The 9th European Congress on Computational Methods in Applied Sciences and Engineering
ECCOMAS Congress 2024
3 – 7 June 2024, Lisboa, Portugal

PUSHOVER ANALYSIS OF MASONRY DOUBLE CURVATURE STRUCTURES SUBJECTED TO HORIZONTAL LOADS: THE ANIME SANTE DOME

ALESSANDRO GANDOLFI[1*], GABRIELE MILANI[2]

[1*] Department of Architecture, Built Environment and Construction Engineering
Politecnico di Milano, 20133 Milano, Italy
e-mail: alessandro.gandolfi@polimi.it
ORCID: https://orcid.org/0009-0007-1100-0865

[2] Department of Architecture, Built Environment and Construction Engineering
Politecnico di Milano, 20133 Milano, Italy
e-mail: gabriele.milani@polimi.it
ORCID: https://orcid.org/0000-0001-5462-3420

Keywords: Pushover, Masonry, Double Curvature Structures, Anime Sante Dome, Rigid Body plus Interface Method, Macroscopic Modelling

Summary. The present paper deals with the seismic analysis of masonry domes in an equivalent static fashion (pushover). The dome is discretised by means of a heterogeneous FE mesh where blocks with homogeneous properties are assumed as elastic, and joints are simply modelled through three-dimensional elements exhibiting either an elasto-plastic or an elasto-damaging behaviour [1]. The present modelling approach, applied to the case study of the Anime Sante Dome (Santa Maria del Suffragio Church in L'Aquila), considers a small portion of the church, namely the dome without the drum. The objective is to produce a benchmark for the development of a code for the automatic generation of a non-linear interface constituted by trusses and shear panels. The possible orthotropy, typically induced by a running bond disposition of the blocks, can be considered either meshing the dome with a truly heterogeneous discretization or adopting a heuristic homogenization where different mechanical properties of the meridian and annular interfaces are assumed, and the rest is linear elastic. The aim is to bring both methods in the non-linear static field, employing them for a fast seismic vulnerability assessment of the architectural heritage. The general objective is to develop a reliable tool for fast structural analyses, design, retrofitting or forensic engineering after earthquake events. The case study dome considered to preliminary benchmark the procedure on previous analyses [2], [3], collapsed during L'Aquila 2009 seismic sequence.

1 INTRODUCTION

In the last decades, since the sensibility toward the conservation of cultural heritage has become greater and the entity of earthquakes began to threaten the stability of historical and recent buildings, the research started to concentrate on the response of constructions subject to seismic actions [4]. The most diffused technology, at least in the surviving historical architecture is masonry. Masonry was exploited because it used to be a fair compromise between mechanical [5], [6], thermal [7], and fire resistance properties [8], and due to the availability of raw materials. Masonry structures, either in stone blocks or clay bricks, are inherently statically undetermined and very rigid. Therefore, they tend to discharge energy forming cracks and hinges, instead of damaging by stress concentration.

Usually in a building, the absence of tie elements between the wall layers or the lack of links between different walls (box behaviour) and panels leads to the overturning of some portions. Horizontally thrusting structures like untied roofing trusses and arches, which transfer concentrated loads at a certain distance from the ground, decreasing the resistance to horizontal accelerations. Besides, thin but massive roofing elements such as domes and vaults are especially interesting in this field. Indeed, sometimes they are opened or further loaded at the top.

This paper represents the beginning of a study regarding the response of masonry double curvature structures subject to seismic actions. In this study, given the way masonry is usually damaged, as a structural material, it is going to be modelled according to a macroscopic approach, which sees the use of elastic macroblocks connected by elasto-plastic interfaces. Once the structural model is meshed in a Finite Element (FE) software environment, a nonlinear static analysis is performed in the G2 mode according to the Italian standard and the capacity curve is obtained.

The general aim is the production of a software for structural analysis of masonry structures (with particular attention to double curvature ones), simple and stable enough to be used in professionals' activity.

1.1 The case study

The approach has been applied to the Anime Sante Dome, which covers the Church of Santa Maria del Suffragio in L'Aquila, Italy (**Figure 1**). It is a single-nave church, built in the Baroque period starting from 1713. An ogival dome (span/thickness = 27) on a drum covers the presbyteral area. Eight windows from the drum and eight from the lantern light up the space (**Figure 2**). A large portion of the dome collapsed during the earthquake that occurred in 2009 (M_L = 5.8, M_w = 6.3). After the event, the monitoring campaign started [9]. A scheme of the collapsed dome is reported in **Figures 3** and **4**. Some years later it has been rebuilt, but its post-earthquake analysis is still interesting to explore the possibilities offered by new models for material and structural analysis.

