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NON-LINEAR DYNAMIC ANALYSES OF AN RC FRAME BUILDING COLLAPSED DURING L'AQUILA 2009 EARTHQUAKE

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Abstract. *This study focuses on the partial seismic collapse of a building during the earthquake of L'Aquila (Italy) on April 6th 2009. Designed in the early '60s, the 7-story building has a reinforced concrete frame structure. In elevation, above a first basement and a ground floor, three wings rise. The collapse affected the North wing, where three separate collapse mechanisms were identified. All the columns at ground story failed with a weak-story mechanism in North-South direction. Three columns, located near the interface with the other wings, failed on the full height. In the same area, subjected to strong distortions due to the difference in vertical displacements following the weak-story mechanism, at stories 1-4 a third collapse mechanism involved a beam supporting a non-structural wall inserted in recent renovation works. Previous investigations carried out with both pushover and nonlinear dynamic analyses explained why the collapse was confined to the North wing, the collapse sequence and the role of the non-structural wall. Based on nonlinear dynamic analyses on a refined 3D FE model, this study focuses on the role of the East and West wings that survived the earthquake. The subdivision of the base shear among the three wings at the collapse onset confirms that only few elements in the East and West wings contributed to resist horizontal forces in the North-South direction where the weak story mechanism took place.*

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1 INTRODUCTION

During the April 6th 2009 earthquake of L'Aquila (Italy), a partial collapse causing a high death toll took place on a seven-story reinforced concrete (RC) building located at about 6 km from the epicenter. The building, named in Italian "Casa dello Studente" (CdS) i.e. *House of Student*, hosted an University dorm. Its partial collapse is one of the cases of failure of RC buildings in the urban area of L'Aquila (EERI Special Report 2009 [1]). The 7-story building (including two underground stories) had an irregular T-shaped plan composed of three nearly rectangular "wings" connected through a common stairwell (Fig. 1a). The building was designed in 1965, following the Italian Seismic Codes of years 1937 [2] and 1962 [3], using the design approach of equivalent static lateral forces. The building collapse is connected to its complex history. Built in 1965 by a private company for a mixed residential-commercial use, in 1979 it was purchased by an University Institution to be used as a student dormitory. In 1982, by virtue of a state law, it became property of the Regione Abruzzo, an administrative partition of the Italian Republic. The rules for the building management changed for three times between 1982 and 2009, and different public institutions were involved in it. To make the building more apt to its usage as a dorm, an extensive refurbishment intervention was performed during the years 1999-2002, which did not involve any intervention on the structural system. Unfortunately, the original design of the structural frame system was affected by an error that was brought to the light neither in the two changes of property nor in the design phase of the refurbishment intervention.

The partial collapse was limited to the North wing facing XX Settembre Street (Fig. 1). Previous studies conducted for legal reasons [4, 5], pinpointed the existence of two distinct collapse mechanisms in the collapsed wing: a) failure of all columns at the ground floor, which triggered a soft/weak story mechanism in the North-South (N-S) direction, and b) failure of three columns along the total height of the building. In the same region, the mid-span failure at floors 1-4 of the beam connecting columns 18 and 29 took place (see Fig. 1b). The beam was affected by the insertion of a non-structural wall during the renovation works of 1999-2001. Legal tests and studies showed that the lack of flexural strength of ground story columns was detected as the main mechanical cause of the collapse.

Given the availability of material properties, acting loads and geometric data on both the structure and the steel reinforcement, the collapse sequence was already investigated through-both pushover analyses [6] and non-linear dynamic analyses on a 3D complete and detailed numerical model of the building [7]. The latter were performed with the OpenSees platform [8], using the accelerograms at the site. In [7] the collapse sequence of the North wing was simulated, and the role of the non-structural wall was examined. This study aims to ascertain the role of the East and West wings that survived the earthquake. The main longitudinal frames of these wings are aligned along the East-West (E-W) direction, but the two transverse frames in each wing are aligned along the N-S direction of the weak Heavy damage was detected for the transverse frames, while the frames along the E-W direction did not show serious damage. In this work, the response of both longitudinal and transverse frames is studied and compared to the above damage scenario.

Section 2 describes the building structure and the earthquake characteristics. The damage observed after the earthquake is described in Section 3. The numerical model adopted in non-linear dynamic analyses is presented in Section 4. Results are discussed in Section 5. Conclusions in Section 6 address the role of longitudinal and external transverse frames on the collapse sequence.

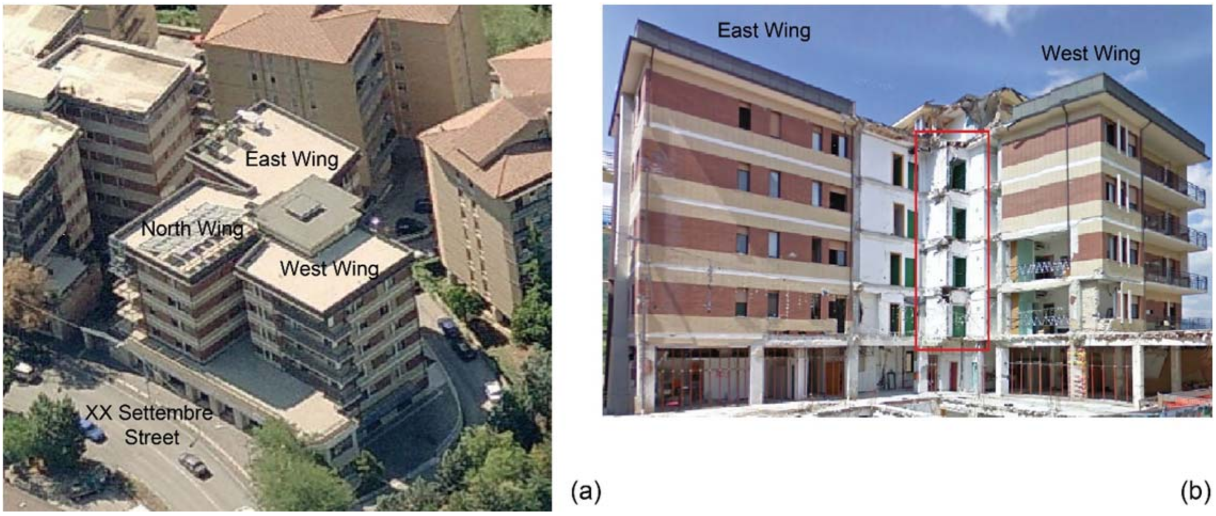

Figure 1: The CdS building: (a) aerial view, original configuration; (b) front view from XX Settembre Street after the removal of debris. In the red box the remaining of beam 18-29. The severely damaged external partitions of the West wing were removed for safety reasons. The jacketed column had several shear cracks. (after [7])

