ANALYSIS OF SEISMIC RESPONSE AND REHABILITATION PROJECT FOR A MASONRY BUILDING SITE AMATRICE (RI), DEMAGED BY THE L'AQUILA EARTHQUAKE OF 06 APRIL 2009

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SUMMARY

The Abruzzo earthquake of April 6, 2009 caused considerable damage to structures over an area of approximately 600 square kilometres, including the urban center of L'Aquila (Italy) and the other adjacent villages.
The article aims to provide a description of the seismic response of a masonry building located in Amatrice (in the province of Rieti, 33 km away from the epicentral area) and damaged by mainshock and aftershocks, as well as a project for the restoration of seismic safety compromised due to the substantial damage on masonry load-bearing structures and secondary structural elements.
Finally, an assessment of the vulnerability of the building will be presented along with pre and post interventions in terms of PGA (of collapse and loss of operation), using a three-dimensional finite element model of the building.

INTRODUCTION: GROUND MOTION IN THE EPICENTRAL AREA AND ITS PROPAGATION UP TO AMATRICE

Figure 1 shows a map of the observed Mercalli-Cancani-Sieberg (MCS) intensity, which gives a picture of the non-uniform and asymmetric distribution of damage within the affected area. The Abruzzo earthquake is the first well-documented strong-motion earthquake instrumentally recorded in Italy in a near-fault area. The main event of April 6, 2009, was recorded by 56 digital strong motion stations which are part of the Italian Strong Motion Network (Rete Accelerometrica Nazionale, RAN), owned and maintained by the Department of Civil Protection (DPC). Fourteen stations are located in the Abruzzo region, while the remaining ones are distributed along the Apennines, mostly NW and SE of the source area. Five strong-motion stations were located within 10 km of the epicenter, on the hanging wall side of the normal fault; all of them recorded horizontal peak accelerations higher than 0.35g.
The peak ground motions calculated from processed data have been compared with the prediction obtained, using different ground motion prediction equations (GMPEs, Figure 3): Sabetta and Pugliese (1996, SP96), Bindi et al. (2008, ITA08), Faccioli and Cauzzi (2008, FC08) and Akkar and Bommer (2007, AkBo07).
It is estimated that during the mainshock of 06/04/2010 the PGA that hit Amatrice was of ≈ 120 cm/s2 (see Figure 2), while the L'Aquila station AQG and AQV registered a maximum value of 479,267 and 644,247 cm/s2. Italian Seismic Code (NTC 2008) provides, for Amatrice, an acceleration equal to 255,06 cm/s2 which is less than estimated for the mainshock.

Table 1. Records from some stations by the main event and the aftershock

STATION	Mainshock (Mw = 6,3) of 06/04/2009		Aftershock (Mw = 5,1) of 13/04/2009	
	Epicentral distance [km]	PGA [cm/s2]	Epicentral distance [km]	PGA [cm/s2]
AMT	≈ 35 km	x	15,602	33,903
AQG	4,392	479,267	14,656	44,541
AQV	4,870	644,247	14,170	59,231

