Analysis of the "Casa dello Studente" collapse during the L'Aquila 6th April 2009 earthquake

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SUMMARY:
The partial collapse of a seven-story RC building with two underground levels, occurring during the April 6 2009 earthquake of L'Aquila (Italy), is analyzed with a comprehensive multidisciplinary approach taking into account all the possible significant factors. Attention is paid to the building history – from 1965 to 2009 – crossing transfers of ownership, refurbishment works and different code regulations; to the quality of the in situ cast material; to the ground motion actually experienced on the site. The comparative analysis between the original design and the picture of the collapse as visible immediately after the earthquake has allowed the explanation of the chain of events leading to the collapse itself. Additional data on material properties and ground motion, together with numerical modelling and analysis, led to the determination of the collapse causes.

Keywords: case study; soft-story mechanism; partial collapse; non-ductile reinforced concrete frame.

1. INTRODUCTION

A magnitude M_w 6.3 earthquake struck the city of L'Aquila, in Central Italy, at 3.32 am of April 6 2009. Among the various cases of failure of reinforced concrete buildings in the urban area of L'Aquila, the event caused the partial collapse of the "Casa dello Studente" (CdS, Student House) building, located at about 6 km from the epicenter. Death toll was very high with eight students killed and one suffering serious injury.

The building, designed in 1965 according to the Italian Seismic Code of 1962, is a 7-story (including two underground floors) reinforced concrete (RC) framed structure having an irregular T-shaped plan composed by three nearly rectangular "wings". The structural collapse took place in only one wing (North wing) with a twofold mechanism: a weak-story mechanism involving the collapse of all the columns at the first story, and the complete collapse of three columns located close to the T center. The legal authority appointed the first Author as a technical/scientific consultant; she formed an academic team for investigating the causes and the mechanism of the collapse. During a nine-months period, the team had a unique chance to analyze in a thorough way the collapse causes, as well as the relationship between the seismic response of the building and its intricate history, involving changes in usage and occupancy as well as delicate legal and administrative issues. Studies and experimental tests were carried on, regarding the ground motion experienced at the site; the soil properties; the mechanical properties of the concrete adopted in construction; the structural and non structural damage in the portion of the building that survived the collapse; the structural elements – parts of beams, columns, joists, floors – coming from the collapsed wing and recovered in a separate area in the months following the earthquake. The on-field tests were supplemented with numerical studies, necessary to characterize the dynamic behavior of the building and its earthquake response. This joint effort, though performed primarily for legal purposes, led to a complete explanation of the different phenomena which caused the collapse.

The paper starts with the description of the building, both as initially designed as a residential building and as subsequently refurbished to be adopted as a university dorm. In Section 3 the original structure and the seismic code adopted in the design are presented. Section 4 is focused on the characterization of the in-situ cast concrete. The studies performed to estimate the ground motion acting on the site are described in Section 5. A numerical model of the structure, providing the fundamental frequencies and the linearized response to the seismic excitation is described in Section 6. The final section describes the failure mechanism, in light of both the initial design and the alterations inserted during refurbishment works. Reasons for the collapse in this case study are presented in the Conclusions.

2. DESCRIPTION OF THE BUILDING

The building was designed in 1965 for a mixed-use, with the ground floor and the underground levels designed for offices and laboratories of a pharmaceutical company, and the remaining portion for residential use; it was built on a lot of 1700 m^2 overlooking Via XX Settembre (XX Settembre Street in the following). From the morphological point of view, the building has a non-compact form, called "star" by the designer, and consists of three parts, two of them arranged in the East-West direction and the third one, perpendicular, placed in the North-South direction. The three wings of the building are grouped around a central stairwell, which is the connective element for the vertical and horizontal distribution to three apartments for each of the four floors dedicated to the residential use. With this shape each of the three apartments per floor has three open sides, with the exception of one part close to the gallery of the stairs. From the functional and spatial point of view, at the time of construction the building consisted of four floors and two basements, one of which was built with deviations from the legislation of the time. Each floor had three apartments of 140 m^2 each, consisting of two entrances, a dining room, a kitchen, two bedrooms with one main bathroom, another bedroom and a service bathroom. From the technological point of view the resisting system was a reinforced concrete framed structure; the facade was made of brick subsequently coated, in accordance with the construction technologies used at that time (Dal Co, 1997).

Figure 1 – Plan of the building: (a) original design, 1965; (b) after refurbishment, 2000.