2 BUILDING DESCRIPTION

A brief description of the building, of the measured ground motion as well as of the observed on-site damage is given below. A more detailed presentation can be found in [5]. In elevation, the CdS building had five stories with a common basement and a second underground story below the East wing, for a total height of 25.5 m. In plan, the RC framed structure had an irregular shape, since it was composed of three nearly rectangular "wings", two roughly oriented in the E-W direction and one in the N-S direction (Figs. 1-2). Several non-structural refurbishments have characterized the building life. Changes in the use of spaces required the introduction and/or modification of several internal partitions and the modification of the technical equipment [4]. Between 1999 and 2002, the layout of the internal partitions was changed to adapt the building to its use as a university dorm. During this period, a fire resistant non-structural wall (REI wall) was placed at stories one to four to comply with fire safety regulations (see Fig. 2). The wall, composed of lightweight concrete hollow blocks 20 cm thick, was inserted from column 18 (C18) on half-span of the beam 18-29. The negative role on the collapse of this non-structural element has been widely discussed in [7].

The structure was designed according to the Italian Seismic Code of 1962 [3], even though the designer in his report affirms to adopt the Code of 1937 [2]. The seismic design principles of both 1937 and 1962 Italian Codes were based on the application, at the center of mass of each floor, of a set of equivalent static forces equal to 5% (1937 Code) or 7% (1962 Code) of permanent loads plus 1/3 of variable loads. A value of 7% was actually adopted in design. Static forces should have been applied in any horizontal direction according to the 1962 Code. Three main longitudinal moment-resisting RC frames are located in each wing (Fig. 2) whose design was driven by the afore-mentioned criteria. The horizontal loads acting on each frame were computed adopting the same tributary areas used for vertical loads, but only in the direction of the frame itself. The building plan configuration in Fig. 2 shows that, of 9 longitudinal RC frames, only the three frames of the North wing (21-24, 25-28 and 29-31) were aligned along the N-S direction. The other frames (1-4, 5-8, 9-11, 15-17, 32-35 and 36-39) mainly resisted the E-W seismic loads. A transverse frame was placed both on the short and on the inclined side of each wing, but without any specific design provision. As a consequence, the design procedure led to a significant underestimation of seismic forces in the N-S direction.

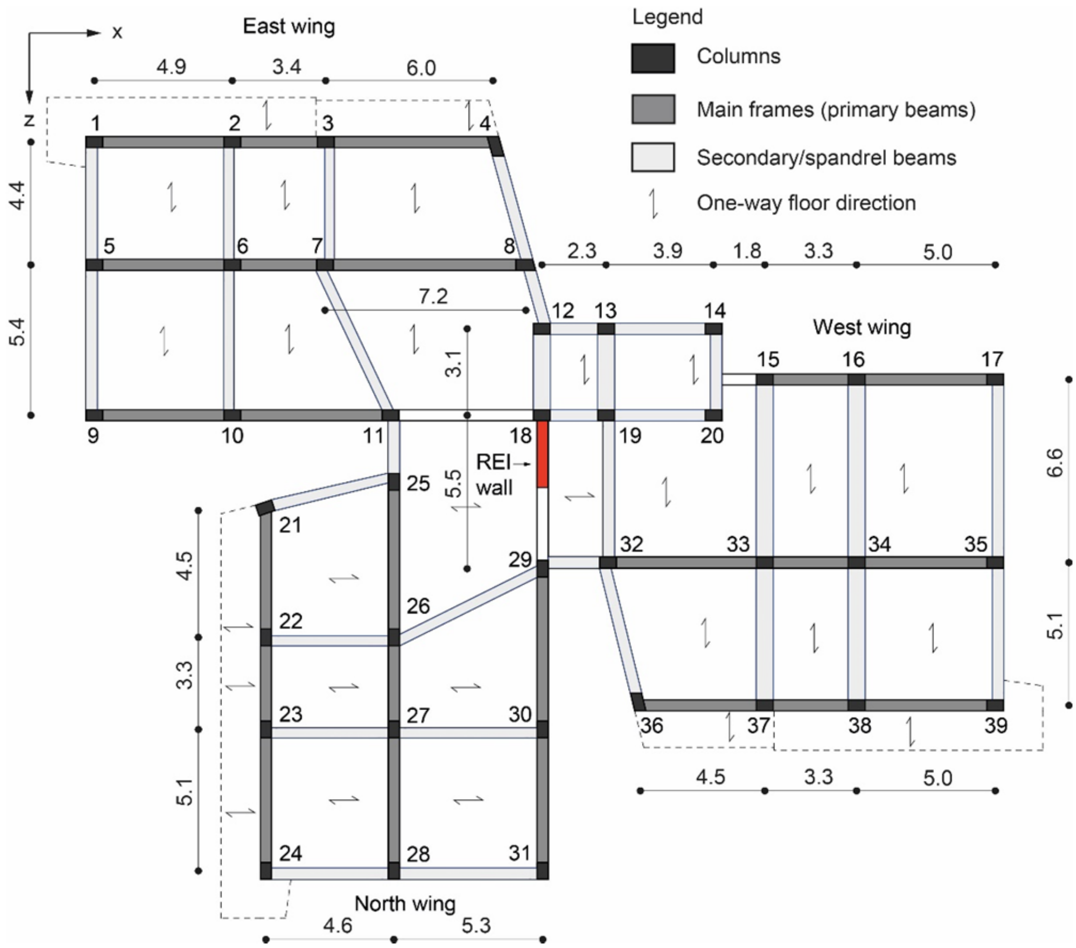

Figure 2: Structural plan layout of a typical residential floor and column numbering, units: m. (after [7])

Field investigations, based on destructive and non-destructive tests, showed that the measured steel mean yield strength value (f_{ym} = 388 MPa) is very close to the value prescribed in design. On the other hand, the measured mean cylinder compressive strength f_{cm} is 13.3 MPa, that corresponds to a concrete of class C8/10 according to the classification in the Italian Code NTC2008 [9], is far lower than the design value, equivalent to the class C16/20.