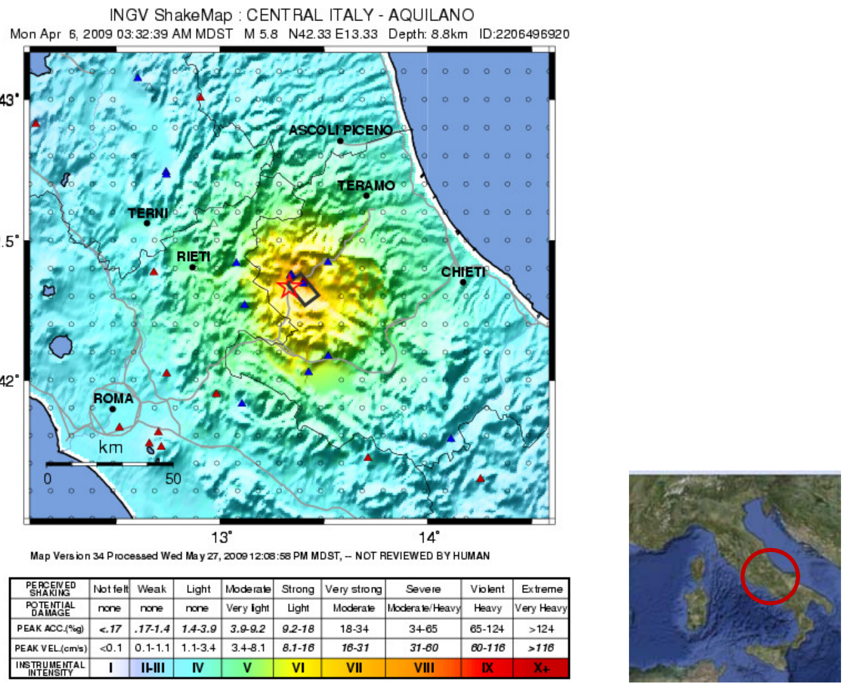

Figure 1. Map of the observed Mercalli-Cancani-Sieberg (MCS) intensity (Galli and Camassi, 2009).

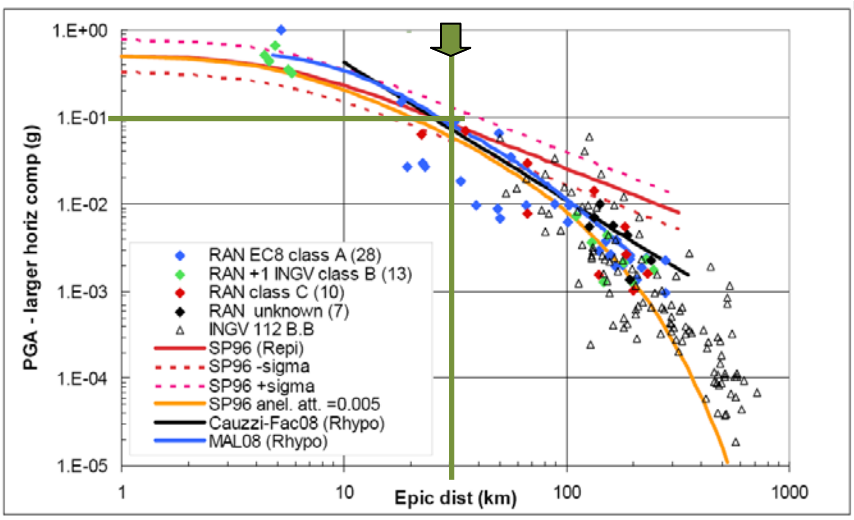

Figure 2. PGA values recorded by the network RAN according to distance from the fault compared with some attenuation relations (Sabetta, 2009).

In Figure 3, the registred response spectrum of aftershock of 13/04/2009 is reported: a maximum seismic amplification was obtained, both in Amatrice and L'Aquila, for similar values of periods (T, sec), shown in Table 2.

The dynamic analysis of building will not show amplification phenomena: for the mainshock and aftershocks the periods of the structure are far away from those amplification of earthquake.

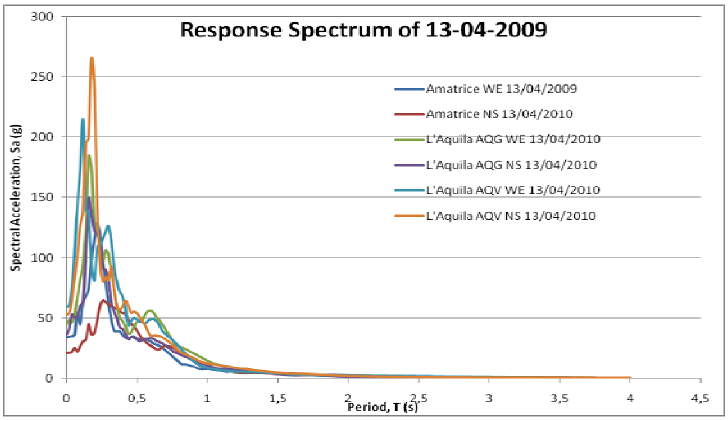

Figure 3. Comparison between 5% damped elastic acceleration response spectra from NS and EW components of accelerograms recorded at Amatrice (AMT) and L'Aquila (AQG, AQV).

