Seismic response of churches façades: comparison between static and dynamic approaches for recent Italian earthquakes

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ABSTRACT
Various earthquakes around the world have highlighted the significant seismic vulnerability of ancient religious buildings. Historical unreinforced masonry constructions are known to suffer local collapses during strong earthquakes, and this is even more evident for churches which, because of their high horizontal slenderness, frequently show an uncoupled response to their main macro-elements. Among them, the church façade is the one with the most powerful symbolic component and, as noticed after several seismic events, a significant propensity to damage. For each recent Italian seismic sequence, the 1997 Umbria-Marche, the 2009 L'Aquila, the 2012 Emilia and the 2016 Central Italy earthquakes, the out-of-plane response of a church façade is modeled. The analyses, based on an accurate survey of the façades geometry, are developed both in static and dynamic terms with reference to the recorded accelerograms of Nocera Umbra, Aquilpark, Mirandola and Norcia stations, next to the examined buildings. The analyses investigate the role on the façade response of the presence of buttresses, tie rods, and vertical component of the ground motion. The outcomes of such analyses are in reasonable agreement with observed damage.

1 INTRODUCTION

Damage observed after strong earthquakes has shown that the seismic behavior of existing unreinforced masonry buildings is frequently characterized by the occurrence of local mechanisms, rather than a global response (D'Ayala and Speranza, 2003; Sorrentino et al., 2018). This is even more evident for churches, because of their architectural characteristics, such as large horizontal and vertical spans (D'Ayala, 2000). Among the possible collapse mechanisms, the out-of-plane response of the churches façade has been frequently observed during earthquakes (Marotta et al., 2017), causing irreplaceable losses to the architectural heritage because of their noteworthy symbolic value.

In the last twenty-five years Italy has experienced several major earthquakes that proved the propensity to damage of the ecclesiastical buildings (De Matteis et al., 2016; Lagomarsino and Podestà, 2004; Penna et al., 2018; Sorrentino et al., 2014b) and particularly of their façades. In 1997 a strong earthquake struck the Northern part of Umbria region, nearby the boundary with Marche region. The first shock (M_w 5.7), occurred on September 26, was followed few hours later by a stronger event (M_w 6.0), causing 11 fatalities overall. The Umbria-Marche earthquake highlighted how inadequate strengthening interventions may affect the seismic response of churches. In fact, in those regions seismic strengthening was the main reason for replacing traditional timber roofs with heavier reinforced-concrete roofs, often not connected adequately to the walls. This kind of intervention frequently led to the failure of masonry walls (Tobriner et al., 1997). On April 6, 2009 the city of L'Aquila and the surrounding areas were hit by a seismic event (M_w 6.3) that caused 308 fatalities. The main shock was followed by thousands of aftershocks, seven of which with magnitude greater than 5.0. About 47% of the churches surveyed in the province of L'Aquila had significant levels of damage and was restricted for use (Da Porto et al., 2012), and the most frequent collapse mechanisms were those involving the churches façade. In May 2012, the Emilia region in Northern Italy experienced a severe earthquake sequence. The main shock occurred on May 20, 2012 (M_w 6.1) and caused 7 fatalities. Few days later, on May 29, a second shock (M_w 6.0) caused the death of additional 19 people. Churches were especially damaged, presenting some of the worst collapses observed. It was estimated that 905 of the churches in the dioceses of Carpi and Modena-Nonantola were uninhabitable (Anonymous, 2012). Recently, between August 2016 and January 2017, nine events of magnitude greater than or equal to 5.0 struck the central part of Italy. The first main shock took place on August 24, 2016 (M_w 6.0) causing severe damage to the municipalities of Amatrice, Arquata del Tronto, and Accumoli, while the second main shock occurred on October 30, 2016 (M_w 6.5) and mostly affected the municipalities of Norcia and Castelsantangelo sul Nera, with almost all the churches of Norcia suffering substantial damage or collapse (Borri et al., 2017).