The intent of this paper is programmatic. The model prepared for the post-earthquake assessment of the Anime Sante Dome will serve as a baseline to check the results of a future macroscopic model, the interfaces of which will be automatically produced starting from a simpler mesh. Up to now, this simplified model is compared with one already available and based on Limit Analysis [10].

Figure 1. External view of Church of Santa Maria del Suffragio. From the cathedral square in L'Aquila.

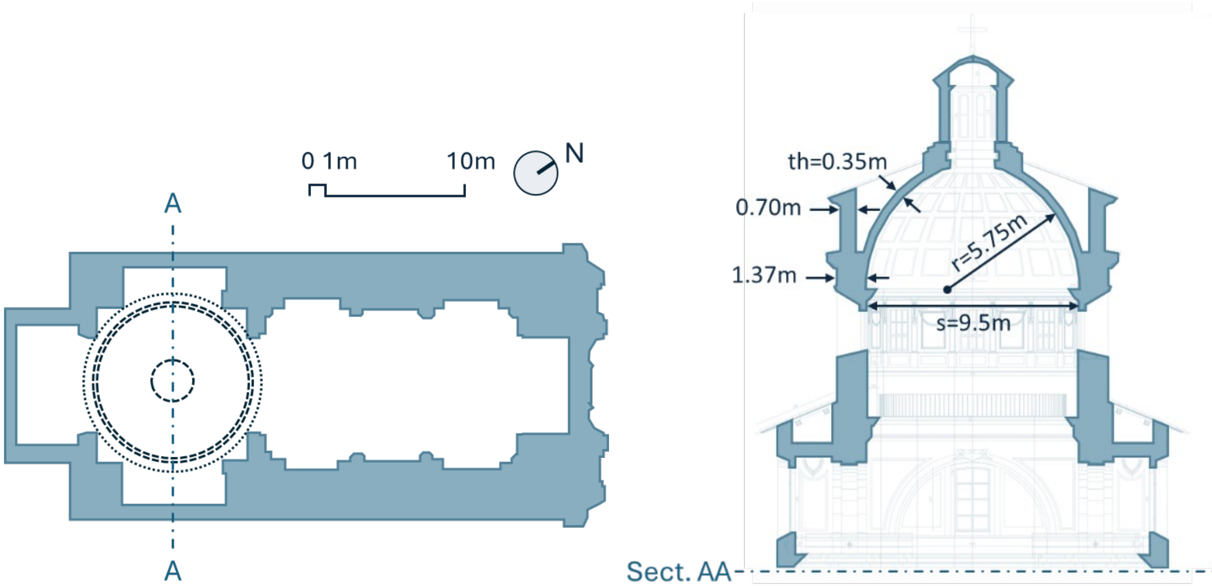

Figure 2. Architectural schemes of the Church. Note: the plan (left) and the section (right) are represented in different scales.

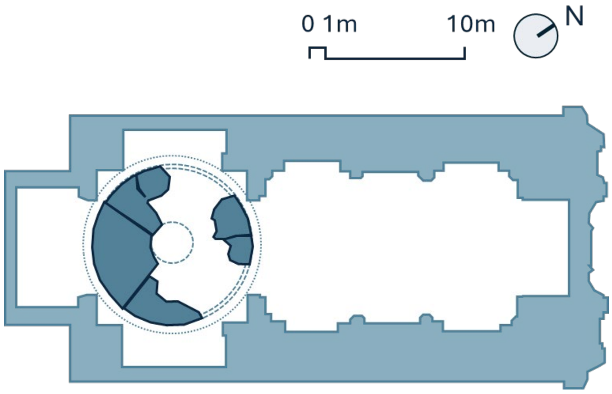

Figure 3. Architectural scheme of the church. The plan shows the position of the dome, its damage and the parts that withstood the earthquake.

Figure 4. Inner view of the dome after the earthquake of 2009. As can be seen, diagonal cracks separated large portions of masonry which could still stand by virtue of the double curvature of the construction.

2 MODELLING

During the 2009 earthquake, the Church of Santa Maria del Suffragio suffered from many damages. Aside from the dome, the walls of the nave showed shear cracks; other cracks appeared on the arches, while the façade and the apse detached from the central body, but no more collapses happened [10]. Indeed, being the structure quite massive, it is herein considered as rigid, therefore able to transfer the acceleration from the ground to the upper architectural elements.