Table 2. Comparison between the natural period of structures and amplification period of L'Aquila Eathquake

Station of recording	Resonat Period, s			Natural Period of Building
AMT	0,22 s	0,16 s	0,26 s	0.07 s (First Period)
AQV	0,1÷0,2 s	0,30 s	0,48 s	0.21 s (Second Period)
AQV	0,15÷0,18 s	0,28 s	0,56 s	0.35 s (Third Period)

GEOLOGICAL AND SEISMOTECTONIC CONTEXT IN THE AMATRICE AREA

The Amatrice area, before the L'Aquila earthquake, was affected by a strong historical earthquake in 1639 (Amatrice earth-quake, I =X), by a number of small-to-moderate earthquakes since 1000 A.D. (I<VIII, fig. 1) and also by three minor earthquake sequences during the last decade (August 1992, M=3.9; June 1994, M=3.7 and October 1996, M=4.0).
The epicentral areas of the 1992, 1994 and 1996 seismic sequences are located between two NNW-SSE-trending regional systems of active normal faults. In the Amatrice Basin, the outcropping stratigraphic succession consists in Early Pleistocene glacis deposits, overlaid by large landslide bodies, and by terraced fluvial deposits presumably of Middle and Late Pleistocene age (Blumetti et al., 1993).
According to the Italian Seismic Code (NTC 2008), the site can be classified in category "C".

BUILDING EXAMINED: DESCRIPTION, STRUCTURAL TIPOLOGY AND DAMAGE

The Abruzzo earthquake of 6 April 2009 caused structural damage to a residential building located in Amatrice (RIETI - Italy). Following the visit of the fire department with reported on 18/05/2009, the building showed a widespread damage in bearing walls due to the mainshock and aftershocks of the earthquake of 06/04/2009, and the need to perform work to realize safety and measures to protect public and private safety. Moscati Palace is a large natural stone masonry building that was built at the beginning of XVIII century, in Amatrice, a little town near L'Aquila (Italy) at 33km.

At the ground floor, the rooms (used as storage) are delimited by a masonry wall ith ribbed vaults that lean on square clay brick masonry columns. Upstairs there are four corridors and numerous rooms, that were originally destined to be bedrooms (as today).
The first floor diaphragm is realized through masonry vaults while the second floor diaphragm has some masonry and barrel vaults, with some areas in which the floor's realized with steel I-beams and clay tiles. In other areas, there are wooden ceilings barely attached to the walls. In particular, the Fire Brigade said they found lesions to bearing walls in some arches and wooden beams; in addition it was found that the damage induced by the earthquake affected almost all of the building with particular concentration on the higher levels and in the central sector.

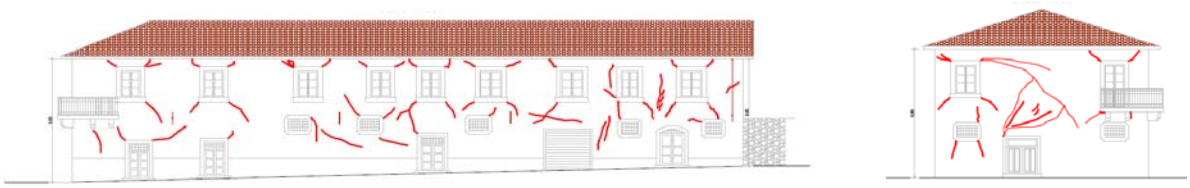

Figure 4. cracks detected on two external facades of the building

The building has a residential current-use with a typical nineteenth-century structural system and an irregular plan; The two floors above ground structure consist in perimeter walls in stone and brick bound with slaked lime and / or mortar in relation to the various interventions of transformation suffered over the decades, the diaphragm are composed primarily of:
- vaults and barrel vaults in solid brick at the ground floor;
- iron girders and small vaults in brick on the first floor, to represent the transformations of the building;
- pavilion roof in reinforcement concrete with clay tiles and beams pushing in the corners (diagonal beams).

The building is in generally good condition, just depending on the trasformations of the past for routine maintenance. A detailed analysis of the structure is the superposition of different phases of construction which in time led, by the late Middle Ages, at realize the superior floor, and the construction methods relating at the central basement of the building. For example we can mention the presence of walls in contact near the main staircase, that caused the presence of injuries, and the spontaneous generation of a friction and dissipative joint, on some hammered walls.

ASSESSMENT OF SEISMIC VULNERABILITY OF MASONRY BUILDING THROUGH NON-LINEAR ANALYSIS

Seismic assessment of existing masonry buildings is a complex problem due to the wide variety of involved aspects, such as the quality of the masonry, the structural systems, the large effort in inspection and diagnosis, the economical and cultural implications.
In the last years significant developments have occurred with respect to the possibilities of experimental and numerical analysis of ancient buildings.
Critical issues related to the seismic response of existing buildings, are the variability of traditional material properties, the different construction techniques, the limited knowledge of previous damage or the limitations in inspections and tests due to conservation issues for buildings of historical value.
On October 2007 Italian "Guidelines for the evaluation and reduction of seismic risk of buildings of the architectural heritage" were published. The above mentioned Guidelines suggest an approach based on three phases:
- knowledge acquisition;

- seismic safety evaluation;
- structural intervention design.

