Proceedings of the 9th International Conference on Structural Dynamics, EURODYN 2014
Porto, Portugal, 30 June - 2 July 2014
A. Cunha, E. Caetano, P. Ribeiro, G. Müller (eds.)
ISSN: 2311-9020; ISBN: 978-972-752-165-4

Seismic performances of modern unreinforced thermal insulation clay blocks masonry houses

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ABSTRACT: In the scope of the transnational access activities of the European research project SERIES, the Laboratório Nacional de Engenharia Civil (LNEC) has provided access to its 3D shaking table to the international construction company Wienerberger AG and to a group of European experts, in order to perform full-scale seismic tests on an industrial solution for buildings using a modern unreinforced thermal insulation clay block masonry structure. Such solution represents a very common construction method in Central Europe but, although some cyclic shear test results are available, its effective dynamic response under seismic events still requires experimental validation. For this purpose, two full-scale mock-ups with different geometries were tested on the 3-D shaking table using a series of seismic records with increasing intensity. The first mock-up is plan-regular, while the second one is designed so as to trigger some torsional effects. This paper summarizes the most relevant experimental results regarding the structural response of the specimens, e.g., the dynamic response evolution, the collapse mechanism identified and the maximum drift values measured. It also focuses on two different easy modelling strategies, i.e. simple model according to a combined Eurocode 6/Eurocode 8 approach and equivalent frame approach. It finally compares the predictions obtained from these two methods with the experimental results.

KEY WORDS: Modern unreinforced masonry houses, thermal insulation clay blocks, Full-scale tests, 3-D shaking table.

1 INTRODUCTION

The Art of building and designing has changed during the last decades, especially for the private dwellings. Besides the structural and mechanical considerations, additional requirements are now necessary and mandatory in terms of energy consumption. In particular, the thermal insulation of buildings has become more and more important to fulfil the standards in terms of heating/cooling energy demand.

As traditional constructive materials and methods weren't appropriate to this purpose, masonry producers have developed new units allowing keeping a similar way of building. For Wienerberger, it results in a new generation (see Figure 1) of high thermal insulating clay block allowing the construction of unreinforced monolithic clay masonry walls as traditionally in countries like Austria, Hungary, etc., situated in Central Europe.

Figure 1. High thermal insulating clay block

For a few years and besides the energy consumption aspects, unreinforced masonry has now to cope with the seismic risk. According to the current seismic design standards [1], Central Europe is located in a low-to-moderate seismic area and a specific consideration for earthquake events is required. In this perspective, several research works have been performed in the past twenty years with the book "Earthquake-Resistance Design of Masonry Buildings" by Miha Tomazevic as first main reference [2], but these ones were dedicated to more traditional types of masonry [3] or consisted in cyclic [4, 5] or pseudo-dynamic [6] tests, leading to a questionable modelling of the seismic action. Other test campaigns were conducted on shaking table, but were focused on specific structural elements or sub-structures [7 – 9].

The characterization and the validation of the new masonry clay units and the global behaviour of structures using such units need therefore to be validated by carrying out appropriate and specific experimental tests. To contribute to these issues, shaking table tests on full scale modern unreinforced thermal insulation clay blocks masonry houses have been performed at the Laboratório Nacional de Engenharia Civil (LNEC) in Lisbon in the framework of the European project SERIES. The two tested mock-ups have been designed as 2-storey structures with different in-plan arrangements in order to compare the seismic response of structures with or without plan regularity.

This paper describes the tested mock-ups and experimental procedure. It also summarizes the most relevant experimental observations regarding the structural response. Details of the collapse mechanisms, maximum drift values and dynamic response evolution are provided. A comparison of test measurements with two different modelling strategies, namely a simple model according to a combined Eurocode 6/Eurocode 8 approach and an equivalent frame model, is also performed.

2 DESCRIPTION OF THE MOCK-UPS AND TESTS

Two full-scale mock-ups have been built at LNEC. These are 2-storey buildings, with a single room delimited by 0.2 m thick walls. The global dimensions of the floors are 3.7 m x 4.2 m. The first mock-up is regular in plan, while the second includes significant stiffness irregularities (see Figure 2).

