Blind predictions of shake table testing of aggregate masonry buildings

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Abstract

In many historical centers in Europe, stone masonry is part of building aggregates, which developed when the layout of the city or village was densified. The analysis of such building aggregates is very challenging and modelling guidelines missing. Advances in the development of analysis methods have been impeded by the lack of experimental data on the seismic response of such aggregates. The SERA project AIMS (Seismic Testing of Adjacent Interacting Masonry Structures) provides such experimental data by testing an aggregate of two buildings under two horizontal components of dynamic excitation. With the aim to advance the modelling of unreinforced masonry aggregates, a blind prediction competition is organized before the experimental campaign. Each group has been provided a complete set of construction drawings, material properties, testing sequence and the list of measurements to be reported. The applied modelling approaches span from equivalent frame models to Finite Element models using shell elements and discrete element models with solid elements. This paper compares the first entries, regarding the modelling approaches, results in terms of base shear, roof displacements, interface openings, and the failure modes.

Keywords: Historical centres; Stone masonry; Adjacent buildings; Shake table test; Blind prediction competition

1. Introduction

During the long periods, various reasons caused the densification of historical centers in Europe. In turn, this caused centers of cities to be characterized by masonry building aggregates. Facades of adjacent buildings often share the structural walls, connected with weakly interlocked stones or by dry joints. Furthermore, since the growth of aggregates was a process spanning through many generations, it is not uncommon for the adjacent buildings to be constructed with different materials, different distributions of openings and floor and roof heights. Post-earthquake observations have shown that the opening of the joint leads to complicated behavior and interaction of the units, often too simplified in the analyses. However, the analysis of masonry building aggregates presents multiple challenges. One of the principal reasons is the lack of experimental data, due to the high cost and the complexity of performing tests on large-scale aggregates. These facts have prompted a joint research program between École Polytechnique Fédérale de Lausanne (EPFL), Switzerland, University of Pavia, Italy, University of California, Berkeley, USA, RWTH Aachen University, Germany and National Laboratory for Civil Engineering, Portugal, named SERA AIMS – Adjacent Interacting Masonry Structures. As a part of this project, a shake table test is to be performed on a half-scale stone masonry building aggregate at the LNEC laboratory in Lisbon, Portugal. Complementary tests on materials and components are to be performed in parallel. As a part of the campaign, blind prediction competition is organized, with multiple participants both from the research community and the industry.

Fig. 1 - Masonry aggregates in Central Italy

One large scale campaign was performed on a half-scale masonry aggregate of similar typology at EUCENTRE in Pavia, Italy [1, 2]. First, the uni-directional shake table test has been performed with the incremental PGA. When significant damage was reached, the specimen was retrofitted and the test was repeated once more with increasing PGA [3]. Valuable insights into the behavior of masonry aggregates and the interaction between the units are provided. However, due to the interlocking of the stones between the units of the aggregate at the interface, no full separation of the units was detected.

Different modeling techniques with different level of details were used to model masonry aggregates. One of the most common approaches is the macro-element approach using Tremuri software [4]. This approach was used in [5] to develop an understanding of the vulnerability of a single structural unit within an aggregate. Numerical and experimental results were compared in [6] to validate the macro-element approach in modeling masonry aggregates. The comparison was performed in terms of hysteresis, lateral displacement and pushover and backbone curves. Advanced numerical analysis results have been used to calibrate a simple non-linear methodology in [7]. Simplified pushover curves of both single structural units and building aggregates were presented and discussed. Analyses results were compared with the Italian Guidelines on Cultural Heritage and correction factors proposed. Simplified assessment procedure has been proposed for assessing a large scale seismic vulnerability of stone masonry building aggregates in [8]. The methodology was inspired by the well-known vulnerability form for the masonry buildings and builds upon it by integrating additional five parameters. These added parameters take into account conditions among adjacent units of an aggregate. Both numerical models and theoretical approaches were used to evaluate the case studies in Central Italy [9, 10].

