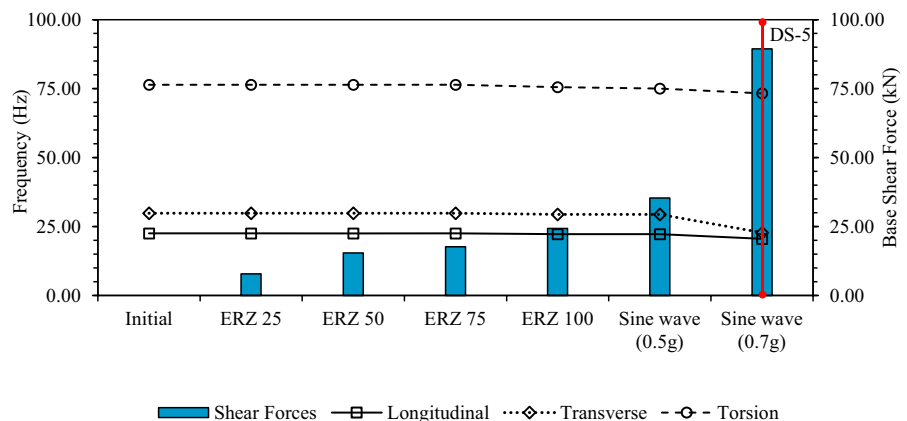


Fig. 18 Variation of maximum accelerations (g) measured

Fig. 19 Fluctuation of natural frequencies based on recorded base shear forces at the seismic excitation



at the slab level and multiplying it by the corresponding accelerations recorded by that accelerometer. The total mass considered for this calculation was 3.75 tons, which accounted for half of the wall masses, the weights of the reinforced concrete slabs, and additional mass. The test results indicate that the base shear forces remain relatively consistent in the tests conducted with natural acceleration records. Furthermore, as the base shear forces increased, there was a trend of decreasing natural frequencies. Specifically, during the test with GM-6, which involved a five-cycle Sine Wave with a peak ground acceleration (PGA) of 0.7 g, resulting in a base shear force of 89.4 kN, notable diagonal cracks with shear displacements on the order of centimeters developed. This seismic excitation led to the highest-frequency degradation of the structures.

4 Numerical campaign

In the study, a masonry structure made of pumice was modeled in three-dimensional nonlinear finite-element analysis, and its behavior against different earthquakes was investigated. In the numerical analysis of the building model, seismic inputs were systematically applied with increasing amplitudes. These amplitudes were expressed as a percentage of the peak ground acceleration (25%, 50%, 75%, and 100%). In addition, two sinusoidal waves were introduced using a method that involved transferring results between different analyses. A numerical analysis was carried out using ABAQUS v2019 [74] software to assess the dynamic characteristics (frequency

and mode shape) of a structure under both undamaged and damaged scenarios associated with the first three modes. The analysis also involved studying the damage patterns and displacement values. To confirm the accuracy of the numerical findings, a verification process was carried out by comparing the results with experimental data. This comparative analysis aimed to assess the reliability of structure in the face of seismic forces. Such an investigation is of paramount importance in evaluating how effectively the numerical model replicates the actual behavior of the structure under seismic loads.

4.1 Material properties

To characterize the concrete material properties and damage behavior, a plastic concrete damage model (CDP) was employed. This model, as demonstrated by Resta et al. [75], has proven successful in finite element numerical simulations for investigating both the static and dynamic behavior of masonry structures. The material parameters used to represent the masonry wall can be found in Table 6. It is important to highlight that detailing the plasticity properties of materials often necessitates a range of experimental tests, which were not conducted as part of this study. As outlined in the research by Abdelmoneim et al. [76], the plasticity parameters were derived through a trial-and-error updating process. The findings revealed a robust concurrence between the experimental outcomes and the numerical models, suggesting the effectiveness of the proposed models in predicting the overall behavior of specimens. In addition, the elastic properties were determined based on test

results from pumice wall experiments. Please refer to the Table 7 for detailed mechanical properties of the concrete and steel used in the study.

4.2 Finite element model of the pumice building

In this study, the researchers developed a numerical model for studying the seismic behavior of half-scale masonry structures (see Fig. 20). The element type used for walls and concrete components (lintels, slabs, and foundation) were modeling using 3D 8-node solid element with reduced integration (C3D8R) from ABAQUS library. Each node of the C3D8R element possesses three translational degrees of freedom, typically in the x, y, and z directions. These degrees of freedom allow the element to model deformations in three-dimensional as indicated before by Zhao and Alex [78]. Two noded truss element (T3D2) was utilized for reinforcing bars model that implanted inside the concrete. Perfect connection was expected between the reinforcement and the surrounding concrete. The macro modeling approach, which is pumice units' interface are spread out in the continuum, was considered to depict the masonry construction. The pumice units are treated as a homogenous and continuous material, and this technique is ideal for developing comparatively larger and more complex masonry structures. Tie contact type between the components of the model was considered. Therefore, the following types of contact constraints were assessed; entire bond (tie), solid to solid, and embedded region restrictions. Geometrically nonlinear behavior was acknowledged in FE model. Different mesh sizes were employed to evaluate the convergence and the final mesh size is decided dependent on the complexity, amount of detail and analysis needs of the structure. When selecting the mesh size in this research, fine meshes of 50 mm

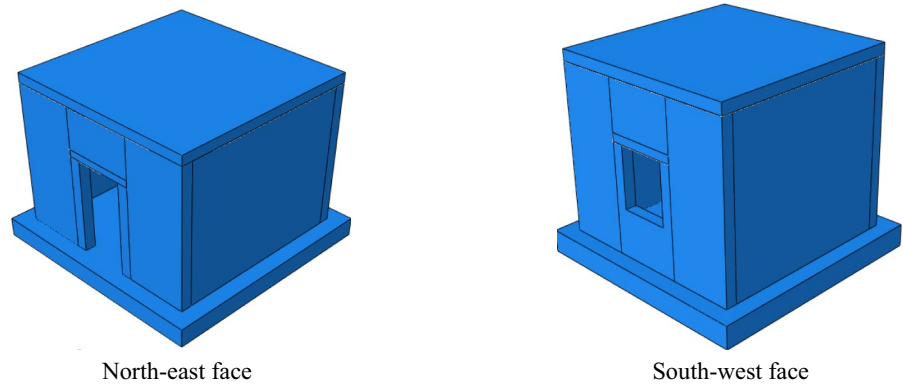
Table 6 Material properties of the pumice for CDP model [76]

Parameters		Values
Elasticity	Density (kg/m^3)	1400
	Modulus of elasticity E_m (MPa)	2006
	Poisson's ratio ν	0.2
	Tensile strength (MPa)	0.22
Plasticity	Dilatation angle	34
	Eccentricity	0.1
	Initial/biaxial/uniaxial ratio	1.16
	Kc	0.67
	Viscosity	0.001

Table 7 Mechanical properties of steel [77]

