Unloading and reloading process for the earthquake damage repair of ancient masonry columns: the case of the Basilica di Collemaggio

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ABSTRACT

The repair of historical masonry columns damaged by earthquakes is a complex engineering procedure. In most cases, these elements support important vertical loads, and, for this reason, the repairing interventions (such as block replacement) could generate critical modifications of the stress pattern and eventually lead to local failures of the masonry material. In order to prevent critical situations, the repair work should account a complete unloading-reloading process of the column. However, at this time, no specific indications are provided by technical codes and guidelines. In the present paper, the unloading-reloading process developed for the stone masonry columns of the Basilica di Collemaggio in L'Aquila (Italy) is presented. Particularly, two specific unloading systems were designed and verified with advanced finite element analyses. A three-dimensional finite element model of the colonnade and supported nave wall was implemented to check if the unloading-reloading process would significantly change the state of stress in the masonry, causing local failures. The geometry of the structure was reconstructed from a complete laser scanner survey of the church, by considering all the intrinsic irregularities of heritage constructions. Finally, a comparison between the numerical results and the corresponding values measured during the worksite intervention was carried out.

Keywords: unloading and reloading process, historical building, stone masonry columns, seismic retrofitting, finite element, construction stage, Basilica di Collemaggio

1. INTRODUCTION

The preservation and structural rehabilitation of historic buildings are relevant topics nowadays, especially in countries with a vast inventory of heritages located in seismic prone regions (Binda and Saisi 2005, Lagomarsino and Cattari 2015). Most of these constructions are constituted by ancient unreinforced masonry (URM) and differ for age, typology and building techniques.

Particularly, religious sites, such as churches, cathedrals, sanctuaries, monasteries, etc. represent an important portion of the monumental building stock and are characterized by high levels of vulnerability to earthquake actions (Lourenço *et al.* 2007, Betti and Vignoli 2011, Roca *et al.* 2013, Chellini *et al.* 2014, Milani and Valente 2015). One of the most dramatic example of the devastating effect of the ground shaking on historic churches was the 2009 L'Aquila earthquake: the post-event reconnaissance revealed that among 240 monumental constructions, 170 of them were heavily damaged or experienced partial failures (Brandonisio *et al.* 2013). As a matter of fact, churches are characterized by complex structural configurations (slender walls, open spaces, absence of stiffening floors, presence of vaults, arches, domes, etc.) and poor quality of the masonry material due to the mechanical properties deterioration over the years (Brandonisio *et al.* 2013). In addition, since these buildings were realized before the development of structural engineering theories, they can suffer from serious structural deficiencies.

One typical characteristic of churches is the presence of colonnades which geometrically divide the space in aisles. Masonry columns usually sustain arches, massive walls, domes, etc. and are consequently subjected to larger vertical stresses with respect to perimeter walls. High compressive loads together with material deterioration, settlements and construction defects can put at risk the stability of these elements, as it was observed in some monumental buildings in the past (Tringali *et al.* 2003, Roca *et al.* 2013, Coronelli *et al.* 2015). In late 60ies, for example, the Cathedral of Milan (Italy) suffered damages in the main pillars of the crossing mostly because of the lowering of the water table of the city and the consequent foundation settlements (Coronelli *et al.* 2015). The differential displacements generated severe overloads in some piers with respect to the others resulting in deep vertical cracks in the marble blocks. Luckily, a prompt rehabilitation work prevented the failure of the pillars. The restoration consisted in the progressive substitution of the damaged blocks with new bigger marble pieces, able to deeply penetrate in the rubble core of the cross-section. A more critical situation interested the Cathedral of Noto (Sicily, Italy) when, in 1996, the right-side colonnade of the central nave suddenly collapsed (Binda *et al.* 2003). In this case the reason of the failure was correlated to the damages suffered by the building during the 1990 Sicily earthquake and to the bad construction quality of the columns filled with an incoherent core. After the failure, the building was completely renovated by reconstructing either right and left colonnades with structurally sound stone masonry cross-sections (Tringali *et al.* 2003).

As a consequence of the 2009 L'Aquila seismic event, the Basilica di Collemaggio, one of the most iconic churches of the Abruzzo region, experienced the collapse of the transept pillars and severe damages in the central nave columns (Gattulli, Antonacci, *et al.* 2013). Particularly, the post-earthquake survey revealed that the transept pillars were characterized by an incoherent filling core while the nave columns seemed to be constituted by compact stone blocks of *breccia aquilana*, a type of limestone from the Abruzzo region. Extensive structural investigations confirmed that this construction characteristic of the columns was probably the reason why they survived the seismic shaking (Crespi *et al.* 2016), despite strongly damaged. During the post-earthquake reconstruction phase, the columns have been subjected to a complex repairing activity which involved a complete unloading-reloading process. The aim of this paper is to provide details of this complex procedure. Two alternative solutions, one proposed during the design phase and the other during the construction phase, are herein presented and analyzed with detailed Finite Element (FE) models. The analysis results are then compared with measurements acquired during the construction work.

2. THE S. MARIA DI COLLEMAGGIO BASILICA

In Italy, historical constructions have been heavily impacted by earthquakes, as was observed in 2009, L'Aquila (Brandonisio *et al.* 2013, Gattulli, Antonacci, *et al.* 2013, De Matteis *et al.* 2016), 2012, Emilia-Romagna (Milani 2013), 2016, Amatrice (Sextos *et al.* 2018, Jain *et al.* 2020), and 2017, Ischia (Briseghella *et al.* 2019) earthquakes. The *S. Maria di Collemaggio* Basilica, one of the most iconic heritage building in Central Italy, sustained severe damages during the 2009 L'Aquila event. Subsequently, thanks to funding provided by ENI company (www.eni.com), the church was reconstructed and retrofitted.

The Basilica was built in the late XIII century and is located on the top of a hill in the city of L'Aquila. The Basilica is an outstanding example of Romanesque architecture. During the centuries, the church had been continuously interested by architectural and structural interventions, spread over the whole building, many of which were consequent to earthquake damages (Carbonara *et al.* 2014, Crespi *et al.* 2016). The church is a three aisles structure built on a basilica plan with two lines of octagonal columns (Fig. 1). The transept together with two chapels and a choir are placed at the opposite side of the facade. The two altars at the far ends of the transept, together with the stucco decoration of walls, vaults and chapels are a residual of the baroque decoration removed in the seventies. The rest of the church shows bare walls with exposed wooden trusses to support a gable roof on the central nave and a pitched roof over the aisles. Just at the right of the majestic two-tone façade, an unusual octagonal base squat tower is located.

Fig. 1. S. Maria di Collemaggio Basilica (source "Soprintendenza Regione Abruzzo").

A considerable amount of research on the Basilica di Collemaggio is currently available in the literature. Interested readers could refer to the following works:

- dynamic response and structural health monitoring of the building (Antonacci *et al.* 2001, 2020, Alaggio *et al.* 2020, Aloisio *et al.* 2020);
- seismic response of the building during the last 2009 earthquake (Cimellaro *et al.* 2011, Cartapati 2012, Ciampi 2012, Gattulli, Graziosi, *et al.* 2013);
- seismic behavior of the colonnade and nonlinear response of the masonry columns (Crespi *et al.* 2016);
- laser scanner survey and building information modelling (Oreni *et al.* 2014, 2017, Brumana *et al.* 2018).

Additionally, some recent works presented the main structural engineering strategies that were implemented during the reconstruction and retrofitting process (Longarini *et al.* 2020, Zucca *et al.* 2020).