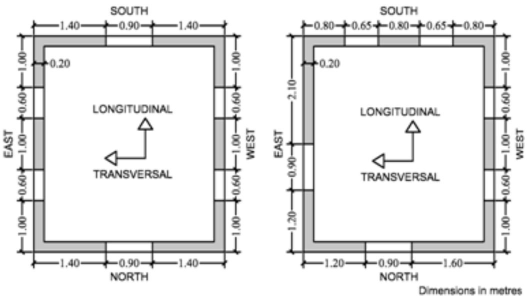

Figure 2. Plan view of the mock-ups

The floors are 2.5 m high and are separated by a prefabricated RC slab of 0.2 m. Another 0.2 m high prefabricated RC slab lies on the top of each mock-up, with plan dimensions of 4.4 m x 4.9 m (see Figure 2), for a total height of 5.4 m. The openings heights are 1.15 m or 1.9 m high, respectively for the windows and the doors. Additional masses are fixed on the first floor slab to emulate usual live load in buildings, leading to a total weight of 31.7 tons for each mock-up. During the tests, the slabs are held by steel suspension loose cables connected to the bridge crane, for safety reasons in case of global collapse.

Figure 3. Elevation view of the mock-ups

In Figure 3, the mock-ups are resting on a steel foundation, specially designed to limit deflections at mid-span (< L/1000) during the transport of the mock-up onto the shaking table. Steel ties have been also used to pre-compressed the masonry elements and mitigate the cracking during the set-up.

Details on the materials properties used for the mocks-up are given in [10]. For further use, the in-plan regular (symmetric) mock-up will be called “Mock-up A” and the other (asymmetric) will be “Mock-up B”.

2.1 Instrumentation layout

The instrumentation used during the tests includes the internal table instrumentation. In addition to this, 26 accelerometers are fixed on the mock-ups at different places and 18 LVDTs measure the relative displacements between specifically chosen elements. Four bi-axial absolute displacement sensors also record the displacements of both slabs. Scheme of the instrumentation layout are given in [10]

2.2 Testing procedure

The mocks-up are submitted to two different types of tests. The dynamic properties are first evaluated on the base of shaking table tests using a low amplitude square-wave displacement time-history signal, creating an impulsive excitation in the mock-up. Such a test is performed in both horizontal directions before each seismic test stages constituting the second type of tests. A total of 8 stages are defined, with alternated uniaxial and biaxial stages. It means that the odd test stages are performed along the two horizontal directions successively and that the even ones have components in the two directions simultaneously. The seismic stages are detailed in Table 1 for the mock-up A and Table 2 for the mock-up B.

Table 1. Mock-up A – Seismic stages.

Stage	NS (Long. Dir.) [m/s²]	EW (Trans. Dir.) [m/s²]	No of shakes [-]
01T	0.096	0.433	6
01L	0.491	0.110	5
02	1.013	0.913	5
03T	0.280	1.388	5
03L	1.419	0.616	5
04	3.734	2.143	5
05T	0.486	2.857	5
05L	2.526	0.844	6
06	3.099	2.684	6
07T	0.646	3.068	7
07L	3.541	0.830	5
08	3.718	5.362	2

Table 2. Mock-up B – Seismic stages.

Stage	NS (Long. Dir.) [m/s²]	EW (Trans. Dir.) [m/s²]	No of shakes [-]
01T	0.092	0.428	5
01L	0.636	0.084	5
02	1.000	0.949	6
03T	0.141	1.249	5
03L	1.505	0.620	4
04	2.016	1.882	5
05T	0.664	2.616	4
05L	3.193	1.148	5
06	3.918	2.105	6
07T	1.008	4.415	4
07L	3.639	1.141	3
08	3.685	4.583	2

The values given in Tables 1 and 2 are the measured PGAs. Contrary to the theoretical input, one can observed that the odd stages have a component in both directions. This comes from some difficulties in perfectly controlling the shaking table and in practically imposing the targeted theoretical input. The last column of Table 1 and 2 gives the number of shakes performed in the corresponding stage. Several successive shakes are necessary to minimize the differences between the target and effective motions of the table. This iterative procedure allows reaching progressively the target displacement and to avoiding undesirable and uncontrolled motions, which could lead to the mock-up collapse.

3 EXPERIMENTAL TESTS RESULTS

The first interesting outcomes are those coming from direct observations of the global behaviour of the mocks-up during the tests and of the most visible collapse mechanisms, without referring to the recorded measurements.

