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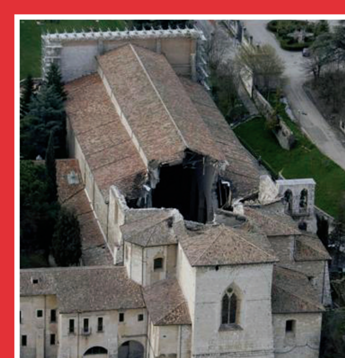
L'Aquila Earthquake of April 6, 2009 in Italy: Rebuilding a Resilient City to Withstand Multiple Hazards

by

Gian Paolo Cimellaro, Ioannis P. Christovasilis, Andrei M. Reinhorn, Alessandro De Stefano and Tatiana Kirova



Technical Report MCEER-10-0010

December 29, 2010

This research was conducted at the University at Buffalo, State University of New York, and the Politecnico di Torino in Italy. It was sponsored by the National Science Foundation and the European Community's Seventh Framework Programme (Marie Curie International Reintegration Actions – FP7/2007-2013 under the Integrated European Disaster Community Resilience Project).

MCEER

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MCEER is a national center of excellence dedicated to the discovery and development of new knowledge, tools and technologies that equip communities to become more disaster resilient in the face of earthquakes and other extreme events. MCEER accomplishes this through a system of multidisciplinary, multi-hazard research, in tandem with complimentary education and outreach initiatives.

Headquartered at the University at Buffalo, The State University of New York, MCEER was originally established by the National Science Foundation in 1986, as the first National Center for Earthquake Engineering Research (NCEER). In 1998, it became known as the Multidisciplinary Center for Earthquake Engineering Research (MCEER), from which the current name, MCEER, evolved.

Comprising a consortium of researchers and industry partners from numerous disciplines and institutions throughout the United States, MCEER's mission has expanded from its original focus on earthquake engineering to one which addresses the technical and socio-economic impacts of a variety of hazards, both natural and man-made, on critical infrastructure, facilities, and society.

The Center derives support from several Federal agencies, including the National Science Foundation, Federal Highway Administration, and the Department of Homeland Security/Federal Emergency Management Agency, and the State of New York, other state governments, academic institutions, foreign governments and private industry.

EARTHQUAKE ENGINEERING TO EXTREME EVENTS

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Alessandro De Stefano[4] and Tatiana Kirova[5]

December 29, 2010
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ABSTRACT

On the morning of Monday April 6th, 2009, at 03:32 local time (01:32 UTC), a strong shallow crustal earthquake with magnitude M_w 6.3 occurred in the Abruzzo region of central Italy. The epicenter was located 6 km southwest of the regional administrative city of L'Aquila and 85 km northeast of the Italian capital, Rome. This report describes the types and causes of damage resulting from the earthquake and the associated social and economic impact.

Funded by the National Science Foundation, a multidisciplinary team of investigators from the Multidisciplinary Center for Earthquake Engineering Research (MCEER), headquartered at the University at Buffalo, together with a team from the Politecnico di Torino in Italy, conducted post-disaster field reconnaissance to examine the impact of the earthquake on physical engineered systems and the response and recovery efforts that followed. By collecting information, the Politecnico di Torino together with MCEER is seeking to develop engineering design strategies and organizational strategies that will make the L'Aquila community more resilient against future earthquakes and any extreme event in general. The report was initiated to present the findings from the field reconnaissance mission, but the topic addressed includes advance damage identification using remote sensing, damage to engineered and historical structures, and organizational decision making primarily in hospitals. This report will contribute to the development of a better understanding of how to cost-effectively enhance the resilience of a community against future extreme events.

ACKNOWLEDGEMENTS

The work in this report is sponsored by MCEER (formerly the Multidisciplinary Center for Earthquake Engineering Research), and by the European Community's Seventh Framework Programme (Marie Curie International Reintegration Actions - FP7/2007-2013 under the project *ICRED - Integrated European Disaster Community Resilience*.

Any opinions, findings and conclusions or recommendations expressed in this report are those of the author(s) and do not necessarily reflect those of the Multidisciplinary Center for Earthquake Engineering Research (MCEER), the National Science Foundation (NSF), the State of New York (NYS) or the University at Buffalo.

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- Gian Paolo Cimellaro
- Alessandro De Stefano
- Tatiana Kirova
- Ioannis P. Christovasilis

The MCEER reconnaissance team arrived in the earthquake-affected area on April 19th, two weeks after the event, to survey damage, interview experts, and collect data. The team presented preliminary findings at a seminar in Buffalo.

We gratefully acknowledge the following individuals and sources for their contributions:

- Professor Mauro Dolce of University of Naples Federico II, for sharing information and data source with us.
- Professor Edoardo Cosenza of University of Naples Federico II for his generous logistical support for the team's field investigations.

Finally, we extend our sincerest thanks to the managers and personnel of RELUIS who took time out of their busy post-disaster schedules to provide tours of the red-zone in downtown L'Aquila.

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SECTION 1
INTRODUCTION

1.1 The Earthquake

On the morning of Monday April 6th, 2009, at 03:32 local time (01:32 UTC), a strong shallow crustal earthquake with magnitude M_w 6.3 occurred in the Abruzzo region of central Italy. The epicenter was located 6 km southwest of the regional administrative city of L'Aquila and 85 km northeast of the Italian capital, Rome. Figures 1.1a and 1.1b indicate the geographic location of the epicenter in Abruzzo, Italy.

The earthquake shaking was devastating for the city of L'Aquila and the villages in the Aterno River valley, southeast of the city-centre, resulting in 304 fatalities (the highest in Italy since the 1980 Irpinia earthquake) and 1,500 injuries. The collapse of residential structures, mainly low-rise unreinforced masonry construction, was the major contributor to life hazard as most people were sleeping when the earthquake struck. More than 200 fatalities were reported in L'Aquila, 37 in Onna, and 17 in Villa Sant'Angelo (EERI 2009). The significant level of structural damage observed in residential construction led to the temporary displacement of 80,000 residents and left 29,000 people homeless. Figure 1-2 illustrates places where loss of life was reported.

The earthquake was associated with a shallow focal depth (8.8 km) and a normal fault mechanism with northwest-southeast strike orientation and southwest dip direction. This suggests an extension of the earth's crust oriented perpendicular to the Apennines mountain range located along the Italian Peninsula. Advanced processing of satellite and GPS data before and after the seismic event – image illustrated in Figure 1-3 released by the Italian National Institute of Geophysics and Vulcanology (INGV) – show that the center of subsidence of the hanging wall was located midway between L'Aquila and Fossa, close to the village of Onna where the ground moved as much as 25 cm (along a line between the satellite's orbital position and the earthquake area). The large green square represents the M_w 6.3 main shock, the smaller green squares represent the $M_w > 5$ aftershocks and the black triangles represent GPS stations used for SAR validation. The yellow line east of L'Aquila shows the location of a ~4 km–long alignment of co-seismic surface breaks observed in the field by INGV researchers. This alignment corresponds to a northwest-southeast strip where the spatial fringe rate seems to exceed the limit for interferometric correlation. This may indicate that the fault dislocation reached, or was very close to, the surface along this line. The observed pattern of ground displacement is in very good agreement with the earthquake source mechanism (the 'beach ball'), confirming that the earthquake source is a normal fault striking 144 degrees (clockwise from north), and dipping to the southwest (Salvi et al. 2009).

The fault dislocation reached close to the surface at the boroughs of Paganica and Tempera, 4 km northeast of the city-centre, where tension cracks and a water pipe failure were observed (EEFIT 2009). This information explains the concentration of damage and human casualties in the area east and southeast of the fault epicenter, as shown in Figure 1-2. Similar observations are deduced from Figure 1-4 that illustrates the Mercall-Cancani-Sieberg (MCS) macroseismic intensity observed throughout the area hit by the earthquake. The highest intensities (IX and X) have been observed close to Onna and southeastward of the epicenter.

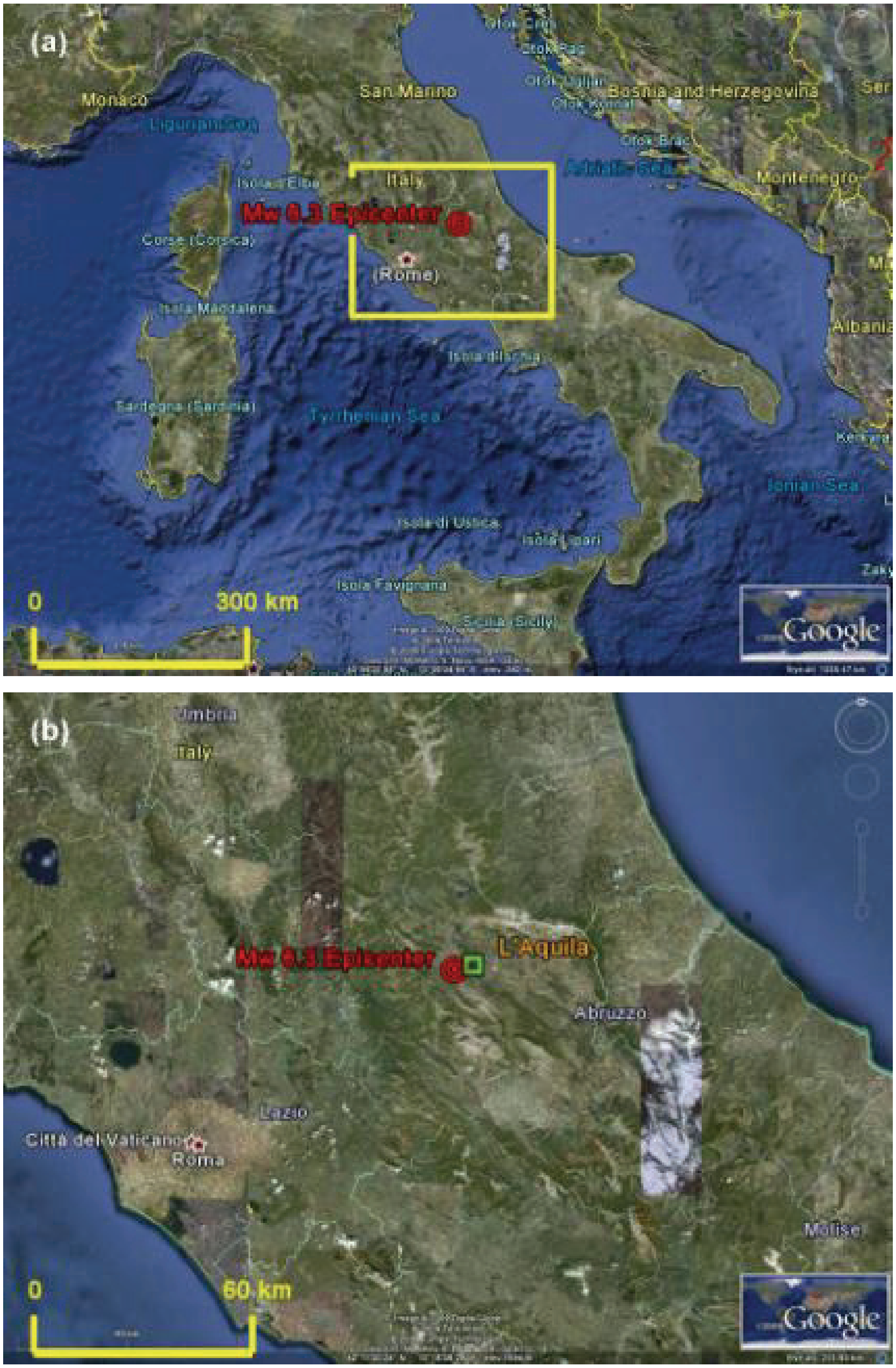


Figure 1-1 Location of the epicenter of the M_w 6.3 earthquake in Abruzzo, Italy

Figure 1-2 Towns and villages where fatalities have been reported

1.2 Seismic Sequence and Characteristics of Foreshocks and Aftershocks

The main earthquake of April 6th was preceded by more than 200 regional seismic events within the year of 2009 (INGV 2009). Most of the events were located close to the epicenter of the main shock with local magnitudes M_l up to 2.5, while the strongest ground motion occurred on March 30th (M_w 4.4) and featured a normal fault mechanism and extension in the southwest-northeast direction.

A great number of aftershocks occurred the first days after the main shock (more than 300 within 4 days – INGV 2009). In total, 7 seismic events with M_w greater or equal to 5.0 were recorded by 23 of April, 2009. The sequence of the aftershocks demonstrates that there were two principal regions of crustal rupture. The first region was the area where the main shock occurred and includes L'Aquila and the southeast part of the Aterno Valley. The second region was located north of the city of L'Aquila, close to Barete, Pizzoli and Campotosto. The fault mechanisms of the strongest aftershocks have been reported to have similar characteristics with the main shock, striking along the Apennine mountain range and dipping to the southwest (INGV 2009). Figure 1-5 illustrates the epicenters of seismic events that have occurred since December 1st, 2009, in the region hit by the earthquake. Table 1-1 lists information for the strongest events recorded before and after the main shock, while Figure 1-6 illustrates the location of the epicenters and the focal solutions of the strongest seismic events recorded.

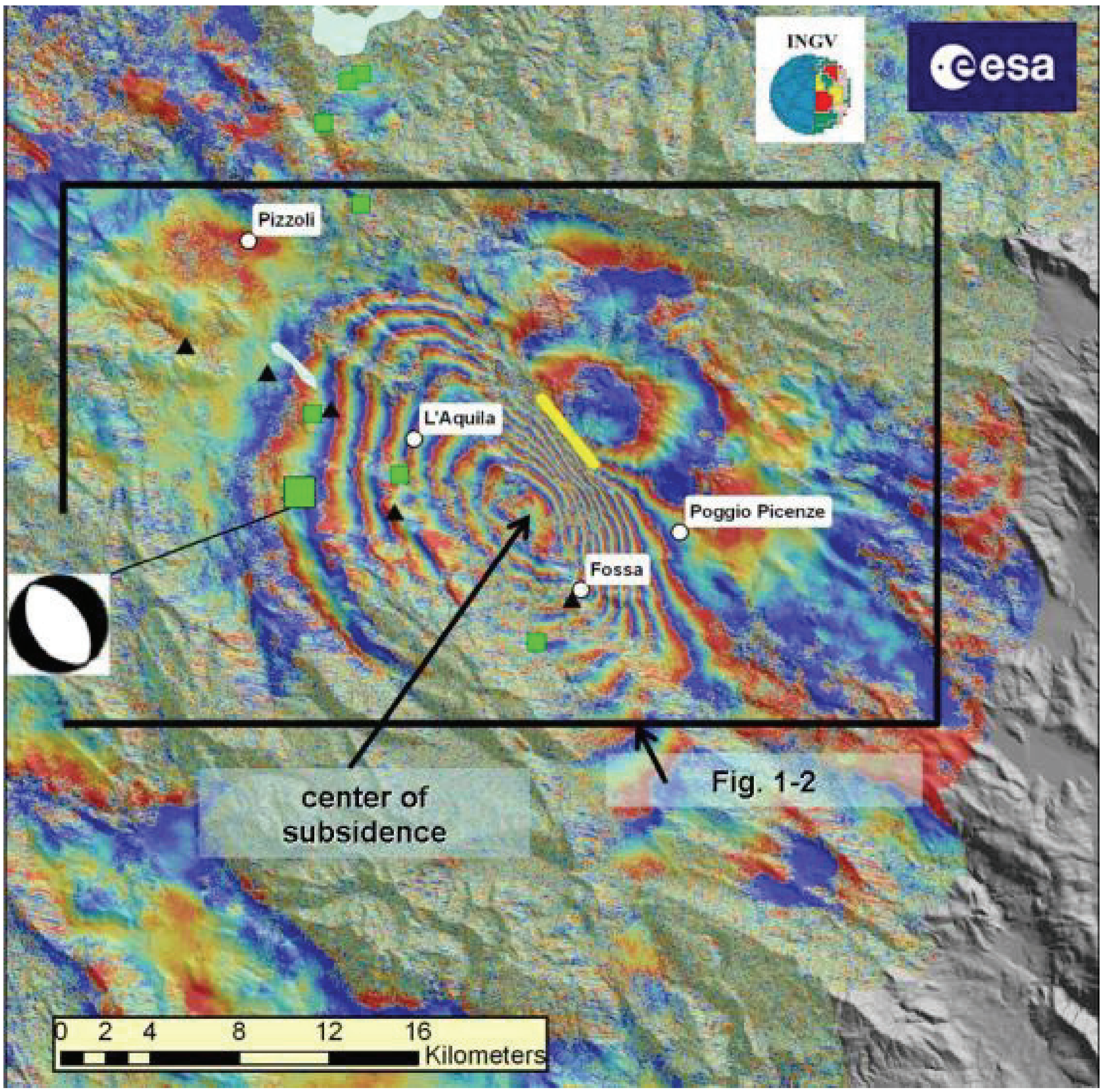


Figure 1-3 Envisat Advanced Synthetic Aperture Radar (ASAR) interferogram

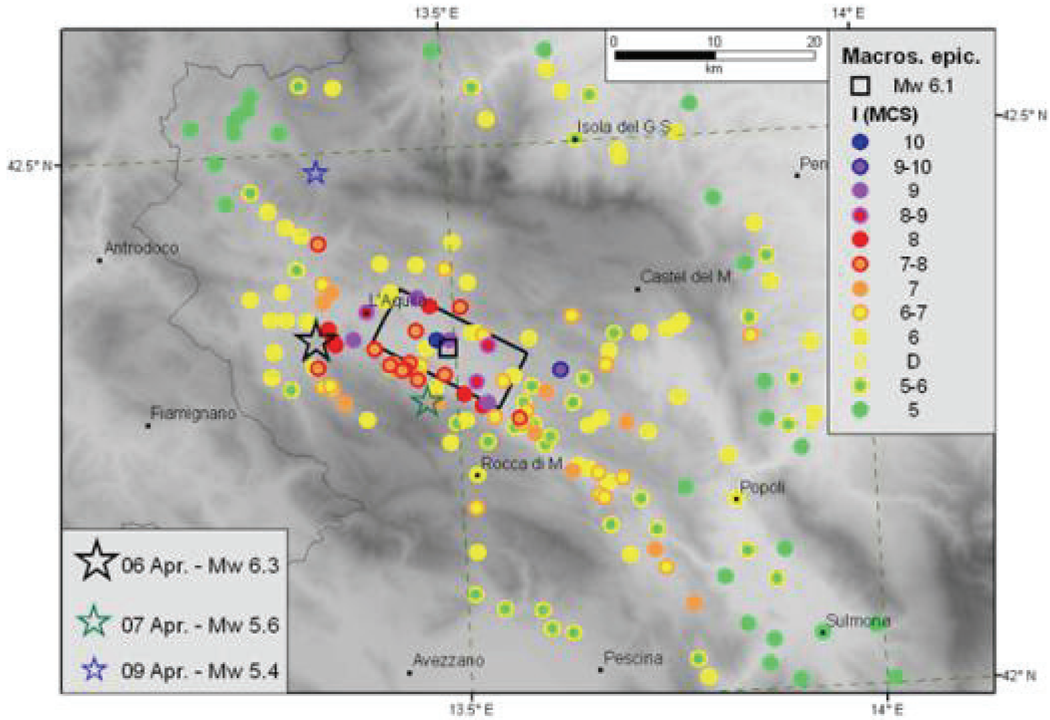


Figure 1-4 Macroseismic intensity MCS observed in the area hit by the earthquake

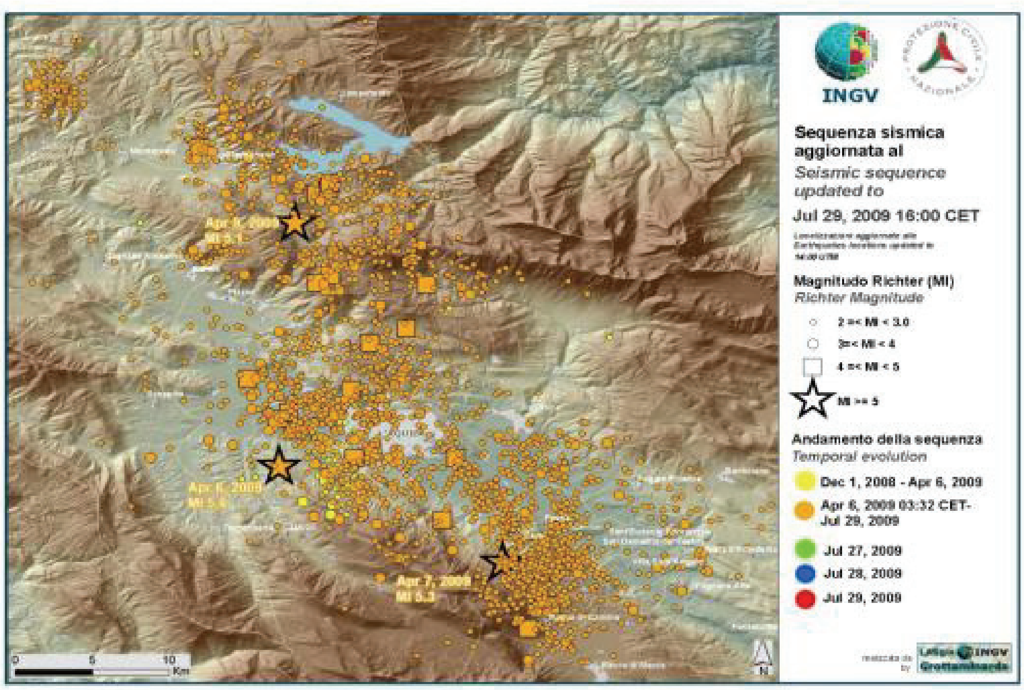


Figure 1-5 Map illustrating the epicenters of seismic events that have occurred since December 1st 2009 in the region hit by the earthquake

Table 1-1 Characteristics of Earthquakes with M_w > 4

Date	Time (UTC)	Lat. (E)	Long. (N)	Depth (km)	M_w
March 30, 2009	13:38:38	42.326	13.362	10.6	4.4
April 6, 2009	*01:32:39*	*42.334*	*13.334*	*8.8*	*6.3*
April 6, 2009	02:37:04	42.366	13.340	10.1	5.1
April 6, 2009	16:38:09	42.362	13.333	10.2	4.4
April 6, 2009	23:15:37	42.451	13.364	8.6	5.1
April 7, 2009	09:26:28	42.342	13.388	10.2	5.0
April 7, 2009	17:47:37	42.275	13.464	15.1	5.6
April 9, 2009	00:52:59	42.484	13.343	15.4	5.4

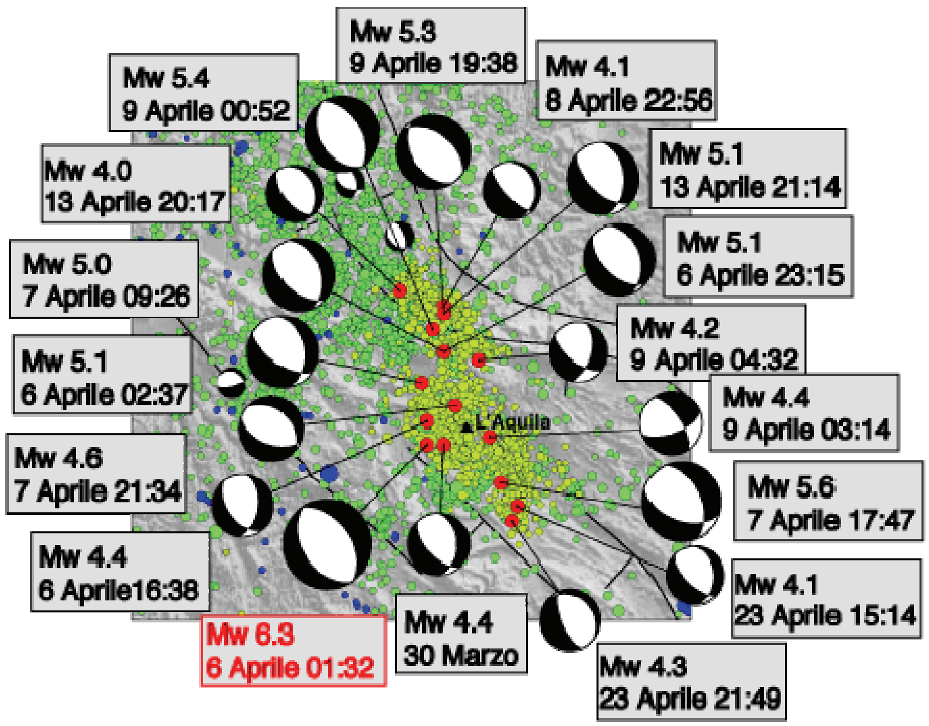


Figure 1-6 Map illustrating the location and focal solution of earthquakes with M_w > 4 (red dots). Location of minor events is illustrated by yellow dots. Blue and green dots illustrate epicenters of earthquakes occurred within the last 30 years with M_w greater and less than 4, respectively (INGV 2009)

1.3 History of the City of L'Aquila

L'Aquila is situated on the left bank of the Aterno River, at an elevation of 650 meters above the sea level, in a valley surrounded by the highest mountains of the Apennines, the Gran Sasso and the Velino-Sirente. The city, named initially Aquila meaning "Eagle" in Italian, was founded around 1240 by Holy Roman Emperor Frederick II and was renamed Aquila degli Abruzzi in 1861, and L'Aquila in 1939.

According to the local tradition, the city was actually built and populated by the inhabitants of 99 castles (villages), situated in the surrounding area. Each castle was given a piece within the city to develop the urban landscape, resulting in the city of the 99 fountains, 99 squares and 99 churches. The patron support of the villages gave growth to the development of the city and established it as an important market for the surrounding countryside. Within a few decades L'Aquila became a crossroads in communications between two extremely powerful and important trading towns, Naples and Florence. This main road connecting south and central Italy, called Via degli Abruzzi, was the safest road between Tuscany and the kingdom of Naples and Sicily. The only other route was through the Vatican States, where dangerous outlaws populated the roads (Scalabrino 2009).

The history of the city has been marked by Pietro Angeleri, a hermit from the Morrone Mountains in Abruzzo, who built in 1287 the basilica of Santa Maria di Collemaggio, a large medieval church of Romanesque and Gothic architecture. In this same church, seven years later, on August 29, 1294, Pietro da Morrone was consecrated Pope Celestine V and the new Pope declared and established an annual religious rite of forgiveness, Perdonanza Celestiniana, which is still held on 28 and 29 of August for more than 700 years. The relics of San Pietro Celestino, the patron saint of the city of L'Aquila, are kept in a mausoleum inside the basilica di Collemaggio.

The largest Renaissance church in Abruzzo, the basilica of San Benardino da Siena, was built in 1446 after the death of Bernardine from Siena, a Franciscan and one of the most effective and widely known preachers of his days, who lived in L'Aquila for a long time. The original dome of the basilica of San Bernardino was designed along the lines of the dome of Santa Maria del Fiore in Florence. In the area surrounding the city of L'Aquila, ruins of the ancient Roman city of Amiternum have been found among ancient monasteries and numerous castles, the most famous of which is Rocca Calascio; the highest castle in Italy and one of the highest in Europe.

1.4 Report Organization

This report contains nine chapters and a list of references. Chapter 1 describes the seismic sequence of the earthquake. Chapter 2 describes the tectonics and the historical seismicity in the region. Chapter 3 analyzes the ground motions histories and gives some considerations about the seismic hazard analysis. Chapters 4 and 5 describe performances of historical and monumental buildings and private residential buildings, respectively, distinguishing among material types. Several case studies for each category have been described in detail.

Chapter 6 describes infrastructural damages that are grouped in essential buildings and lifelines. Chapter 7 describes the societal impact of the earthquake.

Chapter 8 outlines the techniques, materials and design procedures in use for the shoring provisional intervention of historical buildings and monuments damaged by earthquakes. Some examples of effective and not effective shoring up interventions in historical palaces in downtown L'Aquila after the seismic event are presented. Chapter 9 describes a framework to define and measure the disaster resilience of a community using a new concept of Resilience-Based Design. Finally, Chapter 10 describes the main conclusions of this report.

SECTION 2

TECTONICS AND HISTORICAL SEISMICITY

2.1 Tectonic Setting in Central Mediterranean

Central Mediterranean is characterized by a complex tectonic and geomorphical setting, as a result of the geodynamic evolution processes that have been driven by the convergence of the Eurasian plate, at the north, and the African plate, at the south.

The convergence of the two plates initiated more than 100 million years ago (Ma), in the Late Cretaceous period of the Mesozoic era, when the ancient Tethys Ocean spanned the Mediterranean basin. By the end of the Oligocene epoch of the Cenozoic era (23 Ma), the Alpine orogeny had been developed as a product of the collision of continental margins along the north part of the Adriatic Sea, where the subduction of the stiff Adriatic (or Adria) micro-plate resulted in the elevation of the present Alps and Dinarides (Dinaric Alps), and the uplifting of plateaus in the Balkan Peninsula as well as in Corsica and Sardinia (Rosenbaum et al. 2001; Enc. Britannica 2009).

The geodynamic evolution of the Central Mediterranean, since then, is characterized by the sinking of the oceanic seafloor of the Ionian Sea and the retreat of the subduction zone (rollback[1]), originally located in Corsica and Sardinia, towards the southeast, as shown in Figure 2-1 (Gvirtzman and Nur 2001). Around 15 Ma (Early Miocene), Calabria is attached to Sardinia and Tethys Ocean exists where the Tyrrhenian Sea is today. During the Middle Miocene epoch, the eastward rollback of the vertical trench is separating Calabria from Sardinia and by the Early Pliocene epoch (3 Ma), the advance of the Calabrian Arc, with subduction ahead and extension behind, is consuming the old sea floor. Behind it, back-arc spreading[2] is creating the volcanic arc of the Eolian Islands. The separation of Calabria and Sardinia has resulted in the creation of new

[1] Rollback is defined as the migration of a subduction zone resulting generally from downward motion (sinking or gravitational collapse) of the underlying plate more rapid than the forward motion of the overriding plate. In most cases, rollback is associated with back-arc spreading.
[2] Back-arc spreading is the process by which the overriding plate in a subduction zone becomes stretched to the point of rifting, so that magma can then rise into the gap created by the rift.

oceanic crust, while the northern part of the subduction zone has collided with the Adriatic creating the north segment of the Apennine Mountains. Nowadays, the collision of the subduction zone has completed the Apennines range along the length of Italy and Calabria is wedged between Sicily and Peninsular Italy.

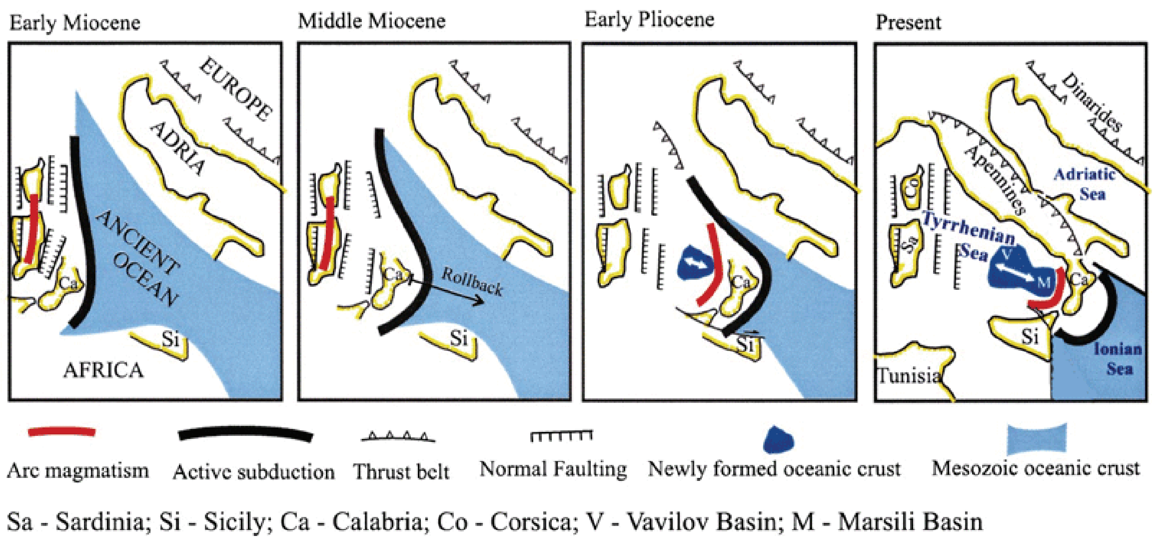


Figure 2-1 Migration of the subduction zone in Central Mediterranean (Gvirtzman and Nur 2001)

The current geodynamic model for the Central Mediterranean, illustrated in Figure 2-2 (Devoti et al. 2008), demonstrates the juxtaposition of compressional and extensional tectonic activity in the area. The Adriatic Sea itself is relatively shallow, and almost the entire ocean floor (a thick carbonitic platform underlain by continental crust) exhibits compressional deformation structures, while the crust of the overriding plate in the Tyrrhenian Sea to the southwest is extended. So although at a broad regional scale two plates are colliding, at a more local level the current back-arc spreading appears to be a major driver of tectonics in Italy, to the extent that thrust faults that built up the Apennines are now being reactivated as extensional normal faults.

Figure 2-3 shows the location of active faults across the Italian territory. In South Italy, major faults exist in the east side of Sicily and along the Calabrian arc. In Central Italy, active faults are distributed throughout the entire Apennine range, and in North Italy, active faults exist in some Alpine sectors.

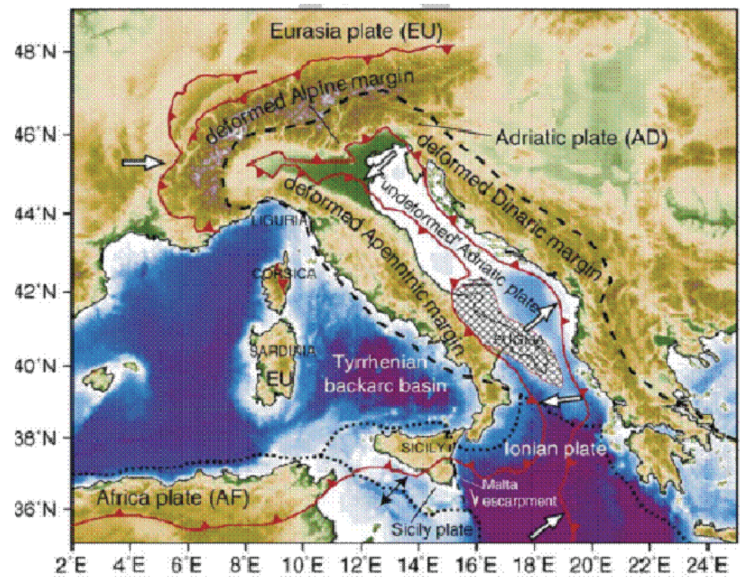


Figure 2-2 Geodynamic model of the Central Mediterranean (Devoti et al. 2008)

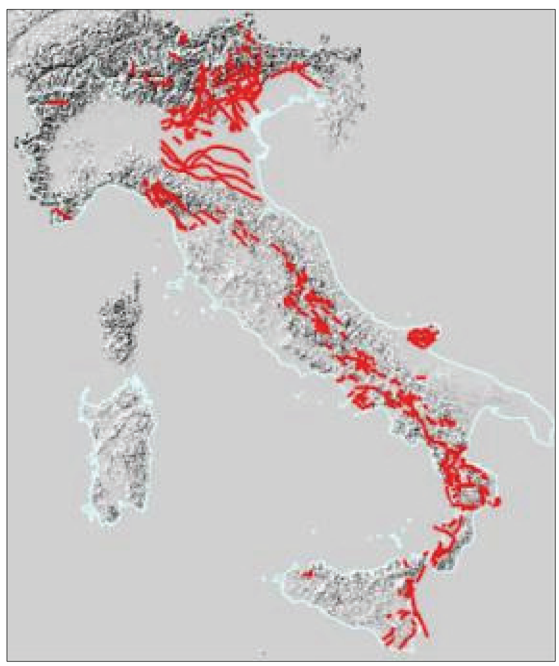


Figure 2-3 Map of the fault locations in Italy

2.2 Tectonic and Geological Setting in L'Aquila

L'Aquila is located on the path of the 800km long segmented normal fault system that accommodates the extensional deformations in the Apennines (Roberts et al. 2002). As a result, the city is in close proximity to active faults, as shown in Figure 2-4, the majority of which is oriented along the Apennine belt, in a northwest-southeast direction. Southeast of L'Aquila, the faulting system has a rather complex structure, since it comprises several overlapping segments, some of which are antithetic to the main southwest dipping fault plane (Papanikolaou et al. 2005).

Most of the central zone of the Apennine chain is formed of stiff calcareous successions of carbonate platforms and turbiditic deposits that were deposited in the foredeep basins and progressively incorporated in the chain migrating from west to east (GEER 2009). In the L'Aquila basin, the bedrock is comprised of limestone formations, Meso-Cenozoic in age, that largely outcrop along the valley flanks and on the ridges located within the Aterno Valley. In addition, Quaternary deposits are encased between carbonate platform deposits, as shown in Figure 2-5.

Downtown L’Aquila is set on a fluvial terrace, which forms the left bank of the Aterno River (De Luca et al. 2005). The elevation of the terrace at the northeast part of the city reaches 900m above sea level (asl) in the northeastern part of the city, sloping down to 675m asl in the southwest part. The terrace ends at the Aterno River which flows 50m below. The alluvial deposits that constitute the terrace are lower Quaternary in age, and are composed of breccias with limestone boulders and clasts in a marly matrix (De Luca et al. 2005). The thickness of the deposits was investigated by Blumetti et al. (2002), who found that the sediments reach their maximum thickness (around 250m) in the center of L’Aquila. In contrast, in the Aterno River valley, north of L’Aquila, the thickness of the sediments is never greater than 100m.

The local site amplification effects of the sedimentary basin, L’Aquila is built on, were investigated and quantified by De Luca et al. (2005), processing recorded data from a regional micro-seismic monitoring network and validating with 2D numerical simulations of a vertical profile of the basin. The authors demonstrate that there is a significant low-frequency (0.5-0.6 Hz) amplification of the horizontal surface motion throughout the historical city-centre.

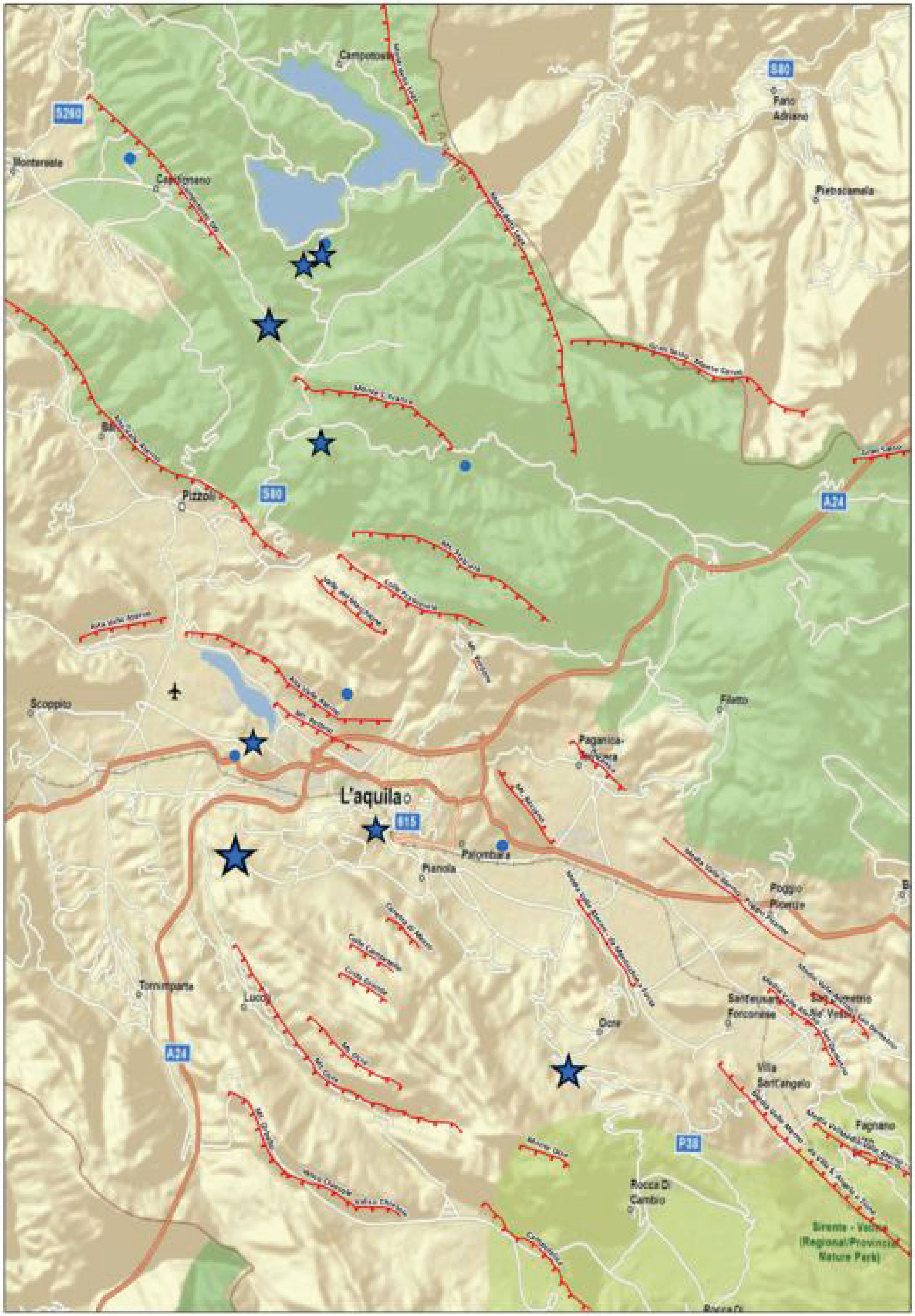


Figure 2-4 Map of active faults near L'Aquila. Features on fault lines are oriented towards the hanging wall. The stars indicate the epicenters of major events ($M_w > 4$) associated with the seismic sequence of the April 6 Earthquake (INGV 2009)

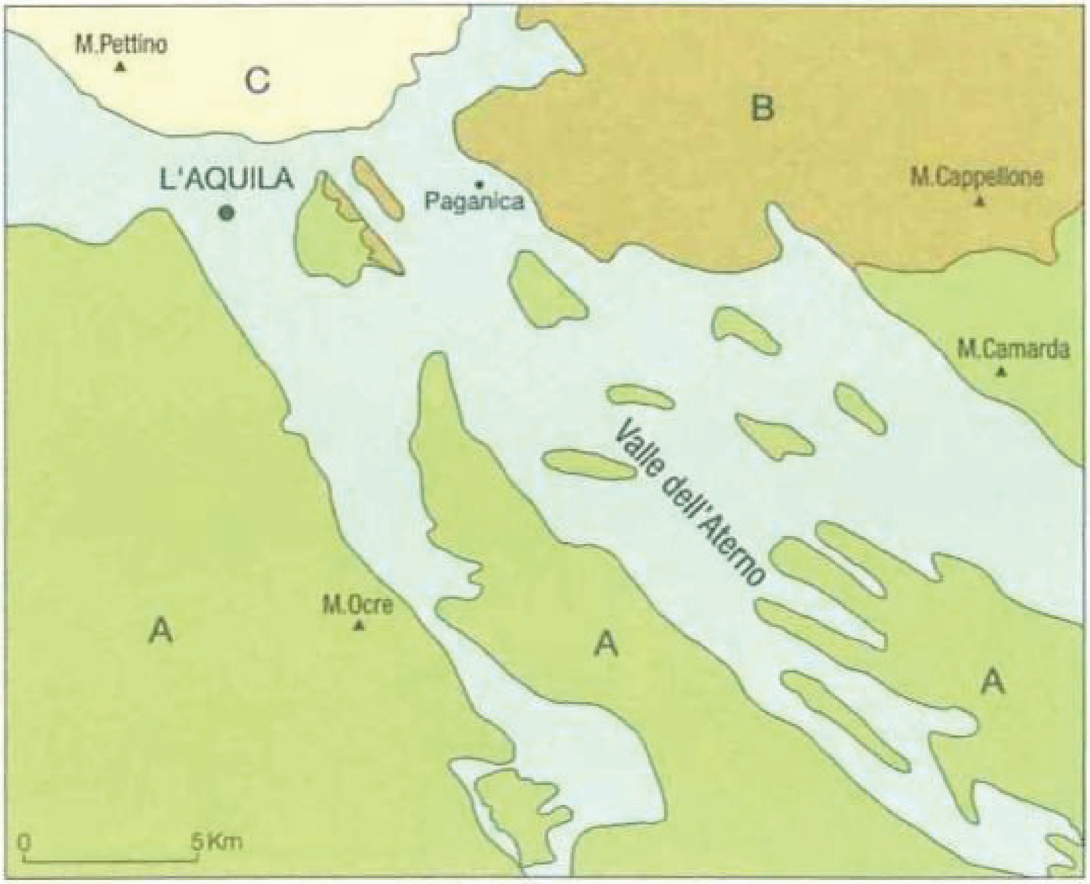


Figure 2-5 Distribution of Quaternary deposits (shown in light blue) and Meso-Cenozoic deposits (shown in different colors [A-B-C]) in the L'Aquila basin and the Aterno River Valley (from Sheet no. 359 of the Geologic Map of Italy at scale of 1:50000 - APAT 2006).

2.3 Historical Seismicity

The map of seismicity in the Central Mediterranean since 1964 (from the European-Mediterranean Seismological Centre - EMSC) illustrated in Figure 2-6 shows that Italy experiences frequent earthquakes with moderate shaking (up to magnitude 5 events). Within the Italian Peninsula earthquakes occur typically at a shallow focal depth (less than 40km) while in the volcanic arc of the Eolian Islands earthquakes initiate at much higher depths (over 150km). However, moderate-to-strong ground motions of magnitude higher than 5 have occurred during the last 45 years, as shown in Table 2-1, which summarizes information for each event.

The L'Aquila Earthquake is the third largest seismic event recorded by strong-motion instruments since 1972, after the 1976 Friuli (M_w 6.4) and the 1980 Irpinia (M_w 6.9)

Earthquakes. It is worth to note that the last two major events before 2009 occurred within Central Italy along the Apennine range (1997 Umbria-Marche Earthquake to the north and 2002 Molise Earthquake to the south of L'Aquila).

