Numerical Modeling of Two Adjacent Interacting URM Structures

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Abstract: ***Masonry structures in addition to their long heritage are still widely used in civil engineering practice. It should be emphasized that a lot of research has already been done on the seismic behavior of masonry structures. However, due to the nature of such a problem, its complexity and seriousness, the development of numerical models and their connection with experimental tests are always important. This is particularly significant considering their vulnerability to the action of horizontal forces generated during seismic excitations. In recent decades, many researchers have tried to capture the behavior of unreinforced masonry (URM) structures or reinforced concrete (RC) frames with masonry infills exposed to earthquakes, using different approaches. This paper tackles numerical modeling based on the finite element method (FEM) for the estimation of the dynamic response of two adjacent interacting URM units, subjected to shaking table motions. Geometrical and material properties of the specimen are provided by the Horizon 2020 project SERA-AIMS (The Seismology and Earthquake Engineering Research Infrastructure Alliance for Europe – Seismic Testing of Adjacent Interacting Masonry Structures). The analyses of dynamic performance were executed in SAP2000 software. Obtained results on the numerical model provide useful guidelines for modeling the nonlinear seismic behavior of masonry buildings.***

Index Terms: *Earthquake, Finite element modeling, Seismic response, Shaking table test, Unreinforced masonry*

1. Introduction

MASONRY as the oldest material in the construction industry has many advantages and is primarily intended for accepting vertical loads. However, due to its sensitivity to horizontal impacts caused by the action of earthquakes, nonlinear numerical modeling of such structures represents a challenging task. The main goal of this research is to provide a quick solution for predicting seismic behavior, but also to give a critical review of the possibilities for numerical analysis of unreinforced masonry structures. This study was created as a result of participation in a competition based on a blind prediction of the seismic behavior of two adjacent interacting stone masonry structures. The emphasis of the project was the examination of a half-scale stone masonry aggregate of two buildings with different dynamic properties, connected by dry joints. The prototype of the masonry building was tested in the National Laboratory for Civil Engineering (LNEC) in Portugal, by performing a shaking table test. The simulation was carried out in several steps under the conditions of the earthquake that occurred in Petrovac – Montenegro in 1979.

In general, the uncertainties of different modeling approaches and possible improvements are best quantified through blind prediction contests. A respectable example of this kind of competition, organized by the Pacific Earthquake Engineering Research Center (PEER) is described in [1]. It gathered even forty-one teams with the objective to assess key response quantities of a full-scale reinforced-concrete bridge column exposed to six successive unidirectional ground motions of varying severity.

The numerical analysis in this study was performed according to the strategy proposed in the original plan before the experimental test was performed. However, the real testing sequences differ from the original plan, as described in [2]. Linear and nonlinear analyses, established on FEM were employed in the phase of numerical modeling. The author initially performed a modal analysis to check the stability of the system. Then, pushover and elastic response spectrum curves were compared for given ground motions to determine the level of excitation at which the nonlinear behavior occurs. Finally, by executing a nonlinear dynamic (time-history) analysis, a complete dynamic response of the system is provided. Roof node displacements and base-shear forces are reported indicators of shaking table when cracks were expected based on previous analyses.

Finally, this study provides an overview of the necessary activities to obtain the full dynamic response of the system. Besides, the main possibilities and limitations of the adopted approach are also covered. The obtained results of numerical analyses and their comparison with the experimental values are very useful for future

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investigations of structures exposed to earthquakes.

2. PROBLEM STATEMENT

URM buildings represent a significant part of the infrastructure of many historical centers around the world, but there is a notable lack of guidelines for their modeling or they are poorly defined. In such structural systems, there is often a subsequent construction of adjacent buildings, where the old and new buildings share a common structural wall that is connected either by weak stones or by mortar joints. Previous experiences show that first damages usually occur in such joints, so it is very important to emphasize the research of the behavior of these elements with the simultaneous presence of different dynamic properties of adjacent units [3].

Experimental research of large-scale prototypes is often very demanding to perform but at the same time very expensive, and therefore the SERA-AIMS project campaign was carried out on a half-scale model. The contribution of such a project is very valuable for the scientific community because the most important indicators of the structure's behavior were reported. Such an experiment is significant not only because of its complexity or cost but also because of the lessons it can offer for the execution of similar facilities on site, during the retrofitting of existing ones, as well as because the fact that it provides valuable guidance for the design of future similar laboratory tests.

