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A CASE STUDY OF COLLAPSE DURING THE 2009 L'AQUILA (ITALY) EARTHQUAKE

Maria G. Mulas[1], **Luca Martinelli**[2], **and Federico Perotti**[2]

[1] 1Department of Civil and Environmental Engineering, Politecnico di Milano
P.zza L. da Vinci, 32 – 20133 Milano, Italy
e-mail: mariagabriella.mulas@polimi.it

[2] 1Department of Civil and Environmental Engineering, Politecnico di Milano
P.zza L. da Vinci, 32 – 20133 Milano, Italy
luca.martinelli@polimi.it, federico.perotti@polimi.it

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Abstract. *A number of masonry and reinforced concrete (RC) buildings suffered failure during the magnitude Mw 6.3 earthquake that struck the city of L'Aquila (Italy) on April 6 2009. Many of the RC buildings were designed and built between the late fifties and the early seventies. The partial collapse of the "Casa dello Studente" (Student House) building in L'Aquila, with its high death toll of eight students killed and one suffering serious injury, is paragdimatic on one hand of the possible fragility of reinforced concrete construction of that period, on the other hand of failure due to an adverse sequence of events.*

The legal authority appointed a forensic team, lead by the first author, for investigating the causes and the mechanism of the collapse. During a six-months period, the team had a unique chance to analyze in a thorough way the collapse causes. The joint efforts of the forensic team led to a complete explanation of the collapse and highlighted the sequence of missed opportunities that let this happen. Collapse took place within part of the North wing trough a twofold mechanism: a weak-story mechanism, which involved collapse of all the columns at the first story, and the complete collapse of three columns located close to the floor center. The two mechanisms were due to an initial design error, never discovered during the subsequent renovations of the building, whose effects were adversely amplified by a non-structural fire-proof wall inserted in recent times to comply with fire safety requirements. This imposed unforeseen strength and ductility requirements on members not specifically designed for this.

This paper presents some of the result of the studies that, not strictly necessary for legal reasons, were only performed after completing the report for the legal authority. Nonlinear analyses are adopted to highlight the role of each of the factors listed above which lead to the structural failure.

1 INTRODUCTION

On April 6th 2009 at 3.32 am a magnitude Mw 6.3 earthquake struck Central Italy, with devastating effects in the city of L'Aquila and the surrounding area. Several reinforced concrete (RC) buildings in the urban area failed. Among them, the "Casa dello Studente" building (CdS, in the following), located at about 6 km from the epicenter, suffered a partial collapse. The death toll was very high, with eight students killed.

The CdS building was a seven-story RC building hosting a University dorm ("Casa dello Studente" means Student House, in Italian). It was designed in 1965, following the Italian Seismic Codes of 1937 [1] and 1962 [2], for a mixed use (residential/commercial). The plan of the residential part, in the upper four stories, was subdivided in three wings, connected through the stairwell, with an apartment in each wing. Ground floor and basement were common to all the wings; a second underground level was only present below the South wing. The ground floor and the underground levels were originally designed for offices and laboratories of a pharmaceutical company. In 1979 the building was bought by a University Institution to be used as student house. A minor refurbishment work took place at the ground floor and at the two underground floors, where a library, offices and a cafeteria for students were located. Subsequently, in 1982, the building became a property of the Abruzzo Region, in force of State and Region Laws. The second change of property did not affect the usage of the building as a student house. A wide refurbishment intervention, not covering the structural system, was performed during the years 1999-2002. The plans of the upper floors were deeply transformed to make the building more apt to its actual usage (Figure 2).

Figure 1. Aerial view of the building in the configuration at the date of the 2009 seismic event.

The partial collapse of the building was limited to the North wing (see Figure 1). Within this wing two distinct collapse zones were detected, from both pictures of the collapsed part (Figure 3) and on-site observations: the failure of all the columns at the ground floor, with a soft/weak story mechanism, and that of three columns (21, 25 and 29, see Figure 4) along the height of the building, at the interface between the North wing and the remaining ones. Previous studies, reported in [3, 4], performed for legal reasons, showed that neither the poor quality of material properties nor the ground motion at the building site could be considered – from the Italian laws viewpoint – among the causes of the collapse. The most likely collapse sequence was established by crossing the design criteria adopted in 1965, the results of modal analyses of the building in its original configuration and after refurbishment, and all the experimental data coming from the analysis of damage in the part of building that survived the earthquake. The lack of flexural strength of ground story columns was detected as the main mechanical cause of the collapse itself.

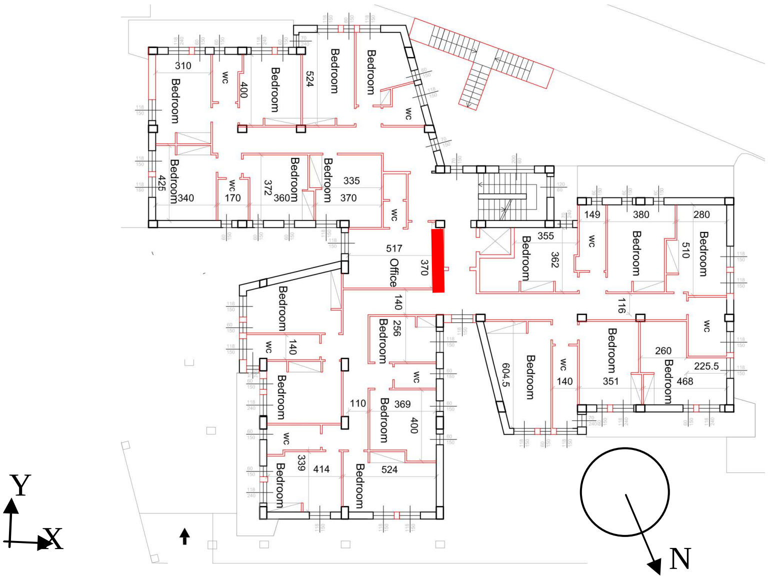


Figure 2. Plan of the typical building floor after refurbishment in 2000. The thick line marks the position of a fire resistant wall on Beam 18-29 which triggered the collapse of this beam at several levels.

