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Performance and damage assessment of 16th century St. Maria del Popolo church in Naples (Italy)

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ABSTRACT: In this paper the case study of St. Maria del Popolo church in Naples, Italy, has been analyzed to validate an interdisciplinary procedure for assessing structural behavior and seismic vulnerability of such structures. The building was seriously damaged during a bombing raid during World War II and by major earthquakes hitting the area. Invasive restorations (e.g. timber roof substitution with a heavier RC roof) were conducted after WWII and other interventions after earthquakes, but very little information was reported in the historical sources. Nevertheless, today the church is heavily damaged and some pillars are clearly not vertical. 3D FEM dynamic analyses of the structural complex and local mechanisms analyses have been carried out to analyze the effectiveness of past interventions today and to design eventual further ones. By this procedure it is possible to demonstrate the formation of main cracks primarily due to earthquakes and long term effects, and the lost of column verticality due to bombing. The reliability of the procedure is assessed by comparing monitored damages to numerical and historical information. The results open problematical issues about the strengthening of this historical building, yet respecting all its stratifications.

1 THE CHURCH DURING TIME: WAR DAMAGES AND RESTORATIONS

An interdisciplinary procedure for assessing the structural performance and seismic vulnerability of the Neapolitan church of St. Maria del Popolo agli Incurabili is presented in the present paper. The approach provides the analysis of the dynamic behavior, damage patterns and collapse mechanisms using both FEM linear analysis of the entire structure and simplified analysis at the “macro-elements” of the church, defined as the parts of the construction characterized by autonomous and unitary structural behavior under seismic actions. In particular, linear analysis is used for detecting tensile and compressive stress zones in the masonry elements and for identifying a probable class of failure mechanism for which the collapse multiplier is then calculated. All calculations took into account, primarily, the static history of the church, an accurate metric survey and the knowledge of mechanical strength of materials (tuff and mortars), deriving from in situ tests.

The church of St. Maria del Popolo, founded in the first half of 16th century (Borrelli 2002), is characterized by a single nave constructed in Neapolitan yellow tuff and concluded by a rectangular apse covered by a dome. The building nave is free from constructions on the South side and confines with chapels and the ancient hospital rooms on the North; it is scanned by six arches on each side, closed at the bottom by masonry panels corresponding to the upper slender windows. A choir, used by the nuns for assisting to religious services, loads the internal façade and it is separated from the nave by three arches, at the present closed by lapillous concrete

blocks. Finally, the whole space is enriched by stuccoes that cover all the surfaces with decorative themes which are frequent in the Neapolitan baroque art. On March 5th, 1943 the bombs directly hit the Hospital of the Incurabili in the ancient centre of Naples while it was intensely operating for lifesaving injured people from the war offenses (Russo 2010): the church of St. Maria del Popolo, enclosed into the historic hospital walls, was seriously damaged in its vertical structures and, particularly, in the roof as a rare photograph taken by Roberto Pane before 1949 testifies (Pane 1949). The effects of bombing raids were evident, as mentioned above, especially on the building wooden roof that supported a wooden coffered ceiling: damages were relevant, above all, near the church main façade on the West because of the loss of the roof beams and of the exposure of the internal space to meteoric agents. The need to operate firstly onto the hospital ruins induced the Allied military forces to postpone any intervention in the church until 1954. A new structure, made of reinforced concrete trusses connected by a slab (fig. 1), was realized in order to close the nave internal space with the scope of calling back the substituted parts with simplified forms and with the reduction of the lacunars in relation to the pre-existent ones (Russo 2010). The 20th century coffered ceiling simulates the previous one at the intrados although it does not reproduce the outline and, so, its ancient design: as it can be noticed today, the trusses' tie-beams define, constructively and alternatively, the transversal ribs of the coffered ceiling slab and they are connected by hollow beams in the longitudinal direction.
During the years following the strengthening works in 1954, the hospital administration asked for further restoration works in order to reintroduce the practice of worship in the church but these interventions, concerning mainly the restorations of the frescoes, were never performed and the church was never re-opened to the public. On November 23th, 1980 the earthquake, caused considerable damages to the city monumental heritage, imposing a “structural test” to the building as it has been modified after Second World War. The vertical trend of the cracking pattern, in fact, outlined in the pilasters, revealed a lack of resistance to seismic actions of the South wall of the nave, considering also that it is not constrained to any other adjacent building.