First of all, a reference can be made to parallel experimental campaigns within the same research program. Indeed, shaking table tests on smaller scale unreinforced masonry sub-structures using a similar material type have been performed in the framework of the same SERIES project [7, 8]. A main outcome of these tests is the observation of a global rocking behaviour. In the present experimental tests, such behaviour also occurred, although quite limited. Some uplifts of walls corners were observed rather than a pure rocking behaviour, certainly due to the different boundary conditions, constraining the motion of the concerned walls.

Collapse of Mock-up A occurred on the second floor, during the last seismic stage. Figure 4 illustrates the failure mode, showing the high damage level in the Northern façade comprising a door opening. The failure mechanisms are a combination of shear failure, sliding and local crushing.

Figure 4. Collapse of the Mock-up A

On the other hand, collapse of Mock-up B occurred in the Northern façade too, but at the first floor (Figure 5). For this mock-up, the failed façade comprised window openings. The main mechanism is the shear failure of the wall standing between those two openings (Figure 5, left). Other secondary collapse mechanisms were also observed, like in particular some local crushing of units, as pictured in Figure 6. In this case, the definite acceleration level leading to collapse is less easy to define since the damage developed progressively through successive shakes.

Figure 5. Collapse of the Mock-up B

Figure 6. Local crushing of the Mock-up B

The total collapse of Mock-up B has been avoided because of the favourable situation of the slab being supported on four sides, in such a way that it remained supported on three sides even after the failure of the Northern façade. It can therefore be concluded that this bearing system is safer through its redundancy.

3.1 *Characterization tests results*

The characterization tests provide results in terms of natural frequencies and associated vibration modes, on the base of the accelerometric measurements. Since these were performed before each seismic stage, they allow the study of the progressive damage of the mocks-up.

The first natural frequency of Mocks-up A and B are tabulated in Table 3. One has to pay attention that the given frequencies are the ones of a global system composed of the mock-up itself and the shaking table where it rests. Therefore, the interaction between the mock-up and the table has to be taken into account.

The analysis of the values in Table 3 shows a general decrease of the frequencies, translating a deterioration of the mock-up stiffness. Further analysis is however required to investigate the absolute values of the measured natural frequencies. For both mocks-up, the decrease is more important in the transverse direction (EW-axis). This statement is in agreement with the observed general evolution of the damage during the tests.

The comparison of the frequencies of the two mock-ups gives similar values in the longitudinal and transverse directions for the Mock-up B, while they are clearly different for the Mock-up A (Figure 7). This can be explained based on simple geometrical considerations.

Table 3. First Frequency in [Hz].

Stage	Mock-up A		Mock-up B	
	Longit.	Transv.	Longit.	Transv.
01T	-	6.9	5.4	5.8
01L	5.7	6.9	5.4	5.7
02	5.5	6.7	5.4	5.8
03T	5.5	6.7	5.3	5.8
03L	5.2	6.3	5.3	5.8
04	5.2	6.1	5.2	5.8
05T	5.2	5.9	5.2	5.2
05L	5.2	5.9	5.2	5.2
06	5.2	6.0	5.2	5.2
07T	5.0	5.5	5.1	4.3
07L	5.1	5.2	5.0	4.1
08	4.8	5.3	4.9	4.1

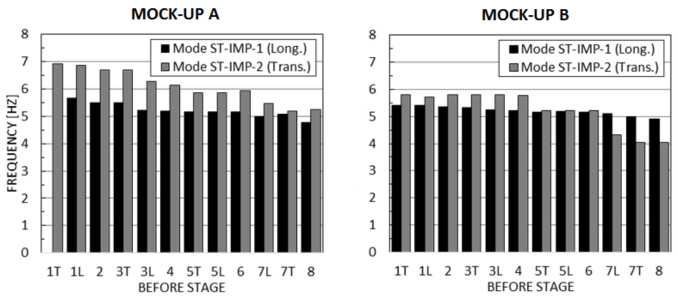

Figure 7. Evolution of the first frequency

3.2 Seismic test results

Although the exploitation of the measurements recorded during the seismic stages still requires additional processing, some results are available to date and deal with the inter-storey drift measurements and with the evolution of the maximum acceleration measured on the mock-ups as a function of the maximum table acceleration.

