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SEISMIC VULNERABILITY ANALYSIS OF S. SEBASTIANO CHURCH DAMAGED DURING THE 2016 CENTRAL ITALY SEISMIC SEQUENCE

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Abstract

The seismic activity that occurred in Italy in recent decades has highlighted the high seismic vulnerability of churches, which represent a good percentage of the Italian monumental building heritage of high historical, artistic, and architectural value. Churches are particular types of buildings that present a high level of vulnerability due to their configuration. Various research, regarding the analysis of damages that occurred in such structures, have highlighted the importance of studying their structural shape and whether the strengthening interventions are effectiveness or not. In particular, a survey campaign of various structures hit by the three seismic sequences of Norcia 1979 (Mw 5.8), Colfiorito-Annifo 1997 (Mw 6.0), Norcia 2016 (Mw 6.5), in the Umbria-Marche region, have been performed. Similarly, to what has been done for other churches hit by the above-mentioned seismic sequences, the church of San Sebastiano was modelled and studied with both linear and non-linear methods. The paper presents the results obtained from the numerical analysis performed with DE and FE methods and a comparison of the damages observed in relation to the past reinforcement interventions.

Keywords: Masonry Churches, Seismic Vulnerability, Numerical Analysis.

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1 INTRODUCTION

In the past years, literature extensively explored the topics of seismic vulnerability and mitigation interventions, with the purpose of developing procedures to assess and preserve structures against seismic events.

These analyses assume particular importance if related to buildings, such as churches, of high historical, architectural, and cultural value.

The study herein reported belongs to a wider project, started in 2019, regarding the large-scale vulnerability analysis of churches in Central Italy [1]. Specific case studies were selected on a dataset of 900 churches stricken by the 2016 seismic sequence, where high structural damages have been detected [2, 3]. Various evaluations of interventions have been carried out with the aim of analysing their effect on the seismic response [4, 5].

In detail, this paper illustrates the preliminary results of the analysis performed on San Sebastiano's church, located in Castel Sant'Angelo sul Nera (MC). Starting from a general description of the strengthening interventions applied to the churches located in the same municipality, non-linear static analysis, based on Finite Element and Discrete Element methods (FEM, DEM), were carried out to evaluate the structural behaviour of the church in both unreinforced and reinforced cases. In detail, the reinforced model matches with the interventions executed on the church after the 1979 earthquake and therefore it aims also at evaluating the effectiveness of these solutions. This research aims to study the particular response of the churches and the influence of the reinforcement interventions.

2 SAN SEBASTIANO'S CHURCH

San Sebastiano's church (Figure 1), located in Castel Santangelo sul Nera, was built in Romanesque style during the 16^{th} century by the architect Pietro da Tolentino. The structure consists of a rectangular plan with a wooden gabled roof, a lateral chapel and a sacristy. The church is 13.10 m length and 11.53 m width, with a height of 9.90 m. A single round arch and a second triumphal arch divide the central nave from the presbytery area. A semispherical dome crowns the semicircular apse area (with a radius of 3.00 m). Laterally, there is a small chapel (4.30 m x 3.60 m) with a lowered barrel vault. The perimetral walls are made by squared stone blocks: the main facade (west direction) has a carved stone portal and a medium-sized oculus while the longitudinal wall to the south is characterised by a secondary door and a small window. The wooden roof has two thrusting beams on the facade located on the diagonal from the center of the arch towards the corners. To counteract that thrust and the one provided by arches, a series of tie rods are arranged in the two main directions. The thrust of the arches is also borne by two tie rods that act as a contrast to the out-of plane mechanism of the side wall. Finally, the roof of the side chapel is characterised by a masonry vault.