The scope of this approach is to create a procedure based on an accurate knowledge of the structure that indicates an objective evaluation of the seismic safety level of the building and suggests the most convenient intervention. The three phases are briefly described in the following.

Knowledge of the building

The knowledge of a building implies geometrical mapping, experimental investigation and historical research. Generally geometrical mapping is easily carried out, while
Hence, the final aim of this phase is to define a model that allows to give a qualitative interpretation of the structural behavior and subsequently to perform a structural analysis able to give a quantitative evaluation of the seismic safety. When the knowledge phase is completed, it is possible to define the confidence factor FC that will be the material safety factor to be used for the seismic evaluation. This factor is calculated through the following equation:

$$F_C = 1 + \sum_{k=1}^{4} F_{C,k} \tag{1}$$

where the four terms are based on the elaboration level concerning:
a) geometric survey; b) material survey and constructional details;
c) mechanical properties; d)geotechnical soil and foundation.
In table 3 all possible values of these terms are reported.

Table 3 Values of the 4 terms necessary to define the confidence factor

Geometric survey	Material survey and constructive detail	Mechanical proprieties of material	Geotechnical soil and foundation
Complete geometric survey FC1 = 0,05	Limited survey of materials and constructive details FC2 = 0,12	Mechanical proprieties obtained by available data FC3 = 0,12	Limited investigations on geotechnical soil and foundations structure FC4 = 0,06
Complete geometric survey and graphic representation of cracks and deformations FC1 = 0	extensive survey of materials and constructive details FC2 = 0,06	Limited investigations on mechanical proprieties FC3 = 0,06	limited investigations on soil and foundations FC4 = 0,03
	exhaustive survey of materials and constructive details FC2 = 0	Extensive investigations on mechanical proprieties FC3 = 0	Extensive or Exhaustive investigations on soil and foundations FC4 = 0

The geometrical survey was conducted with a level of detail coherent with the one utilized in the analytical model. If the geometrical survey also includes a description of cracks and deformations, FC1 can be assumed equal to 0.
The aim of the material survey (masonry typology, slab typology, vault structure, etc.) and constructive details identification (connections between walls, possible weaknesses, type of slabs and degree of connection with the walls, thrust reduction elements, material deterioration etc.) was that to individuate all the constructive typologies of the building and their localization, paying particular attention to the aspects that can trigger local collapse mechanisms.
Regarding the definition of FC3, it is important to underline that often different masonry typologies are used to realize the structure. In these cases it seems correct to correlate the FC3 factor to the masonry typology which is most relevant for the seismic analysis.

The definition of the FC4 factor depends on the influence that the foundation system can have on the collapse mechanisms: if the collapse mechanisms are assumed not to be influenced by the geotechnical parameters, it is possible to use FC4=0. Otherwise the FC4 factor must be chosen depending on the type of investigations carried out.

The seismic safety evaluation

The guidelines introduce a new model for the evaluation of seismic safety through the definition of three levels of investigation:

LV1: territorial-scale simplified seismic evaluation;

LV2: seismic evaluation to be used in case of local interventions on a building;

LV3: deep evaluation of the seismic safety of a building.

LV1 allows to evaluate the collapse acceleration of buildings by means of simplified models based on a limited number of geometrical and mechanical parameters or qualitative tools (visual test, construction features, and stratigraphic survey). LV2 has the aim to evaluate seismic safety when local interventions on single frames of a building are carried out. It is important to underline that LV2 can only be used when local interventions do not modify the structural behavior of the building. Otherwise it is necessary to use LV3. Such a level is based on the use of models that simulate the global structural behaviour of the building and allow to estimate the values of acceleration leading the structure to each limit state. These accelerations will be compared to the ones expected according to the Seismic Code.