The values of dead and live loads at the date of collapse are given in [5]. Loads are obtained from site investigations and estimates from handbooks of the sixties. The self-weight of the slab, furniture and partitions ranges between 6.0-9.3 kN/m^2 at the different floors, while the live load value ranges between 2.0-3.0 kN/m^2, a typical value for a residential building.

In the original structural report, the designer described the reinforcement geometry and layout for the members of one “typical frame” that can be identified as the central frame of the North wing. Also, the designer declared that all the other frames had the same geometry, in terms of cross-sections dimensions and reinforcement layout, of the one actually designed. The reinforcement layout was verified through on-site non-destructive and destructive tests on the East and West wings after the earthquake. The longitudinal column reinforcement was designed to resist internal forces acting on the frame plane only and for this reason was located along each short side of the rectangular cross-section of the columns. The column transverse reinforcement is composed of closed stirrups with a diameter of 6 mm with a spacing, for most of the columns, of about 200 mm without provisions for closer spacing at the two column ends.

The horizontal ground motion at the building site was estimated from the actual motion recorded at the nearby station named AQK and the soil properties at the two sites (CdS and AQK sites), by applying a convolution and deconvolution procedure described in [5]. For the vertical ground motion, the actual acceleration recorded at the AQK station was adopted. The horizontal peak ground motion acceleration along the two principal directions of the building, namely the x and z direction in Fig. 2 is respectively equal to 0.346 g and 0.510 g.

The elastic response spectra along the principal direction of the building for a 5% damping ratio (not shown here for the sake of brevity) points out a prevailing amplitude of the N-S component (the fault normal direction) at long periods, as a consequence of the directionality of the earthquake ground motion in L'Aquila [10]. In the period range of the first three numerical modes (0.93–0.99 s, see Section 4), the elastic spectral accelerations at the site along N-S direction are of the order of 0.55 g, that is about 50% larger than the orthogonal component.

3 OBSERVED DAMAGE

The partial collapse of the structure was limited to the North wing (Fig. 1), where two separate collapse zones were detected. The columns at the ground story failed with a soft/weak story mechanism along the N-S direction (Fig. 3a). In addition, three columns (C21, C25 and C29 in Fig. 2) collapsed totally from the ground story to the top of the building, at the interface between the North wing and the remaining ones (Fig. 3b). Part of the beam 18-29 (where the non-structural wall was not inserted in) collapsed as well.

Figure 3: View of the building before demolition of the collapsed wing: (a) weak ground story mechanism on the North wing. Windows along XX Settembre Street, aligned on the E-W direction, are intact. (b) Collapse of the columns at the interface between the North wing (right) and the remaining ones (left and back). (after [7])

Structural damage was observed in the East and West wings especially in the columns that are part of the transverse frame placed both on the short and on the inclined side of each wing. Figure 4 shows the damage at the beam-column joint of the ground story column C5, belonging to the external transverse frame of East wing.

The most likely collapse sequence was determined in [5] based on (i) the study of the design criteria adopted in 1965; (ii) the results of modal analyses of the building in its original configuration and after refurbishment; and (iii) the analysis of damage in the parts of the building that survived the earthquake. The severe lack of flexural strength of ground story columns of the North wing and the corresponding shear failure, as a result of design and construction errors, appeared to be the main cause of the collapse. Moreover, a recent study [7] proved the independency of the two collapse mechanisms (weak-story mechanism at ground floor of North wing and collapse of part of the beam 18-29) and established their sequence.

Figure 4: Damage at the beam-column joint of column C5 at ground floor.

4 NUMERICAL FE MODELLING OF BUILDING

The simulation of the seismic collapse is performed with the open–source platform OpenSees [8]. The beam–column element with distributed inelasticity and force formulation is selected for the FE model [11]. The element formulation is based on the Navier–Bernoulli's hypothesis. The section response is computed by integrating the inelastic material response through the section depth (fiber section). The geometric nonlinearity is included through a P–Δ formulation. Uniaxial material laws describe the hysteretic response of the materials. The modified Kent–Park model [12] describes the response of the concrete fibers in compression. For concrete in tension, a linear elastic branch is followed by a linear softening branch up to zero stress to include the effect of tension stiffening. The well-known nonlinear model of Menegotto and Pinto [13], as modified by [14] to include isotropic hardening effects, is used for the steel fibers.

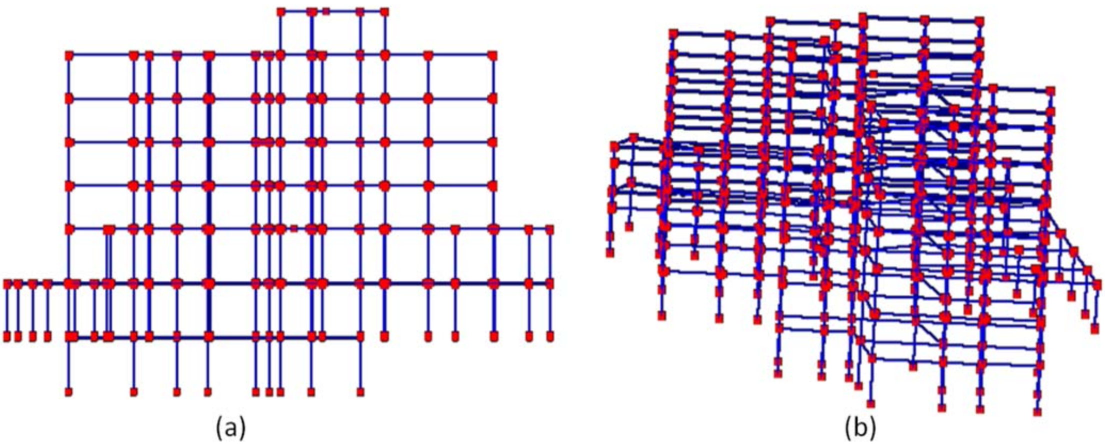

Figure 5: 3D FE model for the whole structure: (a) 2D view in the X-Y plan, North (right) and East (left) wings; (b) 3D view.

