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Seismic Behavior of Some Basilica Churches After L'Aquila 2009 Earthquake

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Abstract In this paper the seismic response of four church buildings in the l'Aquila April 2009 earthquake is analyzed. The buildings are: St. Giusta church, St. Maria di Collemaggio Basilica, St. Silvestro church and St. Pietro di Coppito church. The analyses are carried out by employing a "two-step" procedure suggested by the authors and already applied to similar case studies. The first step consists in three-dimensional linear analysis, while in the second step, two-dimensional non-linear analyses of macro-elements are carried out. The results coming from the two-steps, allow for understanding the observed damage and for approximately assessing the seismic level of the building.

Keywords: L'Aquila earthquake, masonry churches, non linear analysis, pushover analysis

Introduction

The l'Aquila earthquake of 6 April 2009 has confirmed that basilica church buildings are particularly susceptible to damage and to partial or total collapse.

In this paper, four basilica churches strongly damaged during the l'Aquila earthquake, are analyzed through a "two-step" procedure, developed by the authors in Mele et al. (1999). In the first step each building is analyzed in the linear range with 3D finite element models, in order to determine the static and dynamic properties, the distribution of internal forces, the points of stress concentration, the strength demand on each macro-element. Subsequently the complex 3D structure is decomposed in its constituting macro-elements, and each macro-element is analyzed in the non linear range up to collapse, in order to determine its horizontal strength capacity. The results coming from the "two-steps", expressed in terms of strength demand and strength capacity allow to understand the reasons of the seismic damages observed after the l'Aquila earthquake, and, in more general terms, to estimate the safety level of this type of buildings.

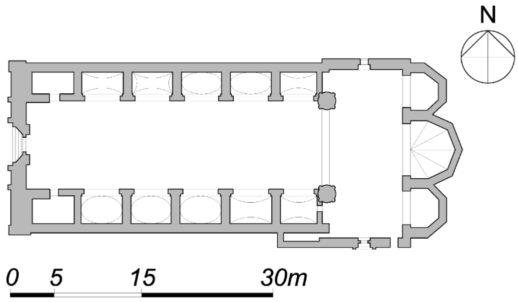
The Case Studies: Description and Observed Damages

St. Giusta Church (SG)

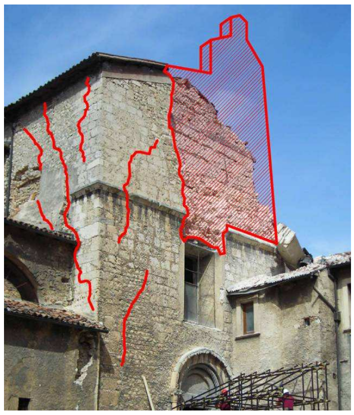
The church of St. Giusta was built in the first decades of the XIV century on the ruins of pre-existing masonry walls. The church has a basilica plan (Fig. 1a) with a central aisle and lateral chapels.
The bearing masonry structures are realized by using sack masonry with external facing made of small calcareous hewn stone and filling in calcareous stone and hydraulic mortar; the triumphal arch and its columns are realized in freestone. The roof structure is realized in no trusting wood trusses.

The severe damage due to the earthquake consists in partial collapses, essentially due to the overturning mechanisms of a portion of the apse and of the south wall of the transept (Fig. 1b); localised damages (crushing and disgregation) have been also noticed in the columns of the triumphal arch under combined compression force and bending moment.

Finally it has been observed a complete separation between lateral chapels and the walls of the central nave, probably due to the fact that the walls of the chapels were realized in a later phase with respect to the external walls of the structure.



(a)

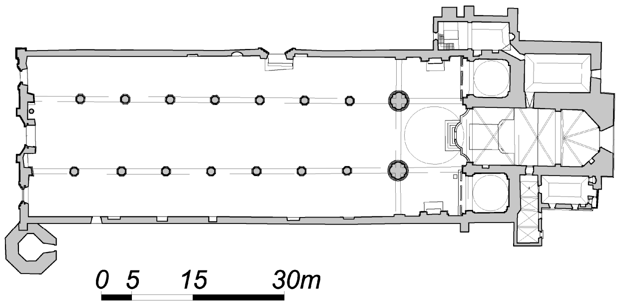


(b)

Figure 1: St.Giusta church: plan (a); damage at transept (b)

St. Maria di Collemaggio Church (SMC)

Also St. Maria di Collemaggio church was built in the XIV century and, following a dismantlement of the Baroque decorations occurred in the 1950's, it presently appears in its native aspect; in particular the building has a basilica plan with three naves, transept and apse (Fig. 2).



