Shaking Table Test and Pushover Analysis on a Scaled Masonry Building

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SUMMARY:
The article deals with the analysis of a two-storey half-scaled masonry building that was tested at the shaking table of the Laboratory of Earthquake Engineering (LEE) as a part of the research project NIKER.
Three different models using well known FEM programs were used in order to perform modal and static non-linear analysis. In particular AEDES PC.E was used to perform modal and non-linear analysis using an equivalent frame model. SAP2000 and ACCA Edilus were used to carry out modal analysis with the former and modal and non-linear analysis with the latter. In both programs the walls were modeled with shell elements while frame elements were used for the wooden slabs and lintels. Both flexible and rigid diaphragm cases were examined. Differences between the analytical results obtained by all programs and the shaking table measurements on the instrumented specimen were identified and commented.

Keywords: Seismic test, Non-linear analysis, Masonry structures

1. INTRODUCTION

The issue related to seismic assessment and retrofit of existing masonry buildings is currently attracting significant attention because of progressive relative reduction of new construction activity with respect to interventions on existing structures. [8]
During an earthquake both out-of-plane and in-plane response are simultaneously mobilized, but it is generally recognized that a satisfactory seismic behavior is attained only if out-of-plane collapse is prevented and in-plane strength and deformation capacity of walls can be fully exploited. A global model of the structure is usually needed when the resistance of the building to horizontal actions is provided by the combined effect of floor diaphragms and in-plane response of structural walls [6].
The building specimen studied in this article was tested at the shaking table for two different configurations. Specifically, at its initial state, the diaphragm constructed of wooden planks and beams was rather flexible while, after testing that caused damages to the structure, it was stiffened to achieve "rigid type" behavior. A comparison of the experimental results for both states of the building is presented.
Modal and non-linear analysis of the building have been carried out using three different models developed with well known FEM programs. Since the role of non-linear static analysis is progressively recognized as a practical tool to evaluate the seismic response of structures, such analysis was carried out and two different approaches have been pursued [3,4,5]. In the first approach thin shell finite elements were employed to model the masonry walls utilizing non-linear constitutive law. In the second approach an "equivalent frame" model was used, also utilizing the same mechanical characteristics of the materials [7,10].
Through comparison between the experimentally extracted and the calculated modal properties, the accuracy of the FEM models was assessed. Once the accurate FEM simulation was validated, non-linear pushover analysis was performed. A comparison between the results obtained with the shell element modelling approach and those obtained with the equivalent frame approach is presented [9].

2. DESCRIPTION OF THE SPECIMEN

The specimen analyzed is a two-storey half-scaled masonry building constructed at the Laboratory of Earthquake Engineering (LEE) of the National Technical University of Athens (NTUA). It was tested at the shaking table of the laboratory as a part of the European research project NIKER [14].
Figure1 shows the floor plan and a photo of the specimen prior to testing on the shaking table. One door and five windows were arranged along the perimeter of the building at the ground floor, whereas six windows were opened at the first floor.