Appropriate design of URM structures can enable the prevention of earthquake disasters and thereby significantly reduce socio-economic losses.

3. THE BEST EXISTING SOLUTIONS

Several authors and SERA-AIMS project participants have tried to develop a prototype of the proposed specimen [4], and to provide various solutions using different tools [5].

Salvatori et al. [6] developed equivalent frame model (EFM) with nonlinear macro elements. These elements are developed by Bracchi et al. [7], [8] and Penna et al. [9], and were implemented using TREMURI [10] software to perform nonlinear static pushover analysis. Final response of the structure was investigated using N2 method of Eurocode 8 [11] and improved MN2 [12], [13] method to accurately capture displacement capacity. However, in the mentioned study a more detailed nonlinear dynamic time-history analysis is missing.

A similar approach using the OpenSees framework [14] with novel three-dimensional macroelements developed by Vanin et al. [15] is proposed by Tomić and Beyer [16]. Such macroelements are able to capture in-plane and out-of-plane behavior. Although capable of accurately predicting the development of damage mechanisms, there is also a lack of time-history results, so it is necessary to additionally calibrate the model in the post-diction phase.

AlShawa et al. [17] performed a study of the same half-scale masonry building using a three-dimensional finite-discrete non-linear dynamic model. Based on 8-node solid finite elements with one integration point, the time-history analysis was implemented using LS-DYNA [18] software. This approach can successfully account for the interaction between units, crack distribution, as well as separation between blocks, beams, and walls. Possible overestimation may occur due to some physical limitations, the size of blocks, and regular geometry compared to actual stone members.

Three-dimensional rigid block modeling for predicting the failure modes was successfully conducted by Gagliardo et al. [19]. The authors have developed a prototype using LiABlock_3D software which is able to generate a model formulated in CAD framework. Analysis has shown successful agreement of failure mode between the numerical model and experimental results. However, apart from the collapse mechanisms, more detailed non-linear static and dynamic analyses are needed.

Similar to the present study, a simplified finite element (FE) method was derived by Ramaglia et al. [20]. Apart from some different conditions, related to the type of structure restraints or stress-strain relationship, this model made satisfactory predictions with slightly underestimated displacement field. This is a possible consequence of the absence of geometric nonlinearities and neglect of large displacements.

4. THE PROPOSED SOLUTION

In this study, the FE method is elaborated for making predictions of structural behavior. This is particularly significant considering the widespread use of this method in various software packages. This enables a wide scope of applicability of such technology, but also the fact that researchers either in science or practice are well familiar with FE. The application of this method often gives satisfactory results for a broad range of real problems, and sometimes allows significantly less time consumption on performing analyses, compared to other techniques. SAP2000 software provides harmony between the need for more complex seismic analyses and the required accuracies of predicting actual behavior. In contrast to other proposed solutions, an often compound time-history analysis is enabled, which provides a more detailed insight into the state of the structure during the action of the earthquake. Softwares often suggest the use of pushover analysis only, as a compromise solution between cost and quality, but such analysis is sometimes not sufficient to obtain a full dynamic response of the structure.

5. *Conditions of the analysis to follow*

Macro modeling implies a kind of approach where the masonry units, mortar joints, and their mutual interaction behave as a homogeneous anisotropic continuum. Such a process, also presented in FEM, enables less computational effort due to simplifications of the applied homogenization system [21].