The maximum inter-storey drifts are plotted in Figure 8 for Mock-up A (2nd storey) and Figure 9 for Mock-up B (1st storey). The x-axis is the ratio of the relative displacement between the floors to the floor height.

Results of Figures 8 & 9 are in good agreement with the general observations and shows out that the values of the measured maximum drifts are in line with the standards recommendations. Maximum drift of the 2nd storey is higher than the one of the 1st storey for Mock-up A, and vice-versa for Mock-up B, corresponding to the respective observed failure modes. Concerning the standards recommendations, a value of 0.4% is suggested for shear failure. This value seems to be respected in the case of Mock-up A (the maximum drift is about 0.3% in Stage 07 and about 0.8% in Stage 08, the collapse stage), while it is probably underestimated for the Mock-up B, although depending on the definition of the collapsed state.

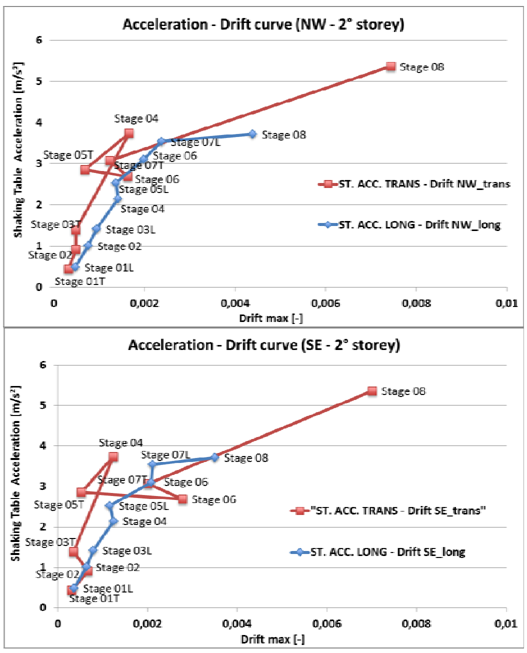

Figure 8. Maximum inter-storey drift – Mock-up A

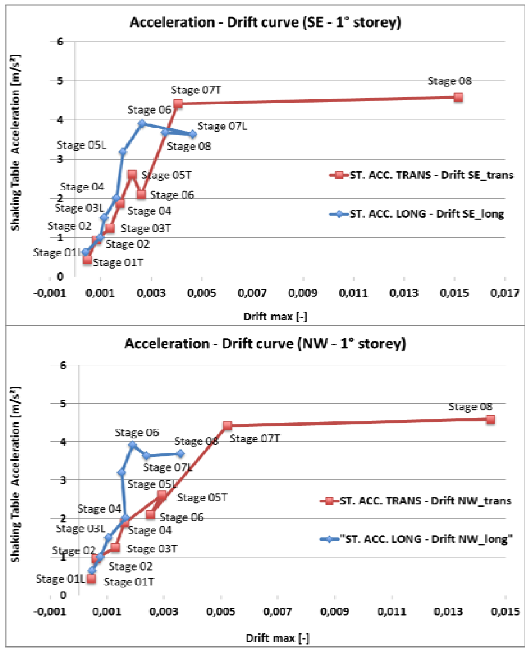

Figure 9. Maximum inter-storey drift – Mock-up B

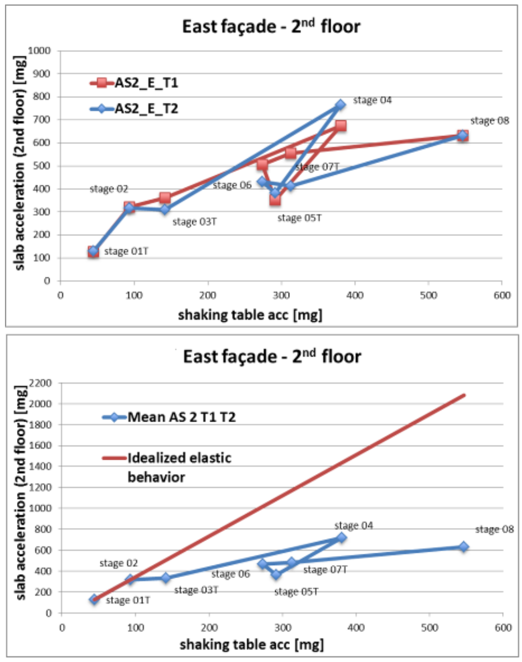

Figure 10. Evolution of transverse acceleration (Mock-up A)