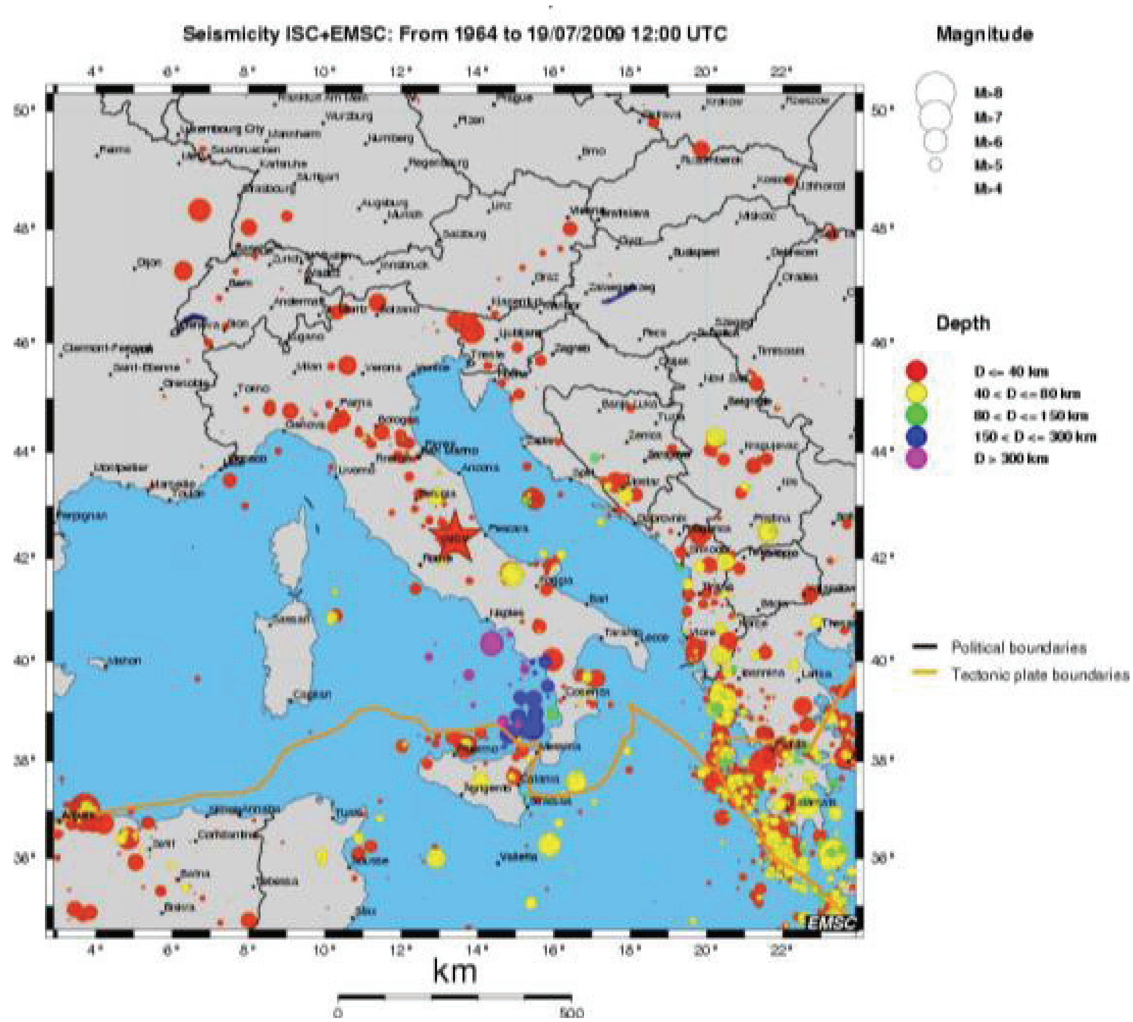


Figure 2-6 Seismicity in Central Mediterranean since 1964 (EMSC)

The regional seismicity in the Central Apennines near L'Aquila has been recently documented and archived (Gruppo di lavoro CPTI 2004) for a period starting at around 1300 AD. Figure 2-7 identifies the location and the time of occurrence of the strongest earthquakes in the region (Rovida et al. 2009). The three strongest earthquakes occurred in 1349, 1461 and 1703 resulting in epicentral MCS macroseismic intensities between IX and X.

In particular, the historical seismicity of the city of L'Aquila is shown in Figure 2-8 in terms of the MCS intensity observed in the city (Stucchi et al. 2007). The three major events mentioned earlier produced the highest shaking reaching intensity IX, while other moderate events occurred in 1315, 1452, 1501, 1646 and 1703. The earthquake of February 1703 caused devastation across Central Italy, destroying the majority of the city and resulting in about 5,000 fatalities. Finally, the event of 26 of November 1461 presents an epicenter location and a distribution of the most damaged sites (Figure 2-9) comparable to the observed damage after the L'Aquila mainshock. The area of maximum intensity was located in the proximity of the southeast part of the city.

Table 2-1 Previous Earthquakes in Italy

Date	Location	Magnitude (M_w)	Fatalities
1968, January 14	Belice in Sicily	6.4	231
1976, May 6	Gemona in Friuli	6.5	1000
1980, November 23	Irpinia in Campania	6.9	2914
1997, September 26	Umbria & Marche	5.7 and 6.0	11
2002, October 31	San Giuliano Di Puglia in Molise	5.9	29

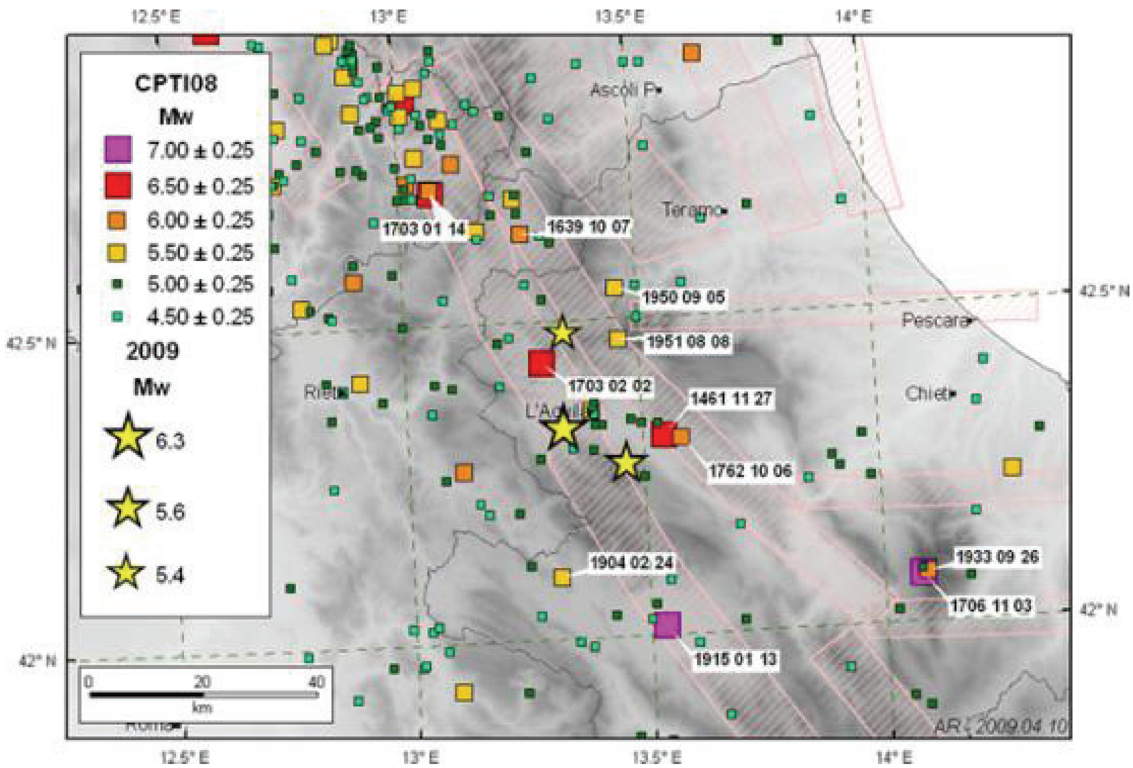


Figure 2-7 Historical seismicity of the central Apennines near L'Aquila (Rovida et al. 2009)

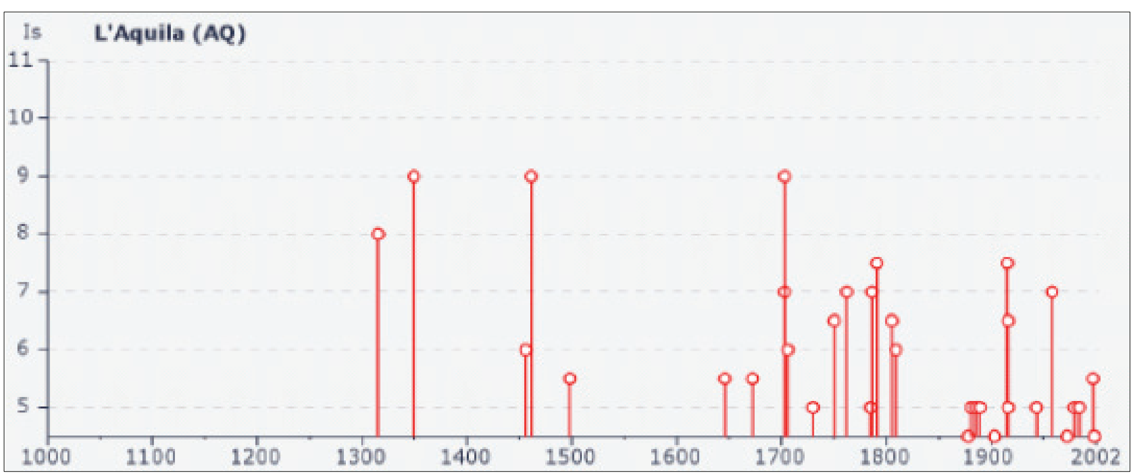


Figure 2-8 Historical macroseismic intensity observed in L'Aquila (Stucchi et al. 2007)

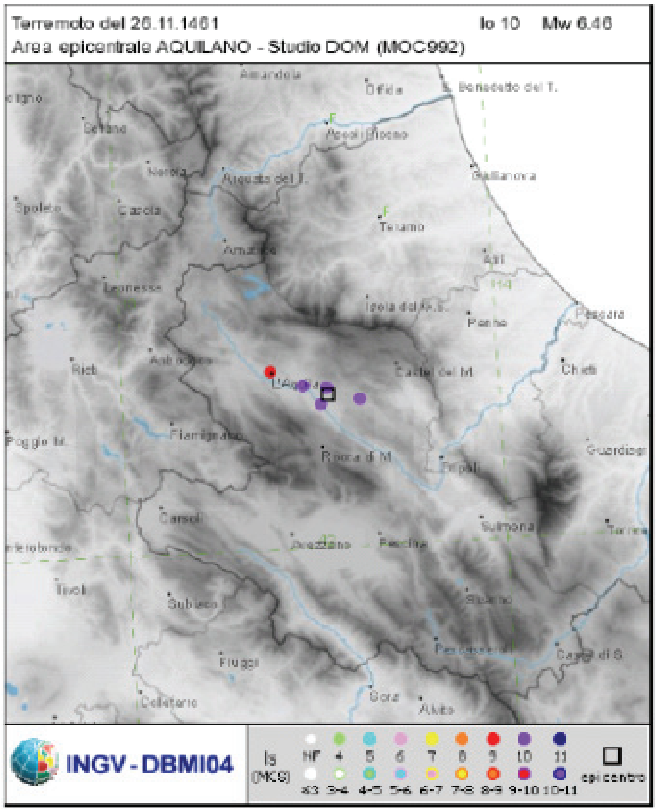


Figure 2-9 Earthquake intensity data of 1461 Earthquake (Stucchi et al. 2007)

SECTION 3

GROUND MOTIONS AND SPECTRAL ORDINATES

3.1 Recording Stations

The Italian territory is monitored by a network of 388 recording stations – RAN (Rete Accelerometrica Nazionale) network – equipped with 269 digital and 199 analog strong motion instruments, a project managed by the Italian Department of Civil Protection (DPC). The main event of L'Aquila Earthquake in April 6th (01:32 UTC) was recorded by 56 digital instruments distributed in the region of Abruzzo (14), as well as in Umbria, Marche, Lazio, Molise and Campania regions, as shown in Figure 3-1.

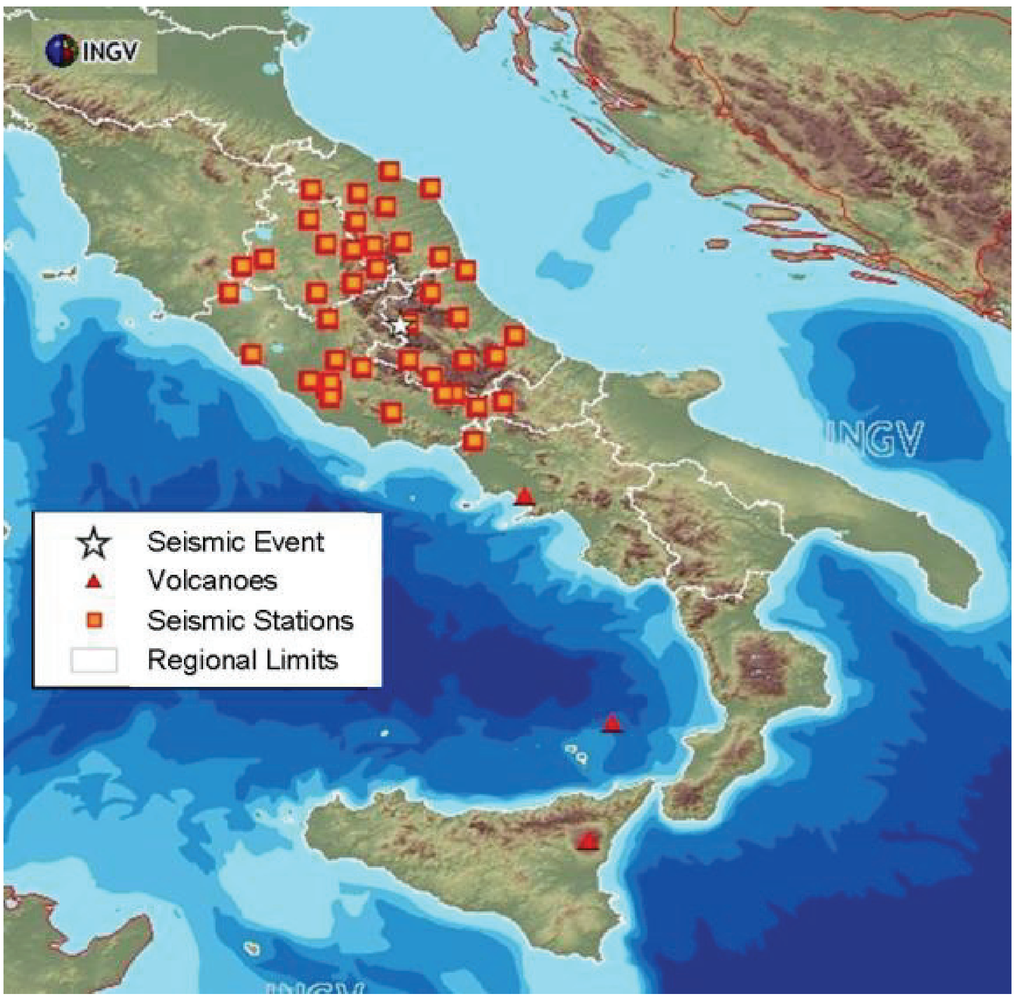


Figure 3-1 Map indicating the seismic stations that recorded the event of April 6th

The recorded data have been processed and metadata with ground motion time-histories and spectral ordinates have been available through ITACA (2009), the Italian Accelerometric Archive (http://itaca.mi.ingv.it/ItacaNet/).

Out of the 14 stations in Abruzzo, the ground motion and spectral characteristics of 9 recordings are presented in this report; those that contain peak ground horizontal accelerations in any of the two components greater than 0.05g. Table 3-1 lists information on the geographical location of each station, the peak ground horizontal acceleration recorded, and the associated epicentral distance. Figure 3-2 shows the locations of the 9 stations with respect to the epicenter of the main event and the city of L'Aquila.

Table 3-1 Information on Recording Stations on April 6, 2009

Station ID	Record ID	Station Name	Lat. (E)	Lon. (N)	Altitude (m)	Site Class EC8**	PGA* (g)	R_{epi} (km)
AQV	GX066	L'AQUILA – V. ATERNO – CENTRO VALLE	42.377	13.344	692	B	0.659	4.9
AQG	FA030	L'AQUILA – V. ATERNO – COLLE GRILLI	42.373	13.337	721	A	0.517	4.4
AQA	CU104	L'AQUILA – V. ATERNO – F. ATERNO	42.376	13.339	693	B	0.444	4.6
AQK	AM043	L'AQUILA – V. ATERNO – AQUIL PARK ING.	42.345	13.401	726	B	0.354	5.6
GSA	EF021	GRAN SASSO (ASSERGI)	42.421	13.519	1,062	A	0.151	18.0
CLN	TK003	CELANO	42.085	13.521	803	A	0.091	31.6
AVZ	BI016	AVEZZANO	42.027	13.426	746	C	0.069	34.9
ORC	CR008	ORTUCCHIO	41.954	13.642	732	A	0.066	49.3
MTR	BY048	MONTEREALE	42.524	13.245	975	A	0.063	22.4

* defined as the absolute maximum among the two horizontal components

** Classification information derived from Ameri et al. 2009

Four of the seismic stations are located on the hanging wall of the fault, within about 5 km epicentral distance, and recorded the strongest ground motions. Three of these stations are sited northwest of the city near Coppito with one reference rock station (AQG – site class A in Table 3-1) and two stations on recent alluvium (AQA and AQV – site class B). The fourth station is sited near the historic center of the city of L'Aquila on Pleistocene material (AQK). The remaining stations are located within epicentral distances from 18 to 50 km, mainly on rock (site class A), except for AVZ station (located on soft soil – site class C).

3.2 Ground Motion Histories and Spectral Ordinates

Table 3-2 summarizes the peak ground motion ordinates associated with each station, for each of the three principal directions. Additionally to the epicentral distance R_{epi}, the Joyner-Boore distance R_{JB} (Joyner and Boore 1981) is provided as a measure of the shortest horizontal distance between the site and the vertical projection of the fault to the surface (fault trace in Fig. 3-2).

Table 3-2 Peak Ground Motion Ordinates L'Aquila, April 6, 2009

Station-Record ID	Peak Ground Acc. (g)			Peak Ground Vel. (m/sec)			Peak Ground Dis. (m)			R_{JB}* (km)	R_{epi} (km)
	EW	NS	UP	EW	NS	UP	EW	NS	UP		
AQV – GX066	0.659	0.546	0.522	0.404	0.428	0.125	0.068	0.034	0.025	0.1	4.9
AQG – FA030	0.475	0.517	0.243	0.312	0.355	0.104	0.060	0.041	0.020	0.1	4.4
AQA – CU104	0.404	0.444	0.470	0.320	0.268	0.094	0.054	0.038	0.018	0.1	4.6
AQK – AM043	0.334	0.354	0.372	0.324	0.362	0.198	0.076	0.126	0.040	0.1	5.6
GSA – EF021	0.151	0.146	0.109	0.098	0.072	0.045	0.025	0.018	0.018	8.6	18.0
CLN – TK003	0.082	0.091	0.045	0.049	0.066	0.071	0.029	0.017	0.019	20.0	31.6
AVZ – BI016	0.056	0.069	0.027	0.108	0.113	0.037	0.039	0.034	0.017	25.1	34.9
ORC – CR008	0.065	0.041	0.031	0.059	0.033	0.037	0.014	0.009	0.008	37.3	49.3
MTR – BY048	0.044	0.063	0.023	0.035	0.029	0.033	0.008	0.006	0.009	15.9	22.4

* Information derived from Ameri et al. 2009

The four stations on the hanging wall of the fault, with R_{JB} practically zero, have recorded peak horizontal ground accelerations that range between 0.35g and 0.65g. Station AQV recorded the highest ground accelerations and velocities in all three directions. Station AQG, located close to AQV but sited on rock, recorded slightly lower horizontal components of acceleration but significantly lower vertical component. Station AQK in the city-centre recorded the highest ground displacements, reaching up to 12.6 cm.

Station GSA, located northeast on Gran Sasso 8.6 km from the fault trace, recorded peak horizontal accelerations of 0.15g, while the remaining stations with JB distances between 20 and 40 km recorded lower values between 0.05g and 0.09g. Along the striking orientation peak magnitudes attenuated more rapidly with distance in the northwest direction (Station MTR) than in the southeast direction (Stations CLN, AVZ and ORC).

Figure 3-3 presents the elastic acceleration response spectra for the horizontal direction, computed as the geometric mean of the spectra of the two horizontal components. Figure 3-4 illustrates the elastic acceleration response spectra for the vertical direction. All spectra are considered for 5% damping ratio. Figures 3-5 up to 3-13 feature the acceleration time-history records and the associated spectra for each station.

The horizontal spectra shown in Fig. 3-3 indicate that mean spectral accelerations in the short period range well exceeded 0.5g for records of sites located on the hanging wall. The horizontal spectra of AQV and AQA reach spectral values of 1.4g and both contain peaks in the periods of ~0.1sec and 0.4sec, as shown in Figs. 3-5 and 3-7. The spectra of the reference rock station AQG is more uniformly distributed with values around 0.8g for periods up to 0.4sec but contains higher spectral ordinates at 1sec and a peak at 0.8sec that is not identified in AQV and AQA (Fig. 3-6). The horizontal spectrum of AQK has a peak of 0.9g at 0.15sec and ranges mostly below the other three spectra in periods below 0.8sec. On the contrary, at 2sec the spectrum has the highest spectral value among all records by a factor greater than 2. This is associated with the site-amplification in the city of L'Aquila, as discussed earlier in Section 2.2, at periods around 2sec (study by De Luca et al. 2005). The horizontal spectrum of GSA contains spectral values above 0.2g for periods up to 0.7sec while the other spectra range below 0.2g.

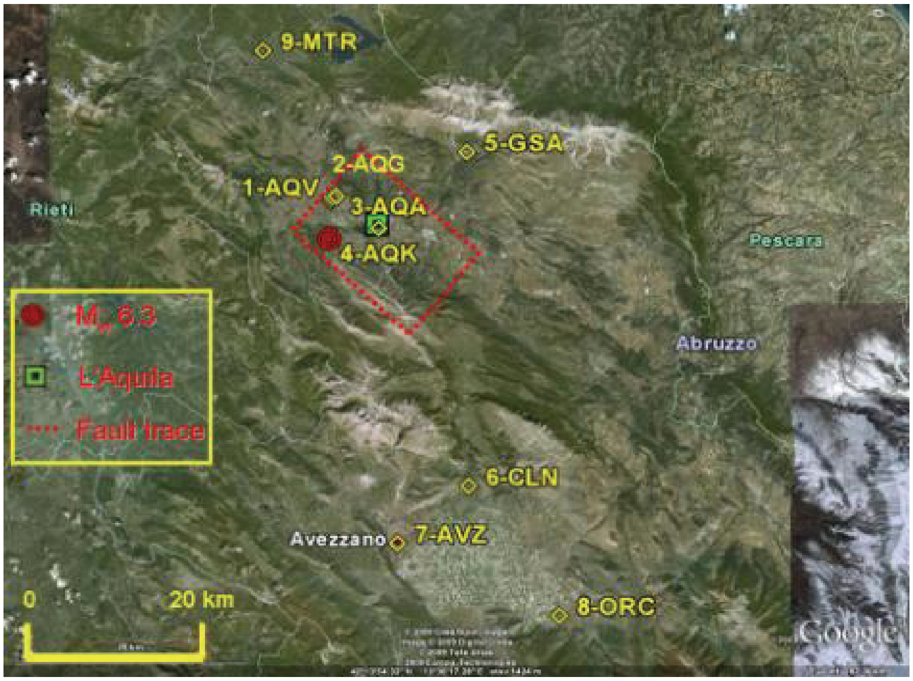


Figure 3-2 Map indicating the 9 seismic stations considered in the report

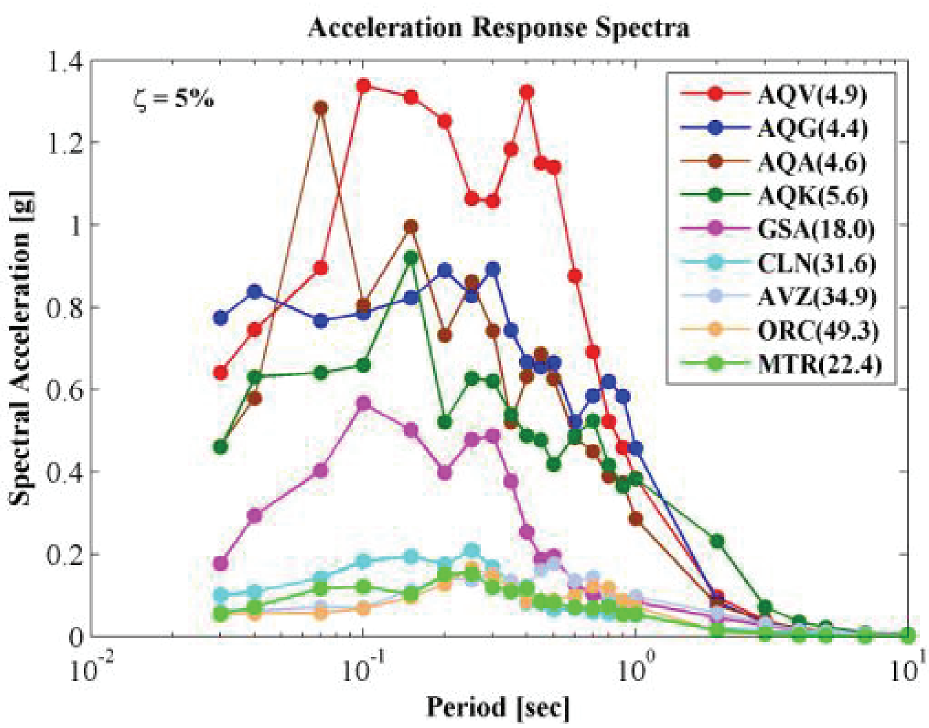


Figure 3-3 Horizontal acceleration elastic response spectra of 9 records from the main event of April 6th. Spectral ordinates for each record computed as the geometric mean of the spectra of the two horizontal components

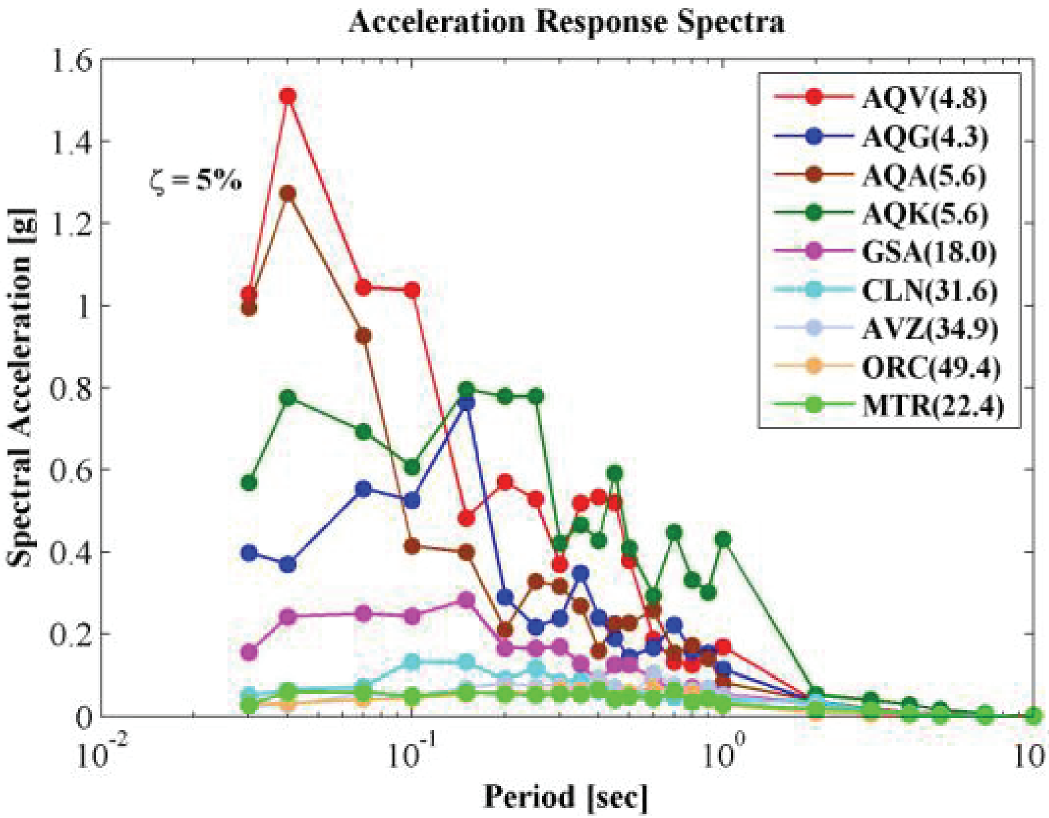


Figure 3-4 Vertical acceleration elastic response spectra of 9 records from the main event of April 6th

