Proceedings of the 3rd Croatian Conference on Earthquake Engineering - 3CroCEE
19–22 March 2025, Split, Croatia

DOI: https://doi.org/10.5592/CO/3CroCEE.2025.119

COMPARATIVE NUMERICAL INVESTIGATION OF TIMBER ADDED FLOORS ON EXISTING RC STRUCTURES

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Abstract

As urban areas become increasingly dense, the vertical extension of existing structures has gained importance, especially in seismically active regions. The challenge is more pronounced for older buildings designed with minimal or no seismic provisions. Light frame timber systems are often preferred for vertical extensions due to their low mass, which minimizes additional seismic forces on the existing structure. At the Institute of Earthquake Engineering and Engineering Seismology (IZIIS) in Skopje, experimental shaking-table tests were performed on two-story reinforced concrete (RC) structures with timber-added floors to assess their structural performance. The goal of these tests was to examine the impact of vertical extension on the structural behavior under seismic loads. To validate the experimental findings, numerical analyses using OpenSees software were conducted. Initially, modal (eigen) analyses were performed to determine the natural frequencies and vibration mode shapes of the structure. This was followed by pushover analyses using a nonlinear static procedure in both the transverse and longitudinal directions. Modal force patterns based on the first mode of vibration were applied to simulate the loading conditions. Lastly, dynamic analyses under seismic ground motion were performed to further assess the behavior of the structure with timber-added floors. The comparative study highlights the effectiveness of using timber for vertical extensions, providing insight into its impact on the seismic performance of existing RC structures.

Keywords: Adding floors, vertical extension, Light frame timber, XLAM,

1. Introduction

A significant number of studies have been conducted on the dynamic behavior of lightweight structures made of wood and steel [1],[2],[3],[4],[5],[6],[7]. However, their behavior when used as materials for vertical extensions on existing structures has not been sufficiently investigated.

To address this, experimental tests were carried out at the Institute of Earthquake Engineering and Engineering Seismology (IZIIS) in Skopje on a shaking table. The tests focused on different extension systems applied to existing two-story RC structures. The aim of this research is to fill the existing knowledge gap by clarifying how the extension interacts with the existing structure and how it alters the dynamic characteristics and behavior of the integral system. Furthermore, the purpose of these tests is to comparatively investigate the impact of each type of extension system on the overall structural behavior.

Numerous dynamic tests were performed on the shaking table at the IZIIS laboratory. The investigation involved harmonic, synthetic, and real earthquake records as input in a single horizontal direction. To verify the test results, numerical analyses were subsequently carried out using the OpenSees software. Initially, modal (eigenvalue) analyses were performed to obtain natural frequencies and mode shapes. Then, a nonlinear static pushover analysis was conducted in two directions using modal forces corresponding to the first mode shape. Finally, dynamic analyses were performed using time histories of real and synthetic earthquakes. It should be noted that these analyses are preliminary, as much of the nonlinear behavior of materials and connections has not been considered.

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This study focuses on RC two-story models without infill walls (RCF1), RC two-story models with infill walls (RCF2), and extension systems made of steel (STL) and lightweight timber (LFT). The paper first presents the geometry of the models and material characteristics. Then, the methods for measuring the models are explained. Afterward, test results and experimental outcomes are presented. Finally, numerical investigations of the basic and extension systems using steel and lightweight timber applied to existing RC structures are discussed.