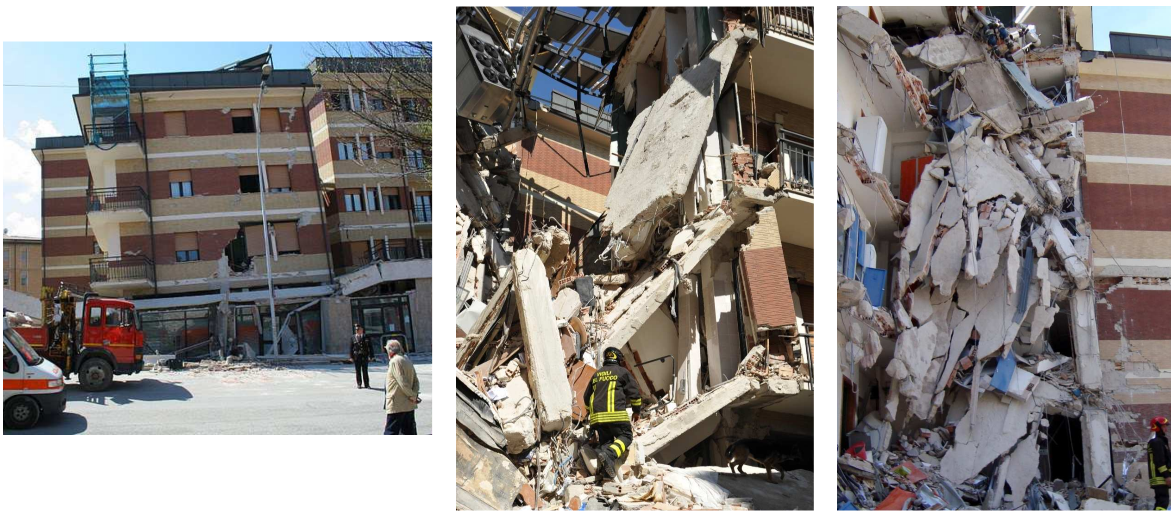


Figure 3. The partial collapse of the building. Left, the soft story of North wing, view from XX September Street. Center, the collapse of the columns at the interface between the North wing and the remaining wings. Right, the remaining of column 29 after demolition of the collapsed region

Starting from the previous results, the collapse sequence has been investigated further in this study. The availability of data on the geometry of the reinforcement has allowed the derivation of a complete and refined numerical model. Non linear static analyses are performed to compute the capacity curve of the structure, with the aim to further investigate on the disproportioned effect that arose by the partial soft-story collapse at ground floor. The results obtained help to clarify the role played by the lack of shear strength of a beam at floors 1-4 on the collapse of the three columns along the height of the building.

2 THE BUILDING: STRUCTURAL DESIGN AND RENOVATION WORKS

The CdS building is a 7-story (including two under-ground floors) RC framed structure (see [4]) having an irregular T-shaped plan composed by three nearly rectangular "wings", two roughly oriented in the East-West direction and one in the North-South direction (Figures 1, 2, 4). The code adopted in the structural design was the Italian Seismic Code [1] of 1937, even though newer provisions were enforced by the Seismic Code [2] of 1962.

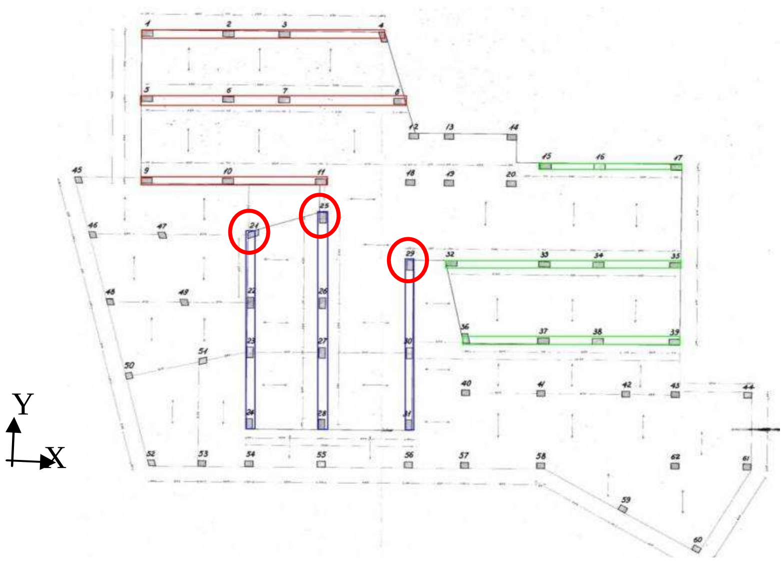


Figure 4. Layout of the vertical members. The colored lines highlight the positions of the main frames within the three wings; circles show the collapsed columns # 21, 25 and 29.

The design principles of both Codes were based on the application, at the centre of mass of each floor, of a set of equivalent static forces, equal to 5% or 7% (according to the 1937 and 1962 Code, respectively) of permanent loads plus 1/3 of variable loads. The 7% value was actually adopted in design. These static forces should have been applied both in transverse and longitudinal directions according to the 1937 Code, and in any horizontal direction according to the 1962 Code. Vertical loads had to be computed as the maximum between:

a) 1.25 times the sum of permanent loads and 1/3 of variable loads,

b) the sum of permanent loads and variable loads.

