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Seismic Response of Reinforced Concrete Structure with Infill Walls for Vertical Forest Applications - Seismic Table Tests

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ABSTRACT

This paper presents the dynamic experimental results of a three-dimensional specimen tested on the seismic table of the Laboratory of Reinforced Concrete and Seismic Design of Structures of DUTH (Democritus University of Thrace). The specimen was constructed as part of the GREENERGY project with the aim of reintroducing vertical forests into existing approaches for repair and strengthening of reinforced concrete buildings. The one-storey test specimen consists of four columns supported on strong foundation beams anchored to the seismic table. The columns are connected at the upper level with hidden beams into a solid slab. The slab protrudes from all four sides at the upper horizontal level, as do the columns along the vertical direction. Furthermore, at the upper level, the beam-column joint has two special joint configurations: a typical one and one without a slab area and beam reinforcement anchoring into the joint region. The seismic excitation is applied in one direction of the structure where there is a partial infill wall eccentrically placed, while in the perpendicular loading direction there are full infill walls functioning out-of-plane. Common techniques used in existing buildings were followed for the construction of the clay-brick infill walls. The loading follows a gradual increase in the peak intensity of the seismic excitation, based on the 1978 Thessaloniki earthquake, at levels from 0.1g to 1.1g acceleration. In this phase, the results from accelerometer measurements, draw-wire displacement sensors and piezoelectric sensors (PZTs) with their initial post-processing are presented. Additionally, the damage to columns and infill walls is investigated at different serviceability limit states. Next, the structure will receive suitable renovations including planters and substitutes of living walls to investigate vertical forests.

Key Words: Shake table testing, RC structures, Infills, Vertical Forest

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Σεισμική Απόκριση Κατασκευής Οπλισμένου Σκυροδέματος με Τοιχοποιίες Πλήρωσης για Εφαρμογές Κάθετου Δάσους - Δοκιμές Σεισμικής Τράπεζας

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ΠΕΡΙΛΗΨΗ

Στην παρούσα εργασία γίνεται η παρουσίαση των δυναμικών πειραματικών αποτελεσμάτων του τρισδιάστατου δοκιμίου στην σεισμική τράπεζα στο Εργαστήριο Οπλισμένου Σκυροδέματος και Αντισεισμικών Κατασκευών του ΔΠΘ. Το δοκίμιο κατασκευάστηκε στα πλαίσια του έργου GREENERGY με σκοπό την επανατοποθέτηση των κάθετων δασών στις υπάρχουσες προσεγγίσεις επισκευών και ενισχύσεων κτιρίων οπλισμένου σκυροδέματος. Το εξεταζόμενο μονώροφο δοκίμιο αποτελείται από τέσσερα υποστυλώματα τα οποία στηρίζονται σε ισχυρά δοκάρια αγκυρωμένα στην σεισμική τράπεζα. Τα υποστυλώματα συνδέονται στην άνω στάθμη με κρυφοδοκούς και συμπαγή πλάκα. Η πλάκα προεξέχει και από τις τέσσερις πλευρές στην άνω οριζόντια στάθμη όπως και τα υποστυλώματα στην καθ' ύψος διεύθυνση. Ακόμη, στην άνω στάθμη ο κόμβος δοκών-υποστυλωμάτων έχει δύο ειδικές διαμορφώσεις κόμβων: μια τυπική και μια χωρίς περιοχή πλάκας και αγκύρωση οπλισμών δοκών στην περιοχή του κόμβου. Η σεισμική διέγερση εφαρμόζεται σε μια διεύθυνση της σεισμικής τράπεζας όπου υπάρχει μερική τοιχοπλήρωση έκκεντρα τοποθετημένη, ενώ στην κάθετη διεύθυνση φόρτισης υπάρχουν πλήρεις τοιχοπληρώσεις που λειτουργούν εκτός επιπέδου. Για την κατασκευή των τοιχοπληρώσεων ακολουθήθηκε κοινή τεχνική των υφιστάμενων κτιρίων. Η φόρτιση ακολουθεί σταδιακή αύξηση της έντασης του σεισμού Θεσσαλονίκης του 1978 σε επίπεδο από 0.1g έως και 1.1g. Σε αυτή την φάση παρουσιάζονται τα αποτελέσματα από τις μετρήσεις των επιταχυνσιογραφημάτων και αισθητήρων μετατόπισης καλωδίου έλξης με την αρχική μεταεπεξεργασία τους. Επιπρόσθετα παρουσιάζονται οι βλάβες στα υποστυλώματα και τοιχοπληρώσεις. Στη συνέχεια, η κατασκευή θα λάβει κατάλληλες ενισχύσεις συμπεριλαμβανομένων φυτοδοχείων και υποκατάστατων ζωντανών τοίχων για τη διερεύνηση των κάθετων δασών.