2 FEM LINEAR ANALYSIS

2.1 FE model

A complete three-dimensional FE model of the church structural complex has been built using commercial computer code SAP 2000. The FE modeling strategy is based on the concepts of homogenized material. In particular the walls have been modeled by means of thick shell elements, while frame elements have been used to model the trusses of the roof. The 3D model (Fig. 1) consists of 14,005 joints, 13,057 shell elements and 245 one-dimensional truss elements. For the masonry tuff elements the assumption of linear elastic behavior has been adopted, with Young modulus equal to 1000 MPa and own weight equal to 19 kN/m^3 (to account also for the plaster, coatings and stuccoes). In order to assess the safety of the church and to clearly justify the damages described in the previous section, both static and seismic analyses have been performed to understand the origin of the cracking patterns and to obtain, from a numerical point of view, the actual state of conservation of the church.

2.2 The static analysis

Linear static analyses have been performed to provide internal stress distributions and also to identify weak points of potential failure in the building. Two different models have been investigated in order to reproduce the changes occurred during the history: the first model took into account the roof made of RC members (present state); while the second one considered the presence of timber roofs (earlier state, before World War II). Two different assumptions have been adopted for addressing the effect of horizontal connection among the vertical structural elements due to the different roof solutions: model (1), with RC roof, considers the insertion of rigid diaphragms, through a kinematic constraint; model (2), with timber roof, instead is modeled with simple frame elements. In this step the structural model was subjected to its dead load only.

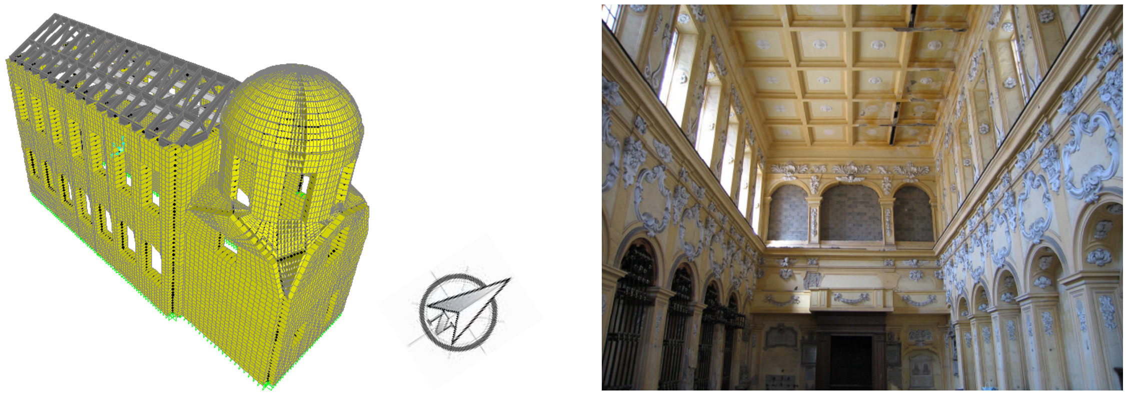


Figure 1. Naples, church of St. Maria del Popolo. Perspective view of the 3D FE model of the church, and inner view of the roof.

The static linear analyses provide the stress distribution inside the structural elements. The maximum value of the compression is reached in the columns along the South wall of the nave: the compression is about 1 MPa. This high value is long term, even if it is feasible for this masonry. As expected, the compressive vertical stresses in the case of RC roof are 20-30% higher than those obtained when timber roof was considered, but still feasible for tuff masonry. Long term effect of masonry under high compression could have led also to damage evolutions, emphasized by seismic actions occurred in the past.