The structural geometry was deduced from the documents provided by the Superintendence for Cultural Heritage of Ancona and from on-site inspections; while, for the characterization of the materials, it wasn't possible to carry out a diagnostic campaign. To overcome this, the Masonry quality Index method [6] was used to define the typologies of the masonry walls and their mechanical properties [7] (Table 1), according to the Italian National Standard [8]. However, since the investigations are limited, the confidence factor calculated according the Italian guidelines for cultural heritage [9] is equal to 1.24 and the level of knowledge reached is KL1.

Figure 1: View of the west facade (left), inside view of the apse (right).

MASONRY TYPOLOGIES	$f_{m,d}$ (N/mm^2)	$\tau_{0,d}$ (N/mm^2)	E (N/mm^2)	G (N/mm^2)	w (kN/m^3)
Ashlars roughly worked rubble masonry	2.0	0.043	1230	410	20
Stone blocks squared masonry	5.8	0.090	2800	950	22

Table 1: Mechanical properties of the masonries used.

2.1 Structural reinforcements related to previous interventions

The analysis of churches response of Castel Sant'Angelo sul Nera carried out on the wide project Reluis 2019-2021, have pointed out a strong correlation between the vulnerability of these particular structures and the consolidation interventions carried out in past years. In detail, for similar seismic actions (between 0.52g and 0.54g), the damages detected in a cluster of 19 churches distributed in the municipality territory are quite different, as is the Damage Index (i_d) evaluated by the A-DC form [10, 11, 12]. This scattering of the index i_d might be attributed to the different vulnerabilities of constructions, especially considering the behavior of the different strengthening interventions performed and not present in all cases.

Different reinforcements were identified and evaluated in terms of structural behavior. The outcomes distinguish some positive and negative interventions. However, for other interventions, like the ring beam, it is difficult to define a univocal judgment. Tie rods and metallic ring beams prevent the out-of plane mechanisms of the wall and guarantee the box-like behavior of a masonry wall with good quality [4], while the reinforcement of the concrete roof slabs decreases the capacity of the structure due to the increased of loads and stiffnesses.
The church of San Sebastiano, like San Martino dei Gualdesi [4], was subject to two restorations following the seismic history of the Valnerina (Norcia 1979 and Foligno/Colfiorito 1997). The state of damage detected after the 1979-Norcia earthquake was characterised by various failures and cracks on the global structure. The facade (West) was severely damaged, and the wooden roof and the central window were characterised by various cracks. Moreover, the lintel of the south arch presented important cracks with expulsion of stone voussoirs. The proposed reinforcement interventions were the following: replacement of the roof with the insertion of a reinforced concrete curb on the upper part of the perimetral walls, arrangement of the rose window, restoration of the lintel of the door on the south elevation and execution of cement injection with additives on the cracks.

The damages detected following the 1997 earthquake were similar to those caused by the previous earthquake: instability of the roof; damages on the facade, expulsion of masonry ashlars; cracks near the intersections between the lateral wall and the apse.

In this case, the restoration project included:

- repair and arrangement of the roof with a reinforced concrete curb inserted inside the masonry;
- replacement of deteriorated wooden parts; waterproofing with sheathing of the pitches and laying of the mantle of roof tiles;
- restoration and sealing of the facade mortar joints;
- injections on the lateral walls;
- insertion of tie rods on the facade in addition to the replacement of the pre-existing tie rods positioned on the triumphal arches.

Figure 2 illustrates the interventions previously described, with a focus to: restoration with lime mortar of masonry joints on the facade (top left), the *cuci-scuci* technique in the south elevation (top right) and tie rods (plan view).