These criteria were used for designing the main lateral load resisting system, i.e. the three longitudinal moment-resisting RC frames located in each wing (Figure 4). There was no provision for transverse frame within each wing, with the only exception of elements along the short sides of the almost rectangular wings. The evaluation of horizontal loads acting on each frame was apparently performed according to tributary areas. Given the building plan configuration, the procedure led to substantial underestimation of seismic forces in the North-South (NS) direction encompassing only three longitudinal frames. In addition, the contribution of perimeter walls was not included; and during construction a 5 cm thick non-structural slab was placed at all floors, that was not accounted for in the structural design. These two facts further increased the underestimation of horizontal forces.

In analyzing the structural configuration and performance it can be also observed that the stiffness and resistance of the floor assemblage have 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.

A wide refurbishment intervention, not covering the structural system, was performed during the years 1999-2002. The layout of the internal partitions was changed to better adapt the building to its use as a university dormitory (Figure 2). To comply with fire safety regulations, a fire resistant wall (denoted, from the French acronym, as “REI wall” in the following) composed of lightweight concrete hollow blocks, 20 cm thick, was placed at floors from the first to the fourth (thick red line in Figure 2), supported by but not connected to the underlying beam This wall spans from column 18 on roughly half of the beam 18-29, that in turn is part of the beam that connects columns 18, 29, 30, and the corner column 31 (beam 18-29-30-31).

The role that this non-structural element, inserted in a non-engineered way, had on the collapse will be investigated in the following.

3 EARTHQUAKE GROUND MOTION AT THE BUILDING SITE

The ground motion at the CdS site was estimated according to the procedure described in [4], based on considerations on both the actual ground motion recorded at the nearby station named AQK and the soil properties at these two sites. The resulting elastic response spectra along the principal directions of the CdS building (namely: N23°E and E23°S; directions Y and X, respectively in Fig. 3) are plotted for a 5% damping ratio in Figure 5. The prevailing amplitude of the N23°E component at the longer periods is a consequence of the directionality of earthquake ground motion in L’Aquila records along the fault normal direction, as pointed out by different studies (e.g., [5]). The estimated motion has been deemed compatible with the magnitude of the static horizontal forces [4], equal to 7% of gravity loads, prescribed by the 1962 Code [2] in a design process based on a working stress approach.

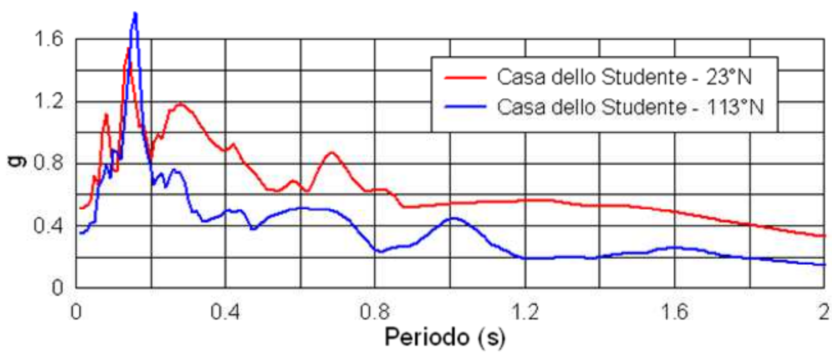


Figure 5. Estimated response spectra along the principal directions of the building, 5% damping [3].

A ratio close to 3 to 1 can be assumed for the spectral ordinates in the likely ranges of period (0.83-1.63s and 0.81-1.45s, respectively), estimated in [4] for the first and the second mode. This is confirmed by the ratio of the extreme values of ground acceleration: the acceleration in direction E23°S was 35% of that in direction N23°E. This fact will be accounted for in the direction the capacity curves are computed.

The wide period ranges depend on the boundary conditions assumed for the model, on the amount of reduction of member stiffness and on the presence of the added fire resistant wall. The boundary conditions addressed the action of the basement wall towards XX Settembre street.