Stone masonry aggregate has been modeled as one unit and as separate units with non-linear boundary conditions in [11]. The seismic behavior of both the intermediate and boundary units was evaluated and compared.

2. Test unit overview

The test specimen is a half-scale prototype of a masonry aggregate, consisting of two units with the 3D model shown in Fig. 2. Unit 2 consists of two floors and a total height of 3.15 m. Unit 1 consists of one floor with a height of 2.2 m. Unit 2 has a rectangular shape with four walls and the dimensions 2.5 x 2.5 m^2. Unit 1 has an u-shape with three walls and dimensions 2.5 x 2.45 m^2. The basic dimensions of the facades can be seen in Fig. 3. Unit 1 wall thickness is 30 cm and Unit 2 wall thickness is 35 cm and 25 cm of the first and the second floor, respectively. Spandrels under the openings have the thickness decreased to 15 cm.

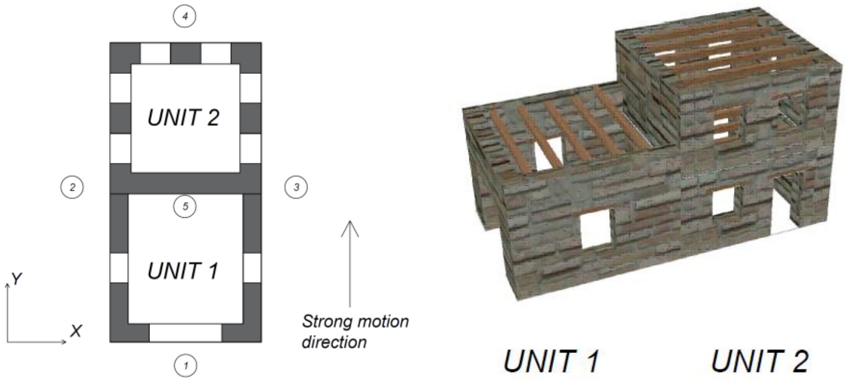

Fig. 2 – General unit orientation

The payload (43 ton) of the shake table at LNEC laboratory in Lisbon, Portugal, imposed constructing the half-scale prototype. It has been common in the past experimental campaigns to use reduced scale specimens to research the seismic response of unreinforced masonry buildings. The most commonly used similitude relationships are Cauchy's or Cauchy-Froude's. However, using Cauchy's relationship would require scaling of gravity acceleration which is not feasible. Using Cauchy-Froude's relationship would require increasing the density of a material, which would increase the total specimen weight. With specimen weight being the critical shake-table limitation, this was not a feasible option. Therefore, the set of scale factors used is the same as the one used in [1]–[3] and listed in Table 1.