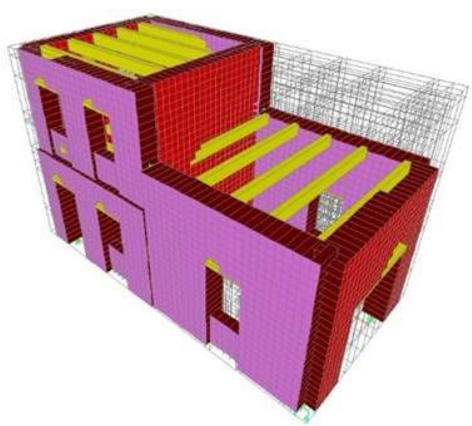


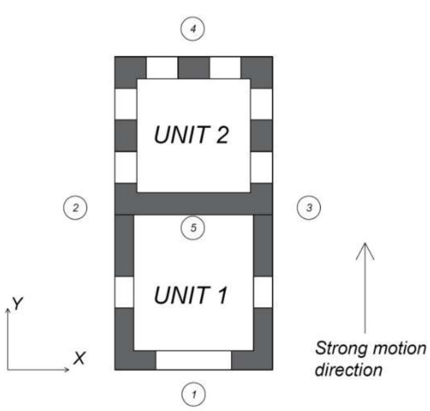
Figure 1: Three-dimensional FE model

FEM is an approximate method of numerical analysis for solving differential equations that describe physical phenomena. In this study, the SAP2000 software was implemented for the purpose of applying the FE method. Masonry walls are modeled by means of nonlinear layered shell elements with permitted in-plane and out-of-plane behavior. The three-dimensional FE model presented in Figure 1 is simply restrained at the base in order to avoid the activation of bending moments, and possible erecting and demolishing walls. Beams and walls are attached using body constraint connection with released moments, due to the difference in stiffnesses. The interface between independent units has been modeled using link-GAP elements with a separation of 3 mm. Such a relationship can disable penetration of units, by putting out-of-action stiffness in tension. Since geometric nonlinearities and large displacements are neglected in this study, in order to avoid numerical instability, P-delta effects are also not assigned to link-GAP elements. SAP2000 enables setting the basic properties of the link elements, such as effective stiffness and effective damping, in the case of performing linear analyses, or stiffness and opening in the case of nonlinear analyses. In addition to the opening of 3 mm, the author adopted a link-GAP element stiffness of 1000 kN/m^2.

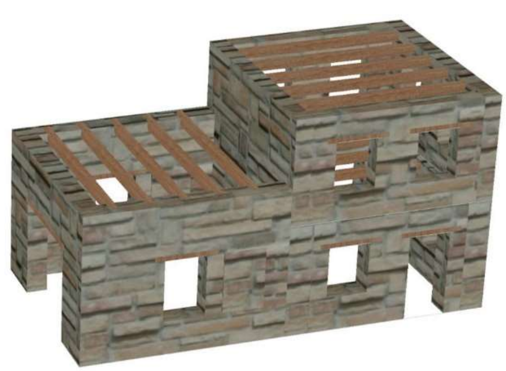
The mentioned conditions were applied to the numerical model in order to adapt as much as possible to the actual environment.

6. *Details of the proposed solution*

The constructed half-scaled test specimen presented in Figure 2, consists of two adjacent disconnected units, first unit with one floor and second with two floors. Units have been built with double-leaf stone masonry with varying wall thicknesses and have different heights and storey levels. Walls are poorly connected with perpendicularly-oriented timber diaphragms. The more detailed model geometry is presented by Tomić et al. [3].



(a) Orientation



(b) 3D model

Figure 2: Specimen orientation and 3D model [22]

It is important to note that the geometrical, dynamic and material properties had to be scaled by a factor λ=0.5, as recommended in Table 1, respectively [23].

Table 1: Scaling of parameters

Parameter	Scaling factor
Length	λ
Area	λ^2
Volume	λ^3
Moment of inertia	λ^4
Displacement	λ
Velocity	$\lambda^{1/2}$
Acceleration	1
Time	$\lambda^{1/2}$
Period	$\lambda^{1/2}$
Frequency	$\lambda^{-1/2}$
Mass	λ^3
Force	λ^3
Density	1
Stress	λ
Strain	1
Young's modulus	λ
Poisson's coefficient	1
Shear modulus	λ
Strength	λ
Cohesion	λ

Due to restrictions of the shaking platform used on a scaled model, the period and time should be reduced by factor $\lambda^{1/2}$ [24]. The test was conducted using earthquake excitation components for two directions (X and Y), recorded from the Montenegro Albatros station in 1979.