The expected acceleration can be adjusted through a γI factor which depends on both strategic relevance and type of use of the building. For the building examined, the Italian Seismic Code proposes a value 0,8 (Type of use of the building = Frequent ; Strategic relevance of the building = Normal).The seismic evaluation is based on the Seismic Safety Index (Iss), obtained as the ratio between state acceleration and expected acceleration for the analyzed building.

$$I_{SS} = \frac{a_{LS}}{\gamma_I \, S \, a_{LS,\exp}} \tag{2}$$

ISS values larger than 1 indicate that the analyzed building is able to resist the expected seismic action; ISS<1 means that the level of seismic safety of the building is lower than the required one. It is useful to underline that the ISS check is not mandatory, but it represents an important quantitative parameter to consider, in order to express a final qualitative evaluation in which other important involved aspects (conservation and preservation requirements, safety demand, strategic relevance and type of use of the building) are considered.

In other words, it is possible to accept values of ISS smaller than 1 if it is demonstrated that interventions needed to fully satisfy structural checks are in conflict with preservation requirements.

Upgrading interventions

Structural interventions, set out in the seismic rehabilitation project (see Figure 5), aiming at seismic vulnerability reduction, have as their main objective the preservation of materials and original resistance structural mechanism, as long as it does not cause early collapse of the building. Moreover their choice must depend on results of the evaluation phase. In particular, interventions will have to reach the safety and durability of the building producing the minimum impact on it and respecting, if possible, both the original structural configuration and all subsequent modifications. From this point of view, damaged structural elements must be repaired as long as possible while element substitution and utilization of innovative systems should be avoided, unless their compatibility with original materials was demonstrated. Finally, particular attention was given to an executive phase of interventions in order to verify their effectiveness and avoid damages that could worse the mechanical properties of the masonry or framework of structural mechanisms.

Figure 5: plans of interventions on the building,

SEISMIC MODELING AND ANALYSIS

The 3D frame equivalent model

The 3-dimensional modelling of the whole unreinforced masonry URM building starts from the identification of walls and floors as bearing structure, both referring to vertical and horizontal loads. A global non-linear seismic analysis was chosen, because the linear analyses are not an appropriate tool for the assessment of seismic behaviour of existent masonry structures, while nonlinear static and dynamic analyses allow to take into account the different level of structural knowledge and follow the damage evolution and the distribution among structural elements. The local flexural behaviour of the floors and the wall out-of-plane response are not computed because they are considered negligible with respect to the global building response, which is governed by their in-plane behaviour (a global seismic response is possible only if vertical and horizontal elements are properly connected). The wall is modelled as a 3D frame equivalent model of non-linear elements (see Figure 6), which constitutive relationship is formulated to approximate the actual damage behaviour of masonry panels. The numerical models and analysis procedures, described in the rest, have been incorporated into the CDM Win program [STS Software]. The presence of stringcourses (beam elements), tie-rods (non-compressive rod elements), previous damage, heterogeneous masonry portions, gaps and irregularities is included in the structural model.

The non-linear frame-element model, representative of a whole masonry panel, is adopted by the 2-nodes elements representing piers and lintels. Rigid end offsets are used to transfer static and kinematic variables between element ends and nodes. The frame-element adopted in this work is a two-nodes bilinear elastic perfectly plastic model which incorporates the shear and flexure strength criteria suggested in the Italian Code (NTC08) and the Eurocode 6 (EC6).

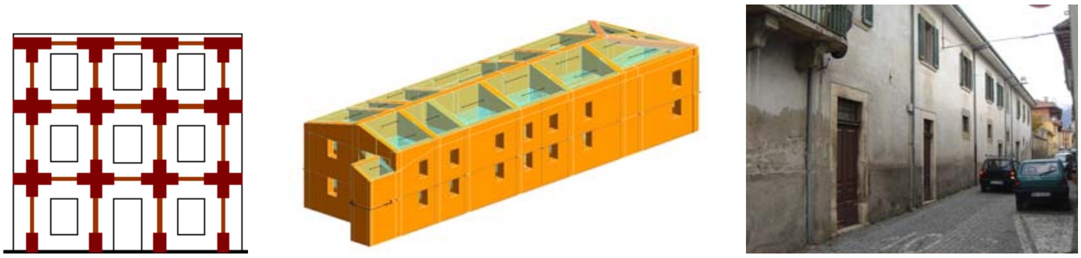

Figure 6. dimensional frame-equivalent model used for non-linear static analysis, CDM Win Model and a view of the building