A comprehensive 3D model of the building reproduces the configuration of April 6th 2009 in terms of geometry, masses and stiffness of structural elements. One nonlinear fiber beam–column element with five Gauss–Lobatto integration points is used for both columns and primary beams. Floors are modeled as rigid diaphragms. The mass and weight of the balconies are directly applied to the perimeter beams. The non-structural wall on beam 18-29 is modeled with 2D plane-stress (membrane) elements. Rigid beam-column joints extend to half of the dimension of the elements framing into them. The FE model has 1835 nodes and 805 elements. Among them, 449 are nonlinear beam elements. Figure 5 presents a schematic view of the nodes and elements layout. Columns at second basement of the East wing and at first basement of the North and West wings are fully restrained at their base. At first basement, a roller on top of columns facing XX Settembre Street restraints the N-S displacement component. The choice of these boundary conditions was driven by in-situ observations.

The average values of material properties coming from on-site and laboratory material tests are used for the unconfined concrete in the numerical analysis. The low amount of stirrups in both columns and beams and the lack of longitudinal reinforcement along the long sides of the rectangular cross-sections limit the confinement effect. The material properties of the Kent-Park model for the unconfined and confined concrete are used in the analysis [7].

Table 1 lists the natural periods for the first 10 modes of the FE model that simulates the building configuration of April 6th 2009. The comparison of modal properties with those of a linear model (including the floor deformability) previously developed [5] provided a model validation. The first mode mainly involves a translation along the N-S direction with a small torsional component. The second and fourth mode combine a translation of the floors in the N–S direction and a predominant torsional component. The third mode involves a translation of the floors in both the E–W and N–S directions without torsional component.

Mode #	1	2	3	4	5	6	7	8	9	10
Period [s]	0.99	0.95	0.93	0.38	0.36	0.35	0.25	0.24	0.23	0.21

Table 1: Numerical model, natural periods of the first ten modes.

5 RESULTS AND DISCUSSION

A static analysis under gravity loads is performed in a first step. Subsequently, the model is analyzed by applying the three components of the earthquake ground motion. The Newmark time integration method of constant acceleration is adopted, with a time step $\Delta t = 0.01$ s. A viscous damping of Rayleigh-type proportional to the mass and initial stiffness is assumed. A damping ratio of 3.0% is imposed on the first and fourth mode. The Newton–Raphson iteration method is used to enforce equilibrium at each time step.

The step-by-step analysis does not include a progressive collapse algorithm simulating the loss of structural elements and the modifications to the geometry of the structure. A previous study [7] showed that the weak-story mechanism at ground floor is activated first. In the real case, once this mechanism is activated, the structural configuration and the dynamic behavior of the building undergo significant changes. Thus, after the collapse onset the numerical results become less reliable. In spite of this limitation, the results up to the onset of collapse give several important indications on the failure modes of the structure and on its response to L'Aquila 2009 ground motion, as it is widely discussed in this section.

Previous nonlinear dynamic analyses indicated that the prevailing building displacement component is directed along the z-axis (N-S direction), parallel to the direction of the North wing frames which, as discussed in Section 2, are the only ones to resist significantly to horizontal forces in this direction [7]. Due to the rigid diaphragm assumption, the numerical displacement magnitude is similar in the three wings. However, columns in the East and West wings did not suffer significant damage (with some exceptions discussed in the following). This seeming contradiction is explained by the fact that their reinforcement layout is designed to withstand moments about the *z*-axis, their weak axis is aligned on the *z*-axis, the strongest earthquake direction and there are no transverse stiff beams. As a consequence, these columns are not highly stressed in spite of high displacement values detected at the top floor.

To study the demand imposed on each wing during the earthquake, the response of their central frames is analyzed. The significant response parameters are shear force and bending moment at the base of ground story columns, i.e. at the level where the weak story mechanism took place. Each frame has a strong in-plane direction: the *x*-direction for the East and West frames, the *z*-direction for the North ones. The out-of-plane direction is the weak one.