As described, the out-of-plane response of several churches façades was observed in all affected areas in the aftermath of each seismic sequence, deeply affecting the national heritage and highlighting the necessity to better investigate the response of such mechanism. The topic is herein studied for four churches, via the use of both static analyses and rigid-body non-linear dynamic models. As for the dynamic approach, the out-of-plane response of the façades is modelled assuming rigid-body mechanisms, following the approach of several recent studies in simulating the seismic behaviour of existing masonry buildings (D'Ayala and Speranza, 2003; De Lorenzis et al., 2007; Derakhshan et al., 2013; Doherty et al., 2002; Lagomarsino, 2015), provided that no masonry disintegration phenomena occur (De Felice, 2011).

2 CASE STUDIES

For each of the aforementioned last Italian earthquakes, the 1997 Umbria-Marche, the 2009 L'Aquila, the 2012 Emilia and the 2016 Central Italy events, the out-of-plane response of an affected church façade is modeled following both the static and dynamic approach. The accelerograms recorded by the permanent and temporary stations of the Italian National Seismic Network, managed by the Italian National Institute of Geophysics and Vulcanology, as well as by the Italian National Accelerometric Network, managed by the Department of Civil Protection, have been used for the two main events of each seismic sequence. Table 1 shows a comparison between the peak ground accelerations (PGAs) of the selected records.

The church of San Filippo Neri in Nocera Umbra was damaged by the 1997 earthquake (Figure 1a) and its façade showed cracks and the failure of the oculus without overturning though. The accelerograms recorded by the NCR station, located in the municipality of Nocera Umbra at 0.30 km from the church, in occasion of the two main events of the seismic swarm have been used for the dynamic analyses.

The church of San Giuseppe dei Minimi in L'Aquila, responded to the 2009 earthquake with evident mechanisms (Figure 1b). The façade was separated from the longitudinal walls and the masonry did not fragment as in other examples in L'Aquila (Sorrentino et al., 2014a). The accelerograms recorded by the AQK station, located in the municipality of L'Aquila at 0.60 km from the church, have been used.

The church of San Francesco in Mirandola showed limited damage in the façade after the May 20, 2012 event, and was able to survive the May 29, 2012 event, despite the collapse of the right nave (Figure 1c). Photos of the church before the 2012 seismic sequence showed cracks on the longitudinal walls close to the façade, thus confirming the study of the façade with the hypothesis of a one-sided rocking body. The accelerograms recorded by the MRN station, located in the municipality of Mirandola at 1.18 km from the church, have been used.

The church of San Benedetto in Norcia was severely damaged by the October 30, 2016, event (Figure 1d), and only its façade remained still standing, whereas most of the rest of the building suffered a collapse. The accelerograms recorded by the NRC station, located in the municipality of Norcia at 0.25 km from the church of San Benedetto, have been used.

Table 1. Peak ground acceleration (PGA) of selected records for the two main events of each considered seismic sequence (NCR for the 1997, AQK for the 2009, MRN for the 2012, NRC for the 2016 earthquakes).

Earthquakes	**Event**	**PGA (g)**		
		E	**N**	**Z**
UMBRIA-MARCHE	September 26, 1997 (00:33 am) - M_w 5.7	0.268	0.495	0.146
	September 26, 1997 (09:40 am) - M_w 6.0	0.423	0.502	0.406
L'AQUILA	April 6, 2009 (01:32 am) - M_w 6.3	0.330	0.354	0.362
	April 6, 2009 (05:47 pm) - M_w 5.6	0.090	0.082	0.055
EMILIA	May 20, 2012 (02:03 am) - M_w 6.1	0.262	0.264	0.303
	May 29, 2012 (07:00am) - M_w 6.0	0.223	0.294	0.857
CENTRAL ITALY	August 24, 2016 (01:36 am) - M_w 6.0	0.360	0.373	0.216
	October 30, 2016 (06:40 am) - M_w 6.5	0.486	0.372	0.375

Figure 1. a) San Filippo Neri (Nocera Umbra): façade before and after the 1997 earthquakes; b) San Giuseppe dei Minimi (L'Aquila): façade before and after the 2009 earthquakes; c) San Francesco (Mirandola): façade before and after the 2012 earthquakes (courtesy of Andrea Penna); d) San Benedetto (Norcia): façade before and after the 2016 earthquakes.