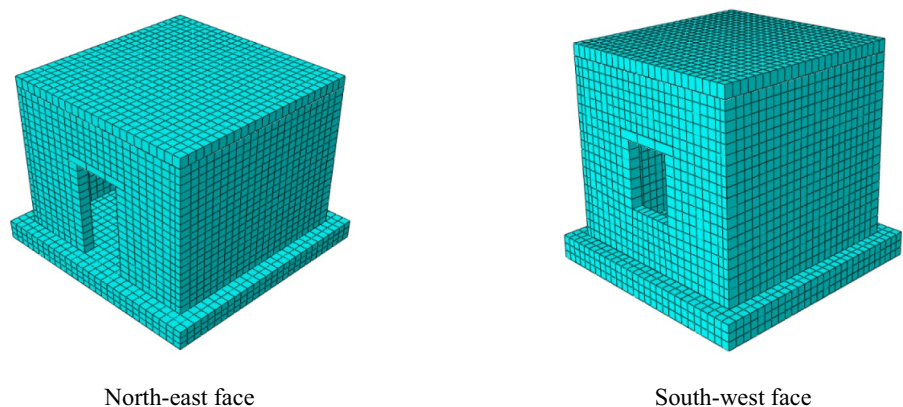
Parameters		Values
Concrete	Density (kg/m^3)	2400
	Modulus of elasticity E_c (MPa)	32,000
	Poisson's ratio ν	0.2
Steel	Density (kg/m^3)	7800
	Modulus of elasticity E_s MPa	210,000
	Poisson's ratio ν	0.3

Fig. 20 FE geometric model of the pumice building



were suited to the building model; rough meshes of 100 mm were applied to the steel materials were selected as the ideal mesh size as shown in Fig. 21. A mesh consisting of 19,630 elements and 27,996 nodes were utilized to model. In addition, the overall weight of the FE model was determined to be 3.750 tons. The fixed base/support, which simulates the impact of a solid base and provides the necessary rigidity for structural analysis, was assumed as a boundary condition for the nonlinear time history analysis of the masonry structure. Therefore, explicit method is more robust in finding solutions and requires less memory to handle large and complicated models [74]. An explicit dynamic solver is recommended for its ability to define comprehensive contact conditions for complex interaction problems while avoiding numerical convergence difficulties. Additionally, this solver employs a consistent large-deformation theory, which can accurately represent significant rotations and large deformations. In this study, the explicit dynamic solver technique is also chosen.

Fig. 21 FE mesh for the pumice building



4.3 Undamaged model

The ABAQUS software was employed to analyze and determine the dynamic characteristics of the pumice masonry structure in its undamaged condition. This analysis involved the investigation of natural frequency values and corresponding mode shapes. The first three mode shapes of the pumice masonry structure are seen in Fig. 22.

Figure 22 illustrates the three identified mode shapes: the first corresponds to a longitudinal mode, the second represents a transverse mode, and the third represents a torsional mode. Table 8 provides the natural frequency values determined for the structure in its original, as-built condition. Rayleigh damping constants, which describe the damping properties, were computed with a 5% damping ratio and were calculated between the first and the third mode for the transient analyses. Specifically, the calculated values for the alpha (α) and beta (β) coefficients of the undamaged model state were 13.403 and 0.00149,

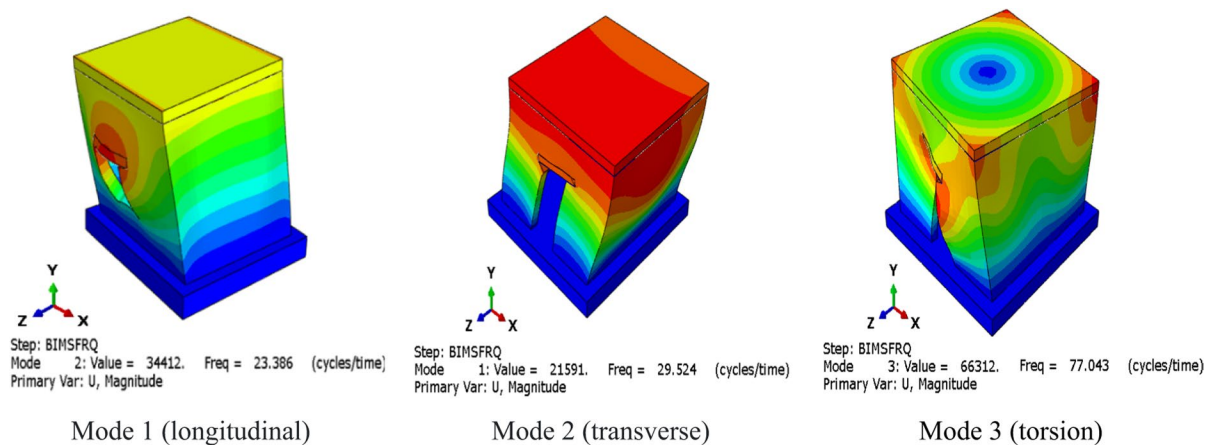


Fig. 22 First three mode shapes of the Initial model

Table 8 Experimental and numerical frequencies of the undamaged model

Mode	Frequency (Hz)		Maximum difference (%)
	Experimental	Numerical	
1st (longitudinal)	22.53	23.39	3.82
2nd (transverse)	29.80	29.52	1.00
3rd (torsion)	76.41	77.04	0.80

respectively. These coefficients are essential for characterizing the damping in the structural system.

4.4 Damaged model

In the numerical analysis, when the structure was subjected to various percentages of the Erzincan peak acceleration records (ranging from 25 to 100%) and sine waves with amplitudes of 0.5 g, no damage was observed on the walls' surfaces. However, significant damage patterns were obtained when the structure was subjected to sine waves with amplitudes of 0.7 g. To validate the numerical model, the natural frequencies from the experimental and numerical studies were compared. After the nonlinear time history analysis, a second analysis, called modal analysis, was conducted using the ABAQUS software. This modal analysis was performed on the damaged numerical model to evaluate changes in its dynamic characteristics. Modal analysis helps in understanding how the dynamic properties of the structure change after it has been subjected to seismic forces or damage.

As shown in Fig. 23, the difference between the first frequency of the experimental model (22.79 Hz) and the numerical model (23.66 Hz) is approximately 3.81%, indicating an acceptable level of accuracy in the numerical model results. Figure 23 illustrates the first three mode shapes. Table 9 provides a summary of the natural frequency values obtained for the damaged condition of the pumice masonry structure.

The changes in the first three natural frequency values of pumice model were recorded and shown in Table 10 for the damaged and undamaged pumice models. The comparison of natural frequency values between the undamaged and damaged models demonstrates how damage affects the first three modes. Specifically, in numerical analysis mode 2 exhibits a significant change, corresponding to a reduction of approximately 20%. The door and window openings in the north and south walls predominantly impact the transverse mode in the corresponding direction. Consequently, the transverse mode exhibited a greater decrease compared to the other two modes.