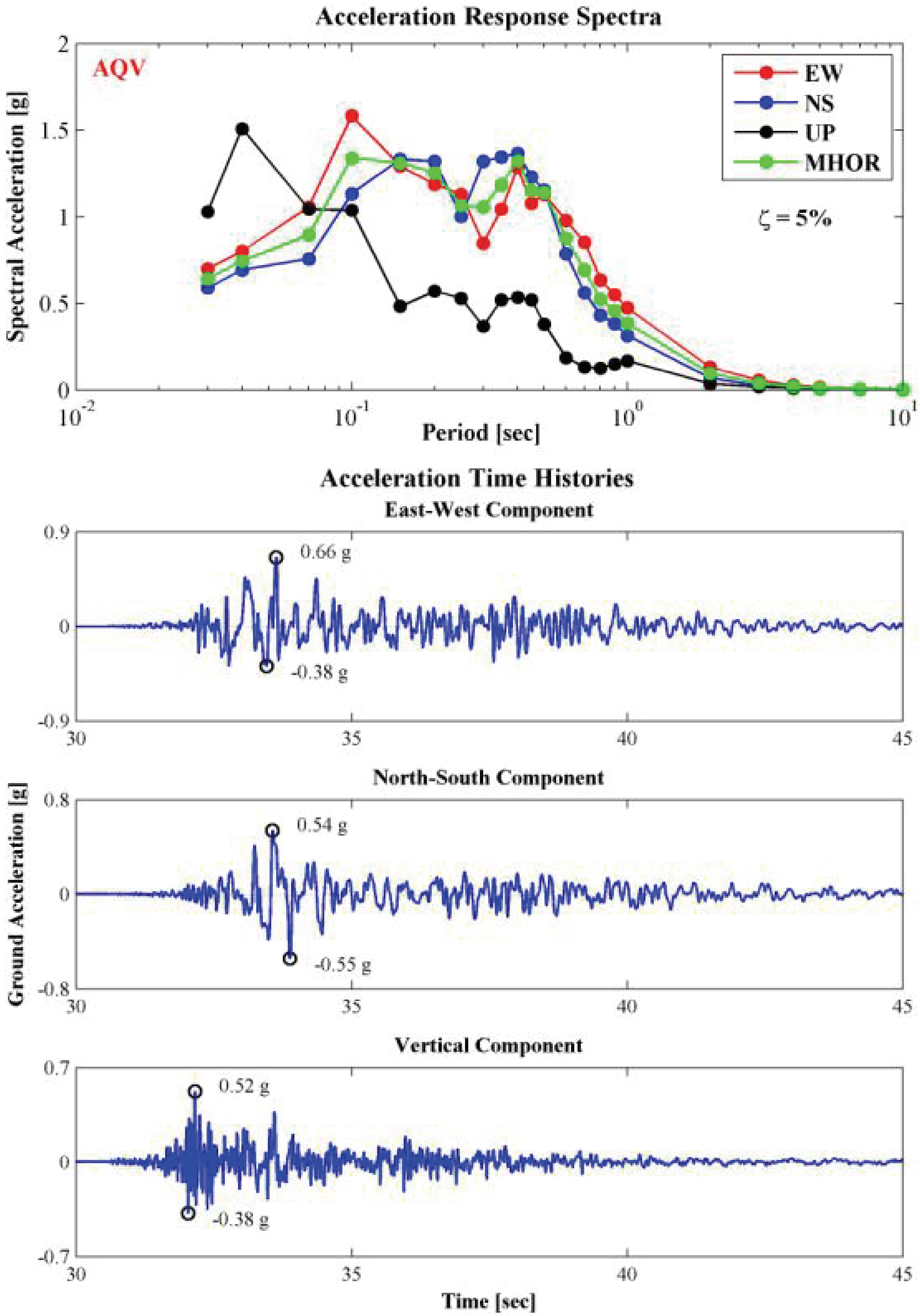


Figure 3-5 Ground motion histories and spectral ordinates for AQV-GX066

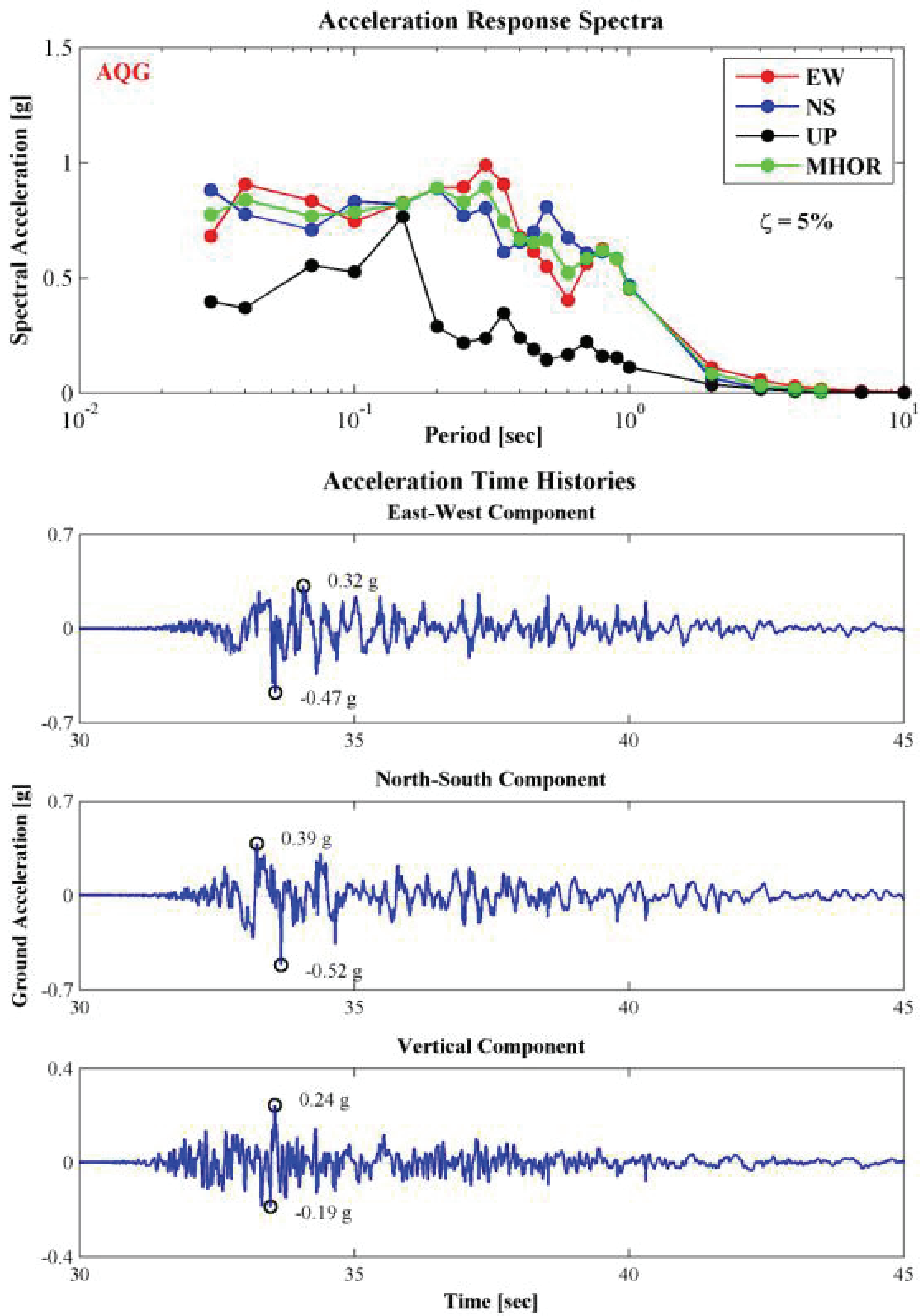


Figure 3-6 Ground motion histories and spectral ordinates for AQG-FA030

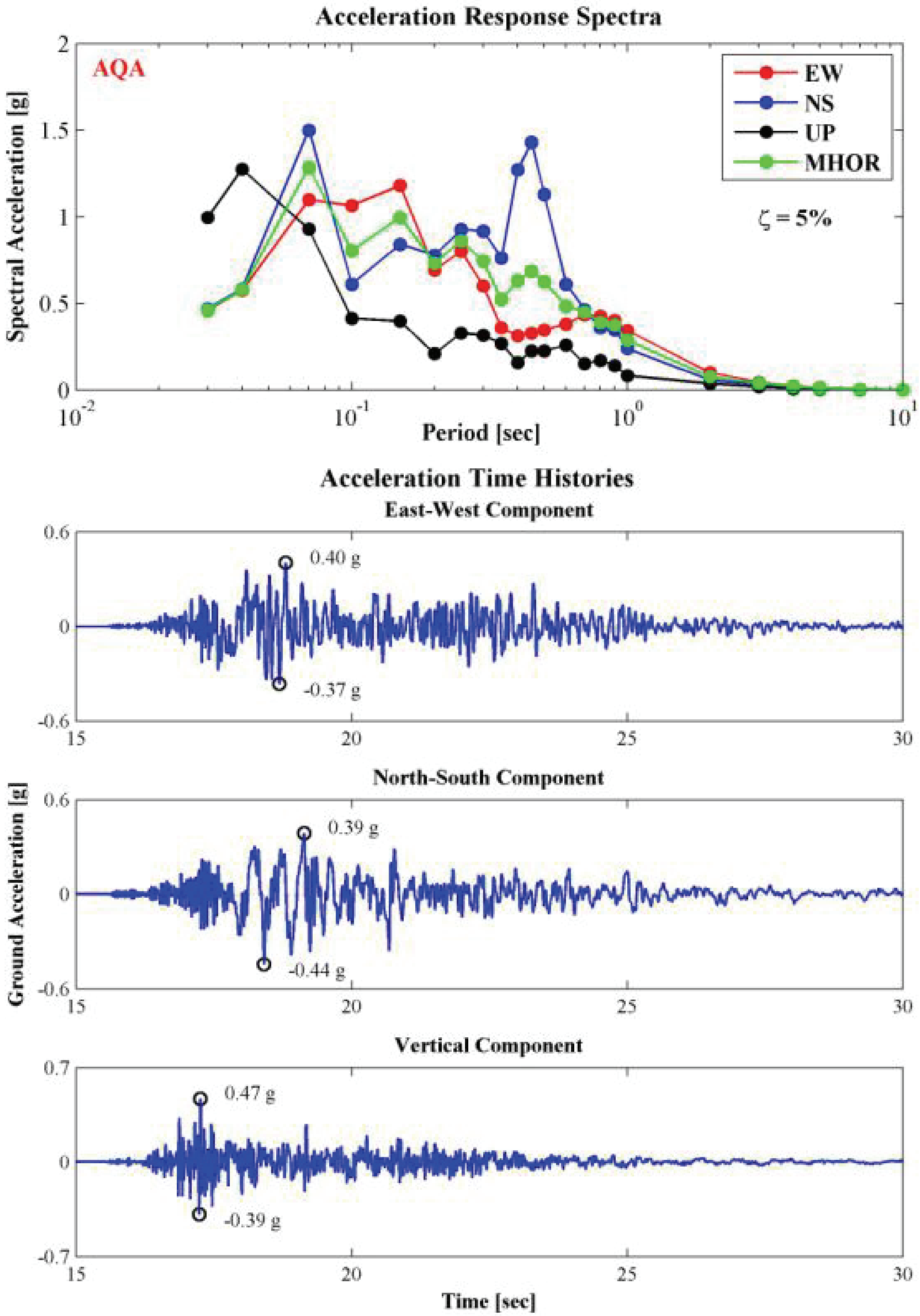


Figure 3-7 Ground motion histories and spectral ordinates for AQA-CU104

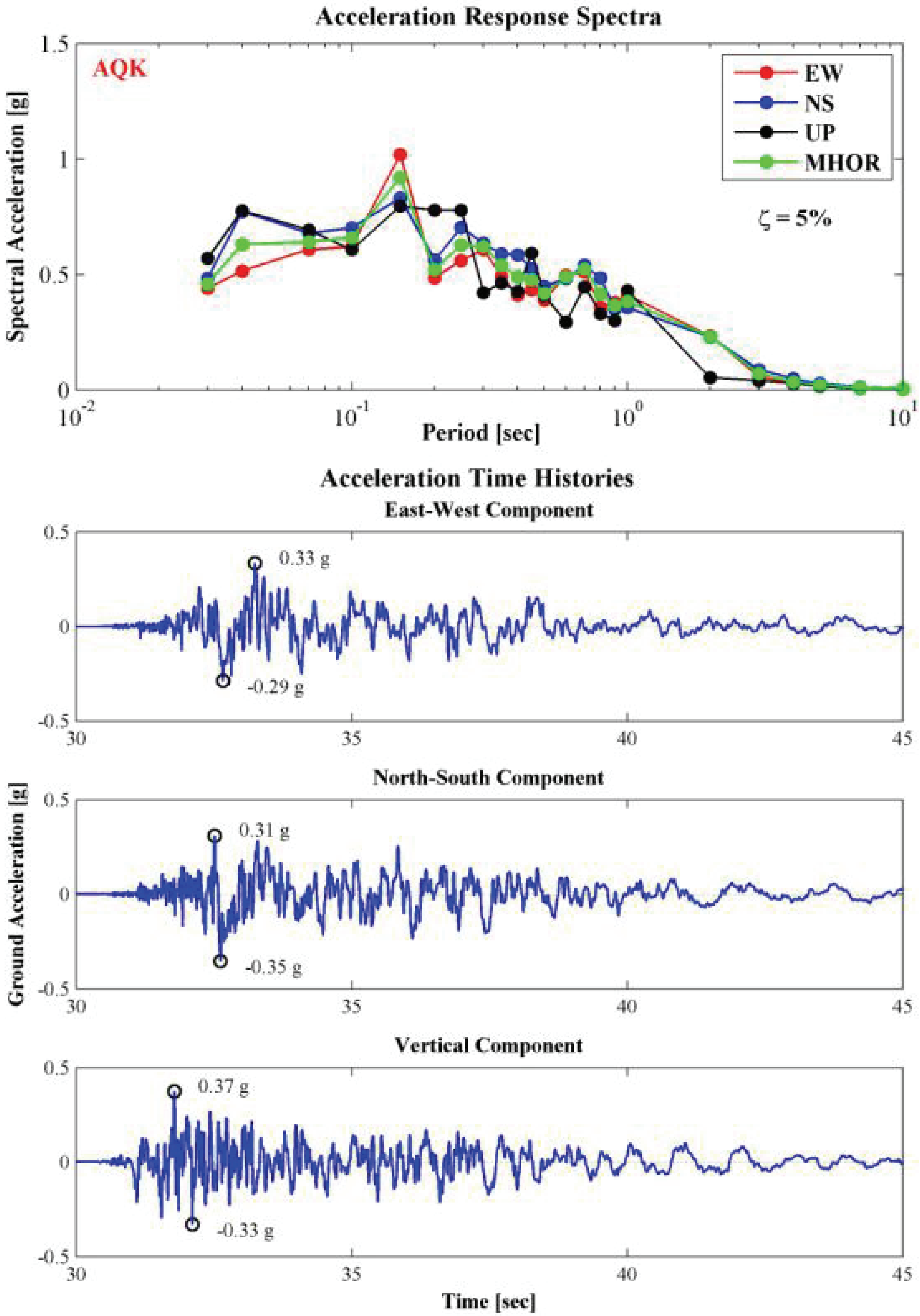


Figure 3-8 Ground motion histories and spectral ordinates for AQK-AM043

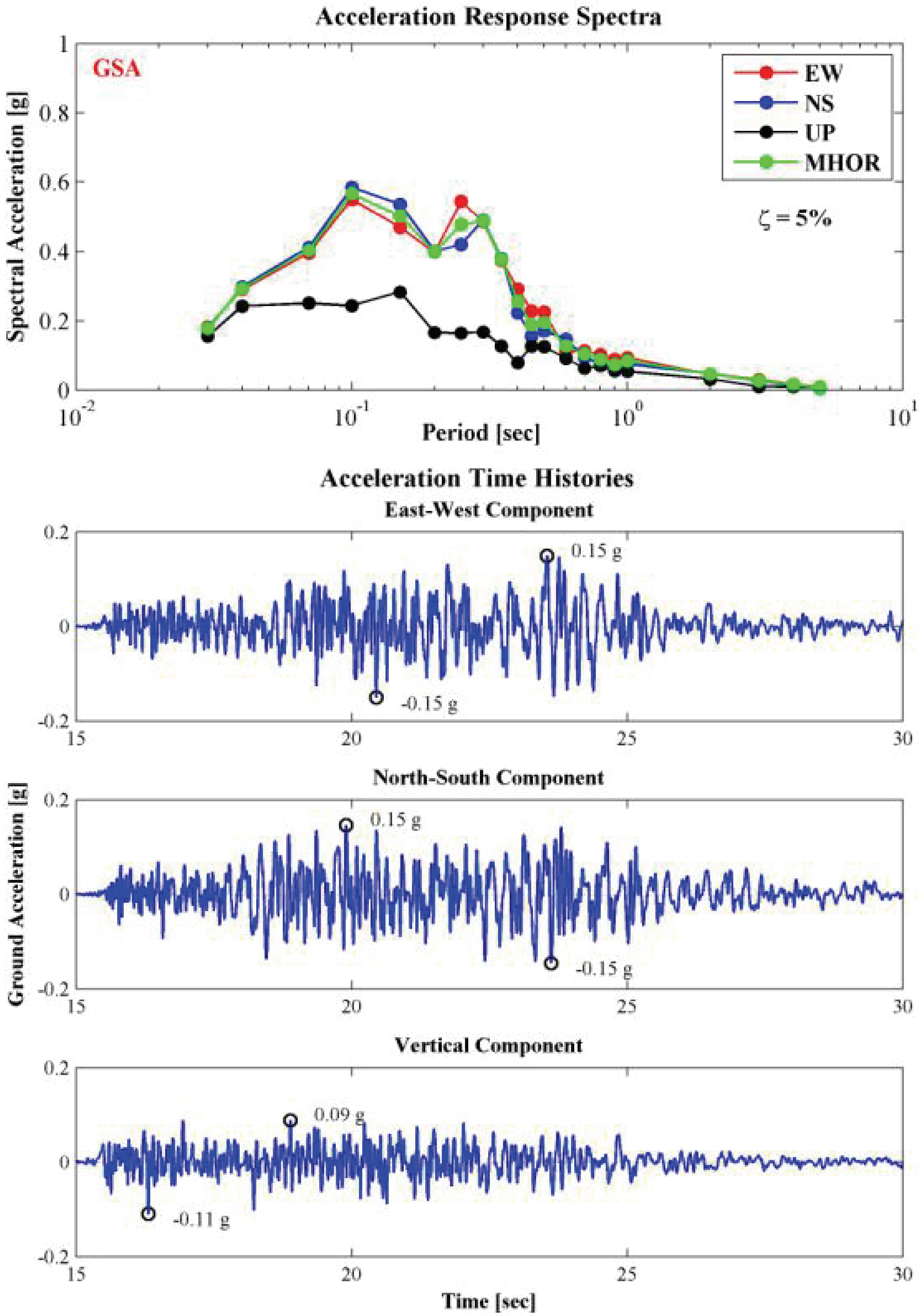


Figure 3-9 Ground motion histories and spectral ordinates for GSA-EF021

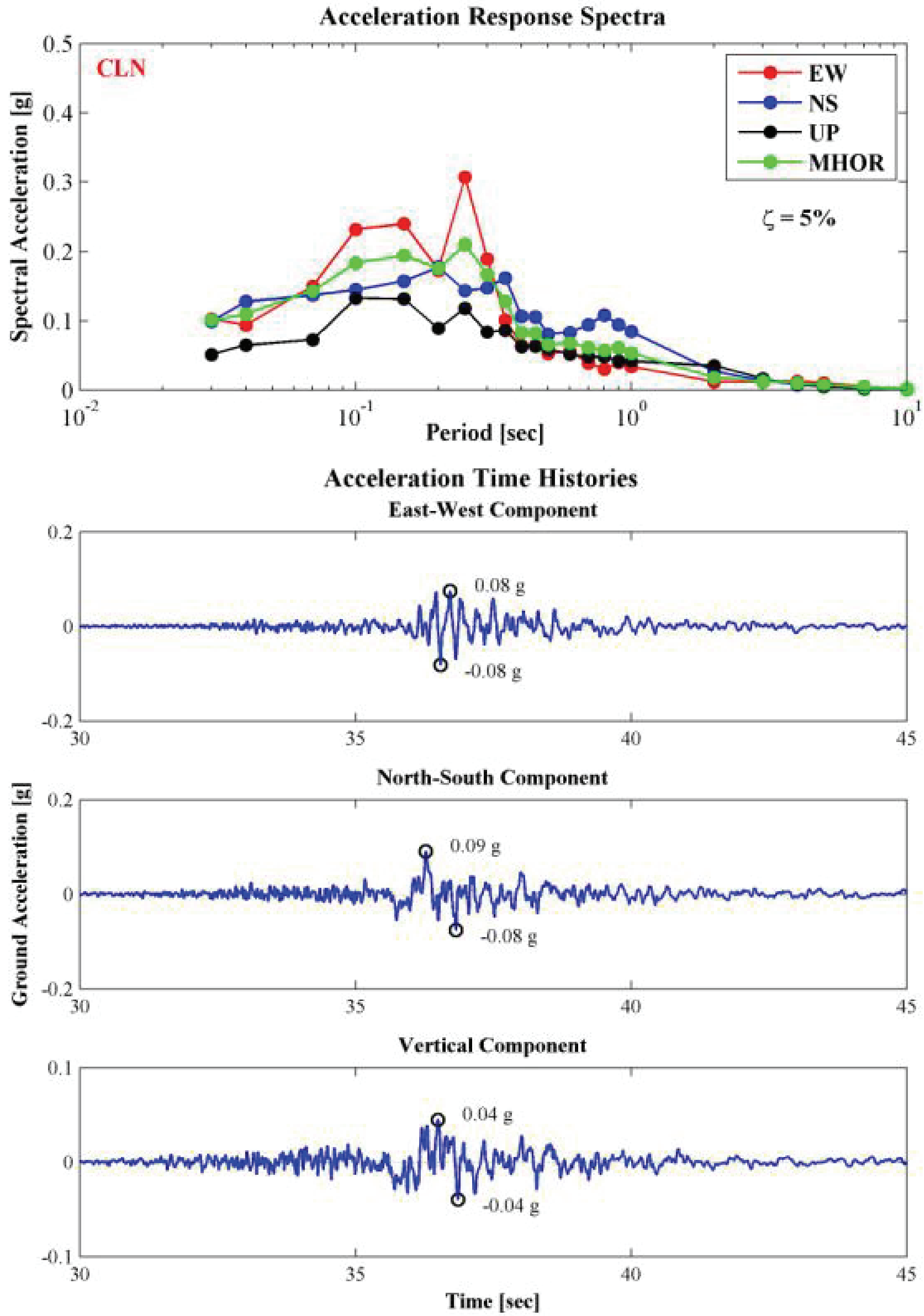


Figure 3-10 Ground motion histories and spectral ordinates for CLN-TK003

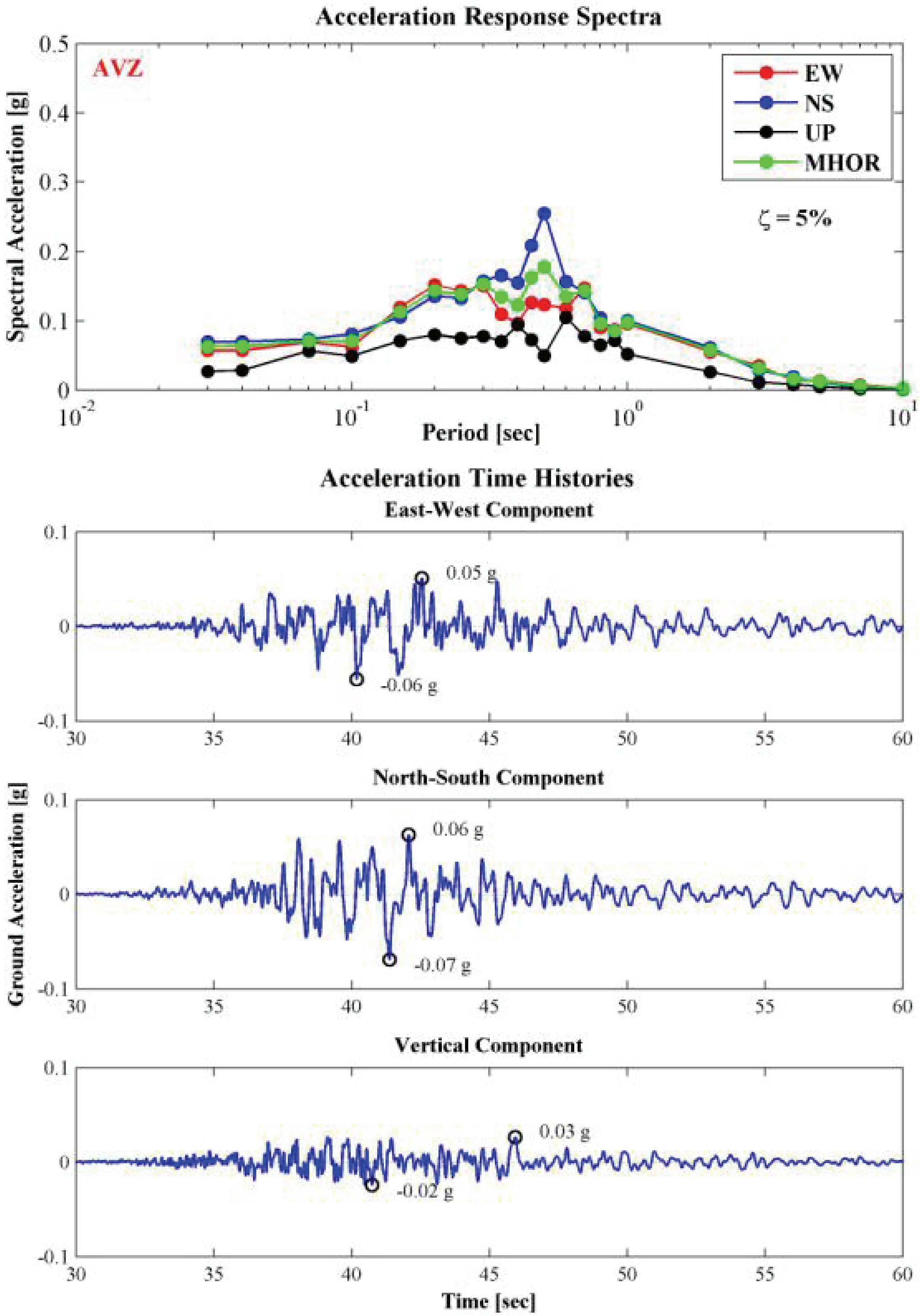


Figure 3-11 Ground motion histories and spectral ordinates for AVZ-BI016

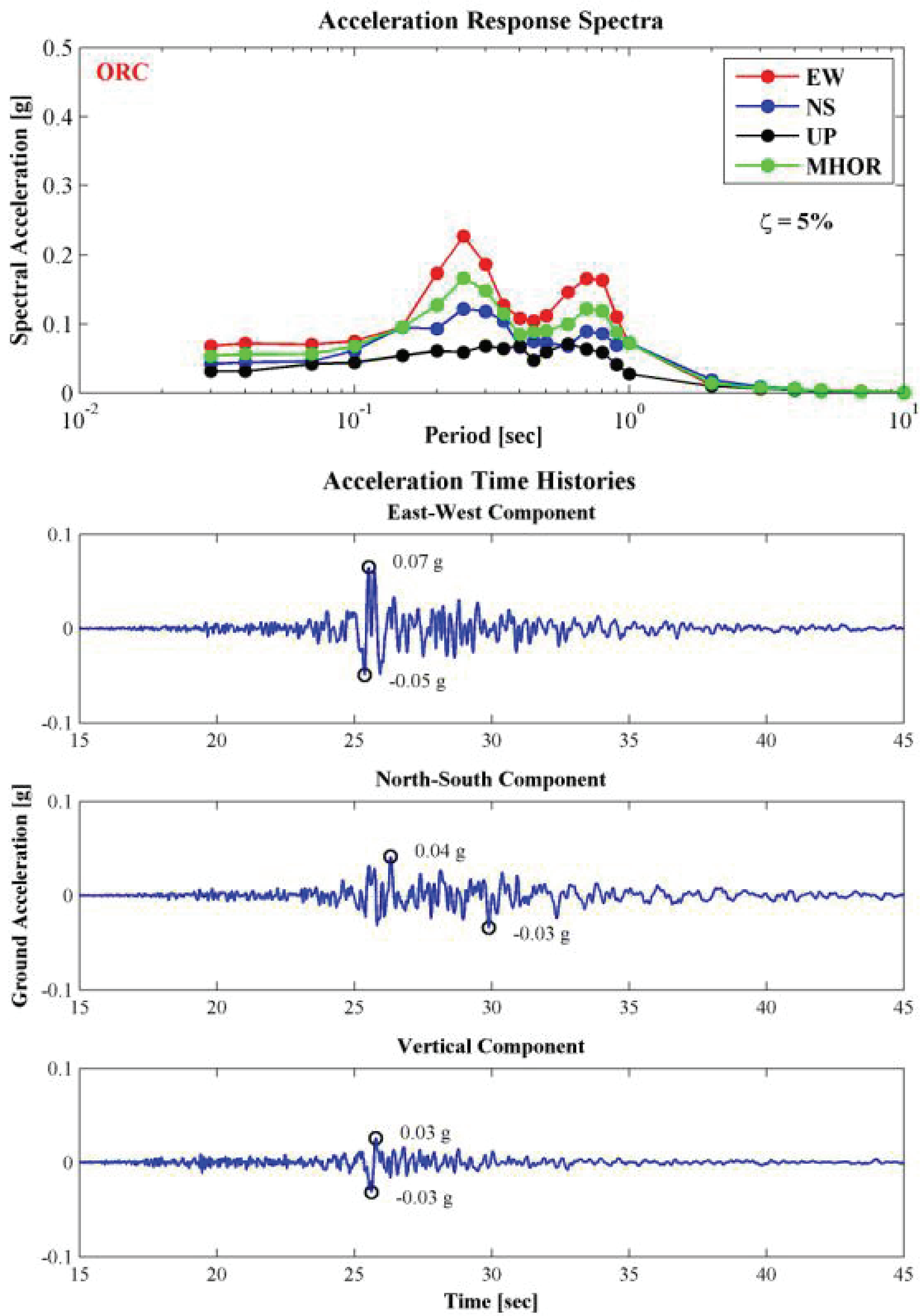


Figure 3-12 Ground motion histories and spectral ordinates for ORC-CR008

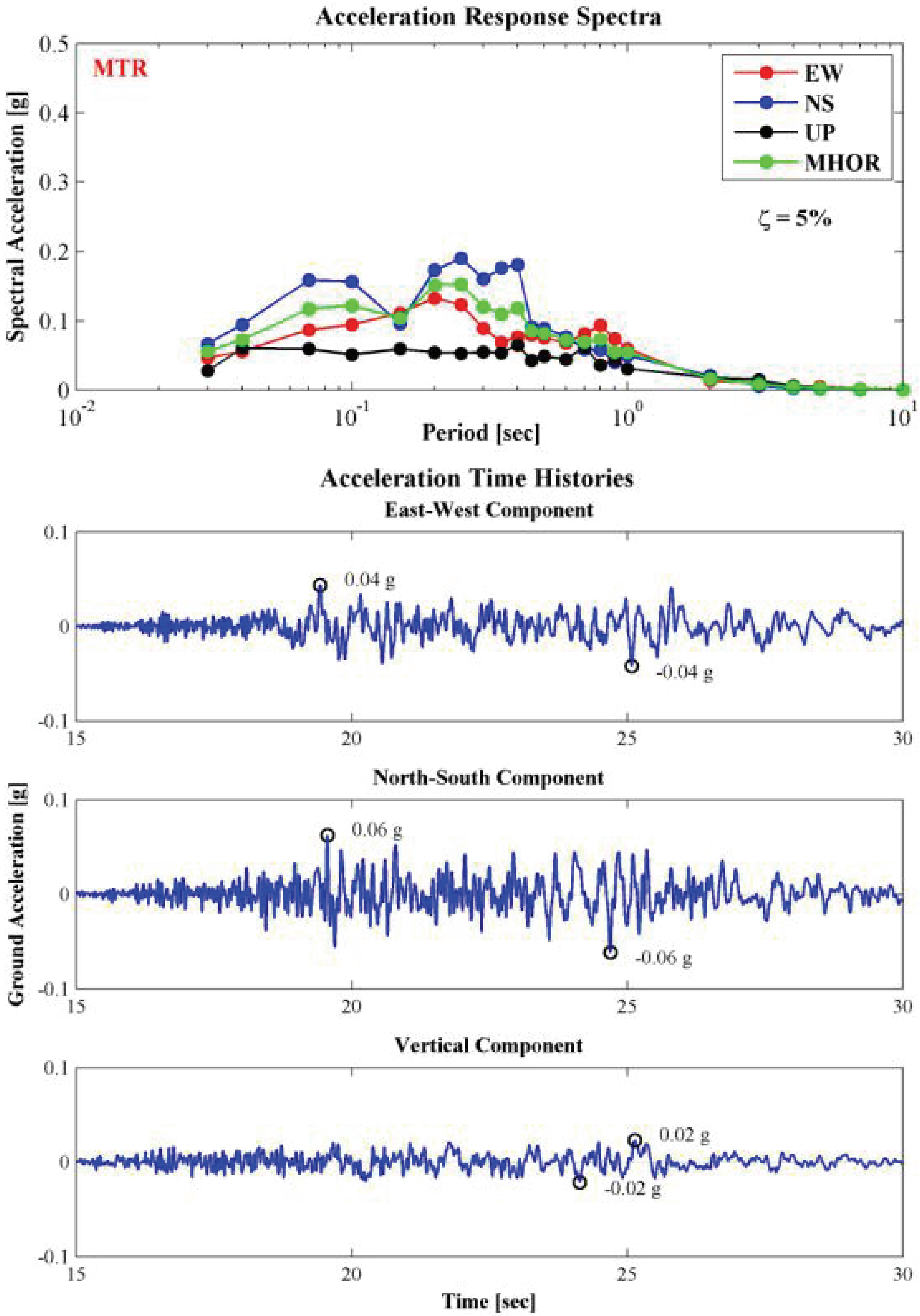


Figure 3-13 Ground motion histories and spectral ordinates for MTR-BY048

3.3 Implications for Seismic Hazard Analysis

After the Messina earthquake in 1908, the Italian government imposed certain regulations for new buildings in seismic regions. Since then, in Italy after each earthquake there is a new code for seismic design. L'Aquila was assessed as a "seismic region" since the Fucino earthquake in 1915 and was classified in the second category of the seismic zones, introduced in 1927.

The main modifications to the Italian seismic standard initiated after 1980 Irpinia earthquake, when the first seismic zonation (Figure 3-14) in Italy was introduced, and concluded in 2003 with the OPCM 3274/2003 seismic code.

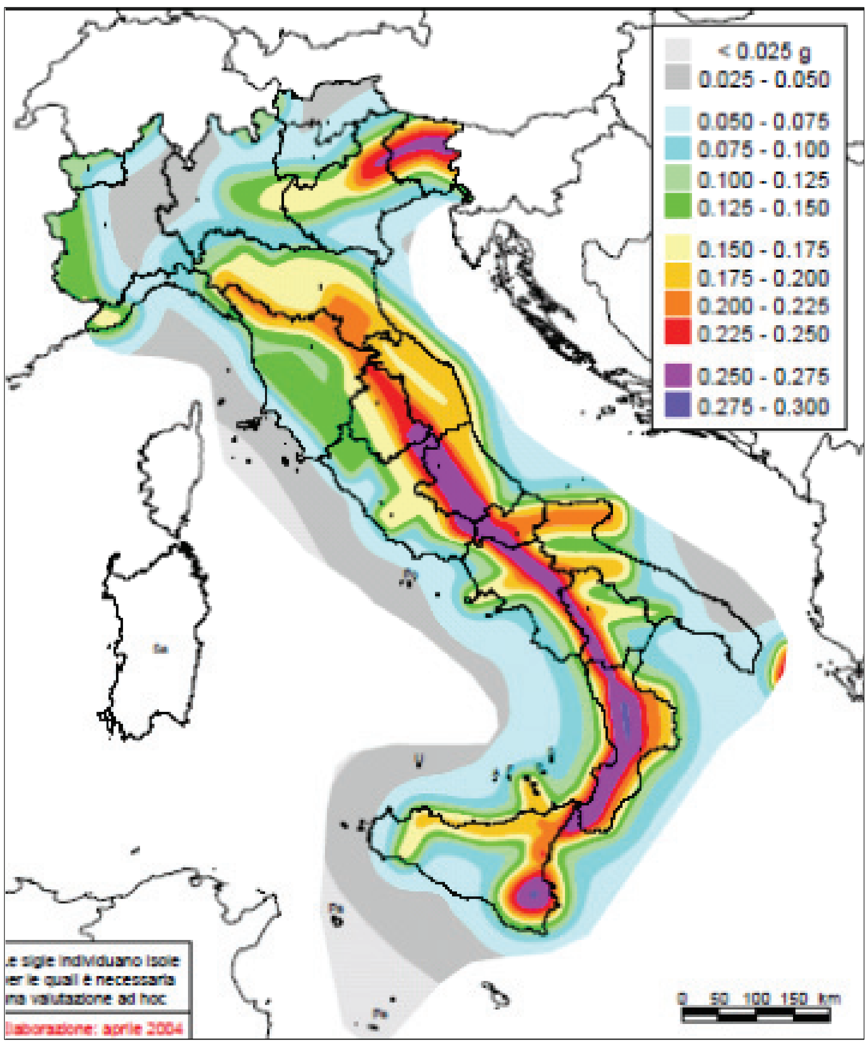


Figure 3-14 Seismic hazard map of Italy

Almost the entire Italian territory was divided in 4 zones (e.g., in the first category, severe earthquakes have the highest probability to appear). L'Aquila was classified in Zone 2 with the latest revised seismic zonation. Today the seismic regions in Italy cover more than 70% of the territory, but only 18% of the buildings respect the seismic code.

The most recent seismic code was released on January 2008 (CS LL PP, 2008), but had not been mandatory for non critical facilities until after June 2009.

When considering the design response spectra in the 2003 seismic code (OPCM 3274, 2003), the elastic spectra was provided with a fixed shape in which the only parameter to change is the peak ground acceleration on rigid soil type. L'Aquila according to this seismic standard belongs to the second category zone (first category is the highest) with a PGA on rigid soil equal to 0.25g. Actual Italian code (CS LL PP, 2008) defines the PGA as a function of the geographic coordinates at the site, therefore L'Aquila (lat 42.38; long 13.35) has a PGA of 0.261g for soil type A (rigid), and for a probability of exceedance of 10% in 50 yrs (Figure 3-15).

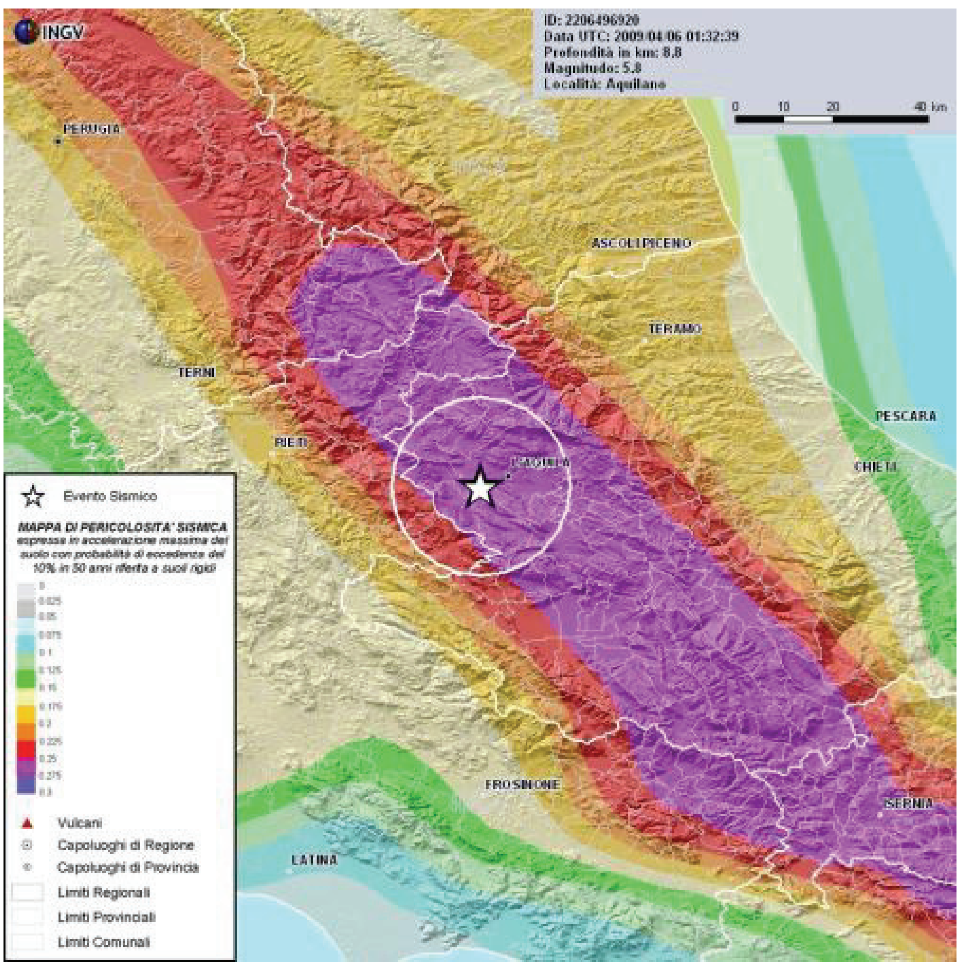


Figure 3-15 Seismic hazard map of Abruzzo region for 10% PE in 50 yrs referred to rock soil

SECTION 4

PERFORMANCE OF HISTORICAL AND MONUMENTAL PALACES

4.1 Introduction

Historical and monumental masonry palaces are very difficult to model analytically both because of their technological and constructive complexity and because of the nonlinear behavior of the masonry material. Therefore, diagnostic observation of the damage to buildings after an earthquake is an important tool for understanding the complex seismic response of masonry buildings.

The 2009 L'Aquila, Italy, earthquake highlighted the fact that seismic vulnerability of historic masonry palaces has increased because of "strengthening" retrofit work that has been done in the last 50 years. Italian seismic standards recommended use of traditional reinforcement techniques such as replacing the original wooden roof structure with new reinforced concrete or steel elements, inserting reinforced concrete ring beams in the masonry and new reinforced concrete floors, and using reinforced concrete jacketing on the shear walls. L'Aquila earthquake has shown numerous limitations of these retrofit interventions, because they lead to increased seismic forces (due to greater weight) and to deformations incompatible with the masonry walls. This report is showing some examples where seismic retrofitting by strict observance of code of practice suggestions has not always led to the improvement of the seismic performance of the building.

The observation of damage caused by the main earthquake showed that the dominant type of collapse for existing masonry structures is associated with out-of-plane kinematic mechanisms at the local and global level, due to loss of equilibrium of walls or assemblies of walls. The limitations of some traditional reinforcement techniques were also highlighted. Significant damage has been reported in buildings that had previously been consolidated after the earthquake of 1997 (e.g., Santa Maria di Collemaggio), particularly due to questionable technical choices and incorrect or poor execution of the retrofit interventions themselves (Cimellaro et al., 2010). Severe damage to old masonry buildings (Figure 4-1) due to earthquakes has caused negative impression among non-experts about the appropriateness and adequacy of the masonry as a building technique in seismic regions. Instead, collapses of these buildings are due to:

i. Poor quality of the materials
ii. Poor building construction quality
iii. Poor understanding of the seismic response of the structure and lack of accurate design
iv. Lack of maintenance
v. Vertical and horizontal development of the building without an accurate analysis of the static behavior of the original building

Figure 4-1 Damage State Map of downtown L'Aquila (Courtesy of ITC-CNR-L'Aquila)

4.2 Building Typology

The greater L'Aquila region contains a mixture of traditional buildings, made of irregular alluvial stones of uneven sizes, and more or less recent reinforced concrete buildings. In the historical centers there are groups of buildings that were entirely abandoned for decades and were already partially in ruins before the earthquake. Overall, the distribution of these building typologies on the territory is highly variable.

4.3 Building Construction Techniques in the Region

Out of the 300 fatalities in L'Aquila, 135 occurred in reinforced concrete structures and 52 in masonry structures, while outside L'Aquila there were only 14 victims in reinforced concrete structures and 97 victims in masonry buildings. It is important to mention that according to statistics, in the town of L'Aquila, there are 24% of reinforced concrete structures (much higher percentage than in the surroundings towns), 68% of masonry structures and 8% of other structures. Around 55% of the buildings were constructed after 1945 including both reinforced concrete and masonry construction.

The reasons of this discrepancy in the number of fatalities in masonry structures between L'Aquila and the surroundings towns is justified by the fact that the masonry buildings in downtown L'Aquila were rebuild after 1703 earthquake with construction rules such as:

- Wooden beams that were built in the wall thickness as small tie rods to improve connections between walls
- External wooden posts to connect the roofs to the top of the walls
- Buttress (Figure 4-2)
- Built-on scarp walls

Figure 4-2 Buttress structure

These techniques can be classified as pre-modern seismic defenses and allowed the structure to act as a unified organism that was able to sustain damage without manifesting fragility collapse.

These rules were also applied in the villages around L'Aquila, but construction in these areas was implemented with lower material quality and less construction experience. However, most of the buildings in the region were built based on earthquake awareness, particularly after the 1703 earthquake, when the use of tie rods to connect walls was systematically established. This technique uses steel tie rods introduced while the wall is raised and it is easily visible today in many buildings after using it for over two centuries. The connection consists in the placement of a wooden element built inside the wall and connects at the extremities by an iron plate and nails and then anchored externally to the corner (Figure 4-3). This technique is very efficient when the wooden element is not placed under too much tension causing weakness near the nails.

Figure 4-3 Tie rods

Other reasons of vulnerability of the surrounding villages are the frequent enlargements, adding floors and transformations, and replacing floors and roofs using materials and techniques incompatible with the original structures that generate dangerous increases of masses and stiffness differences between elements.

Most of the historic centers in the region were developed following the morphology of the ground where they were built on and were affiliated to existing buildings by developing aggregates of mono-celled structures (Figure 4-4).

Figure 4-4 Aggregates of mono-celled structures

4.4 Italian Codes and Seismic Standards

The Italian seismic standard proposes methods of retrofit for unreinforced masonry assuming a "box" behavior and rigid connections among masonry walls and between masonry walls and floor slabs that are assumed rigid in their plane. In order to satisfy these assumptions, the consolidation techniques could be described as: (i) Replacement of the wooden floor and roof assemblies with concrete slabs; (ii) Insertion of RC ring beams on every floor in the masonry; (iii) Use of reinforced plaster and/or injections of cement mixtures to increase the shear strength of masonry.

Several historic buildings, retrofitted within recent decades, have suffered partial and total collapses during L'Aquila earthquake in Abruzzo due to the incompatibility of materials and retrofit intervention techniques used. One of these examples is the out-of-plane collapse triggered by the roof in the Convent of Santa Maria di Collemaggio (Figure 4-5), where the hammering of the ridge beam generated a local collapse of the tympanum at the top of the walls, due to the fact that the roof was not effectively connected to the masonry.

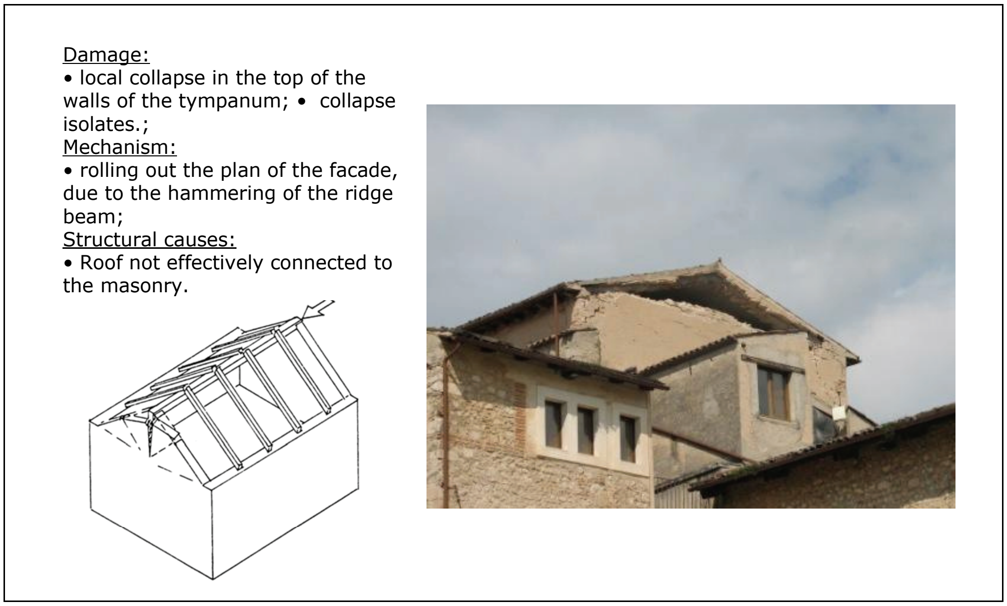


Figure 4-5 Convent of Santa Maria di Collemaggio

Figure 4-6 Poor connection between the ring beam and the masonry in the Convent of Santa Maria di Collemaggio and in Palazzo Margherita

L'Aquila earthquake has also showed that: (i) The analytical models must be adapted to different types of buildings and materials taking into account the actual structural behavior; (ii) The retrofit intervention techniques should also be adjusted, calibrated and improved.

(a)

(b)

Figure 4-7 Wooden roof and ring beam of Palazzo Margherita

The replacement of floor slabs and roofs (Figure 4-8) and inclusion of RC ring beams in the masonry (Figure 4-6) can have both positive and negative effects. In fact, the lack of or the poor connection between the ring beams and masonry generate eccentricity of the loads on each floor and collapse of walls for out-of-plane forces (Figure 4-6).

Figure 4-8 Collapse of half building in Onna triggered by the RC roof

A clear example has been observed again in the Convent of Santa Maria di Collemaggio where a RC ring beam was placed at the top of the masonry walls below the roof to strengthen the masonry walls. The smooth surface at the interface between the masonry and the ring beam shows lack of connection between the two structural elements. Additionally the external lateral walls are made with poor quality mortar and they are not connected properly to each other.

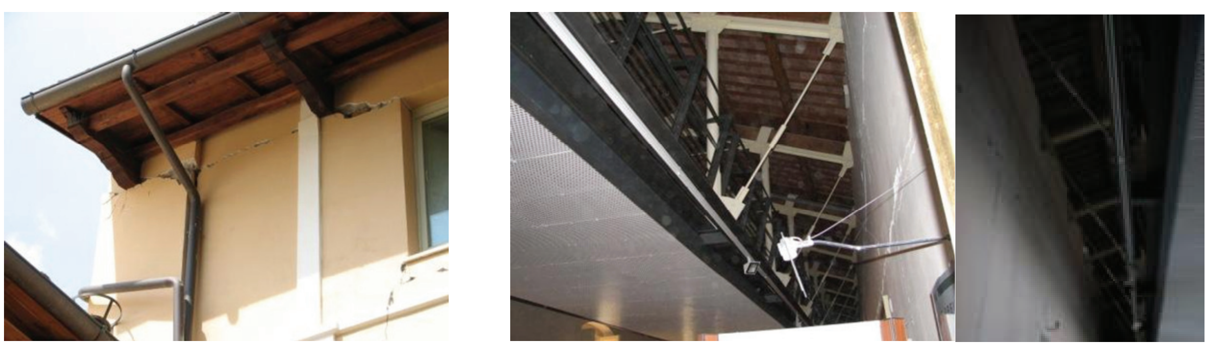
Another example where the RC ring beam had a positive aspect is the case of Palazzo Margherita where, in the early 90's, the architect Macrì made the retrofit intervention of Palazzo Margherita creating a RC ring beam under the roof cover with the aim of giving a box behavior to the whole building. The roof is a light non pushing wooden roof as shown in Figure 4-7a, and the ring beam was properly designed, but it was realized with poor quality mortar and poor quality of construction as shown in Figure 4-7b.

A typical case of ineffective and arguably counterproductive retrofit intervention was the substitution of existing wooden floors or roofs with heavier, stiffer reinforced concrete structures, whose connection with the walls was made by the introduction of “in breach” RC ring beams. Their effectiveness was doubtful and even negative when the ring beam was resting only on the inner leaf of a double leaf walls or in general on low strength, rubble stone masonry.

A typical example of collapse due to the strict application of the Italian seismic standard is given by the response of building in Onna, (Figure 4-8). The collapse of half of the structure has been triggered by the new RC roof that, concentrating a big mass on the roof, generates the overturning of the lateral walls and the consequent collapse due to the eccentricity of the vertical loads.

Another example of incorrect design due to improper choices of nonstructural elements inside the structure is given in the Ex Convent of Santa Maria dei Raccomandati. In this particular case the designer of the interior museum decided to realize a metal walkway that was hanged to the wooden trusses of the roof that in this case is a pushing roof, making the entire roof heavier than before.

The consequences of this choice made by an architect are clearly visible in Figure 4-9.



(a) (b)

Figure 4-9 Collapse of the roof due to the presence of a (b) metal walkway anchored to the wooden roof trusses in the ex-convent of Santa Maria dei Raccomandati

In summary, the replacement of wooden floors with rigid slabs (whose aim would be in principle to increase the integrity of the structural system), has been recognized as not always necessary, and alternative techniques have been developed in recent years and are still being studied to stiffen existing diaphragms and improve their connections with walls.

Another type of retrofit intervention that is recommended by the Italian seismic standard is the use of reinforced plaster and/or injections of cement mixtures to increase the shear strength of masonry.

Most common errors in the application of reinforced plasters (Figure 4-10) are: (i) lack of connection between rebar applied to the perpendicular walls; (ii) lack of overlap between adjacent rebar; (iii) lack of uniform distribution of the rebar; (iv) low durability.

(a) (b)

Figure 4-10 Reinforced plaster in Onna (a) and in Sulmona (b)

4.5 Performance of Horizontal Structures and Vaults

Recent retrofit interventions of horizontal structures were quite diffused in L'Aquila historical center. A common construction technique of the last century consists in developing floors with metallic beams and small brick arches (jack arch flooring system, Figure 4-11). Such retrofit interventions require localized cutaway of the masonry that can generate localized instability of the walls.

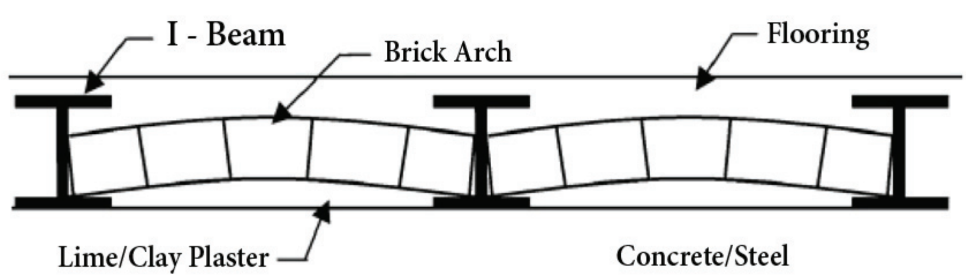


Figure 4-11 General layout of steel beam, jack arch flooring system

The jack-arch flooring system is stable under normal static conditions, but not under dynamic loading. The brick arches transfer the gravity loads, mainly in compression, to the supporting steel beams (Figure 4-12). The load is then transferred via the steel beams in flexure and shear to the load-bearing walls.

Figure 4-12 Steel beam, jack arch flooring system in Sulmona

The use of vaults is very common in L'Aquila center, where the vaults are generally in stone with thickness varying between 20 and 30cm. Barrel vaults are very common structures in L'Aquila region and they performed pretty well, because of correct dimensions of the masonry piers and the proper location of the vault within the whole building.

Low material strength, including weak mortar and lack of proper connections between perpendicular walls and between walls and roof and non homogeneous roofs are but a few parameters that contribute to the general weakness of these elements (Figure 4-13).

Figure 4-13 Collapse of vaults in the Convitto Nazionale inside the Quattro Cantoni block complex due to the movement of end support wall.

The main point of weakness of masonry vaults that contributed to poor seismic response is the inability of the roof to act as a diaphragm, because the load carrying mechanism of these types of roofs is primarily in compression. As a result, they are not capable of restraining the top of their supporting walls during ground shaking, nor are they capable of transferring excessive horizontal inertia forces. Furthermore, another point of weakness is their excessive weight.

SECTION 5

BUILDING DAMAGE

5.1 Private Residential Buildings

5.1.1 Performance of Unreinforced Masonry Buildings

Masonry construction demonstrated its limits, but also its great resistance capacity as well. The collapse was always explainable by the presence of construction defects, incongruous restoration or advance decay, while the good performance of masonry buildings was associated with specific construction details often realized during the construction phases.

Wooden roof structures connected to masonry walls through wooden keys (Figure 5-1) demonstrated good behavior limiting, in many cases, the overturning of the external walls.

Figure 5-1 External roof wooden truss in L'Aquila

The squared stone elements placed around the window openings were rarely connected to the masonry walls and this led to their detachment (Figure 5-2), developing an element of seismic vulnerability of many buildings.

Figure 5-2 Detachment of squared stone elements around the openings in L'Aquila historical center

Localized collapse of the top masonry wall is shown in Figure 5-3 and it is probably related to the poor quality of the masonry pattern at the top of the wall.

Figure 5-3 Localized collapse of a portion of the top masonry wall

Out of plane overturning of walls was observed in the region, but was very limited because of the systematic use of tie rods in building walls. This collapse mechanism was triggered in cases of poor connection quality at the corners and involved failure of portions of the orthogonal walls (Figure 5-4; Figure 5-5).

(a) (b)

Figure 5-4 Out of plane wall overturning

Figure 5-5 Collapse of the corner walls in L'Aquila historical center

In plane wall damage is present in many buildings and the form and placement of shearing varied according to the efficiency of the connections between elements and the quality of masonry.

The presence of shear fractures in masonry walls is a consequence of the box-like behavior of the building, allowing significant energy dissipation and reducing the risk of collapse, therefore, this type of damage should be considered as a positive sign of the good behavior of the masonry building (Figure 5-6).

Figure 5-6 Shear cracks in masonry walls near window openings

Ruinous collapses or overturning of external façade can be justified by the poor quality of the masonry pattern, poor cement quality and excessive quantity of stone elements, as shown in Figure 5-7. Walls were vulnerable to horizontal forces, because of small or badly placed stone elements, lack of regularity in the pattern and interlocking between the facing sections.

Figure 5-7 Collapse caused by defective masonry quality in Castelnuovo

5.1.2 Performance of Reinforced Concrete Buildings

Reinforced concrete (RC) buildings—particularly mid-rise, multi-family condominium structures—experienced significant damage in the 2009 L'Aquila earthquake. The most frequent type of damage consisted of cracking and, in some cases, failure of nonstructural masonry infill walls, as shown in Figure 5-8. Windows or door openings represent a discontinuity in the infill panel, modifying its stiffness and failure mechanism (Figure 5-8b).

Most residential buildings in L'Aquila and surroundings are relatively modern reinforced concrete frame structures with masonry infill on exterior walls and are two to four-stories high, with apartment buildings closer to the city center reaching 5 or 6 stories. The few collapses observed in the damage survey were of four-story reinforced concrete buildings, typically at the ground floor level.


(a) (b)

Figure 5-8 Damage to nonstructural masonry infill walls in Pettino

Figure 5-9 Out of plane mechanism of infill wall

Many structures in this area were not damaged, and where damage occurred, it generally involved relatively minor cracking to relatively severe cracking or collapse of the masonry infill walls (Figure 5-9). Severe structural damage was relatively rare. The reasons of collapse must be found in the older RC buildings in the region that use smooth reinforcing bars, unconventional lap splices, and in some cases, poorer quality construction materials. The frames are almost always designed with no consideration for the layout of masonry infill walls both in plan and elevation, as these are considered to be nonstructural elements, and their exclusion is thought to lead to conservative design. The framing is filled in with one or two wythes of hollow clay or concrete blocks, and sometimes finished with wrap-around clay brick facades or plaster. Partitions are of thin hollow clay blocks.

In summary, reinforced concrete structures suffered minor or no damage during this earthquake and majority of damages were located in the nonstructural masonry infill walls.

5.1.2.1 Case Study: The L'Aquila University Dormitory

The most dramatic RC building failure was the multi-story section collapse of a five-story L'Aquila University dormitory (Figure 5-10), which caused the death of eight young students.

Figure 5-10 Collapsed wing of the L'Aquila House Dormitory due to the lack of one column

The reason of the collapse has been identified in the lack of one of the columns at the base. Furthermore the entire building was not designed to be earthquake resistant. There were no moment resisting frames (Figure 5-11) and the hoop spacing were not properly designed in order to guarantee confinement effects in the concrete at the joints (Figure 5-13, Figure 5-14). Also the quality of the concrete is poor and the electrical, thermal and air conditioning systems were not properly located in the structure generating additional damages as shown in Figure 5-12.

Figure 5-11 Part of a collapsed beam at the ground floor

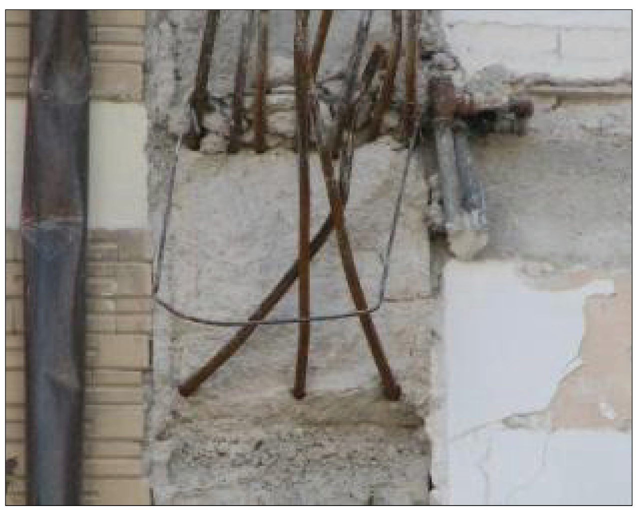


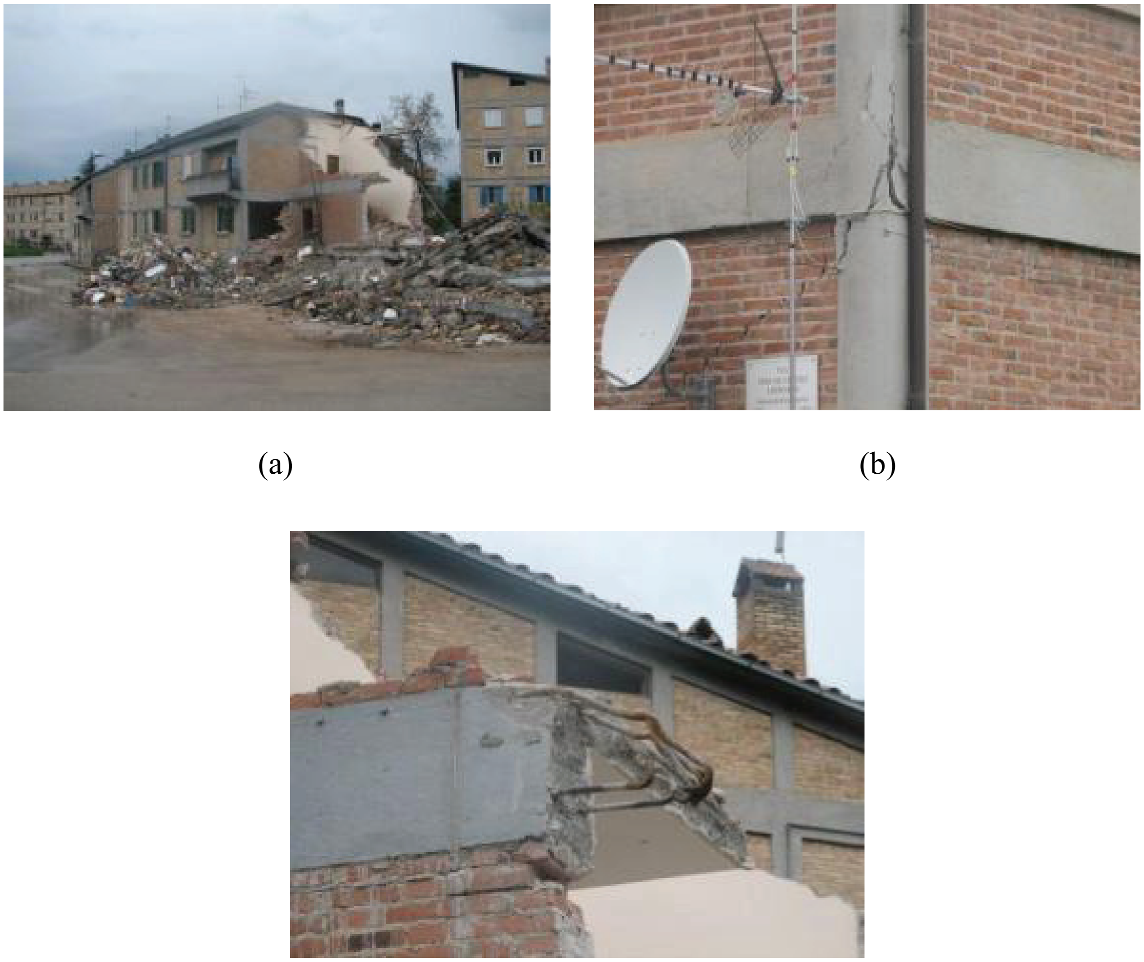
Figure 5-12 Poor quality of construction with thermal pipeline passing through the beam column node

Figure 5-13 Lack of hoops at the beam-column intersection

Figure 5-14 Detail of a hoop with diameter of 6 mm

5.1.2.2 Case Studies of RC Private Residential Buildings

The majority of reinforced concrete damaged buildings was concentrated in the Mt. Pettino region and was originally constructed in the mid-1980s. Pettino is located in the North West suburbs of L'Aquila. The MCEER team observed damage in a series of residential blocks built in the 1980s extending up a hill-side. All these buildings consisted of reinforced concrete frames with hollow core clay bricks as the one shown in Figure 5-15.