6.1 *Material properties*

One of the previously performed test [25] was provided the data of material properties. Major reported masonry properties are density, compressive strength, tensile strength, Young's modulus in compression and Poisson's ratio, with average values of 1980 kg/m^3, 1.30 MPa, 0.17 MPa, 3462 MPa, and 0.14, respectively, as defined in Table 2.

Table 2: Material properties

Properties	Unit	Values
Modulus of elasticity	MPa	3462
Poisson's coefficient	-	0.14
Density	kg/m^3	1980
Compressive strength	MPa	1.30
Strength in tension	MPa	0.17
Damping coefficient	-	0.05
Friction angle	-	0.3
Dilatation angle	-	35°
Hysteresis type	-	Takeda
Stress-strain curve	-	Mander

Some features as friction and dilatation angles were obtained through a model calibration process using a vertical compression test as described in section 7.

6.2 *Masses and additional loads*

In addition to seismic forces, both units are burdened by self-weight and moreover, Unit 2 was loaded with supplementary uniformly distributed masses of 1500 kg on both floors. The total mass of buildings is 23673 kg, i.e. 7415 kg for Unit 1 and 16268 kg for Unit 2.

7. *Analysis*

In order to obtain a full seismic response of aggregate with two masonry units, in the following subsections nonlinear static (pushover) analysis and nonlinear dynamic (time-history) analysis were executed on a 3D FE model. Previously, according to the provided data, it was necessary to calibrate a model, by using vertical compression test [26]. The calibration test is based on applying a vertical compressive force, monotonically or cyclically to the sample, while maintaining the force centered to the wall section. Defined stress-strain constitutive relationship in tension and compression, is presented as a modified Mander curve (see Figure 3).

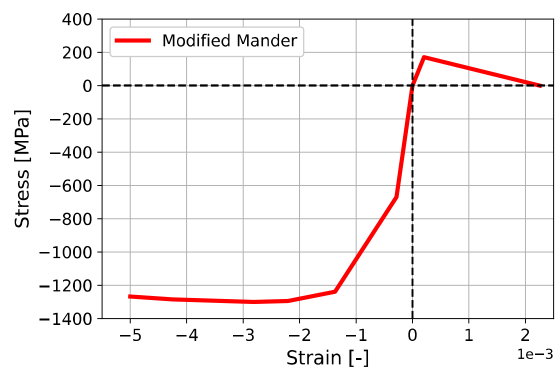


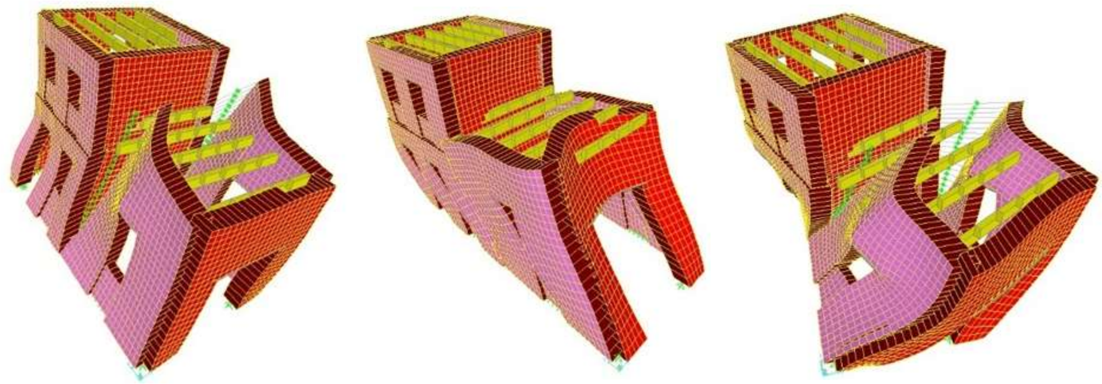
Figure 3: Stress-strain relationship

7.1 *Modal analysis*

The main goal of modal analysis is to discover fundamental dynamic characteristics in forms of mode shapes and natural frequencies. Mathematically, modal analysis represents a transformation between base and principal coordinate systems. It enables engineers to

Table 3: Periods and frequencies

Mode	Period [s]	Frequency [Hz]
First mode	0.0408	24.484
Second mode	0.0379	26.381
Third mode	0.0344	29.034