Nonlinear time-history analyses of the pumice building were conducted using the full Newton method within the finite element simulation. This method allows for the detailed analysis of the structure behavior over time, considering the complex interactions and nonlinearities that can occur during dynamic events, such as earthquakes. In all simulations, a damping ratio of 5% is used. In the experimental results, no damage was observed on the wall surfaces when exposed to different percentages of the maximum acceleration records of Erzincan

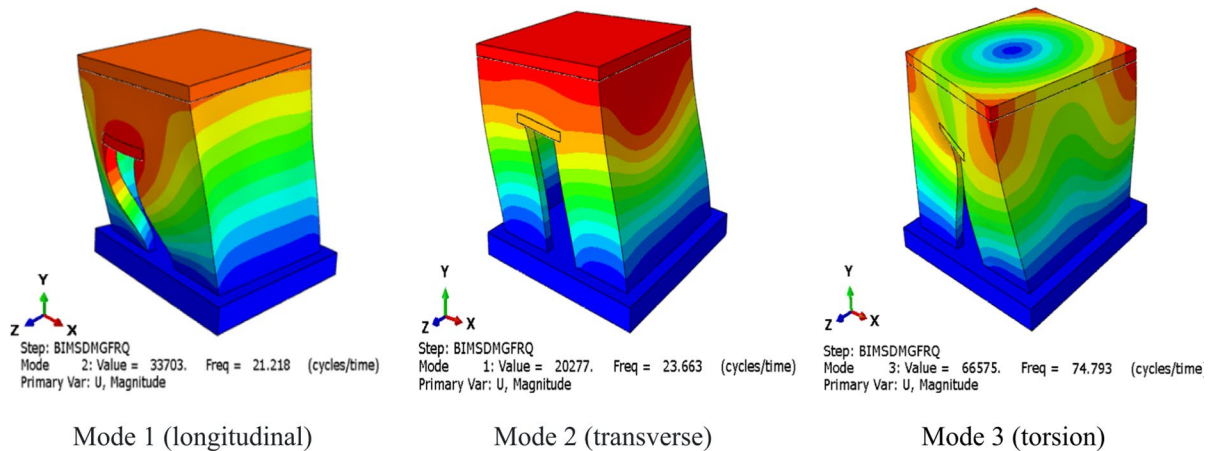


Fig. 23 First three mode shapes obtained from FE analyses for damaged condition (Hz)

Table 9 Experimental and numerical frequencies of the damaged model

Mode	Frequency (Hz)		Maximum difference (%)
	Experimental	Numerical	
1st (longitudinal)	20.55	21.22	3.26
2nd (transverse)	22.79	23.66	3.81
3rd (torsion)	73.29	74.79	2.04

and sine waves with 0.5 g amplitude. Furthermore, numerical analysis results show that the highest first principal tensile values remained below the tensile strength specified for the pumice building (Table 6), indicating that in these cases the masonry walls did not suffer any type of damage. This confirms the strength of the walls under laboratory conditions and the effect of the material used. In order to make the findings of this study easier to understand, a clear comparison of all the options looked at is provided in this section. The no damaged models are presented in Fig. 24 as contours of the principal tensile stress values for the six seismic actions.

The FE analysis results under seismic sinusoidal waves with an amplitude of 0.7 g showed that the maximum first principal stress value reached 0.35 MPa. This value is approximately 1.60 times higher than the tensile strength of the pumice material. This indicates that the tensile stress values would cause significant damage to the pumice model in the event of a severe earthquake. In other words, the structural elements may experience tensile stresses beyond their capacity, potentially leading to damage or failure during strong seismic events. These concentrated stresses align with crack patterns on the four wall faces, as illustrated in the accompanying Fig. 25. The color spectrum highlights regions of high tensile stress in red and areas of low tensile stress in blue.

In contrast, as can be seen from Fig. 26, compressive stress values that occur on the walls are well below the compressive strength of the pumice material (See Table 1). This indicates that the building is safe against compressive stresses.

In the analysis using the CDP model, the tensile and compressive plastic deformation values on the building were calculated (Figs. 27 and 28). The results indicate that certain parts of the north, south,

Table 10 First three natural frequencies of undamaged and damaged model

Mode	Numerical (Hz)		Diff. (%)	Experimental (Hz)		Diff. (%)
	Undamaged	Damaged		Undamaged	Damaged	
1st (longitudinal)	23.39	21.22	9.28	22.53	20.55	8.79
2nd (transverse)	29.52	23.66	19.85	29.80	22.79	23.52
3rd (torsion)	77.04	74.79	2.92	76.41	73.29	4.08

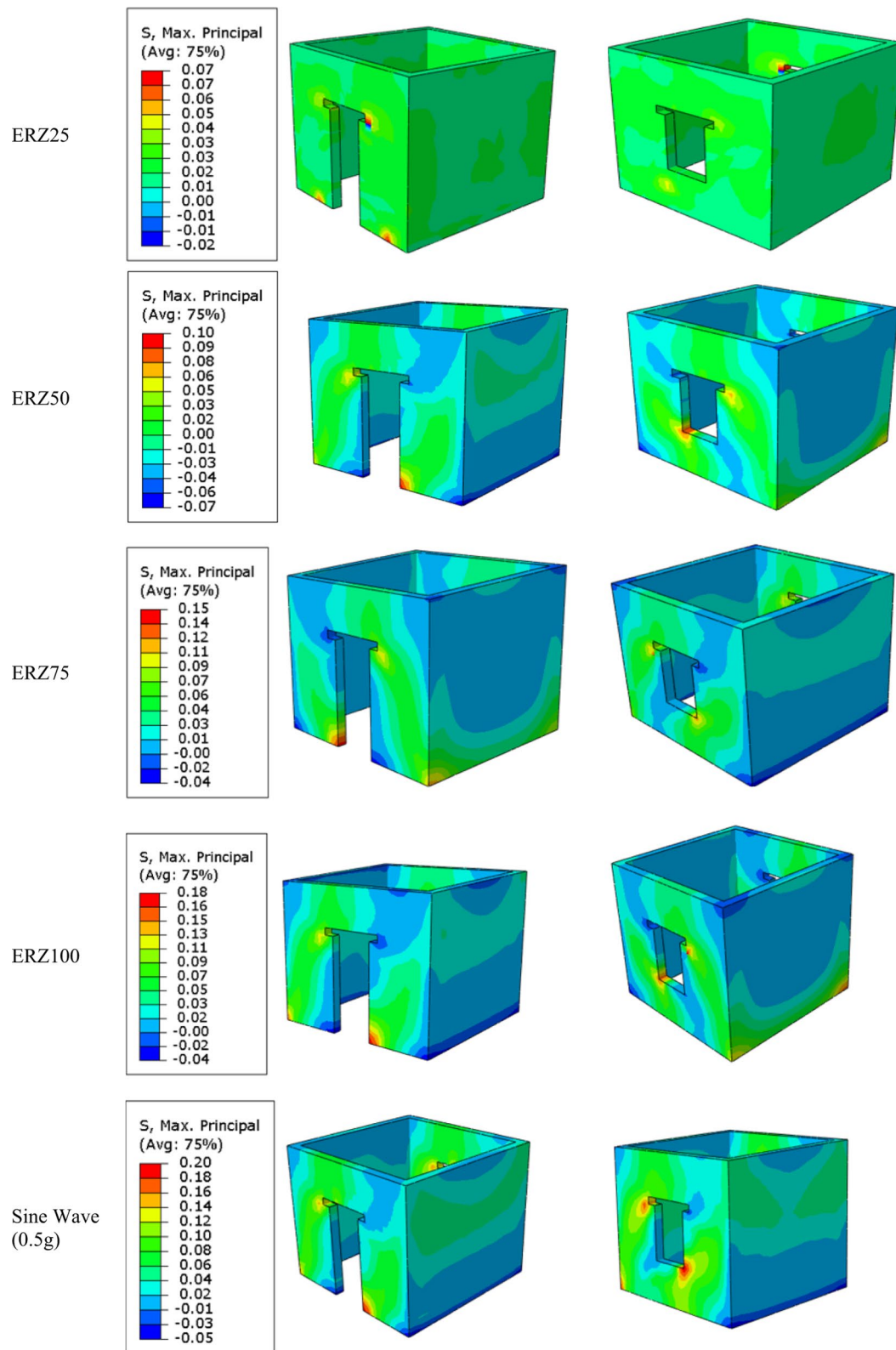


Fig. 24 Propagation of maximum principal stresses in the undamaged FE model (MPa)