At first, the structural model made for this study concentrates on the dome only, simplifying a lot the preprocessing phase but also affecting the results. Indeed, the presence of windows on the lantern has been neglected, as well as the wooden roof covering the extrados of the dome. Even though the drum experienced some damage, at first, it has been considered here as rigid as the rest of the building.

Given the way in which masonry is damaged, usually in portions which are larger than the size of the single units, the macroscopic approach has been applied to this case. In such approach, elastic macroblocks (made by 8-noded hexahedral and 6-noded wedge elements) are connected by elastic perfectly plastic interfaces. Such “Elastic Body Spring Model” (EBSM) follows the Rigid Body Spring Model (RBSM) exploited by Casolo S. [11], [12], [13] and introduced by Kawai [14] in the last century. The mechanical characteristics of these components, listed in **Table 1**, have been chosen by experience with linear dynamic analyses. Indeed, with the elastic moduli as listed, the frequency of the structure stays within the plateau of the spectrum typical for masonry structures. Then, for the present a similar pushover analysis, the aim is to obtain fair values of the ultimate load carrying capacity, independently from the materials properties.

Table 1. Elements mechanical properties assigned to the model

	E [MPa]	**ν [-]**	**ρ [kg/m^3]**	**Nonlinearity**	**Yield Criterion**	**α [°]**	**c [MPa]**
Macroblock	3000	0.2	1600	Elastic	-	-	-
Interface	500	0	1600	Elastic-Plastic	Mohr Coulomb	30	0.01

2.1 Interfaces

The interfaces are the focus of the incipient study. Indeed, in this case, the interfaces have been meshed by elements of the same type as the macroblocks. They have been set as elastic perfectly plastic, responding to a classical Mohr-Coulomb failure criterion, placed to lump the deformation and show damaging. Similar approaches have been previously exploited in the literature, considering elastic-plastic interfaces regulated by Mohr-Coulomb law and modelled either with zero thickness [15], [16], [17] or by rigid bodies [18]. A simple scheme of the mesh is reported in **Figure 5**.

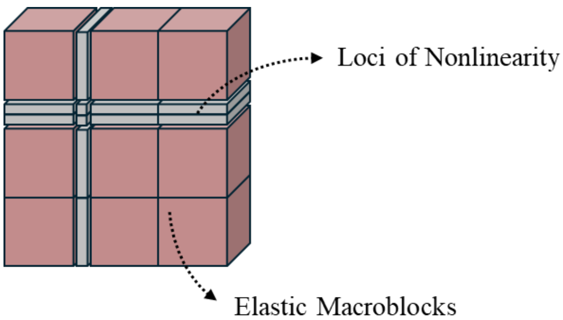

Figure 5. Macroscopic modelling with elastic macroblocks and nonlinear interfaces. Exploded axonometric scheme.

The aim is to make the process as general as possible, making it able to account for the real behaviour of masonry, which is an orthotropic material, in function of the arrangement too.

3 NONLINEAR STATIC ANALYSIS

The structural analysis has been conducted on the model so far described, considering fully fixed supports at the base and rollers preventing the horizontal movements normal to the symmetry plane, which cuts the model in halves (**Figure 6**). Considering only one half of the dome is possible thanks to the symmetry characterising such kind of constructions. It also reduces the computational burden and immediately allows to show the collapse mechanism. A displacement-control nonlinear static (pushover) analysis has been conducted in a simple and fast way. As the Italian standard [19] prescribes, the proportionality of the seismic action to the masses of the structure is ensured by the application of a horizontal gravity field, according to a G2 distribution (constant loading). The boundary conditions applied to the model are schematically represented in **Figure 6**.

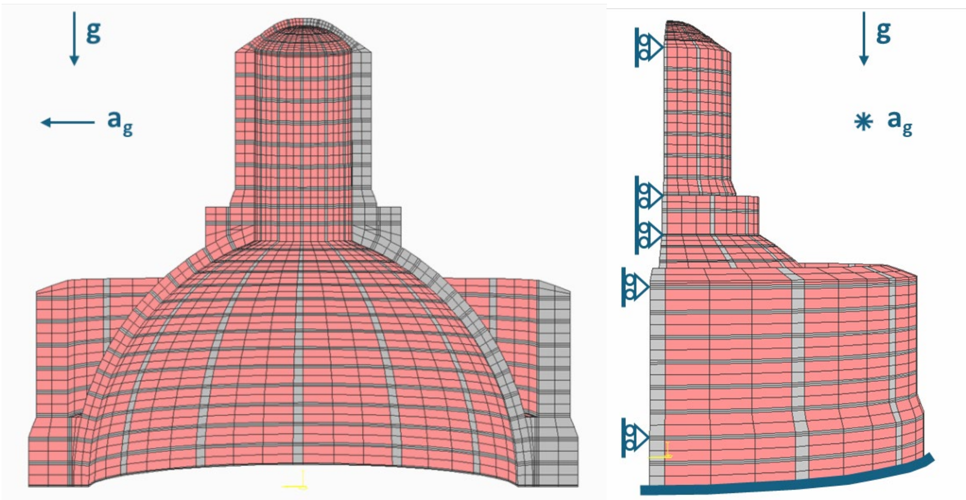

Figure 6. Model meshing and boundary conditions. Fixed support on the base and rollers on the symmetry cutting plane. Loading conditions: gravity acceleration and field of constant horizontal acceleration.