During its lifetime, the building has undergone several transfers of ownership, with changes in building usage, internal distribution and technology, some of which have had a significant impact on its static behavior. In 1979 the building was purchased by the *Opera Universitaria* of the University of L'Aquila, with the aim to allocate it to university residence, but without carrying out modifications to the distribution, with the exception of a partial refurbishment of the ground floor. In 1982 it became property of the Abruzzo Region, by virtue of a regional law. In the years 1999 to 2001 the building was affected by an intense design review that applies to all floors, the vertical connective, the facade and the coverage: the interventions of refurbishment radically changed the functional and spatial asset of the building. The different allocation of the spaces of the standard plan, following the interventions of refurbishment, compared to the original project, is readily appreciated in Figure 1a and 1b. The changing in the uses of the spaces (from 3 apartments per floor to 19 dwellings and 10 bathrooms on each floor) leads to a fragmentation of spaces to be implemented through the introduction of several elements of internal partition and a modification of the technical plant of the building. In addition, the building was adapted to fire regulations that require, in case of "hotel-house for students", the introduction of elements of subdivision for the protection of the escape routes in case of fire and in particular of the stairwell-lift shaft. Therefore, a wall REI 60 was positioned in front of the elevator in the floors one to four, as indicated by the thick red line in Figure 1b. The wall is supported by the beams connecting columns 18-29 (see Figure 9a for the geometrical detail). The symbol REI *n* identifies a building element which must retain, for a given time *n* in minutes, the mechanical strength (R), the resistance to flames and hot gases (E), the thermal insulation (I).

3. THE STRUCTURAL DESIGN

The structural design report of the CdS was issued on May 1965; the reference code adopted by the designer was RD ("Regio Decreto") 11-1937 n. 2105, even though more recent provisions were enforced at the time. The seismic design was essentially based, in RD 2105, on the following principles.

- Application at each floor centroid of an equivalent static force equal to 5% (the adopted value was actually 0.07) of permanent loads plus 1/3 of variable loads; this force must be applied both in transverse and longitudinal direction.
- Application of vertical loads equal to the minimum between 1) 1.25 times the sum of permanent loads plus 1/3 of variable loads, 2) the sum of permanent loads plus variable loads.

The above criteria were used for designing the main lateral load resisting system, i.e. the longitudinal frames located in each wing of the building (red dotted lines in Figure 2a). Loads acting on each frame (Figure 2c) were apparently derived according to tributary areas and to criteria that were typical, at that time, for designing vertical load carrying systems. Given the building plan configuration, the procedure led to substantial underestimation of seismic forces in the NS direction (vertical direction in the plan view 2a), encompassing only three longitudinal frames; note, in this respect, that the adjacent building (on the left in Figure 2b), designed according to the same procedure but characterized by a four-wing plan, i.e by the same number of frames in the two directions, performed satisfactorily during the April 6th 2009 earthquake.

The underestimation of the design seismic forces resulted, in turn, in a significant lack of resistance to lateral forces in the NS direction; focusing on the columns at the ground story, where the collapse sequence was triggered (see Section 7), it can be estimated, by simple limit-state analysis, that overall limit resistance was between half and one third of the value required to resist seismic action, assuming a behaviour factor in the range 1.5-3. In analyzing the structural configuration and performance it can be also observed that the stiffness and resistance of the floor assemblage has proven to be sufficient to perform adequate membrane action but the detailing of the transverse frames located, in each wing, on the facades of the short sides, was not suitable for absorbing in-plane horizontal forces. In addition, the RC frame supporting the staircase mostly contributes to strength and stiffness in the EW direction.

Finally, it must be noted that significant discrepancies were detected between the loads assumed in the structural design and the ones actually present in the "as built" construction; the latter were substantially higher due to a larger thickness of the concrete layer in the floor system and to a higher contribution of the internal partitions. Moreover, the contribution of the external walls was not considered in the evaluation of seismic forces. On the contrary, the actual size and reinforcement of structural members has proven to adequately obey to the original design, even though the overall plan geometry of RC elements shows some discrepancies with respect to the design situation.