Λέξεις-Κλειδιά: Δοκιμές σεισμικής τραπέζης, Κατασκευές Ο/Σ, Τοιχοποιίες πλήρωσης, Κάθετο Δάσος

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1 INTRODUCTION

Seismic table experiments provide a controlled platform for investigating structural behavior under earthquake loading [1]. These laboratory studies enable precise characterization of damage mechanisms, from initial cracking through progressive deterioration, while simultaneously validating monitoring technologies [2] and developing retrofitting strategies [3,4]. The controlled environment allows researchers to systematically examine the coupling between global structural response and local material degradation under known ground motion conditions [5].

Reinforced concrete buildings with masonry infill walls represent a significant portion of existing building stock in Mediterranean seismic-prone regions, particularly structures constructed between the 1960s and 1980s with limited seismic provisions [6]. Experimental investigation of these systems requires replication of construction details, material properties, and loading conditions to generate relevant data for engineering applications. The interaction mechanisms between RC frames and masonry infills under seismic loading require systematic experimental investigation where damage states can be monitored throughout the loading process [7].

Structural health monitoring systems require experimental validation to establish correlations between sensor responses and damage states. Although Piezoelectric transducers (PZTs)-based SHM methods have been thoroughly investigated for RC structures under static loading [8], their effectiveness for masonry structures and under dynamic loading is still largely unexplored.

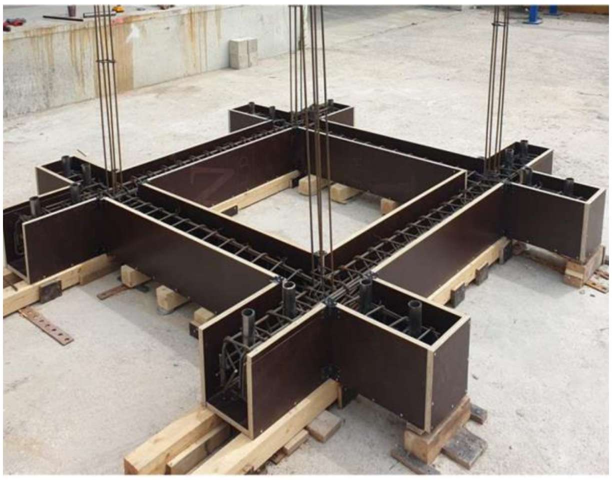
This experimental study (part of GREENERGY project Phase A) investigates the dynamic behavior of a non-seismically designed RC building with brick infills using advanced monitoring techniques on a seismic table. In this Phase A, the results from accelerometer measurements, draw-wire displacement sensors and piezoelectric sensors (PZTs) with their initial post-processing are presented. Additionally, the damage to columns and infill walls is investigated at different serviceability limit states. These tests on the reference building provide the essential baseline for future implementation of Vertical Forest renovation technology [9,10].

2 METHODOLOGY

2.1 Construction and Geometric Characteristics

A specially constructed single-bay, single-story RC frame that is typical of pre-Eurocode Greek construction methods frequently seen in seismically active Mediterranean regions serves as the focal point of the experimental investigation. In order to allow for a realistic assessment of seismic vulnerabilities and subsequent retrofit strategies for vertical forest integration, the test specimen purposefully includes characteristic deficiencies of the aging building stock (inadequate transverse reinforcement in columns, absence of capacity-based design principles, and use of low-strength masonry infills).

In order to meet the operational requirements of the Democritus University of Thrace seismic table (8-ton load capacity, 8-ton-meter overturning moment limit), the specimen plan configuration's key dimensions were 2.70 m × 2.70 m and its overall height was 1.95 m. While preserving testing viability within laboratory limitations, this scaling guarantees representative dynamic response. The construction process of the specimen follows vertical addition as you can see in Figure 1. The construction began with casting the foundation beams using formwork, installing reinforcement bars and connection pipes for seismic table attachment. Next, the four columns were built by placing longitudinal and transverse reinforcement (with embedded strain gauges), setting up formwork, and casting concrete. The slab construction followed, involving scaffolding setup, reinforcement layout with hidden beam zones, PZT sensor positioning, and concrete casting with proper curing. Simultaneously, brick infill walls were constructed using hollow clay units between the RC frames. Finally, shorter columns were built above the slab, and the comprehensive instrumentation system was installed, including accelerometers, draw wire sensors, and the embedded PZT monitoring system. The entire process used on-site concrete mixing with specified material proportions and careful quality control through cylinder sampling.