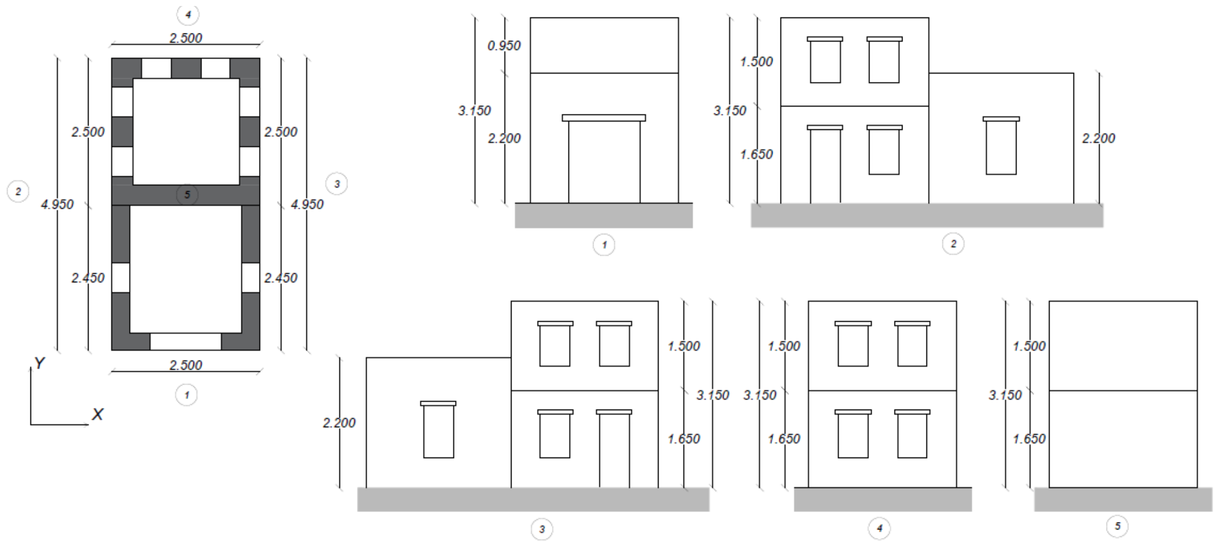

Fig. 3 - Elevation of the facades

Table 1 - Scaling factors for the chosen similitude relationship [2]

Parameter		Scaling factor
Geometric parameters	Length	λ
	Area	λ^2
	Volume	λ^3
	Moment of inertia	λ^4
Dynamic parameters	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
Material parameters	Density	1
	Stress	λ
	Strain	1
	Young's modulus	λ
	Poisson's coefficient	1
	Shear modulus	λ
	Strength	λ
	Cohesion	λ

The material for the construction of the stone masonry walls is chosen to be as similar as possible to the one used for a shake table test conducted at the EUCENTRE [1]–[3]. The mortar is a commercial hydraulic lime mortar mix, with added EPS spheres in volumetric proportions 2:3 of EPS spheres to mixed mortar. EPS spheres lower the stiffness and strength of the mortar to satisfy the similitude relationships. Walls are constructed as double-leaf stone masonry, with no interlocking of the leaves except at the corners and next to the openings. Irregular broken stone pieces are used to fill the voids between the leaves. In this way, it is ensured that the results of the two tests are easily compared. The shake table test conducted at the Eucentre was also conducted at half-scale. Material properties reported were derived in [1] as part of the experimental campaign conducted before the shake table test performed at the Eucentre. At the same time, reported material properties are used for a blind prediction competition before the actual properties are reported.

Fig. 4 - Masonry typology

Two units of the aggregate are connected by a dry joint. Unit 2 is the first unit to be constructed. After the construction of Unit 2, the contact area will be made smooth by mortar to ensure no interlocking between the units. Then, Unit 1 will be constructed, ensuring that the contact between the units is essentially a mortar-mortar interface. This type of connection, paired with different modal properties of two units, facilitates separation and out-of-phase behavior during the test.

Floor diaphragms are composed of 8x16 cm wooden beams and 2 cm thick wooden planks orientated orthogonally to beams. Diaphragms have different orientations, with Unit 1 beams spanning in the x-direction and Unit 2 floors in the y-direction. When subjected to bi-directional seismic record, different orientations will facilitate different behavior of two units. To prevent early out-of-plane failure, PVC tubes are placed into walls, alongside and in direction of each beam. Steel angles are designed to anchor beams into walls. However, being placed just as a precaution for later phases of the test, it is not modeled in numerical analyses.

The total mass of Unit 1 is 7434 kg, and of Unit 2, including the extra weight, 16272 kg. Extra mass is placed evenly distributed on the two floors of Unit 2 to further increase the modal periods of Unit 2, and distinguish it from stiffer Unit 1. Additional mass is applied by steel plates, firmly attached to the floor to prevent sliding. Masses of all test specimen elements can be seen in Table 2.