3.1 Results

In this section, the results of the displacement-controlled nonlinear analysis are presented. For three specific reference points chosen from the most sensible locations of the structure, the capacity curves have been extracted. The results, though focusing on the upper part only, are fairly comparable to those from the literature. **Figure 7** and **8** report the deformed configuration of the structure subject to accelerations of 0.2g and 0.35g respectively, with the indication of the reference points position. The capacity curve in **Figure 9** refers to the node in the middle point of the dome haunch, which is subject to major displacement.

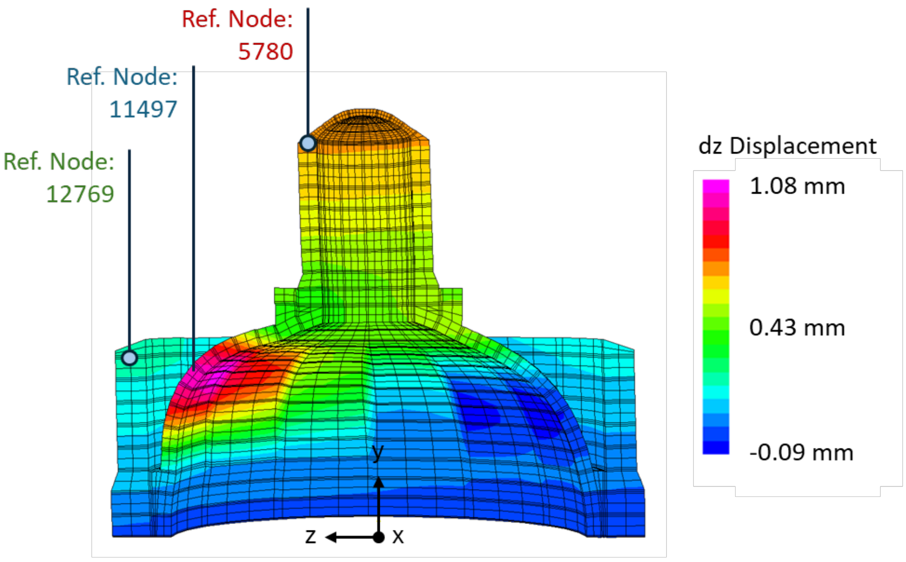

Figure 7. Deformed configuration corresponding to an acceleration of 0.20g (0.97mm displacement for the dome haunch, 0.68mm and 0.21mm for the drum and the cupola respectively).

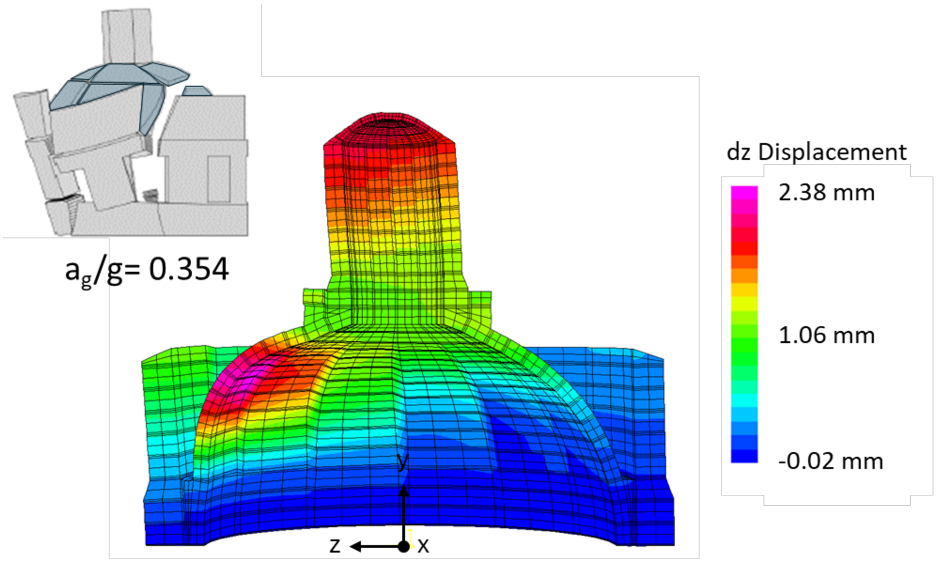

Figure 8. Deformed configuration corresponding to an acceleration of 0.35g (2.10mm displacement of node 11497 on the dome haunch). Comparison with literature results from UB-LA (top left picture).