(a) (b)

(c)

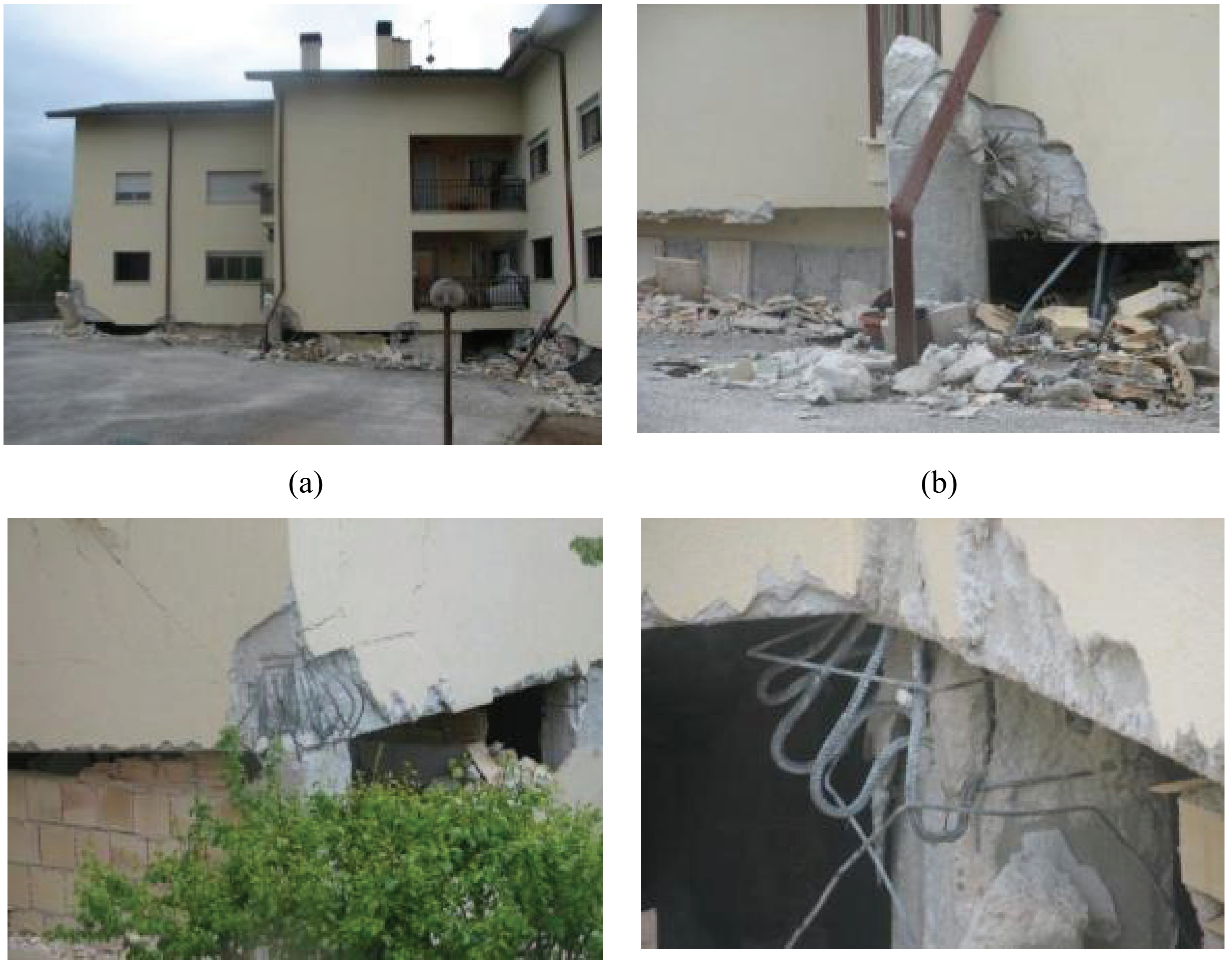
Figure 5-15 Collapse of a two story residential building

A typical diagonal cracking in a concrete external joint that resulted in the loss of monolithic connection is shown in Figure 5-15b. Beam-column joint failure should be avoided in proper seismic design, considering that these elements are characterized by brittle failure mechanisms.

RC frames retrofitted with structural shear walls (Figure 5-16) typically experience less damage because drifts are limited. The 5-story rectangular building shown in Figure 5-16 was constructed with two vertical joints between the three shear walls in order to give a more regular stiffness distribution. However the central shear wall has double mass with respect to the two laterals shear walls and it was severe damaged at the base.

Figure 5-16 Damage to shear walls system in Pettino

Several buildings suffered a typical soft-storey failure, as the one shown in Figure 5-17 that was located on the hill slope in Dante Alighieri Street. The building is T-shape in plan and therefore it can be considered as irregular in plan. The main reason of collapse is the different infill distribution along the height of the building, because infill walls are absent at the first story level due to the presence of garages entrances. However, an adjacent building located on horizontal ground suffered negligible damage to lower storey columns. This suggests that the reason of collapse might be the topographic amplification effects, combined with some vulnerability such as the infill irregularity in elevation and local interaction between infill walls and adjacent columns at the first level that caused brittle failure.



(a) (b)

(c) (d)

Figure 5-17 Soft story collapse of building in Pettino

In Figure 5-17c-d can be observed the buckling of the compressed longitudinal reinforcements, due to not proper hoop spacing.

At the top of the hill, a 5-storey RC Frame (Figure 5-18) with approximately 80m x 20m plan dimensions and built in 1990 suffered from extensive damage to non-structural and structural elements. Shear cracks in connections were observed. Poor reinforcement anchorage and lack of hoops were evident. This is a typical example where local and global interaction effects between infill walls and RC structural elements are not negligible. At the global level the infill walls increase the global stiffness of the structure increasing the spectral acceleration demand and therefore the seismic forces acting on the building. At the local level when the infill wall partially fills the frame bay, it can reduce the effective length of the column, increasing the shear demand and the consequent brittle failure of the column (Figure 5-18d). It is important to mention that two new RC villas had recently been completed nearby and were undamaged.

(a) (b)

(c)

Figure 5-18 Squat column shear failure

5.2 Churches

5.2.1 Case Study: Santa Maria di Collemaggio

5.2.1.1 Introduction

The experience of L'Aquila earthquake (2009) showed that the collapse mechanism of existing masonry structures consists in the activation of cinematic mechanisms with loss of equilibrium in walls or blocks of walls, with a prevalence of out-of-plane mechanisms both globally and locally.

Figure 5-19 Aerial view of the church complex before the earthquake

It also highlighted the limitations of traditional, but also innovative retrofit techniques. Considerable damage was also reported for buildings that had previously been seismically retrofitted. The case of the Santa Maria di Collemaggio Basilica represents an interesting case, because it is the only church in L'Aquila that was retrofitted after the 1997 earthquake in Central Italy and therefore the April 6th 2009 earthquake allows consideration and assessment of the performance of the Church's recent retrofit, including some weaknesses and limitations.

S. Maria di Collemaggio is the largest medieval church in the Abruzzo region (Figure 5-19). It was the site of the original Papal Jubilee, a penitential observation devised by Pope Celestine V, who has been buried here. The church, which therefore ranks as a basilica because of its importance in religious history, is located at the South-West side of the town and it is a masterpiece of Abruzzese Romanesque and Gothic architecture and one of the symbols of L'Aquila. The striking jewel-box effect of the exterior is due to a pattern of blocks of alternating pink and white stone (Figure 5-53, Figure 5-54); the interior, on the other hand, is massive and austere. The basilica has a nave and two side aisles. The dimension of the nave is 61 m long and 11.3 m width with a height, that reaches 18.25 m (Touring Club Italiano, 2005).

The two side aisles are respectively 7.8 and 8 m width with 2 external walls 12.5 m high. Seven pillars, not evenly spaced, on each side separate the nave and two side aisles (Figure 5-21, Figure 5-22, Figure 5-23). The pillars are about 5.25 m high; a covering of well-laid stones encloses their core, which is filled with calcareous stones irregularly arranged in a poor quality mortar (Figure 5-20). The transverse section, approximately circular, is on average about 1.00 m in diameter. The thickness of the external masonry walls varies between 0.95 m and 1.05 m; it is 0.9 m for the two walls of the nave, over the columns. The four walls are connected on one side to the facade of the basilica and, on the other side, to the transept (Figure 5-24).

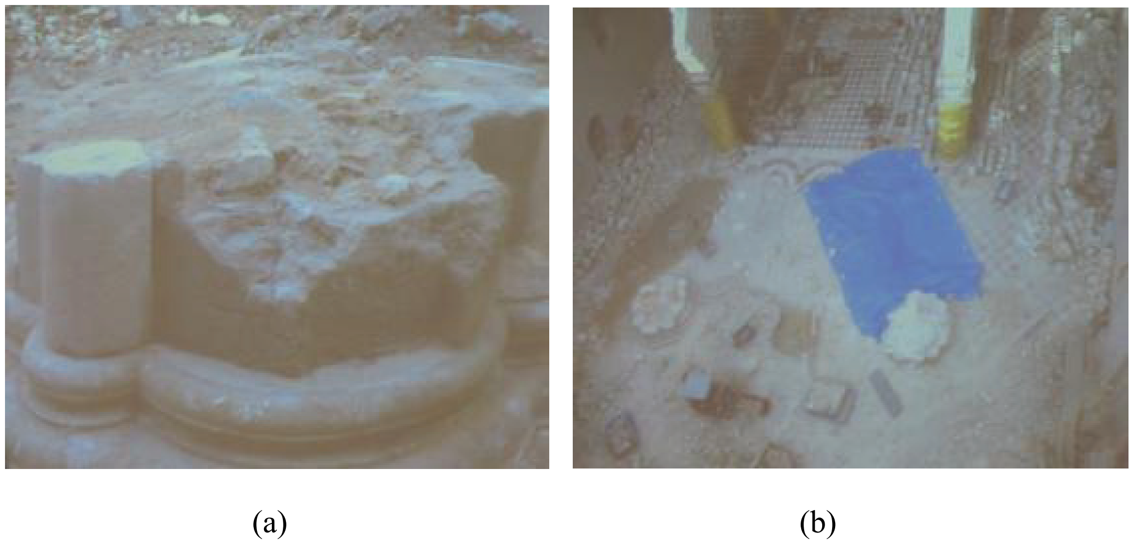


Figure 5-20 (a) Pillar section filled with calcareous stones;
(b) aerial view of the two central pillars

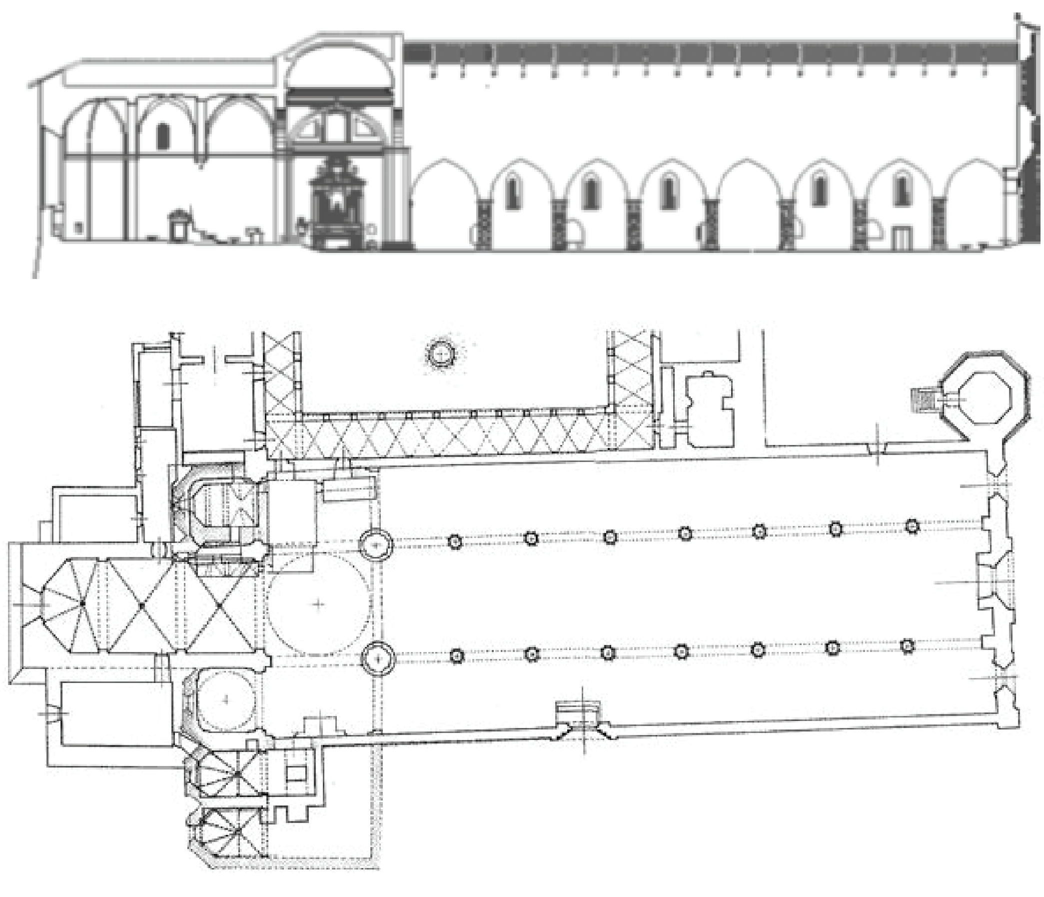


Figure 5-21 Plan and longitudinal view of Santa Maria di Collemaggio

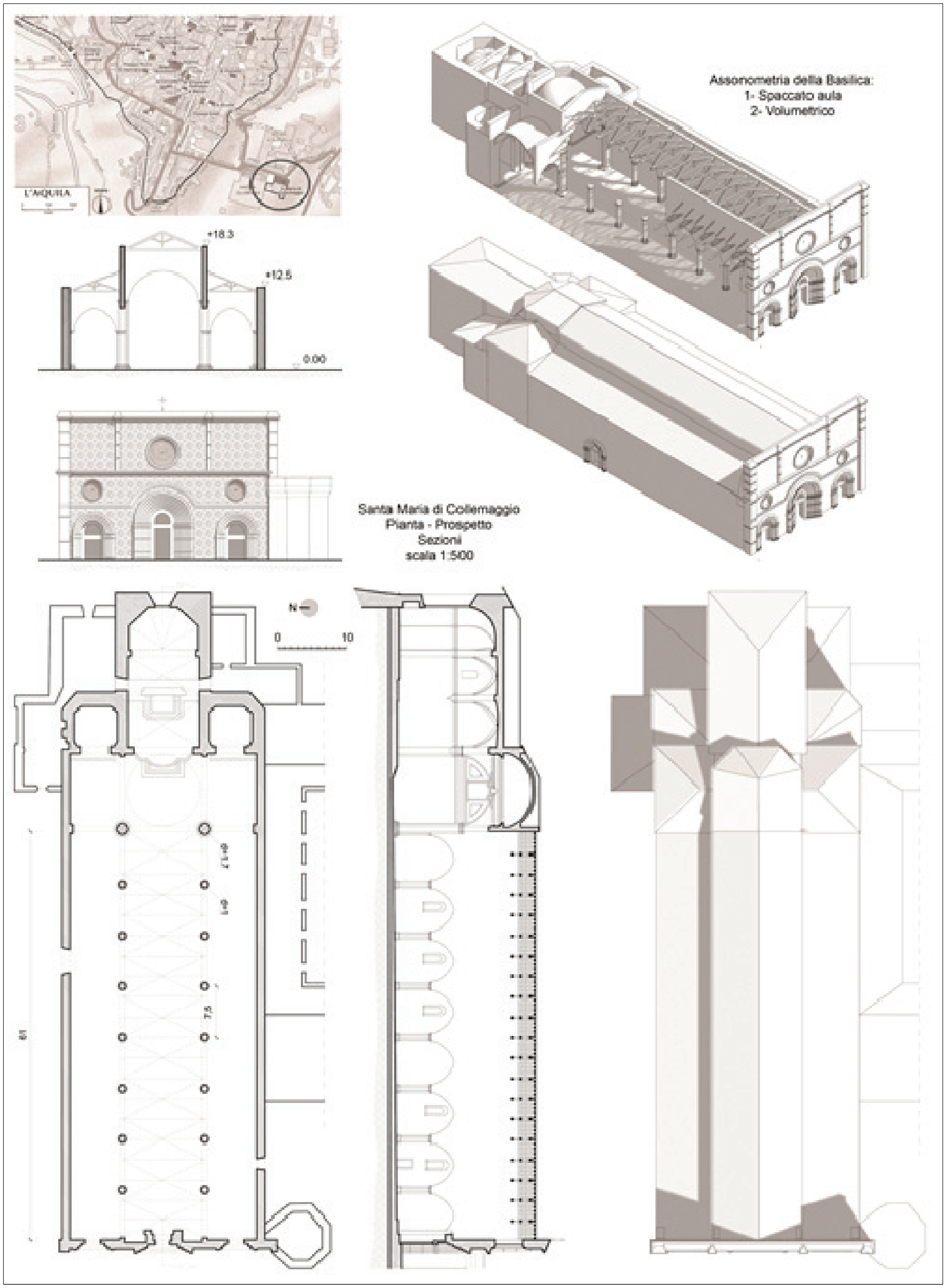


Figure 5-22 Frontal and cross-section isometric view of the Basilica

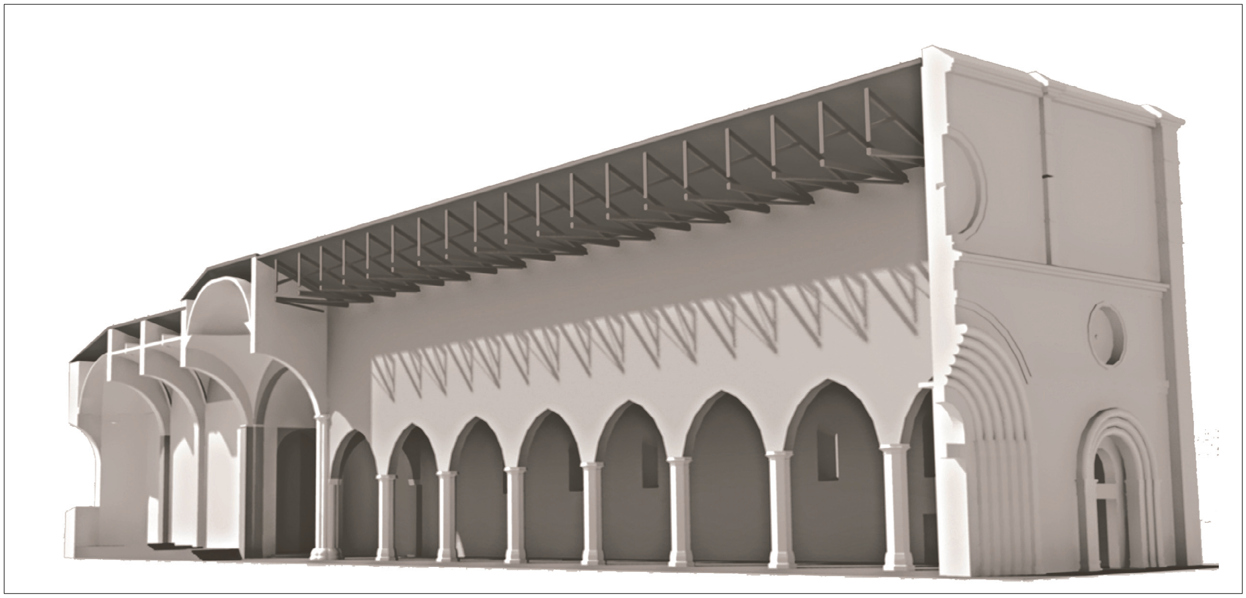


Figure 5-23 Cross-section isometric view of the Basilica to understand the complexity of the system

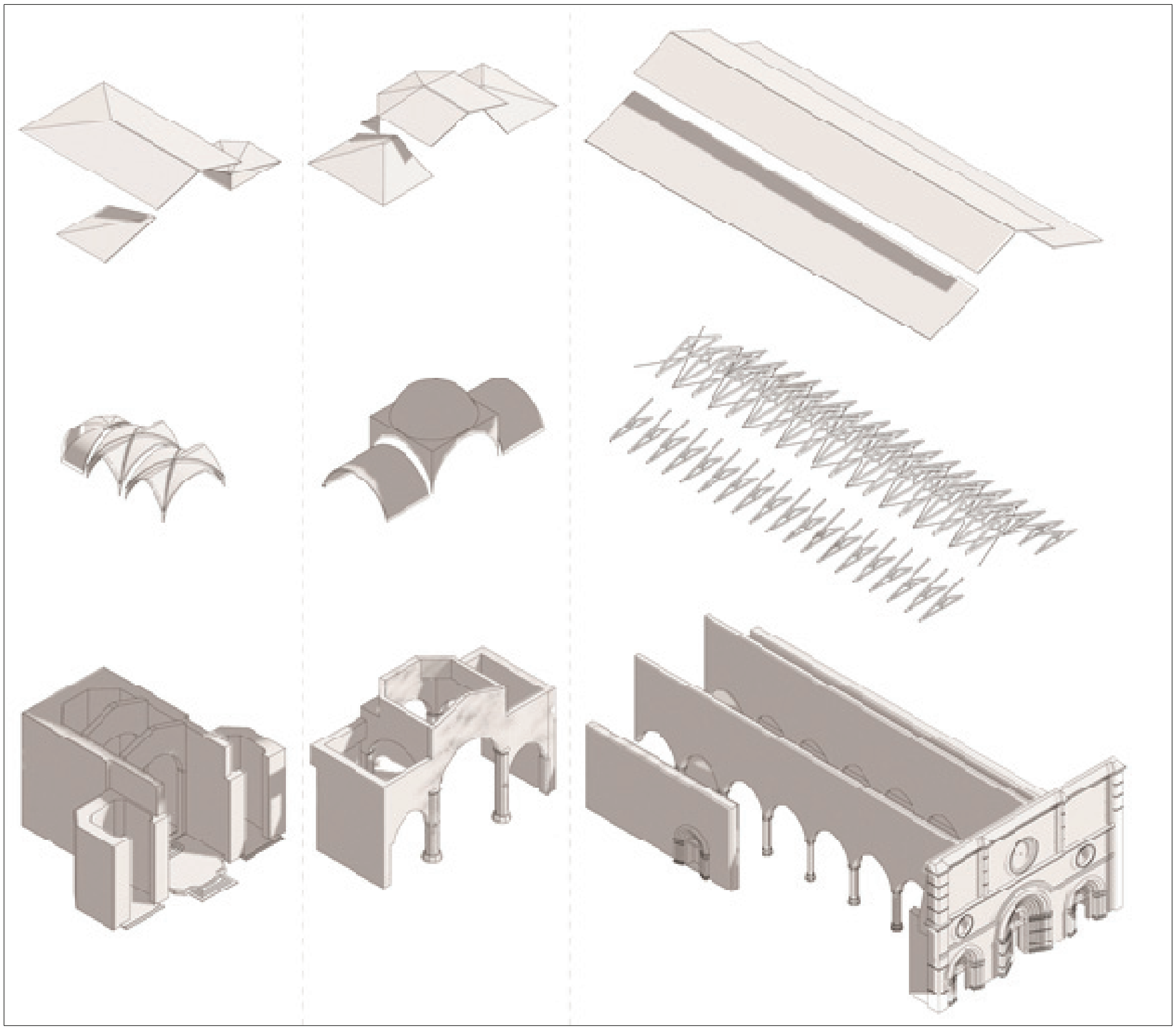


Figure 5-24 Geometric survey of the Church

The facade is attached to a squat octagonal tower on the right corner; another masonry building is adjacent to a part of the wall, about 40% of it, behind the tower. The wooden roof is supported by trusses placed in the cross sectional direction to the walls (*Figure 5-25*).

Figure 5-25 Diaphragm behavior of the wooden roof due to the horizontal truss braces

The soil under the church is of type C, without site effects due to local amplification of earthquake shaking.

5.2.1.2 Previous Restorations of the Last Century

Seismic assessment of historical churches is a difficult task due to the heterogeneity and uncertainty typical of the constituent materials, the intricate geometry configurations, often modified by previous structural or architectural interventions, etc. Usually, heritage masonry constructions are characterized by a wide variability of wall texture, degree and quality of connections, therefore, each structure is a singular case and archival information is required. The present state of the church is the consequence of articulated developments of building phases, with reconstructions and renovations that began right after the construction of the Celestinian church (1287 – 1294) and concluded with the retrofit of the Jubilee (2000).

After 1315 earthquake and after the arrival of Saint Celestine's relics in L'Aquila, in 1327, the construction started again because of the intervention of Matthias Camponeschi who supported economically the realization of the church. Further reconstructions followed the violent 1349 earthquake, during which damaged structures were restored, but also changes to the thirteenth century structure were made. Indeed, the three apses were regularized with the extension of the greater one towards the embankment of the hill on which stands the basilica; the new floor dichromium was built, about 20 cm higher than the previous one; the upper and external walls of the transept was rebuilt with the insertion of arch windows.

Around the first half of the fifteenth century the extraordinary front-facade that began in the early fourteenth century was completed.

On February 2nd, 1703, there was a devastating earthquake with moment magnitude of 6.7, equivalent to the tenth grade of Mercalli Scale that caused more than 3,000 deaths only in the town of L'Aquila. The basilica of Santa Maria di Collemaggio was seriously damaged and it was necessary to restore its facade and the interior.

The following four main interventions can be distinguished in the last century:

1. Between 1918 and 1921, the restoration introduced some concrete beams in the walls to strengthen the walls of the monumental façade with spurs and tie rods.
2. In 1960-1962, the dome of the transept was rebuilt with light RC and the masonry below was strengthened by introducing RC beams.
3. Between 1970 and 1972, the Romanesque structure was restored, removing the wooden baroque ceilings and raising of about 3 meters the walls of the aisles. Additional beams were installed at the top of the wall, where wooden trusses and other roof structures are located.
4. In 2000, a horizontal energy dissipation bracing system with steel hysteretic dampers was installed at the roof level, beneath the wooden trusses of the central nave and the two aisles. These braces (diameter Ø ≈ 30 mm) at intersection include a steel damper that was expected to dissipate energy during the seismic cycles.

The current medieval appearance is a result of the radical restoration carried out in the '70s during which all additions made in the seventeenth and eighteenth centuries were removed.

Today it is a very large room divided into three longitudinal aisles that end by the transept with a dome at the intersection of the three apses. The aisles are separated by arches, eight per side, on octagonal pillars.

Mario Moretti, author of the discussed restoration in the ‘70s, denounced the serious state of degradation of the walls of the transept, because of tons of baroque plaster decoration and by the RC dome realized in the ‘60s. Indeed, for aesthetic and static purposes, he scraped only part of the baroque basilica, removing structures that were also strengthening the entire structure, and he retained the "original" parts of the concrete dome. This intervention weakened the structure, but it does not explain the collapse of the heaviest part: the dome.

This chapter presents an assessment of the performance of the last (2000) seismic retrofit of Collemaggio basilica in the 2009 L’Aquila earthquake in Italy, showing how its limitations may have caused considerable damage to the church.

5.2.1.3 Damage Assessment

The analysis of historical monumental buildings is a complex task. There are many uncertainties in the process which can be summarized below:

1. Information about the inner core of the structural elements is missing
2. The characterization of the mechanical properties of the materials used is difficult and expensive
3. There is a large variability in the mechanical properties
4. Significant changes have occurred in the core and constitution of structural elements, associated with long construction periods
5. Existing damage in the structure is unknown
6. Regulations and codes are not applicable

Nevertheless some considerations about the reasons for collapse can still be carried out to help avoid future improper retrofit and to protect Italian historical heritage.

The following damages of the basilica of “Santa Maria di Collemaggio” were reported following the reconnaissance field investigation done by MCEER in collaboration with the Politecnico di Torino:

1. Collapse of the vault of the transept, of the Triumphal arches and the reinforced concrete dome (Figure 5-26) due to the collapse of the two central pillars (Figure 5-20a);

2. Collapse of the “tamburo” and of the dome (Figure 5-26);
3. Collapse of the roof elements of the transept (Figure 5-27);
4. Slight damage on the stone and the rose window of the façade; overturning of the right side front wall, with moderate cracks in the stones of the Tower.
5. Crushing of the pillars of the nave (Figure 5-28) and broken chains at the anchorages on the North wall;
6. Damage to the frame of the “Holy Door” that is inserted in the masonry (Figure 5-29). The stiffness variation between the masonry walls and the portal is the cause of damage to the marbles of the portal.
7. Severe cracks with imminent collapse in the apse and in the presbytery (Figure 5-45, Figure 5-46, Figure 5-47, Figure 5-48, Figure 5-49).

Figure 5-26 Collapse of the Transept

Figure 5-27 Interior of Santa Maria di Collemaggio

In Figure 5-30, the remains of the reinforced concrete ring beam located on top of the main walls of the nave are visible. The smooth surface at the interface between the masonry and the ring beam show lack of connection between the two structural elements. Additionally, the lateral walls are made with poor quality mortar and are not properly connected to each other. It is not common to observe wooden ties inside the masonry that are not sufficient to guarantee the "box behavior" of the masonry walls.

Figure 5-28 Shoring up of the columns of the nave through confinement to increase their axial capacity

Figure 5-29 Damage to the jubilee "Holy Door" of the Church

Figure 5-30 Detail of the top R.C. ring beam

5.2.1.4 Retrofit with Energy Dissipation Devices

The basilica of “Santa Maria di Collemaggio” in L’Aquila was slightly damaged in the 1997 Marche and Umbria earthquake despite its large distance from the epicenter, therefore the local government of L’Aquila decided to retrofit the church inserting quadrilateral steel hysteretic dampers (Ciampi et al., 1990, Cartapati, 2000) at the roof level that were manufactured by ALGA (Ciampi and Marioni, 1991), an Italian company. The installed devices can be classified as *displacement-activated devices* (Soong and Dargush, 1997; Christopoulos and Filiatrault, 2006), because they were expected to be able to dissipate energy through the relative displacement between the two adjacent walls as shown in Figure 5-31.

In the same period, other devices, with a different shape (C-shaped devices) (Ciampi and Marioni, 1991), were developed following the same principle of the quadrilateral dampers to achieve a uniform plasticization along the member. The device that consists of 16 C-shaped crescent moon elements in a radially symmetric configuration was installed on each pier cap (Marioni, 1997; Marioni, 2000; Ghasemi et al., 2000) in the Bolu Viaduct in Turkey, a 2.3-km long seismically isolated structure which was essentially complete when the earthquake occurred. The viaduct suffered a complete failure of its seismic isolation system and narrowly avoided total collapse due to excessive differential ground, substructure, and superstructure movement, during the destructive 1999 Duzce (Turkey) earthquake (Roussis *et al.*, 2003).

The Bolu viaduct in Turkey as well as the Collemaggio Basilica in Italy demonstrate that the main concern related to displacement activated devices is that the internal forces generated by the dampers are in phase with the maximum deformation of the structure, therefore maximum forces and deformations appeared at the same time in the structure. If viscous dampers would have been adopted instead, the maximum forces and the maximum deformations would have been out of phase (not complimentary), therefore probably stress concentration in the transept of the church would have been avoided and consequently the local collapse of the dome.

From the preliminary field observations of the church done soon after the earthquake, it was clear that neither the design nor the location of the steel dampers were optimal as it will be described in the next paragraphs. In order to understand the retrofit philosophy of the Basilica, a detailed description of the steel hysteretic damper and its behavior is given below.

5.2.1.5 Quadrilateral Steel Damper

The concepts, based on which the steel dampers were installed in the Santa Maria di Collemaggio basilica can be tracked in the literature of the last 25 years (Gluck et al., 1996; Lavan et al., 2008, Occhiuzzi et al, 2009, Cimellaro et al, 2009), and are therefore not considered as new or original. Indeed, they were inspired by the Pall friction dampers manufactured by Pall Dynamics Ltd that were tested in 1985 and applied in the first building in 1987 in Canada (Filiatrault and Cherry, 1990).

The system consists of an articulated quadrilateral steel mechanism introduced at the intersection of the frame cross-braces located at the roof level (Figure 5-31). The main characteristic of this device that is in common with the Pall system is its ability to dissipate energy through the uniform yielding of the steel section and independently of the slenderness ratio of the bracing members (Christopoulos and Filiatrault, 2006) that can indeed have a reduced section (Figure 5-31). In fact, as shown in Filiatrault and Cherry (1990), the compressed diagonal brace that buckles does not work until it recovers the previous total displacement. The quadrilateral could recover part of the axial displacement of the cross brace, due to buckling, increasing its energy dissipation capacity. However, if the device does not yield during cyclic action, the quadrilateral could not be activated and the compressed brace would keep the axial displacement due to the lateral buckling. Furthermore, it is assumed that the yielding displacement of the device is large enough to lengthen the compressed brace, but if this does not happen, the device is not able to dissipate enough energy. In fact, while small drifts do not induce any axial force, for large displacements, the diagonal brace which becomes shorter varies its length more than the other one, and so the entire bracing system is stretched. However, laboratory experimental tests of the quadrilateral steel damper performed at ENEA showed that the slender braces did not always remain in tension as claimed, but they buckled out-of-plane in compression without dissipating energy at large displacements. Therefore, the geometrical nonlinear effects (hardening) at large displacements did not play a role in the structural control of Santa Maria di Collemaggio basilica. In designing damper device after selecting the yielding force and the material, its geometric parameters are selected opportunely by varying the cross section *b(x)* of the quadrilateral steel damper in order to achieve a uniform yielding of the steel section according to the formula given below (Ciampi *et al.*, 1990)

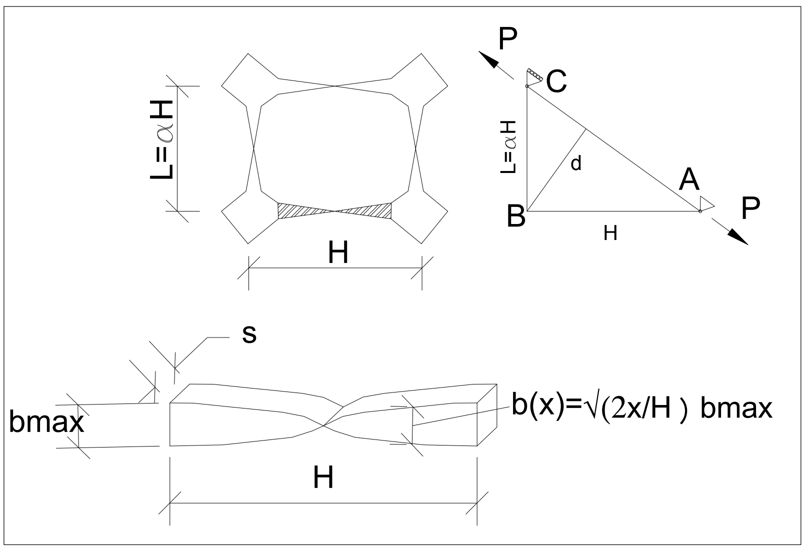


Figure 5-31 Quadrilater steel hysteretic damper (Ciampi et al., 1990)

$$b(x) = \sqrt{\frac{2x}{H}} b_{MAX} \tag{1}$$

The elastic stiffness of the damper K_d is evaluated by the ratio between the yielding force and the corresponding displacement along the diagonal of the brace

$$K_d = \frac{P_y}{\delta_y} = \frac{1+\alpha^2}{(1+\alpha)\alpha^2} \frac{sEb_{MAX}^3}{H^3}; \tag{2}$$

Equations (1) and (2) were derived based on the static scheme in Figure 5-31 assuming an asymmetric linear moment distribution. The maximum moment in B is easily expressed as:

$$M_B = M_C - \frac{P}{2} d \tag{3}$$

and because $|M_B| = |M_A| = |M_C|$ from Eq. (3) one obtains

$$|M_B| = \frac{P}{4} d = \frac{\alpha}{4\sqrt{1+\alpha^2}} PH \tag{4}$$

The yielding moment is

$$M_y = \sigma_y \frac{sb^2(x)}{6} \tag{5}$$

while the yielding force when b is maximum is given by

$$P_y = \frac{2}{3} \frac{\sqrt{1+\alpha^2}}{\alpha} \frac{s}{H} b_{MAX}^2 \sigma_y \tag{6}$$

The plastic moment variation at any section of the device is

$$M(x) = 2\frac{M_B}{H} x \tag{7}$$

In order to maximize the energy dissipation of the device, it is desirable that the plastic moments at any section be reached simultaneously. This can be achieved by varying the geometry of the devices varying the optimum depth b as follow

$$b(x) = \sqrt{\frac{2x}{H}} b_{MAX} \tag{8}$$

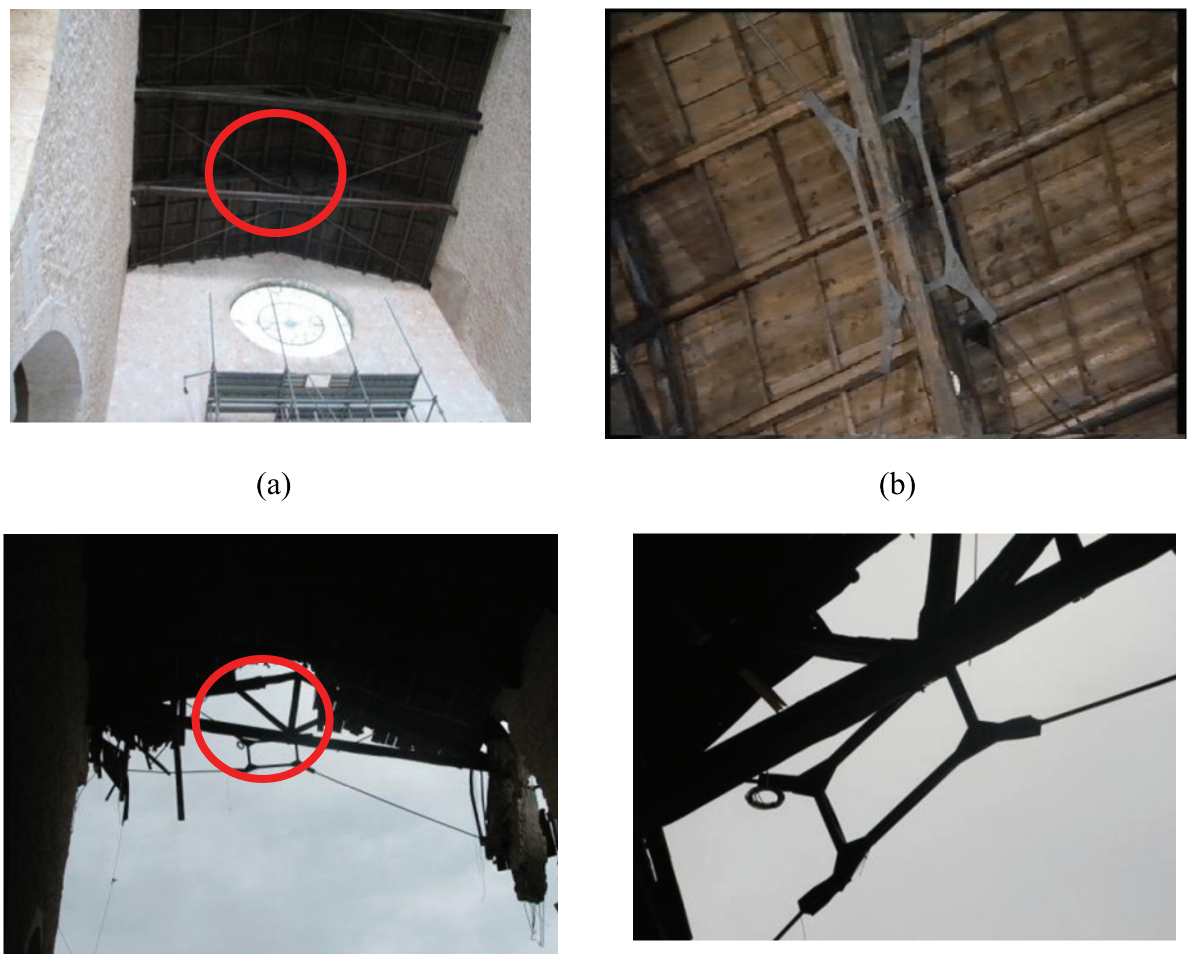
The maximum displacement can be obtained with the general displacement rule

$$\eta = 2\frac{\alpha}{\sqrt{1+\alpha^2}} \left[\int_0^{H/2} \chi(x) x dx + \int_0^{L/2} \frac{1}{\alpha} \chi(y) y dy \right] \tag{9}$$

And after integration yields

$$\eta = \frac{2}{3} \frac{\alpha(1+\alpha)}{\sqrt{1+\alpha^2}} \frac{H^2}{b_{MAX}} \varepsilon \tag{10}$$

So the elastic stiffness in Eq. (2) is given by the ratio between the yielding force given in Eq. (6) and the displacement given in Eq. (10). Based on the dimensions of the quadrilateral device estimated during the field observations inside the church right after the seismic event, a yielding force of around P_y=*47kN* and an elastic stiffness of K_d=3539.6 N/mm has been estimated for the device assuming E=210000N/mm^2, H=1500mm, α=0.4, s=40mm, σ_y=235N/mm^2 and b=65mm. While it was not possible to measure the effective geometric dimensions of the device, in particular the depth of the yielding section by varying s between 30 and 65mm, a total yielding force would be obtained varying between 35.6 and 77.2kN. In any case, the yielding force of the device was lower than the yielding force in tension of the cross brace; that is, P_y=166kN, assuming a diameter of *30mm* and a length *L=6600mm*, while the buckling load of the brace is P_{crit}=*1.8kN*, which is lower with respect to the yielding force of the device. According to the theory based on geometric considerations of the quadrilateral damper, the slender brace should remain in tension and not buckle, however, as previously performed laboratory experimental tests have shown, the slender braces did not always remain in tension, but instead buckled out-of-plane in compression. Therefore, the damper will not be able to dissipate energy at large displacements. In summary, the device might yield in tension before the brace, but it might not yield during reversal of the tension load or in compression.



(a) (b) (c) (d)

Figure 5-32 Location of the two hysteretic steel dampers at the two edges of the nave (Ciampi et al., 1990)

Two hysteretic steel dampers were installed at the roof level, at the two edges of the main nave (Figure 5-32) in the transept and at the frontal façade, respectively. The position in plan of the two dampers is also shown in Figure 5-34.

Figure 5-33 No evidence of yielding in the steel hysteretic damper near the frontal facade (Ciampi et al., 1990)

The main idea of the horizontal brace system at the roof level is to connect the two lateral walls of the main nave generating a diaphragm behavior (Figure 5-40; Figure 5-41). The effect of this connection is the development of a stiffer nave with translation of the stiffness center (CK) toward the frontal façade (Figure 5-34). The stiffening effect of the retrofit is illustrated by Antonacci et al., (2001), which performed numerical analysis by finite element models that were calibrated by dynamic identification tests performed before and after the retrofit, and concluded that the retrofit intervention produced an increment of the first frequency from 1.20 Hz before retrofit to 1.41 Hz after retrofit, while increment of the second mode is from 1.79Hz before retrofit to 2.24 Hz after retrofit.

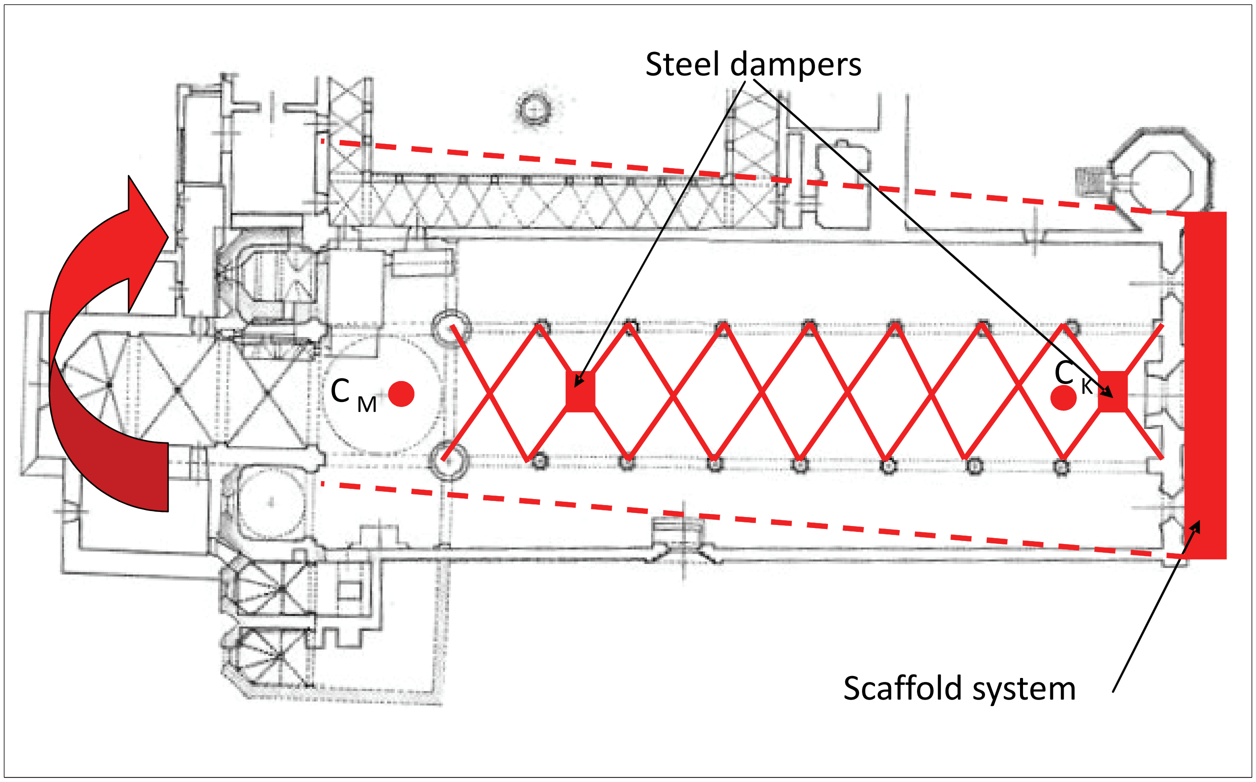


Figure 5-34 Schematic representation of the torsional effects due to the retrofit

The participating modal mass ratios of the first two modes reach 85% of the total mass in the transverse direction of the church (Figure 5-40; Figure 5-41). Therefore, the peak displacements at the roof level can be estimated using the modal combination of the first two modes. The period shift due to the retrofit generated larger reaction forces at the end support of the central nave, because of increments of the spectral demand in the response spectra obtained using the accelerograms recorded in the stations near the Basilica (Figure 5-35).