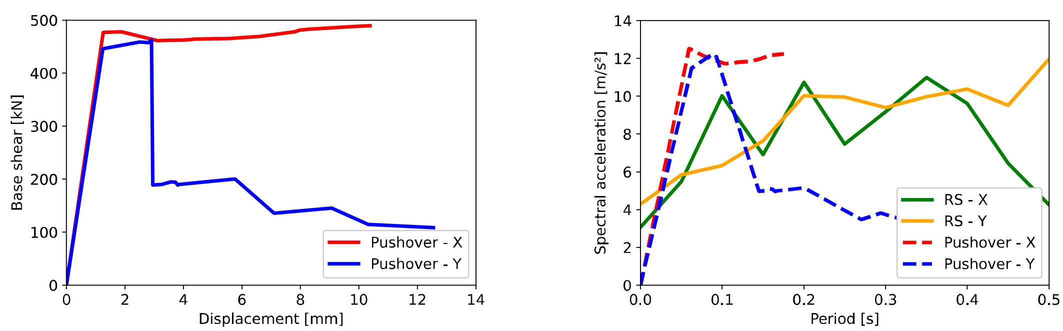
(a) First mode (b) Second mode (b) Third mode

Figure 4: Mode shapes of unloaded structure

predict the dynamic response of a structure to earthquake ground motions.

In fact, modal shapes and frequencies were mapped by calculation of eigenvectors and eigenvalues as yield of the equation of motion, respectively. Values of free vibration periods and frequencies are automatically calculated by SAP2000 software (see Table 3). In accordance with first three mode shapes and their directions illustrated in Figure 4, it can be concluded that an unstable system was not generated in the modeling process.

generated in order to compare it with the RS curves, independently for the X and Y directions. Plot which determines a peak response of the system with single degree of freedom, depending on the dynamic characteristics and for a certain dynamic load is called the response spectrum. To define RS curves, records of the acceleration over time were produced in SAP2000 software. Adopted nonlinear shell elements are able to capture both, in-plane and simultaneous in-plane and out-of-plane nonlinear behavior. Both cases, gave such determination that the structure has the same behavior in the linear domain, which was



(a) (b)

Figure 5: (a) Pushover curves (X and Y directions), (b) Response spectrum/Pushover curves (X and Y directions)

7.2 Pushover analysis and response spectrum

The nonlinear static pushover analysis is a method for performance evaluation of the structure subjected to a gravity loading with monotonic increasing lateral load until reaching an ultimate condition. The aim is to compare capacity and response spectrum (RS) curves for different magnitudes of earthquake, with an intention to predict the occurrence of the failure. As this would be possible, in addition to the main notation of the pushover curve using the base shear force-displacement relationship (see Figure 5a), its acceleration-period form (see Figure 5b) was also

expected. Nevertheless, in the nonlinear domain, an earlier drop in force for simultaneous action is evident and will be considered during the comparison with RS. This is the case in both (X and Y) directions.

As mentioned before, the pushover capacity curve generates base shear forces and displacements as output parameters. The purpose of the comparative analysis with the RS curves is the determination of the load intensity at which cracks could be expected during nonlinear dynamic (time-history) analysis. Comparison demanded to convert the pushover curve by observing periods instead of displacements. It is important to note that this conversion assumes a linear relationship between displacement and period, which may not hold true for highly nonlinear systems.

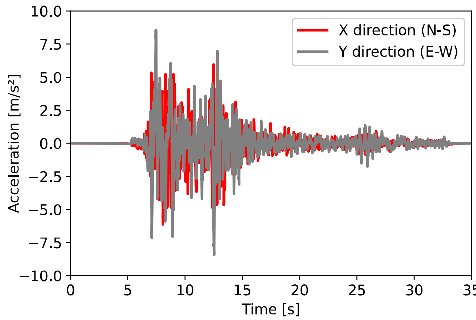


Figure 6: Acceleration records N-S and E-W with the scaled time step

Additionally, the effective lateral stiffness may change during the structure's response, so the conversion will provide an approximation rather than an exact representation. A detailed inspection of the results revealed that the first crack pattern formation could be expected at 50% of the RS scaling factor, as shown in Figure 5(b). Displacements up to that level are almost negligible (i.e. for a 25% scaling factor). Similar conclusions about developing cracks derived from the time-history analyses were reported in [6], [19], [20].