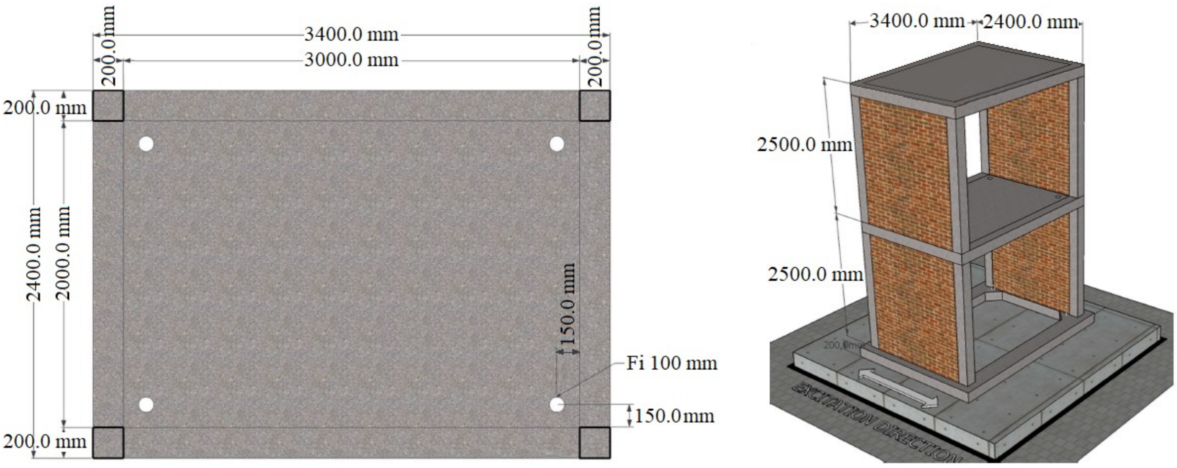
2. Experimental Investigation

2.1. Description of the Models

The basic 3D RC model (referred to in the text as model RCF1) represents a simple two-story structure with a single span and base dimensions of 340cm x 240cm (Fig. 1 and 2 a)). It consists of RC columns (20x20cm), RC beams (20x20cm), and RC slabs with a thickness of 7cm. The columns are reinforced with 4 longitudinal reinforcement bars of Φ16mm and stirrups of Φ8mm/10(20)cm. The beams are symmetrically reinforced in both the top and bottom zones, with a total of 4 longitudinal bars of Φ14mm and stirrups of Φ8mm/10(20)cm.

The RC foundation beams are designed with a height of 20cm and are fixed to the vibration platform using special anchors. Symmetrical longitudinal reinforcement consisting of 4 bars of Φ14mm is placed in the foundation beams, with stirrups of Φ8mm/8(20)cm.

The story height of the model is h = 2.5m (see Fig. 2). A two-story RC frame is rotated by 90 degrees (referred to as model RCF2), and masonry infill (modular hollow blocks, 19x19x29cm) is installed along the shorter direction.



a) b)

Figure 1. a) Building plan view, b) Building 3D view (RCF2)

The XLAM upgrade structure, depicted in Fig. 2 (b), was entirely constructed from 3-layer cross-laminated panels with a thickness of 10 cm. The wall elements were designed so that the outer two layers of the panel, each with a thickness of 3 cm, were oriented vertically, while the middle layer, with a thickness of 4 cm, was oriented horizontally. The XLam roof panel was oriented perpendicular to the direction of the extension. A CLT roof panel was secured to the CLT wall elements with 8 mm self-tapping screws.

Along the shorter side, two wall segments measuring 240 cm each were installed. On the longer side, three segments, each 80 cm in length, were placed—two on one side and one on the opposite. The CLT walls were fixed to the RC slab using 100 mm reinforced angle brackets supplied by Rothoblaas (Jancar et al., 2013).

The light timber frame upgrade (LTF), see Fig. 2 (c), consists of timber frame walls and a CLT roof panel. Two external timber frame wall elements (2.4 × 2.4 m) are placed along the shorter direction,

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while in the longer direction one wall segment measuring 1.2 × 2.4 m is placed on each side. In order to observe differences in dynamic behavior due to changes in stiffness, no mechanical connection was made between adjacent perpendicular wall elements in the LTF 1 and LTF 3 specimens. Where connections were made, self-tapping screws with a diameter of 8 mm were used at a spacing of 100 mm center to center.

The light timber frame wall consists of a timber frame (10/8 cm, C24), with studs placed at 62.5 cm spacing. Each wall is sheathed with three 12 mm thick OSB panels, which are attached to the timber frame using metal staples. The wall elements are anchored to the reinforced concrete slab using 100 mm reinforced angle brackets (Jancar et al., 2013).

To simulate the live load, the first- and second-floor slabs were each loaded with eight lead ingots of 400 kg each (see Fig. 2 a)). In addition, two 400 kg ingots were placed on top of the upgrade systems.

a) b) c)

Figure 2. a) RCF1 model, b) RCF1+XLAM, c) RCF1+LFT1

2.2.Dynamic Excitations

To gain insight into the elastic behavior of the models, tests were conducted using synthetic earthquake records with low amplitude (a modified Landers earthquake) and real earthquake records (El Centro and Petrovac) up to 0.1 g. The acceleration spectra of the aforementioned records are shown in Fig. 3.