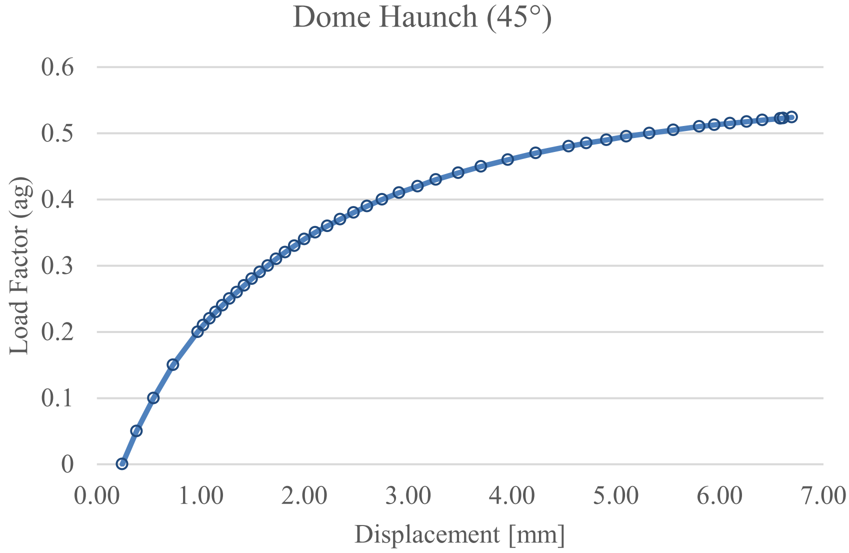

Figure 9. Capacity curve with reference to Node 11497.

Figure 10 reports the deformed configuration of the structure subject to an acceleration of 0.35g. The capacity curve in **Figure 11** refers to the nodes on the top of the drum and at the top of the lantern (at the base of the cupola).

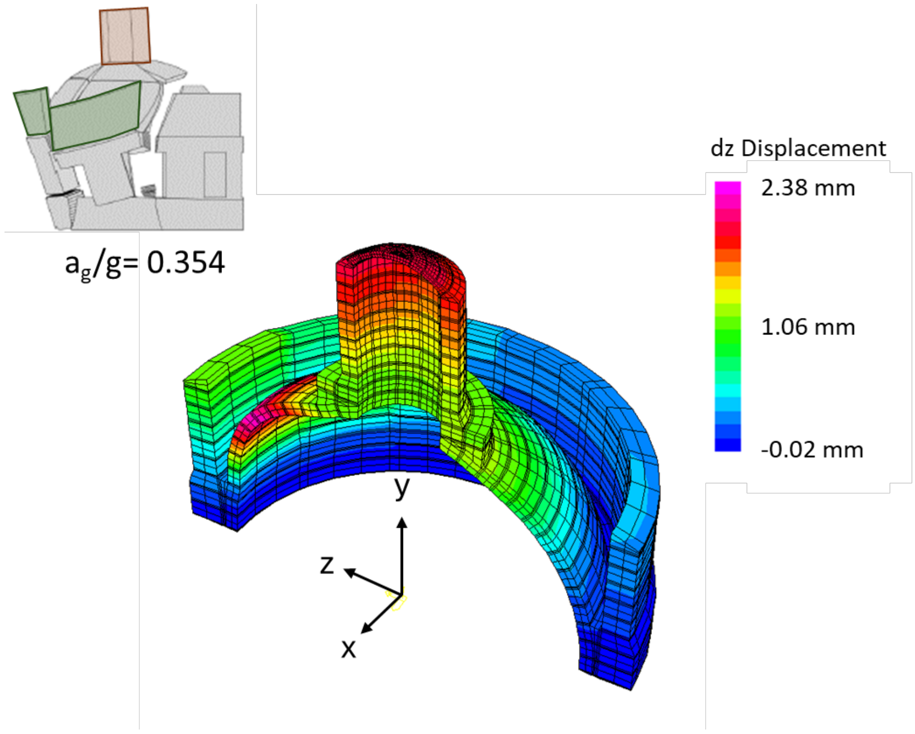

Figure 10. Deformed configuration corresponding to an acceleration of 0.35g (1.95mm and 1.04mm displacement for the drum and the cupola respectively). Comparison with literature results from UB-LA (top left picture).