2.3 The dynamic behavior

The linear investigation was extended to a modal analysis in order to give an estimation of the dynamic response of the church. Substantially this result confirms that churches, designed to withstand vertical (compressive) static loads, are not always adequate to withstand horizontal actions deriving from seismic events, which are probably the main cause of the degradation. The natural periods and the modal shapes of the two 3D FE models are provided in Figs. 2a and 2b respectively in the structure with rigid diaphragm (RC roof) and without horizontal connections (timber roof). As it can be argued from Fig. 2a, the first mode of the “as is” building (model with rigid diaphragms) involves the translation in the longitudinal direction, thus confirming that the building shows a greater stiffness in this direction. The second, third and fourth modal shapes display translation in the weakest transversal direction, with significant out-of-plane deformation of the South wall of the nave. The higher modal shapes do not depicted in the figures, involved torsional modes only. Instead the model with timber roof presents the first three modal shapes in translation in the weakest transversal direction, and the fourth modal shapes in the longitudinal direction. Also in this case the higher modal shapes involve torsional modes only. The natural periods and participating modal mass ratios of the two FE models are provided in Fig. 3. The introduction of the rigid diaphragm, in this case, influences the fundamental modal shapes and participating masses, while it seems to be not influential on periods.

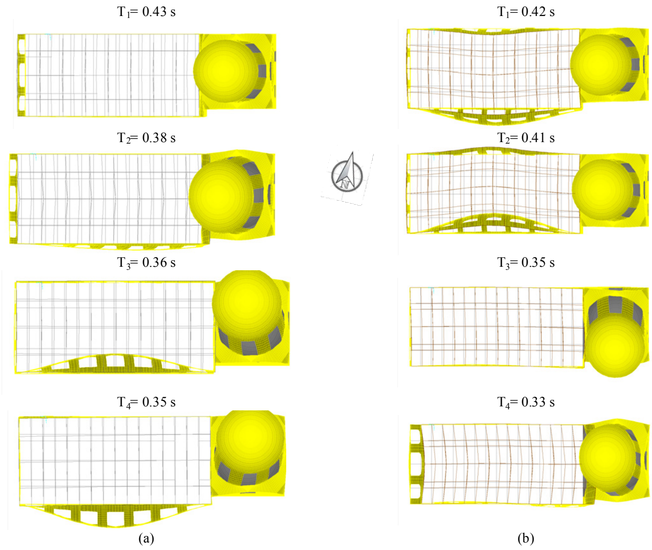


Figure 2. Modal shapes of the church: (a) reinforced concrete roof; (b) wooden roof.

2.4 The seismic analysis

In order to give a structural interpretation of the damage growth due to recent seismic events, dynamic simulations of the church (with RC roof only) are presented considering both the response spectrum registered by National Accelerometric Network at Torre del Greco, that is the closer station to the church area, during the Irpinia earthquake of the 1980 and the inelastic design spectrum provided by Italian Code (NTC '08) for that site with PGA equal to 0.167g. The inelastic spectrum is obtained from the elastic one trough a reduction factor equal to 2, which accounts for the limited ductility and deformation capacity of masonry structures. The seismic actions provided by the NTC'08 have been applied, as suggested by the same code, along the two main directions of the church in two different analyses. Seismic analysis based on response spectrum registered by National Accelerometric Network, has been run combining the three waveform components of the Irpinia earthquake. Linear-direct integration time-history analysis has been run applying ground accelerations of the Irpinia earthquake provided by National Accelerometric Network.