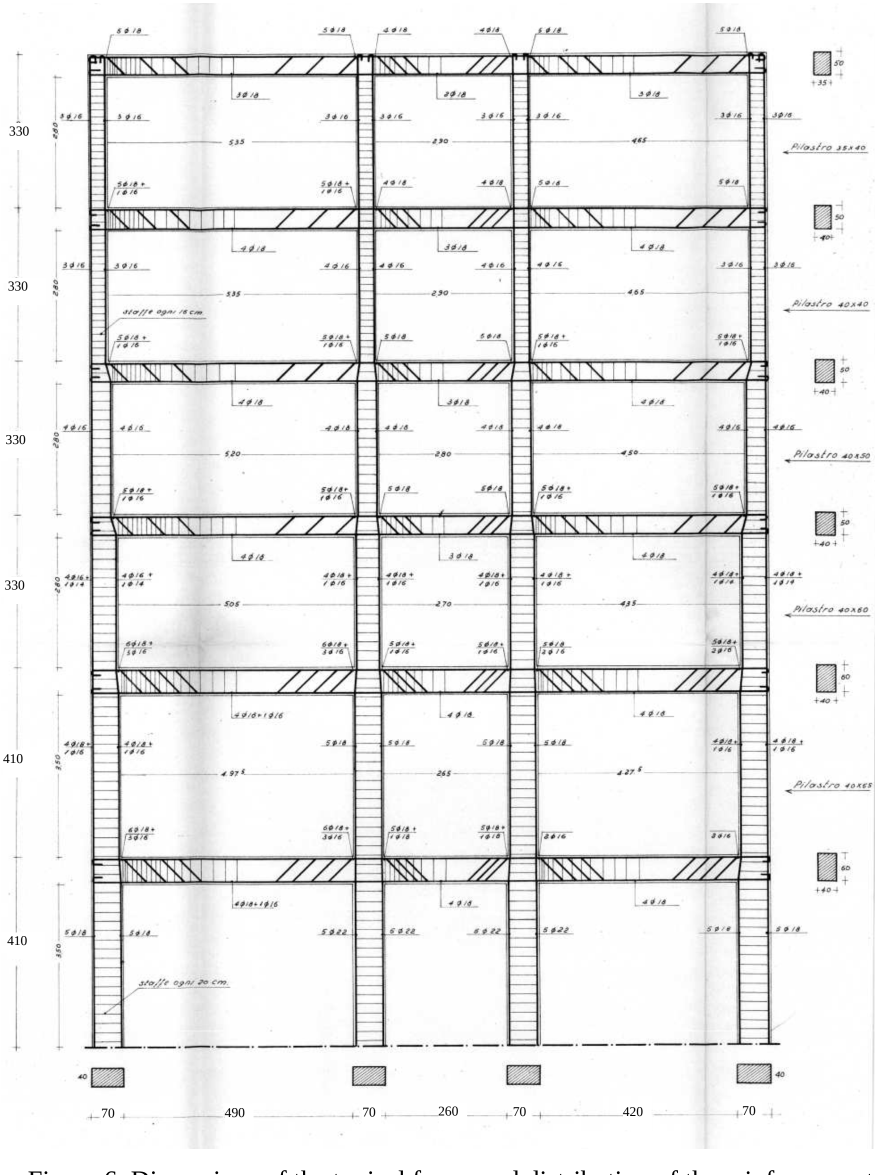


Figure 6. Dimensions of the typical frame and distribution of the reinforcement.

4 NUMERICAL MODEL FOR NONLINEAR ANALYSES

In previous studies, as reported in [3, 4], a comprehensive model was developed to achieve an accurate representation of the geometry and masses of structural and non-structural elements, and of the stiffness of the structural ones. A modification of this model was derived for the static non-linear analyses.

The geometry of the structural elements (beam, columns and floor slabs) derives from both the results of on site surveys and the blueprints recovered by State and County archives damaged by the April 6th, 2009 earthquake. Blueprints describes both the original design and the renovations of the building. As an example, Figure 6 depicts the reinforcement distribution and details of the typical central frame of each building “wing” (colored lines in Fig. 4).

The model derived for the static non-linear analyses reduces the number of degrees of freedom by eliminating the description of the balconies from the model. In the new model, then, the stiffness of the balconies is disregarded but their weight and mass have been applied to the perimeter beams. Moreover, the floor system is considered to be sufficiently rigid in its plane to act as a diaphragm. A view of the solid model is depicted in Figure 6. The new model has been validated by comparison with the previous one in terms of modal periods and correlation of the modal shapes. As in the linear models, the REI wall on beam 18-29 is modeled through 2D quadrilateral and triangular plane-stress (membrane) elements.

The boundary conditions of the model come from in situ observations. All the vertical elements are considered to be fully restrained at their base, since no evidence of movements was detected at the level of the foundations. The perimeter columns on the North and East side are connected to a basement wall.

A static non linear analysis, involving non linear material behavior, requires that beams and columns undergoing non linear response be properly discretized to avoid strain localization effects. In this work beams and columns were discretized in elements of length 20cm and 40 cm, near the members ends and elsewhere respectively. The computer code SISMICAD v.12 [6] is used in the static non linear analyses. This codes discretizes the cross-sections in rectangular sub-domains, here having the longest side less than 7 cm (see Figure 8a). The code assigns 9 Gauss points per sub-domain to follow locally the stress-strain law of the concrete material. A single Gauss point is assumed at the location of a reinforcing bar, as well as perfect bond between concrete and reinforcement. Concrete follows different laws depending on the confinement (confined or unconfined concrete). The elements behave nonlinearly for axial force and bending, being elastic in shear. To avoid spuriously enhancing the bending strength due to moment-axial force interaction, the axial force has been released at one end for all members having both nodes belonging to the same diaphragm or “rigid floor”.

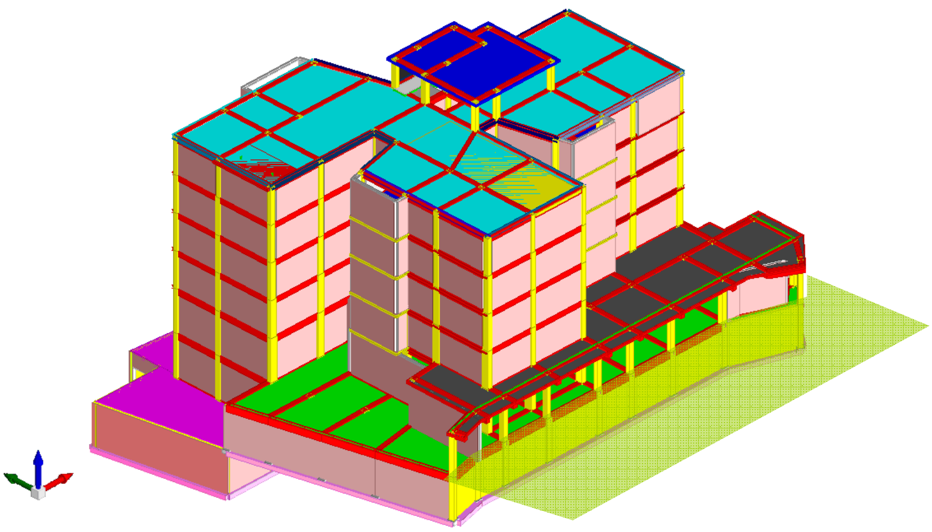


Figure 7. Geometrical model (cantilevered balconies have been disregarded in the modeling).