a) Formwork of the foundation

b) Striping of the foundation and columns

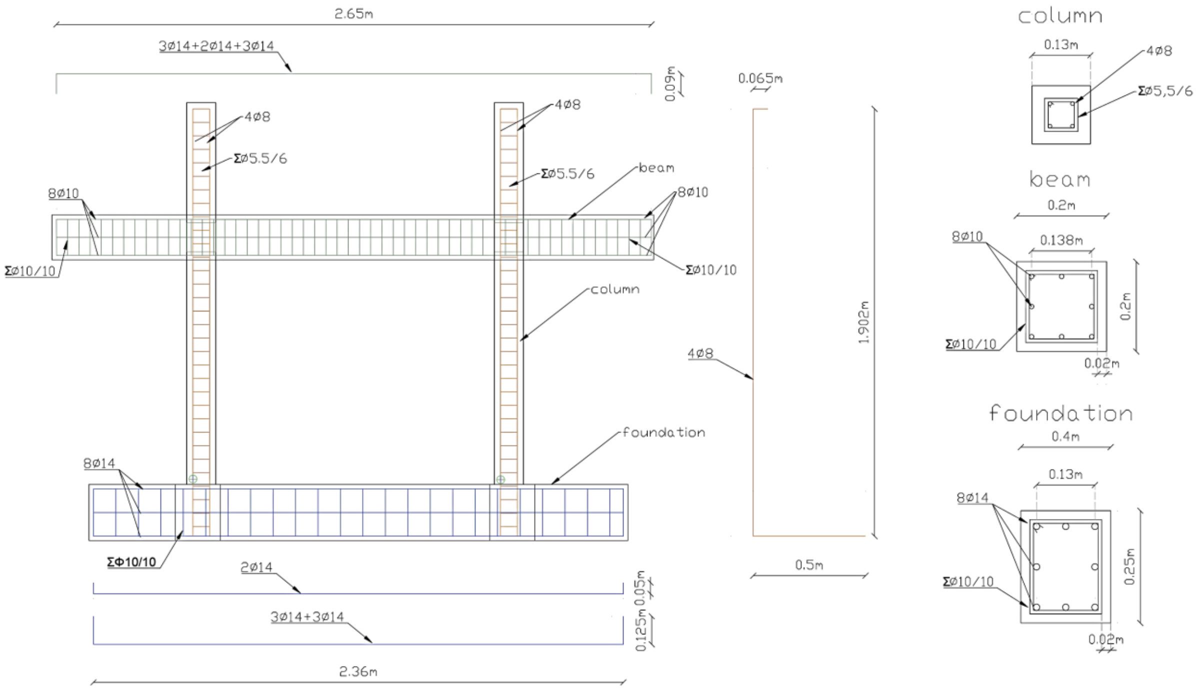
c) Formwork of the slab

d) Final specimen

Figure 1: Different construction stages of the specimen a) formwork of the foundation, b) striping of the foundation and columns, c) formwork of the slab and d) final specimen.

2.2 Structural Reinforcement Detailing and Material Properties

The supporting foundation comprises four rectangular cross-section RC beams that are 250 mm wide and 200 mm high. The longitudinal rebars are placed in three levels, all consisting of rebars with a diameter of 14 mm and Ø10/100 mm transverse reinforcement. At strategically positioned locations, connection vertical tubes were introduced to allow proper and safe attachment of the specimen to the seismic table. A total of four square columns, each measuring 130 mm by 130 mm, rise from the base level to a height of 1.0 m below the main slab and then 0.5 m above it. The main 4Ø8 mm rebars were placed at the cross-section corners, and widely spaced Ø5.5/60 mm stirrups were used for transverse reinforcement. This intentionally weak cross-section detailing demonstrates the problems that already-weak structures have, which allows accurate measurement of their seismic response. The slab has a thickness of 200 mm and is located 1.57 m above the foundation. It serves multiple purposes: it conceals the beam members along grid lines and provides the structure with the appropriate mass for accurate inertial loading in seismic simulation while it allows for future attachment of high-density forestation. The beams contain 8Ø10 mm longitudinal rebars (arranged as 3 top, 2 middle, and 3 bottom) and square stirrups of Ø8/100 mm. The slab extensions extending beyond the perimeter frame constitute a critical design element. These extensions are specifically designed to facilitate the implementation of vertical forest anchoring systems both within and external to the structural envelope during subsequent construction phases. The reinforcement details are depicted in Figure 2.