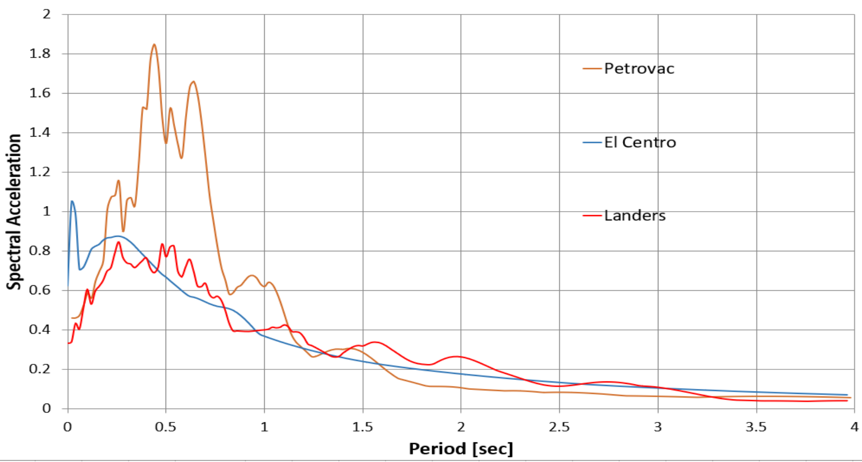

Figure 3. Acceleration spectrum for PGA 0.1g from the modified records used in the tests.

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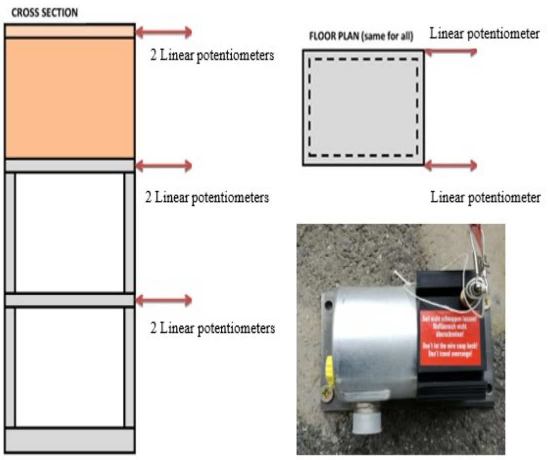
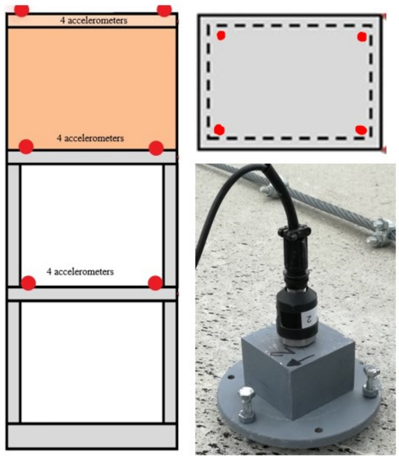
2.3. Instrumentation

The instrumentation concept for the model is shown in Fig. 4 a) and b). Two linear potentiometers were installed on each floor to measure absolute displacements (a total of 6). Additionally, two accelerometers in the transverse direction and two accelerometers in the longitudinal direction were placed on each floor to measure absolute accelerations (a total of 12).





a) b)

Figure 4. a) Two linear potentiometers on each floor, b) Four accelerometers on each floor

2.4. Experimental structure response

Some of the comparative results from the shaking table tests are shown in Fig.. 5-7. Fig. 5 presents the response of the basic two-story model RCF1, while Fig. 11 shows the response of the three-story model RCF1 + LFT1. It can be observed that the displacement amplitudes of the third story in the upgraded model RCF1 + LFT1 are smaller than those of the second story. This leads to the conclusion that higher mode shapes become more significant in the dynamic response when using materials like steel for upgrading RC structures. This phenomenon was also visually observed during the tests.

