Edifice A of the Engineering Faculty of L'Aquila: earthquake damage scenario assessment through nonlinear analyses

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ABSTRACT
Several structures, damaged during the 6th April 2009 earthquake, need important interventions on both the structural and non-structural components in order to improve their seismic behavior. In this context, the Edifice A of the Engineering Faculty of L'Aquila, at the top of Roio hill, composed by seven irregular RC substructures, represents, due to the particular damage scenario, a significant case of study. The present paper discusses the ability of geotechnical and structural modeling in reproducing the structural seismic behavior which justify the observed damage or undamaged scenario for both main and supported components. Firstly, the recent results obtained in site characterization are summarized to obtain a reasonable estimate of the main shock characteristics. Secondly, the main modal properties of the structures are reproduced with the updating of finite element models on the basis of minimization error procedures between measured and modeled quantities. Thirdly, the nonlinear behavior of the structure has been investigated by classical pushover analysis to determine the capacity level reached during the main shock comparing it with on-site tests and observation. Finally, the seismic behavior of the collapsed facade is examined showing the effects of the experienced structural displacements on the overturning of split-face brick walls.

1 INTRODUCTION

During the earthquake which struck the city of L'Aquila, the 6th April 2009, several buildings were seriously damaged. An extensive activity has been carried out to understand the seismic structural behaviour of the impoverished real estate sometimes thanks to functioning monitoring systems (Spina et al. 2011). Numerous investigation efforts aims to achieve an overall comprehension of structural (e.g., Augenti et al. 2010, Ceci et al. 2010a, Ricci et al. 2011) and non-structural (Braga et al. 2011) vulnerabilities under the seismic input. Starting from a first qualitative survey of damage (e.g. Beolchini et al. 2009, Ceci et al. 2010a), developed in the post-event period, the investigation has been deepen through in situ tests (Ceci et al. 2010b, Modena et al. 2010) and numerical analyses (Di Sarno et al. 2011). Several case studies, varying in depth-examination, according to a predefined classification procedure, were developed on specific cultural heritage buildings of L'Aquila in order to understand the seismic behaviour, the damage pattern and to propose a possible restoration intervention on structural and non-structural elements (Carocci et al. 2010, Mucciarelli et al. 2011).

Minor attention was still devoted to analyze the behaviour of RC buildings during the seismic event and to define the damage scenario. Nonetheless an extensive photographic documentation is available in many national and international reports (e.g. Augenti et al. 2010, Fanale et al. 2009, Ricci et al. 2011).

On the whole, the RC buildings demonstrated a good structural behaviour during the seismic sequence. In some cases, the bad compliance with constructive details in the structural elements of older buildings, as the prescription on the node confinement or the spacing and diameter of transversal steel reinforcement, caused low ductility and some localized fragile damage, as concrete injection, buckling of longitudinal steel reinforcement and loss of anchorage for both longitudinal and transversal reinforcement steel.

Instead, concerning the non structural elements, the most frequently observed damage is represented by the collapse of the infill walls, with the overcoming of the serviceability limit state (SLS) of the building. The collapses of infill walls, caused by out-of-plane overturning or by in-plane shear rupture, determined severe economical consequences and they could cause human casualties. To this reason, the loss of functionality represents nowadays, as suggested by the newest codes, a central aspect in the structural design both in strategic and ordinary buildings.

The preliminary investigation activities puts in to evidence the reliance of infill damage on several aspects referable to the seismic action, the geometric and resistance characteristics of structural frame and, not least, the constructive methods of infill walls.

Due to the extensive attention devoted by a research group of the *Training and Research Earthquake Engineering Centre* - CERFIS belonging to the University of L'Aquila, the case study of the Edifice A of the Engineering Faculty, is, here, taken into account.

The Edifice A represents one of the recently-built edifices of the Engineering Faculty Campus of the University of L'Aquila, seats at the top of Roio hill, at S-E of the city. The building host many classrooms, the Faculty Dean office, the student secretary and the laboratories of structures and materials and water engineering.

Due to the ground slope, the Edifice having four levels, presents two of these partially underground.

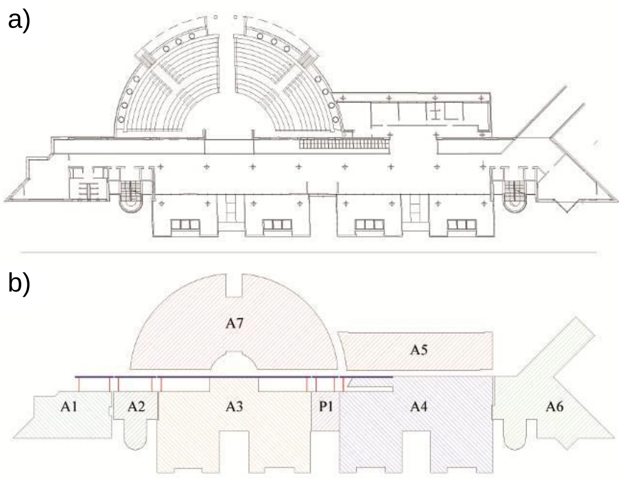

Figure 1: The Edifice A: (a) plan at the entry level, (b) distinction of each substructures.