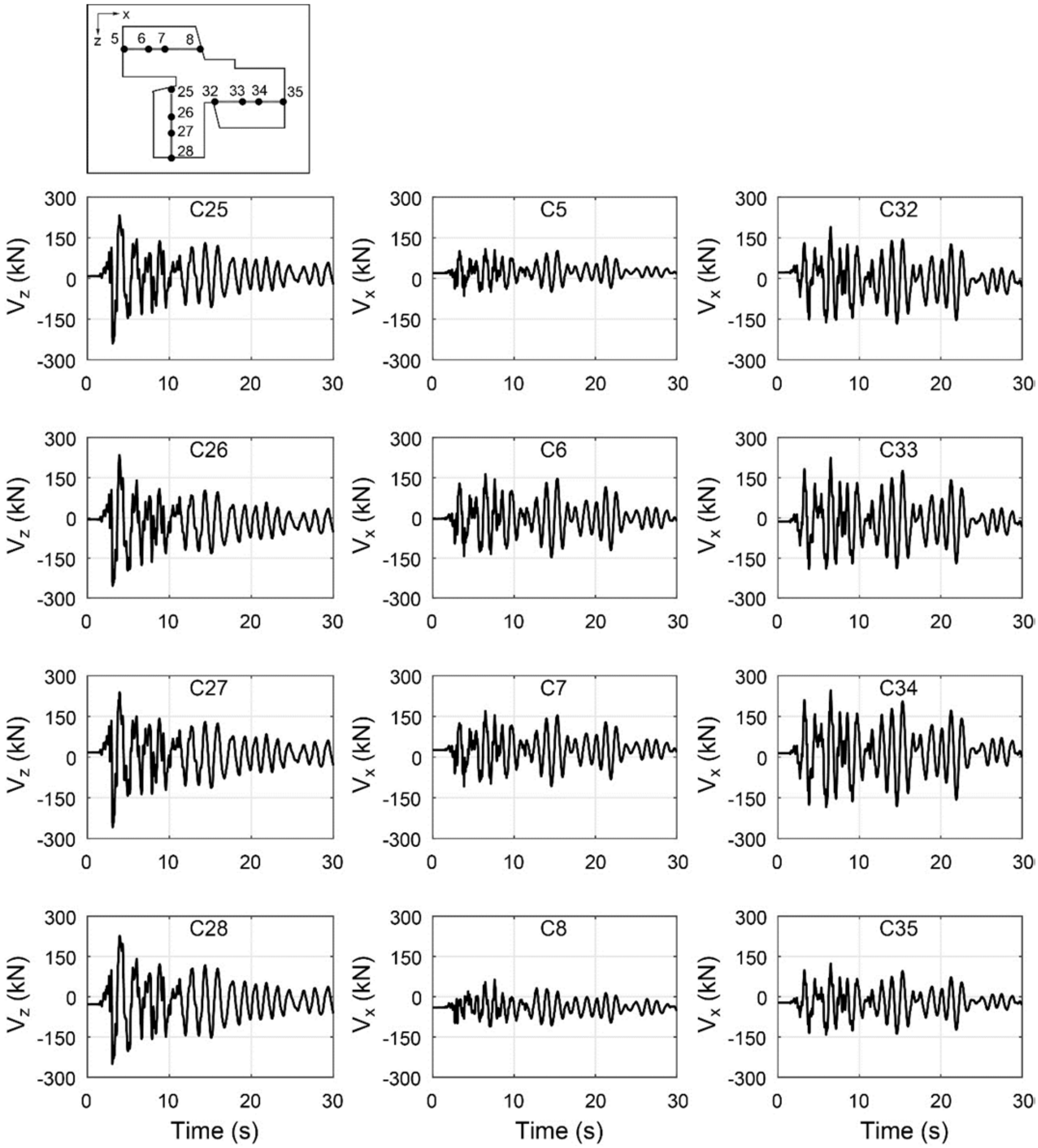

Figure 6: Shear force V_z and V_x along the strong axis at the base of ground story for the central frame columns of the North (on the left), East (at center) and West (on the right) wings.

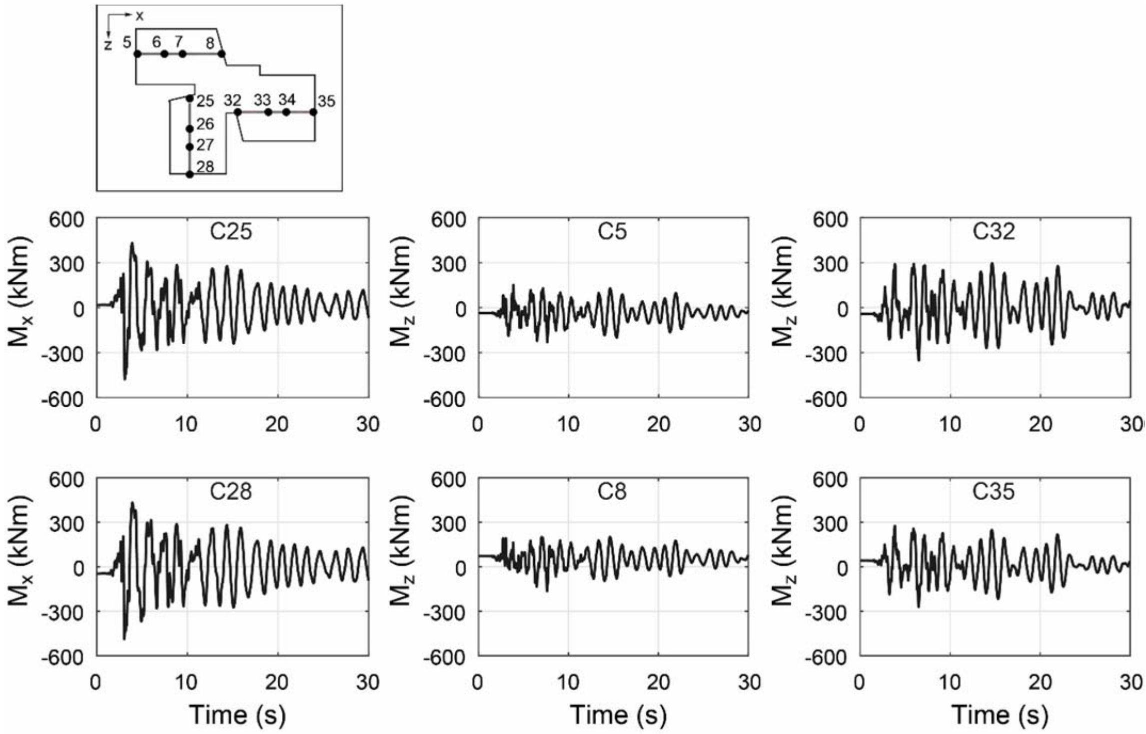

Figure 7: Bending moment M_x and M_z along the strong axis at the base of ground story for the central frame columns of the North (on the left), East (at center) and West (on the right) wings.

Figure 6 shows, for the central frames of the three wings, the shear forces at the base of ground story columns in the plane of the frame, i.e along the strong direction of each frame. The largest value of shear force, in terms of V_z, are attained in the North wing. The demand on East and West wings is lower, but not uniform on the two. The smallest values of V_x are found in the East wing, with a minimum on C5 and C8 at the two ends of the frame. A larger demand is detected in the West wing, with the exception of C35, located at the maximum distance from the North wing.

In a recent work, Mulas and Martinelli [7] have compared the shear demand with the shear capacity for the N-S direction of columns of the North wing. The shear demand during the earthquake largely exceeds that associated to static loads and, to a smaller extent, the shear capacity V_{Rd} evaluated according to the current Italian Technical Code NTC2008 [9]. For the structural members considered, the shear resistance corresponds to a shear failure of the transversal reinforcement. For all the ground story columns of the North wing, the shear strength is exceeded in the N-S direction and in a very short time interval, between 3.002 and 3.034 s. This almost simultaneous exceeding of the shear strength indicates the activation of a weak-story collapse mechanism and justifies the failure of the affected columns. In Figure 6, the maximum shear values in East and West columns are reached after this time interval. Hence, at the instants in which the East and West ground story columns were subjected to the maximum shear demand, the collapse of the North wing had already taken place. The flexural beam-column element here adopted has no provisions for non-linear behavior in shear, which has been detected from the results of the analysis. Consequently, the model does not reproduce the drop of building stiffness in the N-S direction caused by the weak-story collapse. Thus, there is no conflict between the numerical results and the reduced amount of damage of the central frames in the East and West wings.