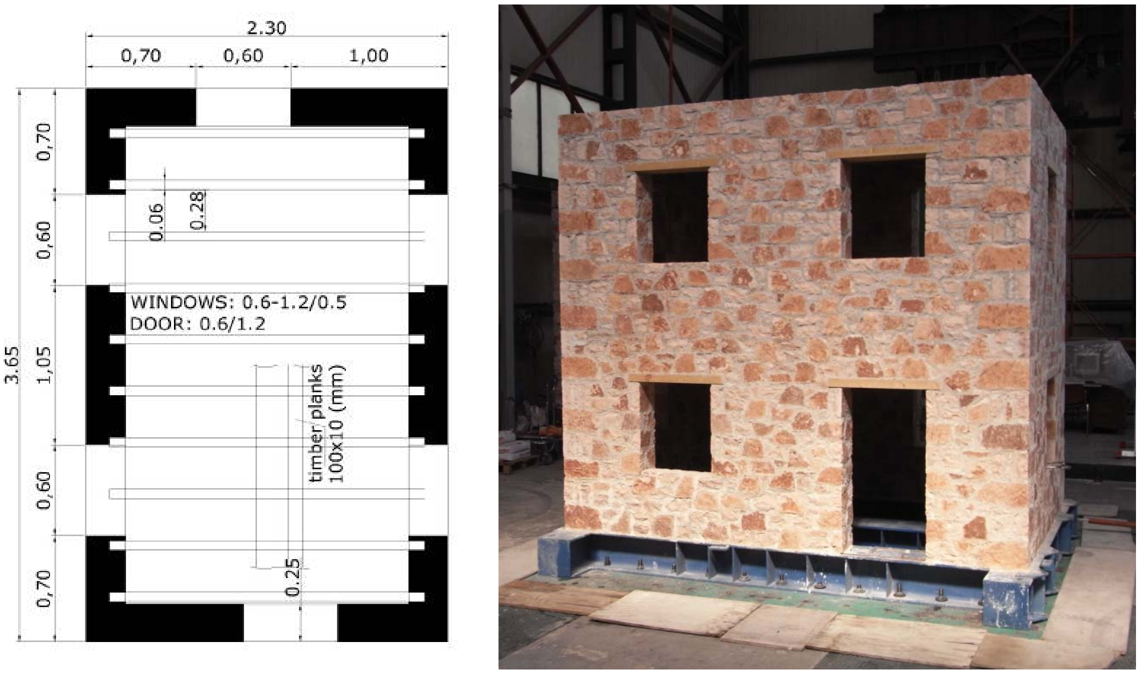

Figure 1. Floor plan and photo of the specimen on shaking table

The specimen was constructed on a steel base to simulate fixed boundary conditions. Also, the base allowed to move the specimen on rollers in order to facilitate the transfer to and from the shaking table.
The walls were made of three-leaf masonry. Limestone with a compressive strength in the order of 100 MPa was used for the construction. The stones thickness did not exceed 10 cm. The building was constructed with a mortar containing a low percentage of cement in order to reduce the curing time needed before testing the specimen.
As shown in Fig.2, the floors consist of timber beams (60x100 mm) placed every 340 mm. Timber beams are placed on collection beams constructed along the walls on the perimeter of the specimen. Also, a timber pavement constructed of timber planks (100x10 mm) nailed to the beams was provided. At the top of all openings timber lintels were placed. For similitude reasons, additional masses were placed over the floors; in particular, 4.5 Mgr were placed on the first floor and 3 Mgr were placed on the second floor.
The specimen building was subjected to simultaneous seismic motions acting along the two main horizontal axes. As a result the parts of the masonry between openings were subjected to combined in-plane and out-of-plane actions. The geometry of the building allowed formulation of squat and slender piers, a specimen design that allowed to study both flexural as well as shear behaviour.

Figure 2. Photos of the wooden slabs and the lintels

Two series of tests were carried out on this specimen:

1. *Building at initial state.* A proper protocol for earthquake simulation tests was applied: namely, low acceleration tests to extract the dynamic properties of the model and seismic action tests with increasing acceleration up to considerable damage or failure. For this purpose, an appropriate accelerogram of a real earthquake was selected, scaled and applied in a sequence in order to create damages that could be repairable. After the development of substantial damages, the specimen was strengthened and re-tested.

2. *Building with grouted masonry and enhanced diaphragm action of the floors.* After the completion of the test, the model was removed from the shaking table. Grouting was applied to masonry walls using a hydraulic lime based grout. Furthermore, the diaphragm action of the floors was enhanced by placing additional layers of timber planks on the top of the existing pavement. The added timber planks were inclined at 45° from those of the initial pavement and properly fixed with nails. The specimen was subjected to a series of tests similar to the first one, that is: “sweep” in order to extract the modal properties after seismic testing. For comparison, some of the seismic loads of the model at its initial state were repeated. The imposed seismic action was gradually increased, until progressively the specimen developed severe damage.

3. INSTRUMENTATION AND EXPERIMENTAL RESULTS

The instruments to measure accelerations (A) and displacements (D) were placed on the specimen following the scheme shown in Fig.3. Eight accelerometers and six displacement transducers were attached to the first floor, whereas seven accelerometers and six displacement transducers were placed at the second floor.

The dynamic properties of the building that is, the natural period of vibration and damping ratio, were extracted through sweep tests applying to the shaking table accelerations of increasing frequency along the longitudinal x-direction and the transverse y-direction. The results are presented in Table 1.