Intending to check the conclusions based on the results of the pushover analysis, but also to compare it with investigations derived by other researchers, a nonlinear dynamic time-history analysis was carried out in the following subsection.

7.3 Time-history analysis

Nonlinear dynamic response of the structure under variable loading, may be monitored step-by-step using time-history analysis. Solving the equations of dynamic equilibrium was performed by the combined application of the modal method known as Fast Nonlinear Analysis (FNA) and the direct integration method. The goal of such fusion was to generate optimal results with as little time consumption. Structural behavior (displacements, base shear forces) was captured by using FNA i.e. modal superposition rule.

The recordings of the acceleration of the ground during earthquake for perpendicular directions (N-S and E-W) X and Y are shown in Figure 6. Based on previous guidelines, time is compressed by a factor of $\lambda^{1/2}$. Measurement points at the top of both units (Rd3, Rd6 for X direction and Rd2, Rd5 for Y direction) are shown in Figure 7.

It is found that after the first step with 25% of scaling factor along the Y direction, only separation of units occurred, with no cracks noted on them. This is in agreement with experimental results as well as with the results provided by [19].

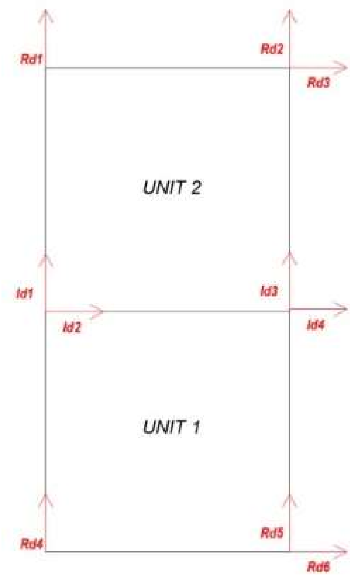
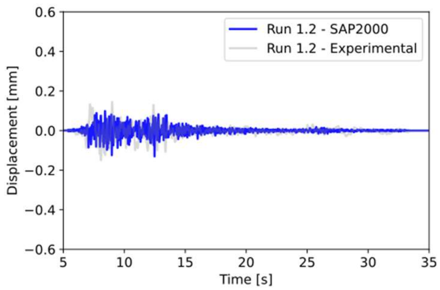


Figure 7: Measurement points [4]

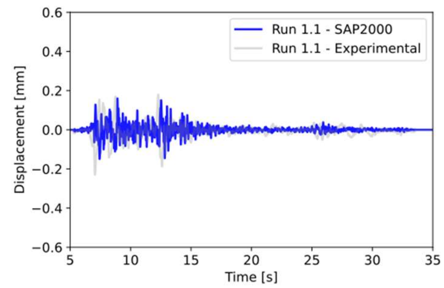
Similarly, as in [20], a slightly underestimation of displacements is noticeable in the lowest sequences (25%) for both directions, which is additionally enlarged for 50% of shaking capacity, as shown in Figures 8 and 9. A possible reason lies in the fact that geometrical nonlinearities and large displacement effects are disabled, as well as the influence of residual stresses/displacements was not considered. Also, the rocking effect after

Table 4: Testing sequence [4]

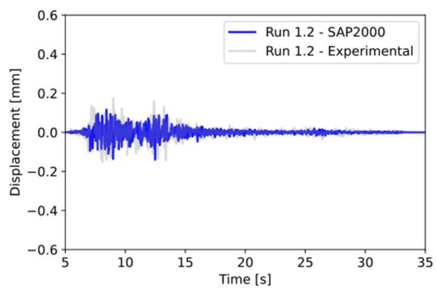
Level of shaking	Substep I	Substep II	Substep III
25% scale factor PGA 0.156/0.219g (X/Y)	Y direction (Run 1.1)	X direction (Run 1.2)	Bidirection (Run 1.3)
50% scale factor PGA 0.313/0.438g (X/Y)	Y direction (Run 2.1)	X direction (Run 2.2)	Bidirection (Run 2.3)
75% scale factor PGA 0.469/0.656g (X/Y)	Y direction (Run 3.1)	X direction (Run 3.2)	Bidirection (Run 3.3)
100% scale factor PGA 0.625/0.875g (X/Y)	Y direction (Run 4.1)	X direction (Run 4.2)	Bidirection (Run 4.3)