3. THE NAVE COLUMNS

According to historians, the stone masonry columns of the central nave of the Basilica di Collemaggio have an important historical and artistic value since they date back to the XIII century. The two colonnades, which include 14 columns, are characterized by several irregularities such as vertical and horizontal misalignments, different arch spans, inhomogeneous column heights and diameters, etc. In order to faithfully reflect their geometrical irregularities, a complete laser scanner survey was carried out (Oreni *et al.* 2017). The average cross-section diameter of the columns is 115 cm for the shaft and 140-160 cm (top-bottom) for the base. The columns are approximately 4 m high, without considering the capital and the base, whose average heights are respectively 0.54 m and 0.81 m. The basement and the capital are made of a local limestone rock called *'breccia aquilana'*. As pointed out by Crespi et al. (Crespi *et al.* 2016), these structural elements are characterized by a good transversal displacement capacity. In fact, the high compressive strength of the stone blocks (experimental average value $f_m = 66.68$ MPa) and the moderate level of dimensionless gravity axial load result in a ductile moment-curvature response of the cross-section (Giordano *et al.* 2017). However, the described behavior is negatively influenced by construction defects and material deterioration over the centuries such as (Fig. 2):

- ineffective interlocking of the stone blocks due to vertical alignment of head joints;
- presence of incoherent/rubble inner core;
- non-uniform vertical stress field over the cross-section caused by past block replacements;
- presence of damaged masonry units;
- heterogeneous blocks' material properties resulting from repair activities.

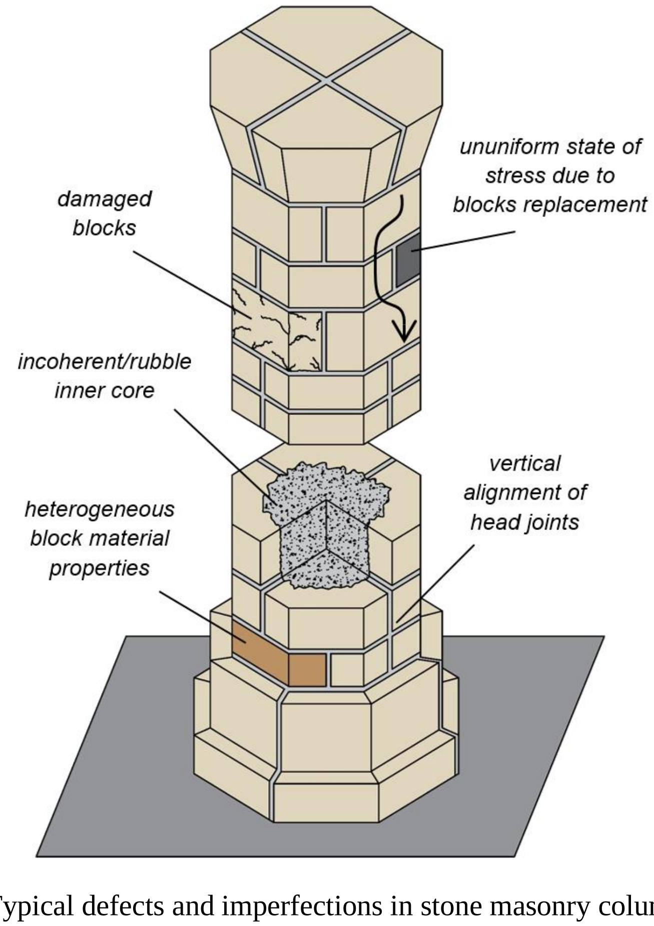

Fig. 2. Typical defects and imperfections in stone masonry columns.

3.1. Damage survey

The post-earthquake survey showed that the central columns of both colonnades were sensibly damaged with respect to the external ones. In fact, as discussed in previous works (Crespi *et al.* 2016), seismic forces acting in transversal direction with respect to the naves generate high bending moments on the central columns. Different types of damages have been identified during the survey:

- relative sliding of adjacent ashlars (Fig. 3a);
- local crushing/spalling of blocks and opening of pre-existent cracks (Fig. 3b);
- vertical cracks due to excessive compression stress (Fig. 3c);
- toe compression failure at the base of the columns (Fig. 3d).

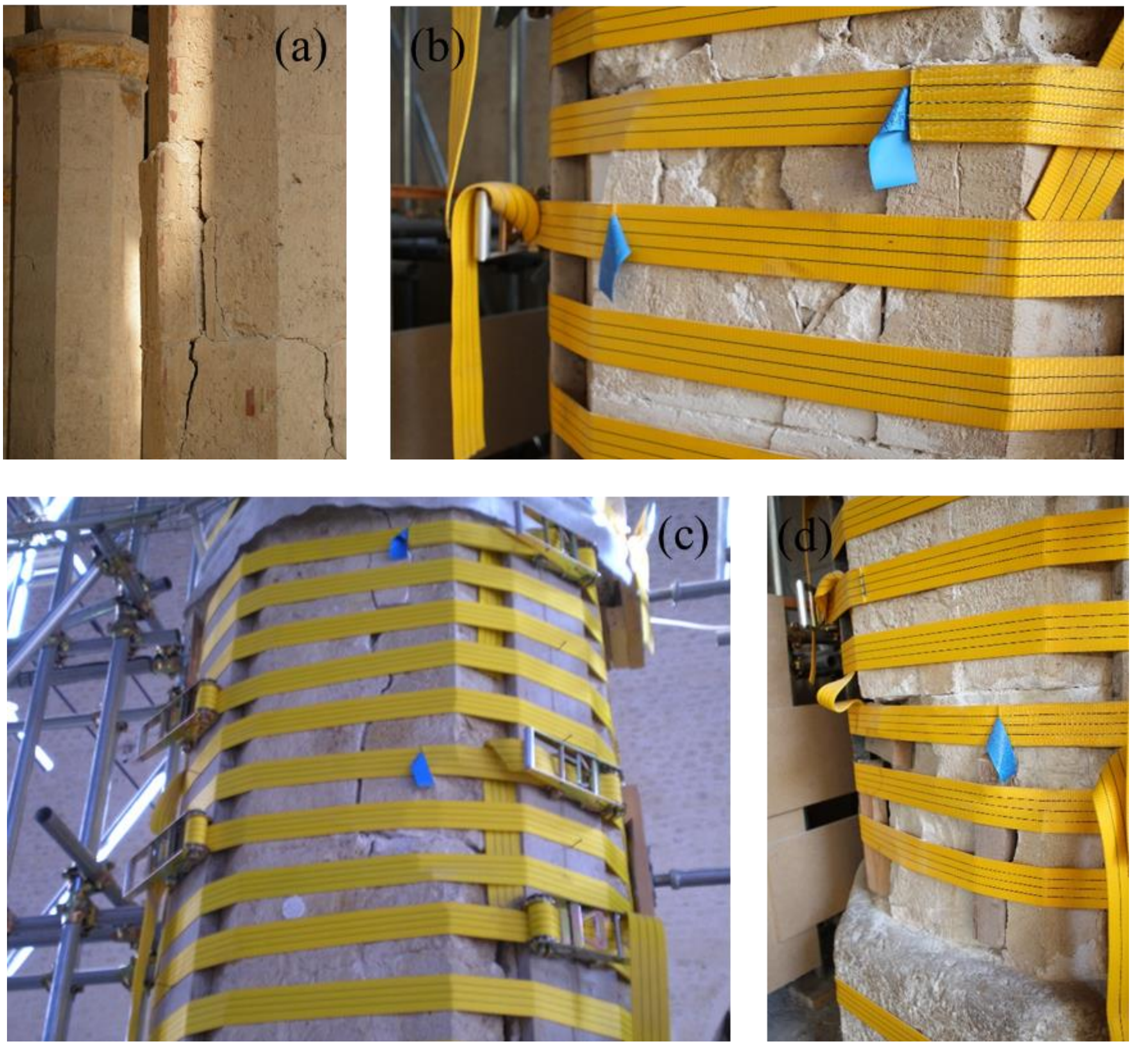

Fig. 3. Typical columns' damage observed after the 2009 earthquake: (a) sliding of ashlars, (b) crushing/spalling, (c) vertical cracks, (d) toe compression failure.

Fig. 4 shows the transversal section of the church and the way the different columns are grouped together in terms of damage level. It is specified that the columns next to the collapsed transept (undamaged columns - highlighted in blue) were rebuilt with a reinforced concrete core in the 70ies and did not suffer seismic damage during 2009 earthquake.