Table 1. Modal properties of the specimen

	Direction	Natural Frequency (Hz)	Natural Period (s)	Damping ratio (%)
At initial state	X	6.0 Hz	0.167	5.0
	Y	4.5 Hz	0.222	7.0
After interventions	X	10.5 Hz	0.095	6.5
	Y	10.5 Hz	0.095	5.8

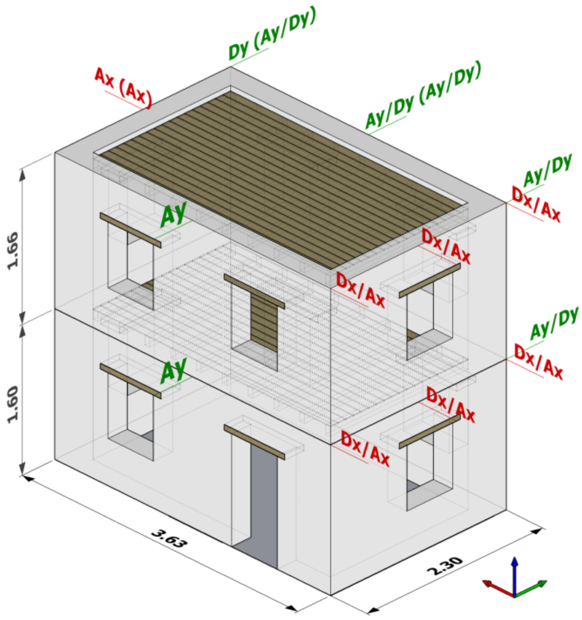

Figure 3. Scheme of the instrumentations

Acceleration time histories were developed based on records from a real earthquake. The spectra of this earthquake, that is, Kalamata (Greece) 1986, along the x and y axis are shown in Fig.4. The acceleration time histories were scaled and applied in a sequence in order to provoke gradual damages to the specimen.

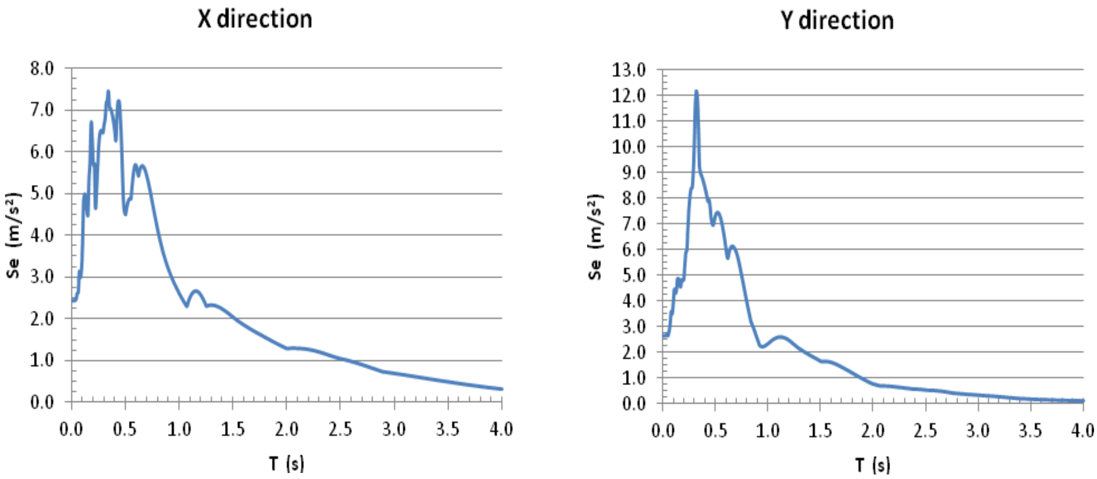

Figure 4. Response spectra of Kalamata earthquake, Greece 1986

4. NUMERICAL ANALYSIS

Three different models were used to run modal and static non-linear analysis for the specimen at its initial state as well as after the interventions. The FEM programs used to create the models and perform the analyses were SAP2000 [11], Acca Edilus [12] and Aedes PC.E [13].