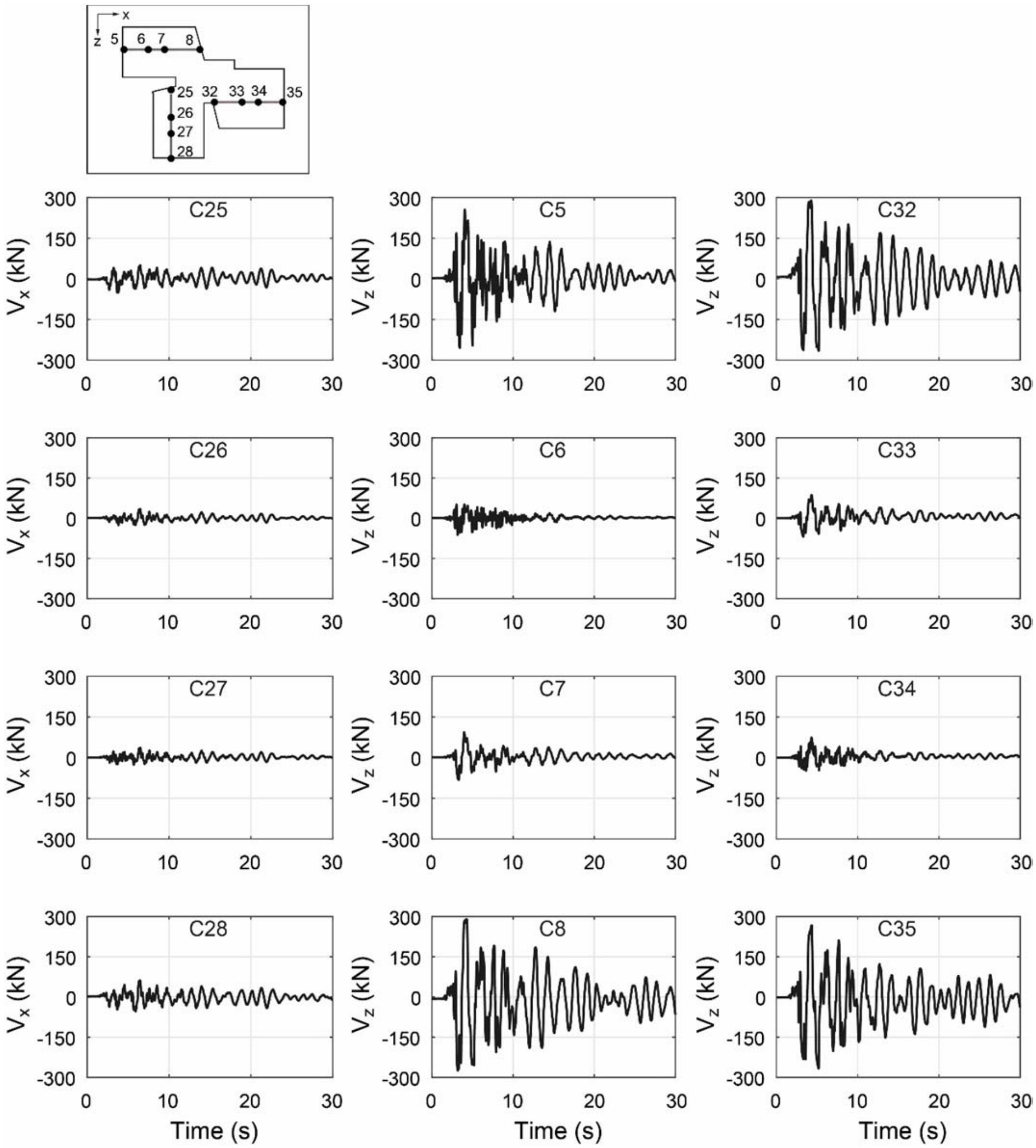

Figure 8: Shear force V_x and V_z along the weak axis at the base of ground story for the central frame columns of the North (on the left), East (at center) and West (on the right) wings.

The in-plane bending moment at the base of ground story columns, for the same frames analyzed in Figure 6, has the same pattern of the in-plane shear. As an example, Figure 7 depicts the results for the columns at the ends of each frame.

Figures 8 illustrates, for the central frame of the three wings, the time history of shear forces at the base of ground story columns, along their weak axis. The out-of-plane demand along the *x*-axis on the North wing is very low. This finding is in agreement with the N-S direction of the prevailing displacement component: the numerical result is confirmed by both the absence of damage in the windows along XX Settembre Street (Fig. 3a) and rescuers' observations. Also, low amplitude values are detected for the central columns of the East (C6 and C7) and West (C33 and C34) wings. On the contrary, external columns of the East (C5 and C8) and West (C32 and C35) wings are subjected to high shear forces.