The external covering is realized by glass curtain and split-face bricks. The principal entry, placed at the 3rd floor, which is hidden by a semicircular bleacher, called "amphitheatre", is overtopped by a 4.50 m height glass curtain, and surrounded by 9.00 m height split-face bricks walls.

At the opposite side, the last two levels are tapered, and are covered uniquely by wide windows. Transversally, perpendicular to the principal facade, a few partition walls divide the different classrooms.

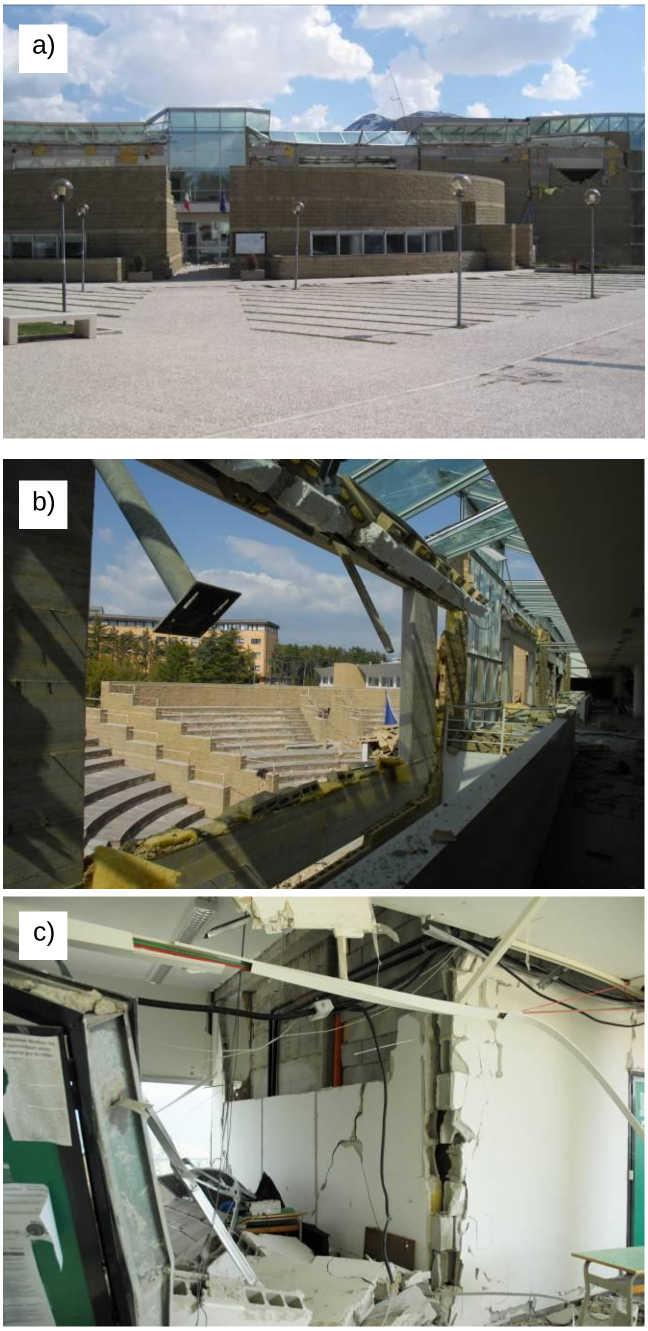

Figure 2. The damages scenario after the mainshock: (a) damages on the principal facade visible from the Campus entrance; (b) damages on the covering glass sheets and the metallic tubulars; (c) overturning and shear cracks on the internal partition walls.

Finally a large glass sustained by a metallic frame were installed longitudinally, to cover the gap between the facade and the remained structures. Even though it appears as unique building, the Edifice A is composed by seven independent RC substructures (A1-A7), with adequate seismic joints. Each substructure has significant irregularities in the geometry characteristics, mass and stiffness distribution. The principal facade covering is supported by planar RC frames which are connected with the principal substructures through RC slabs and horizontal metallic tubes. Between the substructures A3 and A4, simply supported RC slabs are placed at each floor. The resistant structures are realized by shear walls positioned mainly at the low levels and along the perimeter, while internally and at the top two floors, a series of rectangular and circular columns are disposed. Wide beams in the slab heights represent the majority of horizontal elements in the intermediate floors. Two elevator core, and the staircases are located in the substructures A2 and A6; while in A6 a passageway connects the Edifice B. The roof of the substructure A7 sustains the passageway to the main entrance.

The paper aims to described the progress in understanding the overall seismic structural behaviour of Edifice A. An effort to define a realistic level of the reached structural capacity under the main shock is presented. Damage of non-structural elements are discussed on the basis of model response. The acquired knowledge permits specific comments on the retrofitting interventions.

2 SITE EFFECTS

At 01:32:39 UTC (03:32:39 at epicentre), the 6th of April 2009, a devasting earthquake struck the city of L'Aquila and surrounding villages in the Abruzzo Region of Central Italy. The magnitude of mainshock was estimated to be M_w=6.3 (moment magnitude scale) by the UGS – US Geological Survey, or M_w = 6.2 and M_L=5.8 (Richter magnitude scale) by the Italian INGV - Earthquake Remote Sensing Group. The epicentre was located at depth of 8.8km at 5km SW from the city of L'Aquila (42.423°N, 13.395°E) (Akinci et al. 2010). Thanks to a wide covering by the National Accelerometric Network (55 strong motion stations) a great quantities of data in acceleration, velocity and displacements are available to well described the earthquake sequence.