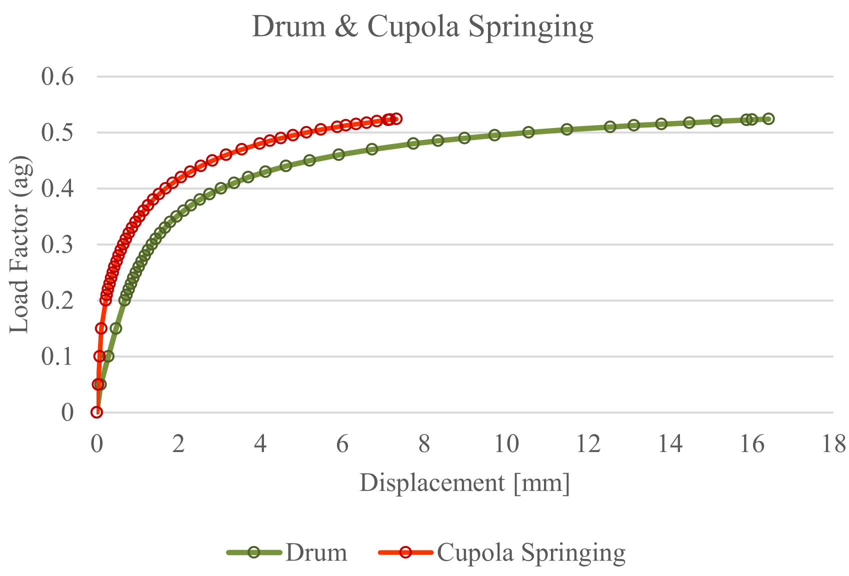

Figure 11. Capacity curve with reference to Nodes 12769 (Green) and 5780 (Red) of the Drum and the Cupola respectively.

Other interesting results can be deduced by analysing the vertical displacements of the elements. As **Figure 12** suggests, the lantern was subject to a shearing action coming from its tendency to overturn.

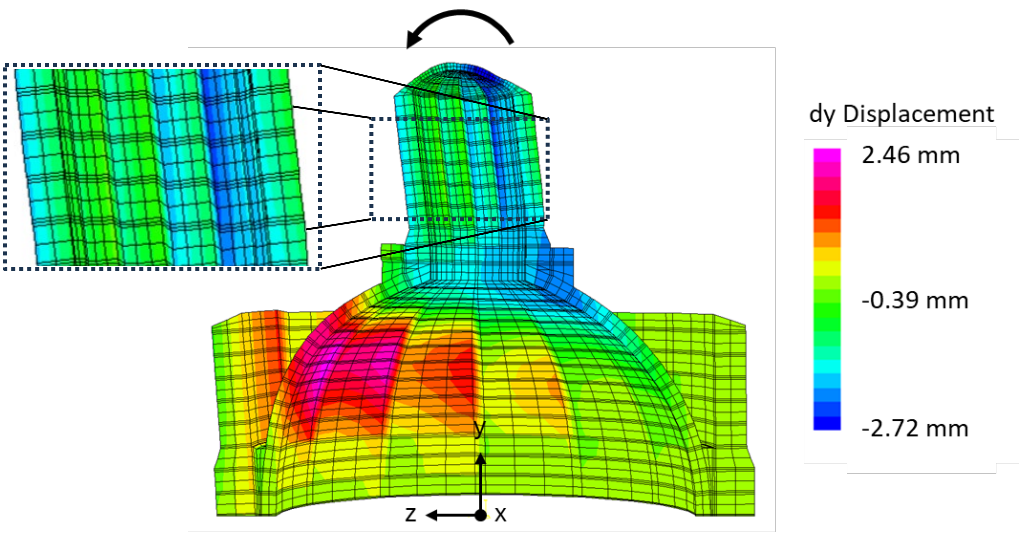

Figure 12. Deformed configuration corresponding to an acceleration of 0.40g. Brick displacement evaluated in Y direction.

4 DISCUSSION AND CONCLUSION

The capacity curves reported in the previous section show very high results of acceleration. However, it should be considered that the results are affected by the inertia of the structure and the neglection of the height effect. Indeed, in this case no contribution from the underlaying structure is accounted for.