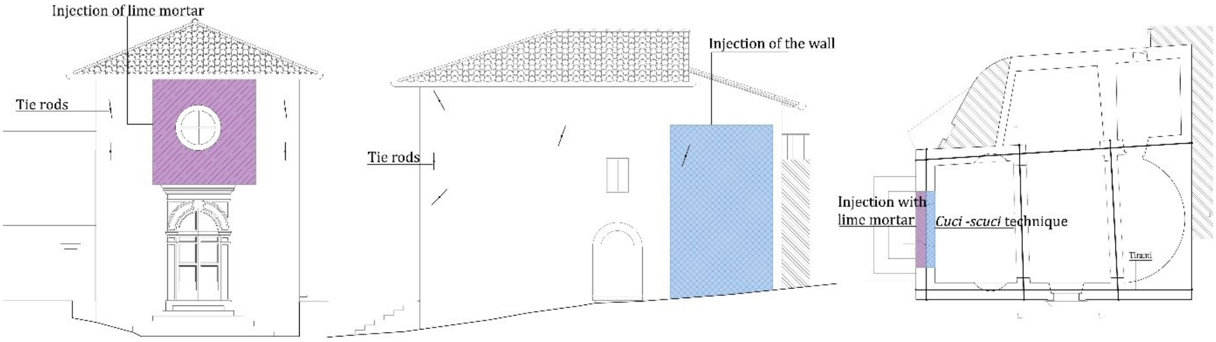

Figure 2: Representation of interventions implemented in 2014 and 2000 to the San Sebastiano’s church.

In order to objectively consider the seismic actions that led to the observed damage, the seismic data collected by the National Accelerometric Network (RAN) of the civil protection identified in "Castelsantangelo sul nera" (CNE) were analysed (https://itaca.mi.ingv.it/). In this case, referring to the seismic sequence of 26 and 30 of October, with magnitude (Mw) between 5.4 and 6.5, the maximum spectral acceleration referred to the seismic action occurred in the E-W direction.

By analysing the current state of damage (Figure 3), the church of San Sebastiano is characterised by various damages. On the outside part, significant cracks are distributed on the south wall and especially in the corner between the facade and the longitudinal wall. At a height of 5.0 m, and in correspondence with the side wall of the apse, a swelling of a wall part can be identified. Finally, a wide crack is visible on the north wall of the sacristy. The internal part of the church is similarly widely damaged. The triumphal arch, placed next the apse, has completely crumbled. This mechanism is probably due to the steel beam located transversally to the arch: the vertical loads of the roof are distributed on the external constraint of the beam without acting as a stabilizing vertical action. On the contrary, on the lateral piers, any cracking mechanisms or widespread damage have been identified. This proves the effectiveness of the past interventions. On the central arch, a similar damage pattern has been identified: the pilasters are well connected to the perimeter masonry, but there are still cracks both in the keystone of the arch and laterally, just above the level of the impost. Finally, the semi-dome and the apse show significant cracks.

By analysing the damage pattern of the church, the past interventions have played a fundamental role on the seismic response of the structure both in terms of structural improvement and deterioration. The two level of tie rods arranged transversely to the facade have ensured a good connection between the lateral walls, avoiding the possible out-of plan mechanism of the facade with part of lateral corner. The perimetral ring beam and the lateral injections could have promoted the box-like behavior of the church nave.