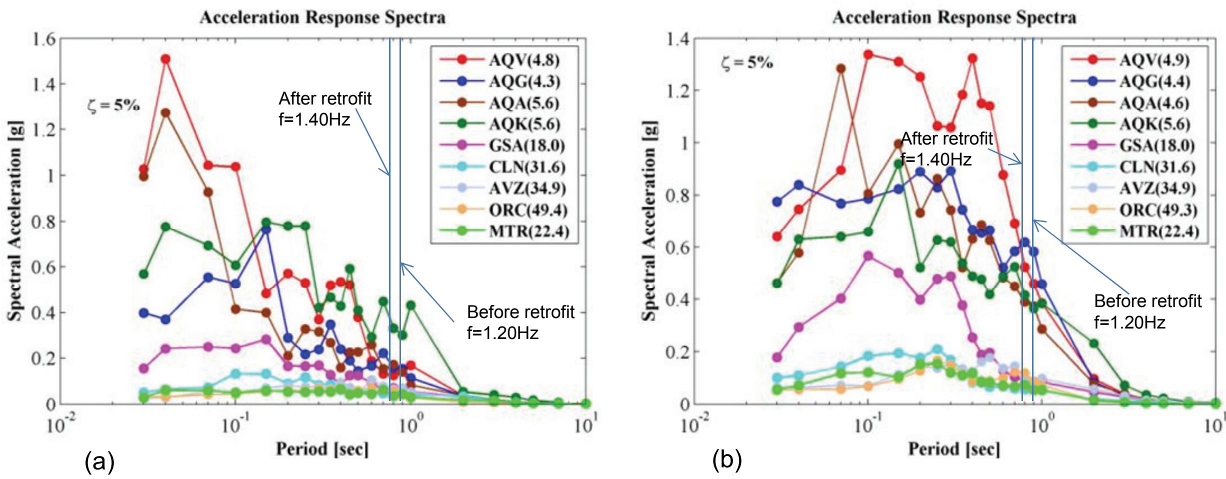


Figure 5-35 (a) Vertical response spectra; (b) Horizontal response spectra of April 6th L'Aquila Earthquake

By observing the damage inside the basilica, two simplified models can be assumed (Figure 5-37):

1. *Simply supported beam model;*
2. *Cantilever beam model;*

The main assumption of the first model is that the central nave is fixed at both ends due to the presence of the frontal façade, external buttresses and heavy walls (Figure 5-36) on one side and the transept on the other side. Using this model, the dampers are located in their optimal position, because in both modes, they are located where the maximum drift (beam rotation) is obtained.

Figure 5-36 Buttress of the frontal façade

If this had been the most reasonable model, then both dampers should have yielded and cracks near the frontal façade should have been observed.

The second model (cantilever beam model) assumes that there is only one fixed point in the frontal façade, while the other side of the transept is free to move. This assumption is reasonable and proved by the fact that the damper near the frontal façade could not be activated during the seismic event of April 6th (no evidence of yielding) (Figure 5-38) because it was located near a fixed point with minimum drift (beam rotation), while the maximum drift was achieved in the middle of the central nave as proved by the presence of a broken tendon at the threaded section (Figure 5-37b, Figure 5-39). If the device was properly designed and located, it would have been the only member that behaved in a nonlinear manner during the design earthquake, while the rest of the structure remained in the elastic range. Instead, from field observations, it is clear that the device was not effective.

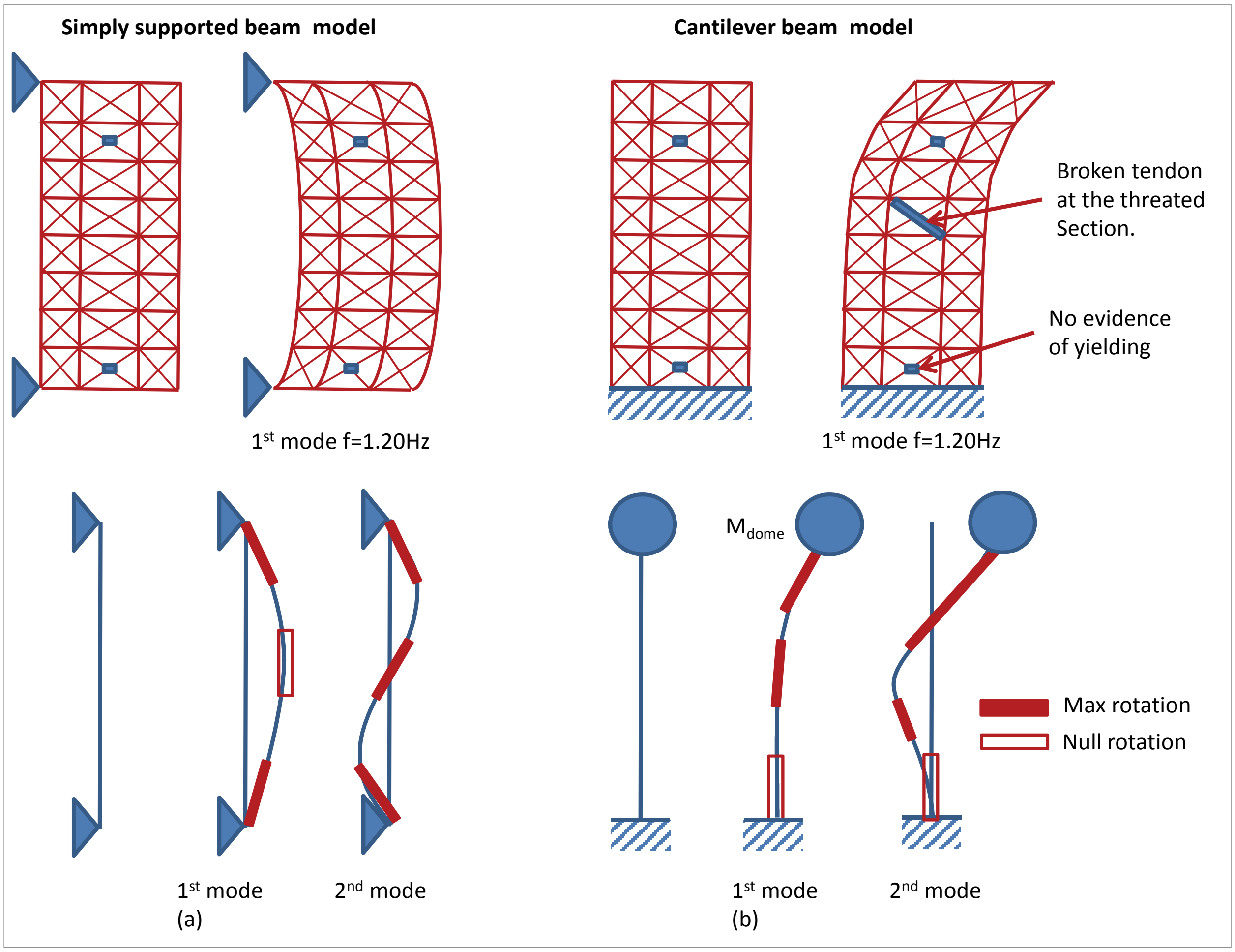


Figure 5-37 Simply supported beam model vs. cantilever beam model

Figure 5-38 No evidence of yielding in the steel hysteretic device and of any damage in the West side of the Church was observed

Figure 5-39 Broken tendon at the threaded section

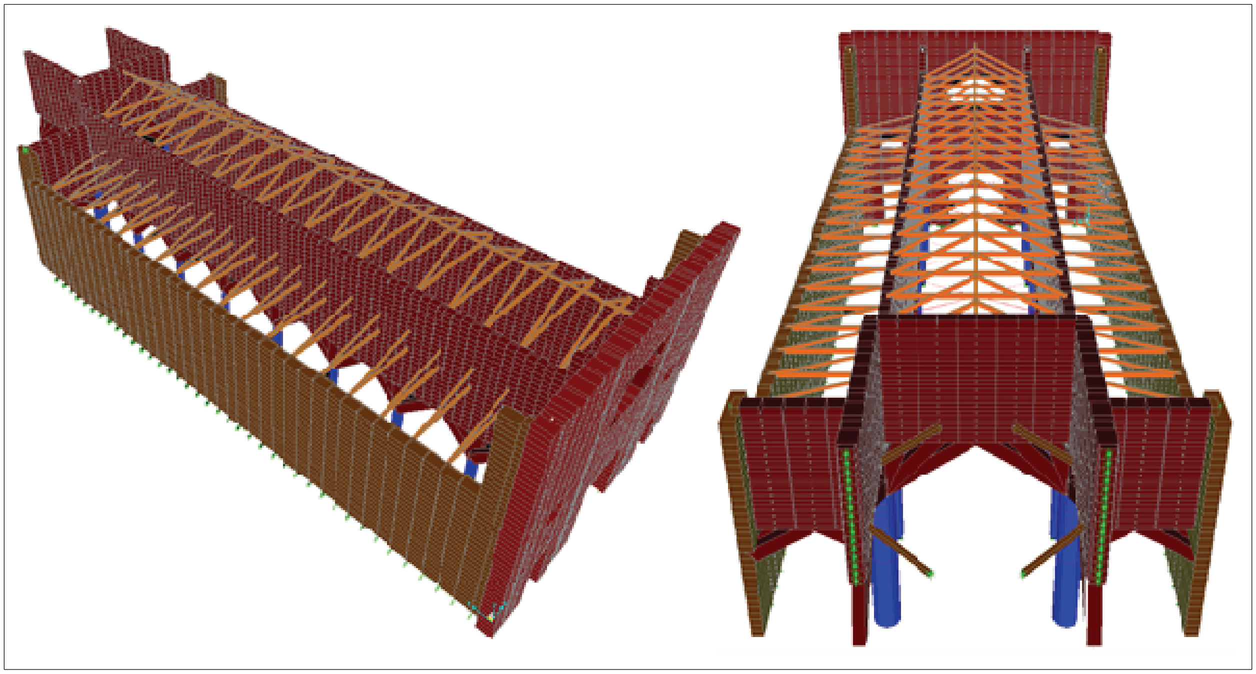


Figure 5-40 FEM model of the Church

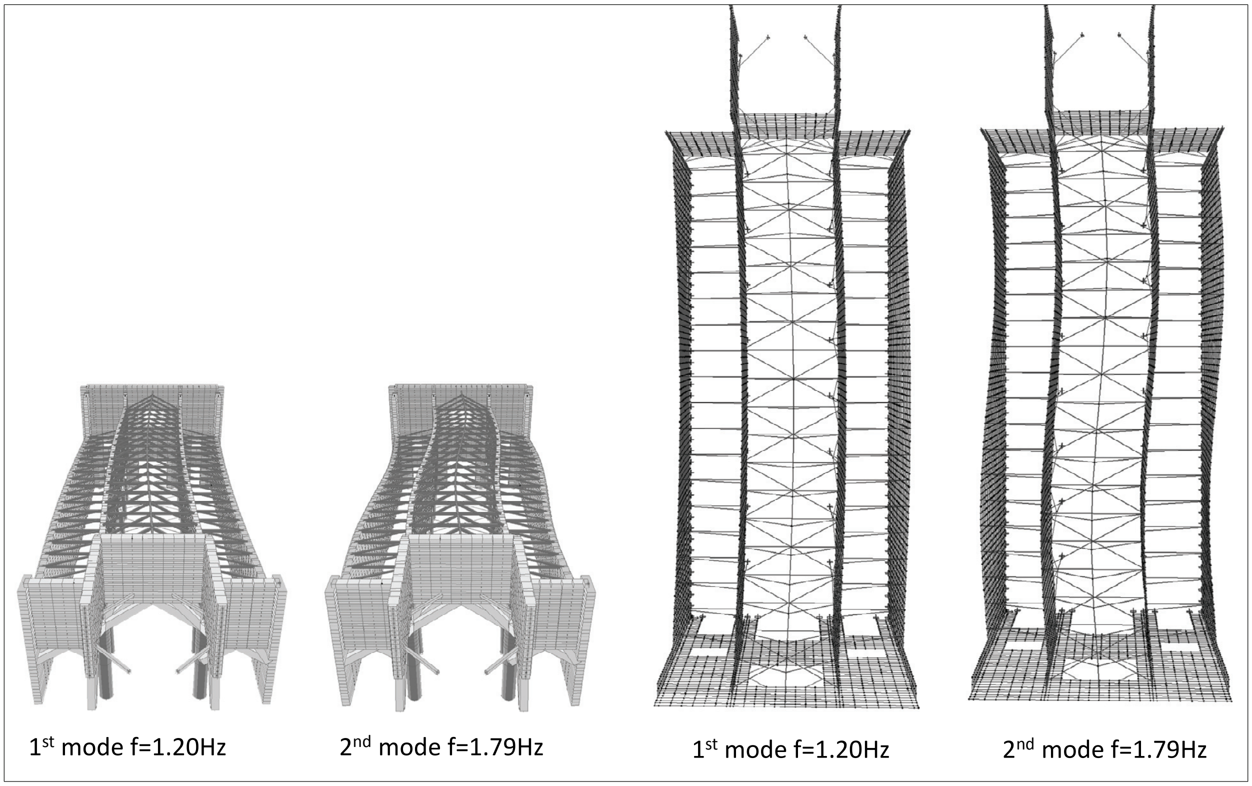


Figure 5-41 Calculated natural mode shapes of the Basilica before retrofit

The global behavior of the basilica during the earthquake is illustrated in Figure 5-34, where the presence of the horizontal diaphragm at the wooden roof level generated a stiffer nave with the consequent translation of the stiffness center (CK) toward the frontal façade. The 2000 retrofit generated a stiffer nave with respect to the transept where most of the mass of the basilica (CM) is concentrated; therefore, the dampers included in the basilica generated an irregular structure, by moving the center of stiffness away from the center of mass which in turn initiated a global rotational behavior of the church during the earthquake around the frontal facade. Evidence of the global behavior described in Figure 5-34 can be found in several parts of the church, and are summarized in Figure 5-42.

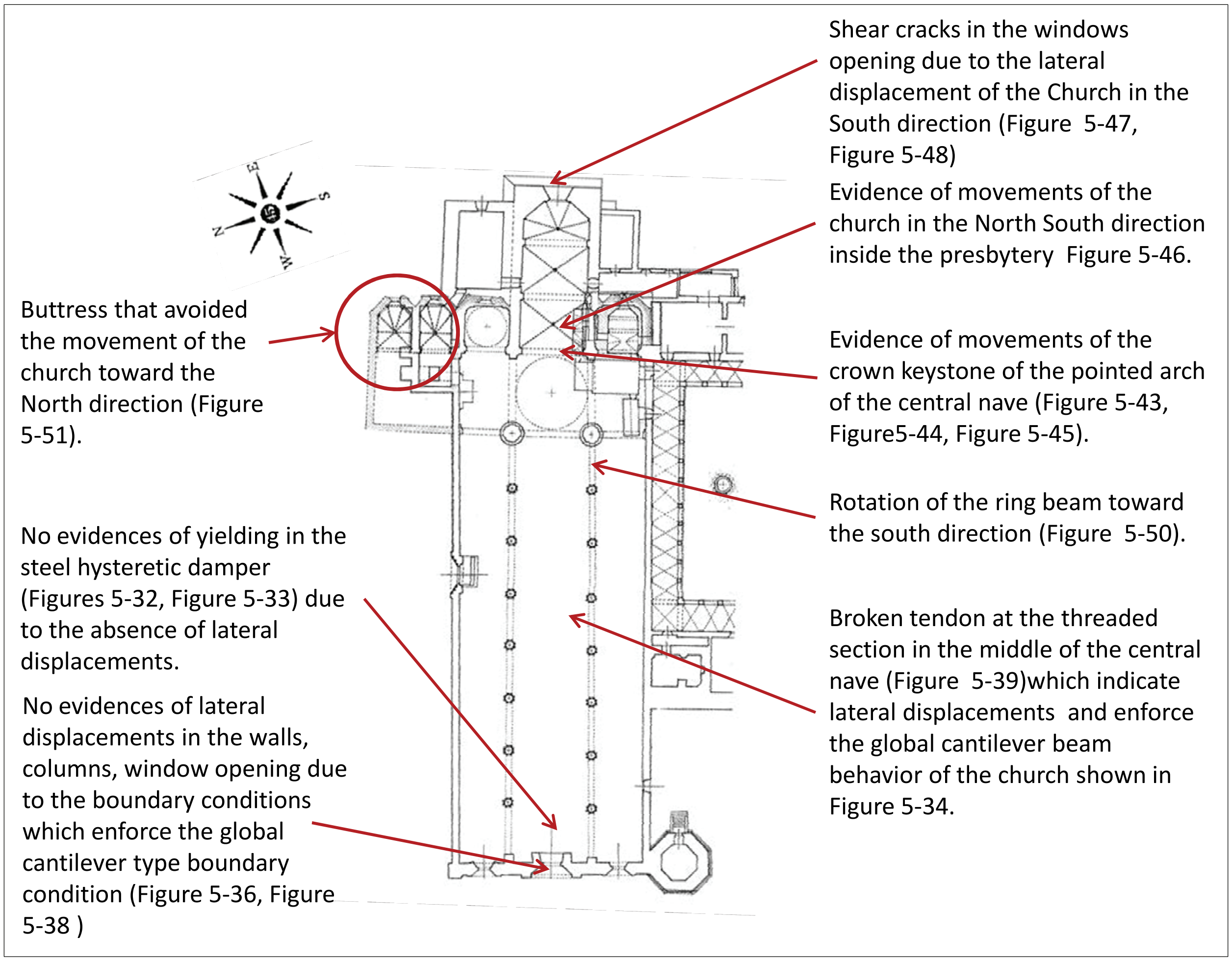


Figure 5-42 Evidence of the global behavior of the church described in Figure 5-34

Figure 5-43 Evidence of movements in the North South direction in the central altar

The pointed arch (Gothic arch) of the central altar shown in Figure 5-43, Figure 5-44 and Figure 5-45 is formed by two curves meeting at the crown with a sharp angle. From Figure 5-44, it is possible to clearly see that the arch collapse mechanism is due to the presence of four plastic hinges at the intrados and extrados which are large enough to transform the arch in a mechanism. Related damage is shown in Figure 5-46, Figure 5-47, Figure 5-48, Figure 5-49, Figure 5-50, and Figure 5-51.

Figure 5-44 Evidence of the lateral displacements in the South direction in the pointed arch and in the presbytery vault

Figure 5-45 Detail of the North South movement of the church in correspondence of the crown keystone of the pointed arch of the central altar

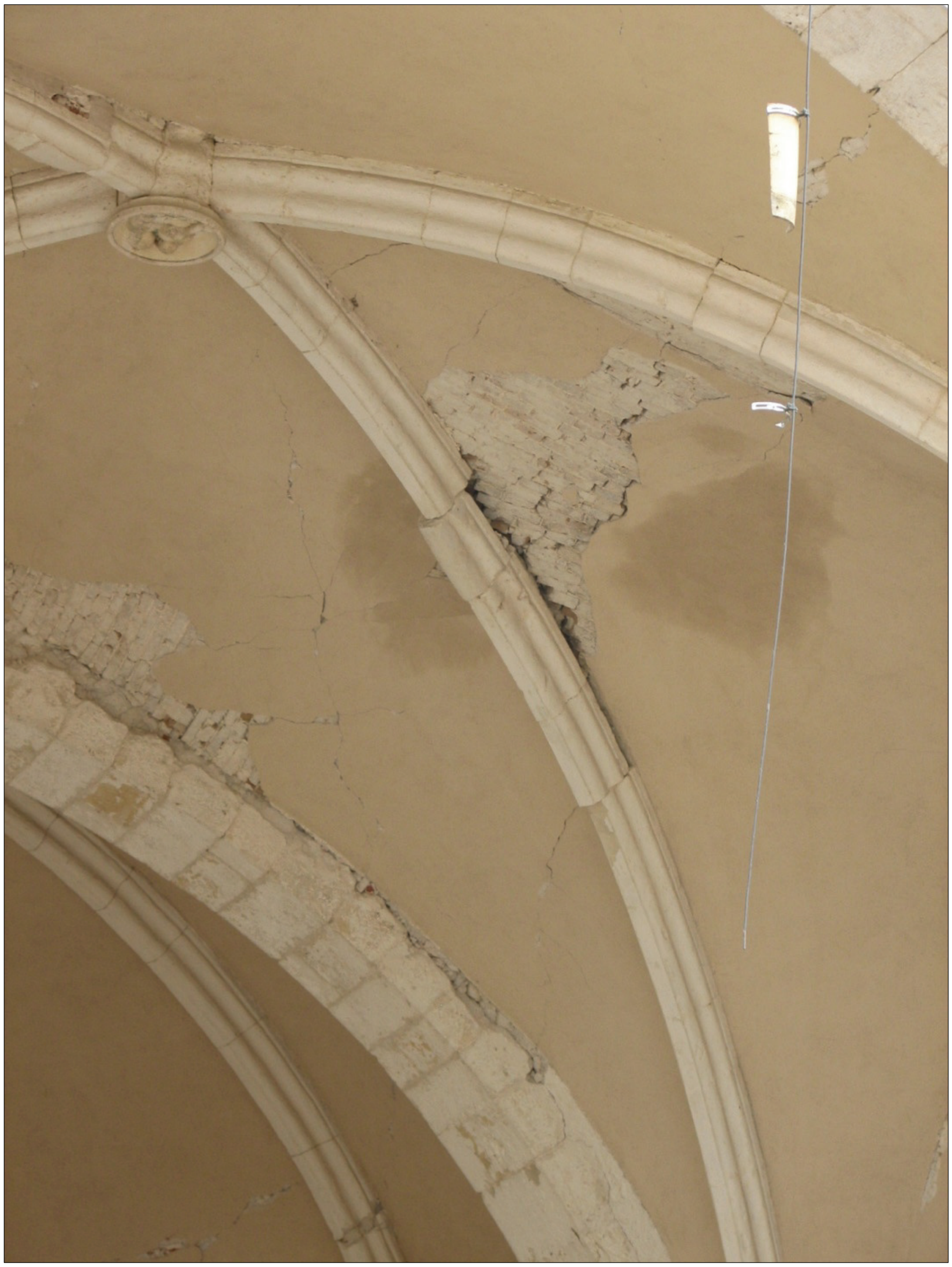


Figure 5-46 Evidence of the lateral moment in the North South direction inside the presbytery

Figure 5-47   Shear cracks due to lateral displacement in the wall of the central altar initiated by the presence of the window opening

Figure 5-48   Shear cracks due to lateral displacement in the wall of the central altar initiated by the presence of the window opening

Figure 5-49 Damage to the altar

Figure 5-50 Evidence of movement of the church toward the South direction due to the twisting of the ring beam in the South direction after collapse

Figure 5-51 Buttress in the North side of the Church which avoided the movements toward that direction

5.2.1.6 Assessment of the Performance of the Diaphragm Behavior

The initial wooden roof system of the nave before the retrofit was flexible and the resultant lateral loads due to the inertial forces during the earthquake were transferred to the portion of the wall just below the transversal truss. In other words, the tributary mass acting on the boundary was limited as shown in Figure 5-52 a.

Following the retrofit, the additional cross horizontal tendons introduced below the roof generated a diaphragm behavior in the nave which increased the tributary mass acting on the boundary shown in Figure 5-52 b and therefore the inertial forces applied at the boundary of the transept (in the frontal façade and in the pillars supporting the dome).

The failure of one side of the church probably was due to the concentration of seismic forces through the horizontal truss which established a diaphragm behavior at the roof level. The tributary area (mass) of the roof increased and therefore the seismic horizontal forces transferred at the supports also increased with respect to the inertial forces before retrofit.

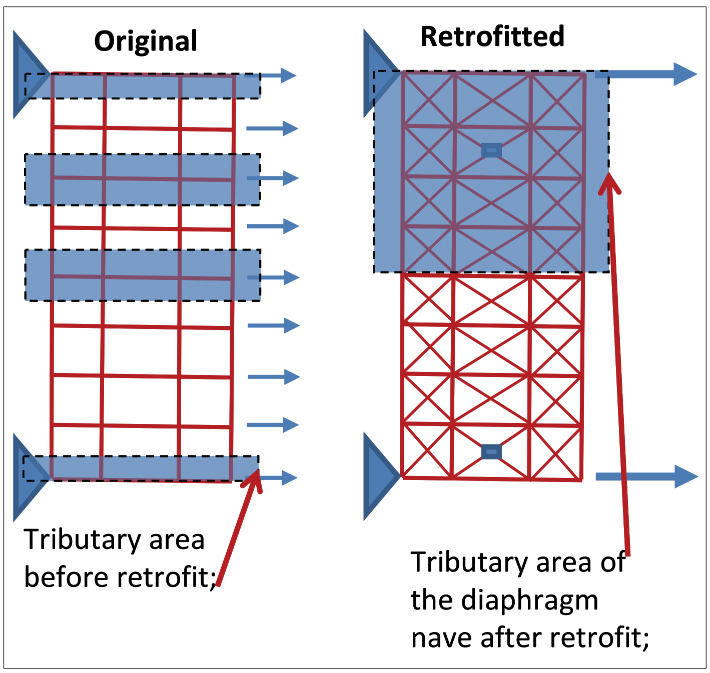


Figure 5-52 Increase of the tributary mass due to the retrofit

5.2.1.7 Concluding Remarks

The field investigation following the L'Aquila earthquake provided evidence of the ineffectiveness of steel dampers that was contrary to the good performance observed in the laboratory under controlled conditions.

In summary, the reasons that might have caused the collapse of the Church can be summarized below:

1. The 2000 retrofit strengthened and stiffened the church as shown in Antonacci et al. (2001) where the first frequency shifted from 1.20 Hz before retrofit to 1.41 Hz after

retrofit, while an increment of the second mode is from 1.79 Hz before retrofit to 2.24 Hz after retrofit.

2. The period shift of the main mode due to the retrofit generated larger reaction forces at the supports of the central nave because of increments of the spectral demand in the response spectra (Figure 5-35) obtained using the accelerograms recorded in the stations near the Basilica.
3. The failure occurred at the supports due to the global rotational movement of the presbytery and the transept shown in Figure 5-43 to Figure 5-46 from the North side toward the South side, which is shown by clear evidence of movements in the North South direction in the altar-transept area, while no evidence of movements can be found in the main entrance (Figure 5-36, Figure 5-38).
4. The global rotational movement is probably due to the contemporary presence of both the temporary scaffolds at the entrance of the church and the improper location of the steel hysteretic devices in the nave (Figure 5-32, Figure 5-33). In fact, the unlikely presence of a scaffolding system (Figure 5-53 and Figure 5-54) in the frontal façade when the earthquake occurred contributed to increase the distance between the center of mass (CM) and the center of stiffness (CK). Thus, torsion forces developed that caused the collapse of the two central pillars (*Figure 5-25*) and consequently of the dome above.
5. The retrofit developed a diaphragm behavior of the central nave which generated an increment of the tributary mass and therefore of the inertial forces transferred to the supports at the end of the nave (frontal façade and the pillars supporting the main dome).

The 2000 retrofit based on stiffening and strengthening the structure may have been an important contribution to the main collapse of the transept, generating deformation incompatibility between the transept and the nave.

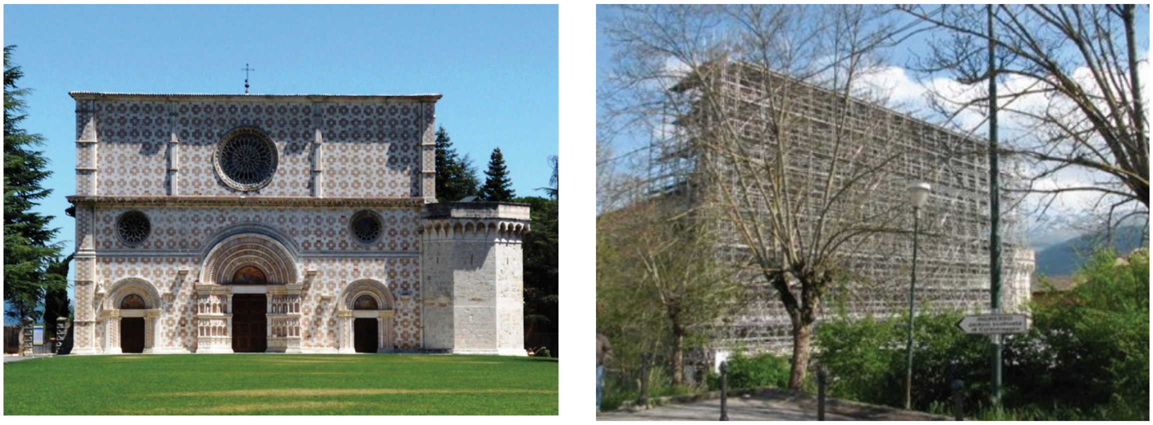


Figure 5-53 Facade of the church before the earthquake and at the moment of the earthquake

Figure 5-54 Structural scaffolds supported the front façade avoiding the collapse (Salvatori)

5.3 Public Historical Palaces

5.3.1 Case Study: The Spanish Fortress

5.3.1.1 History of the Spanish Fortress

The Spanish fortress of L'Aquila - commonly called "il Castello" by the Aquilans, is one of the most impressive Renaissance castles in central and southern Italy.

In the 15th century, L'Aquila had become the second most powerful city in the Kingdom after Naples; there were half a million sheep, wool and saffron exported throughout Europe. All this unfortunately was lost when the Aquilans sided with the French during the war between the French and the Spaniards for the throne of Naples. In 1504, L'Aquila was occupied by the Spanish conquerors. In 1527, the French recovered the city with the support of the citizens and the surrounding towns. But just a year later Viceroy Filiberto of Orange, ruling for King Charles V of Spain, finally defeated the Aquilan rebels and ordered the construction of a fortress in the highest spot, north of the city, exactly where in 1401 King Ladislas had built a garrison to control the unruly and rebellious Aquilans ("*ad reprimendam aquilanorum audaciam*").

The project was entrusted to a celebrated Spanish architect, Don Pirro Aloisio Escriva, a great expert of firearms, who had begun to build Castel Sant'Elmo in Naples. The discovery of gunpowder obliged to new methods of defensive construction. Escribà was in charge of the project for 2 years, leaving the task to Gian Girolamo Escribà.

In the following 30 years, the heavy taxes necessary to build the fortress impoverished the city, which in 1567 begged the Spaniards to stop the construction; the Royal Court granted the request, and works were interrupted, so parts of the castle were never completed. The Fortress had cost an enormous sum for the times, and Aquila was obliged to sell even the thick silver case containing the body of St. Bernardine from Siena.

The Fortress, which had been built not to defend the city, but to control it (cannons pointed to the city as shown in Figure 5-55) and to be a completely self-sufficient structure, was never used in a battle. Its cannons, always ready to fire, were silent throughout the centuries; the only victim was the city itself, whose decline began with the construction of the fortress and went on under the Spanish dominion.

Figure 5-55 Spanish Fortress facing the city of L'Aquila

Escrivà planned a giant fortress, made of four bastions connected through mighty walls, 60 m long, with a thickness of 30 m at the bottom and 5 m at top. The walls were surmounted by massive merlons, with openings for the archers and the long-distance cannons. All around the fortress was a ditch (never filled with water) 23 m wide and 14 m deep, aimed at defending the foundations from the enemy's artillery.

Escrivà did not forget any detail: the slanted walls would reject enemy fire to the sides; each bastion consisted of two separate and completely self-sufficient environments - called "case matte" - almost independent garrisons on their own. Also the aqueduct to the city was deviated so as to supply the fortress first of all, and in case of rebellion, block the water supply.

Moreover, Don Pirro planned a special anti-mine corridor, a kind of empty space between the outer and inner walls which could be walked only by one man at a time (and which can be visited today), aiming at defending the castle in case of explosion in the event that enemy soldiers excavated tunnels to leave mines at the foundations. A whole hill was leveled down to supply the white stone necessary for the fortress, while the city's bells were melted to make the cannons.

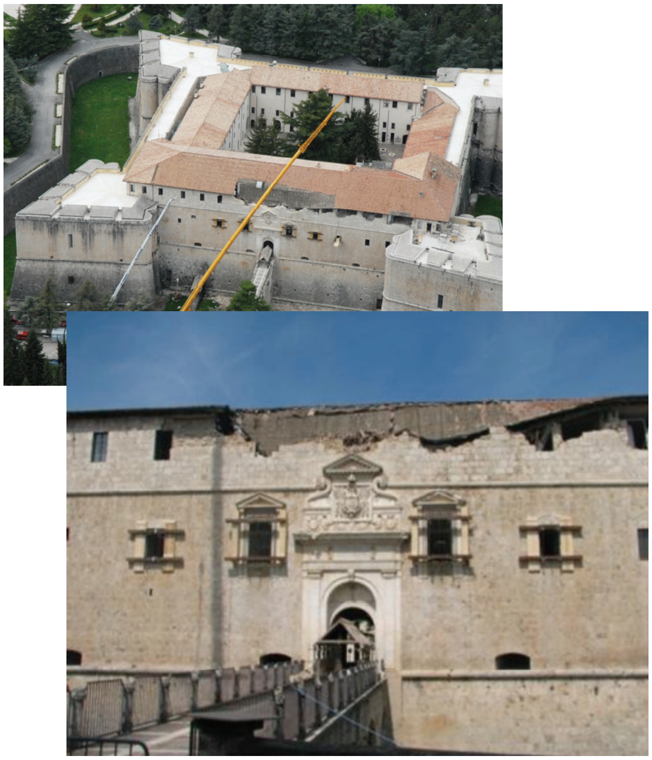
In 1798 the citizens fought against the French who had invaded Italy, attacking, in vain, the fortress. From then on, the building was used as a prison. After 1860 it became a military headquarters, and in the Second World War was occupied and damaged by the Germans. Between 1949 and 1951 the castle was restored, and chosen as the seat of the Museo Nazionale d'Abruzzo.

5.3.1.2 Field Damage Reconnaissance of the Spanish Fortress

The observed damages in the Spanish Fortress are mainly in the new addition to the building as shown in Figure 5-56 and listed in Table 5-1. The details are described in the following sections.

Table 5-1 Observed Structural Damage: Spanish Fortress

Structural damages
Partial collapse of the floors
Partial collapse of the roof cover
Partial collapse of the upper lateral walls
Severe vertical cracks on the vertical walls
Severe overturning of the arcade and of the pillars on the ground floor
Severe damage to the historical monuments of the National Museum of Abruzzo



Damage:
•partial collapse of the floor slab;
•Partial collapse of the roof;
•Partial collapse of the upper lateral wall;
•Severe cracks on the vertical walls;
•Severe overturning of the arcade (loggiato) and of the columns on the ground floor;
•Severe damage to historical heritage inside the National Museum of Abruzzo;

Estimation of Economic losses:
• 50.000.000 Euro;

Estimated Recovery time:
• 8 years;

Figure 5-56 Field damage reconnaissance of the Spanish Fortress

Figure 5-57 and Figure 5-58 show the partial collapse of the upper lateral wall due to the new addition in the old building that is also visible in Figure 5-70. The collapse was triggered by the openings of the lateral walls that make the wall weaker and together with the poor quality of the masonry generated the partial collapse of the roof.

Figure 5-57 Partial collapse of the upper lateral wall

<u>Damage:</u>
• Partial Collapse of the roof coverage;
<u>Mechanism:</u>
• rolling out the plan due to horizontal flexure of the upper part of the façade;
<u>Structural causes:</u>
• Poor connection between the roof slab and the lateral wall weaker due to the presence of openings;
• Poor quality of the masonry;

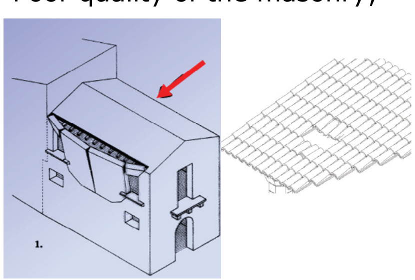


Figure 5-58 Damage to the roof coverage

The internal walls of the courtyard overturned (Figure 5-59, Figure 5-61), generating typical cracks in the cross vaults of the arcades (Figure 5-60) and severe damage in the columns due to axial-flexural behavior (Figure 5-62). Evidence of this mechanism can be found in the internal rooms on the second floor and in the corridor (Figure 5-63), where longitudinal cracks are present at the floor level nearby the windows and in the barrel vault of the corridor (Figure 5-64).

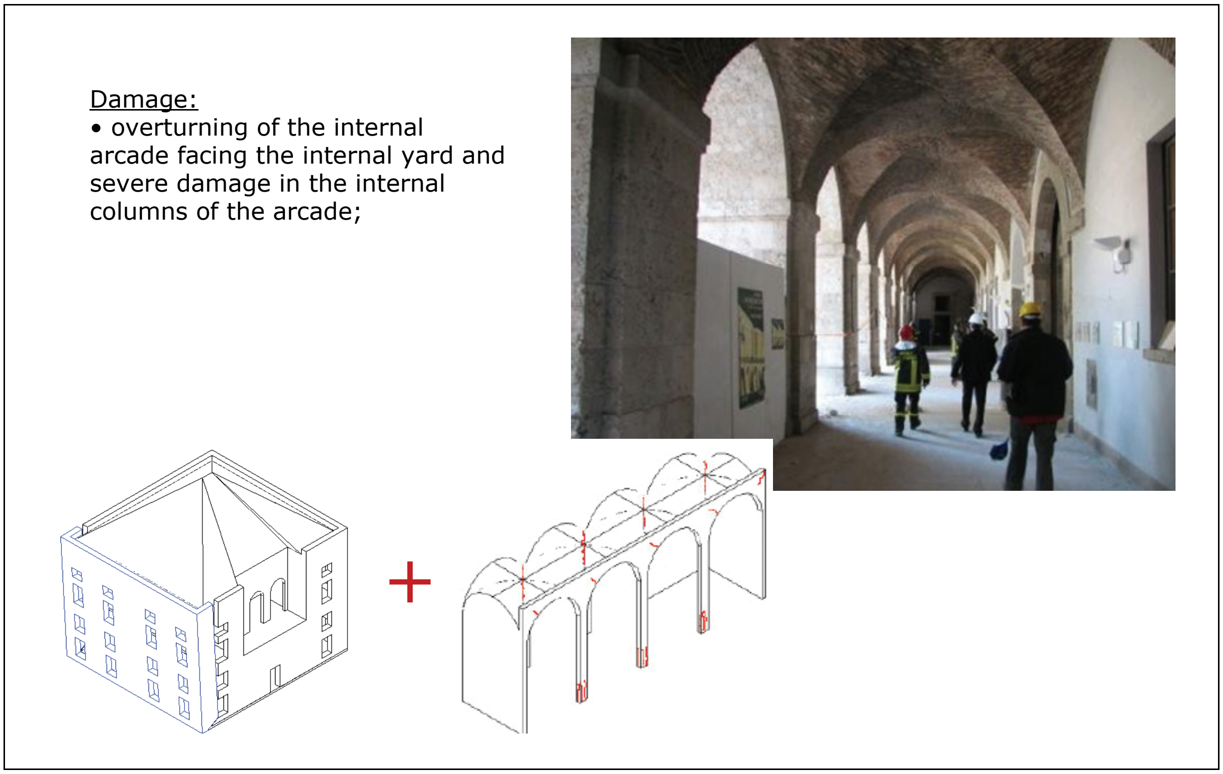


Figure 5-59 Overturning of the arcade

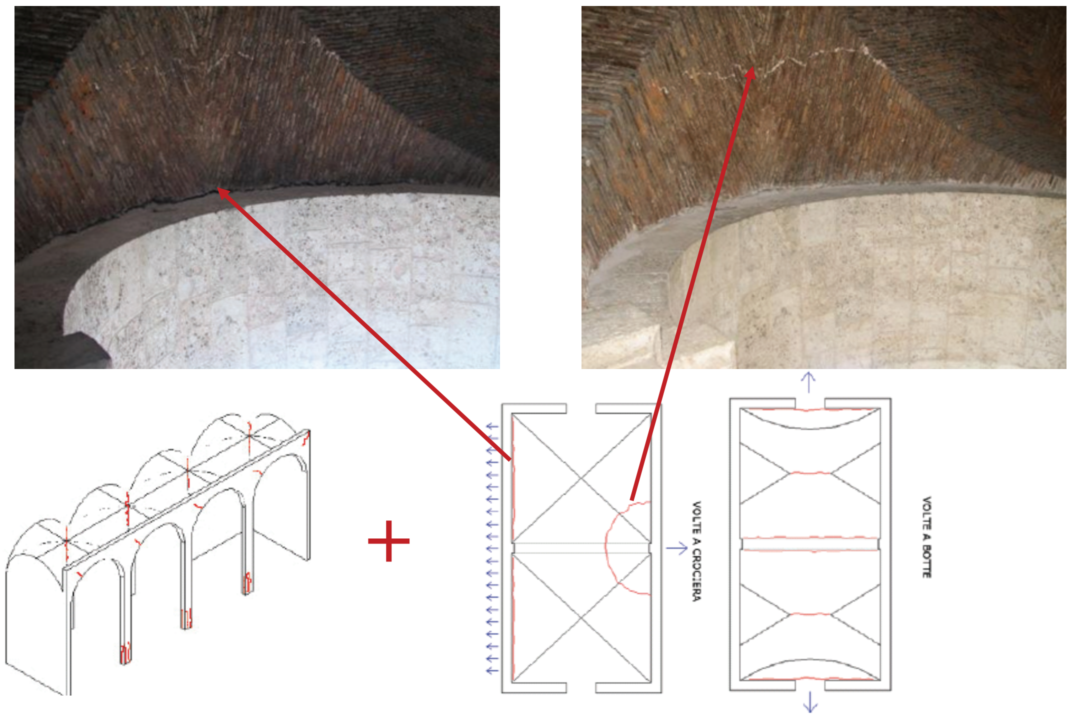


Figure 5-60 Damage to arcade

<u>Damage:</u>

• overturning of the internal arcade facing the internal yard and severe damage in the internal columns of the arcade;

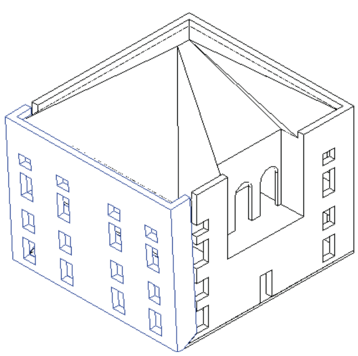


Figure 5-61 Overturning of the internal wall of the courtyard

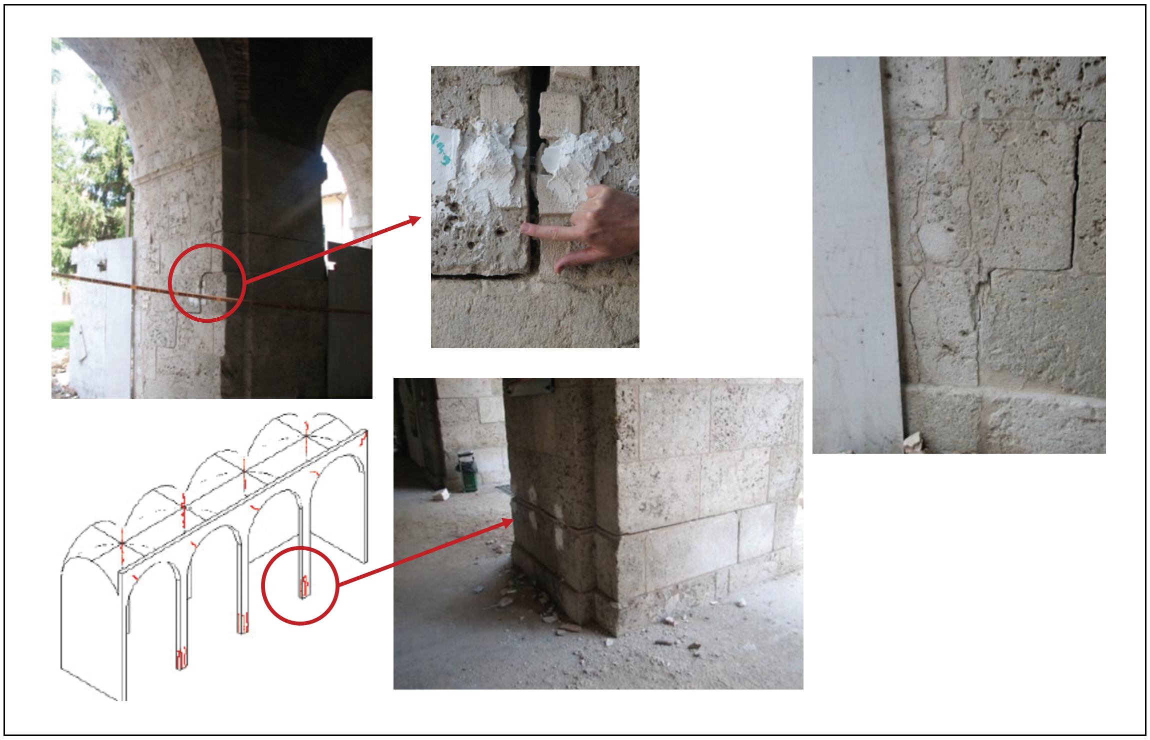


Figure 5-62 Damage to the columns of the arcade

Figure 5-63 Internal evidence of the overturning walls

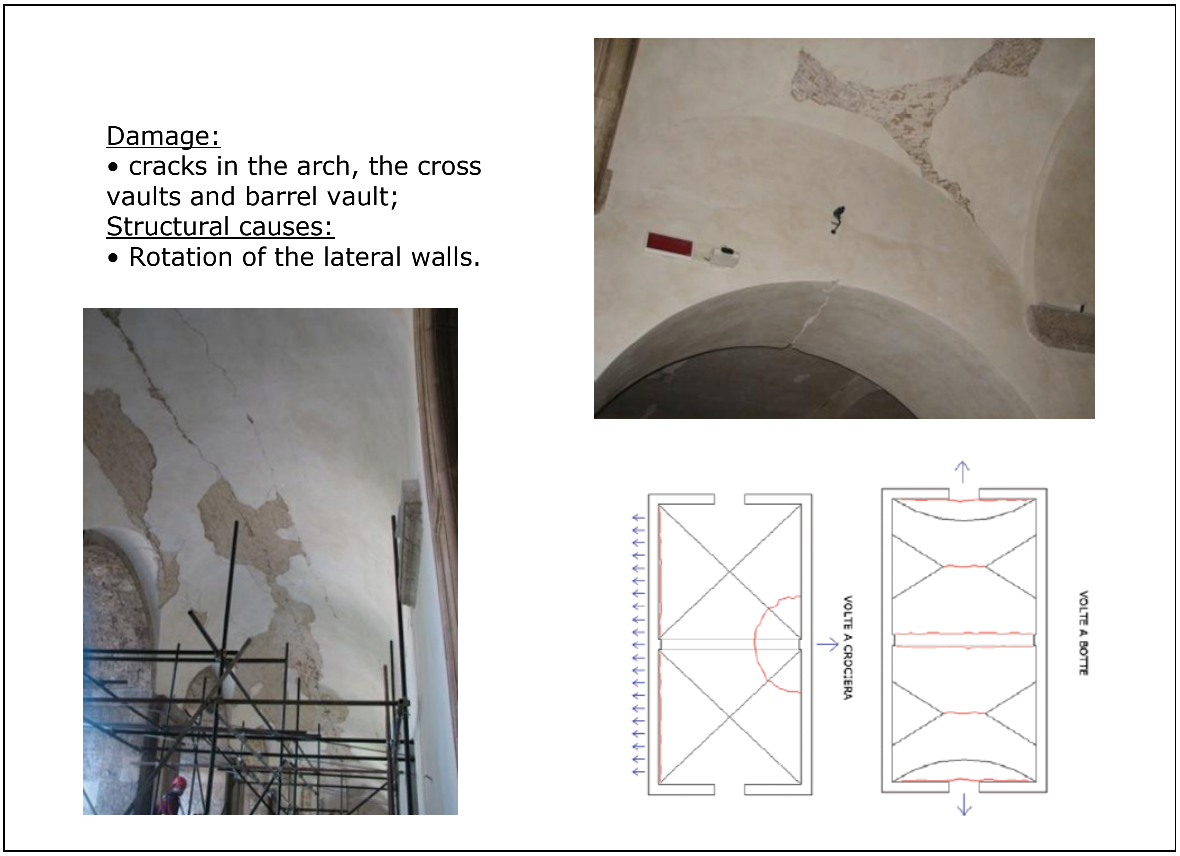


Figure 5-64 Damage to the vaults

Typical X-shaped cracks were observed in the external walls of the internal court yard. They are located mainly below and above the window openings as shown in Figure 5-65 and Figure 5-66. The shear cracks in some cases have been triggered by the presence of tie rod systems above the windows openings as shown in Figure 5-67.