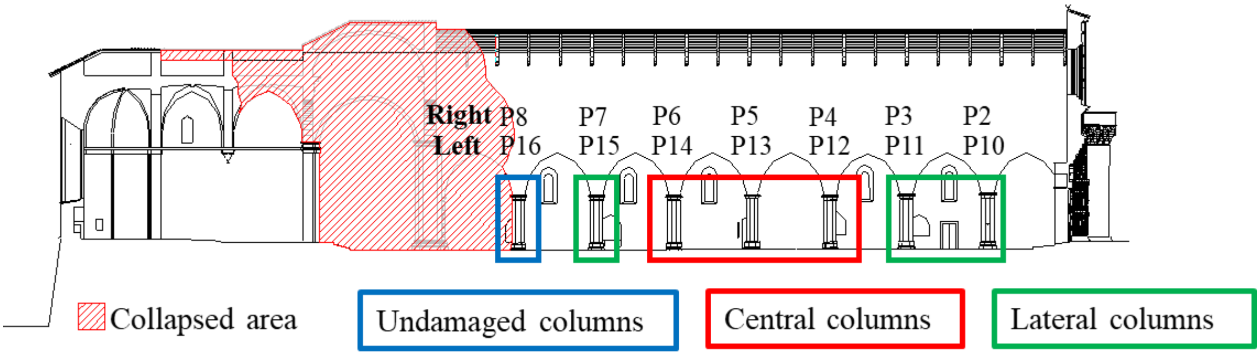

Fig. 4. Transversal section of the church.

4. UNLOADING-RELOADING PROCESS OF MASONRY COLUMNS

Given the different level of damage of the columns (Fig. 4), two repairing techniques were implemented:

- the central columns (heavily damaged) were completely disassembled and rebuilt using a combination of new blocks and original undamaged ashlars. In particular, the damaged blocks were substituted by new elements of *'breccia aquilana'*. In addition, the inner part of the column, originally constituted by rubble masonry, was substituted with a new core of high strength lime mortar;
- the lateral columns were interested by local block replacements and mortar injections, avoiding the complete disassembling of the elements.

To carry out the repairing activities, the columns had to be unloaded and reloaded. Therefore, a specific system to transfer the vertical forces from the columns to the adjacent ground had to be designed. Two alternative supporting steel frames were developed. The frames had to keep in place the arches and the masonry walls until the end of the intervention works and then, after the complete reloading of the retrofitted columns, had to be removed. A fundamental part of the design process was the verification of the unloading system with respect to possible damages to the nave walls. This was validated with advanced FE models by controlling the tensile stresses in the masonry elements during the unloading and reloading process.

The two unloading and reloading solutions developed in the project are presented in the following paragraphs. Conceptually, the first solution is based on the use of three steel ribs + frames that directly supports the arches (Fig. 5a). In this scenario, two columns can be unloaded and repaired simultaneously. The second solution acts on the abutments of the arches through a clamping system (Fig. 5b). To assess the feasibility of the unloading-reloading procedure, a nonlinear construction stage analysis reproducing all the phases of the intervention procedure was implemented in Midas FEA (Midas FEA, Analysis Manual 2015). Since the unloading-reloading procedure affects only the nave structures, a partial 3D finite element model of the left nave wall was developed starting from the 3D point-cloud laser scanner geometry coming from the survey (Brumana *et al.* 2018). In this way, the real shape of the wall, with all its irregularities, was considered. The effect of the other parts of the church were accounted with a set of loads and boundary conditions. In particular, one end of the nave wall is connected into the façade while on the other end is completely free given the collapse of the transept structures.

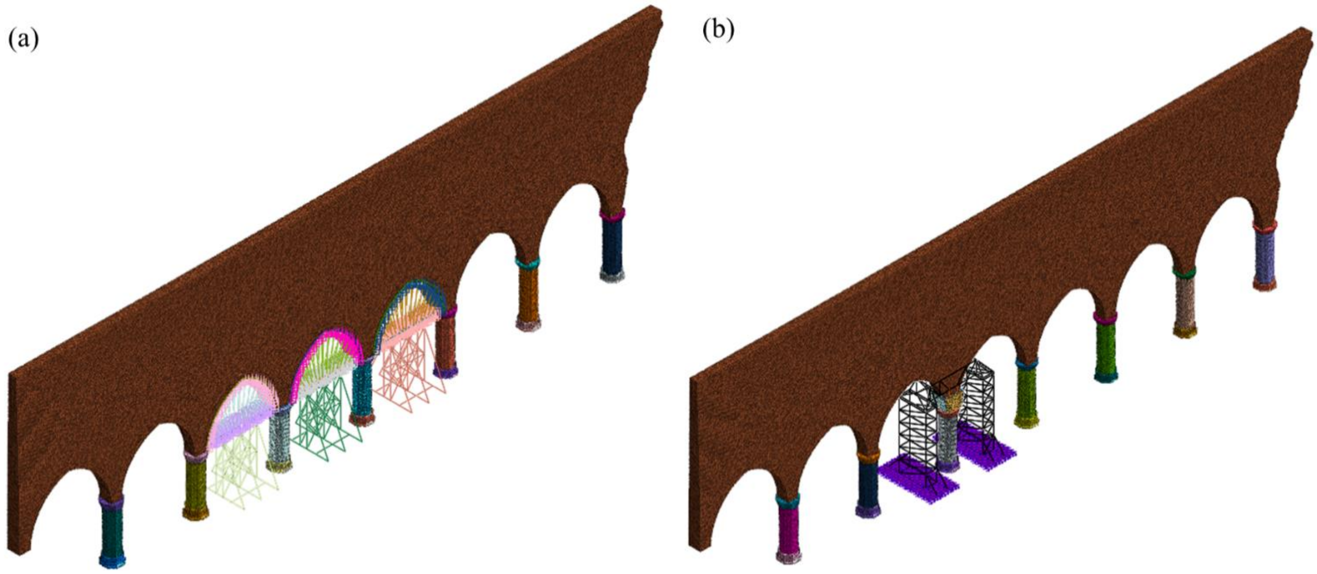

Fig. 5. FE models of the unloading-reloading systems (a) steel-ribs system (b) clamping system.

The following boundary conditions were applied on the FE model:

- fully-constrained ends at the bottom columns' basements;
- transversal supports along the wall edge connected to the façade. This prevents horizontal displacements in both out-of-plane and in-plane directions;
- horizontal simple supports applied at the upper edge of the nave wall to reproduce the out-of-plane restraining effect of the roof.

In addition, a uniform vertical pressure was applied on the upper edge of the nave wall. This pressure load, equal to 26 kN/m^2, was calculated by estimating the tributary area and weight of the roof.

The nonlinear behavior of the masonry material (stone/rubble) was reproduced by adopting the smeared cracking model available in Midas FEA (Crisfield 1986, Vecchio and Collins 1993). This model has been largely used for the assessment of historical masonry structures (e.g. (Clementi *et al.* 2016, Micelli and Cascardi 2020)). Two materials were adopted in the analyses (main mechanical properties reported in Table 1). Stone masonry made of *'breccia aquilana'* ashlars for the outer part of the columns (Fig. 6) and the arches (Fig. 7). Rubble masonry for the inner core of the columns (Fig. 6) and for the nave walls (Fig. 7). It is worth mentioning that, in absence of specific experimental tests on the tensile capacity of these masonries, it was assumed a median value equal to 10% of the compressive strengths in Table 1. Tetrahedral (solid) elements with characteristic size of 100 mm were used to discretize the masonry structures.

Table. 1. Main material properties of masonries adopted for the analyses.