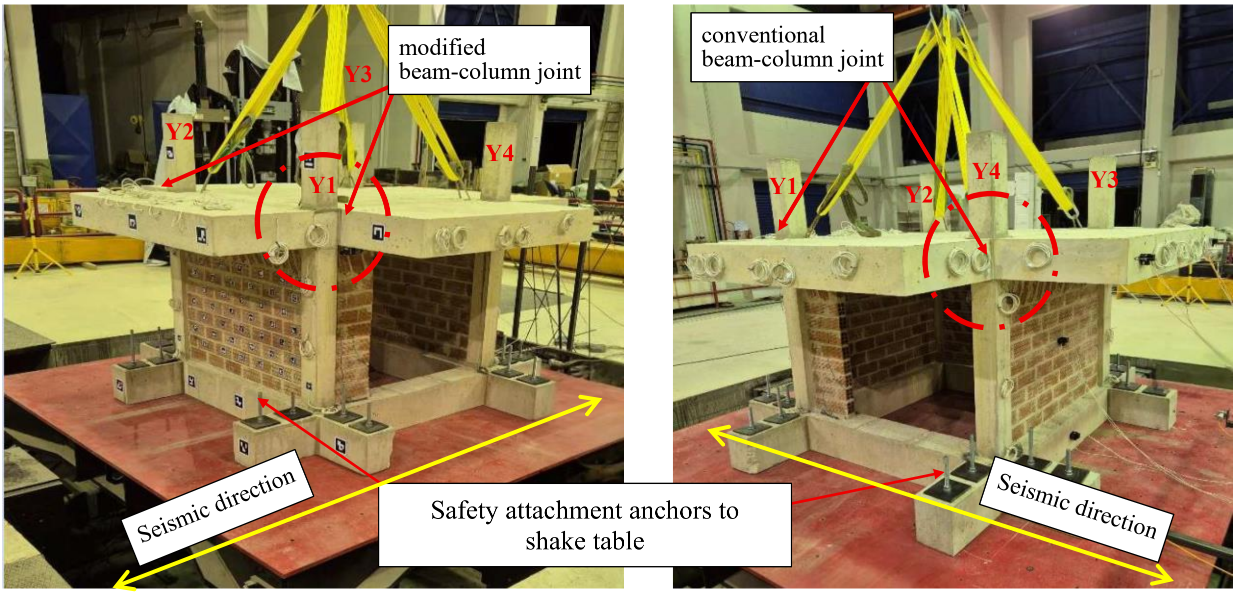
a) b)

Figure 2: Reinforcement of the specimen a) section view b) cross-section details.

The test structure utilized C20/25 concrete for columns and slab with measured compressive strengths of 25.5 MPa and 31.6 MPa respectively, while foundation beams employed higher-grade C30/37 concrete achieving 36.7 MPa strength. Longitudinal reinforcement consisted of B500C steel bars with experimentally determined yield stress of 564 MPa. Infill walls were constructed using hollow clay bricks with compressive strength of 9.8 MPa parallel to holes and 2.1 MPa perpendicular to holes, bonded with lime-cement mortar having 11.6 MPa compressive strength and 2.7 MPa flexural strength. The critical brick-mortar interface exhibited an average shear strength of 0.25 MPa, representing the weakest link in the masonry system.

2.3 Joint Configuration and Infill System

The beam-column connection system incorporates two distinct joint types to investigate differential seismic behavior: conventional connections following standard detailing practices, and modified joints featuring direct beam anchorage into columns (see Figure 3). This intentional asymmetry enables comparative analysis of joint performance under seismic excitation.



a) b)

Figure 3: Details of s two beam-column joint variations a) modified, b) conventional and slab extensions.

Masonry infill configuration varies systematically based on orientation relative to the primary loading direction. Walls perpendicular to seismic excitation feature full-length construction (1.24 m), while parallel walls incorporate partial infills (0.3 m length), enabling investigation of both in-plane and out-of-plane response mechanisms. The masonry components utilize standard clay bricks (60×90×190 mm) with 10 mm mortar bed and head joints. Following traditional Mediterranean construction practices, the uppermost brick course features intentional inclination to enhance mortar-slab interface bonding - a detail commonly observed in older regional structures.