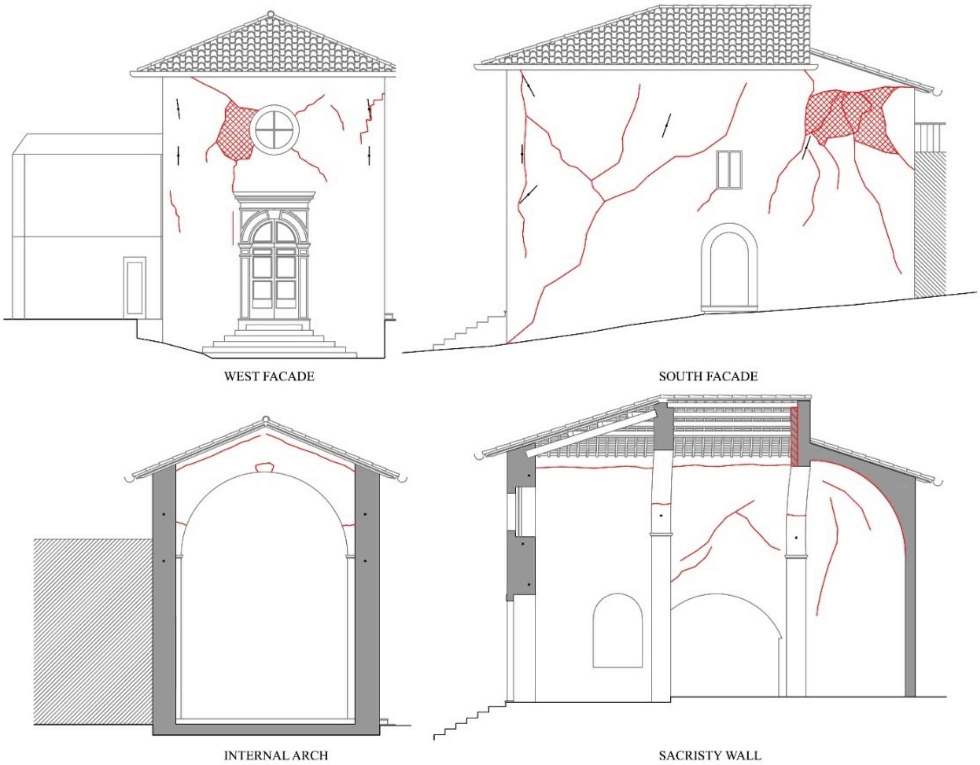

Figure 3: Damages observed on the south and west facades (up). Cracks detected on the internal arch and sacristy wall (down).

3 STRUCTURAL ANALYSIS

The analysis of the seismic response of the church was carried out by implementing a preliminary kinematic analysis (KA) and a non-linear static analysis (NLS) of the structure. For the NLS analysis, the models implemented consider both the hypothesis of the absence and presence of intervention.

The parameters of the seismic action used [4] on the above-mentioned analysis are determined by following the indications provided by the [13]. The nominal life (V_N) is equal to 50 years for a type 2 construction. The structural class, as usual for ecclesiastical buildings, is III with an importance factor equal to 1.5 and a return period of 712 years.

3.1 Kinematic analysis

Starting from the crack pattern analysis of the church, two out-of-plane and one in-plan mechanisms were identified (Figure 4). In detail, the kinematics analysis concerns the simple overturning of the west wall (M1), the horizontal bending of the facade (M2) and the shear mechanism of the south wall (M3). The verification of the analysis refer to the Italian Standard code for construction [8]. The factor ξ represent the safety coefficient defined as the ratio between the seismic capacity and the demand for each macro-element. Values greater than one indicate that the mechanism is verified at the damage limit state (DLS) and at the ultimate limit state (ULS). The M1 mechanism was analysed considering the following hypothesis:

1) Absence of ring beam and tie rods;
2) Presence of the ring beam and absence of tie rods;
3) Presence of both interventions.

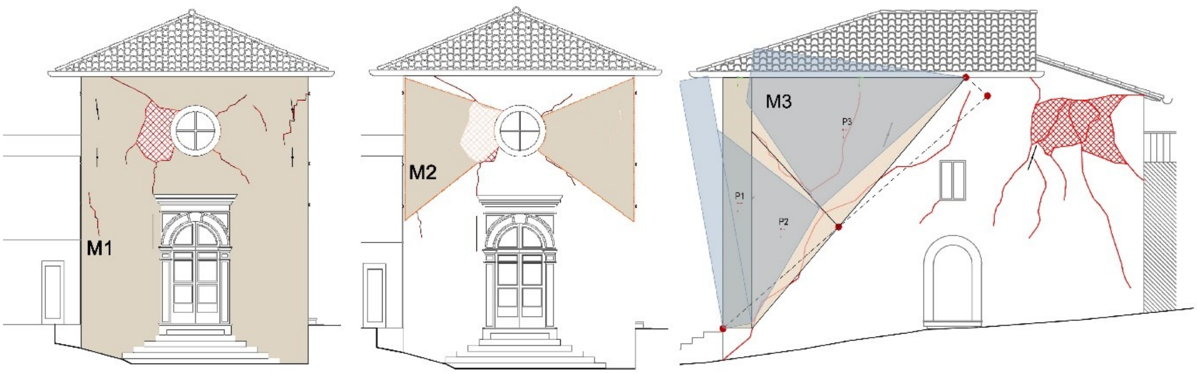

Figure 4: Definition of the macro-elements analysed.