3 DYNAMIC ANALYSES

The non-linear dynamic models used in the analyses are based on an explicit formulation of the equation of motion (**Errore. L'origine riferimento non è stata trovata.**a-b), and on the replacement of the classical two-branch moment-rotation relationship by a three-branch curve **Errore. L'origine riferimento non è stata trovata.**d), where α = arctan (B/H), α_i = angle between R_i and vertical line through O, Δ_1 and Δ_2 are non-dimensional parameters calibrated on experimental tests (Sorrentino et al., 2014a). The equations of motion used for the one-sided rocking assume a façade poorly connected to orthogonal walls that, however, avoid one direction of rotation (**Errore. L'origine riferimento non è stata trovata.**b). According to this model, energy dissipation occurs only at impacts, when $\theta = 0$, by reducing the angular velocity by means of a coefficient of restitution (Housner, 1963; Sorrentino et al., 2011).

Since it has been proven that out-of-plane local collapse mechanisms can be prevented if façades are restrained by tie rods that improve the connections to perpendiculars walls, a nonlinear equation of motion of a monolithic wall restrained by a tie rod is also used, following AlShawa et al. (2019). The tie rods considered in such model (**Errore. L'origine riferimento non è stata trovata.**c), are elasto-plastic, can fail if stretched beyond ultimate elongation and can be placed at any vertical point alongside the wall height. The model also assumes that no failure at wall anchor occurs, neither in the steel connection nor in the adjacent masonry. In the case at hand, tie rods designed according to the force-based procedure established by the Italian Building Code (CMIT, 2019) are considered, and the corresponding characteristics are reported in **Errore. L'origine riferimento non è stata trovata.**.

Table 2. Tie rods characteristics (Code design has been performed assuming a modern steel S235).

Church	**Tie rod height (m)**	**Tie rod length (m)**	**Pre-stress force (kN)**	**Tie rod cross section area (mm^2)**
San Filippo Neri	12.8	30.0	27.2	$1.16\ 10^3$
San Giuseppe dei Minimi	12.2	18.0	37.6	$1.60\ 10^3$
San Francesco	15.4	45.0	25.6	$1.09\ 10^3$
San Benedetto	12.0	47.0	49.7	$2.11\ 10^3$