These shear cracks in the walls do not affect the stability of the building that is able to carry the gravity loads.

Damage:
• typical cross cracks in the wall strips above and below the window openings;

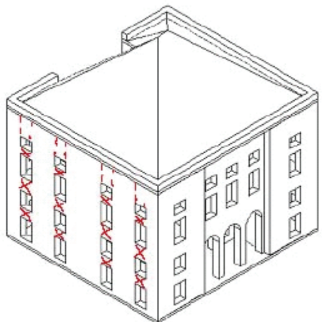


Figure 5-65 Damage in the wall strips

Damage:
• typical cross cracks in the wall strips above and below the window openings;

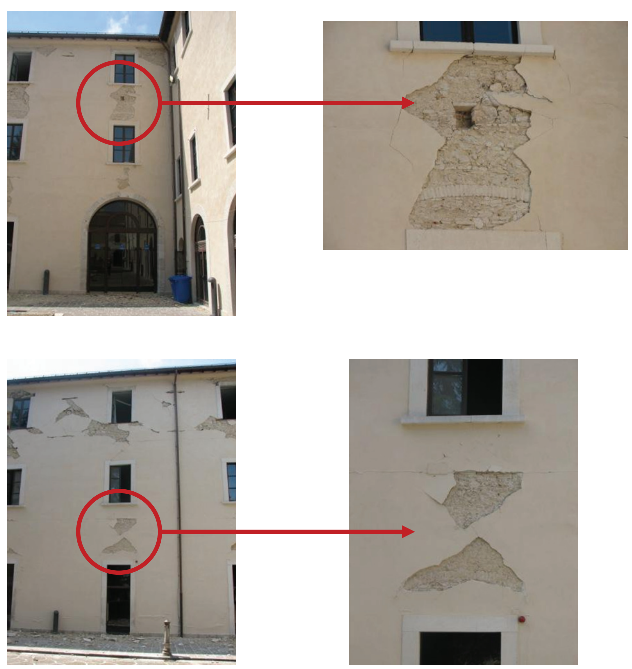


Figure 5-66 Details of damage in the wall strips

Figure 5-67 Damage in the lateral strip walls

Inside the fortress, damages have been observed near the door openings with collapse of the lintels that have been shored up with restraints props (Figure 5-68).

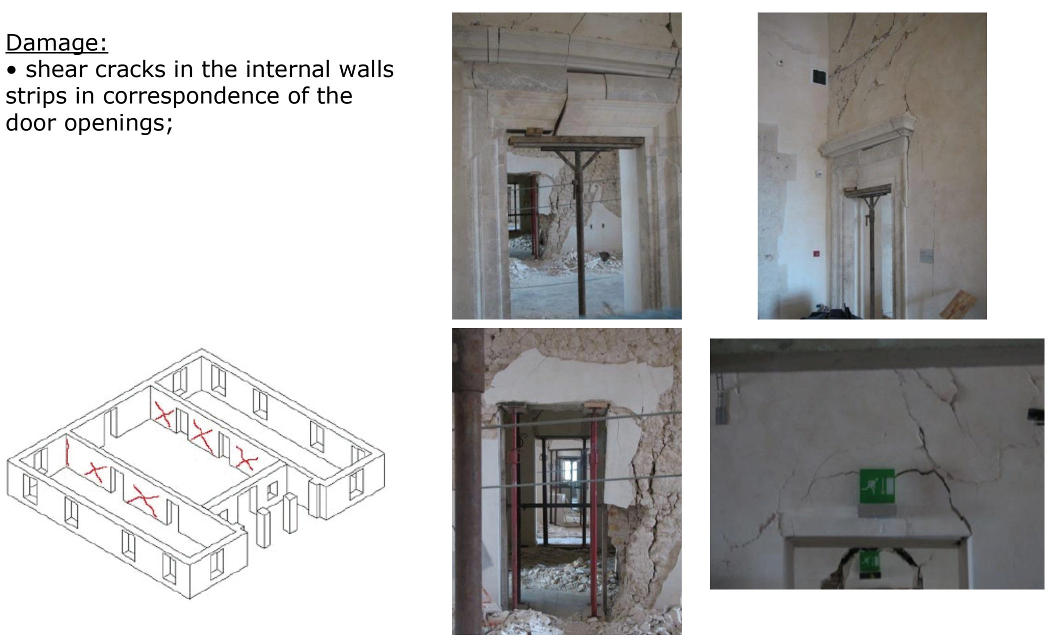


Figure 5-68 Damage in the lintels

The presence of a large quantity of debris was found near the stairs (Figure 5-69) due to the collapse of one of the vaults. This situation generated additional overloads on the stairs that need to be removed to avoid the collapse of the vaults on the lower level.

Figure 5-69 Damage to the stairs

Shoring up interventions started right after the earthquake and, as shown in Figure 5-70, the majority of damages were concentrated in the third floor due to an additional stiffer part in the fortress that was the location of the National Museum of Abruzzo. The majority of masterpieces and statues were rescued by firemen (Figure 5-71) through the opening that was generated on the third floor using cranes from the outside (Figure 5-72).

Figure 5-70 Damage on the third floor

Figure 5-71 Damage to the historical heritage of the National Museum of Abruzzo

Figure 5-72 Damage to the historical heritage of the National Museum of Abruzzo

The main shore up retrofit intervention in the castle consisted of the insertion of steel rods through the windows to connect the external walls facing the courtyard with the external lateral walls of the castle (Figure 5-73). In this way, it was possible to control the overturning mechanism of the internal lateral walls of the court yard. Additionally, in the main entrance of the castle, a wooden roof cover was built (Figure 5-74) to avoid the risk of being hit by falling debris material that was present in the upper part of the external lateral walls.

Figure 5-73 Shoring up of the Spanish Fortress

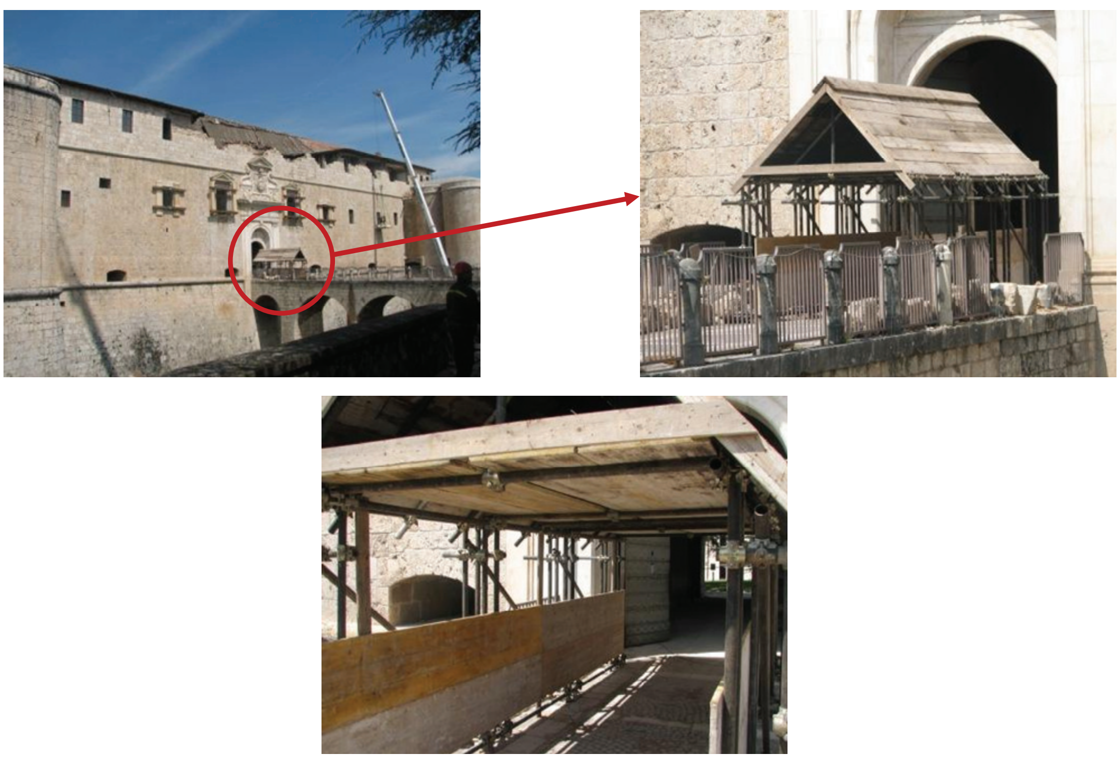


Figure 5-74 Shoring up of the Spanish Fortress

5.3.2 Case Study: The “Quattro Cantoni” Complex

The complex was designed by Alessandro Mancini and built in the late 19th century, reshaping the Jesuit college of the 16th century and using Palladio and Sansovino style (Figure 5-75). The remaining parts of the medieval monastery of St. Francis are: the bell tower, a chapel and the cell of S. Bernardino. The Royal School of Abruzzi was established in 1817 in L'Aquila from Ferdinando I Borbone, in the ex-Franciscan convent. It was the most ancient monastery of the city, built around 1250 with four yards and the church of S. Francis. The convent hosted the cell that still exists where San Bernardino of Siena died in May 20, 1444. In 1877, the St. Francis church was demolished and in the area it was decided to arrange a library independent of the provincial high school. Now the complex is formed by Provincial Library, the High school and the Convitto Nazionale (Figure 5-76). Nowadays, the complex is made of stone masonry. The floors are made with vaults and reinforced concrete beams. The detected damage is listed in Figure 5-77.

Figure 5-75 Quattro Cantoni block

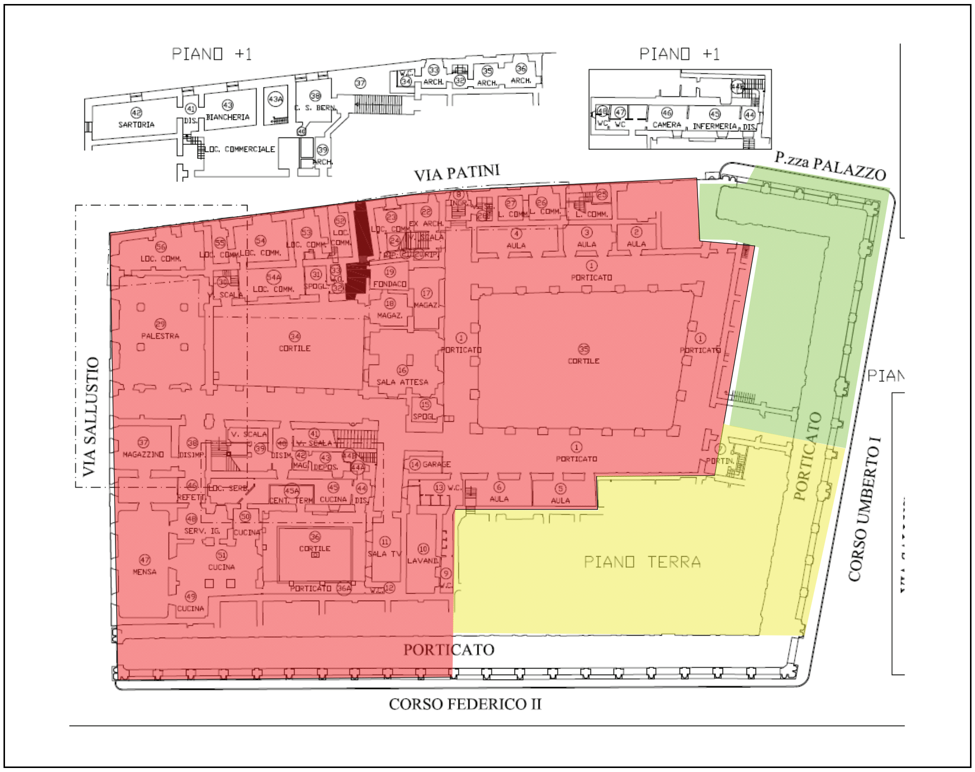


Figure 5-76 Plan view of Quattro Cantoni block

<u>Damage:</u>
•Severe overturning of the façade with signs of compressions on the column of the ground floor;
•Severe shear cracks on the external lateral walls;
•The internal vaults in the stairs are seriously damaged;

<u>Estimation of Economic losses:</u>
• 25.000.000 Euro;

<u>Estimated Recovery time:</u>
• 8 years;

Figure 5-77 Damage assessment of the Quattro Cantoni complex

The principal observed damage in the Quattro Cantoni complex is the simple overturning wall mechanism that is a collapse mechanism characterized by a rigid rotation of portion of the wall around a horizontal cylindrical hinge located at the base, as shown in Figure 5-78.

Figure 5-78 Overturning of the frontal façade of the National library

This rotation initiated another type of damage due to axial compression of the columns of the frontal façade of the National Library (Figure 5-79) that has also been characterized by pounding effects with the adjacent building of the Quattro Cantoni block (Figure 5-80).

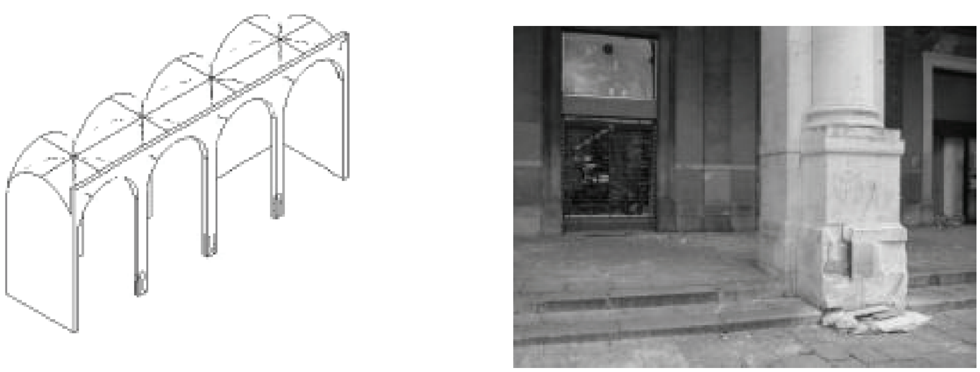


Figure 5-79 Damage due to axial compression at the base columns initiated by the rotation of the frontal façade of the National library

Also, the stone columns of the external arcades in Corso Federico II were damaged due to bending and shear, and are now confined with carbon fiber polymers (Figure 5-75).

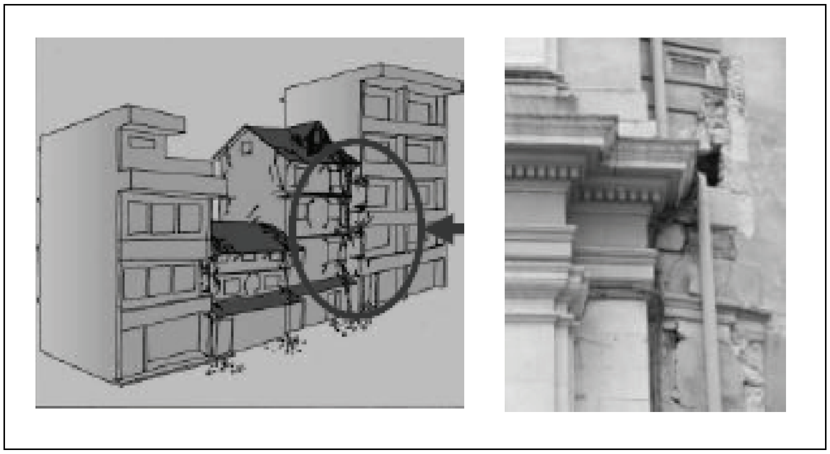


Figure 5-80 Pounding effects between adjacent buildings

The other main external damage mechanism has been observed in Via Sallustio (Figure 5-81 and Figure 5-82) where the entire external wall is characterized by the horizontal flexure damage mechanism that is probably initiated by the hammering effect of the roof beams on the top of the lateral wall.

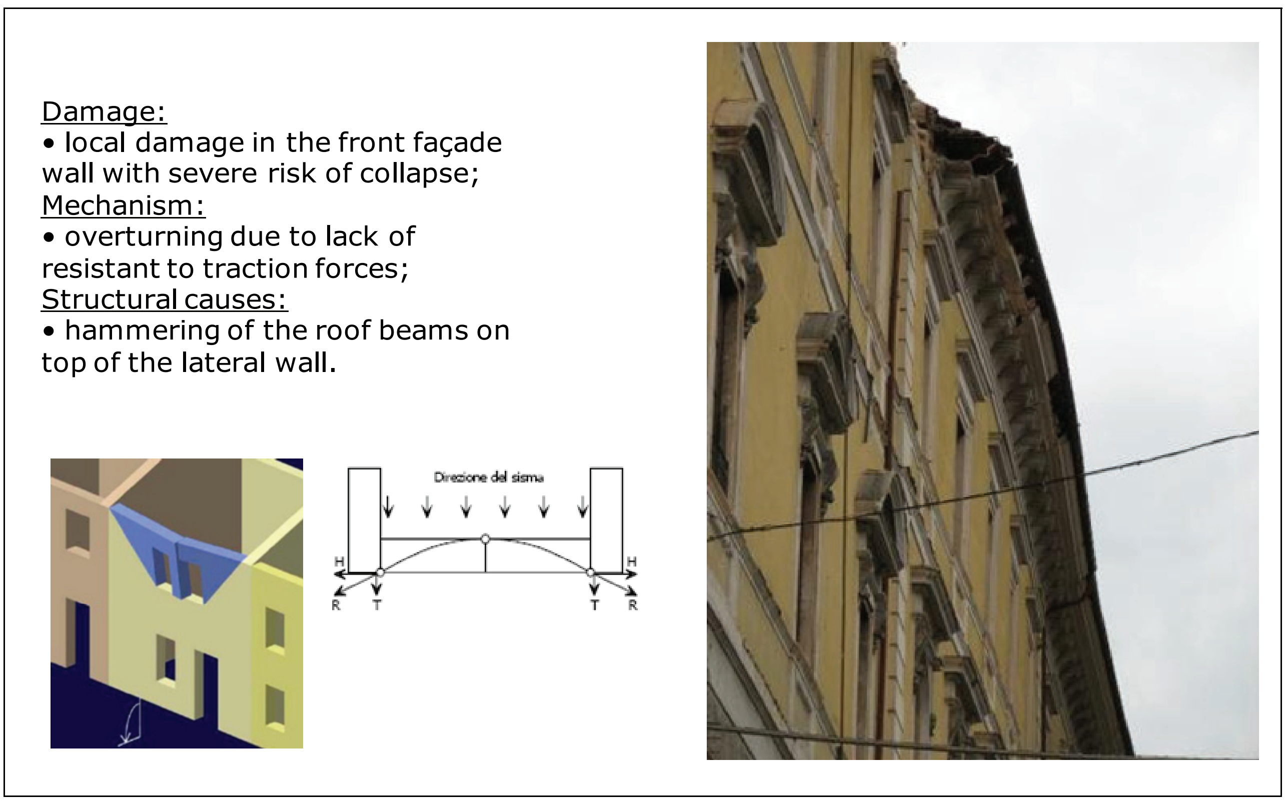


Figure 5-81 Horizontal flexure damage mechanism in Via Sallustio

Figure 5-82 Horizontal displacement of the frontal façade of Via Sallustio of the National Convitto

Finally, the most representative damage mechanism of the complex appeared in the cantonal between Frederick II and Corso Umberto I street where the overturning mechanism was promptly retained by the work done by the firefighters (Figure 5-83).

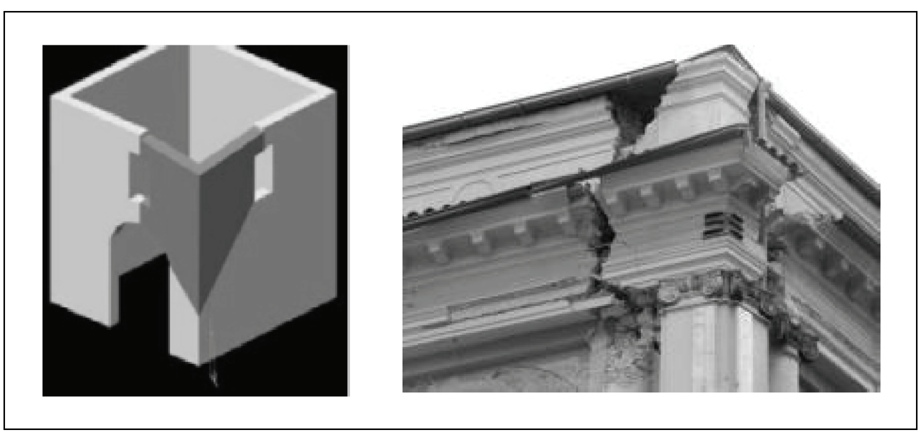


Figure 5-83 Overturning of the corner walls

From an aerial view before (Figure 5-84) and after the earthquake (Figure 5-85) it is clear that the gabled roof is severely damaged with collapses at several places.

Figure 5-84 Aerial view of Quattro Cantoni block before the earthquake

Figure 5-85 Severe damage to roof coverage

Several vaults in the arcades of the internal yards of the complex collapsed due to poor construction quality and rotation of the vertical lateral walls as shown in Figure 5-86.

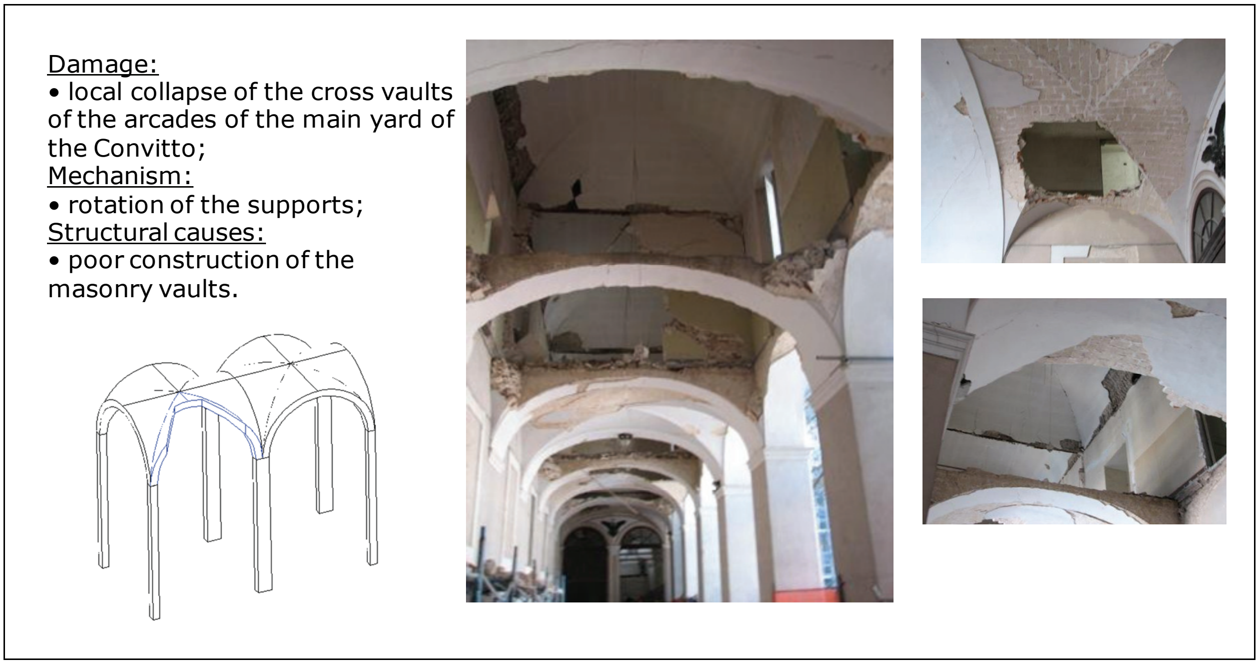


Figure 5-86 Total collapse of the vaults of the arcades of the main internal yard

The poor connection between the upper lateral walls and the roof coverage generated overturning horizontal flexure mechanisms (Figure 5-87).

Figure 5-87 Overturning of the upper lateral wall

Irregular vertical shapes of the lateral walls of the internal yard, combined with poor masonry quality and improper location of tie rods, generated collapse of the upper lateral walls (Figure 5-88). From these damages it can be summarized that vertical and horizontal asymmetry of the blocks and the lack of ring beams and tie rods in the upper part of the masonry are vulnerability points that impacted on damage.

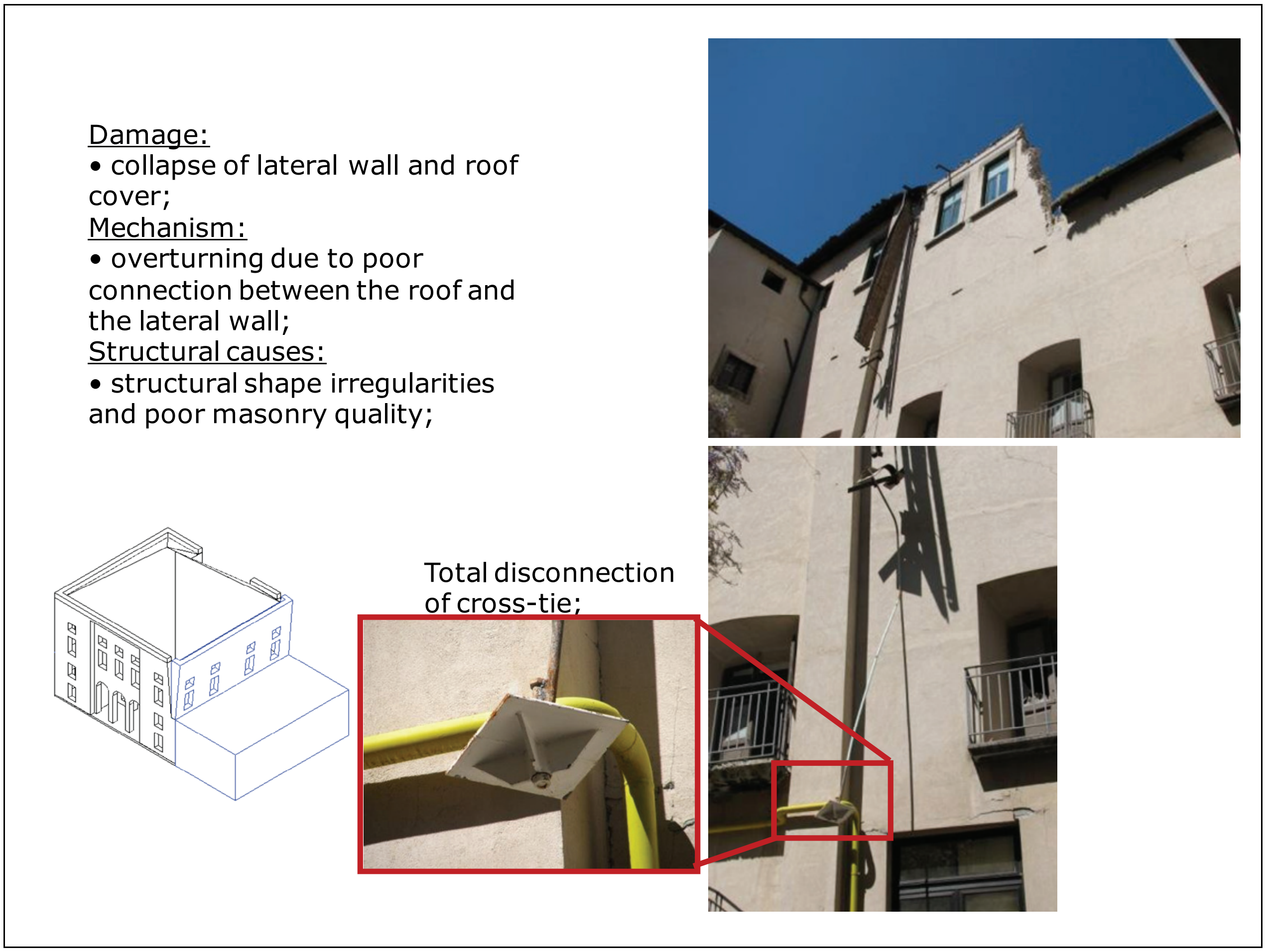


Figure 5-88 Damage due to irregular shape

Inside the complex there is also a church and Figure 5-89 shows the damage of the tower bell. Figure 5-90 shows cracks to the vaults of the stairs that were propped by firefighters.

Figure 5-89 Damage to the bell tower of the internal church

Figure 5-90 Cracks in the vaults of the stairs

Figure 5-91 shows the local collapse of the horizontal ceiling of the court yard due to the falling debris from the upper levels.

Figure 5-91 Collapse of the horizontal ceiling of the court yard due to a falling rock

5.3.3 Case Study: Palazzo Margherita

This section describes the damage classification of Palazzo Margherita, situated in the historical center of L'Aquila in Piazza Palazzo (Figure 5-92). The building is made of two levels with rectangular plan and internal courtyard (Figure 5-93). The structure is made primarily of stone and brick masonry with good quality mortar. The floors are made of masonry vaults with and without tie rods in both directions at the ground floor and of steel and brick at the first and second floor. Tie rods are in the north façade and at the two angles in the east façade (Piazza Palazzo) and west façade (Piazza Santa Margherita). There are no tie rods in the south façade. There are tie rods in the internal courtyard in the east-west façade, but they are not present in the north-south façade.

Figure 5-92 Front view of Margherita Palace (Civic Palace) in Piazza Palazzo XIV sec.

The building is located on a slope and has a height differential of 1.5 m between the north wall in Via Bafile and the south wall in Via delle Aquile. It also has a height differential of 2.40 m in the east-west direction because Piazza Palazzo is elevated compared to Piazza Margherita.

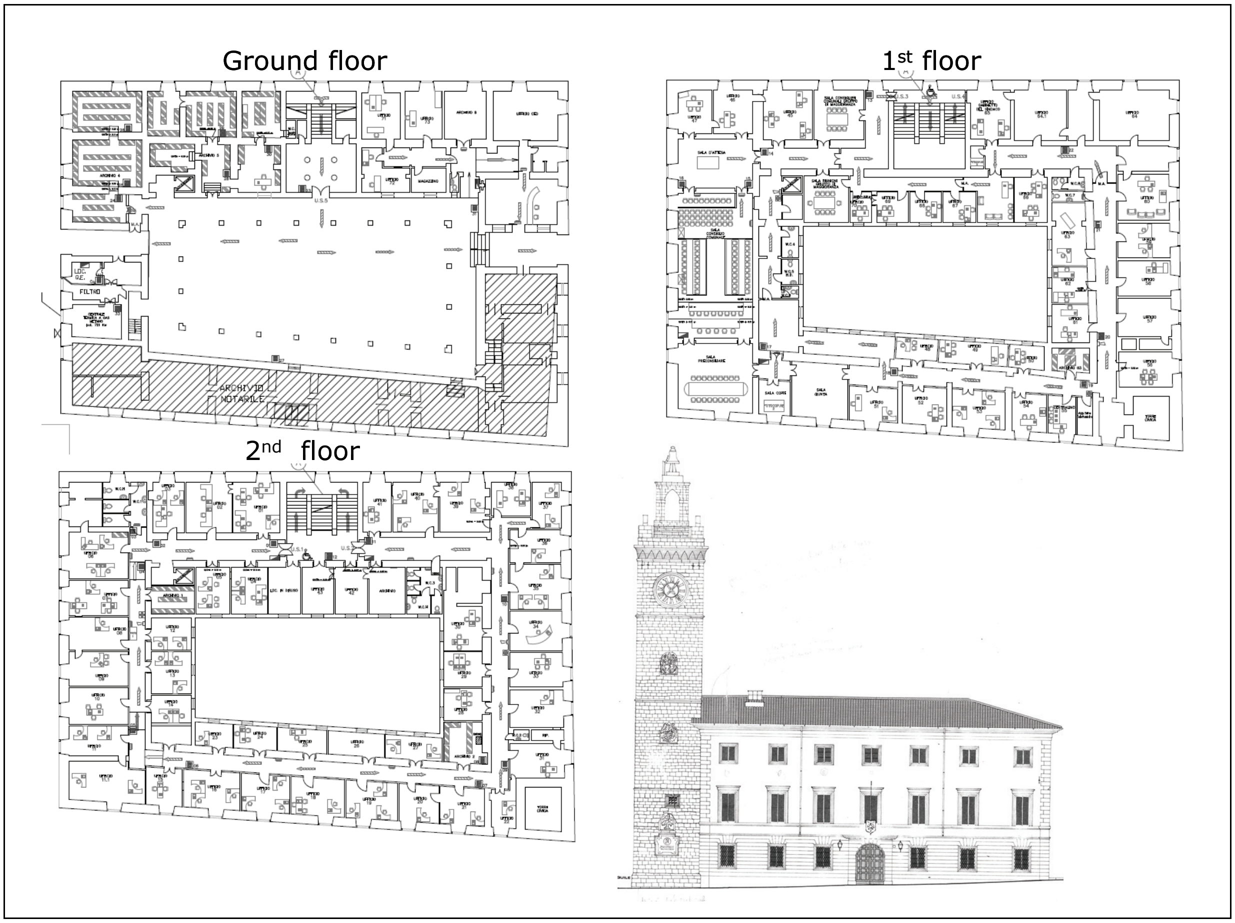


Figure 5-93 Plan views of the buildings and front view in Piazza Palazzo

The analysis of the seismic behavior of the building is characterized by significant margins of uncertainty; therefore this section identifies the most probable collapse mechanisms. Hypotheses on the collapse mechanisms need to be further justified by field tests and numerical modeling.

5.3.3.1 Previous Restorations

The construction of the palace started in 1294 and originally was used as a civic building for the Captain of Justice. Jerome Pico Fonticulano, architect, mathematician and writer, was the designer of the renovation of the Renaissance town hall in 1541, as indicated by a plaque on the facade.

In 1573, the building was almost completely rebuilt by Battista Marchirolo of Naples (decorated with stucco and interior design by Bedeschini and frescoes by Fantilli). It became the house of Margaret of Austria, daughter of Charles V, wife of Alessandro de Medici and later of Ottavio Farnese, who after being clever and energetic governor of the Netherlands was elected governor of Abruzzo. Unfortunately, it seems that the building does not retain much of the original design and of the reconstruction of the second half of the 16th century structures in the current reconstructed version of the 19th century. Only the plan settings and the monumental proportions, and the pleasant courtyard compound, are elements of the late 16th century that recall the importance of the place.

Of the ancient structure has remained only the tower, whose bell tolls to announce the closure of the city gates at night. Currently, the clock strikes 99 strokes each night in accordance with tradition to commemorate the founding of the city.

More recently in the early 90's, the architect Macri` restored Palazzo Margherita creating a ring beam at the top of the masonry walls behind the roof with the goal of giving a box behavior to the whole building.

5.3.3.2 Structural Systems and Materials

The different masonry typologies should be classified inside the building according to their quality. *Equilibrium Limit Analysis* can be used only with good quality masonry walls that avoid local collapse due to disaggregation. Inside Palazzo Margherita, there are different masonry types that have been partially classified in this paragraph. There is a brick arch inside the masonry wall in correspondence of each window facing Piazza Palazzo as the one shown in Figure 5-94.

Figure 5-94 Detail of the brick arch in correspondence of the windows of the East facade

Figure 5-95 shows the detail of squared stone blocks made of hard local limestone with which are made the columns of the internal arcade.

Figure 5-95 Details of the columns of the Arcade made of coarse limestone square blocks

Figure 5-96 shows the good quality of the masonry with which has been realized the low arch vaults of the stairs.

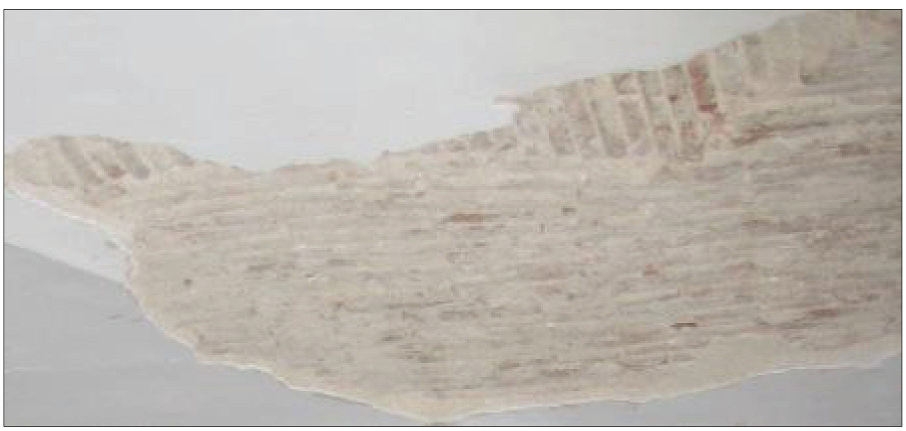


Figure 5-96 Detail of the masonry in the vault of the stairs

Figure 5-97 Detail of stone cladding in hard coarse limestone located in the Civic Tower

Figure 5-97 shows the hard limestone stone cladding in the entrance of the Chapel of the Civic tower located at the first level.

5.3.3.3 Field Damage Reconnaissance of Palazzo Margherita

The observed damage in Palazzo Margherita and the Civic Tower are given in Table 5-2 and are described in detail in the following paragraphs.

Table 5-2 Observed Structural Damage: Palazzo Margherita

Structural damages
Overturning of the external walls
Displacements of the floor slab and total collapse on the second floor
Damage to the columns of the arcades
Cracks of the internal walls
Damage on the external strip walls
Tale displacements and bending of the beams supporting the wooden roof
Damage in the facade due to hammering of the heating system that re-open the holes used in the past to support the old scaffolding system
Cracks in the Civic Tower at the basement level

The evaluation of the seismic behavior of Palazzo Margherita should be done considering the possible local damage mechanisms. In the old masonry structures, the collapse is determined by the lack of connections between orthogonal walls and by discontinuities not always visible. Therefore, attention is given to the local damage mechanisms that are described in numerous studies of Giuffre` (1993), Avorio & Borri (2002) and many others. The principal mechanism located inside Palazzo Margherita is the overturning wall failure.

Simple Overturning Wall Mechanism

This collapse mechanism is characterized by a rigid rotation of a portion of the wall around a horizontal cylindrical hinge located at the base. The overturning mechanism of the internal wall on the north side of the courtyard developed longitudinal cracks in the cross vaults of the arcade (Figure 5-98, Figure 5-99).

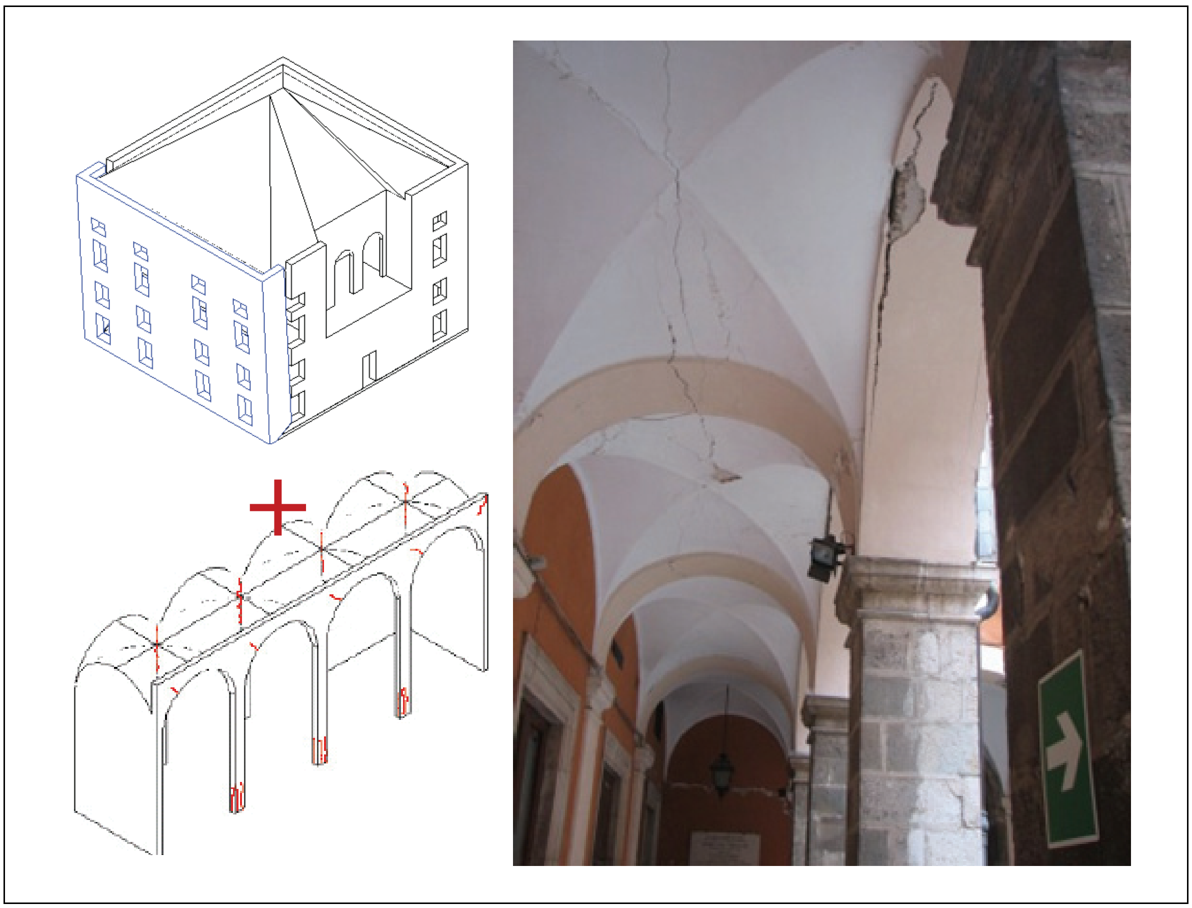


Figure 5-98 Overturning of the internal walls in the courtyard and longitudinal cracks in the vaults of the Arcade without tie rods (North wing of Via Bafile)

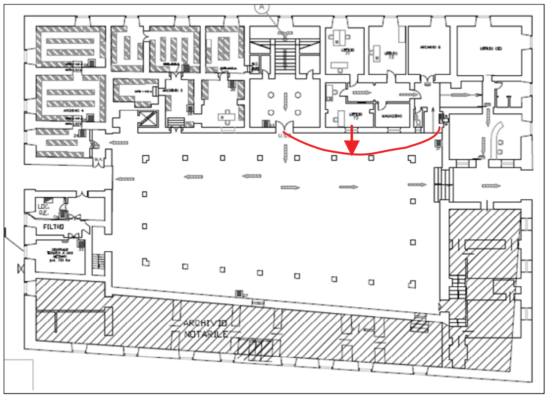


Figure 5-99 Position of the overturning wall in the plan view of the building

The rotations of the internal walls of the courtyard developed cracks in the limestone square blocks of the columns of the courtyard due to compression effects visible in Figure 5-100.

Figure 5-100 Cracks in the columns due to compression

The presence of tie rods on the East and West side of the courtyard (Figure 5-101) prevent the rotations of the walls and avoided the cracks in the vaults of the arcade.

Figure 5-101 Tie rods in the arcades of the West side of Piazza Margherita

A broken tie rod was found in the East wing of the courtyard (Figure 5-102), proving the traction of the tie rod during the seismic event in that direction.

Figure 5-102 Breakage of the tie rod located on the East side of the arcades facing Piazza Palazzo

The absence of tie rods on the exterior south wall facing Via delle Aquile, developed a simple overturning of the facade with displacements at the top bigger that 10cm, well visible in Figure 5-104. The presence of the top RC ring beam was not sufficient to guarantee the box behavior of the building, but it was essential in avoiding the collapse of the palace. The wooden roof was restored in early 1990 and it is a no pushing wooden roof (Figure 5-103).

Figure 5-103 Wooden roof and ring beam of Palazzo Margherita

Figure 5-104 Overturning of the external wall in Via delle Aquile

In correspondence of the South-West corner of Palazzo Margherita, vertical cracks are visible on the West wall (Figure 5-105) that confirm the overturning of the whole external wall in Via delle Aquile. The street of via Bafile is a main road for downtown L'Aquila, therefore it needed to be maintained as a viable road and props could not be used, because they would have occupied the entire road. Thus, rods were used, because they occupy less space and they use less material. In order to reduce intervention inside the building, rods passed through the windows and were able to connect the external walls with the internal walls of the courtyard (Figure 5-106).

Figure 5-105 Vertical cracks on the West side of Piazza Margherita

Figure 5-106 Provisional retrofit on the West side of Piazza Margherita

Damage to the Stairs

In Figure 5-107 and Figure 5-108, visible cracks due to shear in the vaults of the stairs that are highlighted in red in the plan view in Figure 5-109 are clearly seen. The higher stiffness of the stairs with respect to the rest of the building attracted the seismic forces.

Figure 5-107 Shear cracks in the vaults of the stairs on the first and second story level

Figure 5-108 Cracks of the cross vaults of the stairs

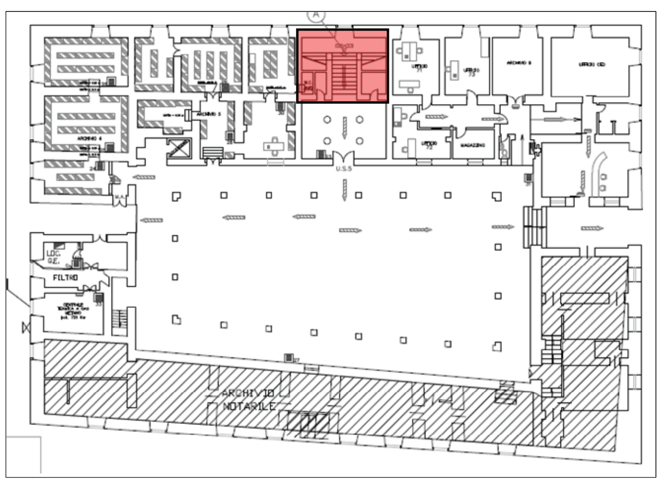


Figure 5-109 Location of the stairs in the plan

In order to avoid collapse of the low vaults, wooden restraint props were located to support the weaker part of the building (Figure 5-110).

Figure 5-110 Shoring up retrofit interventions in the stair of Margherita Palace with wooden restraint props

Partial Collapse of the Internal Vaults

In many places inside the building, collapse of slim false vaults such as the one shown in Figure 5-111 have been observed.