As it is shown in **Figures 7**, **8**, and **12**, the meridian joints in the dome deform showing the classical opening of meridian slices as happens in static conditions [1], [20], plus a shear deformation given by the horizontal acceleration. In **Figure 10**, the effect of the shearing action is visible on the vertical joints of the upper drum. In **Figure 12**, a shearing action in the vertical joints is highlighted and it is given by the tendency of the lantern, to overturn. This is characteristic of such a model and an advancement of the results obtained by UB-LA [10]. The lantern as modelled is smaller and stiffer than the dome. However, though giving a good prediction of the collapse mechanism, the neglection of the windows surely affects the results. Moreover, the way of modelling presented is able to show displacements and the related capacity curve, which was not possible under the assumption of one of reference studies [3].

By comparing the deformed configurations shown in **Figure 8**, **10** and **12** with the literature results and the state of the dome after the event in **Figure 4**, the global collapse mechanism can be guessed, but no proof of actual damage of the model interface and no precise correspondence with real damage (diagonal cracks) can be detected.

As can be deduced from the FE model results and in agreement with literature results [3], [10], the lantern initiated a rotation on the crown of the dome, which caused an uneven stress distribution and the consequent collapse of the dome around the taut portion.

5 FUTURE DEVELOPMENTS

The method applied to this case study is going to evolve into a more general approach. For what concerns the case study itself, further studies will involve the complete drum and all the openings (both on the lantern and the drum), expecting better and richer results. Different G1 loading combinations can be applied separating the drum, the dome and the lantern by virtue of their different height and shape. A sensitivity study will be performed to check the impact of mesh-dependency [21] of such a model on the results.

Regarding the method, the intention of the authors is the reduction of the time needed for the preprocessing phase. As anticipated in the text, an automatic procedure will generate interfaces made by Rigid Beams, Trusses and Shear Panels, as happened in [1], [22], starting from a simpler interface produced in a fast way (by simple 3D elements) in a FE software. The communication will happen by a *.txt file, both in input and output. The new heterogeneous model exported in text form will then undergo NLSA in the FE software or LA procedures developed by the authors' research group. Such a procedure simplifies the explicit meshing process of very large and massive structures like [23]. The results of this modelling will be checked against previously obtained results [24].

REFERENCES

[1] A. Gandolfi, N. Pingaro, and G. Milani, "Simple non-linear numerical modelling for unreinforced and FRP-reinforced masonry domes," *Buildings*, vol. 14, no. 1, p. 166, 2024, doi: https://doi.org/10.3390/buildings14010166.

[2] A. Chiozzi, N. Grillanda, G. Milani, and A. Tralli, "UB-ALMANAC: An adaptive limit analysis NURBS-based program for the automatic assessment of partial failure

mechanisms in masonry churches," *Eng Fail Anal*, vol. 85, no. November 2017, pp. 201–220, 2018, doi: 10.1016/j.engfailanal.2017.11.013.
[3] M. Pavlovic, E. Reccia, and A. Cecchi, "A Procedure to Investigate the Collapse Behavior of Masonry Domes: Some Meaningful Cases," *International Journal of Architectural Heritage*, vol. 10, no. 1, pp. 67–83, 2016, doi: 10.1080/15583058.2014.951797.
[4] M. Acito, M. Buzzetti, C. Chesi, E. Magrinelli, and G. Milani, "Failures and damages of historical masonry structures induced by 2012 northern and 2016–17 central Italy seismic sequences: Critical issues and new perspectives towards seismic prevention," *Eng Fail Anal*, vol. 149, Jul. 2023, doi: 10.1016/j.engfailanal.2023.107257.
[5] B. Ghiassi, A. T. Vermelfoort, and P. B. Lourenço, "Masonry mechanical properties," in *Numerical Modeling of Masonry and Historical Structures: From Theory to Application*, Woodhead Publishing, 2019, pp. 239–261. doi: 10.1016/B978-0-08-102439-3.00007-5.
[6] T. Zimmermann, A. Strauss, and R. Wendner, "Old masonry under seismic loads: Stiffness identification and degradation," in *Structures Congress 2011 - Proceedings of the 2011 Structures Congress*, 2011, pp. 1736–1747. doi: 10.1061/41171(401)151.
[7] M. Ismaiel, Y. Chen, C. Cruz-Noguez, and M. Hagel, "Thermal resistance of masonry walls: a literature review on influence factors, evaluation, and improvement," *J Build Phys*, vol. 45, no. 4, pp. 528–567, Jan. 2022, doi: 10.1177/17442591211009549.
[8] A. Daware and M. Z. Naser, "Fire performance of masonry under various testing methods," *Constr Build Mater*, vol. 289, Jun. 2021, doi: 10.1016/j.conbuildmat.2021.123183.
[9] G. Boscato, D. Rocchi, and S. Russo, *Anime Sante church's dome after 2009 L'Aquila earthquake, monitoring and strengthening approaches*, vol. 446–449. 2012. doi: 10.4028/www.scientific.net/AMR.446-449.3467.
[10] N. Grillanda, A. Chiozzi, G. Milani, and A. Tralli, "Collapse behavior of masonry domes under seismic loads: An adaptive NURBS kinematic limit analysis approach," *Eng Struct*, vol. 200, no. January, p. 109517, 2019, doi: 10.1016/j.engstruct.2019.109517.
[11] S. Casolo, "A three-dimensional model for vulnerability analysis of slender medieval masonry towers," *Journal of Earthquake Engineering*, vol. 2, no. 4, pp. 487–512, 1998, doi: 10.1080/13632469809350332.
[12] S. Casolo, "Rigid element model for non-linear analysis of masonry façades subjected to out-of-plane loading," *Commun Numer Methods Eng*, vol. 15, no. 7, pp. 457–468, 1999, doi: 10.1002/(SICI)1099-0887(199907)15:7<457::AID-CNM259>3.0.CO;2-W.
[13] S. Casolo, "Modelling the out-of-plane seismic behaviour of masonry walls by rigid elements," *Earthq Eng Struct Dyn*, vol. 29, no. 12, pp. 1797–1813, 2000, doi: 10.1002/1096-9845(200012)29:12<1797::AID-EQE987>3.0.CO;2-D.
[14] T. Kawai, "New discrete models and their application to seismic response analysis of structures," *Nuclear Engineering and Design*, vol. 48, no. 1, pp. 207–229, Jun. 1978, doi: 10.1016/0029-5493(78)90217-0.
[15] M. Pepe, M. Sangirardi, E. Reccia, M. Pingaro, P. Trovalusci, and G. de Felice, "Discrete and Continuous Approaches for the Failure Analysis of Masonry Structures