4.1. Model with SAP2000

The masonry walls were modeled using four-node shell elements. Frame elements were used to model the lintels over the windows as well as the timber beams and the timber planks constituting the floors.

The final model shown in Fig.5 consists of 3170 shell elements, 1010 frame elements and 3666 nodes. The timber beams of the floors are pin-connected to the shell elements of the walls and the nodes at the base are fixed. In the model after the interventions a diaphragm action has been imposed to the nodes of the floors in order to account for stiffening in their plane.

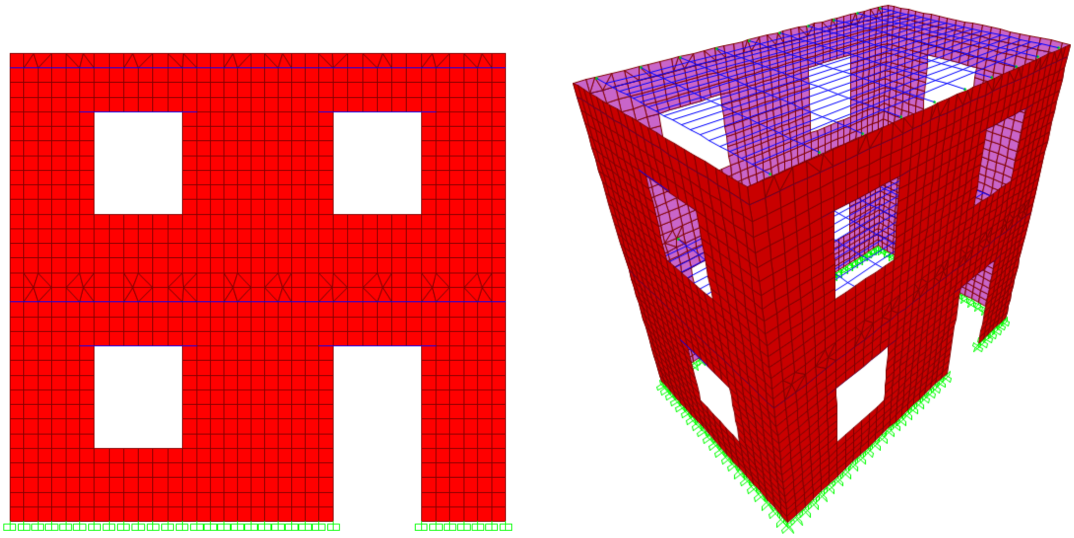

Figure 5. Front view and 3d view of the model with SAP2000

Since the results from compression tests at LEE on half-scaled wallets were not completed, proper mechanical properties of the masonry were selected and shown in Table 2. These properties were used to analyze the specimen at the initial state and after the interventions. In particular, the value of the modulus of elasticity was found by trial-and-error trying to match the natural period of the structure measured with the experiment. The values shown in Table 2 were used in all the other models.

Table 2. Mechanical properties of the masonry

	Modulus of elasticity (MPa)	Shear modulus (MPa)
At initial state	200	80
After interventions	450	180

Table 3 shows the natural mode of vibrations in the horizontal directions of the specimens. For each mode, the period and the participating mass ratio are listed.

Table 3. Modal properties of the model with SAP2000

	Direction	Period (s)	Ux	Uy
At initial state	X	0.166	0.738	0.000
	Y	0.227	0.000	0.752
After interventions	X	0.092	0.867	0.000
	Y	0.111	0.000	0.870

4.2. Model with Acca Edilus

The masonry walls were modeled using three-node shell elements, whereas frame elements were used for the lintels and the timber beams. No finite elements were used to model the timber pavements. Instead a slab object, a special provision of the particular software, spanning from one beam to another was employed in order to collect the load from the floors and distribute it to the adjacent beams. According to program capabilities, such slab object has no stiffness for the building at its initial state, whereas it has infinite stiffness for the case after interventions in order to account for the rapid diaphragm action of the floors. The model shown in Fig.6 consists of 7982 shell elements, 52 frame elements and 4690 nodes. The timber beams of the floors are pin-connected to the shell elements of the walls and the nodes at the base are fixed.