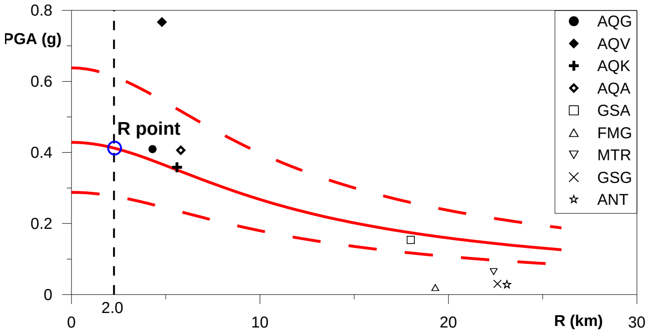

Figure 3. Seismic mainshock mitigation curve with the accelerometer stations closer than 30km. The Engineering Faculty is about 2km from the epicentre (R point, Roio).

Significant data were produced by 21 stations, of the National Accelerometric Network, situated less than 100km from the epicentre, fourteen being in the Abruzzo Region; four of them are placed less than 10km from the main shock epicentre. The horizontal accelerations registered by the stations closer than 30km form the epicentre, was token into account to analyze the mitigation effects in the region (Figure 3). Except for the peak ground acceleration registered by the AQV station, all the stations follow substantially the standard mitigation curve (Sabetta et al. 1987), defined starting from the epicentre magnitude.

The construction of the mitigation curve allows a first preliminary estimation of the PGA at the Roio site. Indeed, both Figures 3 and 4 evidence that the epicentre is closer to the Engineering Faculty than to the AQG station. The distance of 2km from the epicentre permits the estimation of the PGA (0.41g) based only on the position of the site in the attenuation curve (R point in Figure 3). Therefore, the here presented preliminary evaluation is coherent with the site condition of the AQG station, neglecting any local effect at Roio site.

Figure 4. Locations of AQG station and Roio with respect to the epicenter. In the frame the polarization (120°N) of seismic waves due to 3D local effects.

However, this approximation permits the evaluation of the spectrum based on the estimation of the PGA. In Figure 3, looking at the very similar PGA values of AQG and the R-point, it has been assumed that the regularized elastic response spectrum of the main shock at Roio can be desumed by the regularization of the spectrum evaluated through the registration at AQG. This process has permitted to identify the parameters of the NTC08 code spectrum which better matches, in the plateau, the AQG spectrum which are reported in Table 1. Direct comparison is shown in Figures 5 and 6, evidencing the effect of different level of structural damping (2-5-7%).

Moreover, one of the discussed aspect of the 2009 seismic sequence, was represented by the influences of site effects on the observed damages. A deepen inspection of the damaged buildings and other engineering structures permitted to show a map of intensity distribution (Is, MCS scale) of the interested region (Galli et al. 2009). The highest level of Is (≥ 9.0) were distributed along the NW-SE direction, according to the fault orientation. A devasting effects were documented on the city of L'Aquila and in many villages on the Roio hill (Is=8.0).

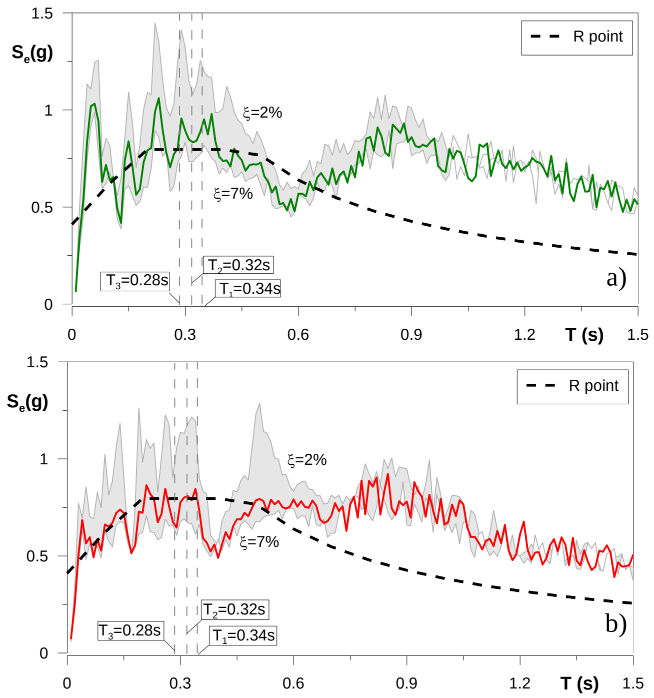

Figure 5. The response acceleration spectrum to the AQG main shock (ξ=5%) compared with the elastic spectrum relative to the target PGA (R point in Figure 3): (a) N-S component, (b) E-W component. The dashed gray-lines represent the three principal modal period of the substructure A3.