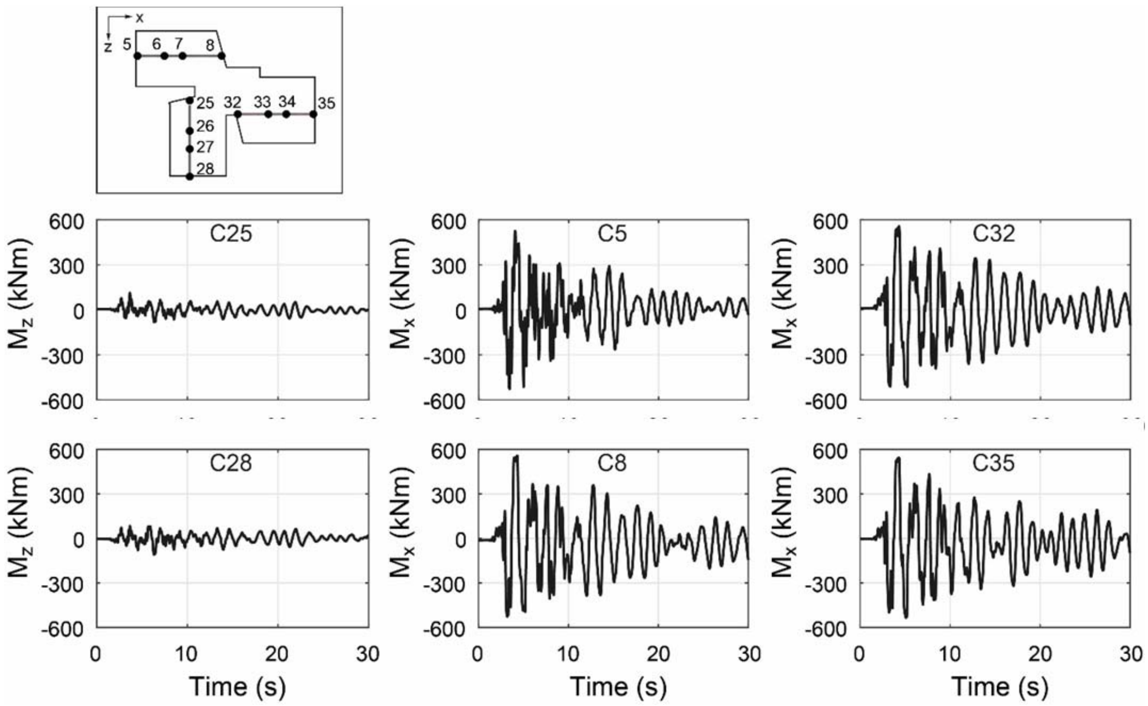

Figure 9: Bending moment M_x and M_z along the weak axis at the base of ground story for the central frame columns of the North, East and West wings.

Thus, for the East and West wings, the out-of-plane shear demand reaches the highest values in the columns where the demand was lowest in the strong in-plane direction. The columns experiencing the highest demands in the N-S direction, i.e. the weak direction for East and West wings, are those belonging to the transverse frames along the N-S direction (Fig. 2).

Figure 9 depicts the time history of the bending moment, for the central frames of the three frames, along their weak direction. As for the response in the strong direction, shear and bending moments have a similar pattern, and only the results for the external columns of each frame are presented. The bending moment magnitude for columns belonging to the two transverse frames of East and West wings is remarkable: these columns are subjected to high demands, showing their contribution to resist to the strong direction of the earthquake.

To investigate further this aspect, the transverse frames placed both on the short and on the inclined side of the East (columns C1, C5, C9 and C4, C8) and West (columns C17, C35, C39 and C32, C36) wings are considered. Figures 10 and 11 show the shear forces V_z along the N-S direction for the ground story of edge columns of East and West wings, respectively. The time instant at which the weak-story mechanism was triggered in the North wing is denoted with a dotted line in each time history. Both figures show that the edge columns belonging to the transverse frames were significantly loaded along the N-S direction, with the exception of columns C9 and C17. These columns are approximately aligned with the structure supporting the stairs, which can have played a role in resisting seismic forces in the N-S direction too. It can be concluded that the edge column on the short side of East and West have contributed to the global shear capacity along the N-S direction. However, it is apparent that this contribution was not sufficient to avoid the collapse of the North wing.

Finally, the subdivision of shear force at the base of ground story among the columns belonging to the three wings and to the stairs, during the time interval before the weak-story mechanism (0-3 s), is analyzed. Both shear forces in the x- and z-directions are considered.

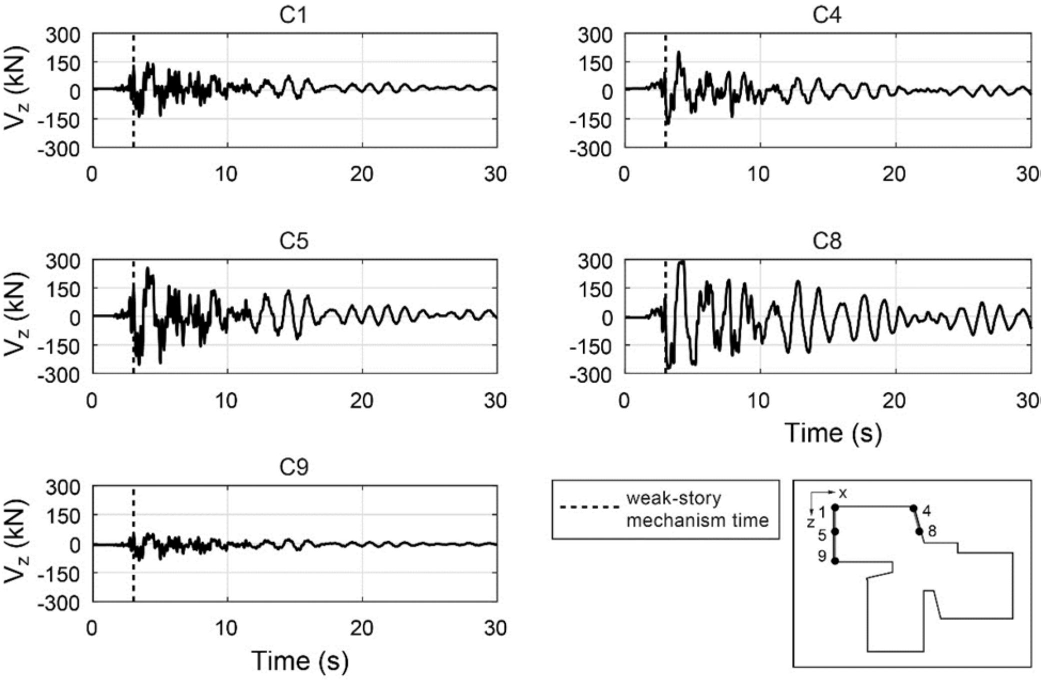

Figure 10: Shear force V_z at the base of ground story for the edge columns of the East wing.