Fig. 25 Propagations of maximum principal stresses under sine wave with a $PGA=0.70$ g (MPa)

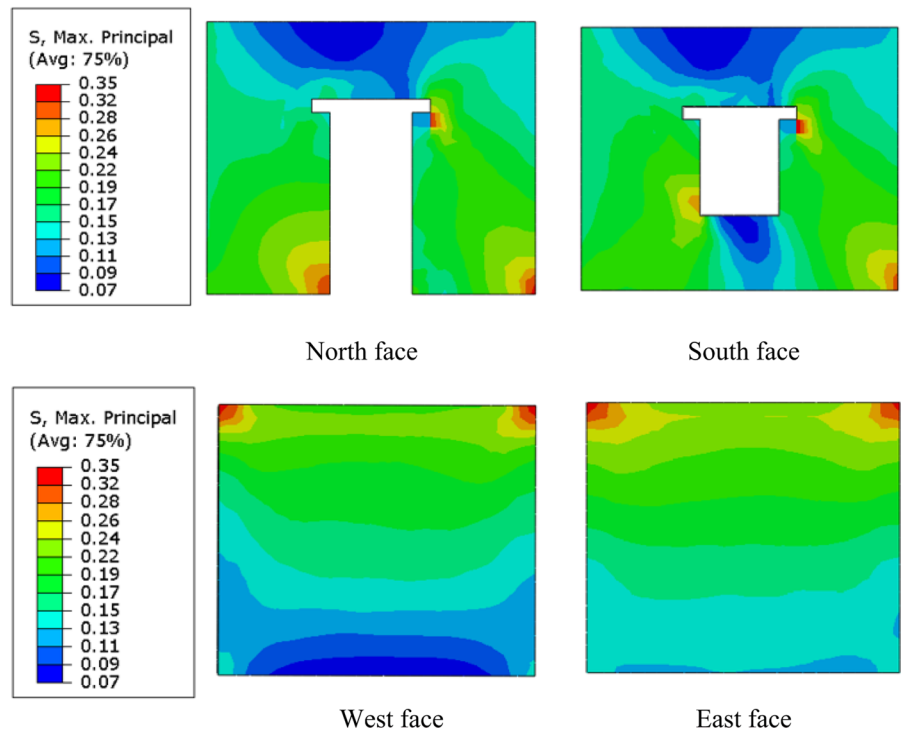


Fig. 26 Propagations of minimum principal stresses under sine wave with a $PGA=0.70$ g (MPa)

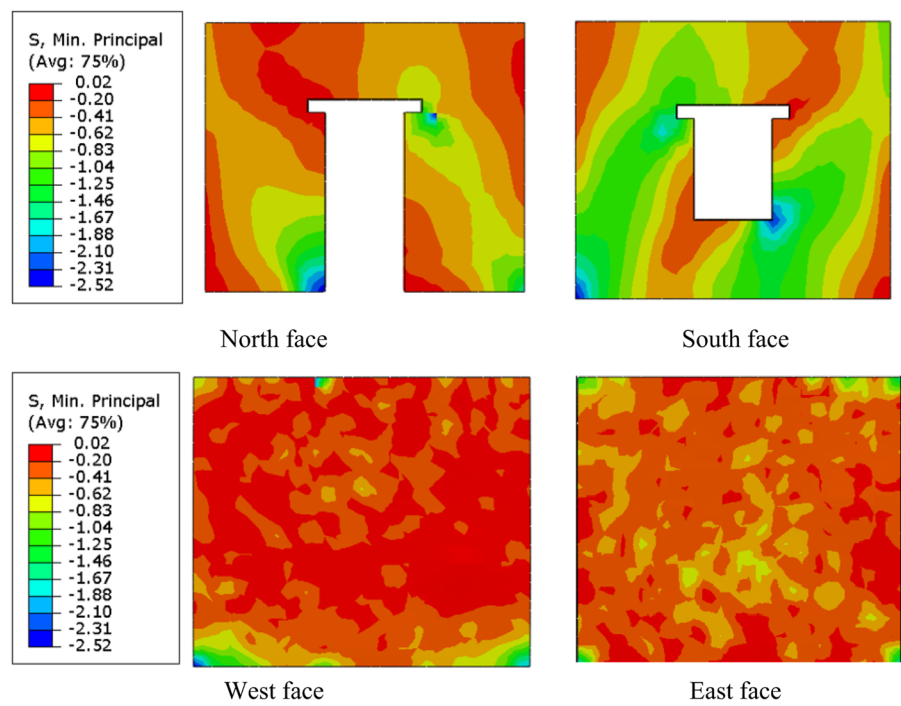


Fig. 27 Tensile plastic strain contour under 5 five-cycle sine wave ($PGA=0.70$ g). (Color figure online)

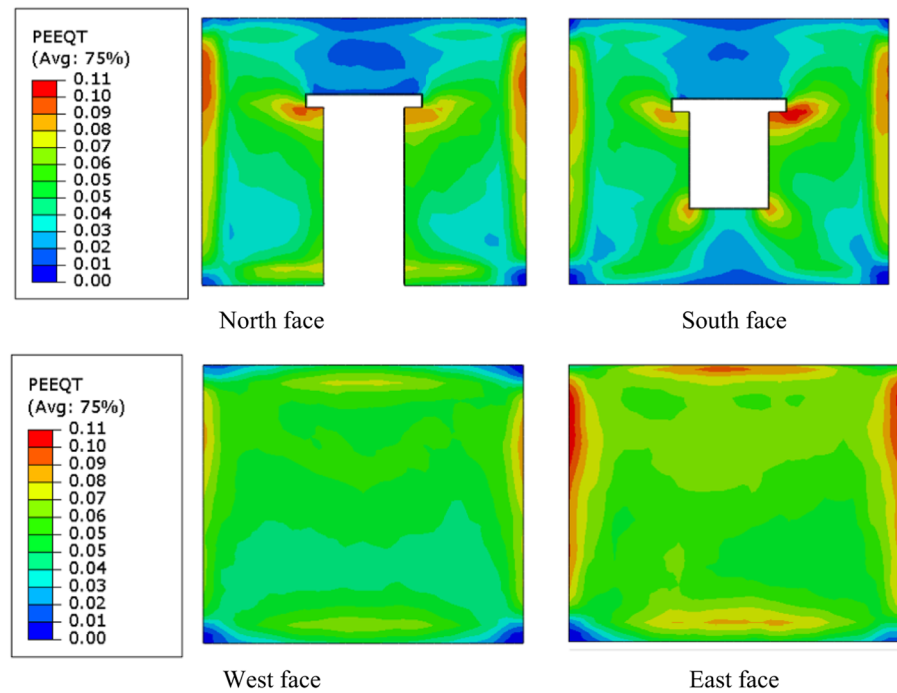
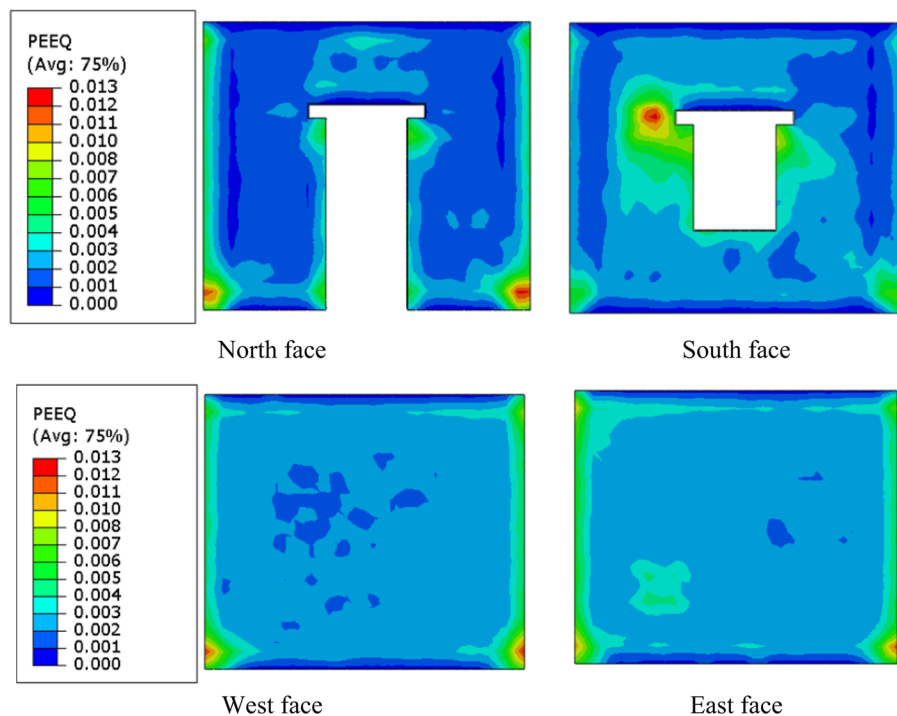