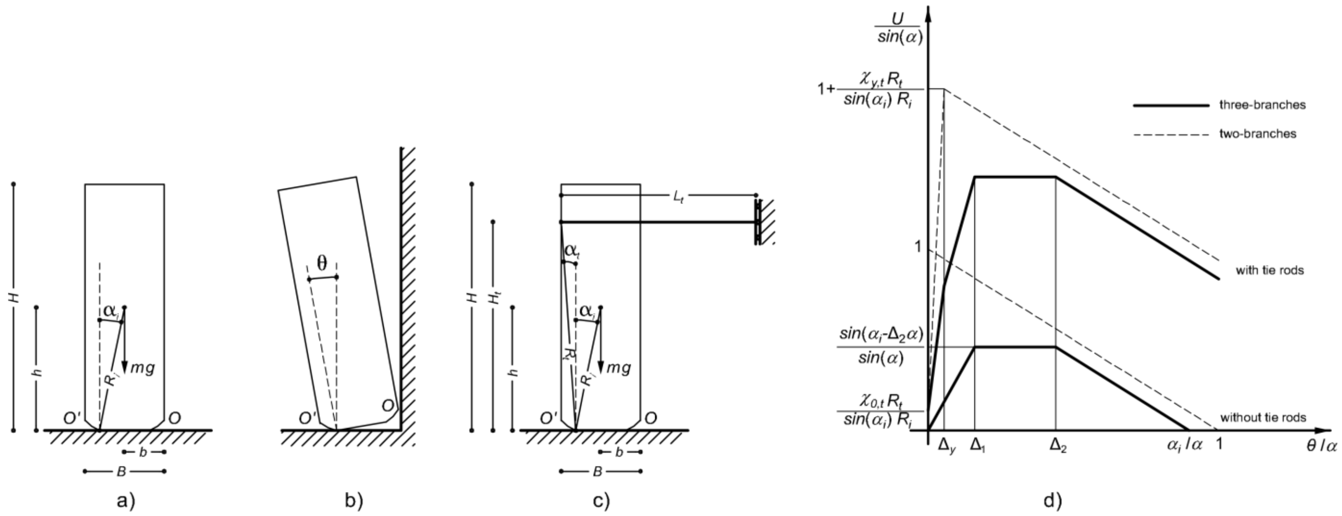

Figure 2. a) Geometrical parameters accounting for hinge indentation; b) One-sided displaced configuration (θ > 0); c) Geometrical parameters accounting for tie rods; d) Normalized moment–rotation relationship.

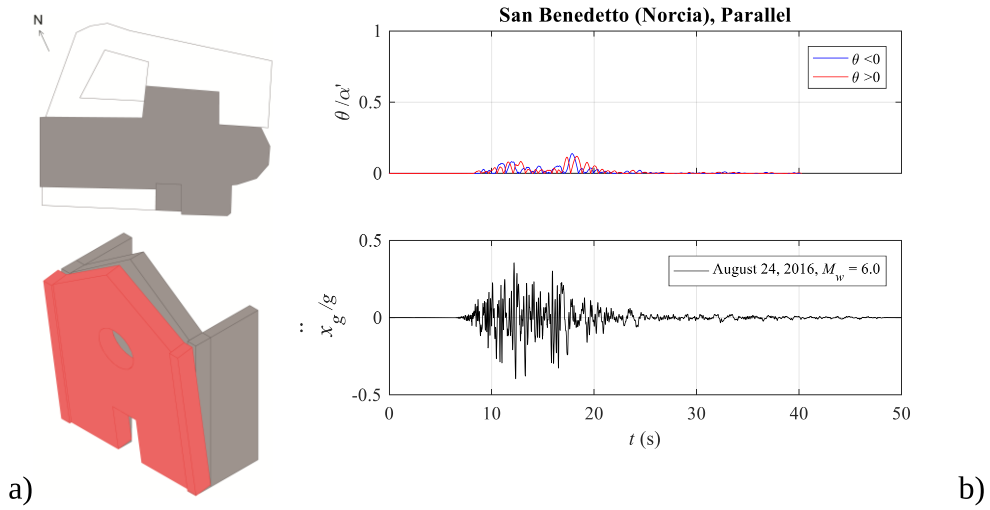

Figure 3. a) Model of the façade of San Benedetto (Norcia); b) Time history of the one-sided rocking rotation response parallel to the façade for the August 24, 2016 accelerograms.

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The façades of the four churches have been modelled accounting for their actual geometry, including pitched top side, openings and buttresses (example in **Errore. L'origine riferimento non è stata trovata.**a), considering one-sided rocking and the two main events for all sequences. Accelerometric records have been projected in the direction normal to the façades and, for comparison, in the parallel direction too. Vertical component is considered unless otherwise noted. Although rigid-body models depend on the position of the rocking hinge along the façade height and on the scale effect (AlShawa et al., 2015), in the cases under consideration the hinge was considered at ground level, as observed in the four churches. An example of time history of the normalized rotation of the one-sided rocking body is presented in **Errore. L'origine riferimento non è stata trovata.**b. Because the problem is asymmetric, the polarity of the rotation has been considered as positive and negative for each record. In Table 3, the façades response is measured in terms of the maximum normalized absolute rotation, θ_{max} / α_i, where θ and α_i are shown in **Errore. L'origine riferimento non è stata trovata.**. Results are in agreement with the behaviour actually observed for each church. As for San Francesco and San Benedetto, no overturning is observed when considering one-sided rocking in both directions of both components, and the rotation is not too large in any of the cases, in reasonable agreement with the cumulative damage observed after the occurred events, after which the façades stayed still-standing despite the collapse of the rear structure. In the case of Norcia, the comparison between the August and the October events shows a noticeable increase of maximum rotation.