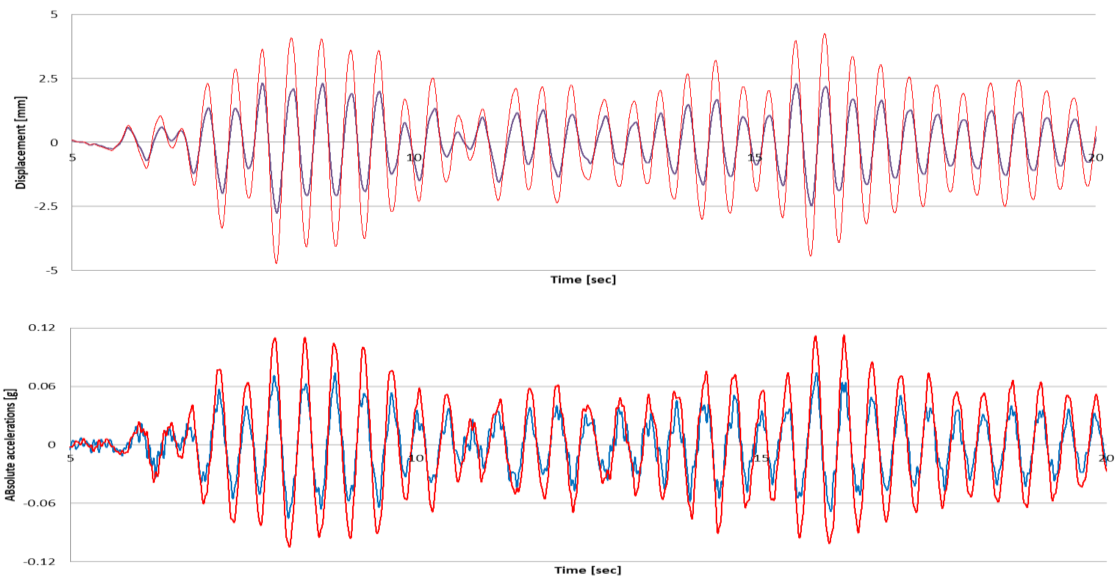


Figure 5. Experimentally obtained relative displacements and acceleration time histories for the RCF1 model: blue line – first floor, red line – second floor (Test 03).