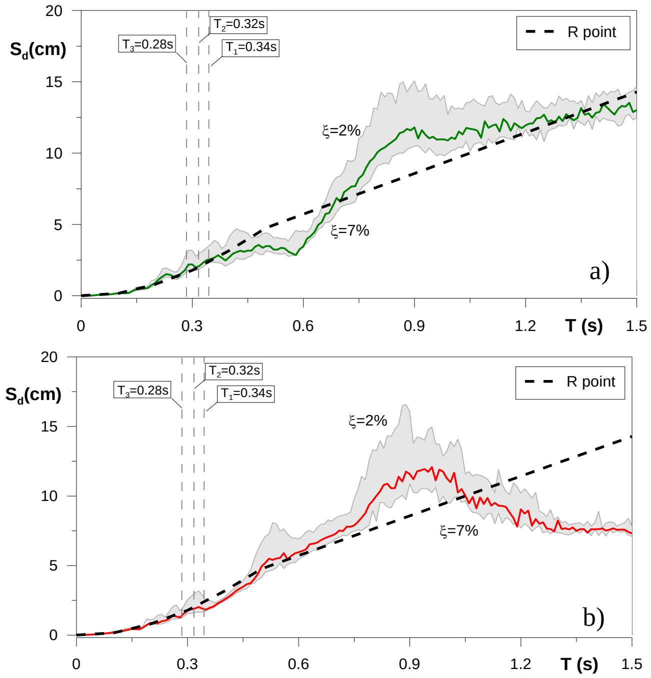

Figure 6. The response displacement spectrum to the AQG main shock (ξ=5%) compared with the elastic spectrum relative to the target PGA (R point in Figure 3): (a) N-S component, (b) E-W component. The dashed gray-lines represent the three principal modal period of the substructure A3.

It has recognized that probable site effects contributed to amplify the shock waves in many villages put on the top of hills, depending also on the geological stratifications.

The study were supported by extensive microzonation campaigns, based on the analysis of aftershock-swarm, starting few days after the main shock. These tests aims to analyze the site effects caused by particular topography and geological features of the hit region.

Comparison between the measured accelerations and the observed damages evidences the possibility of local amplification of the ground motion (Akinci et al. 2010, Bertrand et al. 2011, Marzorati et al. 2011). Focusing the attention on Roio hill, the site amplification effects were analyzed taking into account 152 aftershocks by a six stations network (Bertrand et al. 2011). A summary of the obtained results is here presented in view of the discussion of the damage scenario related to the estimate of the main shock.

Table 1. Parameter for the definition of the spectra in Roio hill (R point).

	a_g	Fo	Tc*	Soil	St	ξ
R point	0.41g	1.28	0.356	B	1	5%

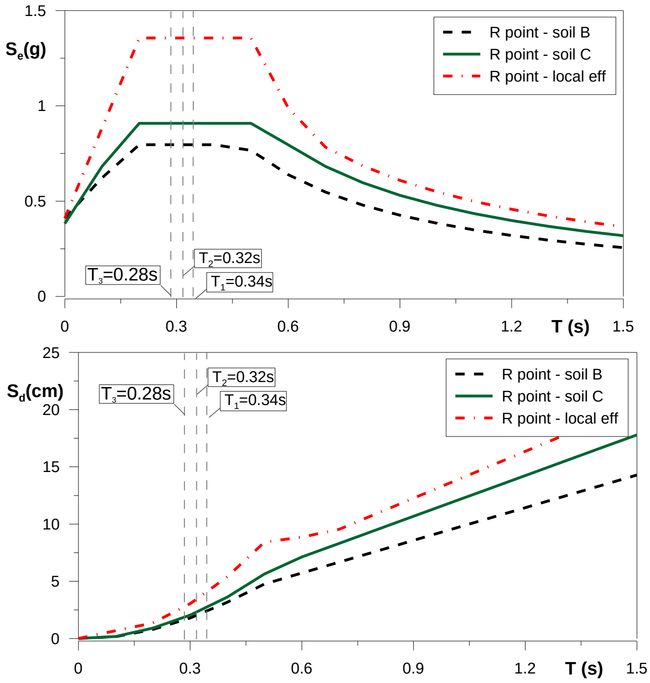

Figure 7. Comparison between the AQG average spectrum, the Roio spectrum and the hypothesized spectrum due to the local effects. The dashed gray-lines represent the three principal modal period of the substructure A3.

Colle di Roio is characterized by the overthrust of the Mesozoic carbonatic deposits above a Miocenic clay-marls succession, with a N130° trending asymmetric anticline.

The conducted tests examined the ground motion at the *Engineering Faculty Campus*, *Poggio di Roio* (two stations), *Colle di Roio*, *Roio Piano* and *San Ruffina*. Signals were processed numerically to obtain the transfer functions between different nodes of the network. The results suggest an overall amplification effect both on alluvium-based and rock-based at the edges of the plain stations. The resonance frequency is characterized by an high variation from 1.3Hz to 4.00Hz. The Engineering Faculty station exhibits an high seismic amplification of horizontal component, especially around 3.6 Hz, with the amplification factor equal to 2 in the range 2-10Hz.