2.4 Instrumentation and Monitoring Systems

The test specimen was comprehensively instrumented with strategically placed sensors to capture the complete dynamic response during seismic excitation, monitoring global displacement and acceleration as well as localized strain development and internal damage progression. Twelve uniaxial accelerometers were arranged in four triaxial clusters to capture the dynamic response. Acc1 through Acc6 recorded three-dimensional movement at the slab level (left and right corners), measuring x-direction (parallel to excitation), y-direction (perpendicular), and zdirection (vertical) responses. Acc7 through Acc9 monitored in-plane and out-of-plane behavior at the infill center, while Acc10 through Acc12 provided reference measurements at the foundation level. Eight draw wire sensors captured horizontal displacement data, with W1 and W2 monitoring lateral drift at the upper slab (left and right sides) and W3 and W4 tracking foundation beam motion. Diagonal sensors W5 through W8 were positioned across partial infill walls to monitor extension and compression during in-plane deformation, replicating typical diagonal cracking patterns in masonry infills. Twenty strain gauges were installed on longitudinal reinforcement bars during construction, with S1 through S8 positioned at expected peak moment locations and potential plastic hinge zones, while S9 through S16 were placed near beam-column joints. Surface-mounted gauges S17 through S20 monitored horizontal and vertical strains at the centers of both partial infill panels. Forty-eight piezoelectric transducers (PZTs) were strategically distributed throughout the structure using two installation methods: thirty-seven sensors were embedded as smart aggregates within concrete elements during casting, while eleven were externally bonded to masonry surfaces.

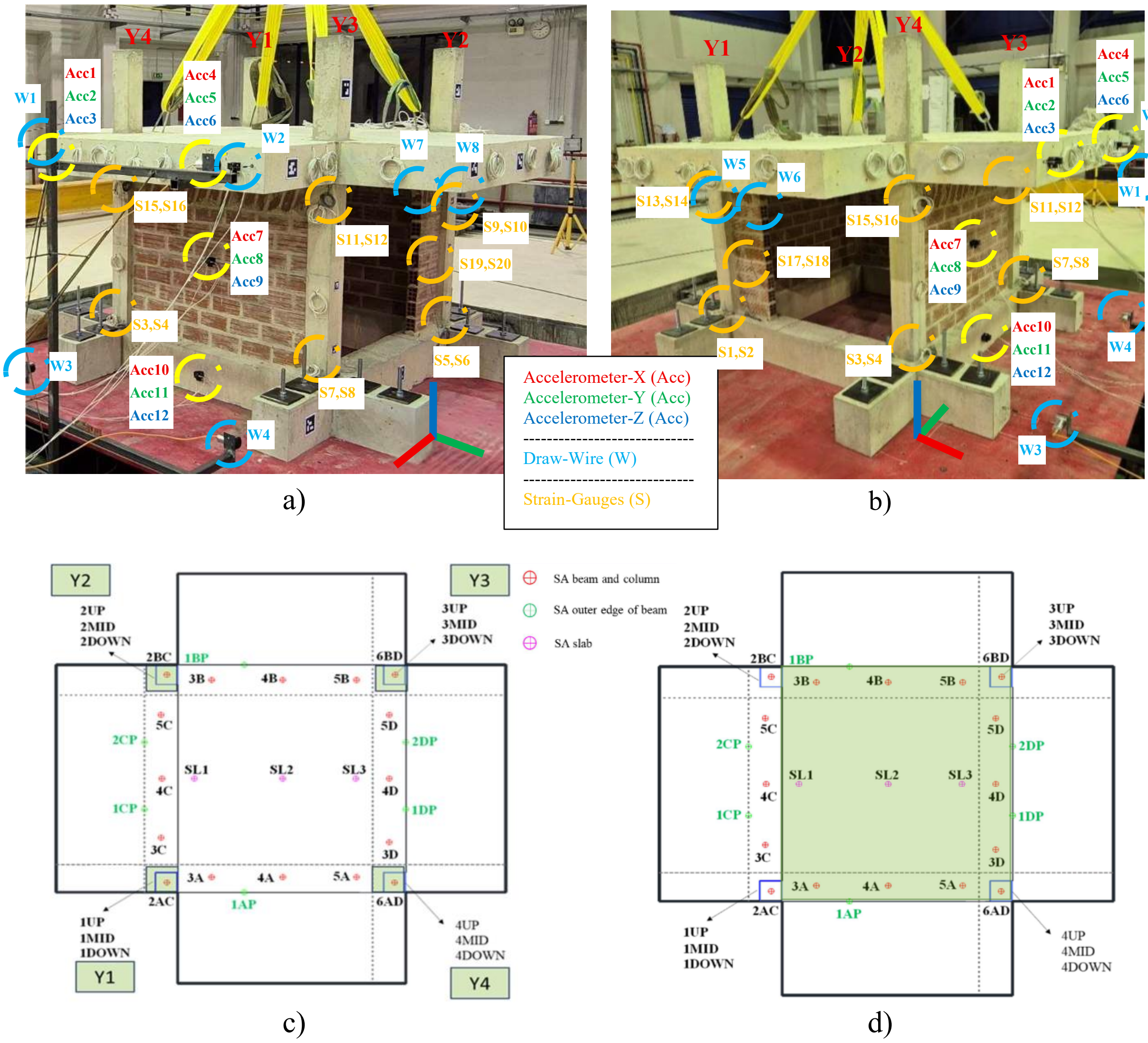


Figure 4: Position of the a)-b) accelerometers, draw-wires, strain-gauges and PZT c) inside the columns d) inside the slab