4.1 Concrete and steel reinforcement models

Concrete has no strength in tension and follows (Fig. 8b) a parabola-rectangle law in compression up to a strain value ε_{c0}. Compression failure is attained at a strain value ε_{cu}. Unconfined concrete has the design nominal values of strength and strain f_{ck} = 16MPa, ε_{c0} = 0.2%, ε_{cu}=0.35%. Confinement has a small effect, due to the low amount of stirrups in columns and in beams, and to a larger extent to the lack of longitudinal reinforcement along the long sides of the rectangular cross-sections. Confinement effectiveness, estimated on a sample column cross-section, accounts for the following values of strength and deformation: $f_{ck.c} = 1.033 f_{ck}$, $\varepsilon_{c2,c} = 1.067\varepsilon_{c2}$ and $\varepsilon_{cu2,c} = \varepsilon_{cu2} + 0.13\%$.

Steel reinforcement follows an elastic-perfectly plastic law (Fig. 8c) symmetric in tension and compression, and has nominal strength and Young's modulus values f_{yk} = 380 MPa, E_s = 2.6 GPa.

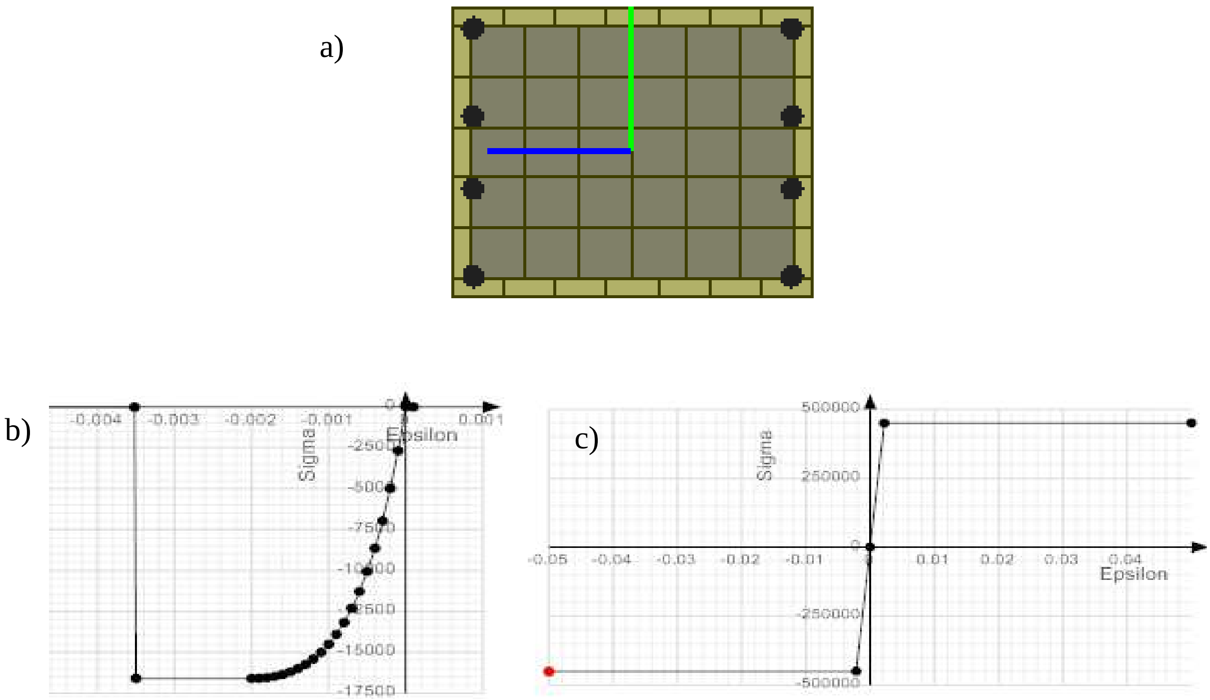


Figure 8. (a) Sample of cross-section discretization for the static non-linear analyses; a lighter color denotes unconfined concrete. Constitutive laws adopted for (b) concrete and (c) steel reinforcement.

The numerical analyses were carried out with both the nominal design values of the mechanical parameters and the average values coming from on-site and laboratory material testing: concrete average compression strength f_{cm} =13.3 MPa, steel average yielding strength f_{ym} =388 MPa. While the yielding strength of the steel reinforcement corresponds to the prescribed value, the one of the concrete is far lower than its prescribed value and corresponds to a concrete of class C8/10, instead of C16/20, according to the classification in the Italian code D.M. 14.01.2008 [8].

4.2 Layout of reinforcement

The reinforcement geometry and layout were derived for the members of the primary frames (colored lines in Figure 4) from design drawings recovered form State archives (e.g. the drawing in Figure 6). The reinforcement distribution was checked during on-site surveys which allowed also to gather further details. Finally, were appropriate, reinforcement details in manuals in widespread use at the time of the design of the structure have been assumed.

As an example of the level of detail with which the reinforcement has been described inside the numerical code, Figure 9 depicts the reinforcement considered for the critical beam 18-31 in the span that supports the REI wall. This span failed in shear at the location of the REI wall.

Bending reinforcement at mid-span is 4ϕ18 mm + 1ϕ16 mm in the lower part of the cross-section, 3ϕ18 + 1ϕ6 in the upper part. In these indications, the symbol ϕ denotes the bar diameter, the first number denotes the number of equal diameter bars, the second number the diameter of the bar in mm. At the supports the upper bending reinforcement is 6ϕ18 + 3ϕ16 while the lower one is 2ϕ18. Shear reinforcement is provided by ϕ6 stirrups at a variable spacing: 240 mm in the central part of the beam and in between 50 to 75 mm at the supports.