Fig. 28 Compressive plastic strain contour under 5 five-cycle sine wave ($PGA=0.70$ g). (Color figure online)



west, and east walls experienced tensile plastic strain values that reached 0.11 at the end of the analysis. This value exceeded the tension strength damage

strain limit of 0.004, as reported in a previous study [76]. This critical region is expected to be damaged during seismic sinusoidal waves with an amplitude of

0.7 g. Conversely, in some parts of the structure, such as the opposite side of the core, compressive plastic strain values of 0.0013 were observed. These values did not exceed the strength limit of 0.015. As a result, damage is expected to occur in this region (indicated by red locations), but the extent of the damage is limited, and the masonry structure can still be considered safe under these conditions.

Based on the numerical results obtained, the damage distribution in the building due to seismic waves occurred by exceeding the tensile strength of the masonry wall. In the analyses, the wall compressive strength was never exceeded.

The present study successfully validated a FE model designed to predict crack patterns in pumice masonry building. As illustrated in Fig. 29, the damage inclined cracks at the window and door corners, also a horizontal crack appeared along the upstream walls without opening. As can be seen, the crack patterns observed in the experimental study (refer to Fig. 8) matched with the damage predicted by the proposed FE numerical model. However, it is worth noting that minor discrepancies in the damage of certain wall regions were observed.

The recommended allowable roof drift limit for masonry structures, according to TBEC [51], stands

at 0.7% of the height of the building. The maximum displacements at the top right point in both numerical and experimental cases are 58.52 mm and 51.97 mm, respectively (see Fig. 30). The displacement from experimental studies is lower than that predicted by numerical simulations, revealing a slight overestimation of the maximum displacement by the FE model. The height of building is 1.60 m, the calculated roof drifts for the experimental and numerical scenarios are 0.0368 and 0.0325, respectively, corresponding to roof drift ratios of 3.68% and 3.25%. Both calculated roof drift ratios significantly much larger than permissible limit 0.7%, which would mean the possibility of non-structural damage in a conventional pumice building. It indicates that the roof of the building is experiencing larger horizontal movements than what is considered permissible and within design requirements. This might possibly lead to significant damage within the pumice building, and the risk of injury or even collapse might rise during catastrophic events like earthquakes or severe storms. The time histories of the displacements and accelerations of the top left point of the slab obtained by the experimental and numerical analysis were comparatively presented in Fig. 30.

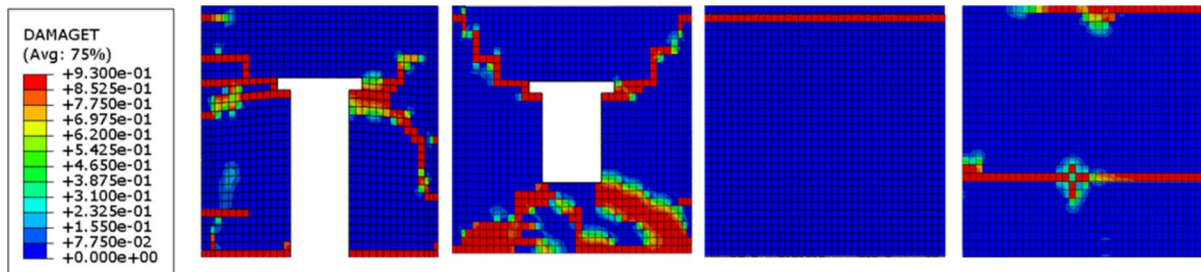
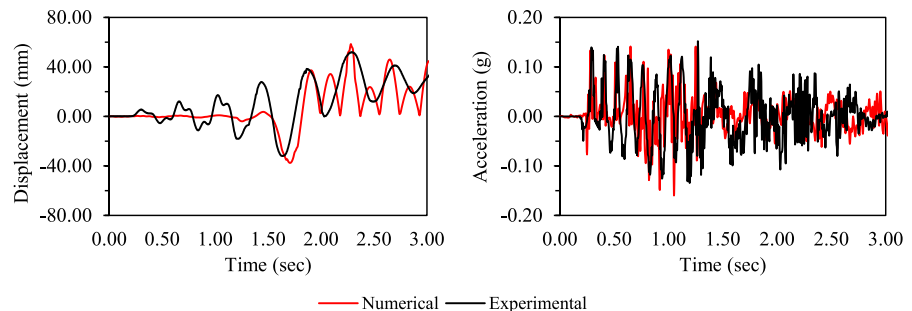


Fig. 29 Propagation of damaged model under Sine waves (0.7 g)

Fig. 30 Comparison between the experimental and numerical displacements and acceleration obtained from top right corner point of the building



Despite these differences, the fact that the numerical simulations often correspond well with the experimental data is acceptable. It shows that the numerical model represents the fundamental behavior of the structure during damage propagations, even when the peak values of displacement and acceleration values are differed. Differences between experimental and numerical data may arise from variations in boundary conditions after each ground movement in the experimental study, as well as from labor errors.

5 Conclusions

Shake table tests are essential for studying the dynamic behavior of masonry structures and conducting structural health monitoring. This study served to validate prior knowledge regarding the dynamic response of masonry buildings. Moreover, it provided valuable insights into how a masonry building responds when subjected to intense seismic excitation in shake table experiments. The tests demonstrated that heightened seismic forces can lead to significant damage and even the collapse of masonry structures. In assessing the seismic performance of a masonry model constructed with pumice concrete under dynamic loading, this study employed six ground motions. These ground motions caused abrupt damage and eventual structural failure. To examine and analyze this damage, the dynamic characteristics of the test model were determined through ambient vibration tests conducted both before and after the experimental tests, covering both undamaged and damaged conditions. It is worth mentioning that the visual and numerical findings across all experiments closely correlate with the outcomes derived from FE analyses conducted with ABAQUS software. Based on the combined findings from both experimental and numerical investigations, the following conclusions can be drawn:

- The frequency values of the pumice model at the end of the shaking table tests showed a decrease of 23.5% for the transverse mode, 8.8% for the longitudinal mode and 4.1% for the torsional mode compared to the undamaged condition (with additional mass).
- The DI values for the pumice concrete model, reaching a DS-5 type damage condition, were cal-

culated as 0.24 for the transverse mode, 0.09 for the longitudinal mode, and 0.04 for the torsional mode.