Regarding the evolution of the measured accelerations at the slab levels vs. the measured table acceleration, the degrading slope of the curve translates the damaging effects. The mock-up behaviour and response effectively remain elastic as long as the slope remains constant, meaning a proportional increase of the different accelerations. If the slope decreases, this may then be due to a loss of stiffness and, thus, to lower natural frequencies and/or to an increase of the damping ratio, leading hence to lower spectral acceleration. From the ratio between theoretical ideal elastic accelerations extrapolated from low intensities to the measured accelerations at failure, behaviour factors q can be estimated (see, e.g. [10]).

An example of this exploitation is illustrated in Figure 10 for Mock-up A, based on accelerometers located on the East façade (transverse acceleration). Top part of Figure 10 shows the accelerations measured by two accelerometers on the slabs, while the mean values and the corresponding idealized elastic behaviour are plotted in the bottom part. The corresponding behaviour factor q is equal to 3.3. This corresponds to the highest value derived from the present experimental campaign, while others, ranging from 2.0 to 2.5, are closer to the recommended values for unreinforced masonry in current standards, namely from 1.5 and 2.5.

4 MODELLING STRATEGIES

This section aims at comparing the experimental reality to current standards modelling strategies. An additional comparison with shell FE modelling is in progress and will be presented in a later contribution.

The modelling strategies aim at determining the maximum ground acceleration that the mock-ups can sustain. The main outcome is therefore the maximum theoretical acceleration leading to failure. This latter can then be compared to the experimental acceleration that effectively led to collapse. Another point of comparison is the natural frequencies, which can also be assessed according to the standards recommendations.

4.1 Simple model according to EC 6/8 approach

The simple model according to EC 6/8 approach does not consider the entire building as a whole but as a set of cantilever shear walls. In this approach, the shear walls are assumed to be continuous elements rising from the bottom to the top of the building. The main lack of the method is to neglect the presence and the contribution of the horizontal spanning elements, such as the lintels and slabs as well as the masonry parts located under or above the openings. Moreover, the building is broken down into two perpendicular directions, namely along the direction of the façades, and the analysis is performed by considering the contribution of walls oriented along one direction at a time only. A second lack can be therefore identified, since the contribution of perpendicular walls is neglected, both in terms of stiffness and resistance. The mechanical properties of the walls can be assessed on the base of Eurocode 6 recommendations, with a Eurocode 8 update to take into account the cracking through a reduction of the elastic stiffness. Consequently, the building is studied as a set of shear walls oriented along the same direction and the building strength is obtained as the sum of the resistance of these walls.

The seismic action is modelled by an equivalent horizontal shear triangularly distributed over the height of the building and transformed into point loads applied at the level of the floors. The conversion of the seismic acceleration into horizontal shears is performed according to Eurocode 8 procedure, using the horizontal elastic response spectrum.

Finally, the individual wall strength is calculated with the resistance model proposed by Eurocode 6 for unreinforced masonry elements submitted to horizontal shear. This model consists in evaluating a compressive length, due to the no-tensile strength of masonry, and assumes a linear stress distribution along the wall base (Figure 11).

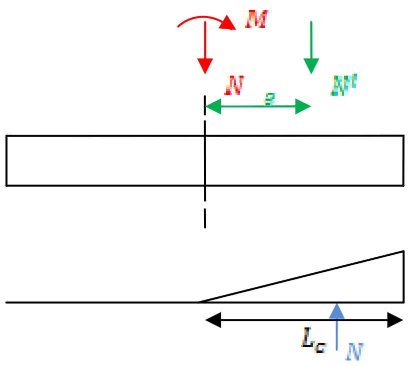

Figure 11. Assessment of the compressive length

$$L_c = \begin{cases} 0 & e \geq L/2 \\ 3\left(\dfrac{L}{2} - e\right) & e \geq L/6 \wedge e \leq L/2 \\ L & e \leq L/6 \end{cases} \tag{1}$$

In Eq. (1), L is the geometrical length of the wall and e is the load eccentricity, defined as the ratio of the bending moment induced by the horizontal shear to the compressive load acting on the wall. The verifications are then carried out to check the shear strength, the overturning and the crushing of units in the wall. An iterative procedure allows the determination of the maximum horizontal shear. This latter is related to a given acceleration according to Eurocode 8, § 4.3.3.2.2, with due consideration of the behaviour factor. This relation requires also the estimation of the first natural period of the structure, which can be compared to the experimentally identified first natural frequency. This approach has been extensively detailed in [11] and led to the results presented in Table 4.