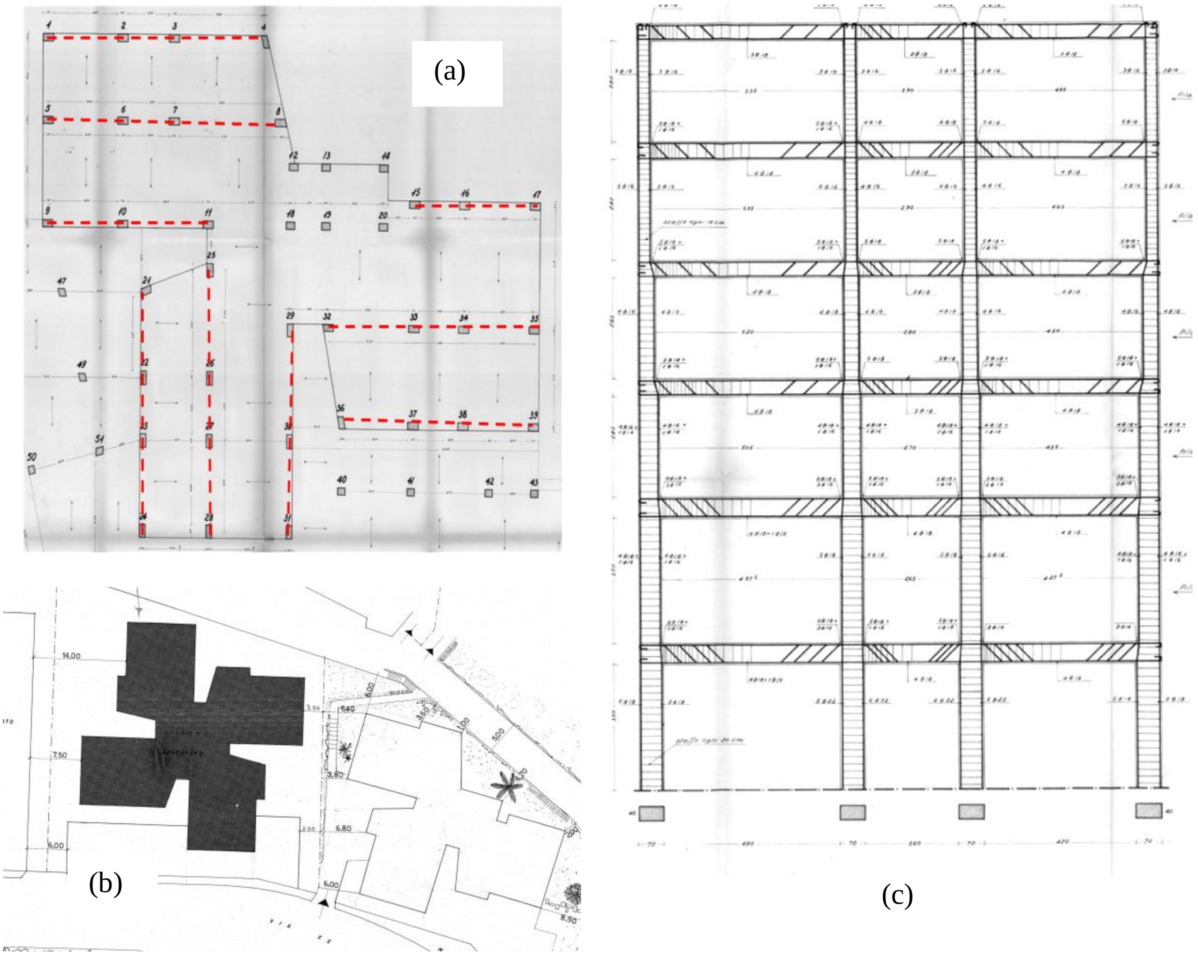

Figure 2 – (a) Structural plan of the building with column numbering and plane frames; (b) comparison with plan configuration of the adjacent building; (c) main structural frame (typical).

4. MECHANICAL PROPERTIES OF THE STRUCTURAL MATERIALS

From the analysis of the design documents and the regulations there quoted, it is possible to estimate that the concrete specified by the designer would correspond to strength class C16/20 (cylinder strength 16MPa, cubic strength 20 MPa), according to the classes defined in UNI-EN 13791 (CEN, 2007). An experimental campaign was carried out including concrete cores, rebound hammer and ultrasonic tests (Sonreb) and tests on bar samples for the reinforcement to establish the strength of the in-situ cast concrete. In a first phase in year 2009, 52 cores were drilled, 42 from columns in the still standing parts of the building and 10 from columns and beams removed from the collapsed part. Non-destructive tests were carried out on 32 columns, 27 of which coincided with columns tested with cores. Compression tests were carried out according to UNI EN 12390-3 (CEN, 2009) and 7 cores failure modes in compression were non satisfactory. The mean cylinder compressive strength $f_{cm,is}$ = 13,3 MPa, standard deviation σ = 3,89 MPa and coefficient of variation COV = $\sigma / f_{cm,is}$ = 0,29. The characteristic “in situ” cylinder strength $f_{ck,is}$ (UNI-EN 13791) is:

$$f_{ck,is} = f_{m,is} - k_2\,\sigma = 13{,}3 - 1{,}48 \times 3{,}89 = 7{,}5 \text{ MPa } (75 \text{ Kg/cm}^2) \tag{4.1}$$

A second sample was considered assuming that the minimum values were relative to cores damaged by cracking, the values of mean and standard deviation in Table 1 were obtained. The strength values are quite low; according to the classes defined in (CEN, 2007) the standard class is C8/10, lower than the class required in design. The concrete density was below 2200 Kg/m^3 for all specimens, with a mean value around 2180 Kg/m^3. The results can be explained in terms of a low quality construction process, concrete mixing casting and curing. This is also indicated by the rather high scatter of the strength (COV=0.29) typical of low-strength concrete, and the irregular distribution of strength values indicated by the histogram (Figure 4.1). The variation of strength with density was coherent with theoretical models (Neville, 1972) and the low density and strength were correctly related. The variation of strength with ultrasonic velocity measurements agrees with results in the literature on low strength concrete; low velocity values agree with the low strength. The strength values obtained with the SONREB method, calibrated on the basis of the core results, are in agreement with the destructive tests. The mean values of strength at each story of the building agree with the sample mean, and no trend can be recognized from the top to the bottom indicating that the cores were not affected by earthquake induced damage. The mean strength and standard deviation of the cores taken from members of the collapsed part of the building agree with the results of the sample.