The embedded PZTs were systematically placed with twelve sensors in beams (three per beam), twelve in columns (three per column at different heights), four at critical beam-column joints, three within the slab, and six in cantilever sections. The masonry-mounted sensors included patches positioned at strategic locations around the corners and the center to monitor infill wall behavior. This comprehensive PZT network operated at frequencies of 10-250 kHz to detect internal damage progression and structural deterioration from initiation through development, providing real-time structural health monitoring capabilities. All sensors location are depicted in Figure 4.

2.5 Seismic Excitation

The structure was subjected to a progressive seismic testing protocol using the ay-component of the severe 1978 Thessaloniki (Volvi) earthquake as the base ground motion (see Figure 5). Five scaled intensity levels were applied sequentially, ranging from 0.1g to 1.1g peak ground acceleration, with each earthquake test preceded and followed by low-amplitude white noise excitations (0.08g) to monitor changes in dynamic properties. The testing campaign comprised eleven tests total (ST1-ST11 see Table 1), systematically increasing the seismic demand to achieve serviceability limit state damage levels in the reinforced concrete structure (serious damage of infill walls while maintaining steel reinforcement strains below yielding).

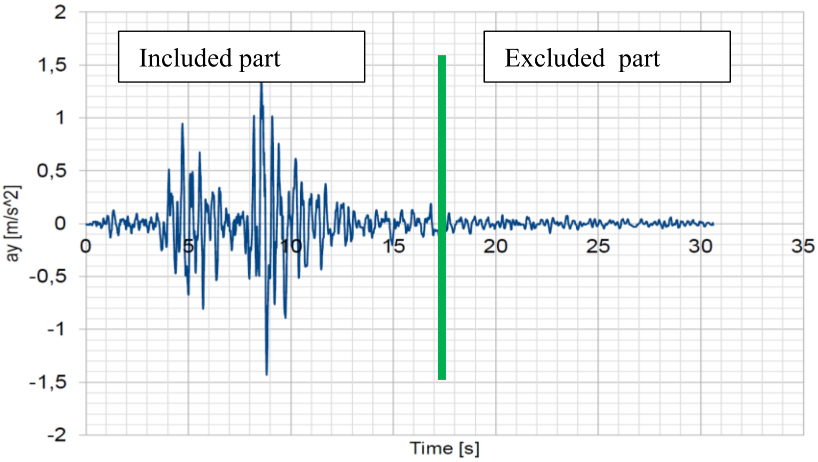


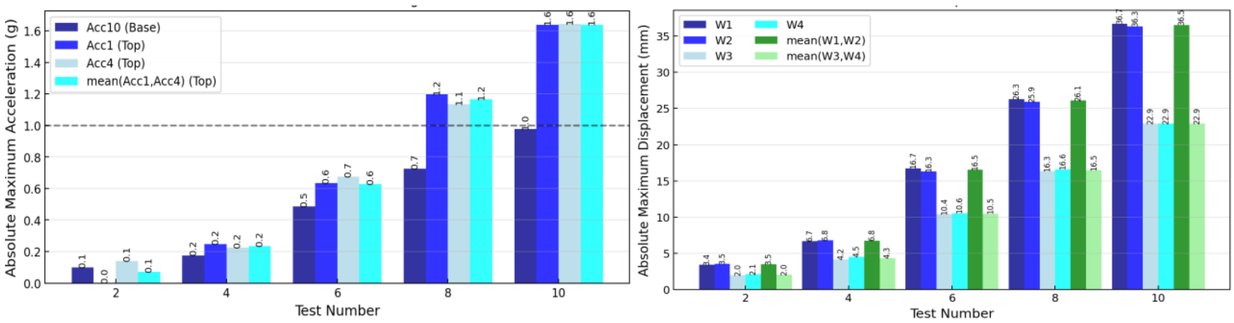
Figure 5: Time history executed (included part) by the DUTh seismic table (ay component of Thessaloniki (Volvi) earthquake record)

Table 1: Complete Test Sequence (11 total tests)