Material	Unit Weight (γ)	Young Modulus (E)	Poisson Ratio (ν)	Compressive Strength (f_m)
	[kN/m^3]	[N/mm^2]	[-]	[MPa]

Stone masonry (*breccia aquilana*)	25*	20000*	0.2**	66.68*
Rubble masonry	18**	1600*	0.2**	2.00*
*University of L'Aquila - In-situ experimental tests, Basilica di Collemaggio (internal document) **Ministero delle Infrastrutture e dei Trasporti 2008, 2009				

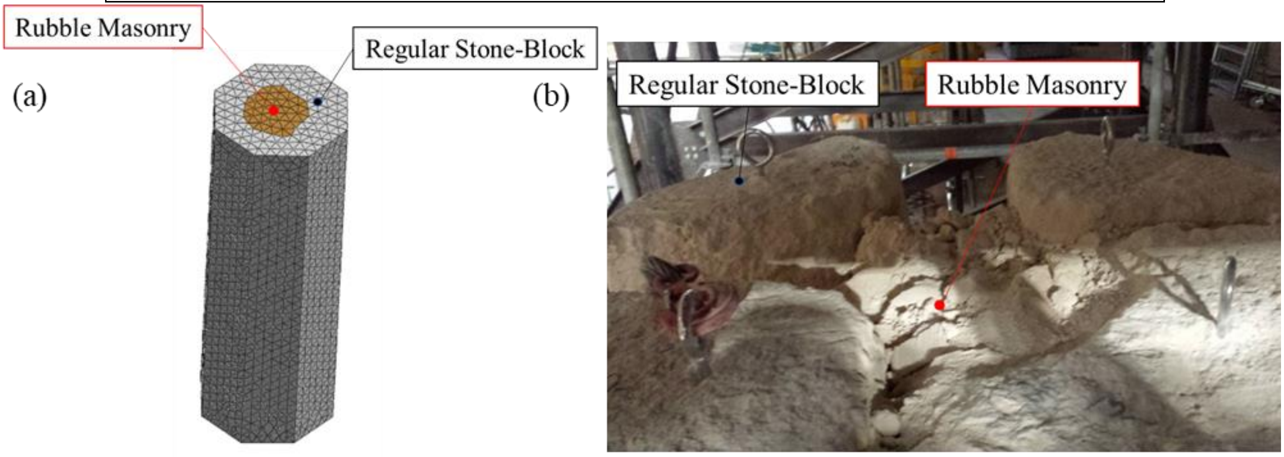

Fig. 6. Indication of masonry characteristics for the column: (a) FE model (b) real structure.

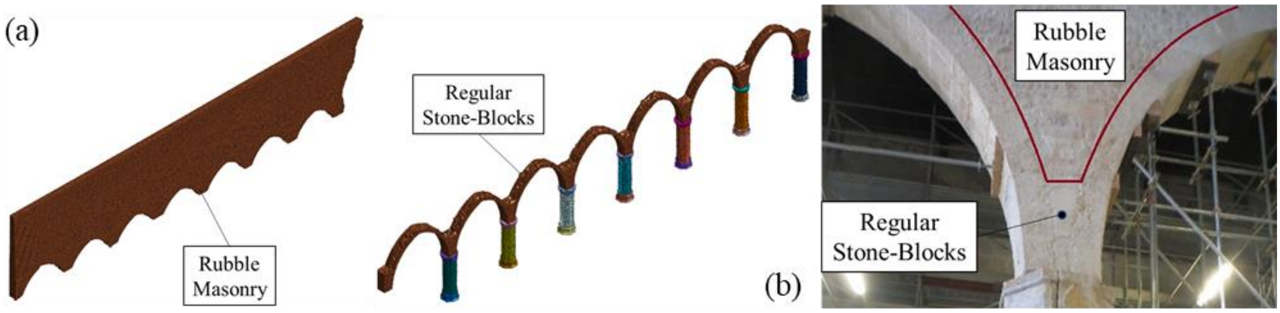

Fig. 7. Indication of masonry characteristics for the nave walls: (a) FE model (b) real structure.

4.1. The struts-ribs system

The struts-ribs system was designed to unload two internal columns of three adjacent arches (Fig. 5a). The frame, characterized by a height of 9 m and plan dimensions of 3.3 m × 4 m, is made of S355 steel profiles (Fig. 8). On top of the frame, a double level of steel ribs and struts is located. The columns are progressively unloaded by applying a distributed vertical-upward pressure on the intrados of the arches. The vertical pressure is generated by six hydraulic jacks that are placed in between the steel frame and the struts-ribs structure. To guarantee a uniform distribution of stresses over the surface of the arches, wooden planks are placed between the masonry and the ribs.

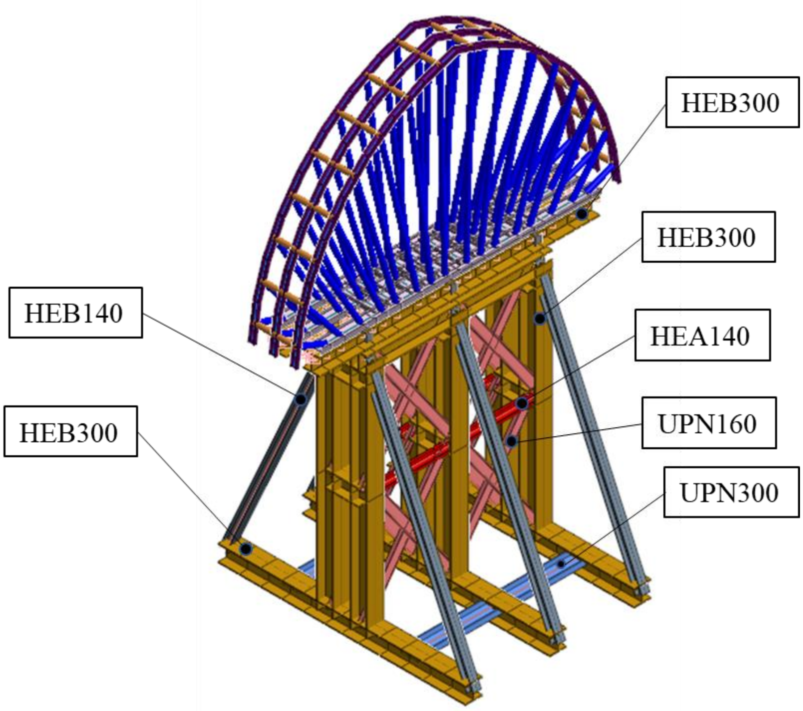

Fig. 8. FE model of the steel frame and ribs of the unloading-reloading system.

4.2. The clamping system

With the clamping system, the unloading-reloading procedure works on a single column by supporting the abutment of the two arches insisting on it (Fig. 5b). The abutment of the arches is confined in a steel clamp, placed 10 cm above the column's capital (Fig. 9 and 10). To guarantee a uniform stress distribution on the abutment, the gap between the clamp and the masonry surface is filled with mortar. Then, the clamp is tightened on the abutment through horizontal pre-stressing bars. This activates a horizontal biaxial compression stress field (compatible with the masonry compressive strength) that allows the lifting of the abutment by friction. The clamp is connected to the supporting steel frame with four threaded bars, and the column is progressively unloaded by applying tension loads to the threaded bars through four hydraulic jacks. Particular attention was devoted to the modelling of the steel clamp region. A mesh refinement was implemented in this part of the model (Fig. 10) to increase the geometrical accuracy. In fact, the thickness of the clamp-to-wall mortar filling is about 20 mm while the thickness of the clamp's plates is 14 mm.