Visual and in situ investigation

All in situ investigations have been carried out according to the Italian Seismic Code in order to have a deep knowledge of the building. The first phase consists of a geometrical and structural survey of the whole building (FC1=0.05) and material and crack pattern investigation (FC2=0.00). Two different masonry typologies were identified. The first was realized with grey tuff units and was utilized for most of the building walls , in the ancient portion to east of the structure; while the second typology was a clay bricks masonry, at the ground and first floor in the area near the cloister. In order to evaluate material mechanical properties, experimental tests were not carried out but the results of experimental tests and the indications on specialized literature (FC3=0.00) (Augenti, www.reluis.it, 2008) were used on the similar walls in the near areas to building examined. Mechanical caratteristics of walls are reported in the table 4. Regarding geological and foundation structure data limited investigations on foundation structure were carried out (FC4=0.06). Finally the confidence factor FC obtained is 1,11.

Mechanical model and analysis

Mechanical parameters of masonry used in the analysis (derived from the current code suggestions) are reported in Table 4.

Table 4. Mechanical proprieties of masonry typologies

Masonry typologies	f_m [MPa]	τ_0 [MPa]	E [MPa]	G [MPa]	W [kN/m3]
Grey tuff masonry	1,25	0,048	850	290	13,5
Clay brick masonry	2,95	0,125	2350	530	17

In order to carry out analysis according to Italian Seismic Code, in situ expected maximum horizontal acceleration has been calculated considering geographic coordinates. The obtained value has been multiplied by γI =0,8 factor to consider strategic relevance of the building (normal) and its expected use (frequent). In this way a seismic event was considered having an excess probability, in the period of 50 years, of 6,5% for severe damage limit state (SDLS) and 40% for limited damage limit state (LDLS).

Moreover a S=1,2 factor was introduced in order to take into account the effects of the topographic configuration on the seismic behaviour of the building placed on the ridge.

Building structural behavior analysis has been carried out in 3 phases: the first one concerned analysis on original building conditions. This analysis, identified by OM-1 code, has allowed highlighting structure mechanical behavior in case of earthquake. Moreover analysis carried out in this phase allowed to individuate all local collapse mechanisms and understanding that the main local mechanisms predictable were overturning mechanisms out of plane for walls on north and east sides. In the second phase a new structural model was created in which several structural elements were designed in order to avoid local collapse mechanisms. This new model, identified with FM-1 code, has highlighted the good effect of interventions: local mechanism activation was avoided and the seismic safety index related to SDLS has passed from 0,83 to 1,22 with an increase of over 45%. This proves that provided interventions, blocking the activation of the local mechanisms, had a beneficial effect also on the whole building behavior. This phase of the study is aimed to carry out analysis to evaluate seismic behavior of the building after interventions. Due to building degradation, the substitution of the existing covering floor with reinforced concrete riddles and a wood floor having a good connectivity to the wall was provided.

Table 5. Result of analyses

Model of Analysis	Expected accelerations		Seismic assessment				Local mechanism activation
	$a_{SDLS-exp}$ [m/s2]	$a_{LDLS-exp}$ [m/s2]	a_{SDLS} [m/s2]	a_{LDLS} [m/s2]	$I_{SS,SDLS}$ [-]	$I_{SS,LDLS}$ [-]	
OM-1	3,98	1,96	3,31	1,84	0,83	0,93	YES
FM-1			4,87	2,21	1,22	1,13	NO

According to "Guidelines for the evaluation and reduction of seismic risk of buildings of the architectural heritage", in Table 4 results of the two analyses are reported. Particularly, information about local mechanisms activation, expected peak ground accelerations at the base of the structure related to both SDLS and LDLS are reported. Moreover accelerations that cause the attainment of both SDLS and LDLS and related seismic safety indexes are reported. The nonlinear static analysis, incremental, has provided the results for single step of calculation (shear to base and displacement of checkpoint), which was built with the capacity curve.