Here, the results of the site characterization are roughly synthesized in terms of elastic response spectra in Figure 7. In particular, firstly is taken into consideration the amplification from soil B-type to soil C-type provided by the National code (NTC08), then the amplification evaluated from the study of the site effects is taken into account in the spectrum with local effect (dot-dashed line in Figure 7). Moreover, a 3D effect which polarized the seismic waves was evidenced. In particular, the maximum amplification denoted in the closer station to the Faculty of Engineering, was obtained for azimuth close to 120°N, as reported in the frame of Figure 4.

The absence of a direct transfer function between the AQG station and the Engineering Faculty node, for the previously presented network, does not allow a complete reconstruction of the main shock characteristics felt by the structures of the Edifice A. However, the previously discussed results permit the determination of a reasonable range in which is contained the elastic response spectrum of the main shock of April 6 at the Engineering Faculty site.

Furthermore, a discussion on the occurred damages with respect to the acceleration estimate can be developed (Marzorati et al. 2003, Irtem et al. 2007), as presented in the following sections.

3 DESCRIPTION OF DAMAGE SCENARIO

Immediately after the mainshock, which occurred at 6^{th} April, 2009, the important level of damage which has been occurred at the Edifice A, was examined either to understand the structural deficiencies under seismic loads, and to register the damage evolution during the aftershocks.

The main emblematic damage scenario was represented by the overturning of portions of split-face bricks walls on the principal façade (Figure 8). Different profiles of damage were visible in the external and the internal layer. Due

Figure 8. The front view of the principal facade, with the description of the damage profile (red line).

to the collapse of the split-face bricks the metallic tubes were seriously damages, along the element and in the anchorage.

The cover glass sheets and the metallic frames were collapsed, probably due to both the relative motion between the planar frames and the principal substructures and the overturning of the walls which supported the structure on the facade-side.

While the external substructures (A1, A2 and A6) were softly damaged, the highest level were visible on A3 and A4; in particular in the last two floor, starting from the entrance, where the transversal length of the edifice were reduced. Internally, in A3 and A4, the transversal partition walls, which did not oppose significant resistance to the drift motion, presented X-shape cracks. The longitudinal partition, which divided the classrooms from the common areas, overturned.

The partially underground floors, due to the presence of wide shear walls appears safe without significant non-structural damages. In all substructures, the RC structural elements appears undamaged. The first surveys put into evidence the exceptionality of the seismic action which provoked important relative displacements between the floors, in special way, where the shear walls were interrupted.

The lower floor, where wide shear walls surround all the substructures, the drift floor did not provoked damages.

The most vulnerable aspect was represented by the brick walls of the facade, which overturning out of plan without any contribution in the dissipation of the seismic energy. Also the internal partitions, due to their low thickness and extension did not furnish resistance during the seismic shock.

4 STRUCTURAL MODELLING

Several different FE models were realized to reproduce the structural behaviour of the Edifice A. The FE models were continuously improved thanks to the information derived firstly by the visual surveys, and subsequently by the on-site testing. A fruitful updating were conducted based on the identified modal properties. Soil-structure interaction at the base foundation has been also considered in some stages of the study.

The models represent only the resistant frames without considering the possible increase of stiffness and damping due to the non-structural elements. This choice should be considered strongly realistic, in the studied case, due to the particular constructive method used for the face-split bricks, which are mounted externally to the RC frame. Moreover, the limited number of partition walls, used to divided the classrooms, together with small thickness and small connection furnishes a negligible augment of stiffness. The modelling effort aims to understand the seismic behaviour of each substructures, to verify the level of adequacy respect to the Italian Code, and, finally, to design the retrofitting.

4.1 Linear model updating

To achieve a reliable knowledge of the dynamic behaviour of each substructures a series of dynamic measurements under ambient vibrations were registered on the A1 and A3 substructures (Foti et al. 2011). The attention was focused mainly to analyze the structures behaviour in the out-of-plane facade direction and the latero-torsional behaviour. The modal characteristics of each substructures were identified through output-only procedures, already used in previous studies (Antonacci et al. 2011, Lepidi et al. 2010) and thanks to the collaboration with a research group of Politecnico di Bari. The measurement testing, which were conducted on the damaged structures, give valuable information on the modal characteristics of the substructures; in particular the role of coupling, considering the reduced number of metallic tubulars, were examined. The lower natural frequency of substructure A1 is equal to 3.86 Hz. A roto-translation shape characterizes the main mode, with prevalent translation in the y transversal direction, and in-phase motion of the facade and the structure. All the identified modes are characterized by a roto-translational shape. Substructure A3 is characterized by the lower natural frequency of 2.77 Hz. The first natural mode is characterized by a y transversal translational shape in the structure and a symmetrical roto-translational shape in the two sides of the facade with a prevalent y direction. Where the interconnections are lacking, out-of-phase motion is possible. The second and third modes are characterized by roto-translational shape both in the structure and in the facade. Due to the position of accelerometers, the different shape of facade are localized at the last floor, over the level of the principal entry.

A measurement campaign will be planned on the substructures during the retrofitting intervention operations and at their conclusion to compare the different behaviour due to the influence of the works done in the structural and non-structural parts. In particular, the measures aim to understand the modified behaviour of the facade planar frame after the increase of stiffness due to the connected split-face brick walls, both internally and externally, in absence and presence of coupling system.