Table 2 - Specimen mass

Specimen mass	
Walls of Unit 1	7270 kg
Floors of Unit 1	164 kg
Total Unit 1	**7434 kg**
Walls of Unit 2	12937 kg
Floors of Unit 2	335 kg
Additional masses of Unit 2	3000 kg
Total Unit 2	**16272 kg**
Steel foundation	7000 kg
Concrete slab on top of steel foundation	11000 kg
TOTAL	**41706 kg**

3. Loading sequence

The specimen will be tested using both horizontal components of the Albatros station records from 1979. Montenegro earthquake [12]. Peak ground acceleration limit of the LNEC shake table is, for the given specimen weight, 0.875 g in the y-direction and 0.625 g in the x-direction. The test sequence comprises four steps, with the ground motion applied at 25%, 50%, 75% and 100% of the shake table limit. In each step, first uni-directional excitation in the y-direction is applied, followed by uni-directional excitation in the x-direction and finally, the bi-directional excitation in both x- and y-direction. Timestep is scaled with $\lambda^{1/2}$, as defined in Table 1.

Table 3 - Loading sequence

Level of shaking	Substep I	Substep II	Substep III
25% shake table capacity PGA 0.219/0.156g y/x-dir	y-direction (EQ-Y-25%)	x-direction (EQ-X-25%)	Both directions (EQ-XY-25%)
50% shake table capacity PGA 0.438/0.313g y/x-dir	y-direction (EQ-Y-50%)	x-direction (EQ-X-50%)	Both directions (EQ-XY-50%)
75% shake table capacity PGA 0.656/0.469g y/x-dir	y-direction (EQ-Y-75%)	x-direction (EQ-X-75%)	Both directions (EQ-XY-75%)
100% shake table capacity PGA 0.875/0.625g y/x-dir	y-direction (EQ-Y-100%)	x-direction (EQ-X-100%)	Both directions (EQ-XY-100%)

4. Numerical modelling

All the participants have been provided a set of material parameters, construction drawings, loading sequence, and all particular construction details. They were asked to describe their modelling approach, report complete time-histories of requested variables, damage description, and to describe the failure mechanism. First entries to the Blind prediction competition, received at the time of writing this article, are reported in this chapter, together with material parameters.

4.1 Set of material parameters

In order to ensure the comparability of results, the material parameters will be as similar as possible to the ones reported for the shake table test performed in Eucentre, Italy in [1]–[3]. The shake table test at the Eucentre was also conducted at half-scale. Material properties reported here were reported as a part of the experimental campaign conducted before the shake table test. Shear-compression tests on masonry wallets and material tests for mortar stones and masonry were performed and reported. The results of the material tests are summarized in Table 4-6.

Table 4 - Mortar properties [1]

Mortar properties	Average	C.o.V. [%]
Mortar compressive strength, f_m [MPa]	1.75	28
Mortar tensile strength, $f_{m,t}$ [MPa]	0.60	23
Mortar Young's modulus, E_m [MPa]	243	35

Table 5 - Stone properties [1]

Stone properties	Average	C.o.V. [%]
Credaro Berrettino stone compressive strength, f_b [MPa]	144	-
Credaro Berrettino stone tensile strength, f_{bt} [MPa]	19	-

Table 6 - Masonry properties [1]

Masonry properties	Average	C.o.V. [%]
Density of masonry, ρ [kg/m^3]	1980	5
Masonry compressive strength, f [MPa]	1.30	2.6
Masonry tensile strength, f_t [MPa]	0.17	7.3
Masonry cohesion, f_{v0} [MPa]	0.233	7.3
Masonry Poisson's modulus, ν [-]	0.14	56
Masonry Young's modulus in compression, E [MPa]	3462	12
Masonry shear modulus, G [MPa]	1524†	17
Masonry shear modulus, G [MPa]	1898‡	58

†*from vertical compression tests*
‡*from diagonal compression tests*

4.2 Modelling approaches

A first approach is an equivalent frame model approach using the OpenSees framework [13] and the newly developed advanced macro-element [14]. The macro-element is a three-node, three-dimensional element capturing the main features of the in-plane and out-of-plane dynamic behavior of masonry walls, including the explicit coupling of the shear and flexural response. Floors are modeled with elastic membrane elements, which is a common practice in equivalent frame models. The model provides only the membrane stiffness components, resulting in a zero-bending stiffness. Wall to wall connections are modeled with a one-dimensional non-linear interface, which provides linear elastic behavior in compression, with no crushing, and a finite tensile strength paired with exponential softening law. Floor to wall connections are modeled with a frictional interface, ruled by the vertical load acting on a floor node and a friction coefficient. The material model can model the pounding of the beam when the slip is in the negative direction. Unit to unit connection of the aggregate is modeled with an n-dimensional zero-length element and the material model, that captures linear elastic behavior in the axial direction (perpendicular to the interface between the units) and a finite tensile strength paired with exponential softening law. In the perpendicular plane, the cohesive-frictional behavior is based on the axial load, friction coefficient and exponential damage law of cohesion.