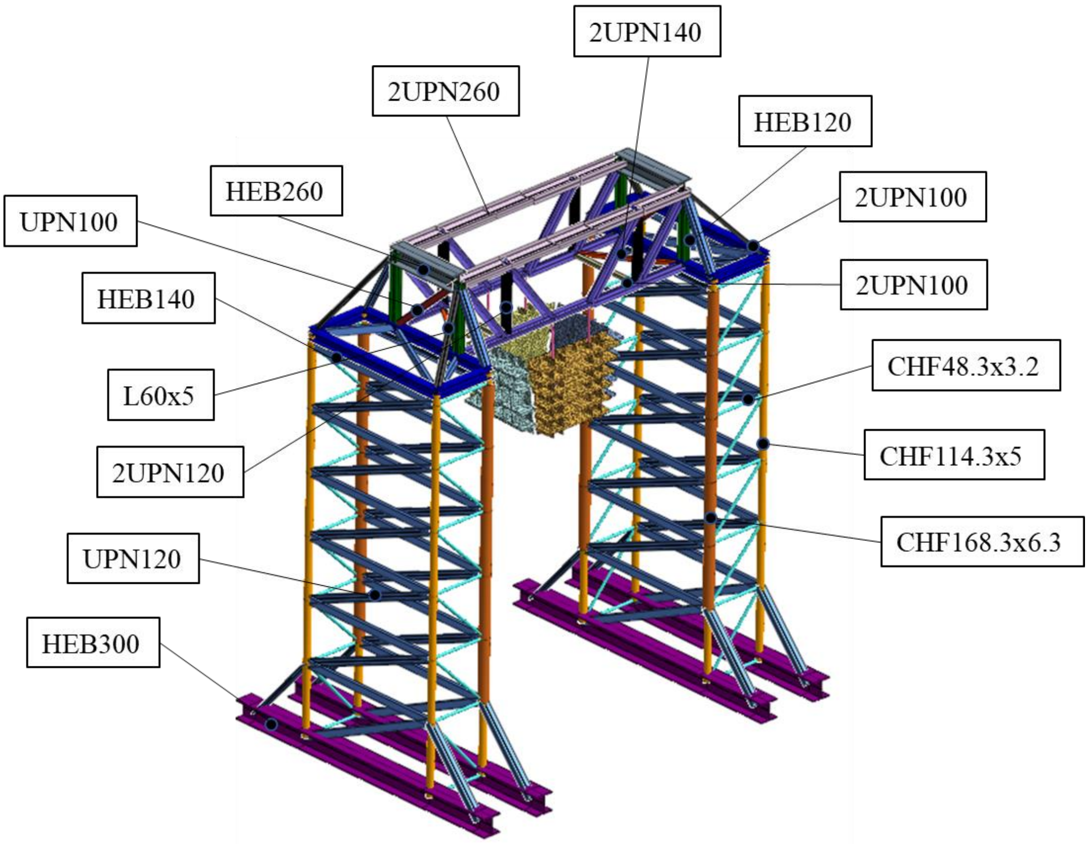

Fig. 9. FE model of the steel frame and clamping system.

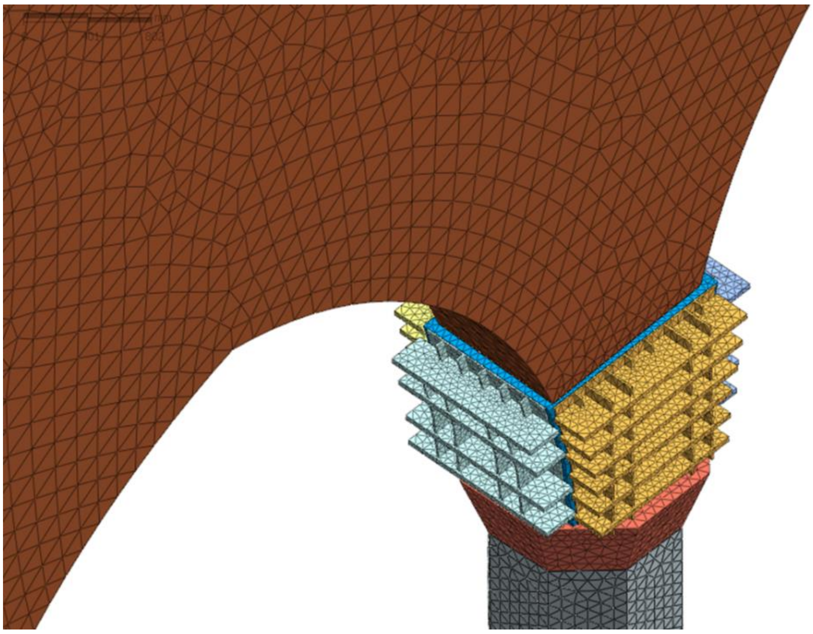

Fig. 10. Detail of the mesh refinement in the region of the clamp.

4.3. Construction stage sequence

To evaluate the stress fields on masonry walls and columns during the unloading-reloading process, a non-linear construction stage analysis was performed. In particular, this paper presents the results of the unloading-reloading process for a central column (third column from the façade) of the left aisle, one of the most damaged columns interested by the complete disassembling and reconstruction.

The implemented construction stage sequence was as follows (Fig. 11-12):

Phase 1. *Initial configuration.* The structure is modelled in its initial (post-earthquake) configuration, where the only applied load is the self-weight of masonry elements and roof. Three sub-stages are defied to reproduce the historical construction sequence: (i) realization of the columns; (ii) building of the stone arches in between the columns; (iii) realization of the nave walls on top of the arches.

Phase 2. *Assembling of the supporting frame.* Phase 2 depends on which unloading-reloading solutions is considered. The struts-ribs system involves: (i) positioning of the steel frames along three adjacent arches; (ii) installation of six hydraulic jacks on top of each steel frame; (iii) positioning of the struts-ribs structures on top of the jacks; (iv) installation of wooden planks in between the ribs and the stone masonry arches. On the other side, the clamping system involves the following sub-steps: (i) positioning of the clamp around the arch abutment and mortar filling. The self-weight of the elements together with the compression loads of the pre-stressing bars are introduced, without introducing changes to the boundary conditions; (ii) installation of the steel frame (no connection with the clamp). For both solutions, the steel frames are located on a temporary foundation of wood joists and sand that protects the Basilica's pavement. This foundation has been represented in the FE models with an equivalent-stiffness surface of springs.

Phase 3. *Unloading phase.* The vertical load on the column is progressively transferred to the provisional steel frame by means of hydraulic jacks. In the struts-ribs system, the loads are transferred on the top HEB300 couple of horizontal beams. In the clamping solution, the loads are transferred from the vertical threaded bars to the horizontal trussed beam. From a numerical point of view, the unloading condition is reached when the sum of the vertical reactions at the base of the column is equal to the self-weight of the column.

Phase 4. *Dismantling of the column*. Once the column is completely unloaded, the mortar joint between the capital and the abutment is removed and the disassembling of the column may begin (Fig. 13). From a practical point of view, the mortar joint between the top of the capital and the abutment is cut away by a diamond wire sawing. The unloaded column is then separated from the upper part of the structure, allowing the execution of the retrofitting works. This was translated in the FE model by eliminating a thin layer of finite elements (100 mm) above the column's capital.

Phase 5. *Repairing works*. The retrofitted column is re-introduced into the FE model through a new set of elements with updated mechanical properties of the inner core (Young modulus, $E = 15000$ N/mm^2, unit weight, $\gamma = 18$ kN/m^3, compressive strength $f_m = 40$ MPa and tensile strength $f_{tm} = 0.4$ MPa).

Phase 6. *Reloading phase.* After the intervention, the column is reloaded by the progressive unloading of the hydraulic jacks.

Phase 7. *Dismantling of the steel frame*. The last stage consists in removing the temporary supporting frames, together with the clamp and the mortar filling. By comparing the stress and displacement fields of Phase 1 and Phase 7 is then possible to quantify the effect of the construction process on the existing structure.