Test No.	Name	Type	Intensity	Test No.	Name	Type	Intensity
ST1	WNb0.1g	White noise	0.08g	ST7	WNb0.8g	White noise	0.08g
ST2	EQ0.1g	Earthquake	0.10g	ST8	EQ0.8g	Earthquake	0.80g
ST3	WNb0.2g	White noise	0.08g	ST9	WNb1.1g	White noise	0.08g
ST4	EQ0.2g	Earthquake	0.20g	ST10	EQ1.1g	Earthquake	1.10g
ST5	WNb0.5g	White noise	0.08g	ST11	WNa1.1g	White noise	0.08g
ST6	EQ0.5g	Earthquake	0.50g				

3 RESULTS AND ANALYSIS

The progressive seismic excitations revealed systematic amplification patterns in structural acceleration response and increasing displacement demands throughout the testing sequence (see Figure 7a). Peak ground acceleration (PGA) to peak floor acceleration (PFA) amplification ratios demonstrated the structure's dynamic characteristics, with floor amplification factors (FAF) ranging from 1.00 to 3.13 as summarized in the comprehensive test results (see Table 2). During earthquake excitations, the amplification factors progressed from 1.26-1.27 at initial serviceability levels (EQ0.1g and EQ0.2g) to 1.55-1.57 during higher intensity excitations (EQ0.8g and EQ1.1g), while white noise excitations consistently produced higher amplification values, reaching a maximum FAF of 3.13 during WNb0.8g testing (see Table 2). The displacement response showed a dramatic increase in structural demands (see Figure 7b), with interstory drift progressing from minimal values of 1.10-1.99‰ during initial excitations to a maximum of 11.37‰ during the final EQ1.1g earthquake test, representing the peak serviceability limit state demand. Despite these significant displacement demands, the structure maintained minimal residual drift values (maximum 0.25mm), confirming elastic recovery and indicating that the structure remained within serviceability limit state boundaries without permanent deformation. The correlation between acceleration amplification and damage progression was evident as the structure transitioned from elastic response during initial excitations to rather nonlinear behavior at higher excitation levels because of damages to partial infills, with the systematic increase in both FAF and drift values documenting the progressive approach to serviceability limit state thresholds.



a) b)

Figure 7: Progressive increase of a) acceleration and b) displacement

Table 2: Summary of results for accelerations and displacements (11 total tests)

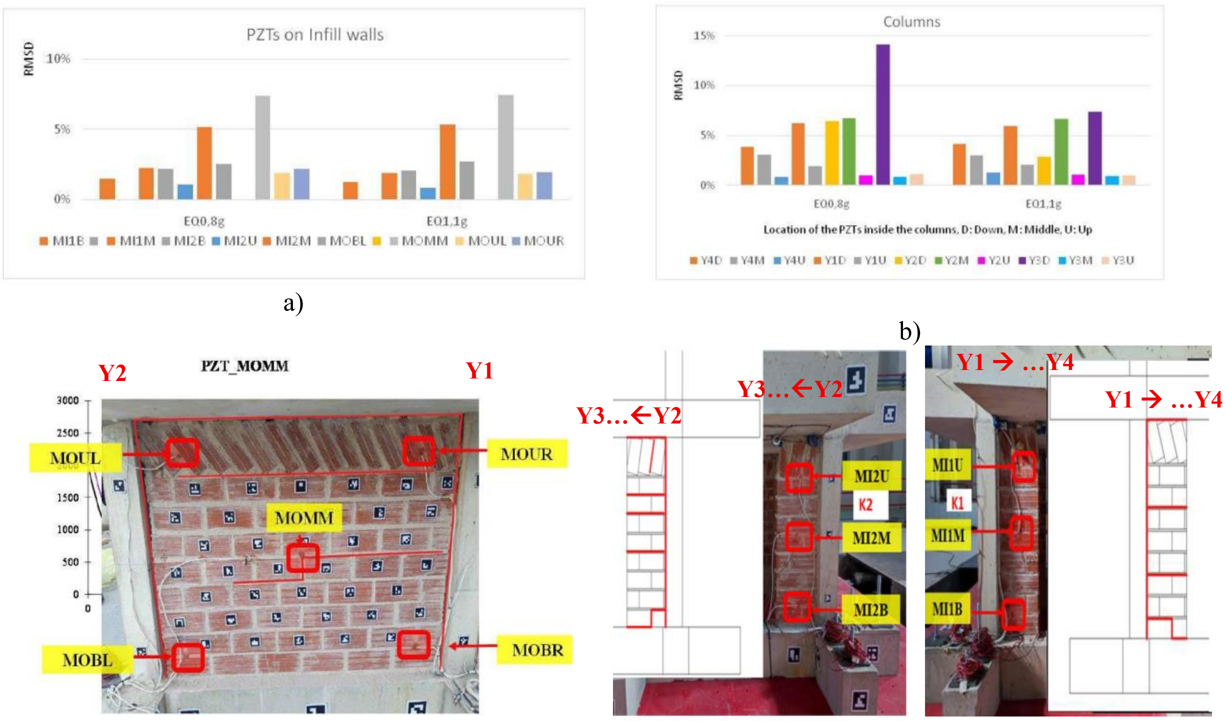
Test No.	PGA (g)	PFA (g)	FAF	drift (‰)	Test No.	PGA (g)	PFA (g)	FAF	drift (‰)
ST1	+0.13/-0.13	+0.11/-0.13	1.00	+1.10/-1.10	ST7	+0.06/-0.08	+0.25/-0.24	3.13	+2.29/-2.32
ST2	+0.07/-0.11	+0.14/-0.09	1.27	+1.20/-0.84	ST8	+0.69/-0.76	+1.19/-1.13	1.57	+8.04/-8.22
ST3	+0.1/-0.11	+0.12/-0.12	1.09	+1.08/-0.96	ST9	+0.06/-0.07	+0.12/-0.13	1.86	+1.32/-1.29
ST4	+0.15/-0.19	+0.24/-0.2	1.26	+1.99/-1.75	ST10	+1.08/-1.00	+1.67/-1.62	1.55	+11.3/-10.9
ST5	+0.06/-0.07	+0.11/-0.11	1.57	+1.01/-0.88	ST11	+0.06/-0.09	+0.19/-0.19	2.11	+2.30/-2.31
ST6	+0.47/-0.52	+0.68/-0.59	1.31	+5.06/-5.14					