(a)

(b)

Figure 2: St. Maria di Collemaggio church: plan (a); collapse at transept (b)

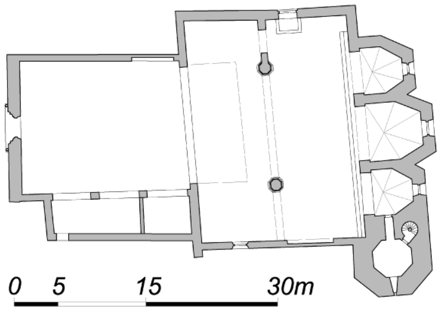
Also in this case the bearing walls are generally realized using the typical sack masonry of l'Aquila; only the most stressed elements, like the triumphal arches and pillars, are realized in freestone. The roof structure is made of wooden trusses for the central aisle and masonry vaults for the remaining parts.

The seismic damage is mainly concentrated in the zone of presbytery; in particular, total collapse of the transept and partial collapse of the apse have been observed. Also the pillars of the nave have suffered damage, namely vertical crushing cracks due to the longitudinal response and the significant vertical component of the earthquake strong motion.

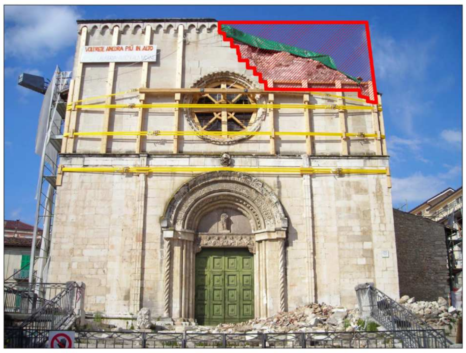
St. Pietro di Coppito Church (SPC)

The church was built at the end of the XIV century and it has the most non-typical plan among the four case studies (Fig. 3a): the hall of the church has a large, almost square nave, with a small lateral aisle

along one side only; also the transept is large and almost square in plan, while the apse is constituted by three polygonal chapels. The church has one of the rare bell towers in the city of l'Aquila.
Similarly to the other cases, the masonry walls are realized in sack masonry and free stone, while the roof structures are wooden trusses, with the exception of the cross vaults covering the apse chapels.
The severe damage consists in overturning mechanism of the façade (Fig. 3b) and in the partial collapse of the bell tower, that in turn has provoked the collapse of the apse roof.



(a)



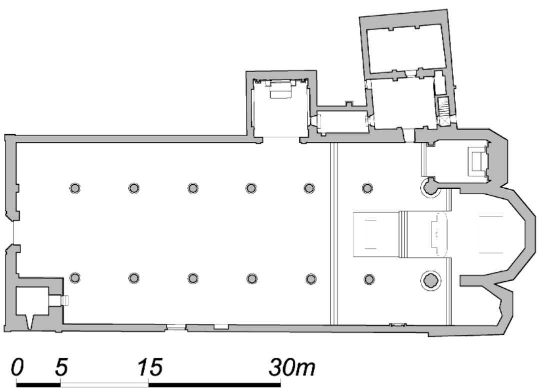
(b)

Figure 3: St. Pietro di Coppito church: plan (a); damage at the façade (b)

St. Silvestro Church (SS)

St. Silvestro church was built it the same age of the other case studies. It has a very simple architectural organism (Fig. 4), with three naves and a apse characterized by polygonal chapels (like in St. Giusta and St. Pietro di Coppito churches). The bearing wall structures are realized, as in the previous cases, in sack masonry or free stone, while the roof consists in wooden trusses, with cross vaults only covering the apse.

With respect to the other case studies, the St. Silvestro church has shown very limited damage, mainly concentrated in the bell tower, where cracks caused by the seismic overturning moment can be observed. Also the façade shows some damage due to both the activation of partial out of plane overturning mechanism, and the dynamic interaction between the façade and the bell tower.



(a)

(b)

Figure 4: St. Silvestro church: plan (a); damage at façade (b)

The "Two-Step" Procedure

Linear Dynamic Analysis
The four case studies have been modelled by using shell elements; the linear structural analyses have been carried out by using the FEM computer code SAP 2000 (2006).

Modal dynamic analyses have been carried out by using both the design spectrum suggested by the Italian Technical Code (NTC'08) and the response spectrum obtained from the acceleration history recorded during the main shock of April 2009.

The results of the modal analyses show a wide dispersion of the modal shapes, due to absence of rigid diaphragm action (roof with wooden trusses): in fact there are no predominant vibrations modes, associated to high values of modal participating mass ratios. The above consideration strengthens the idea at the base of the "two-step" procedure, i.e. of decomposing the entire structures in its macro-elements, that are characterized by an individual seismic behaviour.

The periods of the analyzed structures are within the range 0.1s ÷ 0.6s; only in some cases, i.e. presence of very deformable elements characterized by not negligible masses, the periods reach values up to 3s.