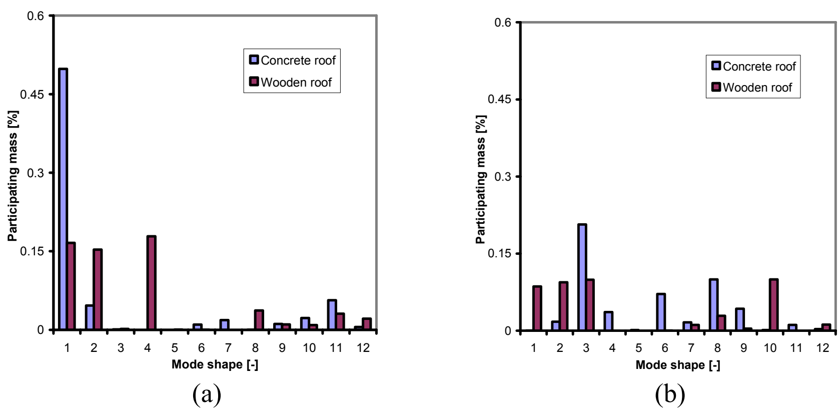


Figure 3. Linear modal analysis: (a) participating mass ratios along longitudinal direction; (b) participating mass ratios along transversal direction.

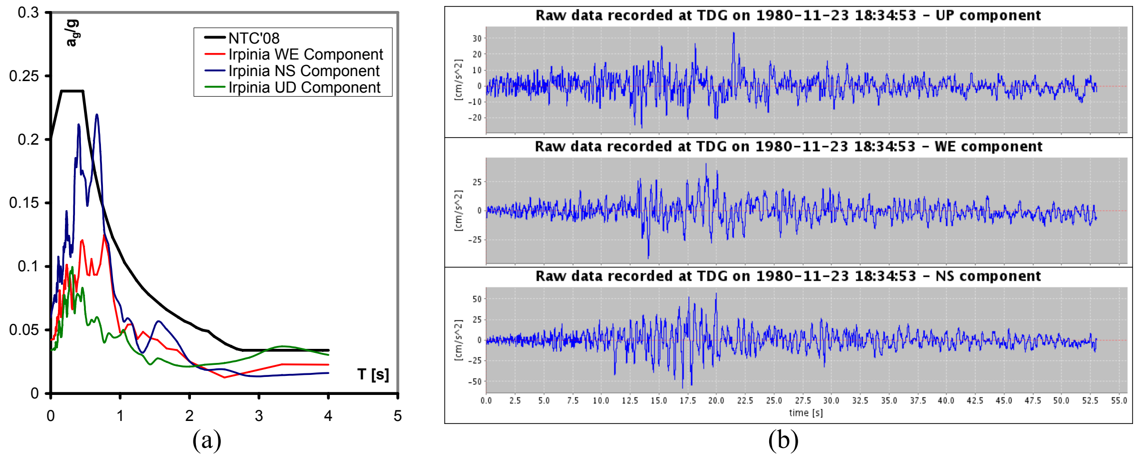


Figure 4. FE input for seismic analyses: (a) response spectrum; (b) ground acceleration.

In Fig. 4a, NTC'08 inelastic spectrum is depicted, together with the three waveform components Irpinia earthquake, while in Fig. 4b the three components of the ground acceleration are depicted. The Italian code inelastic spectrum matches quite closely the real waveform and in any case results conservative as suggested into a design phase. The comparison between the numerical stress distributions inside the structural elements and the in situ surveys of the cracking patterns shows a good agreement between the FE simulations and the actual state of conservation. Figs. 5 show this comparison and it is clear that, according to numerical outcomes, the peak stresses in compression (Fig. 5c) can be found in the central columns of the South wall of the nave, while peak stresses in tension (Fig. 5b) can be found in the keystones of the arches between the columns. It is noted that the same areas are nowadays mostly damaged: masonry crushing and crack openings can be found (Fig 5a) also due to long term effect of high sustained loads.

Fig. 6 summarizes the results of the numerical simulations providing compressive stress values achieved in the critical structural elements. It is clearly highlighted the crucial influence of the seismic actions. Out-of-plane bending in horizontal direction can lead to damages. In terms of stress increments due to bending, the stress values reported in Fig. 6 represent the maximum values across the thickness of the wall. Furthermore, local mechanisms can be activated by out-of-plane actions leading to further damages; this aspect is specifically discussed in next section. RC trusses present small stresses: the only damages nowadays are related to rain water leakage.