The linear verification for M1 macro-element is not verified in case of absence of ring beam and tie rods and with the presence of the ring beam and absence of tie rods. The only mechanism verified at the ULS, is the kinematic characterised by both the strengthening interventions. Regarding M2 and M3 macro-elements, the mechanisms are verified at both the DLS and ULS, without any type of interventions (Table 2).

M1 – Simple out-of plan mechanism			
	α (t/2)	**ζ (DLS)**	**ζ (ULS)**
No interventions	0.083	0.32	0.32
Ring beam	0.082	0.33	0.32
Ring beam and tie rods	0.258	1.01	1.00
M2 – Horizontal flexion mechanism			
	α (t/2)	**ζ (DLS)**	**ζ (ULS)**
No interventions	0.505	1.65	1.63
M3 – In-plane shear mechanism			
	α (t/2)	**ζ (DLS)**	**ζ (ULS)**
No interventions	0.578	2.36	2.33

Table 2: Kinematics of M1, M2 and M3 macro-elements.

3.2 FEM analysis

The numerical analysis of San Sebastiano follows the procedure analysis applied for the case study of San Martino dei Gualdesi church [4]. In detail, the case study was modelled with DianaFea software and analysed both with linear and non-linear static analysis. The model of San Sebastiano was defined through three-dimensional quadrilateral elements with 8+8 nodes (BQ24S8) and through beam elements characterised by three nodes (CL18B). The mesh size was set as 0.25 m. The mechanical parameters were defined through the previous investigations (Masonry Quality Index analysis), as well as the thicknesses and the geometries of the elements. The constitutive model adopted is the Total Strain based Crack model, with an exponential function for the tensile behavior and a parabolic function for the compression behavior. In detail, the energy parameters have been obtained from the literature [14] while the resistance parameters refer to Table 1. The roof of the model was simplified with a two-dimensional element with a very limited plane resistance and its weight was inserted as a distributed load up to the walls. Finally, the boundary conditions were defined with hinges at the base.

In order to better analyse the seismic response of the structure, with particular attention to the current state of damage of the church, important consideration on the model were done: first, an *unreinforced model (URM)* without the strengthening interventions was implemented; then, the model with *tie rods* was defined.

Regarding the model without intervention (*URM*), a linear static analysis of the vertical loads was performed to check the overall weight of the structure, with a global vertical reaction of 7284 kN. The vertical deformation, due to its own weight is located mainly on the roof (1.3 mm). After this, a non-linear static analysis of the model was carried out. The own weight of the structure was applied in a single load step, followed by the application of a global acceleration (proportional to the masses) into different load steps. The main directions investigated are the south and west direction.

The analysis referred to the transversal direction (south) shows the bending mechanism on the transversal wall (Figure 5, right) due to the thrust of the central arch with a capacity coefficient equal to 0.17. By implementing the non-linear static analysis in the west direction, the deformations are concentrated on the central facade opening and between the facade and the longitudinal walls (Figure 5, left). In this case, the limited tensile strength of the masonry wall leads to the overturning of the facade. Equally critical are the area between the arch of the apse and the adjacent dome. In this case, the α coefficient increase to 0.24.