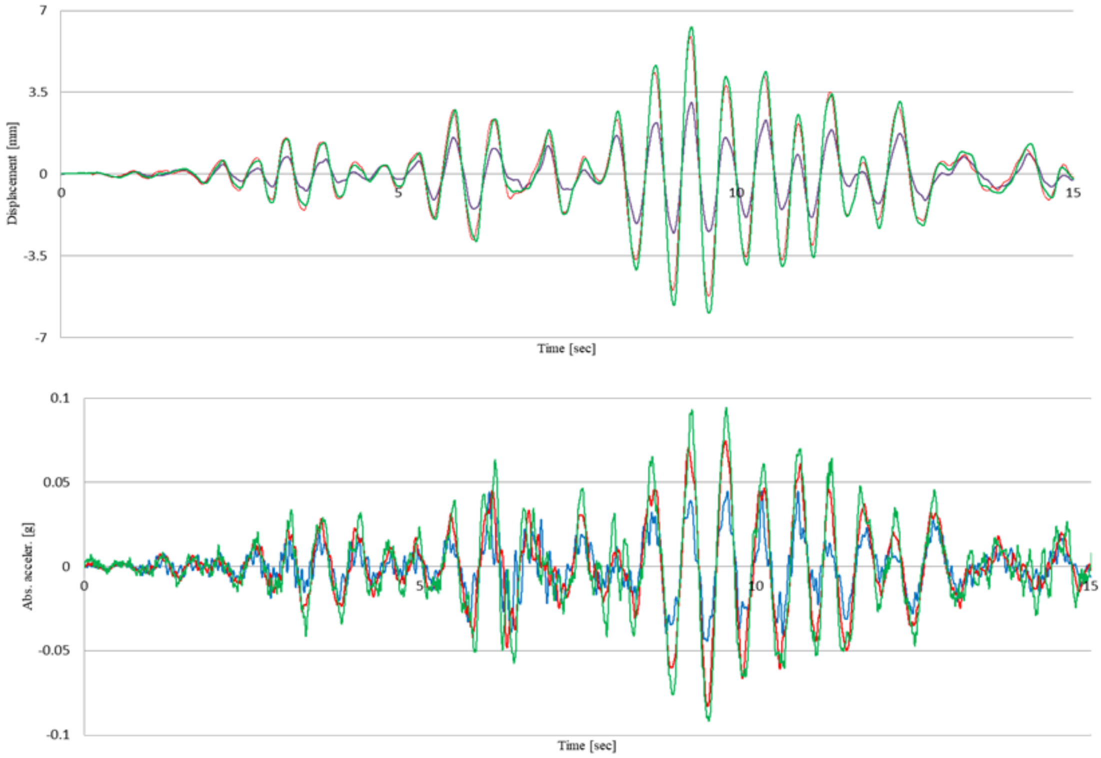


Figure 6. Experimentally obtained relative displacement and acceleration time histories for specimen RCF1+XLAM2: blue line – 1st story, red line – 2nd story, green line – 3rd story (Test 17)

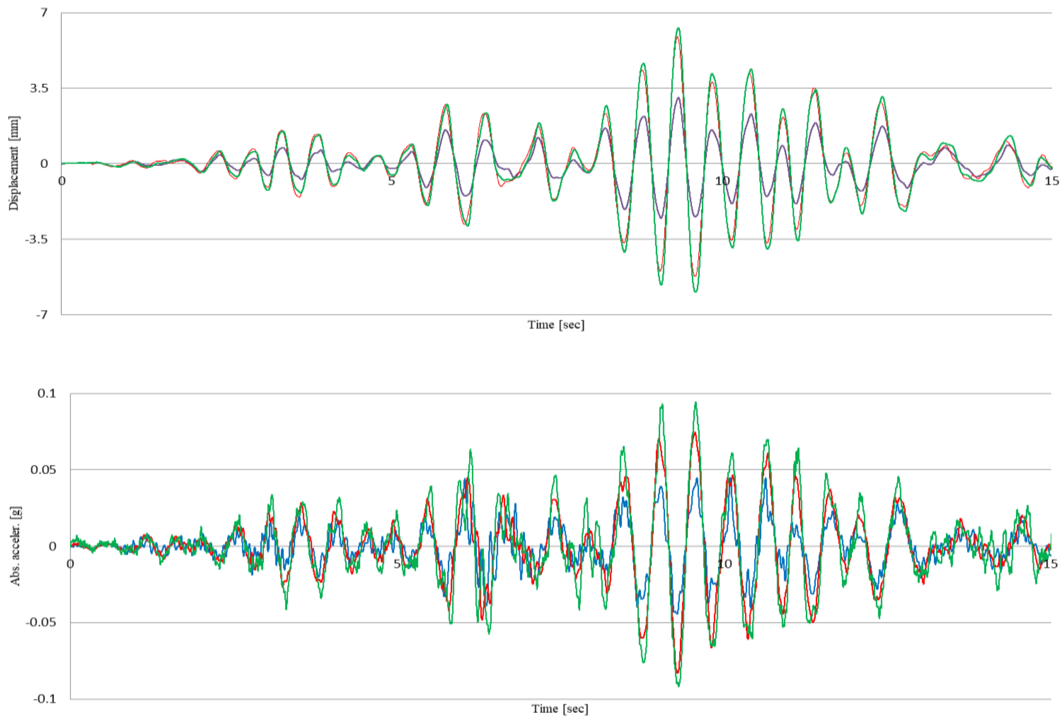


Figure 7. Experimentally obtained relative displacements and acceleration time histories for the RCF1+LFT model: blue line – first floor, red line – second floor, green line – third floor (Test 26).

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3. Numerical Investigation

3.1.Computer Model of the Specimen

Nonlinear analyses were conducted in OpenSees. The columns and beams were modeled as single-plasticity elements, consisting of an elastic element with two "fiber" sections at each end of the elastic element. For modeling in OpenSees, "Beam with Hinges" elements were used. The nonlinear behavior of the masonry was modeled by replacing the wall with an equivalent diagonal strut. The nonlinear behavior of the diagonals was defined by a force-displacement relationship, following the procedure proposed by Fardis et al. [9].

In OpenSees, truss elements were used to model the masonry, with the behavior of the masonry represented by a hysteretic uniaxial material. Rigid diaphragms were assigned at each floor level. The total mass and moment of inertia of the structure were applied to specific nodes located at the center of mass. The mass nodes from the total weight were horizontally connected with a rigid beam element to the center of the structure. Vertical loading was applied as concentrated loads at the columns.