3 ANALYSIS OF LOCAL MECHANISMS

The seismic collapse of a masonry structure is often due to a loss of equilibrium of a structural portion (macro-element), rather than attaining maximum strength of the materials. The limit analysis is a particularly effective tool for estimating the collapse load of macro-elements providing a bound of the horizontal capacity. Under the following assumptions on the masonry behavior: (i) no tensile strength, (ii) infinite compressive strength and (iii) absence of sliding at failure; the masonry material becomes an assemblage of rigid parts, held up by mutual pressure, and collapse of the structural elements is characterized by the development of non-dissipative hinges transforming the structure into a mechanism. The term "mechanism" indicates a displacement distribution in the structure produced by inelastic deformations (the formation of hinges) which occur in a finite number of sections due to disconnections and cracking.

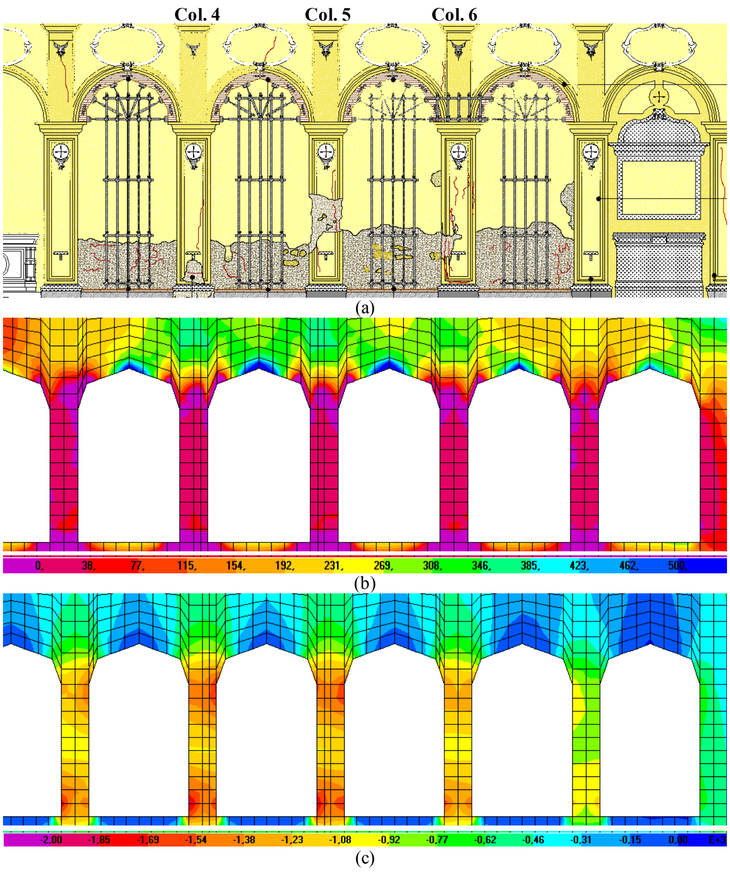


Figure 5. Comparison between numerical analyses and state of conservation of nave: (a) cracking patterns; (b) tensile stress (kPa); (c) compressive stress (MPa).