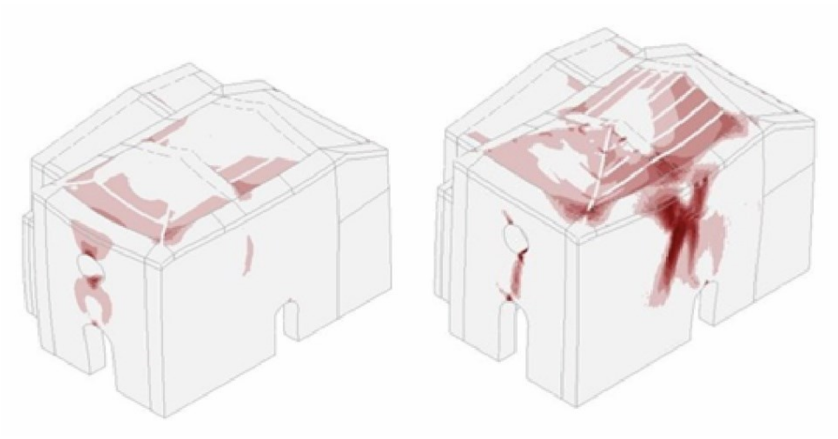

Figure 5: *URM.* Principal deformations for the analysis in west (left) and in the south (right) directions.

The model with tie rods interventions was modelled with beam elements with diameter of 30 mm at a height of 7.0 m and 5.3 m. The positions of the tie rods consider both the interventions made it in different periods. The properties of the tie rods follow the elastic function. By applying the horizontal action to the south direction, the damage pattern of the structure in terms of deformations is similar to *URM* (Figure 6, right). The capacity coefficient α is equal to 0.23. The tie rods intervention shows good results in terms of deformations in the analysis with the horizontal action to the west (Figure 6, left): the crack pattern better agrees with the real state of damage. The structure capacity reaches an α coefficient equal to 0.30. In this case, the out-of-plane overturning of the facade is partially inhibited by the longitudinal tie rods. Another effect of this intervention is the identification of the vertical and diagonal cracks on the lateral wall, starting from the limit position of the ring beam.

Regarding the facade, as for the unreinforced model, the activation of the out-of-plane flexural mechanism, which affects the rose window above the door, is observed.

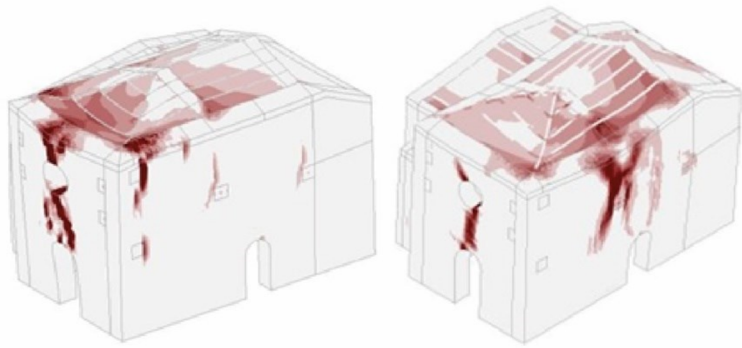

Figure 6: *Model with tie rods*. Principal deformations for the analysis in the west (left) and south (right) directions.

3.3 DEM Analysis

The third modeling approach used to study and evaluate the response of San Sebastiano's church, involved the use of the discrete element method (DEM). The model was defined with rigid, and non-deformable, blocks and interfaces in order to reduce the computational efforts of the analysis and to evaluate the masonry characteristics in the most realistic/correct ways. The 3D-model geometry was first created through the Rhinoceros® software and then imported in 3DEC. Regarding the mechanical properties, only the dead weight was assigned to the single block. The parameters referred to the tensile and compressive behavior of the masonry blocks were applied on the interfaces: the compressive resistance was assumed infinite and the tensile strength (j_{tens}) was defined according Table 3. Finally, following the Mohr-Coulomb constitutive law, the friction coefficient μ was defined equal to 0.567 (angle of friction of 30°), and the cohesion coefficient j_{choe} was calculated through the average shear strength τ_0 [8].