The damage assessment revealed systematic deterioration patterns that were effectively captured through the piezoelectric transducer (PZT) monitoring system using Root Mean Square Deviation (RMSD) indices.

$$\mathrm{RMSD} = \sqrt{\frac{\sum_1^M \left(|V_p(f_r)|_D - |V_p(f_r)|_0 \right)^2}{\sum_1^M \left(|V_p(f_r)|_0 \right)^2}} \tag{1}$$

where $|V_p(f_r)|_0$ is the absolute value of voltage output signal at baseline/healthy state, $|V_p(f_r)|_D$ = absolute value of voltage output signal at damaged state, M = number of measurements in the frequency band and f_r = frequency.

The structure remained undamaged through initial serviceability levels (EQ0.1g and EQ0.2g excitations), with no visual damage or elevated RMSD values detected. The first brick infill serviceability limit state threshold was reached at EQ0.5g excitation (ST6), where initial damage accumulation commenced in infill wall 3-4, coinciding with the first appearance of elevated RMSD values from PZT sensors on infill walls (see Figure 7a). Progressive RC serviceability limit states at EQ0.8g and EQ1.1g excitations revealed systematic damage escalation: infill walls developed extensive horizontal and diagonal crack patterns with wall-to-frame debonding (see Figure 7d-e), while columns exhibited minor edge cracking at critical sections. The PZT monitoring system effectively captured this damage progression, with RMSD values increasing proportionally across different RC serviceability levels (see Figure 7b). Infill wall sensors recorded the highest RMSD increases in central areas where major horizontal cracks developed, while column sensors showed elevated values particularly in lower regions corresponding to observed cracking (see Figure 7c-e).



a) b)

c) d) e)

Figure 7: Damage details for a) PZTs on infill walls, b) PZTs on columns and visual damage observation on infill between c) columns 1-2, d) columns 2-3 and e) columns 1-4

4 CONCLUSIONS AND FUTURE WORK

This experimental investigation successfully established the baseline seismic performance of a representative non-seismically designed RC frame with masonry infills for subsequent vertical forest renovation implementation. The structure was excited up to RC serviceability limit state damage levels with maximum interstory drift of 11.37‰ and floor acceleration amplification factors ranging from 1.0 to 3.13, while maintaining minimal residual deformation (0.25mm). Initial damage commenced at EQ0.5g excitation with negligible horizontal bed joint mortar cracking of the out of plane (with respect to the dynamic excitation) infill wall 3-4 and negligible partial debonding from the concrete columns. After EQ1.1g the damage progressed to characteristic infill wall cracking within the mortar joints (both for in plane and out of plane ones) and complete wall_infill-to-RC_frame debonding from the top slab and adjacent columns (both for in plane and out of plane infills). No beam-column joint disintegration was observed. The PZT monitoring system validated real-time damage assessment capabilities, with RMSD indices successfully correlating with visual damage observations. The next phase will implement vertical forest renovation technology on the tested specimen, incorporating simulated planters with trees and simulated vertical living wall systems. It aims to investigate the potential of high-density urban forestation in existing buildings through suitable vertical forest renovation.

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