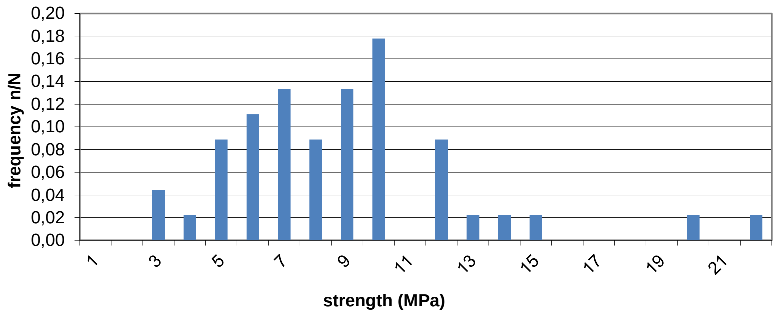

Fig. 3 – Histogram of core cylinder strength

A second testing campaign was carried out in 2011 to check the results of the previous tests. The sample of 8 cores was selected choosing the 5 columns with the lowest values of the previous series of cores, and the 2 columns with the highest values. The former were chosen to check if the cores taken from those columns could have been damaged by cracking or the specimen preparation. The tests were preceded by non-destructive tests. The results of all tests were higher than in the previous campaign with $f_{cm,is}$=16.5MPa and σ = 5.1MPa ($f_{ck,is}$=8.9 MPa). The values of strength and density of the second sample agreed with the previous sample. It was concluded that the cores with the lowest values in the first sample were damaged.

Table 1 – Core compression testing results

	$f_{cm,is}$ (MPa)	St Dev. (MPa)	$f_{ck,is}$ (MPa)	$R_{ck,\,is}$ (MPa)	Class (EN 13791)
Sample 1 (2009) – all values	13,3	3,89	7,5	9,0	C8/10
Sample 1 (2009) – w/out minima	13,1	2,6	9,3	11,2	C8/10
Sample 2 (2011)	16,5	5,1	8,9	10,7	C8/10

Steel samples were taken from the collapsed part of the building, made of smooth reinforcement from beams and columns, with average diameter around 18 mm. The tension tests measured mean yield strength 388 MPa, mean strength 554 MPa with values ranging between 500 e 600 MPa, average elongation at failure 27%. These values are in good agreement with design values.

5. EARTHQUAKE GROUND MOTION AT THE BUILDING SITE

As shown in Figure 4, the CdS is built on the SW side of the alluvial terrace on which the historic centre of L'Aquila is located, which raises about 50 m on the Aterno River valley. As described by Milana et al. (2011), the terrace is formed by alluvial Quaternary breccias consisting of limestone clasts in a marly matrix, superimposed in the Southern part to the lacustrine sediments of the Aterno valley. In the southernmost part of the terrace, the breccias show a high degree of alteration at surface with shear wave velocity V_s that drops down to 300–400 m/s. Therefore, it can be argued that ground motion at the CdS site may have been affected by local site effects, due to the combination of both stratigraphic and topographic amplification.

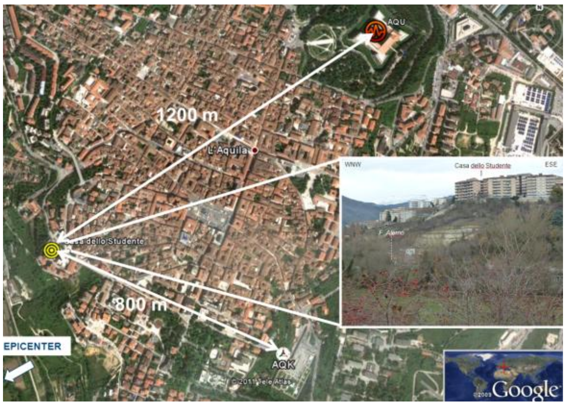

Figure 4. Location of Casa dello Studente with respect to the AQK and AQU recording stations.

To quantify ground motion at the CdS site during the Apr 6 2009 earthquake, the AQK record was considered, available at the ITACA strong motion database (http://itaca.mi.ingv.it). This was preferred to the AQU record, obtained at the L'Aquila Castle site, because AQK lies at similar epicentral distance as CdS, with similar geological conditions. Furthermore, processing of aftershocks data by Milana et al. (2011) clarified the reliability of AQK records up to at least 10 Hz.