The second set of results presented here is obtained using a novel Macro-Distinct Element Model (M-DEM, see [15]), namely a hybrid Finite-Distinct macro-element approach aimed at combining the efficiency of simplified modeling strategies with the possibility of accounting explicitly for the interaction among in-plane and out-of-plane-loaded URM components. In the M-DEM framework, each URM member is idealized an assembly of six deformable Finite Element macro-blocks, characterized by an internal tetrahedral mesh, and connected to each other by means of nonlinear spring layers. The interface is modelled as frictional (zero cohesion and zero tensile strength).

4.3 Comparison of results

The response of the OpenSees model is governed by the in-plane rocking response of Unit 2 and the combined in-plane and out-of-plane response of Unit 1. Cracking of spandrels initiates already at 25% load step. Starting from EQ-Y-50% flexural damage spreads through units and the first significant separation of the units at the interface occurs. Starting at EQ-X-75% significant rocking of Unit 2 first floor occurs, potentially leading to the failure depending on the displacement capacity of the piers. The last substep of 75% load step (EQ-XY-75%) causes widespread flexural damage in the piers and spandrels of both units and out of plane displacement of lateral walls of Unit 1. The latter failure mode is highly dependent on the floor-to-wall non-linear connection, floor model and its ability to restrain out-of-plane displacement.

The response predicted by the M-DEM model is mainly governed by the rocking response of Unit 2, which exhibited a first-floor soft-story mechanism, with most of the damage concentrated at the top/bottom of the piers and in correspondence of the openings. Starting from EQ-Y-50% diagonal shear cracks, as well as minor flexural damage, induced by the dynamic interaction among the two sub-structures, are also detected on the squat piers of Unit 1. The top portion of the party wall of Unit 1 suffered some two-way bending out-of-plane failure mechanisms, as depicted in Fig 5b (where only flexural/shear cracks through joints are represented). Near-collapse conditions were reached at the end of EQ-X-100%, although a significant increase in terms of 2nd floor displacements (Unit 2) was detected from EQ-Y-75%.