Table 1 lists the geometry, at ground floor, of the cross-sections of the columns of the North Wing. The reinforcement is located along each short side of the rectangular cross-section of the columns (see Figure 8a). Shear reinforcement for the columns is provided by 6 mm diameter closed stirrups. On average, the stirrups spacing is 200 mm for most of the columns, without provisions for closer spacing at the ends.

Column	Cross-section dimensions (cm)	Number of bars along short side	Bar Diameter (mm)
21	65x40	5	18
22	40x65	4	18
23	40x65	4	18
24	40x65	4	18
25	40x65	5	18
26	40x65	5	18
27	40x65	5	18
28	40x65	5	18
29	40x65	5	18
30	40x65	4	18
31	40x65	4	18
1	65x40	4	18
9	65x40	4	18
17	65x40	4	18
39	65x40	4	18

Table 1: Cross-section dimensions and reinforcement at ground floor in columns belonging to the North Wing.

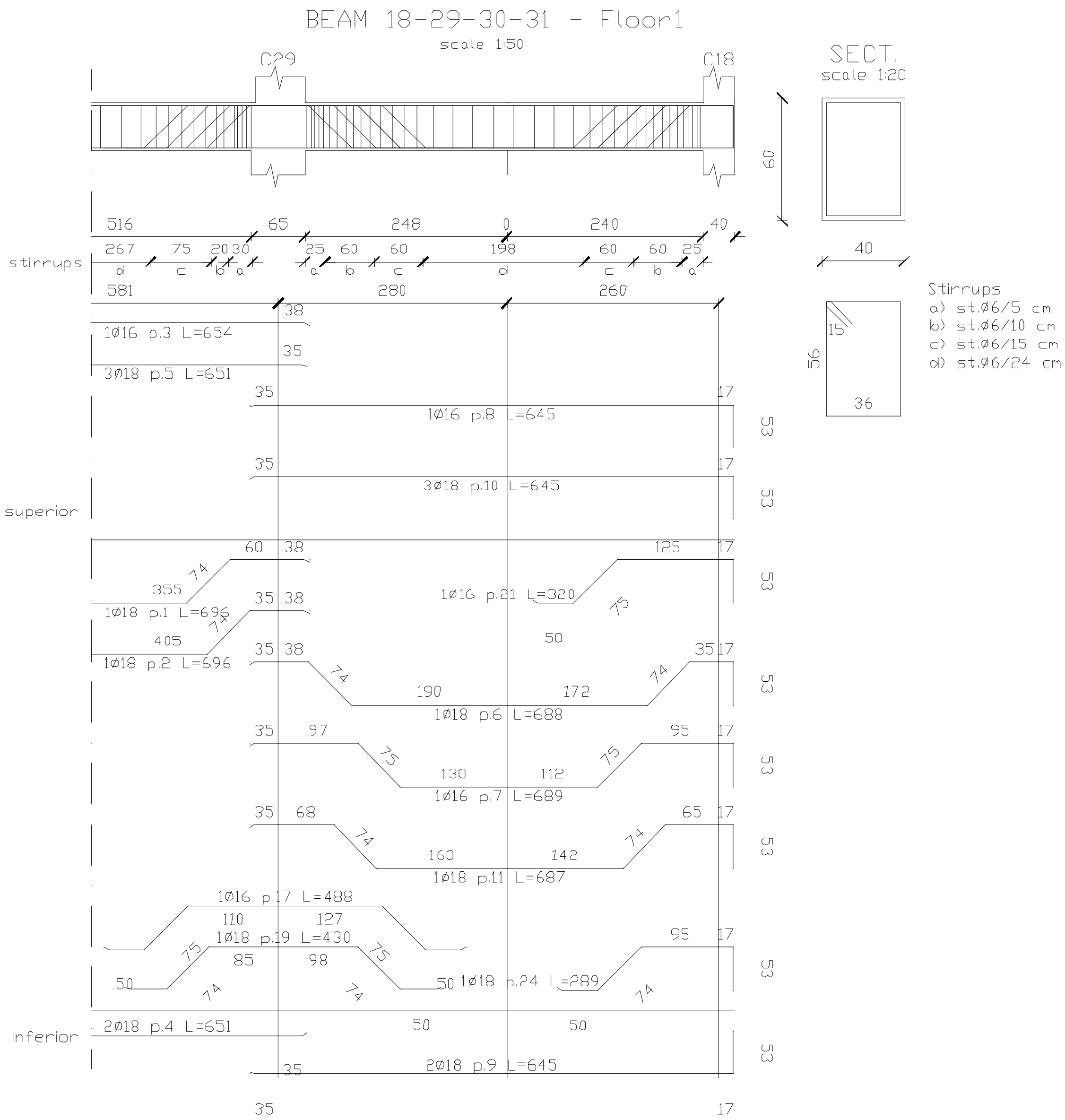


Figure 9. Detail of the reinforcement in beam 18-31 at first floor.

5 NONLINEAR ANALYSES

The non linear static analysis procedure in the present Italian Code NTC 2008 [7] has been adopted to compute the capacity curves for the CdS building. Aim of the analyses it to gain a better insight on the disproportioned effect that arose by the partial soft-story collapse at ground floor. In [7] the nonlinear static analysis, or pushover analysis, has to be performed applying to the structure first the gravity loads and then, for the considered direction of the seismic action, a fixed pattern of horizontal forces proportional to the inertia forces and having as resultant (base shear) the force F_b. The horizontal forces, acting at the floors, are increased monotonically up to the condition of local or global collapse. The horizontal displacement d_c of a control point, located at the center of mass of the last level, is monitored and the curve F_b-d_c is the capacity curve of the structure.

According to NTC2008, this analysis has to be performed with two different distributions of horizontal forces, named primary (Group 1, GR1), and secondary (Group 2, GR2) force distribution. The force distributions relate to the largest value of participating mass of the natural modes for a chosen direction of the seismic action.