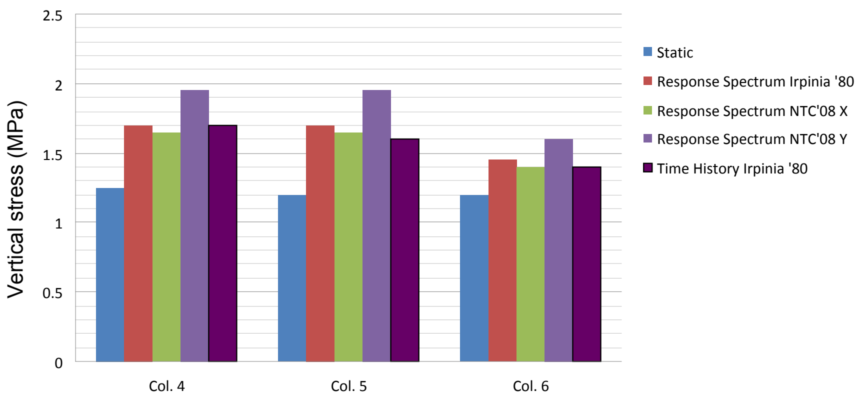
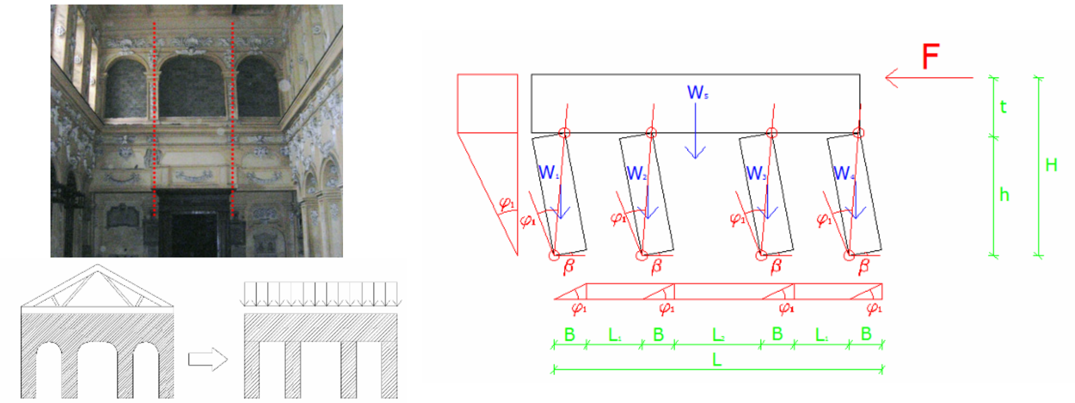


Figure 6. Summary of vertical compressive stresses in masonry columns 4, 5 and 6, for each analysis.

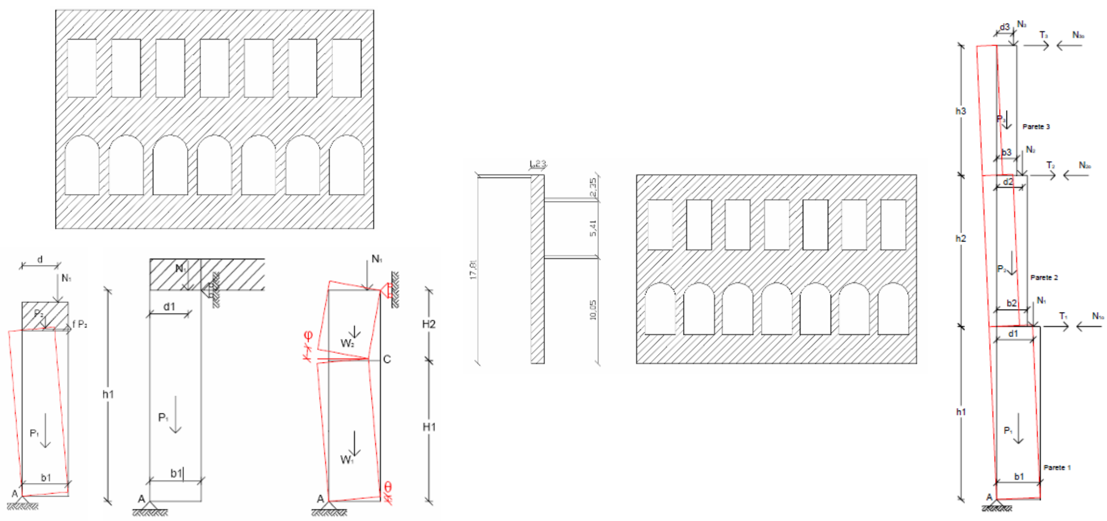
In the present section, the collapse multipliers of a distribution of horizontal loads have been evaluated for the following three macro-elements of the nave: (i) West wall; (ii) South wall; (iii) North wall. The analysis has been carried out by applying the kinematic theorem to the structural elements, subjected to vertical loads, and to the seismic load distribution. The mechanism and, more generally, the collapse mode depend on geometrical and mechanical factors. In general, the failure mechanism can be represented by a kinematic chain. The displacement components, derived by the kinematic chain, are used in the equation of the principle of the virtual works to calculate the horizontal collapse multiplier, from which it is then possible to calculate the value of the acceleration that activates the failure mechanism and to compare it with the design response spectrum, derived by the NTC'08 for the damage (SLD) and ultimate (SLU) limit state, and the response spectrum along the two main direction of the Irpinia earthquake. In particular the west wall is geometrically simplified to masonry portals (Fig. 8a). Storey mechanism is analyzed for the selected macro-element as suggested by the actual state of conservation that underlines the presence of a residual drift of the columns. Fig. 8b shows the geometrical configuration, loads on the portal and the kinematic chain of the mechanism.