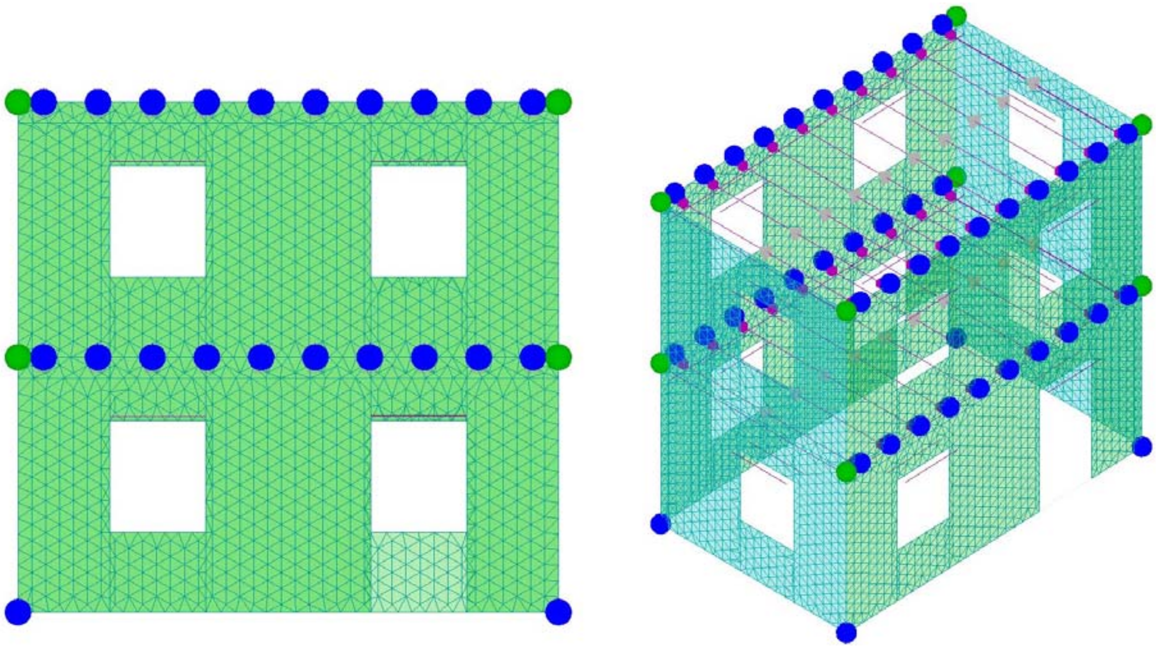

Figure 6. Front view and 3d view of the model with Acca Edilus

First a modal analysis was conducted. Table 4 shows the natural mode of vibrations in the horizontal directions of the building. For each mode the period and the participating mass ratio are presented. The values are quite close to the ones obtained with the experiment. The only exception is the natural period of the building at the initial state along the Y axis (0.310s versus 0.222s). This difference can be attributed to the way the slabs were modeled: the timber pavement was defined only as a load applied to the beams which are the only ones that contribute to the stiffness of the slab, whereas in the model with SAP2000, that better calculate the corresponding period, the slabs were modelled with more detail.

Table 4. Modal properties of the model with Acca Edilus

	Direction	Period (s)	Ux	Uy
At initial state	X	0.174	0.797	0.000
	Y	0.310	0.000	0.617
After interventions	X	0.108	0.829	0.000
	Y	0.127	0.000	0.829

4.3 Model with Aedes PC.E

The model is based on an equivalent frame idealization of the structure. Masonry walls are simulated with pier, spandrel and joint elements. The pier and the spandrel elements are frame elements with shear deformation, while the joint elements are infinitely stiff and modeled by means of rigid offsets placed at the ends of the pier and the spandrel elements. No finite elements used to model the floors. Specially, mono-directional slab objects, a special feature of the particular software, spanning from one wall to the other collect the load and distribute it to the walls were employed. For the building at its initial state they have no stiffness, while they are infinitely stiff for the building after the interventions. The model shown in Fig.7 consists of 59 frame elements and 40 nodes. The nodes at the base are fixed.