The role of the buttresses of the façade on the dynamic response has been investigated repeating the previous analyses for a model presenting the same global geometry but without the buttresses effectively present in three of the four façades. Compared to previous results, there is a reduction of the maximum rotation in just two cases, an overturning in two cases and a marked increase of the response in the remaining cases.

Such performance is related to the decrease of the angle α, when no buttresses are present. The reduction in rotation is present only in the case of the church of San Benedetto in Norcia, for the October 30, 2016 event and is related to the different rotations of the façade at the instant before the occurring of the pulse (Figure 4).

In fact, in the case without buttresses, at the time of the pulse the façade is moving back to the vertical configuration, thus resulting out-of-phase with the ground motion, while in the case with buttresses, despite an initial smaller rotation, there is a combined effect of pulse and rebound after impact.

In order to assess the relevance of the vertical component on the seismic performance of the façades, the same analyses have been repeated without the vertical component (values in brackets). The behavior is almost coincident for almost all cases, while a few differences are found only for the churches of San Giuseppe dei Minimi in L'Aquila and San Francesco in Mirandola, for the façade configuration without buttresses. While in L'Aquila the vertical component of the April 6, 2009 shock was not that prominent, the May 29, 2012 shock induced very large vertical accelerations in Mirandola, with a vertical PGA of 0.87 g.

The role of the tie rods designed according to the force-based procedure established by the Italian Building Code is significant on the out-of-plane response of each considered façade. Reductions of the maximum normalized absolute rotation are observed in all cases, confirming the efficacy of this intervention in preventing the overturning of masonry buildings façades.

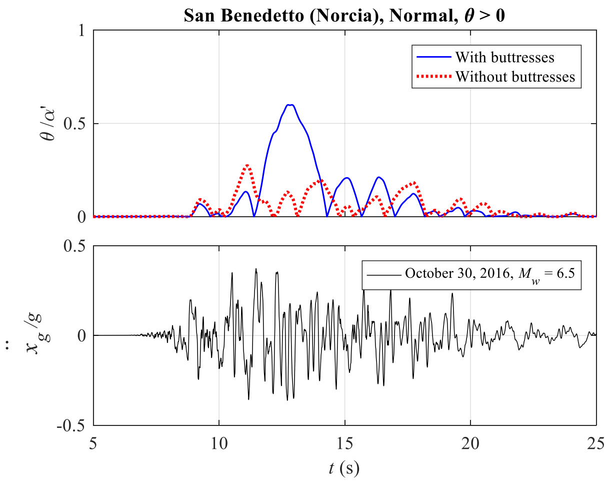

Figure 4. Time history of the one-sided rocking rotation response perpendicular to the San Benedetto façade for the October 30, 2016 accelerogram, considering both the presence/absence of buttresses.