Whenever the largest participating modal mass is less than 75% of the building mass in the seismic configuration and the period of the first mode is larger than the corner period T_c between the constant–spectral–acceleration and the constant–spectral–pseudo–velocity ranges of the response spectrum, as it is the case here, the GR1 force distribution is derived from the seismic floor shears calculated in a linear dynamic analysis for the chosen direction of the seismic action. The Group 2 adopts a uniform distribution of forces, that would correspond to a uniform distribution of accelerations along the height of the building.

5.1 Structural configurations at study

Two structural configurations were analyzed, namely: with and without the REI wall on beam 18-29. In addition, two sets of material parameters were considered, derived either from design specifications or from material testing. For each structural configuration and set of material parameters, several push-over analyses were carried out applying the seismic action in different directions. The floor forces were obtained from linear dynamic analyses with the response spectrum in [7], combining the effects of the seismic action in the X (East-West) and Y (North-South) direction (see Figure 4) as it follows ±X, ±Y, ±0.3X, ±0.3Y. The most severe combination turned out to be -0.30X-Y. This combination is very close to that corresponding to the extreme values of the ground acceleration at the site, characterized by an X-acceleration that is 35% of the Y-acceleration. For this ratio of seismic actions the effects of accidental eccentricities equal to 5% of the floor X and Y dimensions were also introduced.

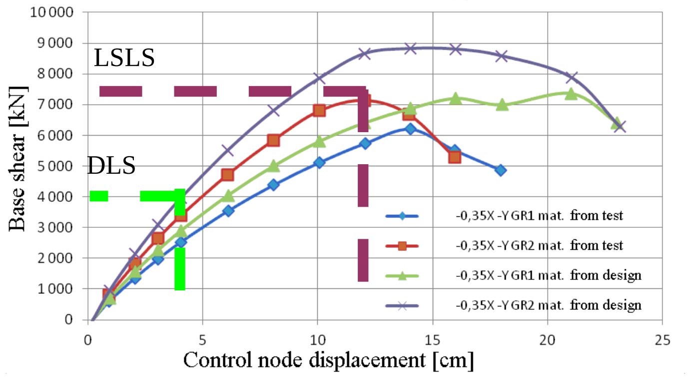
5.2 Results of the pushover analyses

For the sake of brevity, only the results obtained for the direction -0.35X-1.0Y of the seismic action will be presented in the following. Figure 10 depicts the capacity curves of the structure, for GR1 forces distribution, as well as the estimate of the displacement demands at the Damage Limit State (DLS) and the Life Safety Limit State (LSLS). The last one corresponds to the ultimate limit state for seismic loading in EN 1998-1:2005 [8]. Figure 11 shows the deformed configurations with and without the REI wall for a displacement of the control node d_c = 23 cm.

The estimated displacement demands in Figure 10 (4.0 cm at the DLS, 11.8 cm at the LSLS obtained with the response spectrum in [7] evaluated at the CdS) clearly highlight that also the original configuration, without the REI wall, would probably have collapsed under the earthquake of April 6th, 2009. The activation of a soft/weak-story mechanism is clearly visible in both the original and renovated configuration. In the second configuration the mechanism is more apparent, so that it would seem the insertion of the REI wall has adversely affected also the ground-story performance.

From the analysis of the sequence of local collapses, it can be observed that, when the REI wall is included, the beam 18-29 fails in shear at the first floor for a displacement of the control node $3 \leq d_c \leq 4$ cm, then at the upper stories for $4 \leq d_c \leq 6$ cm. The collapse of the North wing columns at ground floor starts at $4 \leq d_c \leq 6$ cm with failure in shear of column #29. The structural configuration without the REI wall sees the first collapse at a larger value of the displacement of the control node, $6 \leq d_c \leq 8$ cm, when some of the North wing columns at ground floor fail in shear. Whatever the material properties, the force distribution and the configuration (original or renovated) considered, the elements reaching failure are the same, and coincide with those that actually experienced the collapse during the 6 April 2009 earthquake.

a)



b)

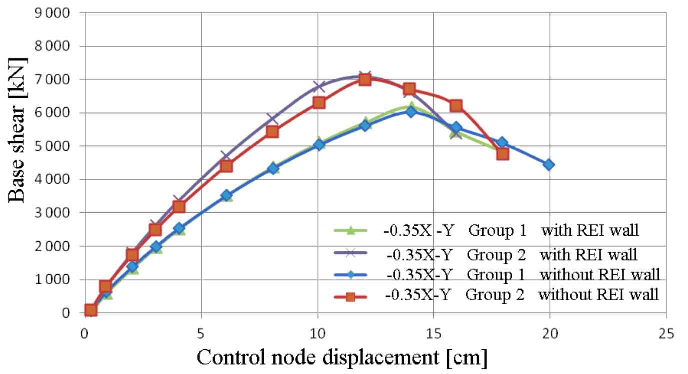


Figure 10. Capacity curves for Group 1 and Group 2 forces distributions: (a) REI wall included and material parameters from design and from tests; (b) with and without the REI wall, material parameters from tests.

The effect of the material properties is investigated in Figure 10a, where the results for the different materials are compared for the two force distributions considered. The poor quality of the materials affects more the base shear capacity of the structure than its ductility. The final value of the displacement under GR1 force distribution is practically the same, regardless the material properties adopted.

The effect of the REI wall is investigated in the Fig. 10b. The wall tends to exacerbate formation of a soft story at the ground floor and induces early failure in shear of beam 18-29. The failure of this beam accounts for the collapse of portion of the floor system in the zone from the end of the REI wall to column #29. This failure can be clearly detected at several floor in the pictures taken immediately after the seismic event of April 6th, before the remains of the North wing were taken down (see for example Figure 3 right). The collapse of column #29 can be ascribed to the increased shear force at the beam interface, causing ultimately the collapse of the column joints in shear and the loss of support of the beam 29-30. The wall plays an adverse role for this column too.