In the post-earthquake phase, different, partially conflicting, Vs profiles were obtained at the CdS site by downhole (DH) investigations carried out by different teams in the same borehole. Eventually, reference was made to the V_s profile (DH) obtained by SOLGEO (Solgeo, 2011) specifically for this investigation, for two reasons: (i) it was in agreement with surface wave measurements and (ii) it provided substantial agreement in terms of CdS/AQK amplification with aftershock data provided by Milana et al. (2011).

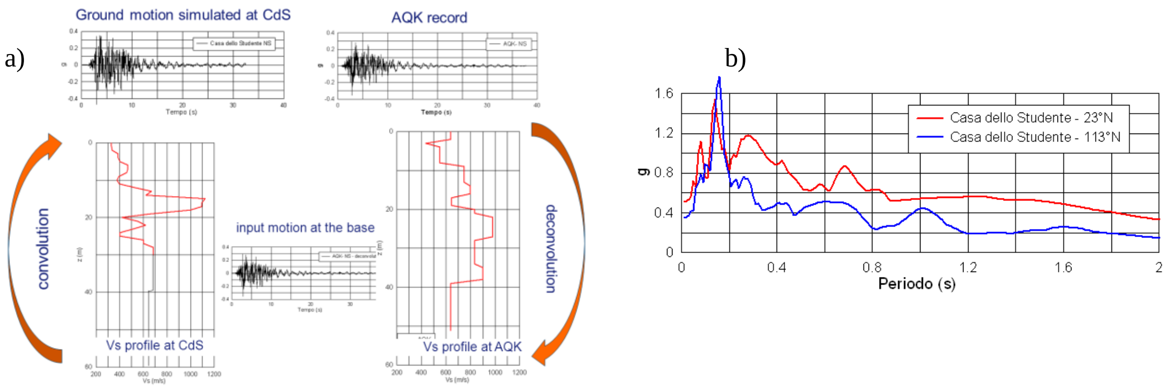

Figure 5. (a) Sketch of the procedure to obtain ground motion at the CdS site based on the AQK record; (b) simulated elastic response spectra, rotated along the principal directions of the building.

With these preliminary remarks, ground motion at the CdS site was predicted by the following approach, illustrated in Figure 5a:

a) the AQK record was deconvolved considering the 1D stratigraphy available at the ITACA web site;
b) the base motion obtained at step (a) was convolved by the local 1D stratigraphy at the CdS site;
c) a topography amplification factor S_T=1.1 was calculated based on 2D numerical simulations of SH wave propagation and applied as a scaling factor to the ground motion obtained at step (b);
d) the NS and EW components were rotated clockwise by 23° to be directed along the principal directions of the CdS building.

The resulting elastic response spectra for a 5% damping ratio, in Figure 5b, show the prevailing amplitude of the 23° N component, as a consequence of the directionality of earthquake ground motion in the L'Aquila records along the fault normal direction (e.g. Paolucci and Smerzini, 2010).

6. NUMERICAL MODELS AND ANALYSIS OF THE BUILDING

A structural model has been developed to achieve an accurate representation of the geometry and the masses of structural and non-structural elements, and of the stiffness of the structural ones. The geometry of structural elements (presence, size, position) derives form the results of on site surveys by and from the blueprints of the several renovations of the building that it has been possible to recover after the damage suffered by state and county archives due to the April 6, 2009 earthquake.

Given the highly irregular shape in plan, the floors were modelled as elastic membranes having the thickness of the concrete slab collaborating with the joists in the floor system (5 cm from the design data of 1965). The mechanical properties are that of a low strength concrete C16/20, similar to the design value. To reduce the size of the numerical model, the rigid floor assumption was adopted for the basement floor. The building outer walls consist of two nonbearing layers of brickwork (hollow tile clay-fired) of different thickness, with an air gap in between. The outer layer is interrupted by the openings (doors and windows); only the outer layer is present above and below windows. This allows not to consider the mechanical effect of the outer cladding in the response. For this reason, only the masses and weight of the outer walls are modelled, approximated with a uniform load distribution per unit length on the perimeter beams. Similarly, internal partitions are modelled only in terms of weight (and mass), and are approximated with a uniformly distributed load per unit area. This assumption is justified by the presence of openings in the partitions and since these mostly do not have columns and beams on all the sides. In the structural configuration resisting the earthquake of April 6, 2009, the beams 18-29 at floors from first to fourth were supporting a REI 60 wall on a part of their span. This effect is included in the model representing this wall by plate finite elements having appropriate modulus of elasticity. The stairs are modelled with an equivalent concrete beam for the flight, supported by orthogonal beams at the landings. Finally, the constraint conditions for all the vertical elements are of full restraint at the extrados of their foundations. A view of the geometric model of the structure, from which the finite element models are extracted, is shown in Figure 6a; the detailing of a typical ground floor column is represented in Figure 6b.

A stone retaining wall is present in the basement facing XX Settembre street, which partially clamps and incorporates the perimeter columns and supports the ground floor slab. On-site surveys have shown that these columns show no detachment from the stone wall, suggesting that this could have operated as a constraint against displacements in the North-South direction for the ground floor. A concrete wall is present also towards the East side of the basement, incorporating some perimeter columns. This wall shows signs of slight damage at the basement ceiling, indicating that the wall itself acted as a restraint to the movements of the ground floor slab.