	Unit	Wall blocks	Voussoir
J_{kn}	[Pa/m]	5.348×10^9	1.239×10^{10}
J_{ks}	[Pa/m]	1.78×10^9	4.13×10^9
μ		0.567	0.567
J_{tens}	[N/mm^2]	0.084	0
J_{choe}	[N/mm^2]	0.056	0

Table 3: Interface properties of the model.

To constrain the model, a specific interface, schematized as a platform, was defined between the church and the ground. In this case, the interface was modelled with zero tensile strength and zero cohesion, against a high friction value to avoid the effect of sliding and to not overestimate any non-real soil retention effects. The DEM analysis herein reported consider a pseudo-static approach based on vertical loads follow by an incremental horizontal load (to the west and south directions). For a better control of the model, multiple control points were identified.

In order to highlight the behavior of the structure and to compare the kinematic and FEM analysis previously described, the model without reinforcements *(URM)* and the model with the tie rods interventions were analysed.

By applying the acceleration in the south direction to the *URM*, a bending of the longitudinal wall is observed, with an important thrust of the central arch (the closest to the west facade) probably due to the greater area of influence. Moreover, an important diagonal fracture is observed on the facade (Figure 7, right), starting from approximately the center rose window and ending on the south side of the entrance opening. This crack suggests an outset of overturning of the south wall. The capacity coefficient is equal to 0.14. Regarding the analysis in the longitudinal direction (toward west), the main deformations are located in the longitudinal south wall. In detail, two cracks are identified: one close to the wall and the other starting from the closest arch. An evident overturning mechanism of the facade is observed (Figure 7, left) with an α coefficient, equal to 0.25, similar to the FEM unreinforced model and equivalent to the in-plane mechanism M3.

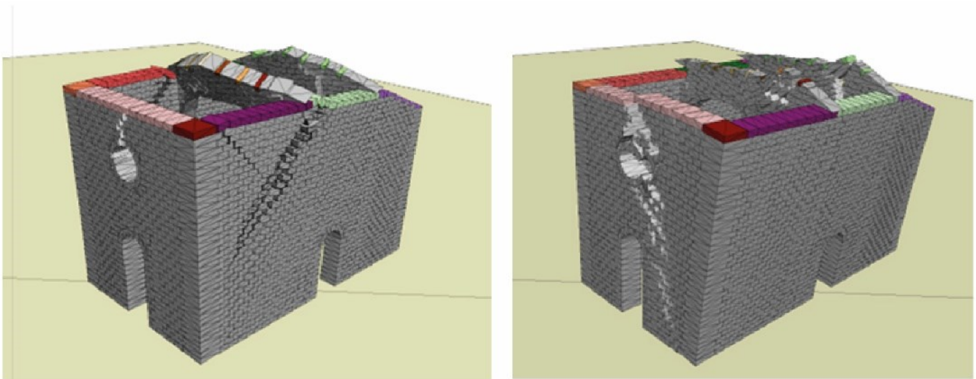

Figure 7: *URM*. Representation of the displacements for the analysis in the west (left) and south directions (right).

Regarding the reinforced model, the tie rods were defined by following the FEM model dimensions. However, due to geometry incompatibilities within the 3DEC software, the chains were modelled with a rectangular section. With the horizontal action acting towards the south, the deformation is located on the upper part of the south wall. In this case, the transversal tie rods provide an effective constraint, by increasing the mechanism capacity of the facade and then activating the overturning of the masonry central portion (Figure 8, right). The α coefficient is equal to 0.23. Due to the horizontal west action, the mechanism observed is the overturning of the facade with its lateral wedge (Figure 8, left). The α coefficient is equal to 0.32.