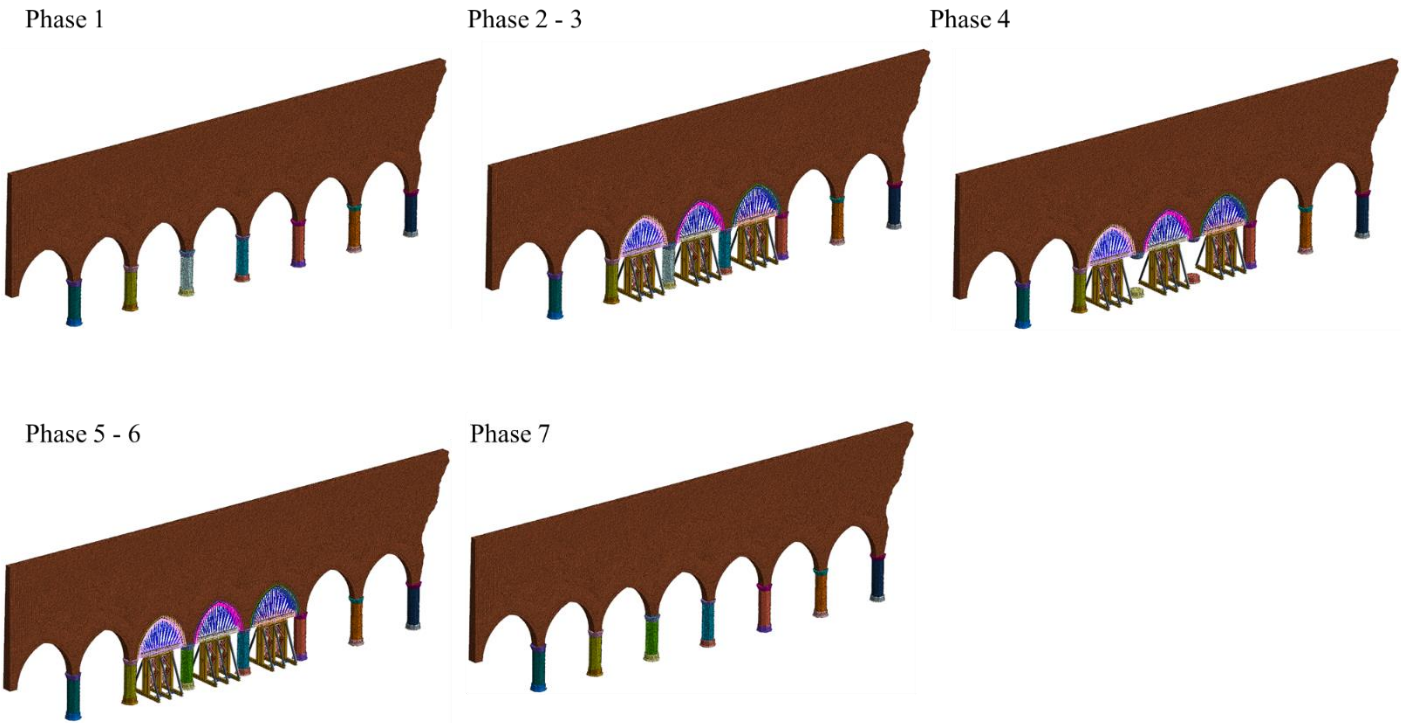

Fig. 11. Construction stage sequence of the struts-ribs solution.

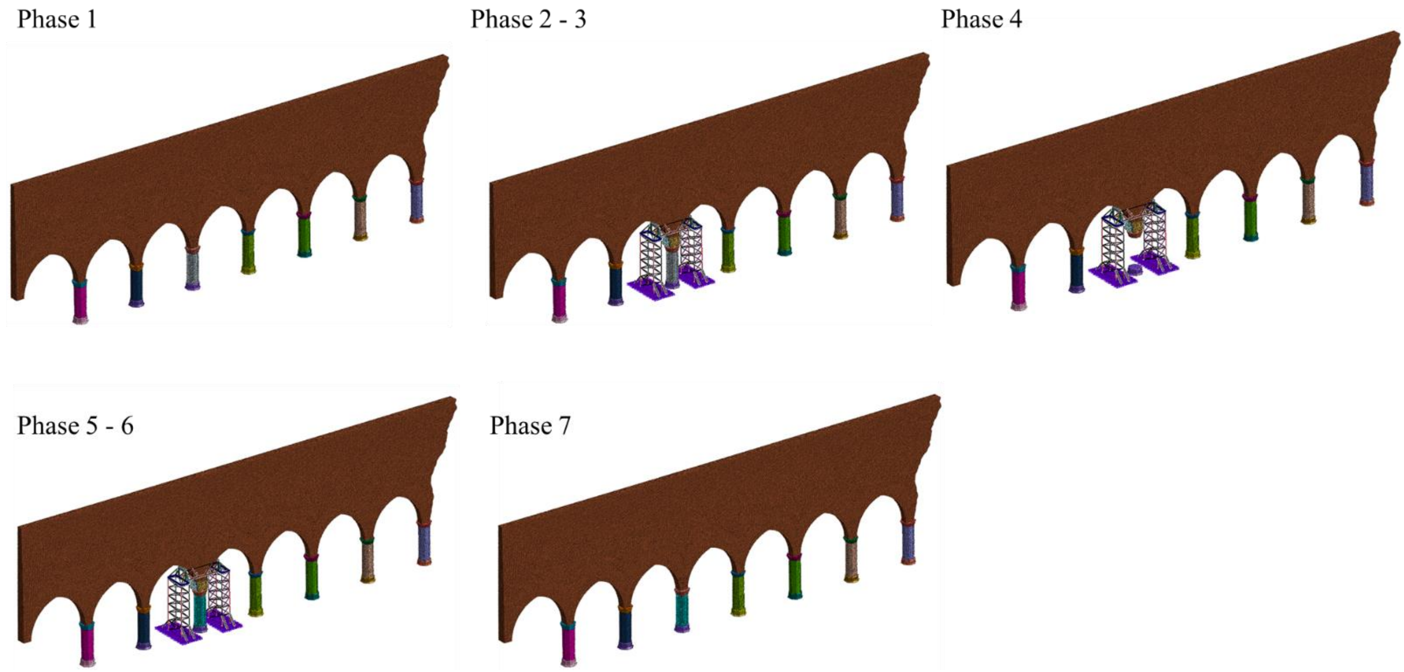

Fig. 12. Construction stage sequence of the clamping system.

Fig. 13. The clamping system: disassembling of the column.

5. DISCUSSION OF THE RESULTS

The results obtained from the nonlinear FE construction stage analyses confirmed the feasibility of the proposed repairing procedures. The vertical base reactions versus construction phases diagrams of the supporting steel frame, together with the unloaded column (P12) and the other two columns beside it (P11, P10), are represented in Fig 14, for both the struts-ribs system and clamping system. Notice that, in Fig. 14a, the curve of the steel frame represents the summation of the reactions of the three steel frames simultaneously present, divided by two in order to have a comparable "per column" value.

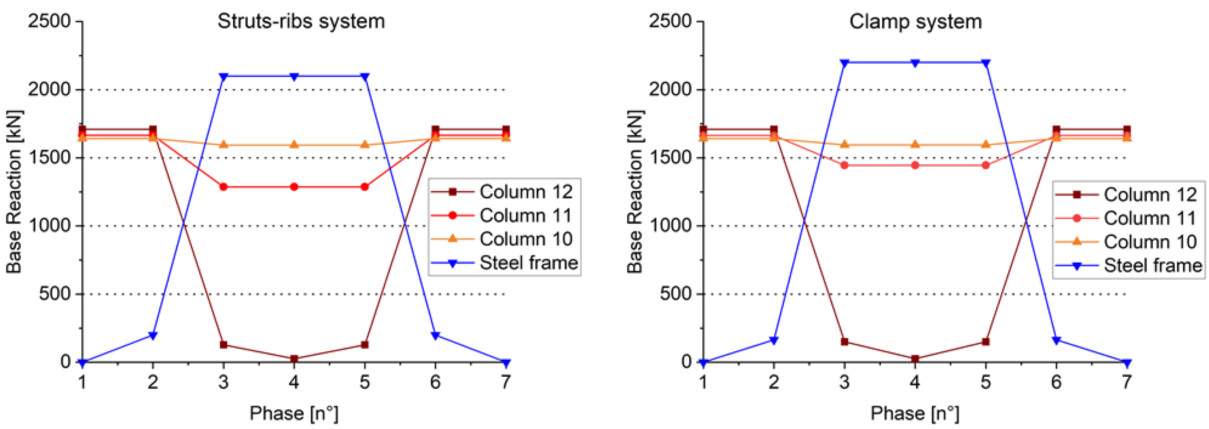

Fig. 14. Base reaction trend for steel frame and columns (P10, P11 and P12) during the unloading-reloading process: (a) struts-ribs system, (b) clamping system.