Table 4. Results of the simple model approach

Stage	Mock-up A		Mock-up B	
	Longit.	Transv.	Longit.	Transv.
Acc. [m/s²]	1.0864	1.469	0.9675	0.6302
Freq. [Hz]	5.615	6.468	7.001	5.643

The comparison with the experimental results highlights the under-estimation of the maximum sustainable acceleration obtained by this very simplified procedure. The first natural frequency is however more or less well assessed, especially in the case of Mock-up A. The differences can be explained by the previously identified shortcomings.

4.2 Equivalent frame model

In order to consider the contribution of the horizontally spanning elements, a basic equivalent frame model has been also derived and is detailed in [11]. Such a modelling does not consider any longer the walls separately, but analyses the entire façade as a global frame with the walls behaving as piers and the horizontal elements as equivalent beams. It accounts also for the possible load redistribution among the piers. The principles of the analysis and the verifications remain the same as in the previous simple model but are now based on internal forces in the wall calculated from the frame model. Another difference between the two methodologies deals with the derivation of the assessed maximum horizontal acceleration. To this purpose, the N2-method [12] has been adopted. The ultimate limit state is assumed being reached when the maximum total displacement reaches a specific threshold value defined from recommendations given by [13], depending on the geometry and the compression ratio. The results obtained with the equivalent frame model are tabulated in Table 5.

Table 5. Results of the equivalent frame approach

Stage	Mock-up A		Mock-up B	
	Longit.	Transv.	Longit.	Transv.
Acc. [m/s²]	9.8	5.65	2.09	5.4

The results of the equivalent frame method provide reasonable estimates of the measured experimental accelerations, except for Mock-up A in the longitudinal direction, where the results are highly overestimated. This overestimation is potentially explained by a poor definition of the limit state considered in the theoretical model.

A major difference between the theoretical model and the experimental reality deals with the predicted collapse mechanisms. In the case of Mock-up B, the theoretical model foresees a collapse triggered by one of the external walls, while the real failure was actually observed in the central wall. This difference is suspected to be induced by the non-consideration of the walls perpendicular to the seismic action in the theoretical model, while these walls clearly strengthen the side walls with respect to the central one. As illustrated in Figure 12, the connection between these perpendicular walls appears to be severely damaged at the end of the tests. This collapse mechanism is however not considered in the current standards although potentially critical.

Figure 12. Collapse of the wall connection.

5 CONCLUSIONS

This paper 3-D shaking table tests performed on full-scale unreinforced masonry houses with high insulation clay blocks and summarizes the first experimental results. Two mocks-up were tested. The tests were stopped due to the near-collapse state of both mock-ups. The following preliminary conclusions can be drawn:

- Each mock-up reached a near-collapse state along the transverse direction. The collapse mechanisms were a combination of shear failure, sliding and local crushing.
- A frequency drop is observed for both mocks-up, translating the damages of the mocks-up and is in agreement with the observed collapse mechanisms.
- Based on measurements taken during seismic stages, it is observed that the maximum inter-storey drift values are equal or higher than the ones proposed in Eurocode 8, part 3. These records also allow the calculation of the behaviour factor *q*. The results lead to the conclusions that the recommended values for unreinforced masonry are slightly lower than the values reached by the present experimental mock-ups.

The comparison of the experimental measurements with the modelling strategies used for a preliminary assessment highlights some lacks of the current standards model. In particular, it is shown that a specific consideration must be taken for the horizontal spanning elements and for the contribution of walls perpendicular to the seismic action. Additional failure criterion should be taken into account in the verification to check local collapse such as the connection between perpendicular walls. The present tests are or course expected to be deeper exploited in a near future.

ACKNOWLEDGMENTS

The research leading to these results has received funding from the European Union Seventh Framework Programme (FP7/2007-2013) under grant agreement n° 227887, SERIES. The authors also acknowledge the contribution of Marco Mastinu for the work carried out during his Master Thesis.

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