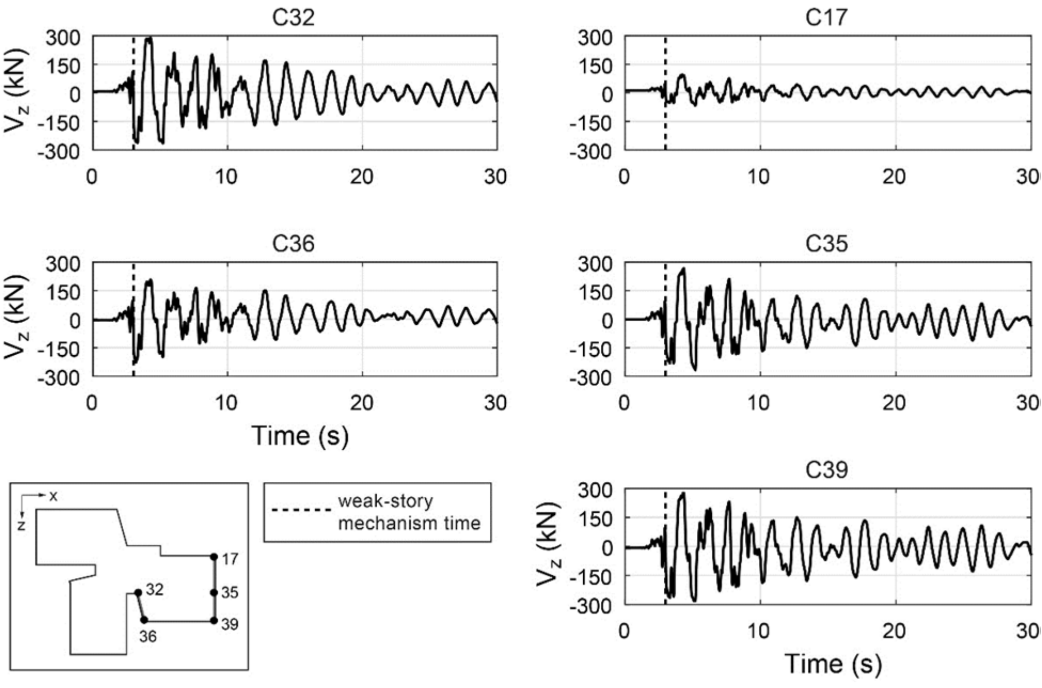

Figure 11: Shear force V_z at the base of ground story for the edge columns of the West wing.

Let us examine, as an example, the columns from C1 to C11 of the East wing. At each time instant t_i, it is possible to calculate a dimensionless index $\nu_x(t_i)$ in the x-direction:

$$\nu_x^{East}(t_i) = \frac{\sum_{C1}^{C11}|V_x(t_i)|}{\sum_{C1}^{C39}|V_x(t_i)|} \tag{1}$$

A similar index $\nu_z(t_i)$ can be defined in the z-direction:

$$\nu_z^{East}(t_i) = \frac{\sum_{C1}^{C11}|V_z(t_i)|}{\sum_{C1}^{C39}|V_z(t_i)|} \tag{2}$$

Equations (1) and (2) are also applied to the other wings and to the stairs. The indices $\nu_x(t_i)$ and $\nu_z(t_i)$ provide a measure of the level of shear demand along *x*- and *z*-directions for the columns of the different wings. The mean value of $\nu_x(t_i)$ and $\nu_z(t_i)$ on the time interval 0-3 s represents a compact index of the level of shear demand. Figure 12 shows the mean values of ν_x and ν_z: approximately 80% of the total base shear in *x*-direction is almost equally subdivided on the East and West wings (40% in each Wings). The North wing is subjected to very low shear forces in *x*-direction (less than 10% of the total). The situation is completely different in the z-direction where the North wing attracts more than 50% of the total shear actions, while the level of shear in each of the East and West wing is around 18% of the total.

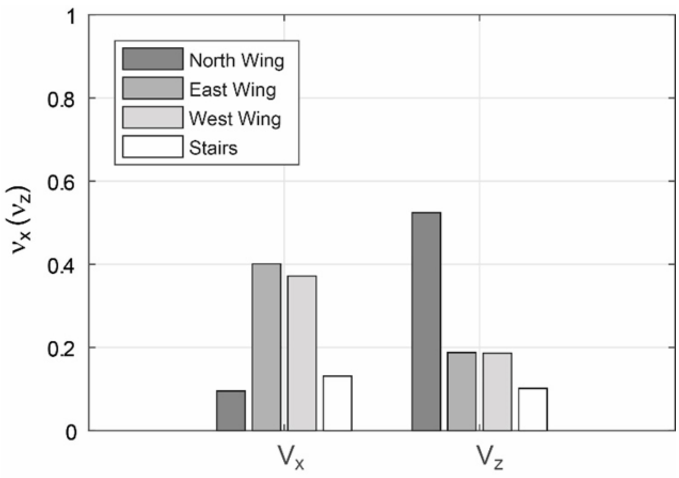

Figure 12: Shear force subdivision on the three wings and stairs at the base of ground story during the time interval 0-3 s.

6 CONCLUSIONS

The numerical simulation of the partial collapse of a 7-story RC building during the 2009 L'Aquila earthquake is presented in this study. The collapse involved the North wing of the building and was characterized by different partial mechanisms: a ground weak-story mechanism in the strong N-S direction of the earthquake, the collapse of three columns along the full height and the failure of a beam at stories 1-4. In this work a nonlinear FE model of the whole structure is adopted, reproducing the building characteristics at the time of collapse, in terms of geometry, material properties and loads [7]. The estimated seismic input at site is used in the step-by-step analysis. The reason why only the North wing collapsed and the analysis of the sequence of the mechanisms were the object of recent research studies [5-7]. In [7], with the same 3D FE model here adopted, the collapse sequence of the North wing was simulated. This study focuses on the role of the East and West wings that survived the earthquake. While the building has globally a bidirectional resistance, in each wing mono-directional frames are placed, following a typical design procedure of the 60's. In the East and West wings, the main longitudinal frames are aligned along the East-West (E-W) direction, but the two transverse frames in each wing are aligned along the N-S direction.

The numerical results indicate that the building developed a bidirectional resistance, but the demand was not distributed as the masses in each wing. The East and West wings attract around 80% of the total base shear V_x along the E-W direction, while the North wing frames are weakly loaded in the E-W direction. A completely different trend was observed along the N-S direction, where the North wing attracts more than 50% of the total shear actions V_z, while the level of shear in East and West wings is around 18% of the total. The two external transverse frames of the East and West wings, aligned along the N-S direction, attract significant shear forces in the same direction. Nevertheless, the shear capacity of these frames was not sufficient to avoid the weak-story mechanism at the ground story of the North wing. It can be concluded that the building collapse can be directly ascribed to the lack of bidirectional resistance within each wing, resulting from the design procedure described in Section 2.

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