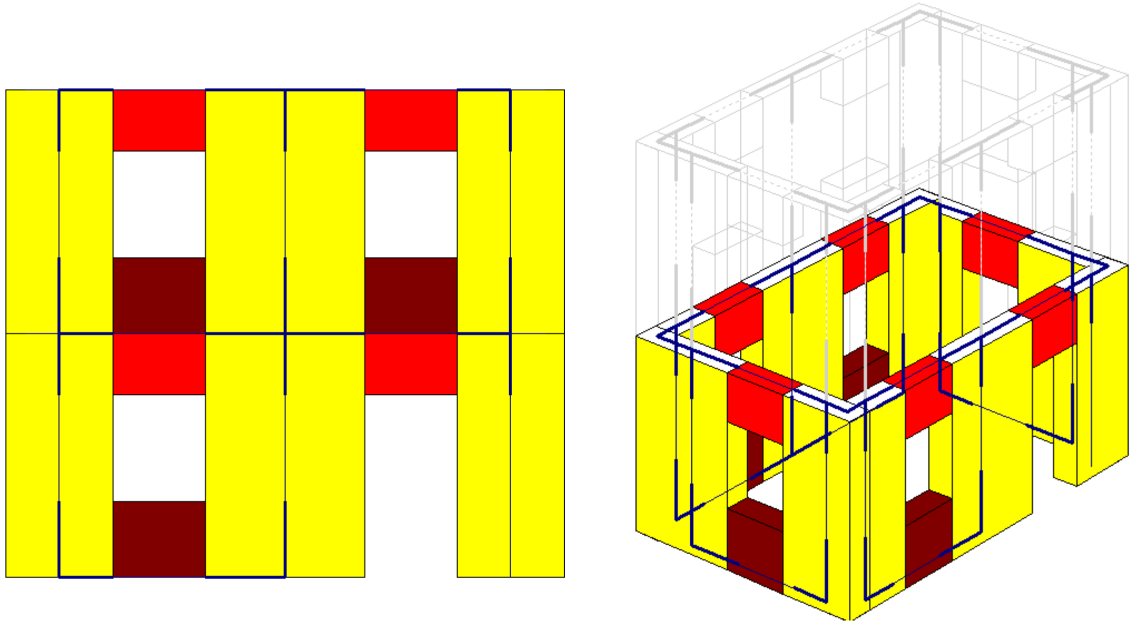

Figure 7. Front view and 3d view of the model with Aedes PC.E

A modal analysis was conducted. Table 5 shows the fundamental mode of vibrations along the horizontal directions of the building. For each mode the period and the participating mass ratio are presented. As for the model with ACCA Edilus, contrary to the other values that are quite close to the experimental results, the natural period of the building along the y-axis shows a considerable difference. Even in this case the difference from the experimental result (0.222 sec) can be attributed to the way the slabs were modelled.

	Direction	Period (s)	Ux	Uy
At initial state	X	0.162	0.806	0.000
	Y	0.308	0.000	0.539
After interventions	X	0.106	0.819	0.000
	Y	0.143	0.000	0.805

4.4. Static non-linear analysis

Linear and non-linear analysis have been performed but, because of space limitations, only non-linear analysis results are presented. Static non-linear analyses were carried out using the equivalent frame model (PC.E) and the shell elements model (Edilus). A uniform load distribution has been considered along the positive X and Y directions. Figure 8 shows the pushover curves obtained with the equivalent frame model, while the pushover curves obtained with the shell elements model are shown in Fig.9. In each figure the continuous curve refers to the building at the initial state, while the dashed one corresponds to the building after the interventions.