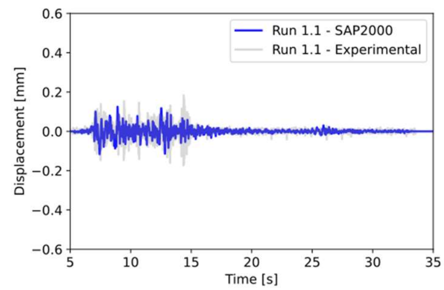
(a) Rd3



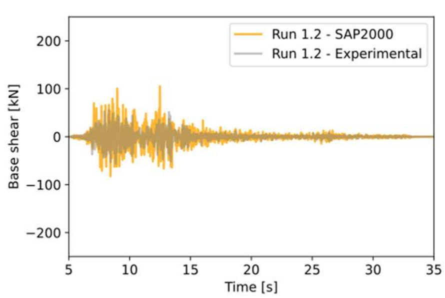
(b) Rd2



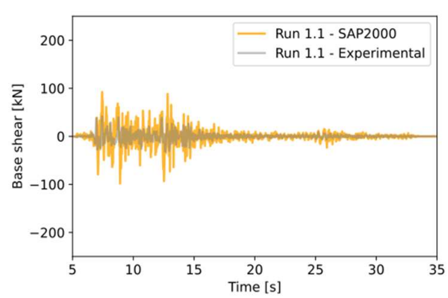
(c) Rd6



(d) Rd5



(e) Base shear X



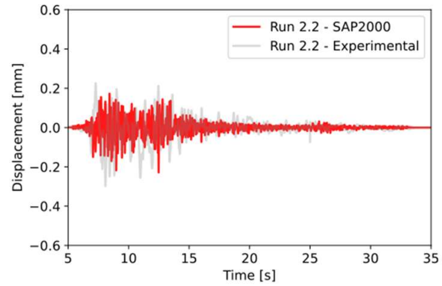
(f) Base shear Y

Figure 8: Comparison of numerical and experimental results of structural response: (a)-(d) Displacements (scale factor 25%, runs 1.1 and 1.2), (e)-(f) Base shear forces (scale factor 25%, runs 1.1 and 1.2)

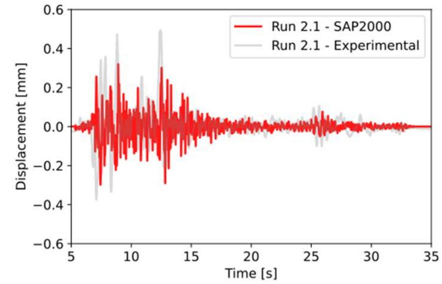
the failure can affect the FEM prediction.

The application of layered shell elements for modeling plates may also cause uncertainties of the results, which require deeper investigation. Conclusions about the damage patterns can be derived indirectly, by monitoring the stress level in the finite elements during the time-history analysis. The model satisfactorily highlighted the stage of the first damages in the structure (50% of scaling factor), which also found consensus with other studies. After the first cracking, model fails to predict further its development. The justification for such result can be found in the fact that the experimental test was subjected to some modifications, which requires a more detailed post-diction. The initial setting of the sequence of earthquake excitations proposed in the experimental program is presented in Table 4. The same sequence was applied in the numerical modeling. Prediction of base shear forces has also some deflections compared to [19]. Accumulated forces at the end of the second stage (50%) with peak ground accelerations (PGA) of 0.313/0.438g, are 210.29 kN for X direction and 196.43 kN for Y direction, respectively. Gagliardo et al. [19] reported that the first damage could occur between 0.208 and 0.593g, which suits the proposed FE model in present study.