4.2 Nonlinear model updating

An exhaustive and coordinated on-site testing campaign was conducted by the researchers of the CSE-DISAT and CERFIS of the University of L'Aquila on the structural elements of the Edifice A to improve the level of knowledge. A series of inspection in situ was done to assure the correspondence of the design geometric characteristics with the real structures. Moreover, an exhaustive campaign of material tests, through non destructive and destructive methods, aims to define the mechanical characteristic of the concrete, the presence of steel reinforcements, coherently with the design plans, and their mechanical characteristics.

The testing locations were carefully selected at different points of each floor level to represent the spatial distribution of the material characteristics on each substructure.

The results show relatively high values of the concrete compressive strength: both the underground levels and the highest level are characterized by R_{ck}>35MPa, with a maximum of 41.40 MPa at the first underground level. Instead, the lowest value of the compressive is related to the ground level strength (26.42 MPa).

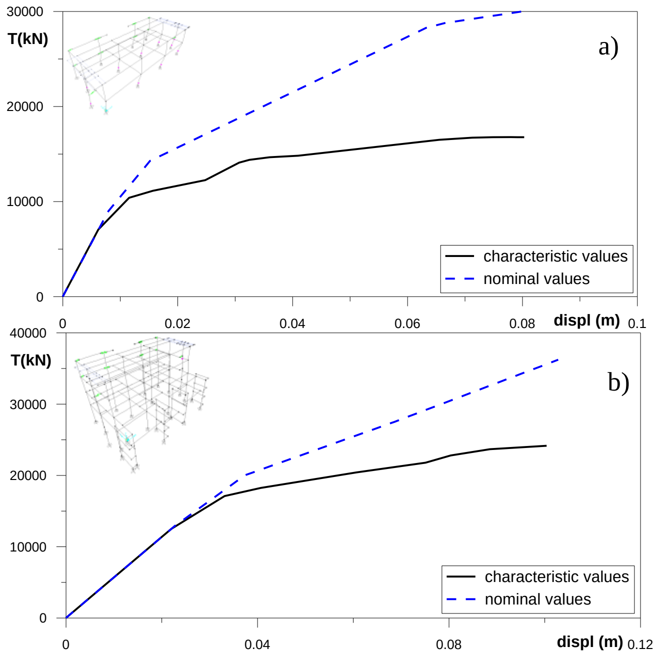

Figure 9. Structural capacity in out-of-plane facade direction using the material resistance characteristic values and nominal values in (a) FE reduced model (3rd and 4th floor); (b) complete FE model.

A core drilling specimen on one of the transversal shear walls which achieves the top of the substructure A3, showed a cracked RC enable the possibility to realized correctly the compression tests. This occurence, which could be caused by a cyclic exciding of the elastic behaviour of the structural element, represents an useful information to understand the level of capacity achieved during the seismic event by the substructure.

Finally, the testing on steel specimens assigned the steel reinforcements in FE44k class, according to the Italian Standard.

The effects of the assumed values for the material resistance on the nonlinear behaviour of the substructure A3 were examined through a pushover analysis. The capacity curves in the out-of-plane facade direction considering the mean and the characteristic values are reported in Figure 9. The analysis shows a significant increasing of resistance due to higher values of material strengths which does not correspond to an increasing of ductility, with the ultimate displacement capacity that remains constant.

5 SEISMIC RESPONSE ANALYSIS

An examination of the seismic performance has developed, focused the attention on both structural elements and split-face brick walls.

Firstly, the linear models were previously used to examine the modal characteristics of each substructures (Table 2). The analysis of modal properties of the substructures, some of which are characterized by the coupling with the planar frames, shows different shape of displacement due to translational and torsional component of motion. In particular, an important latero-torsional coupling is detected in substructure A1 and A6, due to their irregularity. The substructure A3 and A4 are characterized by regular motion in the facade out-of-plane transversal direction. Finally, for the high irregular geometry of A6, the first natural mode results to have a translation in the *x* longitudinal direction.

Table 2. The natural modes of the five substructures which sustain the principal facade, in terms of frequency and the mode shape. (R=rotational, T=translational, *x*=longitudinal direction, *y*=transversal direction).

A1	A2	A3	A4	A6
3.86Hz	4.31Hz	2.94Hz	2.51Hz	1.84Hz
(R-T*y*)	(R-T*x*)	(T*y*)	(T*y*)	(T*x*)

5.1 Nonlinear structural behaviour

An improvement of FE models, through the definition of non-linear material constitutive laws, allows to conduct a series of non linear static analyses to define the capacity curve of each substructure in both the principal directions. Due to the severe damages occurred, and based on inadequacy detected during preliminary linear analyses with the design spectra, in terms of lateral drift at different levels, as requested by the National Code, the non-linear behaviour of substructure A3 was deeply examined.

The most vulnerable aspect was represented by the interruption of the shear walls at the entry level, left the circular columns characterized by an insufficient level of stiffness.

Two different FE models were developed: one of which represents the whole substructure, and the second one that neglects the more stiff part, partially at the underground level, considering only the 3rd (the level of the principal entry) and 4th floor.