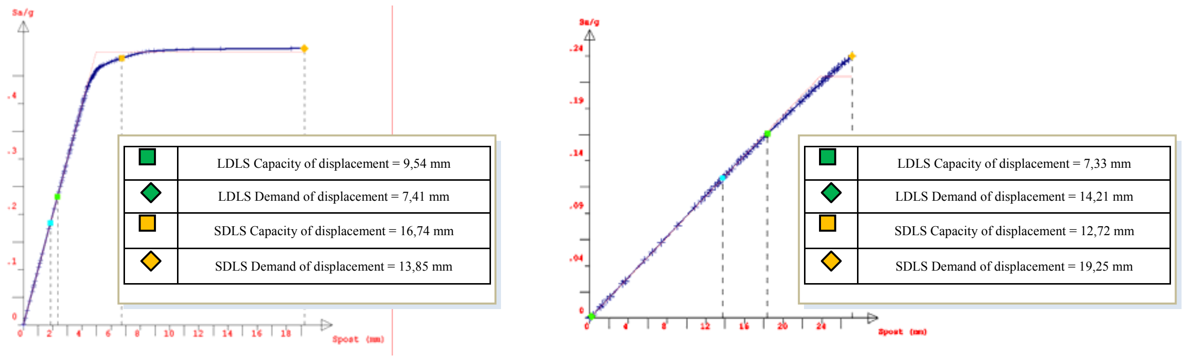

Figure 7. capacity curves obteined for OM-1 and FM-1 models, with non-linear static analysis

In Figure 7 the capacity curve on the ADSR Spectrum are reported, for OM-1 and FM-1 models; the interventions on the building provide an increase of ductility structure: the ability to shift becomes greater than the displacement demand.

In particular, figure 8 displays at the ends of any individual auction hinges colored according to their commitment plastic (and the sequence of the formation of plastic hinges on the structure, with different colors for different levels of damage). Please note that the rotation is slightly damaged at the yield point, while the damage is severe and last 3/4 of the rotation, and for the collapse of the last rotation. Any mode of collapse appears fragile with double zipper with color corresponding to the situation of collapse. The results indicate that the damage found after the mainshock (06/04/2009) and aftershocks was almost similar to that provided by the 3D-model. The area of the collapse of the structure is central, near the stairs, where there is a generic nucleos that has worked as a fulcrum for torsion.

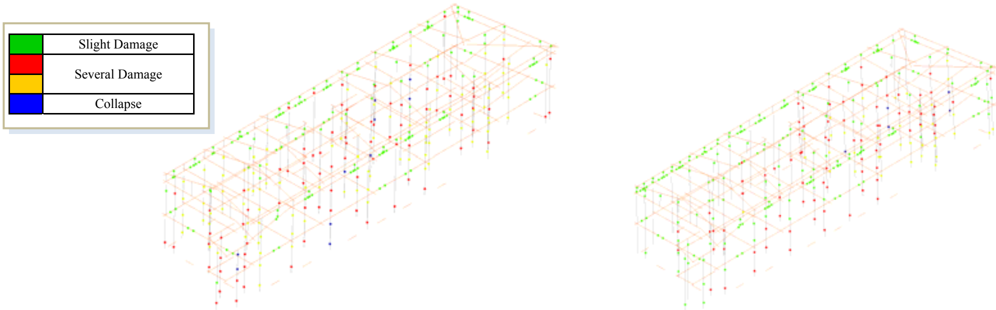

Figure 8: sequence of the formation of plastic hinges on the structure, obtained from OM-1 and FM-1 models (with non-linear static analysis)

From data reported in table 5, it is possible to notice that interventions with tie rods, as well as having prevented local mechanisms activation, have considerably improved the seismic safety index at severe damage limit state, while substitution of the covering floor did not change the mechanical global behavior of the structure. In fact it is possible to observe that passing from original configuration (OM-1) to the final configuration (FM-1), there is a noticeable increase of ISS.

CONCLUSION

Seismic vulnerability analysis of the building, has highlighted the importance of investigation phases that guarantee a deep knowledge of the building and help to create correct structural models. Building analysis, organized in three phases, allowed to evaluate the global level effectiveness of interventions realized to avoid collapse local mechanisms. From data reported in table 4.3.2., it is possible to notice that a considerable improvement of the seismic safety index at SDLS was achieved due to the introduction of steel tie rods and reinforced concrete riddles. Infact, the building's vulnerability is mainly due to the lack of connection between orthogonal walls and between walls and floors. Proper connection devices (e.g. tie-rods), following the result of the pushover analysis for FM-1 model, can increase the seismic safety with respect to local damages and they allow the building to behave as an entire structure with a seismic response governed by the in-plane behaviour of walls and horizontal structures (floors, vaults and roofs). The interventions proposed for the structure increase the global ductility, prevents the activation of local mechanisms of collapse, and restore the building after the damage suffered by the L'Aquila earthquake of 06/04/2009.
The building so, it becomes appropriate at the Italian Seismic Code (NTC 14/01/2008).

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