Based on the above considerations, to delimit a plausible range for the natural periods of the structure, several numerical models were derived from the geometrical one, differing in the constraint conditions towards XX Settembre street and in the average flexural stiffness of beams and columns: a) effective action of the wall on the basement towards XX Settembre street and of rollers constraining the North-South displacements of the ground floor slab at ground level (level of XX Settembre street); b) only the wall is considered; c) ineffectiveness of the wall and absence of the rollers. This structural configuration is the most flexible. Three stiffness reductions of beams and columns, to take into account the damage of the structural elements during an earthquake, have been considered: 1) no stiffness reduction; 2) reduction of 50%; 3) reduction of 70%, to take into account the increased damage due to previous seismic events, prior to that under examination. The stiffness reduction is implemented by reducing the second area moment of the cross-sections and it is sufficiently broad to cover any variation for the modulus of elasticity of concrete. This choice reflects a) the main interest of the numerical model, that is an estimate of the possible range of natural periods of the building before collapse; b) unavailability of detailed information on the damage already suffered by the building and on material properties of the collapsed wing; c) code practice. The presence or absence of the REI 60 wall on part of beam 18-29 was also considered as well.

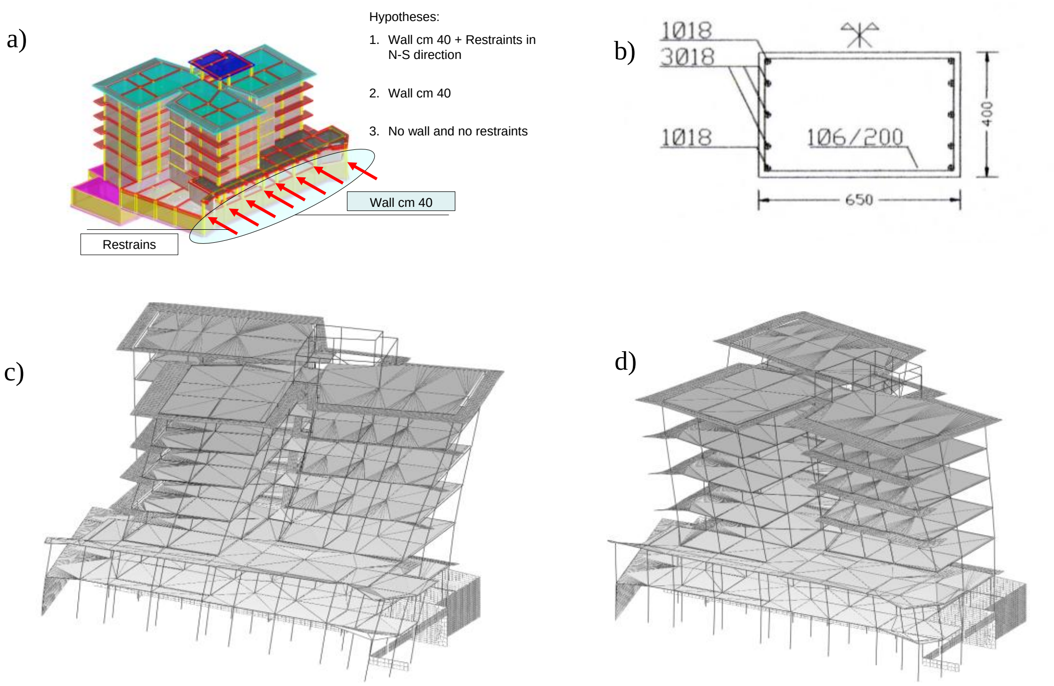

Figure 6. a) Geometric model of the building; b) typical column detailing at ground floor; c), d) modal shapes for 1st and 2nd mode, respectively.

Based on the models developed with the above given criteria, both static and dynamic linear analyses were carried out. The former, with static horizontal forces equivalent to the earthquake, to the aim of assessing the compliance of the structure as originally built to the seismic standards in force at that time. The latter, for the dual purpose of: (a) defining the range of variation for the fundamental period of the building in the North-South direction depending on boundary conditions and members stiffness; (b) determining the structural effects of the renovations the building underwent to, by comparing the estimate of the floor seismic horizontal forces obtained through a modal analysis adopting the response spectrum estimated at the CdS site (Figure 5b).

Figure 6 c),d) shows the first two mode shapes of the structure in the configuration at the date of the seismic event, in the most flexible configuration, the one without the constraints at the level of XX Settembre street, without the basement wall towards XX Settembre street and including a 70% reduction in stiffness of beams and columns. Boundary conditions at the basement, as well as reduction of members stiffness, induce little modification on the first mode shapes. The period of the first and the second mode falls in the ranges 0.83-1.63s and 0.81-1.45s, respectively. For the most rigid structure the values are only slightly different while for the most flexible they differ to a larger extent but anyway fall in regions of the response spectrum that have almost equal spectral ordinates. Involving both a rotational component with respect to a vertical axis, the first two modal shape can be combined in the response giving rise mainly to a translation in the North-South direction, the same of the North wing main frames.