The second-order effect (P-Delta theory) was also considered. The columns were fully fixed at the base. Since the columns and beams are well-reinforced with adequate transverse reinforcement (stirrups), it is assumed that their shear behavior remains elastic. The effective width of the slab in the inverted T-section of the beams was calculated using the method proposed by Eurocode 2 (SIST EN 1992-1-1, 2004) [10]. The plastic hinge length for the RC elements was determined according to Eurocode 8 (SIST EN 1998-3, 2005). The three-dimensional layout of the mathematical model is shown in Fig. 8.

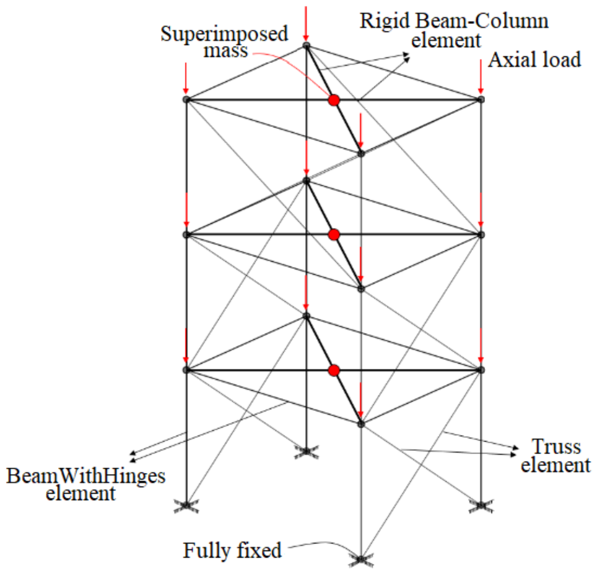

Figure 8. Three-dimensional representation of the mathematical model.

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3.2. Modal analysis

First, modal (eigenvalue) analyses were performed, and the results for the obtained periods and mode shapes of vibration were compared with the experimental results obtained from the shaking table tests. The correlation between the test results and the analysis results is presented comparatively in Table 2 (values in parentheses correspond to the simply supported model).

Table 1. Dynamic properties of models

Model		RCF1	RCF2	RCF1+STL	RCF1+XLAM	RCF1+LFT1
Values from experiment						
Mode	**1**	**0.390**	**/**	**0.510**	**0.55**	**0.542**
	2	**0.320**	**/**	**0.460**	**0.27**	**0.478**
	3	**0.190**	**/**	**0.250**	**0.14**	**0.21**
	4	**0.120**	**/**	**0.210**	**0.12**	**0.125**
Analytically obtained values						
Mode	**1**	**0.399**	**0.398**	**0.467 (0.617)**	**0.490**	**0.683**
	2	**0.357**	**0.118**	**0.403 (0.454)**	**0.468**	**0.676**
	3	**0.177**	**0.077**	**0.249 (0.372)**	**0.340**	**0.41**
	4	**0.118**	**0.052**	**0.204 (0.302)**	**0.117**	**0.36**

3.3. Dynamic analysis

- **General**

Dynamic analyses were performed using a stepwise increase in vibration amplitude (10%, 25%, 50%, and 100% of the Kobe earthquake amplitude). The dynamic analyses were conducted separately in both directions. The results presented for the RCF1 model correspond to excitation in the longitudinal direction, while those for the RCF2 model correspond to excitation in the transverse direction. In the dynamic analysis, 5% proportional mass damping was considered.

- **Response of the RCF1 Model**

The floor displacements and acceleration response history corresponding to the Kobe 100% record are shown in Fig. 9. The maximum displacement of approximately 80 mm was observed at the top of the structure during the final excitation. This corresponds to a 1.6% drift at the top. The peak acceleration at the top of the building, under excitation with the same intensity, reached approximately 1g. The maximum base shear force was approximately 100 kN.

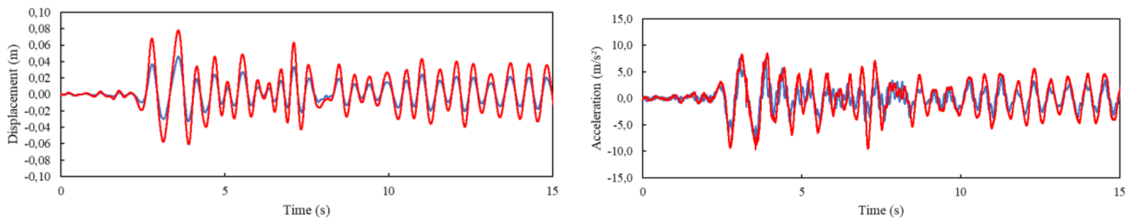

Figure 9. Analytically obtained relative displacements and acceleration time histories for the RCF1 model: blue line – first floor, red line – second floor (Kobe Test 100%).