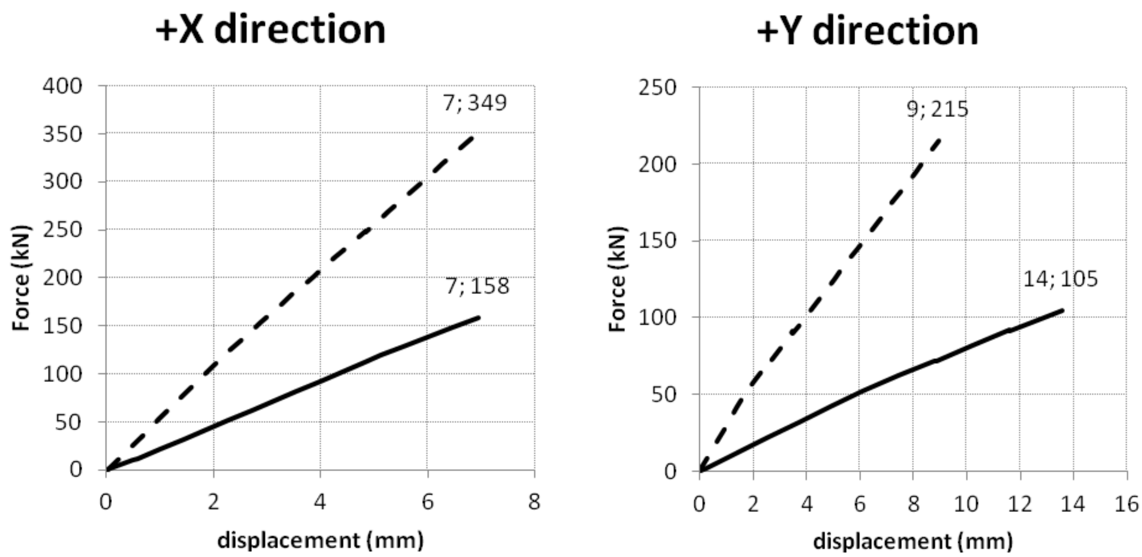

Figure 8. Capacity curves obtained with Aedes PC.E. a) x-direction. b) y-direction

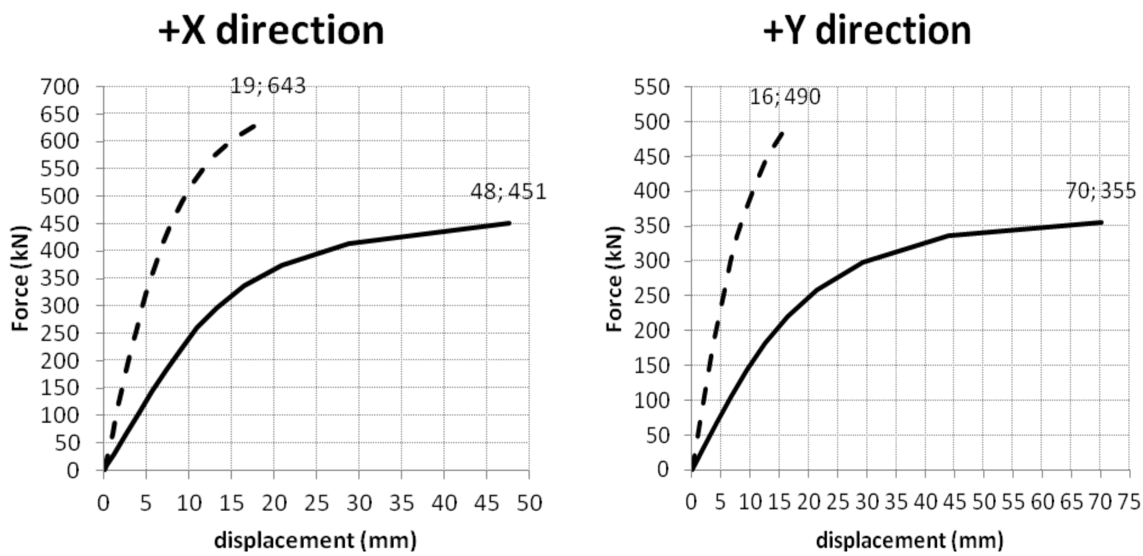

Figure 9. Capacity curves obtained with Acca Edilus. a) x-direction. b) y-direction

5. CONCLUSIONS

Based on the comparison between the experimentally extracted and the analytically determined modal properties, it can be stated that all models provided similar results and in close agreement.

Regarding static non-linear analysis, by observing Fig.8a and Fig.9a it is clear that in the PC.E model a "failure" of the specimen occurs at a smaller displacement than the one predicted by the Edilus model. However, both software provide practically the same base shear at the ultimate displacements of the PC.E capacity curves.

A similar observation for the maximum displacements along the Y-axis can be made regarding their values. However, the corresponding base shear differ by almost 100%. At this stage of the research project we were not able to explain this difference. Hopefully, this issue will be clarified when all experiments and associated analyses are completed.

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