The comparison between the results of the modal analyses associated to the design spectrum suggested by the NTC'08 and the recorded spectrum (Fig. 5), in the following appointed as "06/04/09", shows slight scatters in term of base shear force, with base shear action V in order of 30% of the vertical load W_{tot} both in transversal and longitudinal direction (Fig. 6).

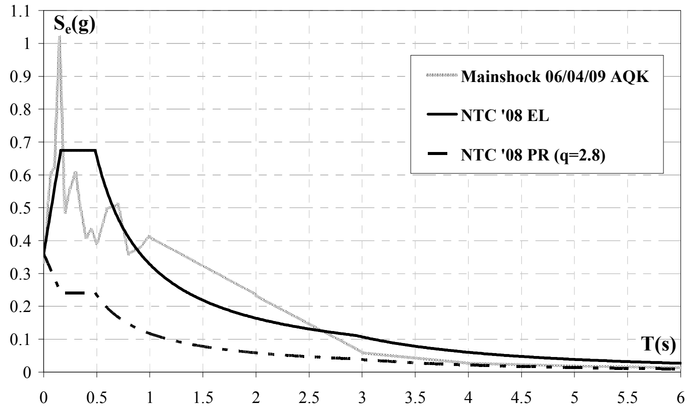


Figure 5: Comparison among main shock spectrum (06/04/09 AQK), elastic (EL) and design (PR) spectra provided by Italian code NTC'08

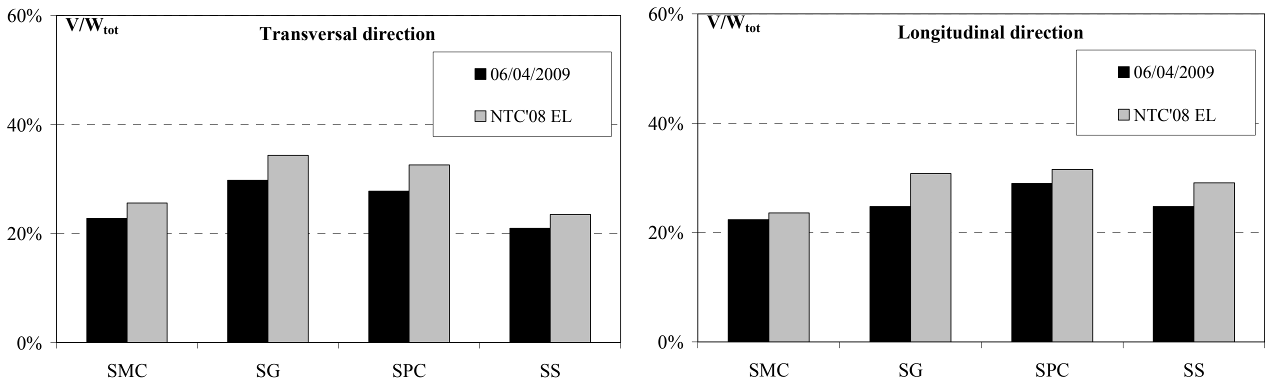


Figure 6: Normalized base shear in the four churches: NTC'08 VS. 06/04/09 main shock

Non Linear Static Analyses

In the second step of the analysis procedure, the church macro-elements have been analyzed in the non linear range, by applying constant gravity loads and increasing horizontal actions. The horizontal loading has been simulated through a set of forces, proportional to masses and distributed throughout the joints of the 2D F.E. models (Fig. 7).

MACRO-ELEMENT	SG	SMC	SPC	SS
1	AP	T4	AP	AP
2	T8	T3	T4	T2
3	T7	T2	T3	
4	T2-T6	TRANSV	T2	TRANSV
5	T1	T1	T1	T1
6	L1 L5	L1 L4	L1	L1
7	L2-L3	L2-L3	L2	L2-L3

Figure7: Macro-elements: (1) Apse; (2) First triumphal arch; (3) Second triumphal arch; (4) Nave section; (5) Façade; (6) Longitudinal Front; (7) Arcade/Clerestory