Figure 5-111 Partial collapse of the brick cross vaults (vaulted brick sheet)

Hammering of the Pipelines of the Heating System Connected to the Radiators

In the facade of the building facing Piazza Palazzo are evident cracks that initially seem to be the hammering of the floor beam at the second level. A more careful analysis has shown that the cracks were higher with respect to the floor level and they were in correspondence of each window. The presence of the radiators located below the windows and the pipelines connecting the radiators to the walls, determined the spill out of the cover of the hole that was used in the old times to support the scaffolding system (Figure 5-112, Figure 5-113 and Figure 5-114).

Figure 5-112 Radiator connected below the window of the facade

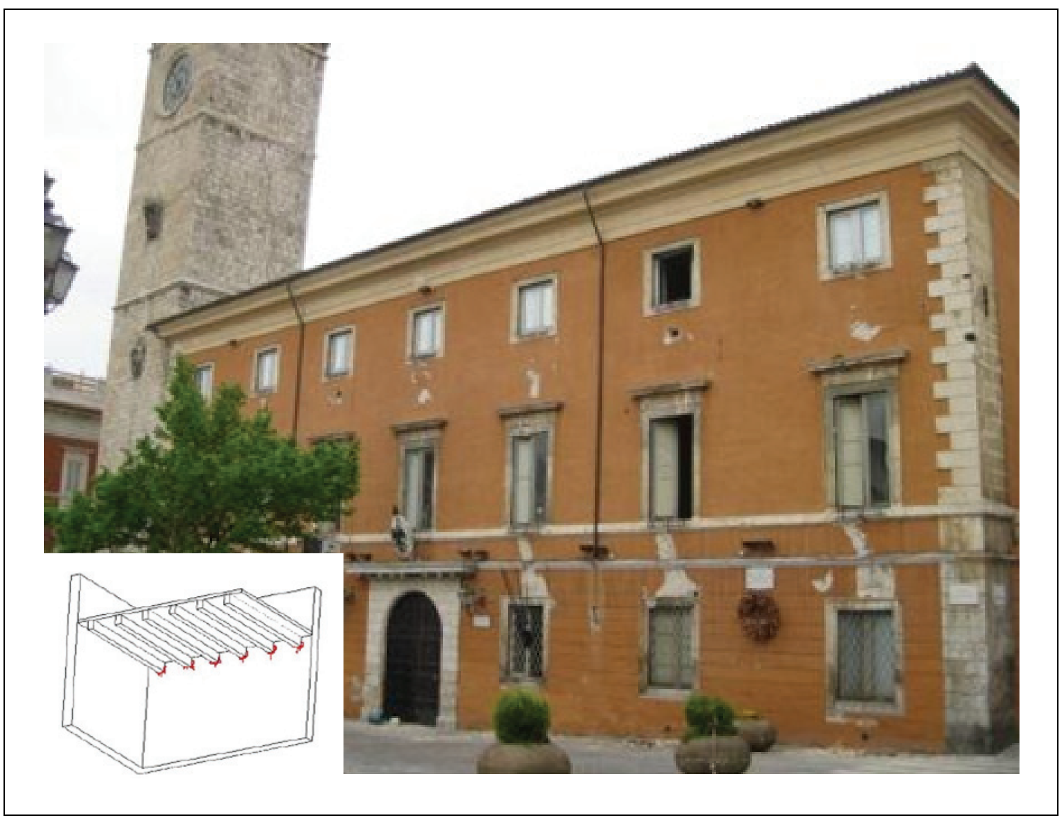


Figure 5-113 Hammering of the pipelines of the radiators anchored below the windows on the facade of Piazza Palazzo

Figure 5-114 Detail of the hammering of the pipelines of the heating system that spill out the cover of the holes that were used in the old times to support the scaffolding system on the facade of Piazza Palazzo

Local Collapse of the Floor Slab

The absence of tie rods on the North side and the presence of vaults in conjunction to the poor connectivity between the floor beams and the masonry walls, determined the local collapse of the floor slab at the second level (Figure 5-115 and Figure 5-116). In addition, it contributed to the collapse of the floor also because of the large distance (more than 7m) between the main orthogonal walls.

Figure 5-115 Collapse of the floor slab on the north wing (Via Bafile)

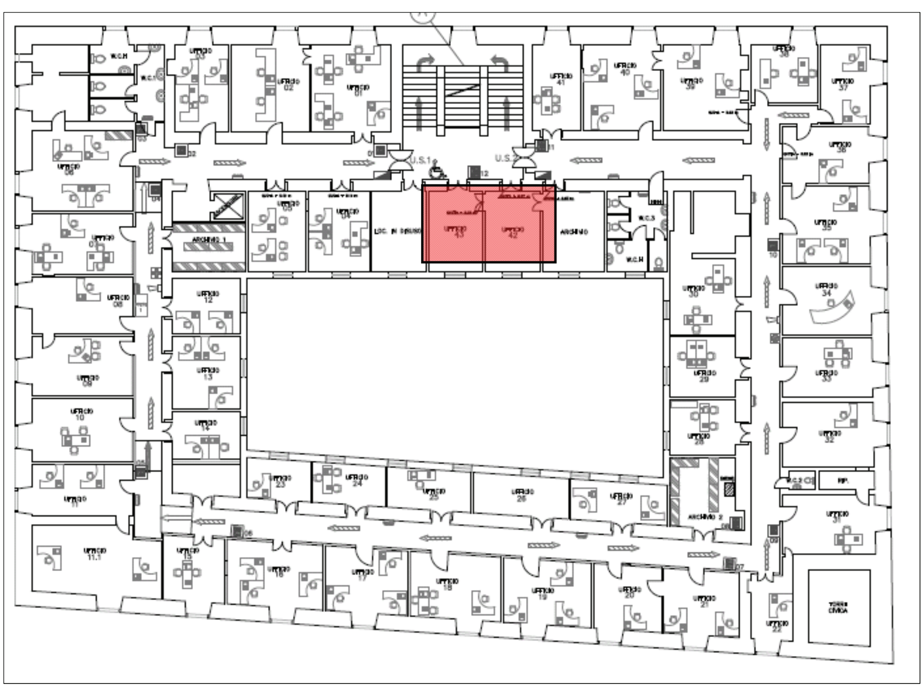


Figure 5-116 Location of the floor collapse at the second level

Shear Cracks in the Masonry due to in Plane Actions

The damage mechanism presented in Figure 5-117 can be observed frequently in the buildings damaged by a seismic event, however it is not considered dangerous for the stability of the building, because damage due to shear cracks in the building allows carrying vertical loads anyway. The presence of shear cracks in the inter-story strips is explained due to the small thickness and the lower strength of the masonry strip with respect to the main bearing walls. This mechanism can be observed in the internal walls of the courtyard in Figure 5-118.

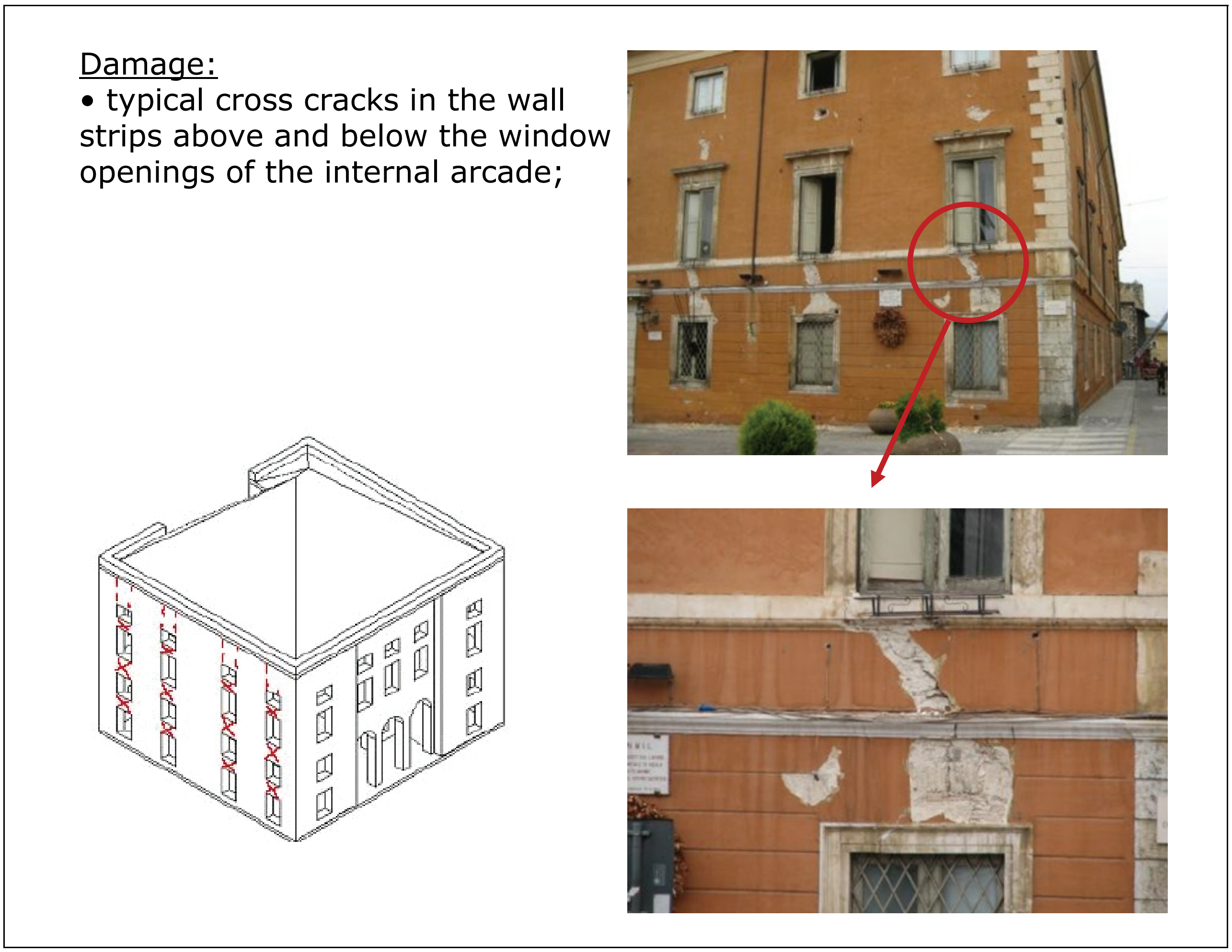


Figure 5-117 Cracks in the masonry strips of the facade of Piazza Palazzo

Figure 5-118 Cracks in the masonry strips of the internal walls of the courtyard

In Figure 5-118, the RC ring beam located at the top below the roof and realized by the architect Macri` in the early 90's can be observed. The goal of the intervention was to guarantee a box behavior according to the seismic standard. This can be considered as one of the few examples where the ring beam had a beneficial effect in preventing the collapse of the building.

Simple Overturning Mechanisms of the Internal Walls

The partial overturning of the orthogonal wall occurred nearby the stairs and was triggered by the bad quality of the masonry structure (Figure 5-119). Similar overturning mechanisms appeared to the internal partition walls (Figure 5-120).

Figure 5-119 Simple overturning of the masonry at the second floor nearby the stairs due to the bad quality of the masonry

Figure 5-120 Simple overturning mechanism of a partition wall made with brick sheet

Damage to the Roof

Tails have been moved at the roof level due to the seismic event (Figure 5-122).

Figure 5-121 Aerial view of the roof of the Palace

Flexural deformations of the beam supporting the roof was observed at the east wing of Piazza Palazzo (Figure 5-123), at the location marked with a red circle in Figure 5-121.

Figure 5-122 Damage to the roof cover

Figure 5-123 Bending of the beam supporting the roof

Cracks due to Shear in the Internal Masonry Walls

In several offices and corridors, X-shaped cracks were observed at the internal walls (Figure 5-124), typically close to the openings that most likely weaken the masonry walls. The internal walls damaged due to shear cracks are always able to carry gravity loads, therefore this mechanism is not considered dangerous for the stability of the building. However, the presence of irregular and spread cracks as the ones shown in Figure 5-124 is an indicator of masonry without monolithic behavior.

Figure 5-124 Cracks due to shear in the internal masonry walls of the corridor

5.3.3.4 Damage Mechanisms in the Civic Tower

The civic tower is part of Palazzo Margherita and it is the only part of the palace that was built in the 14th (XIV) century and was still standing after the earthquake. In 1310, the tower was 52 m high, and in 1320, a bell of the total weight of 7 tons was located on the tower. First renovation works were realized in the 16th century. In the past, one hour after the sunset, the bell was rung 99 dongs reminding the citizens that the door of the city should be closed. 99 dongs were rung because according to the legend, the union of 99 villages built the town of L’Aquila. Even nowadays, the clock of the tower rings 99 dongs after the sunset, remembering the foundation of the town of L’Aquila. The terminal part of the tower collapsed during the earthquake of 1703 and consequently the tower was restored reducing its height to 40 m. The plan dimensions of the tower are 6,27 × 6,42 m, and it is realized with a wall with thickness of about 2 m with an external cover of limestone squared blocks. In correspondence of the second level there is an internal cladding of one head topped brick inside the tower (Figure 5-125). It was realized probably after the earthquake of 1703 and it reduces through the tower height. The access to the tower clock is possible through a wooden stair that rotates around the tower and is shown in Figure 5-125.

Figure 5-125 Brick internal cover inside the Civic Tower

In Figure 5-125 and in Figure 5-126 are visible the vertical cracks along the entire West wall of the tower. The presence of tie rods in Figure 5-126 avoided the effects of buckling from 10 m up. In fact, the wise location of the tie rods (Figure 5-127) in both directions of the Civic Tower conferred to the tower a box behavior especially in the upper part, thus avoiding collapse during the earthquake of April 6th.

Figure 5-126 Observed vertical cracks in correspondence of the second level

Figure 5-127 Tie rods located near the tower clock

The access to the Chapel that used to keep the "Bolla del Perdono" of the Pope Celestino V and that is located inside the Civic Tower is possible through a door at the first level (Figure 5-97). In the chapel are present frescos of the XVI century representing the four patron saints of the town of L’Aquila (San Pietro Celestino, San Bernardino da Siena, San Massimo e Sant’Equizio) together with some landscapes of the town of L’Aquila.

The base of the tower at the ground level is not accessible, but probably there is another room of the same dimensions of the chapel that has been closed with filling material. However, this assumption needs to be verified.

5.3.3.5 Crack Pattern of the Civic Tower

The cracks are the evidence in structures of pathological disruptions, of which should be understood accurately both the current situation and their evolution over time, in order to understand the full life of the building. Therefore, the reading of the crack pattern should be correlated to material and constructive knowledge of the structure. On the front (Figure 5-128) is clearly visible the oldest unit of the national emblem that is composed of a carved stone eagle at the base of the Civic Tower in Piazza Palazzo. The cracks that can be clearly observed at the base of the tower on the south and east-side are the one of 1703 earthquake and are probably due to a stress concentration on the edge due to axial flexural movement of the tower. These cracks have been reopened during the earthquake of April 6th 2009 as shown by the cracks in the inside plaster (Figure 5-130 and Figure 5-131).

Figure 5-128 Vertical cracks at the base of the tower in the East side of Piazza Palazzo

The tower was supported by the adjacent palace on the north and west side, but the lack of an equivalent stiffness in the south and east side has led to a stress concentration in the southeast corner at the base that caused the formation of cracks and the subsequent rupture of one of the rods on the south wall (Figure 5-129). The extent of cracks is more than 2 cm as shown in Figure 5-128. The presence of the crack pattern in Figure 5-128 was generated by the poor confinement effects of the tie rods at the base of the tower that have less density distribution with respect to the top. The cracks are also visible inside the chapel in Figure 5-130 and in Figure 5-131 and were probably favored by the presence of the opening in the walls.

Figure 5-129 External broken chain at the base of the tower in the south side

Figure 5-130 Cracks in the cross vault and at the window of the chapel inside the Civic Tower (Photo by Giacosa and Degiovanni)

Figure 5-131 Cracks nearby the opening visible also from the outside

5.3.3.6 *Shoring up of the Civic Tower*

The provisional retrofit on the civic tower was realized with external rods, because with respect to props, provide better seismic performances during the earthquake vibrations, they occupy less space, they use less material and can be left in place as definitive retrofit intervention. The intervention shown in Figure 5-132 was not necessary, because tie rods were already inserted inside the tower and they guarantee a good box behavior. For the reasons above, the shoring up intervention of the tower could be limited only to the lower part where more cracks were visible.

Figure 5-132 Civic tower wrapped with external rods

5.4 Private Historical Palaces

5.4.1 Case Study: Palazzo Rivera

The Rivera palace is located in Santa Maria di Roio square (Figure 5-133). It is a regular rectangle plan of two stories and is connected on the posterior site with another palace. The building is private property of an old noble family of L'Aquila (the Rivera's family). The palace has an artistic value, because inside are present 9 painted vaulted ceilings on the first floor and on the staircase.

Figure 5-133 View of Rivera Palace in L'Aquila historical center from Santa Maria di Roio square

Figure 5-134 Overturning of the corner walls

In both corners of the frontal façade it is possible to observe cracks due to rotation out-of-plane of the corner walls, because of interacting forces with corner walls. The mechanism has been initiated by the window openings as shown in Figure 5-134.

Previous restoration interventions were done recently in 2003 at the roof level and they had a positive effect on the seismic performance of the whole palace, because displacements between the roof and the external walls were limited as shown in Figure 5-135.

Figure 5-135 Horizontal cracks due to small displacements of the wooden roof

Damages were observed in the perimeter walls of the posterior court yard in the form of instabilities and shear cracks (Figure 5-136). Rotation of external perimeter walls was limited by the presence of tie rods that worked pretty well in tension and limited the damage inside the palace (Figure 5-137). However these retrofit interventions were not able to avoid the collapse of one of the floors due to excessive rotation of the vertical walls as shown in Figure 5-138.

Figure 5-136 Wall instability in the first posterior court yard and shear cracks

Figure 5-137 Sign of tension in the tie rods located in the external perimeter walls

Figure 5-138 Collapse of the horizontal floor due to rotation of the lateral walls

Figure 5-139 Vertical cracks in the vault staircase

Many cracks in the ceilings of the painted vaults of the staircase (Figure 5-139) and in the rooms of the first floor (Figure 5-140) were observed. Shear cracks (Figure 5-141) are present in the interior walls, proof of the good capability of dissipating energy of the whole structure.

Figure 5-140 Damage in the painted vaults due to rotation of the vertical walls

Figure 5-141 Shear cracks in the internal walls

SECTION 6

INFRASTRUCTURE DAMAGE

6.1 Essential Buildings

6.1.1 Hospitals

The MCEER team inspected the San Salvatore Hospital (Figure 6-1), that is located in L'Aquila in Coppito. The hospital consists of 13 wings and was constructed from 1972-2000 according to a 1967-era "intermediate" seismic design.

Figure 6-1 Aerial view of San Salvatore hospital

It was constructed over a period of 30 years and it costs nine times more than expected; therefore it differs in construction details and materials in different parts of the hospital. A common characteristic of all buildings is the brick wall covering, which is partially or not at all connected with the infill panels and the reinforced concrete frame structure (Figure 6-2).

(a) (b)

Figure 6-2 Brick covering of the external walls: (a) shear wall damage mechanism; (b) out of plane damage mechanism

The hospital was shut down some hours after the earthquake, likely because of damage to various floors and to a few concrete columns in three of its many buildings (Figure 6-7). Patients were transferred elsewhere. Only 3 buildings out of 15 suffered considerable structural damage in the hospital, while there was relatively limited nonstructural damage in only a few buildings. The most widespread nonstructural damage was on the exterior façade bricks (Figure 6-2) that were partially or completely detached, because they were not attached to the interior walls.

In the case shown in Figure 6-3, the out of plane mechanism was initiated by the attachment at the exterior walls of the advertisement signs of the San Salvatore hospital that collapsed blocking the entrance to the emergency walkway (Figure 6-4).

Figure 6-3 Out of plane mechanism of the exterior masonry walls at the entrance of the emergency walkway

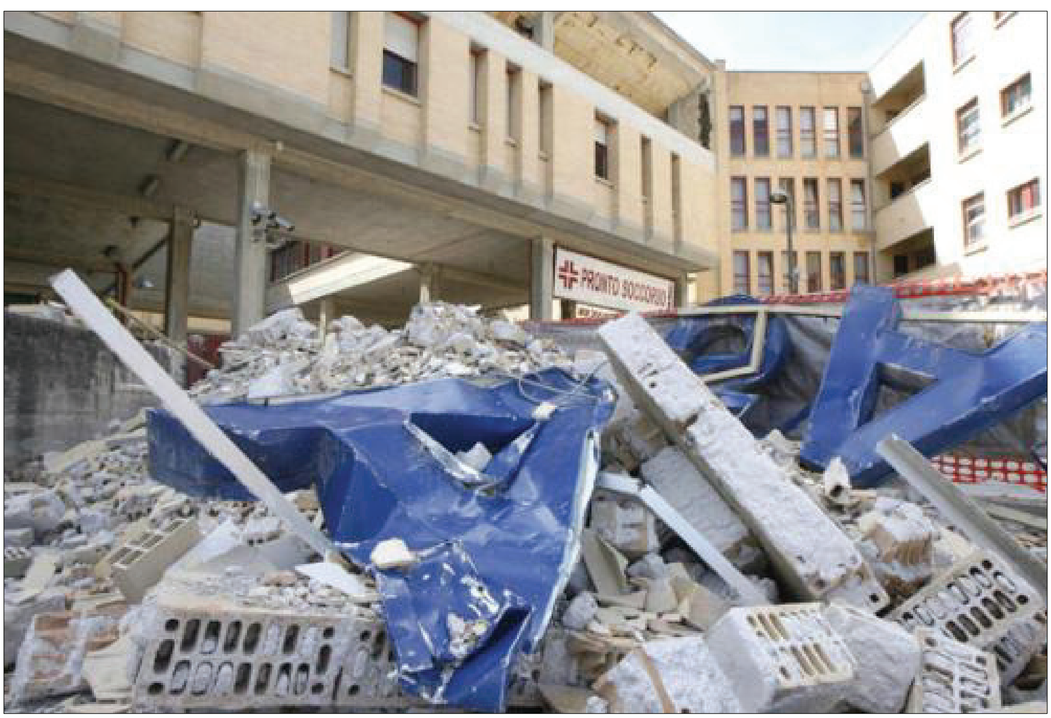


Figure 6-4 Entrance of the emergency walkway

The internal nonstructural components like mechanical devices and equipment did not suffer from any significant damage. Damage was observed in the internal suspended ceiling systems (Figure 6-5).

Figure 6-5 Damage to suspended ceiling systems inside the hospital

The nonstructural infill masonry likely contributed to resisting the earthquake lateral forces and limited the frame deformation (Figure 6-6).

Figure 6-6 Damage to internal infill walls

The main structural damage was the shear failure of some columns of the covered walkway over the driveway to the Emergency Room (Figure 6-7) with spalling of the concrete cover with the consequent instability of the longitudinal reinforcement (Figure 6-8) due to the absence of transverse reinforcement.

Figure 6-7 Damage to column due to poor construction quality and propping intervention

Figure 6-8 Instability of the longitudinal reinforcements due to inadequate transverse reinforcement with hoops of 6 mm and spacing of more than 25 cm

The presence of smooth bars indicates an old construction practice, but it cannot be considered as a cause of the failure of the column, because experimental tests proved their good performance during cyclic testing.

In the column shown in Figure 6-9, the shear failure was initiated by the poor construction quality and the insertion of electric tubes inside the columns of the reinforced concrete structure.

Figure 6-9 Poor construction quality: electric tubes passing through reinforced concrete column of the emergency walkway

No other significant damage to the concrete frame was detected except for the structural column damage in few zones due to improper construction details, such as the realization of the expansion joints shown in Figure 6-10, the inappropriate confinement of the structural elements and the insufficient concrete cover.

The overall status of the buildings inside the hospital did not justify the evacuation of the entire hospital, because the most extensive damage was mainly nonstructural. However, most of the buildings were usable provided some simple short term countermeasures such as removing falling cladding plaster and detached parts of the coating.

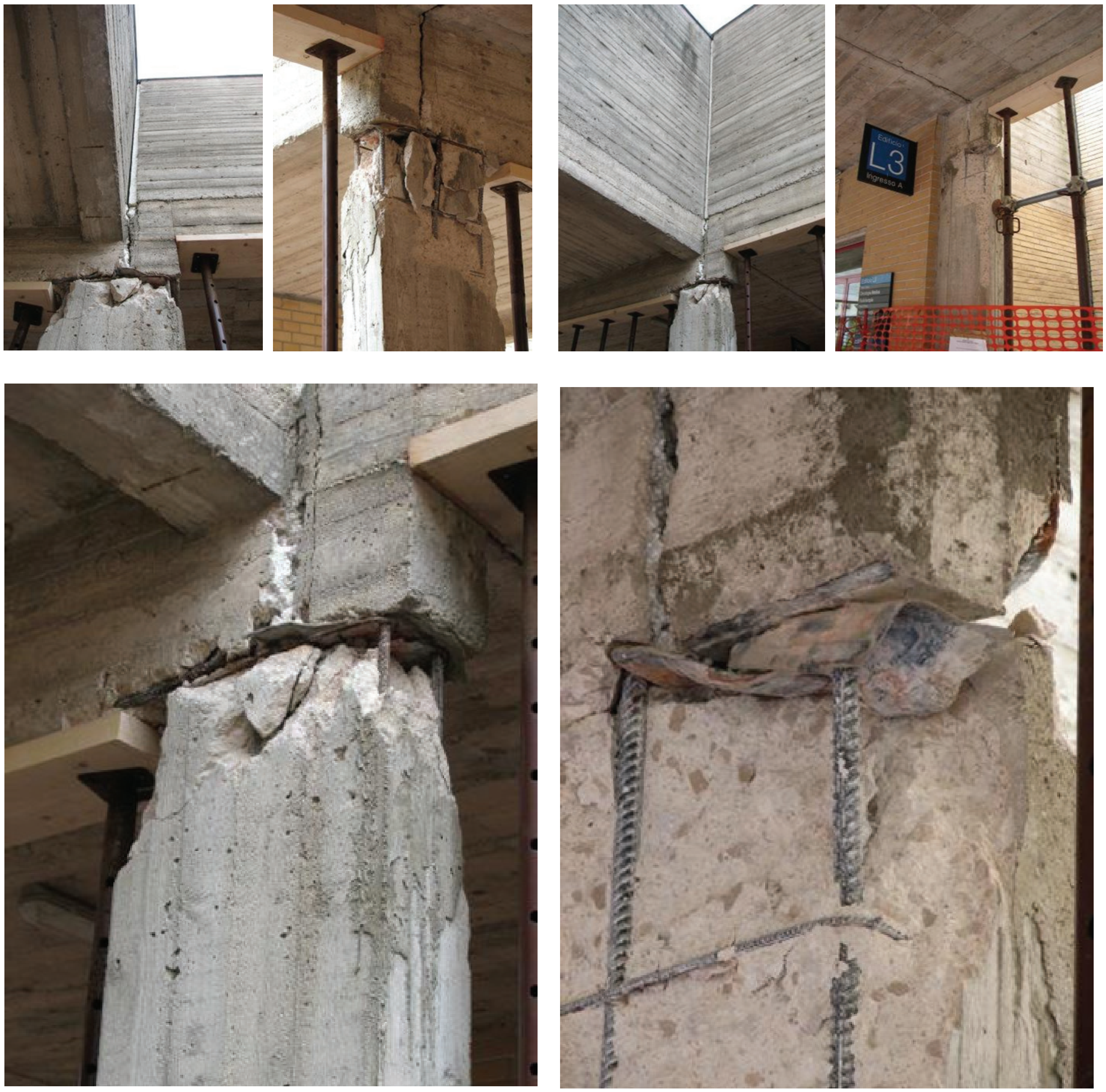


Figure 6-10 Poor construction quality of the expansion joints

6.1.2 Governmental Buildings

The Palazzo del Governo in Piazza della Repubblica, should have been the location of the emergency response of the Civil Protection Department. This building was declared as seismic vulnerable in a report written in June 2008, about a year before the earthquake. Due to April 6th earthquake it completely collapsed, as shown in Figure 6-11, and only the four columns of the main entrance remained standing.

Figure 6-11 Governmental building

6.1.3 School Buildings

6.1.3.1 Elementary School "De Amicis"

6.1.3.1.1 Historical Notes and Previous Restorations

The Gothic portal and other elegant specimens remain of the 15th century S. Salvatore hospital structure (Figure 6-12), while the majority of the building has been restored in the 18th century with austere forms. The building is made of brick masonry on the ground floor, and mostly of stone on the upper floors.

Damage:
- Partial collapse of the roof;
- Partial collapse of the upper lateral wall;
- Cracks in the internal walls with partial collapse of ceilings;
- Dislocations of horizontal slabs;
- Collapse of cross vault;

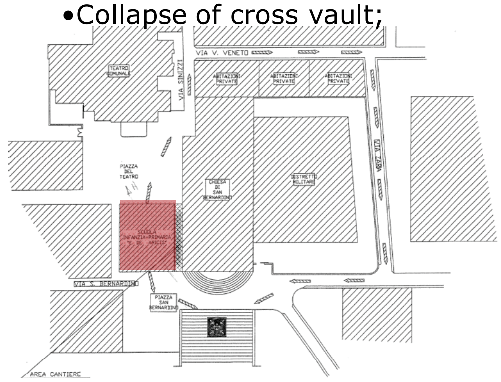


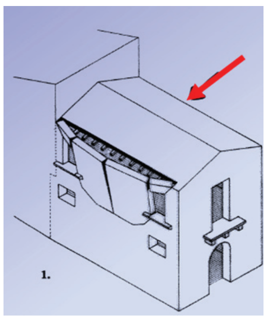
Figure 6-12 De Amicis Elementary school

In the ground floor there are arcades with stone pillars and foil vaults, with the absence of tie rods in the north side. On the upper floors, the floor slabs are made with horizontal RC beams that are the result of subsequent alterations. The gabled roof was restored in 1990 with a RC ring beam.

Other relevant changes were probably done at the beginning of the 20th century, which introduced a system of reinforced concrete beams that support the steel beams of the floor slab covered with concrete.

Restoration interventions of the roof cover started in 2004 and they were still going at the moment of the earthquake as shown by the presence of scaffolding system inside the courtyard and on the façade of Via San Giovanni da Capestrano. The scaffolding system located in the courtyard collapsed, as shown in Figure 6-13. Additionally, the roof structure of the north wing has been completely replaced by a new metal structure with wood planking (Figure 6-14), after inclusion of a RC ring beam on top of masonry walls.

Damage:
• Local collapse of the lateral wall and the roof;
Mechanism:
• Overturning of the lateral wall;
Structural causes:
• Poor connection between lateral walls and roof slab. The poor quality of masonry and presence of openings initiate the mechanism;



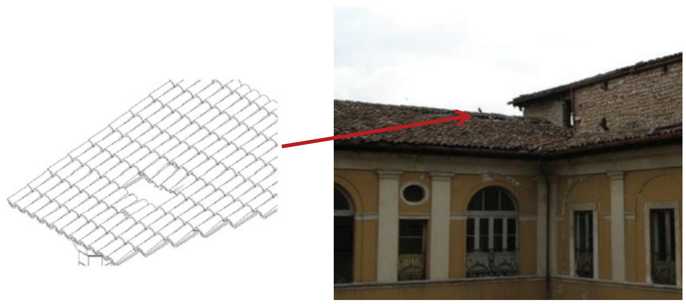


Figure 6-13 Damage to the roof coverage

Figure 6-14 Collapse of the barrel vault at the second floor

6.1.3.1.2 Geometric Description

The building is located in downtown L'Aquila in Piazza San Bernardino near San Bernardino Church on a slope with a height difference in the north-south direction of about 2.5m. The building has a rectangular plan (54×46 m) with a large courtyard inside and three story levels in total with a total height of about 18 m and no basement. Thickness of the walls ranges between 1 and 1.5 m. The squared stone columns of the courtyard are 0.55×0.55m. Horizontal slabs are different at different stories and globally five different typologies have been detected.

The ground floor is made with cross vaults, while steel beam slabs with clay pots are used on the first floor. On the second floor, however, the slab is made with a wooden squared mesh and a false ceiling made of plaster and vegetable fibers.

Local amplification effects might be developed in Piazza San Bernardino, due to topographic effects together with sediments of soil type B, because the school is located on top of a hill.

6.1.3.1.3 Damage Mechanisms

The detected damage is listed in Table 6-1.

Table 6-1 Observed Structural Damage

Structural damages
• Partial overturning of the lateral walls in the North South direction as shown by the signs of compression on the columns of the arcades and the cracks parallel to the façade in the vaults and on the floor of the first level • Damage to the columns of the Arcades • Shear cracks in the internal and external walls mainly in the North South direction • Partial collapse of the roof and of the upper lateral wall triggered by the hammering of the roof beams • Cracks between orthogonal walls • Dislocations of horizontal slabs • Collapse of two barrel vaults at the second floor

The main damage from the external observation is visible in the frontal façade of Piazza San Bernardino on the upper left roof corner (Figure 6-15).

Damage:
• Overturning of the upper corner lateral wall;
• Partial collapse of the roof cover;

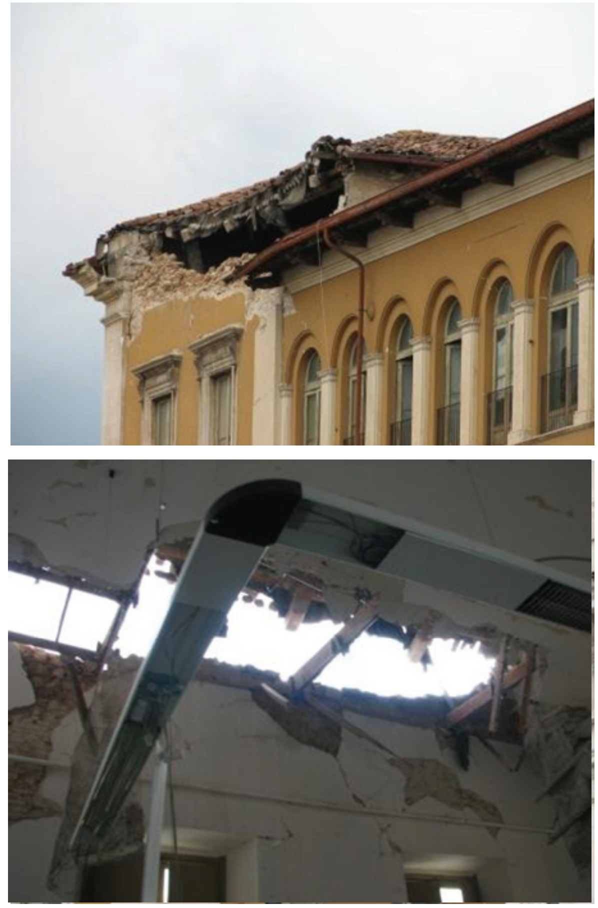


Figure 6-15 Damage to roof coverage due to composite tilt mechanism

The composite tilting mechanism was accompanied by dragging a piece of masonry belonging to the unconstrained corner. The mechanism has been probably triggered by the hammering of the roof wooden beams and by the poor quality of the masonry walls.

The horizontal flexure mechanism of the upper lateral walls has been triggered by the poor quality of connections between orthogonal walls and horizontal slabs, but also by the presence of the window openings, as shown in Figure 6-15.

The damage mechanism shown in Figure 6-16 has been observed in several buildings in downtown L'Aquila, but is not considered dangerous for the building's stability, because masonry walls damaged with shear cracks are still able to carry vertical gravity loads. Shear cracks in Figure 6-16 appeared between two window openings at different stories due to the short wall thickness and therefore lower stiffness with respect to lateral walls between window openings at the same level.

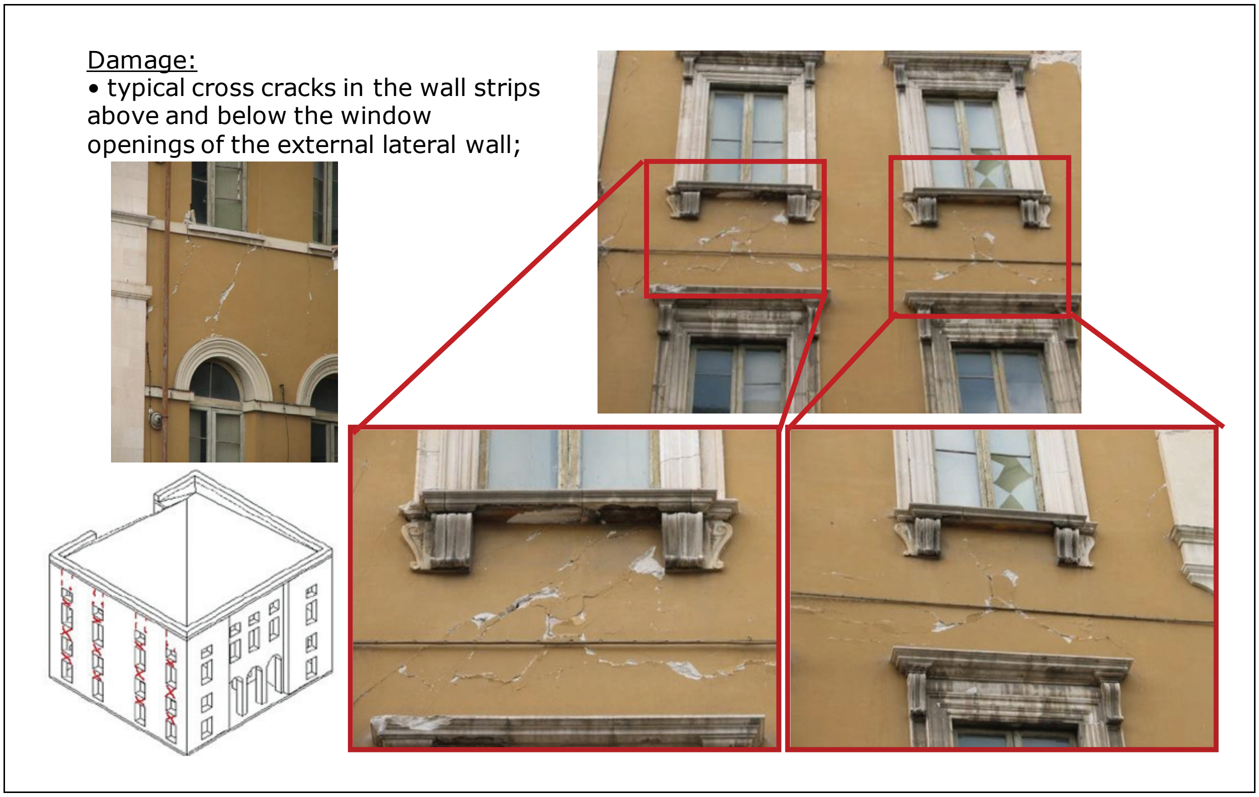


Figure 6-16 Damage in the strip walls

Tilting mechanism of the external lateral walls are present in the East, West and South side, as shown by the presence of cracks parallel to the longitudinal walls and diagonal cracks in the transverse walls.

An interesting damage in the frontal facade has been observed both in the Elementary School "De Amicis" and in Margherita Palace (City Hall). Holes were observed in the external masonry walls below each window, therefore based on first observation it was thought that it was due to the hammering effect of the floor beams (Figure 6-17).

Damage:
• local damage in correspondence of the wall strips in the facade;
Mechanism:
• damage to the front facade, due to the hammering of the radiator of the heating system;
Structural causes:
• radiator not effectively connected to the wall masonry.

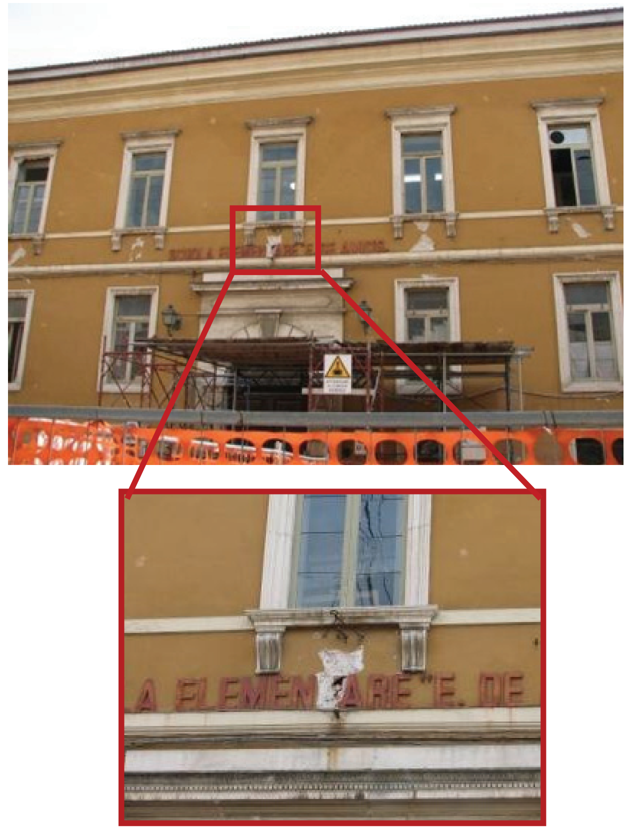


Figure 6-17 Hammering of the radiators of the heating system that reopen the holes made for hanging the scaffold system

However, after checking the hole and its height more closely, it was concluded that that hole was already in the masonry and it was used in the last century to support the scaffold system. Eventually the holes were re-opened by the hammering effect of the radiators of the heating systems located below each window.

Damage to arcades are shown in Figure 6-18 due to the presence of longitudinal cracks in the vault and arches keys in the north and west side. Damage to the columns of the arcades is observed due to the presence of vertical cracks describing the flexural damage mechanism initiated by the tilting of the lateral walls.

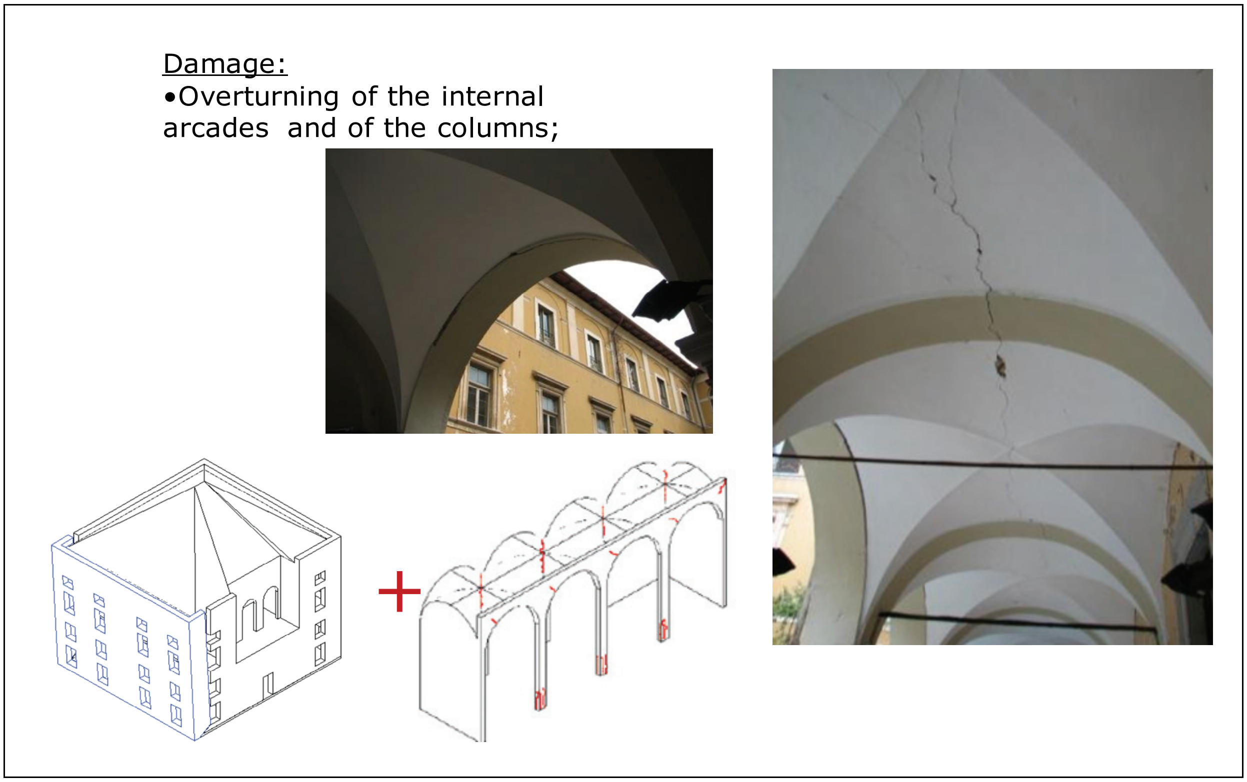


Figure 6-18 Damage in the arcades

6.1.3.1.4 Internal Damage Mechanisms

Internal damage is well spread through the entire building at every story level and it affects both structural and nonstructural components.

Following the shock that occurred during the school hour service on March 30th, 2009, the school was evacuated and never used after that date. This is confirmed by the presence of clothing and books on the class desks as shown in Figure 6-19. The vertical elements are characterized by extreme variability in texture and in the state of conservation. Damage in the internal walls due to shear are not clearly visible due to the different patterns of the internal masonry walls, but also due to the presence of arches within the walls.

The presence of various transformations resulted in changes to the original convent, in fact in several cases, thin hollow core clay bricks are used as partitions. Such situations are indicative of potential vulnerabilities that have occurred mainly with local crashes and damage due to shear in the thin elements as in the one shown in Figure 6-19.

Slippage of the steel beam heads is observed in all the horizontal slabs as shown in Figure 6-20. This damage mechanism is the consequence of the principal tilting mechanism of the external lateral walls. Local collapse of the foil barrel vault of the second floor is shown in Figure 6-14. This collapse is triggered by the overturning of the supporting lateral walls.

Figure 6-19 Damage in the internal partition walls

Figure 6-20 Damage in the roof ceiling

In summary, the presence of two corner towers in the frontal façade introduces some vertical irregularities that are emphasized by the ground height difference of about 2.5m. Furthermore, considerable amount of significant structural transformations that are not always compatible with the material and historical characteristics of the building, introduce some vulnerabilities in the building. Furthermore, the absence of ties rods in parts of the building, the foil vaults and the partial realization of the ring beam represented elements of vulnerability. However, the strong and widespread interaction between the thick transverse and lateral walls gives a positive contribution to the safety of the building. The restoration should aim at restoring the connection between structural elements, using metal tie rods, in order to improve the connections between floors and walls.

6.1.4 Industrial Facilities

Several factories and industrial plants were located in the earthquake-affected area and suffered minor (Figure 6-21) to heavy damage during April 6th 2009 L'Aquila earthquake. The damage sustained by electrical and mechanical equipment, special structures e.g., on grade steel tanks, industrial buildings, and nonstructural and secondary components caused several industrial complexes to be put out of commission. Damage to industrial facilities not only caused direct losses, but several indirect economic and social impacts also occurred due to the work stoppage at industries. Industrial buildings in the region are mainly constructed from reinforced concrete precast elements and they suffered damage due to insufficient saddles of precast beams and insufficient strength of nonstructural infill panels.

Figure 6-21 Damage to windows

Most of the damage of the precast industrial buildings depends on the relative displacements at the connections between structural elements (Figure 6-22).

Figure 6-22 Relative displacement at the beam-column connection of precast elements

A chemical company of plastic material suffered from damage to the steel tanks, which experienced total collapse, while others suffered from local or global buckling, depending on the level of the liquid contained. Most of them were full when the earthquake struck (EERI, 2009).