Moreover, the non linear model of A3 was utilized to better understand the role of the tubular tubes and the RC slabs in the coupling between the principal structure and the partially connected planar frame of the facade. Different hypotheses on the tubular connection were done: no connection, rod connection and beam connection. While in the first hypothesis the planar frame are connected with the principal structure uniquely by the RC slabs, in the other cases the presence of tubular, considering all the elements as in the undamaged structure, were take into account as either only the axial stiffness (rod connection) or the axial stiffness and bending moment (beam connection).

In Figure 10 the capacity curves of each model, following the different hypotheses are reported. The incremental forces applied in facade out-of-plane direction, are proportional to the principal mode shape in that direction. A significant difference between the case with facade connected with the tubulars or only with RC slab coupling can be appreciated.

Indeed, the tubulars sustain the shear walls of the planar frame, transferring axially the tension in the three-dimensional structure, assigning more ductility to the whole substructure.

Although, a few differences on the global behaviour are visible between the rod connection and the beam connection. These small differences are due only to the low rotational stiffness of the tubular modelled as rod, which leaves unchanged the stiffness ratio between the planar frame and the main system. In the complete model, there are larger differences between models using rod connection or beam connection.

Taking into account the FE models with rod connections which result the more realistic, confirmed by some basilar deductions on the technical anchorage characteristics, the capacity curves are compared with the demand, in terms of maximum displacement, considering both the spectrum at Roio with soil type-C and the spectrum which included the local effects (Figure 11). In both cases the demand is quite low with respect to the ultimate displacement capacity.

Moreover, the demand is lower than the displacement required to achieve the first yielding in one element (represented as the blue point on the graphs), which occurred at the two external shear walls transversally oriented, the only ones that arrive at the top of the structure. Considering the complete model, the structure result more flexible and the demand, in terms of maximum displacement, increases. Taking into account the spectrum of local effects, the demands result rather close to the first yielding point.

This consideration can explain the anomalous cracked concrete found during the material testing, only in the shear walls.

This results are coherent with the investigation tests and the in situ surveys, confirming that the substructure suffer uniquely of large displacement which damaged the non-structural elements but left undamaged the structural elements.

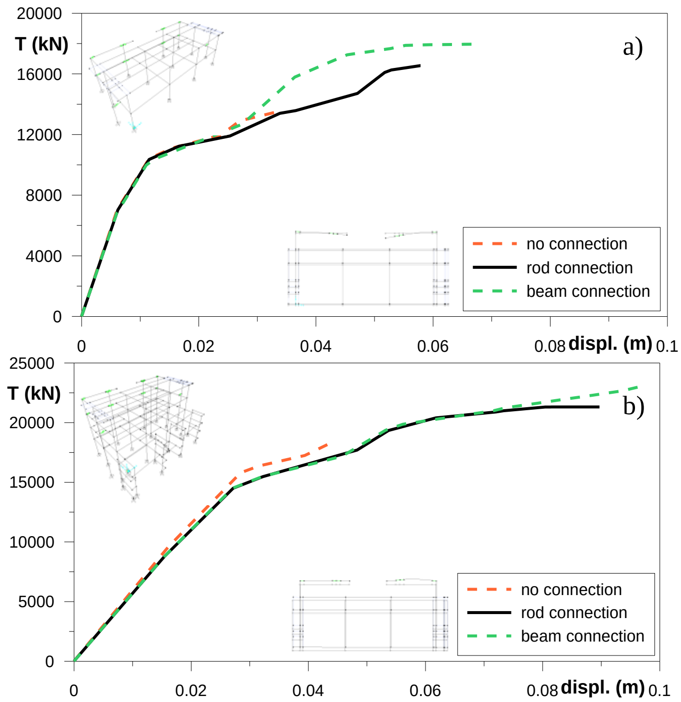

Figure 10. Structural capacity in out-of-plane facade direction with different connections between the facade and the main structure: (a) FE reduced model (3rd and 4th floor); (b) complete FE model.

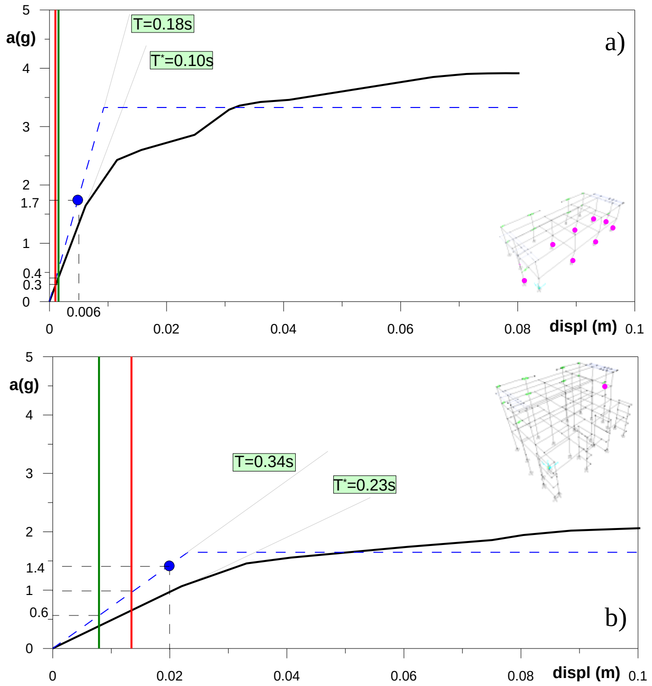

Figure 11. Structural capacity in out-of-plane facade direction, with the displacement demand of main shock, taking into account the (a) FE model reduced at the 3rd and 4th floor, and (b) the complete FE model. The green line represents the demand of Roio soil C spectrum, the red line takes into account the local effects.