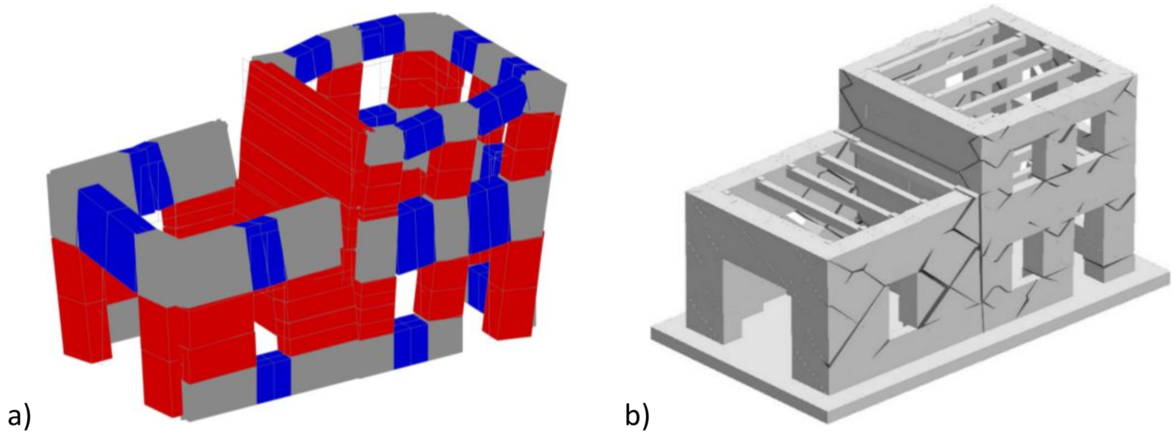

Fig 5. - Final deformation shapes: a) OpenSees model; b) M-DEM model

Base shear, roof displacement of Unit 2 and opening of the interface between the units obtained using both modelling approaches have been compared in x-direction and y-direction in Fig. 6 and Fig. 7. For the brevity, only results from the last substep (bi-directional excitation) of every load step are reported. In the case of the Equivalent Frame OpenSees model, it is important to note that using realistic drift limits, the reaching of the ultimate limit state is expected at the load step EQ-XY-75%. However, to compare the results, the analysis has been run without imposed drift limits. With reference to the M-DEM, analyses have been run for the part of the initial record considered significant, starting at 3.6577 s and ending at 16.67 s of each record, originally 39.17 s long.

The two models show different performance in terms of roof displacements in both directions, with the M-DEM model showing larger values. The opening of the interface in the x-direction shows more similar results between the models, with the difference of the M-DEM model showing residual deformations of the interface, unlike the OpenSees model. The M-DEM model predicts a larger opening at the interface in y-direction already at EQ-XY-50% step, whereas the OpenSees model predicts a larger opening at the final step, with the same difference regarding the residual deformations as the x-direction.