The detected shear failure of beams and columns may depend on the formula adopted for the computation of the shear strength of these members. The sequence of failures, however, do not change if one adopts a standard ideal truss with 45° struts or the more generous (in terms of shear strength) variable inclination strut method in [8].

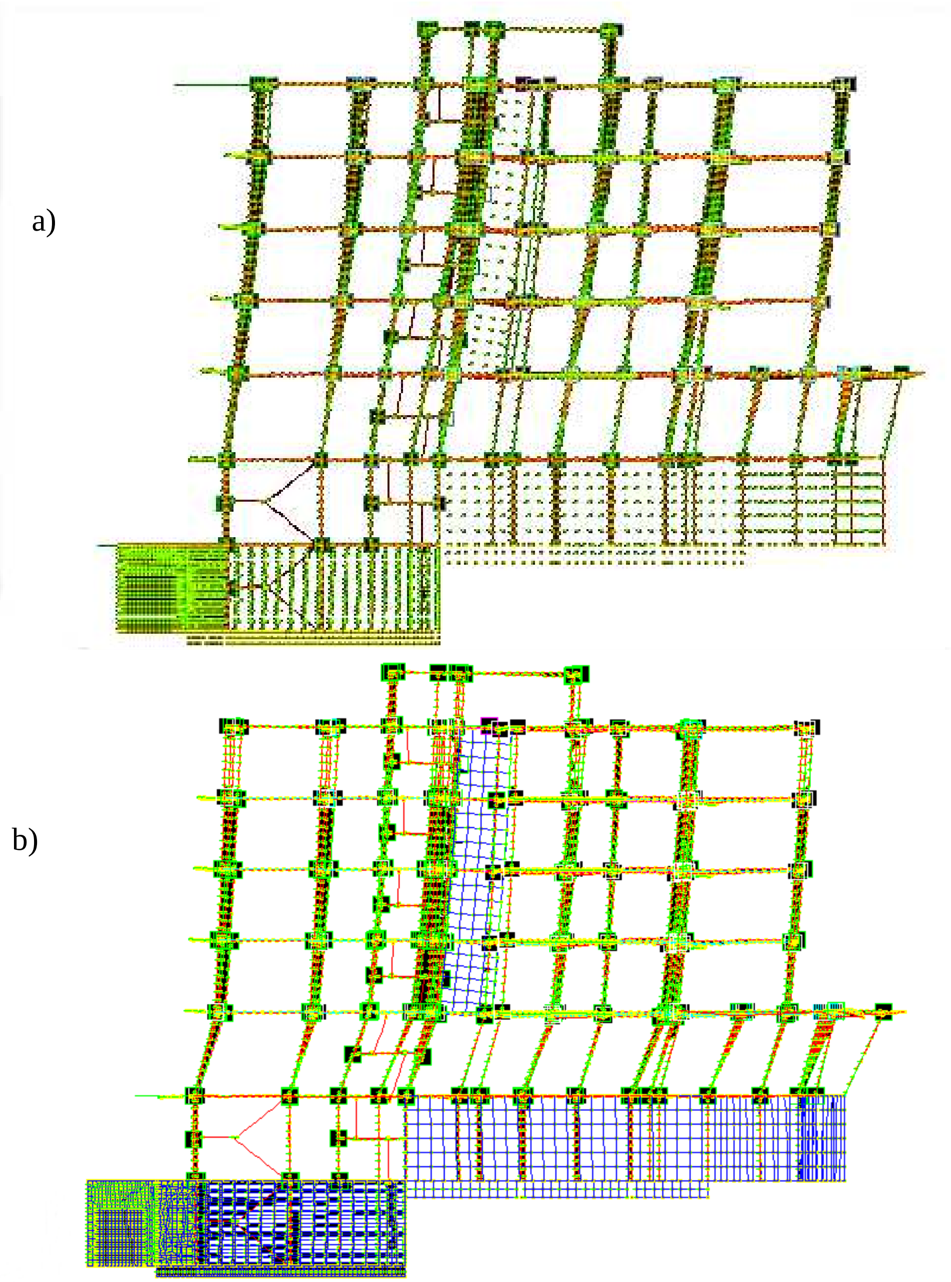


Figure 11. Displaced structure for Group 1 forces distribution and seismic action -0.35X-1.0Y (a) without the REI wall (b) with the REI wall. The control node displacement is 23 cm in both cases.

6 CONCLUSIONS

The results of the pushover analyses confirm the activation of a partial collapse mechanism, restricted to the part of the building where it actually took place, as already found from linear analyses and simple hand computation in [4]. The numerical analysis predicts well also the elements that fail during earthquake; if performed before the earthquake, could have issued a warning about the inadequacy of the building at study to sustain the design earthquake.

From the numerical results here presented the effects that the REI wall, inserted during refurbishment works, had on the structural behavior are well highlighted. The collapse sequence points out that the combined effect of the REI wall and of the lack of shear strength of beam 18-29 is responsible for the collapse of one of the three failed columns, column #29. For the other two, the collapse can be ascribed to the soft/weak story mechanism at the ground floor, in turn triggered by shear failure of the columns. A partial soft story collapse would have taken place also in the original configuration of the building.

The numerical code here adopted detects a shear failure according to the rules of the Italian Code [7], and accounts for a linear behavior in shear even after detecting a failure. Further analyses with a research oriented code are underway, aimed to better describe the collapse sequence.

DISCLAIMER AND ACKNOWLEDGEMENTS

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