Subjected to Settlements," *Front Built Environ*, vol. 6, Apr. 2020, doi: 10.3389/fbuil.2020.00043.

[16] C. Zhao, Y. Xiong, X. Zhong, Z. Shi, and S. Yang, "A two-phase modeling strategy for analyzing the failure process of masonry arches," *Eng Struct*, vol. 212, 2020, doi: 10.1016/j.engstruct.2020.110525.

[17] G. Milani, M. Rossi, C. Calderini, and S. Lagomarsino, "Tilting plane tests on a small-scale masonry cross vault : Experimental results and numerical simulations through a heterogeneous approach," *Eng Struct*, vol. 123, pp. 300–312, 2016, doi: 10.1016/j.engstruct.2016.05.017.

[18] E. Reccia, A. Cazzani, and A. Cecchi, "FEM-DEM Modeling for Out-of-plane Loaded Masonry Panels: A Limit Analysis Approach," *Open Civil Engineering Journal*, vol. 6, no. 1, pp. 231–238, 2012, doi: 10.2174/1874149501206010231.

[19] 17 gennaio 2018 Decreto del Ministero delle Infrastrutture e dei Trasporti, *Aggiornamento delle «Norme tecniche per le costruzioni»*. Rome: Ministero delle Infrastrutture e dei Trasporti, 2018.

[20] G. Milani, "Simple lower bound limit analysis model for masonry double curvature structures," *Comput Struct*, vol. 269, p. 106831, 2022, doi: 10.1016/j.compstruc.2022.106831.

[21] G. Milani and E. Bertolesi, "Quasi-analytical homogenization approach for the non-linear analysis of in-plane loaded masonry panels," *Constr Build Mater*, vol. 146, pp. 723–743, 2017, doi: 10.1016/j.conbuildmat.2017.04.008.

[22] N. Pingaro and G. Milani, "Simple non-linear numerical modelling of masonry arches reinforced with SRG using elasto-fragile and elasto-ductile truss finite elements," *Eng Struct*, vol. 293, no. July, p. 116637, 2023, doi: 10.1016/j.engstruct.2023.116637.

[23] A. Gandolfi, N. Pingaro, G. Milani, and S. Ghosh, "Nonlinear static analysis of Global Vipassana Pagoda by means of a novel FE-based method: Modelling Strategy," in *IEEE Metrology for Living Environment (MetroLivEnv)*, Chania, Jun. 2024.

[24] A. Gandolfi, N. Pingaro, G. Milani, and S. Ghosh, "Nonlinear static analysis of Global Vipassana Pagoda by means of a novel FE-based method: Results," in *IEEE Metrology for Living Environment (MetroLivEnv)*, Chania, Jun. 2024.