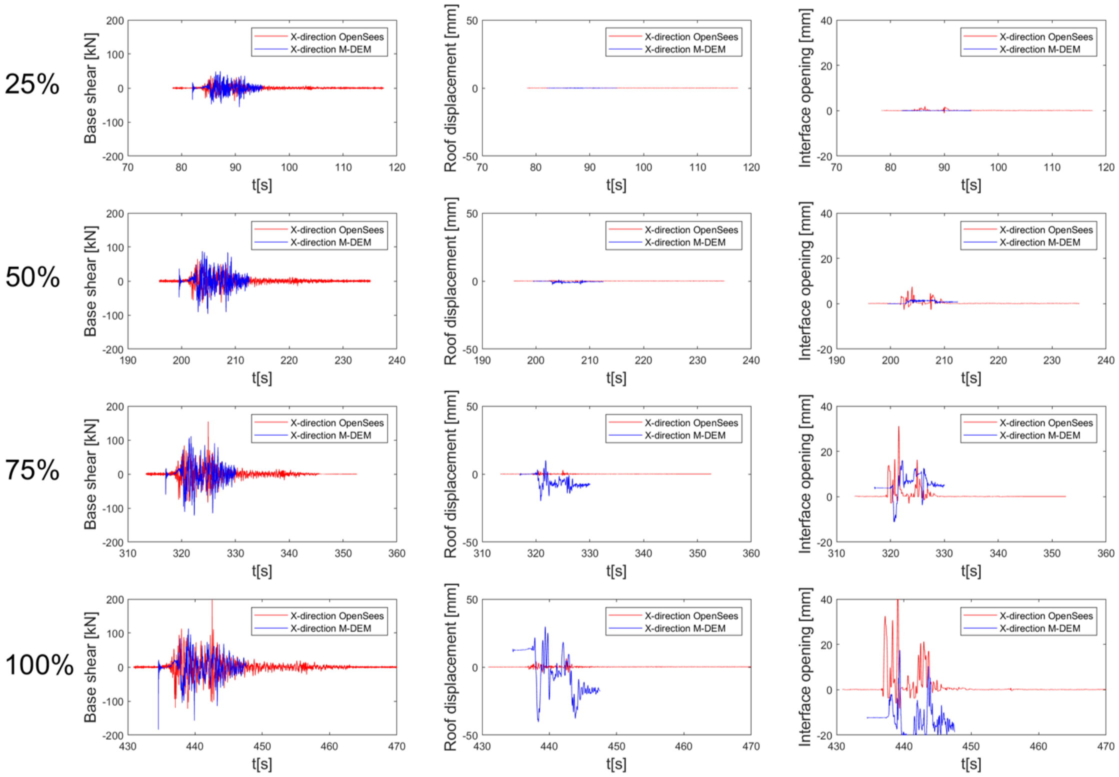

Fig. 6 - Comparison of results in the x-direction

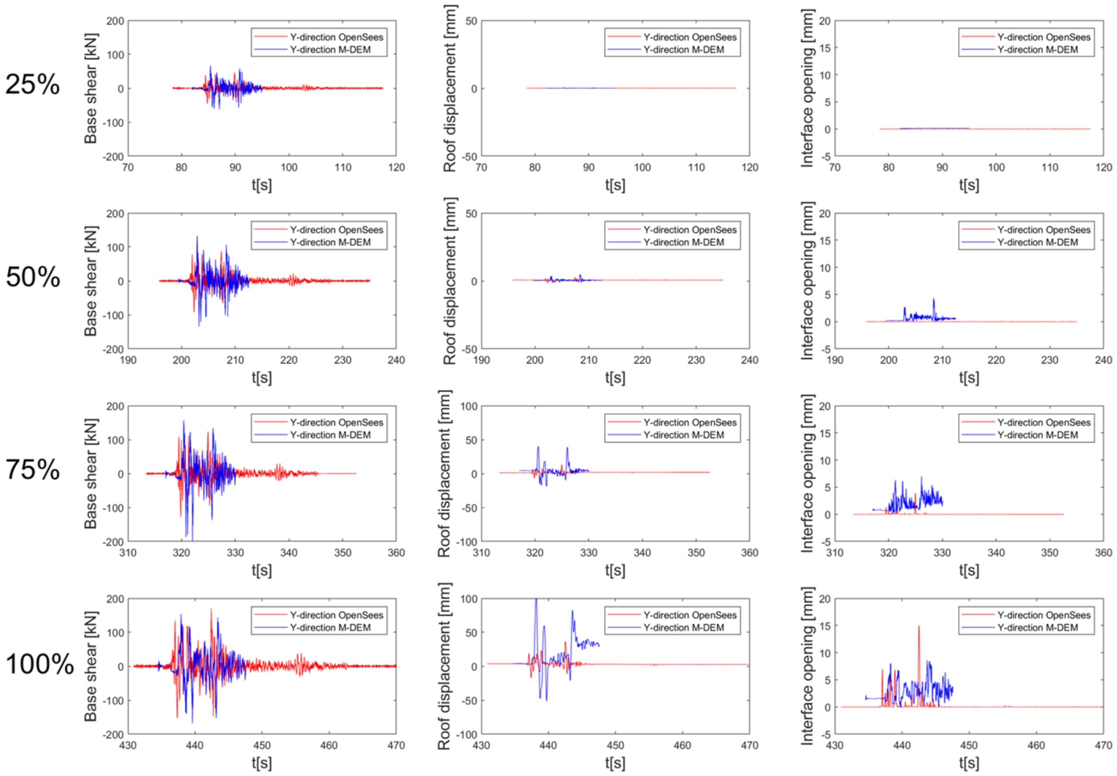

Fig. 7 - Comparison of results in the y-direction

5. Conclusions

Prior to the shake table test on a stone masonry aggregate, the blind prediction has been organized with participants from academia and industry. All the participants have been provided with the complete set of construction drawings, expected material properties, earthquake inputs, and testing sequence. The first set of results is analyzed and compared. Comparison of the results between different modeling approaches points to the fact that different approaches can lead to significantly different results in terms of roof displacement-base shear curves, interface opening and failure modes. Apart from inherent masonry heterogeneity, every modeling approach carries its intrinsic uncertainties whose influence can be evaluated according to the experimental results. As a part of a blind prediction competition, many more results, coming from different modeling approaches are expected. After the actual shake table test and comparison of numerical and experimental results, all the numerical approaches will be evaluated and conclusions and possible modeling recommendations drawn.

6. Acknowledgments

The project leading to this application has received funding from the European Union's Horizon 2020 research and innovation programme under grant agreement No 730900.

7. References

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