- The noticeable distinctions in the relative displacements recorded at the corners of the slab were primarily caused by the varying sizes of the openings in the north and south-facing walls. Consequently, this led to torsional effects being observed at the level of the slab during the shaking table tests. It has been emphasized in previous studies that the openings left for architectural purposes in such structures have critical effects on the structural behavior. However, it is obvious that more studies should be carried out to determine the ideal opening ratio.
- The inter-story drift ratios obtained during the experiments are partially consistent with the proposed thresholds. A possible reason for the partial agreement can be attributed to limitations in continuous monitoring of the relationship between engineering parameters (displacement, Inter-story drift ratios, etc.) and damage during shake table tests.
- The experimental findings of the damage sequence and pattern correlated well with the predictions of a FE model using the CDP model. This modeling method has been widely employed to simulate crack propagation under damaged conditions.
- The model verification performed in the study was carried out only for the undamaged case. No verifications were made for damaged cases. The initial verified numerical model and experimental dynamic characteristics of pumice model demonstrated acceptable differences when compared to each other. The FE method analysis for the undamaged model revealed errors of less than 5% in the modal frequency values. For undamaged and damaged model, the first three experimental mode shapes were found in good agreement with numerically mode shapes.
- The decrease in natural frequency values between the undamaged and damaged states is a result of the loss of stiffness in the building caused by damage to certain regions of the pumice walls.
- Both the experimental and the numerical results indicate that the expected damages in the building occurred in the same regions. These results indicate that nonlinear FE models can be helpful in identifying potential damage pattern locations.

- In particular, the developed FE model effectively demonstrated the capability of predicting the behavior of the structure under seismic loading conditions. This type of study is useful in engineering to assure the safety and security of buildings, especially in areas prone to seismic activity.
- The results obtained in this study also shed light on future studies. In fact, seismic excitations, or incremental excitations, which are very difficult to perform experimentally due to laboratory facilities, can be easily performed numerically by using the data in this study.

The results obtained from this study can potentially lead to the development of new approaches or methods for strengthening masonry structures. Based on this research, it can be concluded that the structural behavior of similar structures can be positively improved by the application of innovative strengthening materials in the regions identified by analyzing the structural behavior and damage conditions.

Acknowledgements This work was supported by Scientific Research Projects Coordination Unit of Karadeniz Technical University. Project numbers: FAY-2021-9635 and FAY-2018-7715.

Author contributions A.K.: investigation, writing—original draft B.R.: software, visualization S.A.: supervision, writing—review & editing F.S.: methodology, investigation A.F.G.: methodology, investigation M.G.: methodology, writing—review & editing, Supervision A.C.A.: methodology, writing—review & editing, supervision

Funding Open access funding provided by the Scientific and Technological Research Council of Türkiye (TÜBİTAK).

Declarations

Conflict of interest The authors declare no competing interests.

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References

1. Ganbaatar A, Mori T, Inoue R, Danzandorj S (2022) Strength performance of the connection between brick and SPF lumber. *Buildings* 12(4):465. <https://doi.org/10.3390/buildings12040465>
2. Kišiček T, Štepinac M, Renić T, Hafner I, Lulić L (2020) Strengthening of masonry walls with FRP or TRM. *J Croat Assoc Civil Eng* 72:937–953. <https://doi.org/10.14256/JCE.2983.2020>
3. Zhang Y, Lin X, Wang T, Skalomenos K (2021) Seismic behavior of masonry walls strengthened by precast reinforced concrete panels with different connection details. *Eng Struct* 242:112597. <https://doi.org/10.1016/j.engstruct.2021.112597>
4. Yao X, Guo ZX, Basha SH, Huang Q (2021) Innovative seismic strengthening of historic masonry walls using polymer mortar and steel strips. *Eng Struct* 228:111507. <https://doi.org/10.1016/j.engstruct.2020.111507>
5. Garcia-Ramonda L, Pelà L, Roca P, Camata G (2022) Experimental cyclic behaviour of shear masonry walls reinforced with single and double layered steel reinforced grout. *Constr Build Mater* 320:126053. <https://doi.org/10.1016/j.conbuildmat.2021.126053>
6. Ponte M, Penna A, Bento R (2023) In-plane cyclic tests of strengthened rubble stone masonry. *Mater Struct* 56:41. <https://doi.org/10.1617/s11527-023-02126-8>
7. Kabaş HT, Kusain FE, Anıl Ö (2023) Experimental behavior of masonry infilled RC frames with openings strengthened by using CFRP strip. *Compos Struct* 312:116873. <https://doi.org/10.1016/j.compstruct.2023.116873>
8. Estevan L, Torres B, Baeza FJ, Varona FB, Ivorra S (2023) Masonry walls strengthened with Textile Reinforced Mortars (TRM) and subjected to in-plane cyclic loads after real fire exposure. *Eng Struct* 296:116922. <https://doi.org/10.1016/j.engstruct.2023.116922>
9. Karthika RB, Vidyapriya V, Nandhini Sri KV, Merlin Grace Beaula K, Harini R, Sriram M (2021) Experimental study on lightweight concrete using pumice aggregate. *Mater Today Proc* 43(2):1606–1613
10. Tran Q, Ghosh P (2020) Influence of pumice on mechanical properties and durability of high-performance concrete. *Constr Build Mater* 249:118741. <https://doi.org/10.1016/j.conbuildmat.2020.118741>
11. Rashad AM (2019) A short manual on natural pumice as a lightweight aggregate. *J Build Eng* 25:100802. <https://doi.org/10.1016/j.jobe.2019.100802>
12. Rashad AM (2021) An overview of pumice stone as a cementitious material—the best manual for civil engineer. *SILICON* 13(2):551–572. <https://doi.org/10.1007/s12633-020-00469-3>
13. Alqarni AS (2022) A comprehensive review on properties of sustainable concrete using volcanic pumice powder ash as a supplementary cementitious material. *Constr Build*