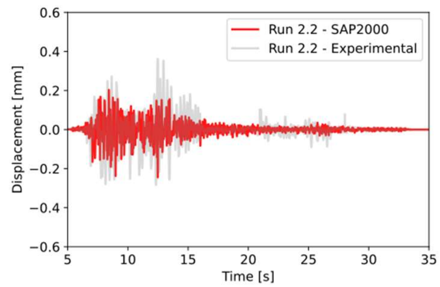
It should be noted that in the first cracking phase (Run 2.1), Unit 1 has been affected and experienced minor in-plane damage in the walls next to Unit 2. This kind of behavior can also reveal the cause of the obtained slightly larger displacements of the top of Unit 1 compared to Unit 2 taking into account the difference in heights, as presented in Figure 9. Under bidirectional excitations that followed, damages were spreaded and also fracture mechanism formed. Gagliardo et al. [19] have also noted the failure of Unit 1, but with the damage consisted of out-of-plane overturning. After the first step, proposed model



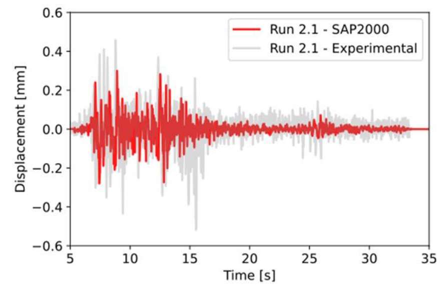
(a) Rd3



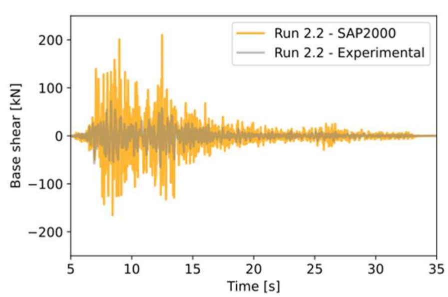
(b) Rd2



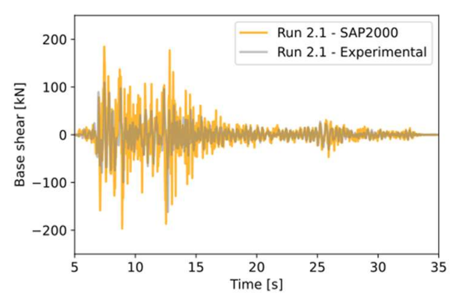
(c) Rd6



(d) Rd5



(e) Base shear X



(f) Base shear Y

Figure 9: Comparison of numerical and experimental results of structural response: (a)-(d) Displacements (scale factor 50%, runs 2.1 and 2.2), (e)-(f) Base shear forces (scale factor 50%, runs 2.1 and 2.2)

wasn't able to accurately capture further collapse mechanism, which requires more refined models, and additional a-posteriori predictions.

8. Conclusion

This work provides predictive results of the seismic behavior of two adjacent interacting masonry units. The finite element method was employed for modeling, using SAP2000 software. Blind predictions performed by several groups of authors were performed under the auspices of the SERA-AIMS project. Obtained outcomes were compared with experimental tests as a part of a brief post-diction study. Output results in this study were given in terms of displacements and base shear forces. SAP2000 performs well in static analyses, but shows certain limitations in the application of nonlinear dynamic analyses, especially after reaching the first damages. A more detailed examination of dynamic analysis using FEM is required, especially when a significant number of cracks is reached. It can be concluded from the numerical results that the first cracking pattern is well-recognized.

Simultaneously, the model underestimates displacements while overestimating the stiffness of the structure, which is reflected in higher base shear values, especially in the case of a 50% scaling factor and beyond. A possible reason lies in the employment of nonlinear layered shell elements for FE modeling, partial neglect of non-linear effects, or is related to the input of material properties, which may be reasons for uncertainty predictions.

Apart from the fact that FEM is a relatively fast method compared to others, present models show certain discrepancies in prediction, which is why more detailed analyses of the behavior of masonry structures are necessary. The adopted methodology has a significant drawback since it cannot depict damage patterns to complement the numerical results that are achieved. Hence, it is recommended to consider the use of more robust models. In particular, attention should be focused on important assumptions in modeling, the influence of residual displacements as well as the rocking effect after reaching the first damage with more careful detection of damage mechanisms. More information and additional materials about masonry structural aggregates and conducted shake table tests covered by this study can be found in [2], [3].

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