4 STATIC ANALYSES

Static analyses of the churches façade response have been performed. In the case of static analysis, the seismic action is expressed by horizontal forces whose intensity is represented by the coefficient α, computed as the ratio between the horizontal forces and the corresponding weights present in the structure. The analysis can be performed in terms of acceleration, linear kinematic approach, or displacement, nonlinear kinematic approach (Sorrentino et al., 2014c). The first approach consists in the comparison between the acceleration that can activate the mechanism and the PGA of the dynamic input. The nonlinear kinematic approach requires to determine of the horizontal action that the structure is able to withstand at the evolving of the mechanism. The demand displacement, obtained from the non-decreasing spectrum with 8% damping according to CMIT (2019), is compared to the one corresponding to the achievement of the accounted limit state that, in the case at hand, is the life safety limit state. In agreement with the dynamic model, each façade has been considered as a single-body wall restrained by a tie rod. Following observed damage, connection to transversal wall, which can be crucial for earthquake performance (Casapulla and Argiento, 2016), has been neglected. For all churches, the analyses in terms of acceleration are not satisfied, and this is due to the high values of PGAs of selected records. The analyses in terms of displacement are not verified in the case of the church of San Giuseppe dei Minimi in L'Aquila for the first event of the sequence both in the direction normal and parallel to the façade, and in the case of San Francesco in Mirandola for the second event of the sequence in the direction parallel to the façade. This latter result highlights the larger displacement demand in the direction parallel to the façade that is close to the NS component, for which a more severe rocking response has already been reported (Sorrentino et al., 2014c). Moreover, because it has been proven that the presence of metal tie rods improves the earthquake performance of the churches façades, the static analyses have been conducted also considering the ties used in the dynamic analyses and designed according to the Italian Building Code (CMIT, 2019). Results are given in Table 4.

Table 3. Response of the façade of the four churches to recorded accelerograms, for both events of each seismic sequence, for horizontal and vertical components (horizontal component alone) in terms of the maximum normalized absolute rotation, θ_{max}/α_i. Effect of component and polarity of the record for the one-sided rocking mechanism are shown, as well as the influence of buttresses and tie rods.

San Filippo Neri (Nocera Umbra)

Component	**Polarity**	**Effective configuration**		**Without buttresses**		**With tie-rods**	
		Event 1	Event 2	Event 1	Event 2	Event 1	Event 2
Normal	$\boldsymbol{\theta < 0}$	0.03 (0.03)	0.10 (0.10)	0.07 (0.07)	0.26 (0.27)	0.01 (0.01)	0.03 (0.03)
	$\boldsymbol{\theta > 0}$	0.03 (0.03)	0.14 (0.14)	0.07 (0.07)	0.19 (0.18)	0.01 (0.01)	0.03 (0.03)
Parallel	$\boldsymbol{\theta < 0}$	0.02 (0.02)	0.05 (0.05)	0.03 (0.03)	0.20 (0.19)	0.01 (0.01)	0.06 (0.06)
	$\boldsymbol{\theta > 0}$	0.03 (0.03)	0.11 (0.11)	0.06 (0.06)	0.18 (0.19)	0.02 (0.02)	0.07 (0.07)

San Giuseppe dei Minimi (L'Aquila)

Component	**Polarity**	**Effective configuration**		**Without buttresses**		**With tie-rods**	
		Event 1	Event 2	Event 1	Event 2	Event 1	Event 2
Normal	$\boldsymbol{\theta < 0}$	0.43 (0.42)	0.02 (0.02)	N/A	N/A	0.02 (0.02)	0.01 (0.01)
	$\boldsymbol{\theta > 0}$	0.89 (0.81)	0.02 (0.02)	N/A	N/A	0.09 (0.09)	0.00 (0.00)
Parallel	$\boldsymbol{\theta < 0}$	0.15 (0.08)	0.02 (0.02)	N/A	N/A	0.01 (0.02)	0.00 (0.00)
	$\boldsymbol{\theta > 0}$	0.07 (0.16)	0.02 (0.02)	N/A	N/A	0.01 (0.01)	0.00 (0.00)

San Francesco (Mirandola)