7. THE COLLAPSE MECHANISM

At today's date, the South and West wings are still standing and can be accessed without major problems, due to the absence of relevant damage, both structural and non structural. The North wing, that suffered a partial collapse (Figure 7a,b), was completely demolished by April 9, to allow for fire-fighters to rescue, in safety conditions, the bodies trapped in the collapsed region (Figure 7c). Immediately after the earthquake, the situation of the building – as witnessed by many pictures - was the following:

1. The ground story – at the level of the road – had a partial soft-story mechanism concerning all

the columns of the North Wing frames (Fig. 7a).

2. The soft-story mechanism was associated to a displacement in the direction of the frames of the collapsed wing.
3. The three columns of the North Wing at the interface with the other Wings (# 21, 25 and 29 in Figure 8a) collapsed totally from the ground story to the building top (Figure 7b).
4. No significant damage was found in the structures of the two underground floors.

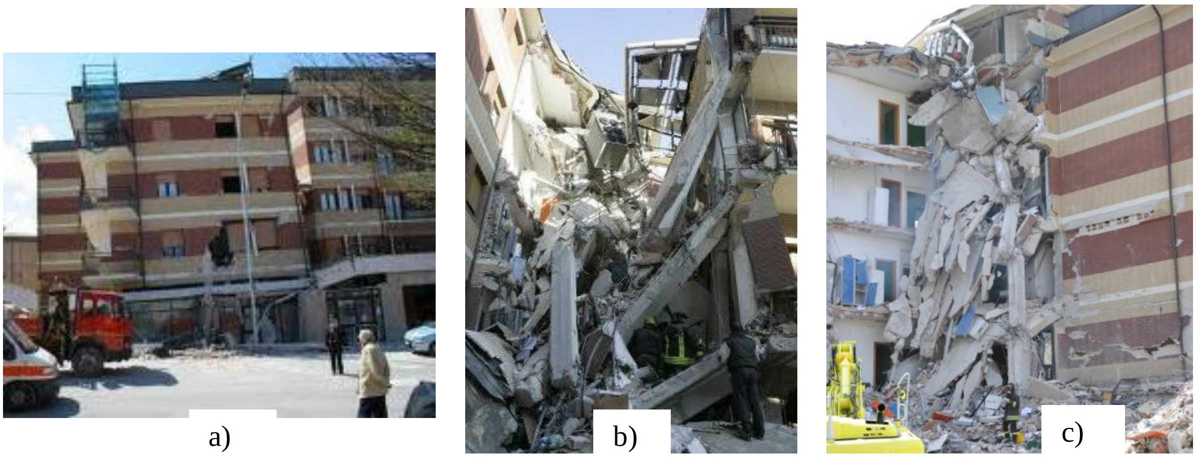

Figure 7 – North Wing Collapse : a) North view; b) connection zone; c) demolition before removal of col. 29.

From the description above, it can be stated that actually two separate collapse mechanisms took place in the building. A first one, that can be defined as “the collapse initiating event” was the failure of all the columns at the ground story in the North Wing, due to an excessive ductility demand on the plastic hinge at top and bottom of each column. With the soft-story mechanism restricted to the North Wing, the connection zone between this and the other wings is subjected to a dramatic vertical relative displacement: the chord of the beams at the border of North Wing is shown with red solid lines in Figure 9b. The original design of the connection zone is affected by unfavorable characteristics, as the misalignment of columns and a reentering corner (Figure 8a). In addition the beam 18-29 supports the nonstructural wall REI 60 that increases stiffness and strength of a limited part of the beam itself. The beam failed in shear at the cross-section corresponding to the end of the wall between the green and the grey zone (Figure 8a); the region between the wall and the elevator (see Figure 1b) is still standing in the building. The floors at levels 1-4 in the yellow zone in Figure 8a underwent a strong distortion, that in turn caused tension forces on the floor itself and on columns 25 and 21. These forces, acting out of the frames plane, induced instability of columns and the final catastrophic failure along the building height, a second event caused by the joint presence of the soft story mechanism, the lack of regularity in plan and the collapse in shear of the beam 18-29. The presence of strong tension forces is witnessed by: (a) the cracking pattern of a few structural elements recovered after the partial failure of the North Wing and its demolition; (b) tension cracks, perpendicular to the red arrows in the yellow zone in Figure 8a, still visible in the ceilings.