5.2 *Deformability in the elastic range*

Some information on the non-structural elements behaviour could be obtain through the evaluation of inter-drift performance through linear analysis on the complete FE model of the substructure A3 (Figure 12). The values obtain both neglecting and considering the local effects were compared with the limitations defined by the Italian Code. While the lower grounds satisfy the limitations, the higher level, due to the stiffness reduction, exceed them.

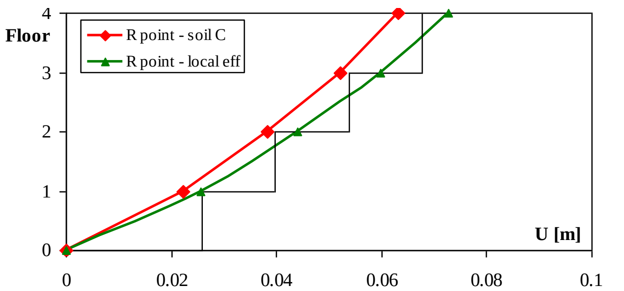

Figure 12. Inter-drift evaluation in A3: comparison between the seismic demand and the Italian Code limitations (black line).

5.3 *Split-face brick wall behavior*

A preliminary considerations were done on the out-of-plane capacity of the face-split brick walls, against the seismic action, considering a monolithic overturning behaviour. However the damage scenario was probably caused by the cyclic nature of the seismic action, the particular constructive method, which was realized through two external layers respect of the planar frame, allows to consider the wall as independent element, horizontally supported by the planar frame. Except for the substructure A1, the damages profile appear essentially horizontal suggesting the monolithic collapse of the non-structural elements. In Table 2, the values of capacity in terms of PGA and maximum displacement, trough, respectively, linear and nonlinear analyses, and the respective adequacy levels, were calculated for each damaged substructures. An overall inadequacy of the collapsed walls were demonstrated. Subsequently, the minimum value of horizontal force which the connections between the wall and the planar frame, will be able to provide to achieve the full adequacy against the seismic action, was calculate.

The analysis aims to highlight the importance of a proper connection between the planar frame and both the layer of the facade wall to avoid disastrous collapses.

6 CONCLUSIONS

The paper aims to describe, through the case study represented by the Edifice A of the Engineering Faculty of the University of L'Aquila, the seismic behaviour of existing RC structures during the main shock of the 6th April 2009. A discussion on a realistic seismic main shock which struck the Roio hill is presented. Moreover, based on a microzonation study, which analyzed the effect of amplification around the Roio hill, some deductions on the local effect around the Edifice A were done. A careful attention was devoted to the substructure A3 which presents the higher level of damage either on the facade and on the principal structure.

Table 3. Level of adequacy of the collapse portion of the split-face brick walls. W and H represent the wide and height of the portion.

	A1	A2	A3	A3-A4
W (m)	7.60	8.20	30.4	5.20
H (m)	4.20	1.40	5.60	8.40
PGA (g)	0.410	0.410	0.410	0.410
a_g (g)	0.047	0.176	0.042	0.033
$\alpha_{lin}=a_g/PGA_{slv}$	**0.11**	**0.43**	**0.10**	**0.08**
d (m)	0.178	0.104	0.208	0.246
d_u (m)	0.029	0.029	0.030	0.028
$\alpha_{nonlin}=d_u/d_{slv}$	**0.16**	**0.28**	**0.14**	**0.11**
T (kN/m)	1.22	0.16	1.40	0.03

A nonlinear static analysis was developed, based on the FE models improved by continuous updating effort, on both a reduced and complete model, to define the capacity curve of the substructure, focusing the attention on the principal facade out-of-plane direction. The role of the metallic tubular in the coupling of the planar frame with the three dimensional frame was examined; the whole structure benefits of the presence of the connection with a significant increasing of ductility. Finally, the comparison between the capacity curves and the predicted demand during the main shock demonstrates the coherence of the results with the in situ surveys and investigation tests. Indeed the demand, either neglecting or considering the local effects, is lower than the displacements related to the first yield in the structural elements. Some linear analyses, allow some deductions on the damage pattern on the facade walls. The modal analysis suggests the vibration shapes of the planar frames which sustained the walls, which are characterized by different profile of collapse. Moreover some local analyses, considering collapse due to monolithic mechanisms, were developed. A substantial inadequacy of all the collapsed walls were detected. These results evidences the importance of a proper anchorage between the non-structural wall and the planar frame. An overall understanding of the substructure A3 behaviour was achieved, furnishing exhaustive analyses on the linear and nonlinear structural behaviour.

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