It is clear that also the reactions evaluated at the basement of the columns adjacent to the unloaded one decrease, especially using the struts-ribs system. On the contrary, considering columns far away from the column involved in the retrofitting intervention, this phenomenon does not occur.
Going more in detail, during the whole unloading-reloading process, the stress fields of the masonry elements do not exceed strength limits of the materials, i.e. the model behaved within the elastic range at any step of the construction stage analyses. The maximum tensile stress on the stone masonry material of arches and abutments is about 1.5 MPa for the struts-ribs system and 1.3 MPa for the clamped solution. These values are indeed significantly lower than the tensile splitting strength of the stone blocks, assumed at 10% of the compressive strength (e.g. (JCSS 2011)). In the upper portion of the nave wall, made of rubble masonry, the maximum tensile stress over the entire retrofitting process is 0.1 MPa (Fig. 15) and 0.12 MPa (Fig. 16) for the struts-ribs system and clamped system, respectively. These values are significantly lower than the tensile strength of the material, assumed as 0.2 MPa. The vertical state of stress inside the investigated column (P12) is reported in Fig. 17 (struts-ribs system) and 18 (clamping system), by considering phases 1, 3 and 6. It can be observed that the vertical stress field of the column is not significantly affected by the unloading-reloading process. In addition, the comparison between Phase 1 and Phase 6 shows that a more uniform distribution of the vertical stresses over the cross section of the column can be highlighted at the end of the intervention, thanks to the higher mechanical properties of the new inner core.

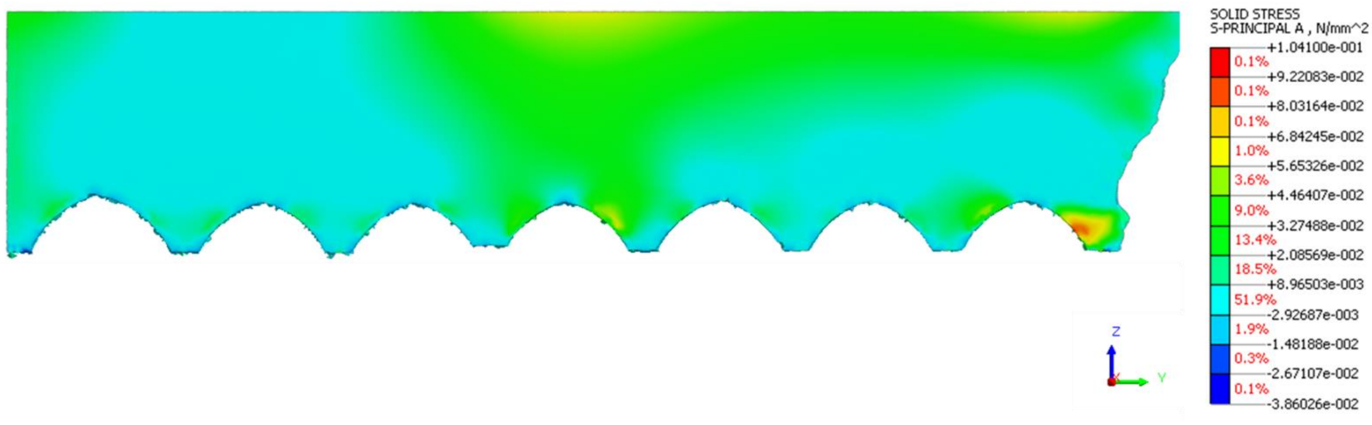

Fig. 15. Struts-ribs system: maximum tensile stresses acting on the wall (phase 3).

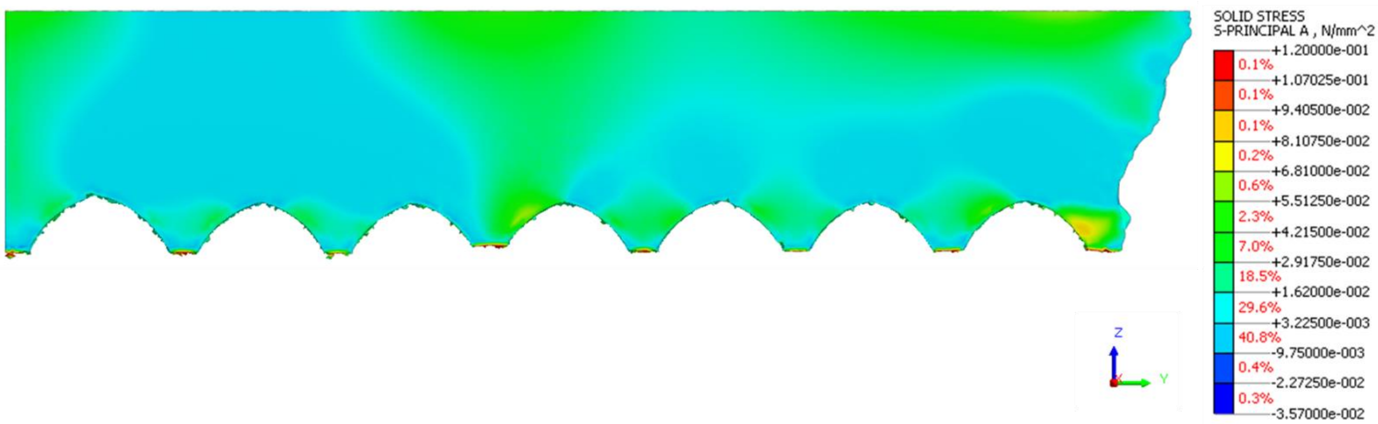

Fig. 16. Clamping system: maximum tensile stresses acting on the wall (phase 3).

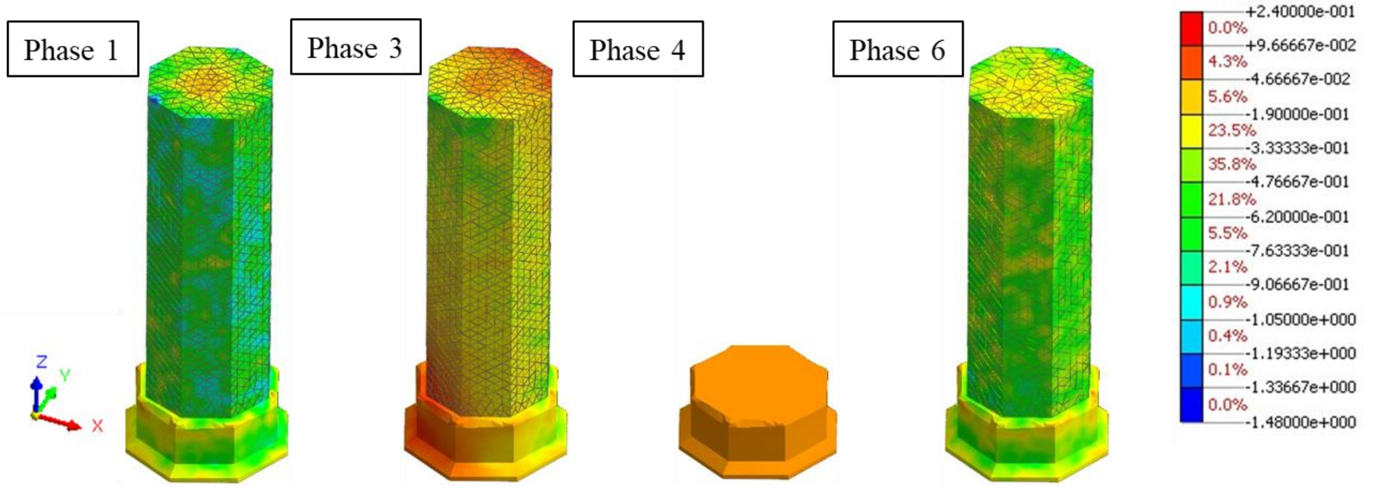

Fig. 17. Struts-ribs system: vertical stresses on the investigated column (P12) during phases 1, 3, 4 and 6.