- Mater 323:126533. <https://doi.org/10.1016/j.conbuildmat.2022.126533>
14. Pavlík Z, Vyšvařil M, Pavlíková M, Bayer P, Pivák A, Rovnaníková P, Záleská M (2023) Lightweight pumice mortars for repair of historic buildings—assessment of physical parameters, engineering properties and durability. *Constr Build Mater* 404:133275. <https://doi.org/10.1016/j.conbuildmat.2023.133275>
 15. Tomaževič M (1978) Computation of the shear resistance of masonry buildings. Technical report. Institute for Testing and Research in Materials and Structures-ZRMK: Ljubljana, Slovenia
 16. Tomaževič M, Turnšek V (1980) Lateral load distribution as a basis for the seismic resistance analysis of masonry buildings. In: *Proceedings of the international research conference on earthquake engineering*, Skopje, Yugoslavia, pp 455–488
 17. Chacara C, Reategui R, Ore A, Suarez P, Aguilar R (2023) Integration of NDT, 3D parametric modelling, and non-linear numerical analysis for the seismic assessment of a vaulted stone-masonry historical building. *J Build Eng* 70:106347. <https://doi.org/10.1016/j.jobte.2023.106347>
 18. Aldemir A, Altuğ Erberik M, Demirel IO, Sucuoğlu H (2013) Seismic performance assessment of unreinforced masonry buildings with a hybrid modeling approach. *Earthq Spectra* 29(1):33–57. <https://doi.org/10.1193/1.4000102>
 19. Kocaman I, Gedik Y, Okuyucu D (2024) Assessment of seismic behavior of historical masonry cupolas: case of Emir Saltuk Cupola. *J Build Eng* 82:108275. <https://doi.org/10.1016/j.jobte.2023.108275>
 20. Federal Emergency Management Agency (FEMA) (2005) FEMA 440 improvement of nonlinear static seismic analysis procedures. Federal Emergency Management Agency (FEMA), Washington, DC, USA
 21. CEN (European Committee for Standardization) (2005) Eurocode 8: design of structures for earthquake resistance—part 3: assessment and retrofitting of buildings. CEN (European Committee for Standardization), Brussels, Belgium
 22. Consiglio Nazionale delle Ricerche (CNR) (2013) CNR-DT212 recommendations for the probabilistic seismic assessment of existing buildings. Consiglio Nazionale delle Ricerche (CNR), Rome, Italy (In Italian)
 23. ASCE/SEI 41-13 (2014) Seismic evaluation and retrofit of existing buildings. American Society of Civil Engineers, Reston, VA, USA
 24. Lagomarsino S, Cattari S (2015) Seismic performance of historical masonry structures through pushover and non-linear dynamic analyses. In: Ansal A (ed) *Perspectives on european earthquake engineering and seismology*. Geotechnical, geological and earthquake engineering, vol 39. Springer, Cham. https://doi.org/10.1007/978-3-319-16964-4_11
 25. Baraldi D, Reccia E, Cecchi A (2018) In plane loaded masonry walls: DEM and FEM/DEM models. *Crit Rev Mecc* 53:1613–1628. <https://doi.org/10.1007/s11012-017-0704-3>
 26. Gubana A, Melotto M (2021) Evaluation of timber floor in-plane retrofitting interventions on the seismic response of masonry structures by DEM analysis: a case study. *Bull Earthq Eng* 19:6003–6026. <https://doi.org/10.1007/s10518-021-01190-1>
 27. Panto B, Cannizzaro F, Caddemi S, Calì I (2016) 3D macro-element modelling approach for seismic assessment of historical masonry churches. *Adv Eng Softw* 97:40–59. <https://doi.org/10.1016/j.advengsoft.2016.02.009>
 28. D'Ayala DF, Paganoni S (2011) Assessment and analysis of damage in L'Aquila historic city centre after 6th April 2009. *Bull Earthq Eng* 9:81–104. <https://doi.org/10.1007/s10518-010-9224-4>
 29. Sorrentino L, Cattari S, da Porto F, Magenes G, Penna A (2019) Seismic behaviour of ordinary masonry buildings during the 2016 central Italy earthquakes. *Bull Earthq Eng* 17:5583–5607. <https://doi.org/10.1007/s10518-018-0370-4>
 30. Meoni A, D'Alessandro A, Mattiacci M, García-Macías E, Saviano F, Parisi F, Lignola GP, Ubertini F (2024) Structural performance assessment of full-scale masonry wall systems using operational modal analysis: laboratory testing and numerical simulations. *Eng Struct* 304:117663. <https://doi.org/10.1016/j.engstruct.2024.117663>
 31. Franck SA, Bretschneider N, Volker S (2020) Safety analysis of existing masonry arch bridges by nonlinear finite element simulations. *Int J Damage Mech* 29(1):126–143. <https://doi.org/10.1177/1056789519865995>
 32. Mughal UA, Qazi AU, Ahmed A, Ajmal MM, Danish M (2023) Investigation of torsion in confined masonry structures originating due to unsymmetric openings. *Eur J Environ Civil Eng* 1–20. <https://doi.org/10.1080/19648189.2023.2293817>
 33. Kaya A, Adanur S, Bello RA, Genç AF, Okur FY, Sunca F, Altunisik AC, Sevim B (2023) Post-earthquake damage assessments of unreinforced masonry (URM) buildings by shake table test and numerical visualization. *Eng Fail Anal* 143:106858. <https://doi.org/10.1016/j.engfailanal.2022.106858>
 34. Tanrıverdi S (2023) An experimental and numerical study of the strengthening of masonry brick vaults. *Structures* 47:800–813. <https://doi.org/10.1016/j.istruc.2022.11.098>
 35. Altunisik AC, Genç A, Günaydin M, Adanur S, Okur FY (2018) Ambient vibration-based system identification of a medieval masonry bastion for health assessment using nonlinear analyses. *Int J Nonlinear Sci Numer Simul* 19(2):107–124. <https://doi.org/10.1515/ijnsns-2017-0004>
 36. Günaydin M (2020) In situ dynamic investigation on the historic “İskenderpaşa” masonry mosque with non-destructive testing. *Smart Struct Syst Int J* 26(1):1–10. <https://doi.org/10.12989/sss.2020.26.1.001>
 37. OMA (2006) Operational modal analysis, Release 4.0. Structural Vibration Solution A/S, Denmark
 38. Pepi C, Cavalagli N, Gusella V, Gioffre M (2021) Damage detection via modal analysis of masonry structures using shaking table tests. *Earthq Eng Struct Dyn* 50:2077–2097. <https://doi.org/10.1002/eqe.3431>
 39. Okur EK, Altunisik AC, Okur FY, Meral M, Yılmaz Z, Günaydin M, Genç AF (2021) Dynamic response of a traditional himis, mansion using updated FE model with operational modal testing. *J Build Eng* 43:103060. <https://doi.org/10.1016/j.jobte.2021.103060>