- **Response of the RCF1+XLAM Model**

The floor displacements and acceleration response history corresponding to the Kobe 100% record are shown in Fig. 10. The maximum displacement of approximately 163 mm was observed at the top of the structure during the final excitation. This corresponds to a 3% drift at the top. The gradually reduced stiffness of the structure can be observed from the presented results. A nonlinear response of the building was obtained during the final excitation. The maximum base shear force was approximately 120 kN.

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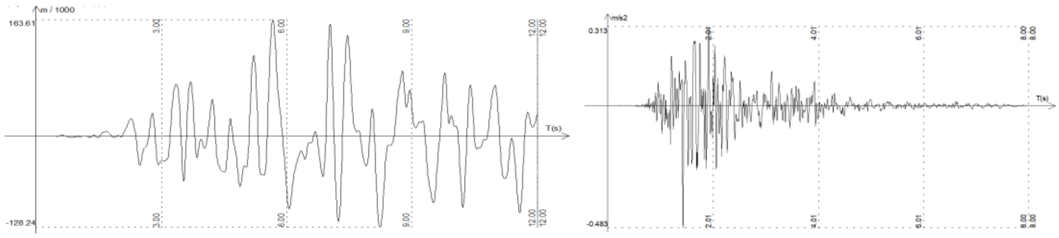

Figure 10. Analytically obtained relative displacements and acceleration time histories for the RCF1+XLAM

- **Response of the RCF1+LFT Model**

The floor displacements and acceleration response history corresponding to the Kobe 100% record are shown in Fig. 11. The maximum displacement of approximately 179 mm was observed at the top of the structure during the final excitation. This corresponds to a 3.5% drift at the top. The gradually reduced stiffness of the structure can be observed from the presented results. A nonlinear response of the building was obtained during the final excitation. The maximum base shear force was approximately 120 kN.

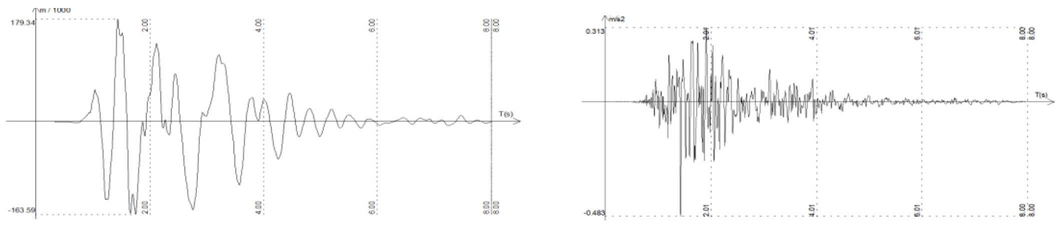

Figure 11. Analytically obtained relative displacements and acceleration time histories for the RCF1+LFT model: third floor (Kobe Test 100%).

4. Conclusion

From this research, as observed from the tested specimens, it can be concluded that all upgrades contribute differently to the overall dynamic response of the integral system. In this case, higher mode shapes were shown to be more significant. The lightweight timber structure(LFT) exhibited improved behavior compared to the steel structure. The connections with the concrete slab certainly had an impact on the response mechanism of the extension. the most favorable behavior due to compactness and properly distributed stiffness along the height has been observed in the case of the X-Lam panel upgrade (XLAM). The masonry also contributed to the different responses of the base and upgraded models. From the numerical analysis conducted, it was concluded that the model closely resembled the real structure in terms of modal analysis. The results from the earthquake records (Kobe) indicated similar characteristics to the real model. The displacement time history showed larger amplitudes in models without masonry. The model with masonry and a steel extension exhibited significantly higher amplitudes in the acceleration time history. In the future, a more detailed analysis will be conducted, including all earthquake records, as well as upgrades made from cross-laminated timber (CLT) and composite structures of wood and glass, to determine the behavior factor for this type of structures.

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