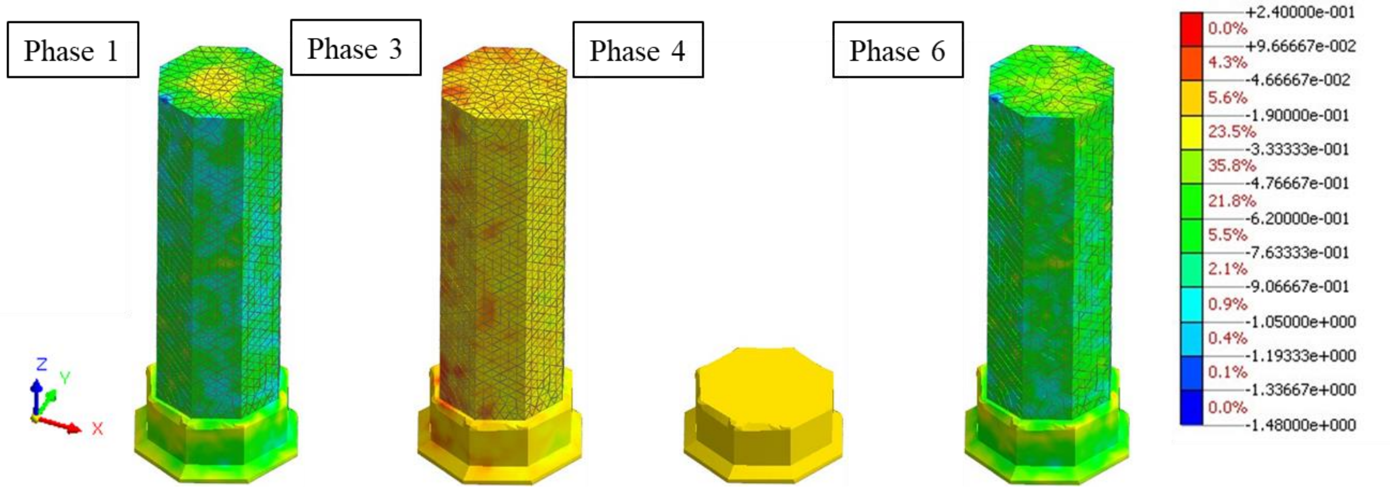

Fig. 18. Clamping system: vertical stresses on the investigated column (P12) during phases 1, 3, 4 and 6.

An additional structural check was carried out for the clamping system to verify the state of stress of the clamped portion of the masonry abutment (Fig. 19). In this case the focus is on phase 2 in order to check the effects of the post-tensioning loads applied to the horizontal ties of the clamp.

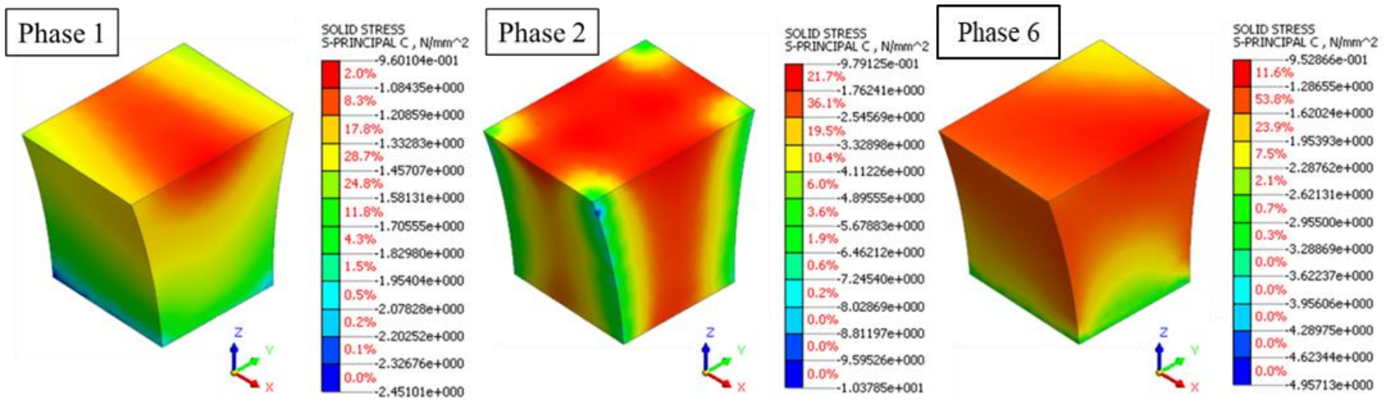

Fig. 19. Clamping system: principal compressive stresses of the masonry abutment (phases 1, 2 and 6).

As previously mentioned, the bidirectional-horizontal compression self-stresses applied by the clamp should generate an adequate friction capacity in the vertical direction. At the same time, however, they must not damage the masonry abutment by exceeding its compressive strength limits. The maximum compressive stress evaluated during the unloading-reloading process is about 10.38 MPa (Phase 2), which is consistently lower than the average compressive strength of the regular stone masonry elements (66.68 MPa).

6. COMPARISON BETWEEN NUMERICAL RESULTS AND MEASURED VALUES

As discussed in Section 5, both unloading-reloading systems represent a valid solution to intervene on the historical masonry columns of the church. For construction site and operational reasons related to the Basilica di Collemaggio project, it was eventually decided to adopt the clamping system. On site, the clamping steel frame was instrumented with a set of transducers in order to monitor the real behavior of the system during the various stages of the unloading/reloading process. Thanks to the availability of the acquired data of this set of instruments, a comparison between numerical results and in-situ measurement collected during the unloading process of the considered column was carried out. As an example, the vertical displacement evaluated at the midpoint of the horizontal truss of the steel frame was considered for the comparison. Five load sub-steps were implemented in the numerical model, corresponding to the actual load steps implemented to reach the complete unloading of column P12 through the hydraulic jacks. Fig. 20 clearly shows the good agreement between the results obtained by the numerical model and the on-site measurements, with an average error of about 3%. Moreover, as expected, the trend of the force-displacement curve is almost linear.

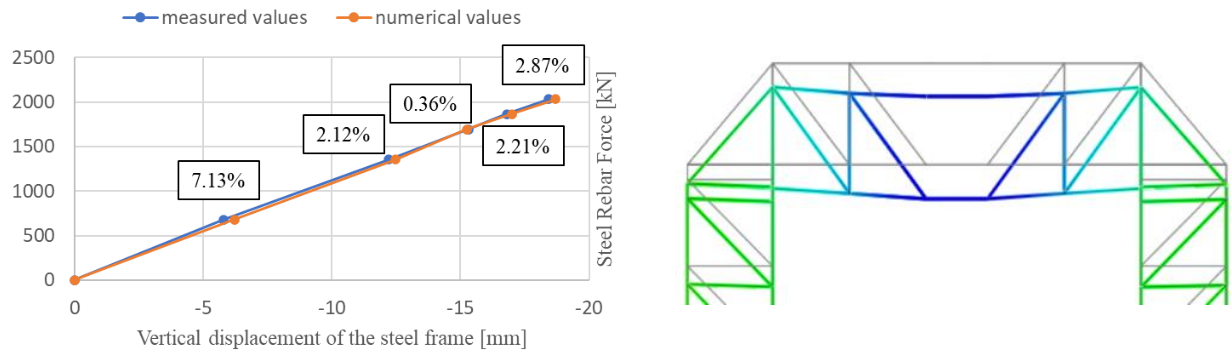

Fig. 20. Numerical vs. experimental comparison of vertical displacement of the steel frame.

Finally, it is observed that the numerically estimated unloading force is 2067 kN while the corresponding in-situ value is about 2000 kN.

7. CONCLUSIONS

In this paper, two different solutions to carry out the unloading-reloading of historical masonry columns have been discussed with reference to the S. Maria di Collemaggio Basilica. In this project, the unloading-reloading activity represented a challenging but fundamental engineering task to execute the post-earthquake repairing interventions. The first system (struts-ribs) is designed to unload two central columns within three adjacent arches by directly pushing on the intrados of the

arches with a system of struts and ribs. The second system (clamping system) works on a single column by supporting the abutment of the arches with a post-tensioned steel clamp positioned around the abutment section of the arches. In this configuration, the column is progressively unloaded by pulling up the clamp with suspension bars.
The feasibility of these two solutions was checked by implementing a nonlinear FE construction stage analysis that reproduce the unloading-reloading process in full. The numerical results have confirmed the validity of the proposed solutions. In fact, the modification of the state of stress generated by the unloading-reloading procedure remains within the material strength limits. In addition, the in-situ measurements collected during the implementation of the clamping system show a very good matching with the numerical predictions.
Furthermore, it should be remarked that the results of the unloading-reloading process here presented for column 12 are also applicable to the other columns, including the lateral columns interested by the stitching technique.
In conclusion, the solutions and analysis methods proposed in this work represent a useful reference for engineers involved in the repairing of historical masonry columns damaged by earthquakes.

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