40. Altunisik AC, Genç AF, Ertürk E, Günaydin M, Okur FY, Sevim B (2022) Soil–structure interaction and earthquake input models effect on the structural response of the Santa Maria church and Guesthouse building. *J Earthq Eng* 27(14):4094–4125. <https://doi.org/10.1080/13632469.2022.2158967>
41. Gunaydin M, Sunca F, Altunisik AC, Ergun M, Okur FY (2022) Nondestructive experimental measurement, model updating, and fatigue life assessment of Çarşamba suspension bridge. *J Bridge Eng* 27(2):05021017. [https://doi.org/10.1061/\(ASCE\)BE.1943-5592.0001818](https://doi.org/10.1061/(ASCE)BE.1943-5592.0001818)
42. Gunaydin M, Demirkir C, Altunisik AC, Gezer ED, Genc AF, Okur FY (2023) Diagnosis and monitoring of historical timber velipasa han building prior to restoration. *Int J Archit Herit* 17(2):285–309. <https://doi.org/10.1080/15583058.2021.1919239>
43. Ayengin S, Bagbanci MB, Bagbanci OK (2024) Evaluation of material characteristics and structural dynamic properties of a historical church: the case of 19th century Bursa Derekoy Church (Ayia Paraskevi). *Period Polytech Civil Eng* 68(3):797–811. <https://doi.org/10.3311/PPci.22520>
44. Civera M, Mugnaini V, Fragonara LZ (2022) Machine learning-based automatic operational modal analysis: a structural health monitoring application to masonry arch bridges. *Struct Control Health Monit* 29(10):e3028. <https://doi.org/10.1002/stc.3028>
45. Sun Q, Rainieri C, Rem WX, Yan WJ, Fabbrocino G (2023) Automated operational modal analysis of bell towers subjected to narrowband input. *Structures* 54:78–88. <https://doi.org/10.1016/j.istruc.2023.05.034>
46. Bayraktar A, Altunisik AC, Sevim B, Türker T (2011) Seismic response of a historical masonry minaret using a finite element model updated with operational modal testing. *J Vib Control* 17(1):129–149. <https://doi.org/10.1177/1077546309353288>
47. Xie Q, Xu D, Zhang Y, Yu Y, Hao W (2020) Shaking table testing and numerical simulation of the seismic response of a typical China ancient masonry tower. *Bull Earthq Eng* 18:331–355. <https://doi.org/10.1007/s10518-019-00731-z>
48. Panto B, Chisari C, Macorini L, Izzuddin BA (2022) A hybrid macro-modelling strategy with multi-objective calibration for accurate simulation of multi-ring masonry arches and bridges. *Comput Struct* 265:106769. <https://doi.org/10.1016/j.compstruc.2022.106769>
49. Cosgun SI, Kaya A, Gencturk B (2023) Computational modeling of the axial behavior of corroded and buckled short steel piles strengthened using concrete-filled GFRP jackets. *Eng Fail Anal* 147:107147. <https://doi.org/10.1016/j.engfailanal.2023.107147>
50. Addessi D, Sacco E (2018) Homogenization of heterogeneous masonry beams. *Meccanica* 53:1699–1717. <https://doi.org/10.1007/s11012-017-0758-2>
51. TBEC-2018 (Turkish Building Earthquake Code), Specifications for buildings to be built in seismic areas. Ministry of Public Works and Settlement, Ankara, Turkey
52. Saleem MU, Numada M, Amin MN, Meguro K (2016) Shake table tests on FRP retrofitted masonry building models. *J Compos Constr* 20(5):04016031. [https://doi.org/10.1061/\(ASCE\)CC.1943-5614.0000684](https://doi.org/10.1061/(ASCE)CC.1943-5614.0000684)
53. Das A, Deb SK, Dutta A (2016) Shake table testing of un-reinforced brick masonry building test model isolated by U-FREI. *Earthq Eng Struct Dynam* 45(2):253–272. <https://doi.org/10.1002/eqe.2626>
54. Harris H, Sabnis GM (1999) Structural modelling and experimental techniques, 2nd edn. CRC Press LLC, Boca Raton Florida, p 808. <https://doi.org/10.1201/9780367802295>
55. Zhang H, Kuang JS, Yuen TY (2017) Low- seismic damage strategies for infilled RC frames: shake- table tests. *Earthq Eng Struct Dynam* 46(14):2419–2438. <https://doi.org/10.1002/eqe.2911>
56. Carvalho EC (1998) Seismic testing of structures. In: Proceedings of the 11th European conference on earthquake engineering, Paris, France
57. Quintana Gallo P, Pampanin S, Carr AJ, Bonelli P (2010) Shake table tests of under-designed frames for the seismic retrofit of buildings-design and similitude requirements of the benchmark specimen. In: Proceedings of the NZSEE Conference, Wellington, Paper 39
58. Ramos LF (2007) Damage identification on masonry structures based on vibration signatures. Dissertation, University of Minho
59. Tomassetti U, Correia AA, Candeias PX, Graziotti F, Campos Costa A (2019) Two-way bending out-of-plane collapse of a full-scale URM building tested on a shake table. *Bull Earthq Eng* 17(4):2165–2198. <https://doi.org/10.1007/s10518-018-0507-5>
60. PEER (2022) Pacific earthquake engineering research center ground motion database. University of California, Berkeley
61. Khan FZ, Ahmad ME, Ahmad N (2021) Shake table testing of confined adobe masonry structures. *Earthq Struct* 20(2):149–160. <https://doi.org/10.12989/eas.2021.20.2.149>
62. PULSE (2006) Analyzers and solutions, Release 11.2. Bruel and Kjaer, Sound and vibration measurement A/S, Denmark
63. Grünthal G (1998) European macroseismic scale 1998 (EMS-98). European Seismological Commission (ESC)
64. Como M, Di Carlo F, Coccia S (2019) Dynamic response of rocking cracked masonry walls. *Meccanica* 54:381–398. <https://doi.org/10.1007/s11012-019-00949-w>
65. Onat O (2019) Fundamental vibration frequency prediction of historical masonry bridges. *Struct Eng Mech* 69(2):155–162. <https://doi.org/10.12989/sem.2019.69.2.155>
66. Mendes N, Lourenço PB, Campos-Costa A (2014) Shaking table testing of an existing masonry building: assessment and improvement of the seismic performance. *Earthq Eng Struct Dynam* 43(2):247–266. <https://doi.org/10.1002/eqe.2342>
67. Kallioras S, Correia AA, Graziotti F, Penna A, Magenes G (2020) Collapse shake-table testing of a clay-URM building with chimneys. *Bull Earthq Eng* 18(3):1009–1048. <https://doi.org/10.1007/s10518-019-00730-0>
68. Miglietta M, Damiani N, Guerrini G, Graziotti F (2021) Full-scale shake-table tests on two unreinforced masonry cavity-wall buildings: effect of an innovative timber retrofit. *Bull Earthq Eng* 19(6):2561–2596. <https://doi.org/10.1007/s10518-021-01057-5>

69. Calvi GM (1999) A displacement-based approach for vulnerability evaluation of classes of buildings. *J Earthq Eng* 3(03):411–438. <https://doi.org/10.1080/13632469909350353>
70. Penna A, Mandirola M, Rota M, Magenes G (2015) Experimental assessment of the in-plane lateral capacity of autoclaved aerated concrete (AAC) masonry walls with flat-truss bed-joint reinforcement. *Constr Build Mater* 82:155–166. <https://doi.org/10.1016/j.conbuildmat.2015.02.057>
71. Arslan ME, Celebi E (2019) An experimental study on cyclic behavior of aerated concrete block masonry walls retrofitted with different methods. *Constr Build Mater* 200:226–239. <https://doi.org/10.1016/j.conbuildmat.2018.12.132>
72. Kayirga OM, Altun F (2021) Investigation of earthquake behavior of unreinforced masonry buildings having different opening sizes: experimental studies and numerical simulation. *J Build Eng* 40:102666. <https://doi.org/10.1016/j.jobe.2021.102666>
73. Bothara JK, Dhakal RP, Mander JB (2010) Seismic performance of an unreinforced masonry building: an experimental investigation. *Earthq Eng Struct Dyn* 39(1):45–68. <https://doi.org/10.1002/eqe.932>
74. ABAQUS (2017) Dassault Systemes Simulia Corporation, Providence, Rhode Island
75. Resta M, Fiore A, Monaco P (2013) Non-linear finite element analysis of masonry towers by adopting the damage plasticity constitutive model. *Adv Struct Eng* 16(5):791–803. <https://doi.org/10.1260/1369-4332.16.5.791>
76. Abdelmoneim E, Mohamad AB, Chen Z (2016) Experimental and numerical analysis of the compressive and shear behavior for a new type of self-insulating concrete masonry system. *Appl Sci* 6(9):245. <https://doi.org/10.3390/app6090245>
77. Cosgun SI (2014) Numerical investigation of reinforced concrete plates behavior under impact load. Master thesis, Karadeniz Technical University, Trabzon
78. Zhao G, Li A (2008) Numerical study of a bonded steel and concrete composite beam. *Comput Struct* 86(19–20):1830–1838. <https://doi.org/10.1016/j.compstruc.2008.04.002>

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