8. CONCLUSIONS

In this paper a comprehensive approach has been presented to analyze the partial collapse of a RC building during the earthquake of April 6th, 2009 in L’Aquila (Italy). Even though the research work was performed for legal reasons, the resulting multidisciplinary effort covered all the aspects that could be relevant on both the explanation of the collapse mechanism and the detection of the causes leading to the collapse itself. It can be concluded that the building collapse is directly related to the lack of stiffness and strength in the N-S direction of the two wings in the East-West direction, due to the original design in 1965. The frames in the North-South direction of the North Wing, due to their own stiffness, attracted a large part of the seismic forces of the two other wings, without possessing a sufficient strength and ductility to withstand these forces. It can be observed that the modal shape arising from the combination of the first two modes resembles the collapse mechanism eventually activated. Even though the quality of the in-situ cast concrete was found to be significantly lower than the design value, it must be noted that the lack of strength of the overall structure was such that a concrete having the design properties would have not avoided the collapse. The analysis of the ground motion at the site and the evaluation of a reasonable range for the natural period of the structure has shown that the level of ground excitation experienced by the CdS during the seismic event of April 6th was consistent to the original design action. This observation is supported by the evidence of the buildings close to the CdS and in the same topographic situation. They suffered an amount of damage compatible with the design level of an earthquake excitation acting on structures designed through obsolete codes not guaranteeing the proper level of ductility, but did not collapse. The collapse explanation is coherent with the findings of modern conceptual design, suggesting to avoid non-simple structures, re-entrant corners and uni-directional resisting systems.

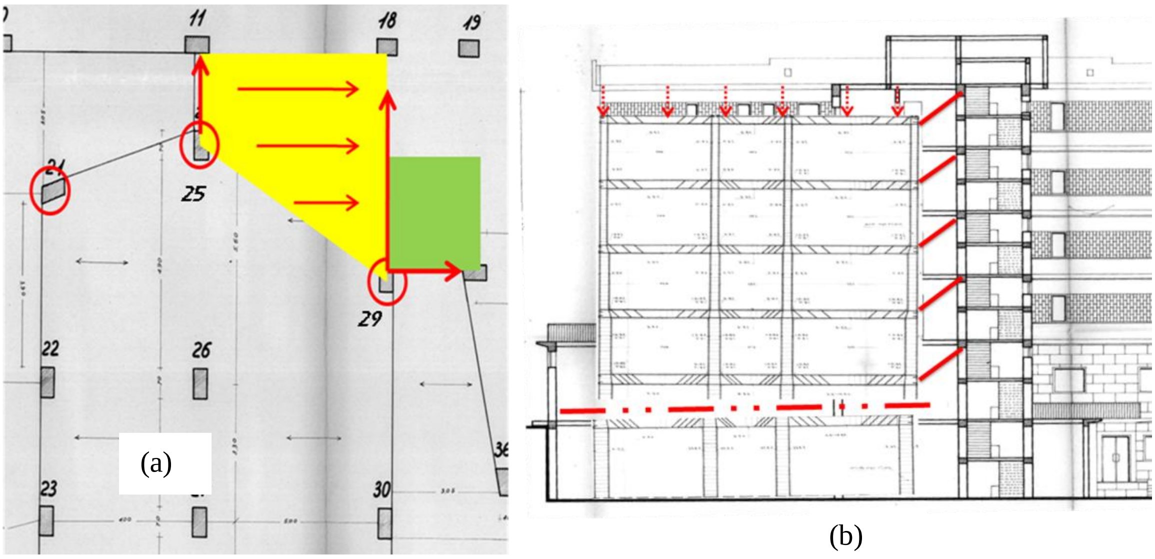

Figure 8 – Collapse mechanism: (a) plan of the collapsed region at a typical floor; (b) view in N-S direction.

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REFERENCES

Dal Co F. (Editor). (1997). Storia dell'architettura italiana. Il secondo Novecento. Electa, Milano.

Milana G., Azzara R. M., Bertrand E., Bordoni P., Cara F., Cogliano R., Cultrera G., Di Giulio G., Duval A.M., Fodarella A., Marcucci S., Pucillo S., Régnier J., Riccio G. (2011). The contribution of seismic data in microzonation studies for downtown L'Aquila. *Bulletin of Earthquake Engineering*. **9**, 741-759.

Neville, A., M., (1972). Properties of Concrete. Pitman Publishing, 686 pp.

Paolucci R. and Smerzini C. (2010). Strong ground motion in the epicentral region of the MW 6.3, Apr 6 2009, L'Aquila earthquake, Italy. *Proc. 5th Int. Conf. on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics*, Invited presentation, Paper EQ4, San Diego, CA, 2010.

SolGeo. Indagine Down-Hole e M.A.S.W. per la caratterizzazione sismica nell'intorno di un sondaggio. (*In Italian*). Relation n. S11SN09. June 2011.

CEN (2007). Assessment of in-situ compressive strength in structures and precast concrete components. EN 13791.

CEN (2009). Testing hardened concrete - Part 3: Compressive strength of test specimens, EN 12390-3.