(a) (b)

Figure 8. West wall macro-element: (a) simplified masonry portal; (b) loads and kinematic chain.

Out of plane failure mechanisms are analyzed for both the South and North walls macro-elements. Fig. 9 shows the geometrical configurations, loads and the kinematics of failure mechanisms.



(a) (b)

Figure 9. Out of plane failure mechanisms: (a) South wall; (b) North wall.

In particular the South wall is investigated against both out-of-plane overturning and vertical flexure of the wall as a monolithic block; while the North wall is analyzed against overturning as multiple rocking blocks. In both cases the analysis takes into account the presence of the RC beams of the roof and the intermediate storey. Table 1 summarizes the results of limit analysis of local mechanisms. It is clearly highlighted that the Irpinia earthquake is not able to activate the analyzed failure mechanisms; while out-of-plane overturning mechanism is crucial for North wall considering the limitation of the Italian code (activating value is lower than spectrum one).

Table 1. Summary of local mechanisms results.

	Acceleration that activate the failure mechanism	Spectrum acceleration			
		NTC'08		Irpinia earthquake	
		SLD	SLV	N-S component	W-E component
West wall	0.1547	0.1010	0.1440	0.1038	0.0718
South wall (overturning)	0.3498	0.0708	0.1008	0.0599	0.0420
South wall (flexure)	0.3073	0.0708	0.1008	0.0599	0.0420
North wall	0.0690	0.0708	0.1008	0.0599	0.0420

4 CONCLUSIONS

In this paper the St. Maria del Popolo church has been analyzed as a case-study to present an interdisciplinary procedure for assessing structural behavior and seismic vulnerability. For this purpose, 3D linear analyses of the structural complex have been run through FE in the static and dynamic range to investigate the origin of the surveyed crack pattern and a limit analysis has been carried out to analyze the local failure mechanisms. By this procedure it has been possible to obtain a proof of the configuration of the cracks, which was found to depend primarily on the Irpinia earthquake in 1980. The reliability of the model was assessed by comparing the monitored main damages with the numerically predicted ones. According to the FE model stress concentrations were detected, with peak values close to the strength of masonry. By a comparison between the stresses due to the seismic shocks acting in the horizontal directions it was observed that the church is especially vulnerable in the along-nave longitudinal direction, due to low stiffness and to the presence of localized thinned portions of the walls. In the same way, the comparison of demand (design seismic acceleration) vs. capacity (acceleration that activates the local failure mechanisms) in the limit analysis confirms the susceptibility of the North wall to the out-of-plane overturning failure mechanism. As a consequence, the results of the research open, today, delicate issues concerning the possible ways of strengthening the "mixed" building respecting all its historical values (Linee Guida 2008): that means strengthening and "historically" evaluating the post war interventions, too.

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