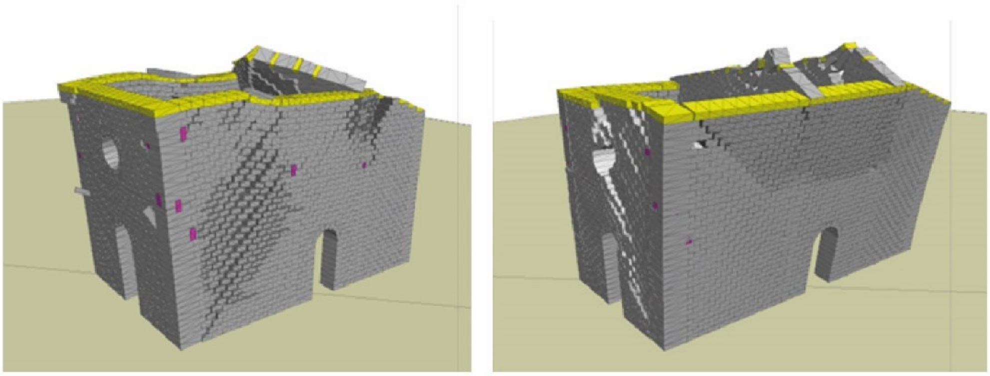

Figure 8: *Model with tie-rods*. Representation of the displacements for the analysis in the west (left) and south directions (right).

3.4 Comparison of the results with the damage observed

The results obtained from the analysis previously described were compared among them and with the state of damage of the church (Figure 9). Considering the UR model, the representation of the church before the strengthening interventions, the deformations identified are mainly located on the central arch and on the apse. In this case, the flexural out-of-plane mechanism of the arch could have acted orthogonally to the wall and the poor quality of the internal walls could not have guaranteed the monolithic of the damaged macro-elements. In this case, both type of modelling demonstrate the vulnerability of the central arch that activates first. However, the DEM model is more affected by interlocking between blocks with the consequent onset of shear cracks: on the central part of the facade and on the lateral south corner.

The reinforced models with the tie rods have shown greater similarities to the state of damage. It is interesting to observe how the introduction of chains does not prevent the cracks near the apse since are anchored only to the second arch and not until the end of the wall. Moreover, the force acting in the tie rod, produced a shear diagonal crack on the south wall. On the apse area, the different masonries, in terms of mechanical properties, could have influenced the activation of the mechanism. Although there are slightly different damage mechanisms between the modeling methods, the results obtained for the west horizontal action are totally in agreement with the real crack pattern. In detail, the vertical deformation next to the anchor chains and the diagonal crack on the south wall are very similar to the crack observed in site, as the expulsion of masonry on the apse area. Taking into account that the seismic action during the earthquake had the E-W horizontal component as principal, all of these results find additional confirmation.

Figure 9: Current state of damage of the church.

In addition to the observed agreement on the crack pattern and the overall structural response, the numerical results (Table 4) of the DE and FE models demonstrated a strong correlation between the two modelling approaches.

	Unreinforced model		Reinforced model	
	FEM	*DEM*	*FEM*	*DEM*
West direction	0.24	0.25	0.30	0.32
South direction	0.17	0.14	0.23	0.23

Table 4: Numerical results of FE and DE models.

4 CONCLUSIONS

- The paper presents the studies carried out on San Sebastiano church, by demonstrating that the use of various type of approaches (i.e. kinematic, Finite Element and Discrete Element models), lead to achieve reliable results, compared to real on-site observations.
- The kinematic analysis, in the presence and absence of reinforcing interventions, demonstrated to provide a good definition of the most vulnerable mechanisms and in defining the activating accelerations.
- The static non-linear analysis, through the Finite Element Method, proved to be a reliable method on detecting a possible damage pattern, and to be a valuable structural modeling in defining the structure capacity.
- The Discrete Element analysis, through the modeling of blocks and interfaces, as well as reflecting the properties of the masonry in a more realistic way, proved to detect a crack pattern very similar to the real one.
- The good agreement among the numerical results of the three modelling approaches demonstrates their reliability. Moreover, this comparison helps in validating results, especially when real damages are not present.
- Future works aim at evaluating the structural behavior of the church by implementing models with different type of interventions.

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