Component	**Polarity**	**Effective configuration**		**Without buttresses**		**With tie-rods**	
		Event 1	Event 2	Event 1	Event 2	Event 1	Event 2
Normal	$\boldsymbol{\theta < 0}$	0.15 (0.15)	0.05 (0.05)	0.22 (0.22)	0.11 (0.13)	0.07 (0.07)	0.02 (0.02)
	$\boldsymbol{\theta > 0}$	0.11 (0.11)	0.05 (0.04)	0.26 (0.27)	0.10 (0.16)	0.07 (0.07)	0.03 (0.03)
Parallel	$\boldsymbol{\theta < 0}$	0.17 (0.17)	0.61 (0.60)	1.00 (1.00)	1.00 (0.47)	0.11 (0.11)	0.05 (0.05)
	$\boldsymbol{\theta > 0}$	0.33 (0.32)	0.52 (0.52)	1.00 (1.00)	1.00 (0.53)	0.12 (0.12)	0.15 (0.15)

San Benedetto (Norcia)

Component	**Polarity**	**Effective configuration**		**Without buttresses**		**With tie-rods**	
		Event 1	Event 2	Event 1	Event 2	Event 1	Event 2
Normal	$\boldsymbol{\theta < 0}$	0.06 (0.07)	0.14 (0.16)	0.09 (0.09)	0.18 (0.18)	0.04 (0.04)	0.07 (0.07)
	$\boldsymbol{\theta > 0}$	0.07 (0.07)	0.57 (0.60)	0.17 (0.19)	0.28 (0.27)	0.03 (0.03)	0.12 (0.12)
Parallel	$\boldsymbol{\theta < 0}$	0.06 (0.06)	0.13 (0.14)	0.11 (0.12)	0.26 (0.25)	0.02 (0.02)	0.07 (0.07)
	$\boldsymbol{\theta > 0}$	0.04 (0.04)	0.20 (0.12)	0.06 (0.06)	0.24 (0.29)	0.02 (0.02)	0.11 (0.10)

Table 4. Demand/ Capacity ratio of each church according to the force- (F) and displacement-based (D) procedure of the Italian Building Code for static analyses.

Church	Event	Component	D/C (Effective configuration)		D/C (With tie rods)	
			F	D	F	D
San Filippo Neri (Nocera Umbra)	Event 1	Normal	3.61	0.12	1.44	0.01
		Parallel	1.95	0.06	0.78	0.01
	Event 2	Normal	3.65	0.23	1.45	0.03
		Parallel	3.08	0.25	1.23	0.03
San Giuseppe dei Minimi (L'Aquila)	Event 1	Normal	7.80	2.77	1.16	0.18
		Parallel	5.19	1.15	0.77	0.08
	Event 2	Normal	1.96	0.34	0.29	0.02
		Parallel	1.51	0.39	0.22	0.03
San Francesco (Mirandola)	Event 1	Normal	4.88	0.54	1.14	0.04
		Parallel	4.88	0.77	1.13	0.05
	Event 2	Normal	4.21	0.59	0.98	0.04
		Parallel	5.25	1.38	1.22	0.10
San Benedetto (Norcia)	Event 1	Normal	6.45	0.33	1.23	0.02
		Parallel	4.94	0.56	0.94	0.04
	Event 2	Normal	5.93	0.78	1.13	0.06
		Parallel	6.28	0.83	1.20	0.06

5 CONCLUSIONS

Unreinforced masonry churches frequently respond to earthquakes with local collapse mechanisms involving separate macro-elements. This phenomenon, highlighted by several seismic events, has been investigated by considering the performance of four churches affected by the last four strongest Italian sequences, and specifically analyzing their façades response. The dynamic response of the considered façades has been modelled using rigid rocking bodies, excited by close-by recorded accelerograms. The computed time histories show reasonable agreement with observed performances after both main events of the four seismic sequences. The role of the vertical component is negligible in most cases, because these mechanisms are sensitive to periods much longer than those of the vertical component. Buttresses are beneficial in reducing the response, but much less than tie rods.

Additionally, the static response of the four façades has been evaluated for comparison reasons, delivering rather conservative results, especially when tie rods are present.

The obtained results are useful for the understanding of observed performances of façades during earthquakes, and can guide future seismic assessment of unreinforced masonry churches.

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