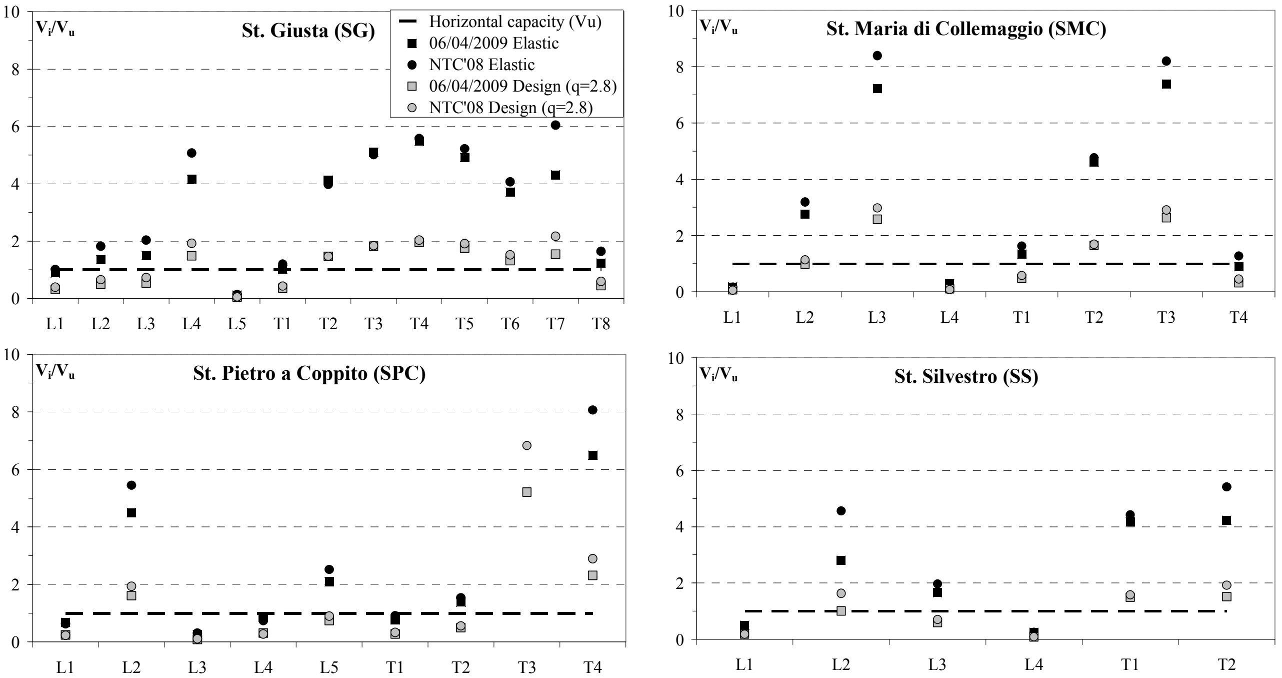


Figure 8: Macro-elements: ratio between horizontal strength demand and horizontal strength (V_i) capacity (V_u)

The non linear analyses have been carried out using a smeared cracking approaches as implemented in the computer code ABAQUS (Simulia 2007); according to this approach, the geometry is fixed and the cracking process is accounted for only through constitutive law. More details concering the application on such approach to masonry modeling can be found in Brandonisio et al. (2008)

The results of the analyses on the single structural elements allow for deriving push-over curves, stress distributions, collapse modes and ultimate horizontal strengths; in this paper, for sake of brevity, only the values of ultimate lateral strength are discussed, particularly in the comparison with the strength demand obtained for each macro-element in the first step of the procedure.

In Fig. 8 the ratio of strength demand (V_i) to capacity (V_u), is provided for each macro-elements; in particular the strength demand has been calculated both ad “elastic” and “inelastic” demand, i.e. according either to elastic spectrum or inelastic design spectrum.

It can be noted that, with the exception of some perimeter elements, the macro-elements of churches usually have a strength capacity smaller than the demand counterpart ($V_i/V_u>1$). These results, together with the observed damage patterns, confirm the high seismic vulnerability of churches.

Conclusions

In this paper, the seismic behaviour of four masonry churches of l’Aquila (Italy) has been briefly analyzed. In particular, the damage patterns caused by the earthquake of 06/04/09 have been studied; then the churches have been analyzed through a “two step” procedure, i.e. both through a global 3D model in the linear field, and through 2D models of single macro-elements in the non-linear range. From the first step, the distribution of the seismic force among the macro-elements of each church has been obtained, showing high force concentration in the perimeter elements and, in some cases, in the transept elements, while lower stress levels generally occur in the internal elements. In the second step of the procedure, non linear analyses of all macro-elements of each church have been carried out in order to obtain the horizontal plastic strength.

The comparison between the L’Aquila recorded spectrum and the conventional acceleration spectrum provided by the Italian provisions NTC’08, shows that actual seismic demand is in good agreement with the Code values (Fig. 5).

Furthermore, the dynamic analyses of the case studies have shown that, the dispersion of the vibration modes, leads to normalised values of base shear demand V/Wtot in order of 30% (Fig. 6).

Finally, on the basis of the results coming from the “two-step” analysis and from the comparison with the actual damage suffered by the analysed churches, it can be stated that an approximate assessment of the safety level of the churches can be obtained through the proposed approach.

The procedure, as implemented here to the four case studies, confirms that basilica churches are particularly vulnerable to earthquake loadings.

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