These liquid tanks are very slender made of steel and the horizontal radial pressure due to the silage material inside the tank has a stabilizing effect against the buckling of the silos walls (Figure 6-23). Typically, it's the compressive vertical membrane stresses that resist overturning moments, that cause local crushing of the ring beam and then the earthquake vertical component initiated the damage overturning the entire tank. The instability of the wall panel was caused by the effect of the axial force in compression due to the friction between the liquid material and the walls.

Figure 6-23 Instability of tanks in the polypropylene stock of VIBAC in Bazzano near Onna

Infill walls of several framed storehouses experienced minor to heavy damage due to the strong ground shaking. Major damage to the infill walls was:

- Out-of-plane failure of the infill walls (Figure 6-24)
- Vertical cracks close to the wall-to-frame junction (Figure 6-25)
- Diagonal cracks at the wall panels

The observed damages are mostly related to their flexibility and to the inadequate performance of the connections. Lack of vertical and horizontal anchors and poor detailing were the main reasons for the damage.

Figure 6-24 Failure of the infill wall due to the lack of anchorage to the frame

Figure 6-25 Vertical cracks close to the wall-to-frame junction

Many car dealers' shops in the region experienced structural and nonstructural damage (Figure 6-26).

Figure 6-26 Car dealer

Although the performance of the industrial structures during the L'Aquila earthquake was better than the seismic performance of conventional structures, e.g., residential buildings, damage to industrial facilities caused a significant amount of economic losses and social impact.

6.2 Lifelines

6.2.1 Water, Gas, Electric Power, and Telecommunications Performance

The electric company that provides electric energy in the region affected by the earthquake (ENEL) was active in the hours immediate after the earthquake on April 6th. 118 personnel were employed, 77 of which for the electric network and 41 for the gas network.

30 power generators were available right after the earthquake and they increased up to 45 in the following days. The electric company provided a continuous support through the operative center of the Department of the Civil Protection (DPC) that was located in the school of the Financial Police.

Around 15,000 clients were left without power at 3:32 a.m. of April 6th, but they were reduced to 4,500, 5 hours after the earthquake, and to 400, 9 hours after. After 4 p.m., power supply was completely restored and ordinary management of damages in the low voltage network was initiated. 277 lines of the low voltage network were shut down for safety reasons according to DPC and the Fireman Brigade (VV.FF.) and they were supplying power to about 10,000 clients. The gas network was shut down as well for safety reasons.

On April 7th, 8 tower emergency lights were provided in some strategic areas. The first post-earthquake retrofit interventions focused on the electric power generator of the 170 tents made for rescuers, which provided about 40 MW.

The electric power supply to these tents was guaranteed until their dissembling in October 2009. Thereafter, electric power was supplied to four new building typologies:

1. C.A.S.E. (Complessi Antisismici Sostenibili ed Ecocompatibili)
2. M.A.P. (Moduli Abitativi Provvisori)
3. M.A.P.- Frazioni (Moduli Abitativi Provvisori Frazioni del Comune di L'Aquila)
4. M.U.S.P. (Moduli ad Uso Scolastico Provvisori)

The electric power was shut down for buildings severely damaged or collapsed. About 500 private residents decided to move their commercial activities in new buildings and asked for electric supply in these structures.

The *gas distribution network* consists of 621 km of pipelines, with 234 km pipelines with gas flowing at medium pressure (2.5-3 bar) and the remaining pipelines with gas flowing at low pressure. The gas network was closed right after the earthquake in less than two hours in order to avoid explosions and fires. Then, in the following days, all the gas valves outside the residential buildings were closed as well. In the second phase of the emergency, the gas was provided to strategic structures such as hospitals, the operative center and several tents through the use of 7 special gas wagons. In less than three months, the gas network was restored and it was possible to restart the gas distribution for all the end-users with a safe home.

The total damage estimation of the electric power network is about 15 million euro, while it is 20 million euro for the gas network, while the initial post-earthquake retrofit interventions in both networks were about 10 million euro.

The *water distribution system* has a network of 900 km of large pipes with pressure between 30-50 bar and it is stored in about 200 tanks. The distribution network is made of cast iron and steel pipes with pressures ranging between 6-8 bar, for a total length of 1100 km reaching around 100,000 customers. After the earthquake, the main pipeline (diameter of 600 mm) of Paganica was shut down, because a joint failed due to the earthquake fault crossing the pipe. The repair was limited to the welding of the pipes at the joint. No other damages were observed to the main water distribution system. The water distribution was not restored in L'Aquila historical center in order to prevent flooding in damaged buildings. Water provision was restored gradually in different zones, according to the priorities, but in the "red zone" in L'Aquila the water network was still closed at the time of this reconnaissance mission.

Damage to the *telecommunication network* was very limited, because it was out of service only for two hours right after the earthquake. Cell phones have been the major communication system in the region. Firefighters and police used their own radio system as the preliminary communication tool.

6.2.2 Performance of Transportation Systems

The road network performed very well during this earthquake. The highway connecting L'Aquila with the city of Rome (A24) (Figure 6-27 and Figure 6-29) was closed only the day after the earthquake for security reasons in order to inspect bridges and viaducts, fixing all the dangerous situations that posed a safety threat.

Figure 6-27 Highway A24 Rome-L'Aquila

During inspections, trucks and commercial vehicles were driving through local roads (Figure 6-28). After that, highway A24 was closed for trucks with weight over 7.5 ton in order to facilitate the movement of emergency vehicles.

Figure 6-28 Trucks driving through local roads

Bearing failure has been observed in the viaduct of the A25 highway close to the town of Popoli at almost 40 km from the epicenter.

Figure 6-29 Map of the two main highways A24 and A25

National road SS5 (Tiburtina Valeria) was closed to traffic due to severe structural damage. In the afternoon of April 6th, it was interrupted between 160 and 169 km due to the presence of falling stone on the road, therefore traffic was moved on provincial road 9. It was closed to the traffic also between 176 and 176.4 km due to a structural failure near Corfinio, and traffic was detoured to national road SS5. Traffic was also interrupted on *national road SS696* due to the presence of falling stones on the road and on national road 81 due to the presence of a collapsing church. These are only two examples that show that falling rocks and landslides triggered by the earthquake and by the rain in the days following the earthquake are the main problem that affected the network mobility in the region, especially in the mountain area around L'Aquila.

Regarding the historical center of L'Aquila and neighborhood towns, many roads were closed to traffic (Figure 6-30) for safety reasons due to the presence of debris and bricks that fell down from the buildings along the road or due to the permanent displacement of the deck of the viaduct (Figure 6-33). One of the first issues of the emergency response was to handle the traffic in L'Aquila due to the closure of the main road that ran through the city. Later, the new traffic requirements developed by the new provisional accommodations for rescuers needed to be satisfied.

Figure 6-30 Closed road in L'Aquila historical center

Heavy damages were observed only on two bridges crossing the Aterno river near Onna (Figure 6-31 and Figure 6-32). The bridge shown in Figure 6-31 collapsed because the displacement exceeded the limit of the bridge bearings, while the second bridge shown in Figure 6-32 failed due to ground failure at the abutment.

Figure 6-31 Collapse of Bridge on the Aterno River

Figure 6-32 Collapse of Bridge on the Aterno River

Figure 6-33 Viaduct closure to the traffic in L'Aquila due to permanent displacement of the deck

6.2.2.1 Railroads

The train company (Trenitalia) interrupted the traffic on the connection between Sulmona and L'Aquila, but starting from 8 p.m. of April 6th the connection between Rome and L'Aquila was reestablished. Due to the following aftershocks, some railroad bridges were inspected in the region interrupting the traffic between Scafa and Torre de Passeri.

6.2.3 Lifeline Considerations on Emergency Response

It can be concluded that the lifeline network was not as severely damaged as the residential and monumental buildings; therefore, the emergency management provided a rapid and resilient response.

SECTION 7

SOCIETAL IMPACTS

7.1 Emergency Response

On the morning of Monday April 6th, 2009, at 03:32 local time, the earthquake occurred and the response was immediate, as shown by the events below:

- 3:35 a.m.: The response is activated
- 4:15 a.m.: The operative committee meets and sends the first team
- 6:30 a.m.: The team of the Civil protection Department (DCP) reaches the site of the earthquake
- 9:30 a.m.: Volunteers mobilization
- 1:00 p.m.: Conference of the Prime Minister

After realizing that the governmental building collapsed (Figure 6-11) during the earthquake, the Headquarters of the Operative Center that is suppose to coordinate the emergency response during the earthquake was located in the police academy campus in Coppito, just outside L'Aquila, while the local management was delegated to seven different centers in the affected area. The structures that were organized by the Department of the Civil Protection were respectively called:

- DICOMAC (Direzione di Comando e Controllo), The Headquarter Operative Center
- COM (Centri Operativi Misti), The local management centers

DICOMAC is the Headquarter Operative Center that coordinates the emergency response in the area affected by the earthquake. The DICOMAC was coordinated by the Vice president of the Department of the Civil Protection and it was located physically in the area of Coppito (Figure 7-1).

The COM is the Mixed Operative Center and it is one of the Operative Centers of the Integrated Model of the Civil Protection (Augustus Model). When an extreme event happens, all the Operative Centers are activated to coordinate the emergency response.

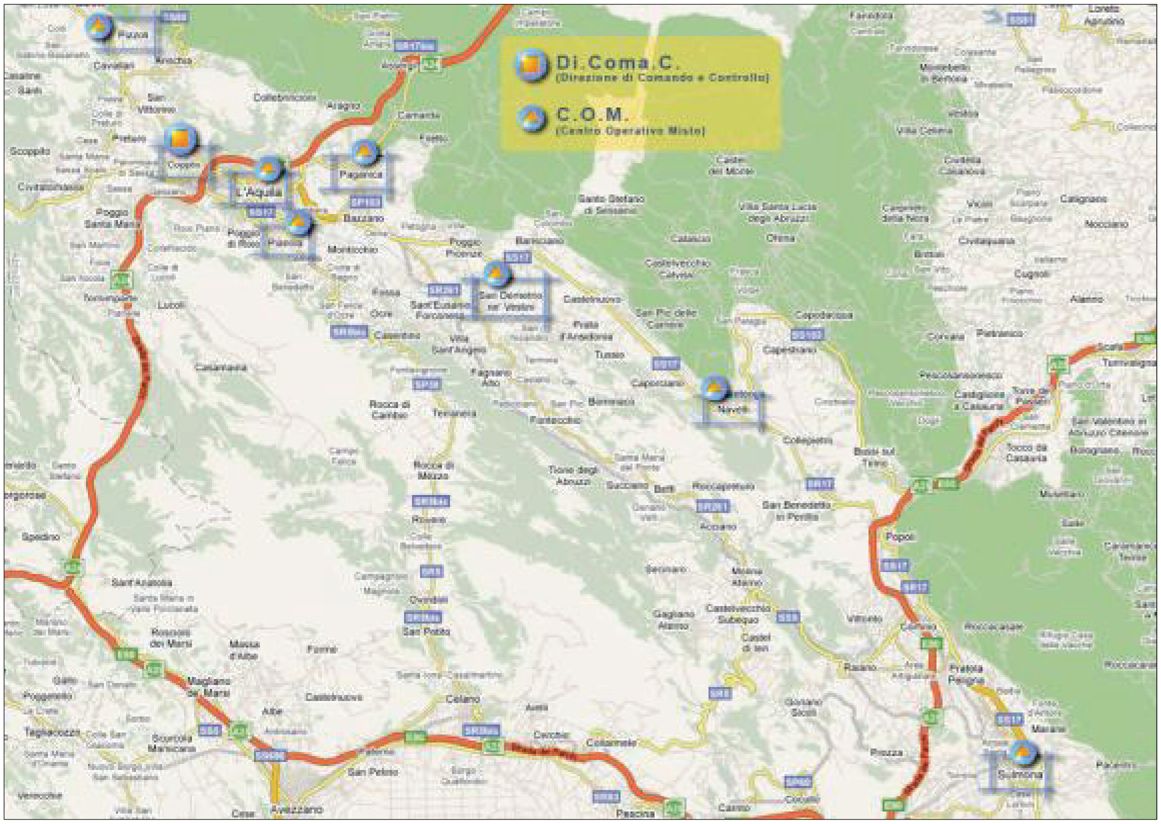


Figure 7-1 Operative Centers of the Civil Protection during the post-disaster response

Abruzzo is a region of sparse population and the main area of damage appears to be limited to a 25 km radius around the epicenter. Hence, the Civil Protection response to the emergency posed no exceptional challenges other than the need to conduct rescues in precarious circumstances and to prepare for the eventuality of rainy weather. In two days, 2,250 firefighters, 1,500 army personnel, 2,000 policemen, 3,000 volunteers and more than 1,000 technical employees of the Abruzzo Regional office were working in the region.

In the weeks following the earthquake, 31 tent camps were built (Figure 7-2) to house 17,772 homeless people, and 171 hotels on the Adriatic coast were allocated to host an additional 10,000 (Table 7-1). Field hospitals were also installed to provide first emergency aid. The facilities for the care and housing of homeless residents increased over time, with about 1,700 private residences in the region opened for displaced people. Globally, 160 tent camps were built, with a total of 5,700 tents for 8 people each.

According to the peer-review, the merit of Italy's Civil Protection is that it "*can rapidly and efficiently mobilize operational resources for emergency management and recovery both at home and in the rest of the world thanks to a coherent, multi- risk approach that integrates scientific research and technological expertise into a structured system*".

Figure 7-2 Tents in L'Aquila

7.1.1 Search and Rescue

A negative aspect of this earthquake is that it occurred at night and at the world scale, most of the injuries occur in nocturnal earthquakes (50-90% of mortality is nocturnal), because a sleeping person is not able to react rapidly. A positive aspect is that it happened at the end of the week-end and many people had not returned home yet.

Relief efforts started soon after the earthquake. The response to the earthquake should be divided in three temporal ranges:

- Immediate (24 hrs);
- Intermediate (first week);
- Current (from second week on);

In the first period range, the role of the firemen that helped to rescue people that were buried under the collapsed buildings was essential. As in all seismic events, the L'Aquila earthquake confirmed the role of the National Fire Brigades Corps as the lead agency, owing to its primacy in technical rescue and making urban areas safe. During the intermediate temporal range when all people were rescued, building safety inspections started and it extended in the current temporal range. In the current temporal range, the reconstruction phase starts.

7.1.2 Mass Media Response

The initial mass-media coverage during L'Aquila earthquake is the one typical of any disaster event:

- Human interest stories,
- coverage of VIP visits to the area,
- news of world reaction to the event,
- efforts to convey the flavor of being at the scene of the action
- updates on the statistics of casualties, homelessness and emergency responses.

On a positive note, the description of the stoicism and dignity of the survivors was reported in news bulletins.

Disaster sociologists have shown that antisocial behavior is minimized right after the disaster by the accession of the so-called 'therapeutic community' (Barton 1970).

However, it is clear that mass media prefers to inform about looting and other examples of anti-social behaviors that tend to be exaggerated.

A further aspect of press reporting shown during this disaster was the common tendency to exaggerate. Both the city of L'Aquila and the small village of Onna, 8 km to the SSE, were described as being 'destroyed'. Aerial views of the buildings of Onna show 100 per cent damage and 50-60 per cent outright collapse. Images of L'Aquila show sporadic damage, including the partial or total collapse of single large buildings and areas in which groups of buildings have battered each other down. In both cases this is far from total devastation. However, aggrandizement is a common feature of disaster reporting in any setting.

7.1.3 Predictions and Warnings

Italian laboratory technician Giampaolo Giuliani predicted a major earthquake on Italian television a month before, after measuring increased levels of radon emitted from the ground. He was accused of being an alarmist by the Director of the Civil Defence, Guido Bertolaso, and forced to remove his findings from the Internet. He was also reported to police a week before the main quake for "causing fear" among the local population when he predicted an earthquake was imminent in Sulmona, about 50 km (31 mi) from L'Aquila, on 30 March where a 4° quake happened (later Sulmona only suffered minor damages by the 6 April earthquake). Predicting earthquakes based on radon emissions has been studied by scientists since the 1970s, but enthusiasm for it had faded due to inconsistent results.

7.1.4 Damage Assessment

The building safety inspections started in the days after the earthquake. The damage and seismic usability assessment of public and private buildings were the most demanding activities in terms of needed resources. They were coordinated by DPC with support from the regions, provinces, Firemen Corps, etc. In order to improve the recovery process, priority was given to public buildings such as hospitals, schools, headquarter buildings, as well as to commercial buildings. The damage assessment has been carried out using the AeDES survey form (Baggio et al, 2000), that is based on the experience gained during several earthquakes starting from Umbria and Marche earthquake in 1997. According to this form, buildings are classified from *A* to *F* where *F* means unusable. According to the outcome of the survey, slightly more than 50 % of the buildings are usable (cat. A), while 26.3% are unusable (cat. E) as shown in Figure 7-3.

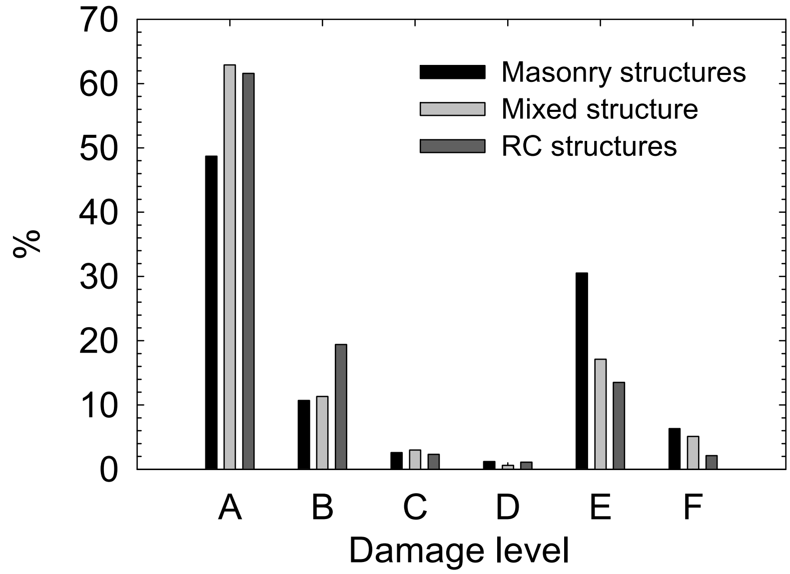


Figure 7-3 Usability distribution for different buildings typologies

When focusing on cultural heritage, the seriousness and the extension of damage due to the earthquake were significant due to the dimension and strategic importance of L'Aquila as capital of the Abruzzo region. The activities to protect the heritage have been conducted on two parallel levels of damage survey and of design and implementations of temporary safety measures. They started one week after the earthquake on April 14th, 2009. Damage survey involved different subjects such as Cultural Heritage officers, experts on structural engineering from universities and Fire Brigade teams. Together they filled standardized damage survey forms for churches and palaces developed by DCP. The structural damage survey is based on the identification of macro-elements that constitute a masonry building and on the evaluation of the level of activation of the kinematic mechanisms associated to the macro-element itself. The survey form for the churches has already been used in the previous earthquake in Molise, while the survey form used for the evaluation of the palaces has been used for the first time in the Abruzzo region. Difficulties were noticed by the MCEER team in filling up the form that is probably connected to the difficulty of bringing together all the non religious historic buildings into the same typology defined as "palaces".

The total cost of damage to buildings is estimated to be 2Bn to 3Bn Euro (AIR Worldwide), whilst the insured loss to be in the range 200-400M Euro (AIR Worldwide).

7.1.5 Economic and Social Effects on Business

Damage to buildings also led to significant disruption of the community and social fabric, leading to the temporary or permanent closure of government offices, small businesses, restaurants, churches, and schools. Slowly these shops started opening again, but the red zone in downtown L'Aquila was still closed at the time of this reconnaissance mission together with small business, restaurants, churches and schools. Whenever possible, shop owners opened again their activities outside the red zone in L'Aquila.

7.1.6 Casualties and Injuries

In total, the earthquake killed 308 people (202 in the town of L'Aquila), injured over 1,500 people (202 serious and 898 triaged) and approximately 44,000 people were homeless. The main percentages of deaths are over 70 yrs and in the 20-29 yrs range. The large number of young death is due to the presence of many university students in the area. However in downtown L'Aquila, the number of deaths was clustered in an area of topographic amplification and poor quality buildings.

7.1.7 Displaced Persons

Over 65,000 people, approximately half of the population of L'Aquila and surrounding areas, were still in temporary housing (tent cities, hotels on the Adriatic coast, and private residences) at the time of this reconnaissance mission.

Around 40,000 people who were made homeless by the earthquake found accommodation in tented camps and a further 10,000 were housed in hotels on the coast. However, many people living in hotels along the coast would have preferred to stay in tent cities rather than in a hotel far away from their home, because they would have more contact with their neighbors and could do things such as cooking and be less restricted by hotel rules.

Table 7-1 Persons in Hotels Along the Coast

Teramo	6000	110 hotel
Pescara	2000	31 hotel
Chieti	2000	30 hotel
TOTAL	**10000**	**171 hotel**

7.2 Consequences and Implications

Given the extent of devastation, a wide range of social, economic, cultural, and technical issues will pose major challenges for reconstruction of L'Aquila in the coming years.

The L'Aquila earthquake raises many questions that should be addressed in future social science and policy research. Follow-up research is needed to better understand long-term impacts upon the surviving population, including psychosocial impacts, and to track the social and economic recovery process of households and businesses. Special attention should be paid to at-risk populations, such as those who suffered very severe losses, disrupted families, and youth.

Additionally, research is needed to evaluate the provision of services to affected populations, including mental health care, temporary and permanent housing aid, and assistance provided to businesses. Studies assessing the effectiveness of governmental aid programs and policies could provide valuable lessons both for Italian society and for other nations. A number of conferences and workshops have already been held in L'Aquila under the auspices of other agencies to identify "lessons learned" as a result of the earthquake.

SECTION 8

EMERGENCY INTERVENTIONS FOR SAFETY MEASURES

The 2009 L'Aquila earthquake is one of the most dramatic examples of destruction of historical monuments by natural forces. Immediate shoring is vital, as is the freeing of the structure from the overloading caused by debris which has fallen onto it and in addition its protection against rainwater seepage. Nowadays, most of this debris has been piled in the streets creating mountains of debris, therefore the main problem that is facing L'Aquila downtown is the removal of the debris, in fact without its removal, is not possible to start any reconstruction.

Seismic aftershocks create more damage to weakened buildings, therefore the inability to act quickly, -even to shore up the structure on a temporary basis- can result in the building demolition. Penetrating rain, uneven settlement and continuing aftershocks can easily undermine a building's remaining structural integrity.

This chapter presents some examples of effective and not effective shoring up interventions in historical palaces in downtown L'Aquila after the seismic event. Most of the shoring interventions have been directed reasonably well, but sometimes they were done on half-destroyed buildings that were in that condition even before the earthquake and they should instead be only demolished.

The main purpose of this engineering analysis of the buildings is to obtain the best solution that minimizes the intervention and, consequently, to balance between the need for safety and integrity, and the need for preservation of the original structure.

When an earthquake damages the buildings, one of the first actions needed is to prevent progressive damage due to aftershocks, protect life of people and rescuers and quickly restore normal economic and social activities. The urgency of immediate action can be achieved only with provisional retrofit interventions which are simple to install and use available materials. These safety structures are provisional and they will be removed after a certain period of time. They are necessary to allow access to the monument to do in-situ measurements and investigations, laboratory tests and rigorous structural analyses.

The objective of this chapter is to outline the techniques, materials and design procedures in use for the shoring provisional intervention of historical buildings and monuments damaged by earthquakes. The retrofit intervention techniques can be grouped into three categories: (i) props, (ii) rods (iii) and hoops.

8.1 Props

Props are devices to support the weaker parts of the building. They have a *replacement function* when they substitute the capacity of the structural elements damaged by the earthquake.

They have a *caution function* when the building might suddenly collapse even if it is still standing. They have a protective function when there is risk of detachment of some parts of the building.

Props that are designed to prevent the fall by tilting of portion of walls are called *support props*.

Props that are designed to avoid the collapse of arches, vaults, lintels (Figure 8-2, Figure 8-4) are called *restraint props*.

Support props can be oblique and fixed on the ground or horizontal pushing against other buildings.

Restraint props are always vertical or sub-vertical (Figure 8-1).

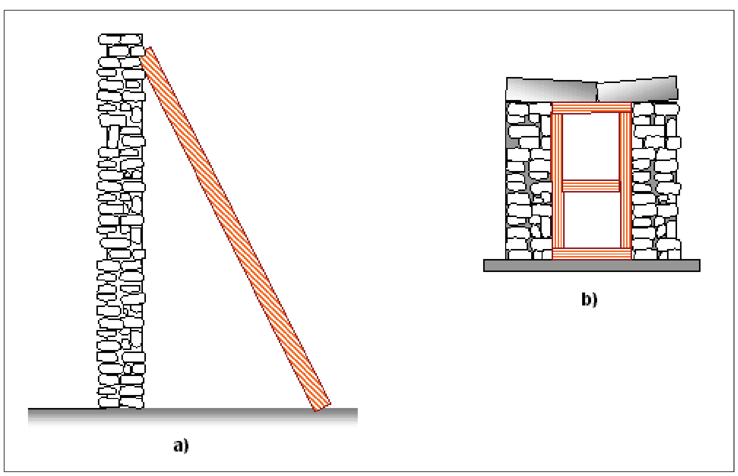


Figure 8-1 (a) Support props and (b) restraint props

The fundamental elements of shoring are the props that can be made of either steel or wood (Figure 8-3). Wooden props should be made with good quality materials, free of cracks and with few knots and seasoned (Figure 8-2a). Nailing should be perpendicular to the strength path. Steel props can be adjusted in height with length that can vary between 1.7 and 4.0 m (Figure 8-2, Figure 8-3b).

(a) (b)

Figure 8-2 (a) Restraint metal prop in the Spanish fortress to avoid the collapse of the lintel and (b) restraint wooden props in the Palazzetto dei nobili

Figure 8-3 (a) Wooden props and (b) metal props of De Amicis school in L'Aquila

Figure 8-4 Shoring of the stairs of Margherita palace

8.2 Rods

Rods or chains are steel elements linking two parallel walls to keep them stuck to the floor or to the orthogonal walls (Figure 8-5). They eliminate the possibility of wall openings due to the absence of ring beams or to the presence of a "pushing roofs". They can be used to keep together vaults and arches (Figure 8-5b).

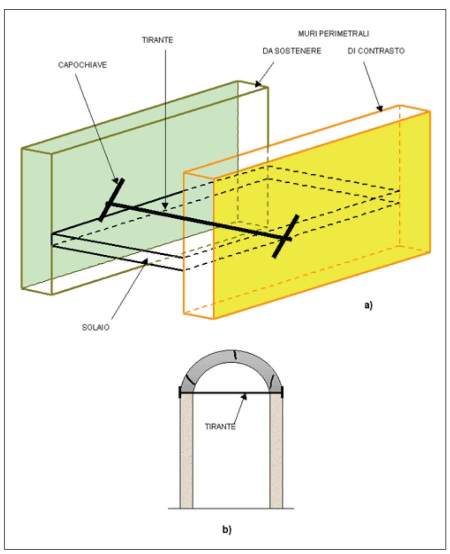


Figure 8-5 (a) Rods on walls and (b) rods on cracked arch

Rods, with respect to props, provide better seismic performances during the vibrations due to the earthquake, they occupy less space, and they use less material and can be left in place as definitive retrofit intervention.

Figure 8-6 Civic Tower of L'Aquila wrapped with external rods

They have the disadvantage of requiring operations inside the building to pass the rod through the walls (Figure 8-7), unless they are placed outside to achieve a hoop of the building (Figure 8-6). Installation of provisional roofing of resistant plastic is necessary for some buildings to protect their content against rain and snow.

Figure 8-7 Detailed view of holes inside the building to pass the rods

Tie rods are anchored to the walls through bolt or circular plates (Figure 8-8). The masonry wall around the plate should be of good quality in order to guarantee a proper behavior of the tie rod.

Figure 8-8 Close up of the bolts of the tie rod in the Spanish fortress

8.3 Hoops

Structural elements subjected to vertical compression tend to shrink vertically and expand transversally. Hoops avoid transversal expansion, thereby increasing the resistance to compression, by including rings equally distributed, or spiral bands of high-strength polyester (Figure 8-9, Figure 8-10).

Figure 8-9 Hoops retrofit intervention of the columns of Santa Maria di Collemaggio Basilica

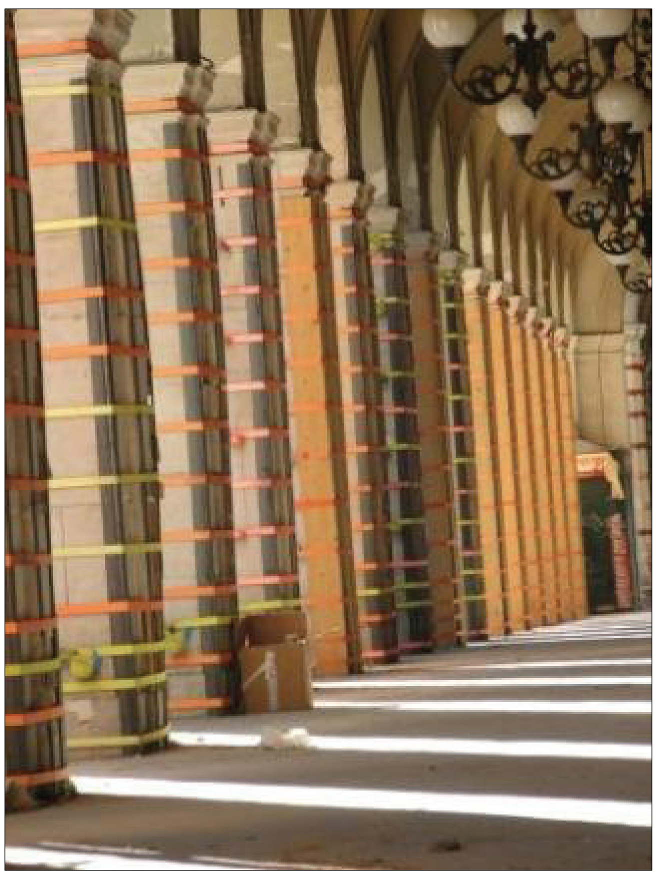


Figure 8-10 Hoops intervention of the columns of the "Quattro Cantoni" complex block

The hoops can also be made on whole buildings, as an alternative to the shore intervention (Figure 8-11).

8.4 Provisional Intervention to Avoid the Posting of the Front Facade

The damage develops due to lack of connection between the facade and the orthogonal walls or to the absence of ring perimeter beams. The retrofit intervention can be obtained through wooden props up to a height of 6-7 meters, and metal props can be used for greater heights (Figure 8-3b), or with the hoops of the whole building (Figure 8-11). It's important that the props are arranged all in the same way. The shoring wall will then split into equal parts and each strip corresponds to a shore. When the area below the building should need to be cleared, it is possible to use the propped up against another building provided the following conditions: (i) the adjacent building must be robust and in good condition; (ii) the two buildings should be close; (iii) the props on the adjacent wall should be located in correspondence of the orthogonal wall; and (iv) the props must be connected with both walls so that in case of an earthquake, they can avoid becoming detached and start hammering on one of the two walls.

Figure 8-11 Hoops intervention of the whole building of the University: Palazzo Carli

8.5 Provisional Interventions to Avoid the Posting of the Damaged Corners

The damage occurred due to poor connection of the perimeter walls, for lack of ring beams, and/or the presence of openings (doors or windows) in the vicinity of the corners.

(a) (b)

Figure 8-12 (a) Hoop retrofit intervention on damaged corner in L'Aquila with (b) high-strength polyester bands

The retrofit intervention can be done through the prop of the two sides involved or through the hoops of the building (Figure 8-12, Figure 8-13).

Figure 8-13 Retrofit intervention of the damaged corner of Quattro Cantoni with hoops

8.6 Ex Monastery of the Lauretane

Ex Lauretane monastery is a XV century building that now is property of the Province of L'Aquila and is a good example of bad shoring intervention. It is an assembly of three buildings of rectangular shape with a backyard (Figure 8-14). In the complex are visible tie rods in the frontal façade on the west side on the first and the second story level.

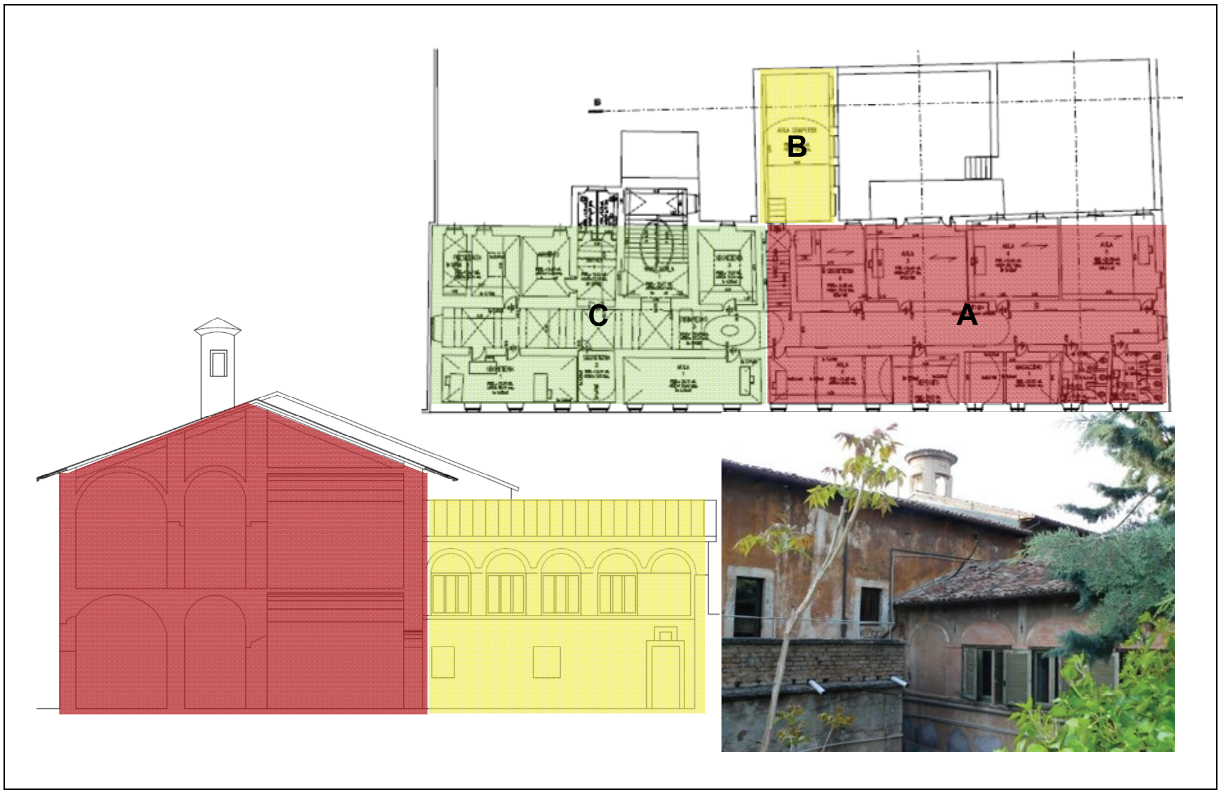


Figure 8-14 Pushing props that increased the damage in the barrel vault of the Ex monastery of Lauretane

Main damages observed in the building are:

(i) High risk of collapse of the lantern

(ii) Damage in the two adjacent buildings due to the hammering of the ridge beam

(iii) Longitudinal crack openings in the barrel vault of the corridor

(iv) Tilting of the side walls of the building C

The longitudinal crack openings in the barrel vault of the corridor appeared first after the foreshock of March 30th. They tried to shore the vault using wooden props located as shown in

Figure 8-15. However, the presence of the pushing wooden props created an extra horizontal force that increased the crack openings after the seismic event of April 6th.

The longitudinal crack opening in the corridor was in building A, but not in the corridor of building C, because building A was retrofitted with chains that were stopped at the corridor level (Figure 8-15), probably for esthetic reasons, and therefore during the seismic event the building behaved as two independent structural elements.

Additionally, the hammering effect of the ridge beam of the orthogonal building B also increased the longitudinal crack opening of the barrel vault of building A. In conclusion, the retrofit intervention, before the seismic event and also the provisional shoring intervention after March 30th not only were ineffective, but they contributed to increase the structural damage of the ex monastery.

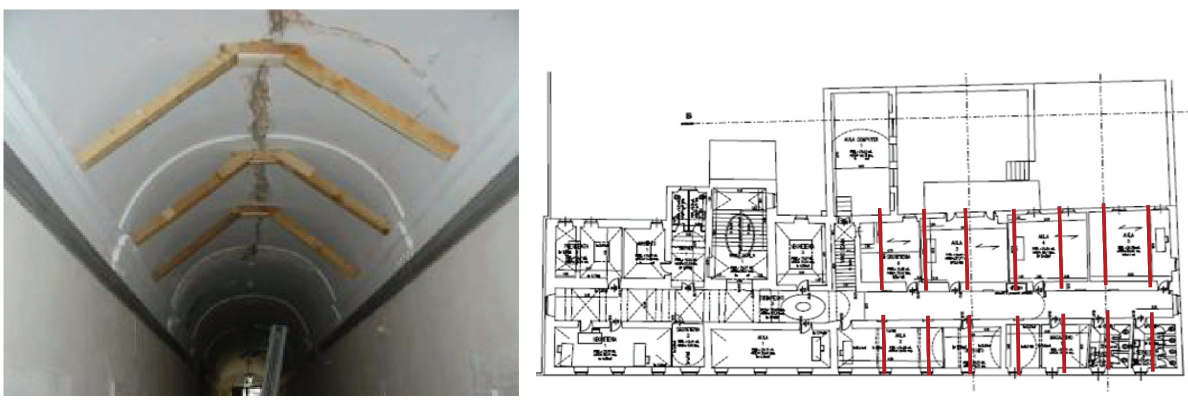


Figure 8-15 Pushing props that increased the damage in the barrel vault of the Ex monastery of Lauretane

8.7 Palazzo Margherita

The palace started being built in 1294 and originally was used as a civic building for the Captain of Justice. The architect Jerome Pico Fonticulano was the designer of the renovation of the Renaissance town hall in 1541.

Figure 8-16 Provisional intervention with rods in Palazzo Margherita

In 1573, the building was almost completely rebuilt by Battista Marchirolo of Naples and it became the house of Margaret of Austria, daughter of Charles V, wife of Alessandro de Medici and later of Ottavio Farnese, who after being clever and energetic governor of the Netherlands, was elected governor of Abruzzo. Unfortunately, it seems that the building does not retain much of the first building. In addition, it does not remain significant parts of the reconstruction of the second half of the sixteenth century structures in the current version of eight-nineteenth century. Only the plan settings and the monumental proportions, and the pleasant courtyard compound, are elements of late sixteenth century that recall the importance of the place.

More recently, in the early 90's the architect Macri made a retrofit intervention in Palazzo Margherita creating a curb of concrete under the roof cover with the aim of giving a box behavior to the whole building.

The building is made of two levels with rectangular plan and internal courtyard. The structure is made primarily of stone and brick masonry with good quality mortar. The floors are made of masonry vaults with and without tie rods in both directions at the ground floor and of steel and brick at the first and second floor. The evaluation of the seismic behavior of Palazzo Margherita is done considering the possible local damage mechanisms. In the old masonry structures, the collapse is determined by the lack of connections between orthogonal walls and by discontinuities not always visible.

The principal damage mechanism of Palazzo Margherita is the “simple overturning wall mechanism”. This collapse mechanism is characterized by a rigid rotation of portion of the wall around a horizontal cylindrical hinge located at the base. The overturning mechanism of the internal wall on the North side of the courtyard developed longitudinal cracks in the cross vaults of the arcades and other damages inside the building.

The street of via Bafile is a main road for downtown L'Aquila, therefore it needs to be maintained as a viable road and props could not be used, because they would have occupied the entire road. Then rods were used, because they occupy less space and they use less material. In order to reduce intervention inside, the building rods passed through the windows and were able to connect the external walls with the internal walls of the courtyard (Figure 8-16). The retrofit intervention on the civic tower was done with external rods that were used until the top of the tower. The authors question the necessity of this retrofit intervention, because tie rods were already inserted inside the tower and they guarantee a good box behavior. For the reason above, the shoring intervention of the tower could possibly be limited to the lower part where more cracks were visible.

SECTION 9

FROM PERFORMANCE–BASED DESIGN TO RESILIENCE-BASED DESIGN

9.1 Introduction

In recent years, the concept of resilience has gained attention recognizing the fact that not all threats or disasters can be averted. Indeed, societies are turning their attention to efforts and ways that can enhance the resilience of entire communities against various types of extreme events. Resilience is clearly becoming increasingly important for modern societies as communities come to accept that they cannot prevent every risk from being realized but rather must learn to adapt and manage risks in a way that minimizes impact on human and other systems. While studies on the disaster resilience of technical systems have been undertaken for quite some time (Chang and Shinozuka, 2004), the societal aspects and the inclusion of various and multiple types of extreme events are new developments. In this regard, communities around the world are increasingly debating ways to enhance their resilience.

In earlier work by Bruneau et al. (2003), resilience was defined including technical, organizational, economic, and social aspects and with four main properties of robustness, redundancy, resourcefulness, and rapidity. The quantification and evaluation of disaster resilience was based on non-dimensional analytical functions related to the variations of functionality during a "period of interest," including the losses in the disaster and the recovery path (Cimellaro et al., 2009). This evolution over time, including recovery, differentiates the resilience approach from other approaches addressing only loss estimation and its momentary effects.

The objective of this chapter is to describe a holistic framework for defining and measuring disaster resilience for a community at various scales. Seven dimensions characterizing community functionality have been identified and are represented by the acronym PEOPLES (Renschler et al. , 2010): *Population and Demographics, Environmental/Ecosystem, Organized Governmental Services, Physical Infrastructure, Lifestyle and Community Competence, Economic Development, and Social-Cultural Capital*. Further details about the description of each one of the dimensions of the PEOPLES framework can be found in Renschler et al. (2010a, 2010b). The framework provides the basis for development of quantitative and qualitative models that measure continuously the functionality and resilience of communities against extreme events or disasters in any or a combination of the above mentioned dimensions.

A new more general design methodology is developed in this chapter called "Resilience-Based Design" (RBD) which can be considered as an extension of Performance-Based Design (PBD) which is only a part of the total "design effort". The goal of RBD is to make individual structures and communities as "Resilient" as possible, developing technologies and actions that allows each structure and/or community to regain its function as promptly as possible.

9.2 Limitations of Performance-Based Earthquake Engineering (PBEE)

The main equation of PBD is based on the total probability theorem de-aggregating the problem into several interim probabilistic models (namely seismic hazard, demand, capacity and loss models) and it is defined as follows (Cornell and Krawinkler, 2000)

$$\underbrace{\lambda(dv < DV)}_{Seismic\ risk} = \iiint \underbrace{G(dv|dm)}_{Loss\ Analysis}\, \underbrace{dG(dm|edp)}_{Damage\ Analysis}\, \underbrace{dG(edp|im)}_{\mathrm{Re}sponse\ Analysis}\, \underbrace{|d\lambda(im)|}_{PSHA} \tag{9.1}$$

where *im*=intensity measure (e.g $Sa(T_1)$, epsilon, $S_{dinelastic}$, duration etc.); *dm*=damage measure (e.g physical condition & consequences/ramifications); *edp*=engineering demand parameters (e.g., drift ratio (peak, residual), acceleration, local indices etc.); *dv*=decision variable (e.g., loss, functionality, downtime, casualties etc.); $\lambda(dv)$=mean annual frequency of a decision variable (*dv*); G(a|b) is the probability of exceedance $a>a_0$ given *b*. Each component of Equation (9.1) needs to be determined statistically. Although this methodology is rapidly spreading, there are parts that the PBEE does not cover such as:

1. Portfolio assessment
2. Community assessment

The concept of Performance-Based Design /Engineering can be applied to describe the behavior of a single building or structure, but the performance of an individual structure is not governed by its own performance, but interacts heavily with the performance of other entities within the same community. A clear example of these interdependencies between the building and the community is for example a hospital, which will not be able to perform without electricity and water even if the structure has no structural damage. Another example of the limitations of PBD is given by 2009 L'Aquila earthquake, during which the small town of

Castelnuovo was completely destroyed, except a single housing unit that was standing after the earthquake and suffered minor damage. According to PBD the building is ok, but in RBD the housing unit would be not operational, because it is not able to interact with other entities inside the same community.

9.3 Resilience-Based Design

Today, designers and engineers approach a structure as if it stands alone, without considering the interaction with the community, which instead should be considered as an integrated part of the design process. There is now a new fundamental way of looking at all the problems. The building is not considered alone, but as a group of buildings using the "Portfolio Approach" which will allow regional loss analysis. Thus, it will be moved from housing units to housing blocks. This concept is borrowed from the financial industry, where Modern Portfolio Theory (MPT) was developed in the 1950s through the early 1970s and was considered an important advance in the mathematical modelling of finance. MPT is defined as a theory of investment which attempts to minimize risk for a given level of expected return (performance), by carefully choosing the proportions of various assets. MPT is a mathematical formulation of the concept of diversification in investing, with the aim of selecting a collection of investment assets that has collectively lower risk than any individual asset. Analogously, the concept of diversification can be applied in the field of disaster resilience, where diversification in retrofit of different buildings in a given region can increase resilience collectively more than any individual retrofit.

MPT models an asset's return as a normally distributed function, defines risk as the standard deviation of return, and models a portfolio as a weighted combination of assets so that the return of a portfolio is the weighted combination of the assets' returns. Similarly the risk in RBD can be defined as the standard deviation of the performances of each housing unit; therefore the performances of a given community can be defined as the weight combination of the performance of each housing unit. It is important to mention that community is a complex system where losses as well as the recovery process are coupled dimensions which involve several parameters which are not only engineering parameters such as drift and accelerations, but also other parameters such as socio-economic gender, age of the population etc. All these parameters are used to define the recovery that becomes part of the design process in RBD and therefore should be planned upfront.

9.3.1 Definition of Resilience

At this time, there is no explicit set of procedures in the existing literature that suggests how to quantify resilience in the context of multiple hazards, how to compare communities with one another in terms of their resilience, or how to determine whether individual communities are moving in the direction of becoming more resilient in the face of various hazards. Considerable research has been accomplished to assess direct and indirect losses attributable to earthquakes, and to estimate the reduction of these losses as a result of specific actions, policies, or scenarios. However, the notion of resilience suggests a much broader framework than the reduction of monetary losses alone.

Resilience (R) may be defined as a function indicating the capability to sustain a level of functionality or performance for a given building, bridge, lifeline networks, or community, over a period defined as the control time T_{LC} that is usually decided by owners, or society at large, for example, and corresponds to the expected life cycle or life span of the building or other system. Resilience, R, is defined graphically as the normalized shaded area underneath the function describing the functionality of a system, defined as Q(t). Q(t) is a non-stationary stochastic process, and each ensemble is a piecewise continuous function as shown in Figure 9-1, where Q(t) is the functionality of the region considered. The change in functionality due to extreme events is characterized by a drop, representing a loss of functionality, and a recovery.

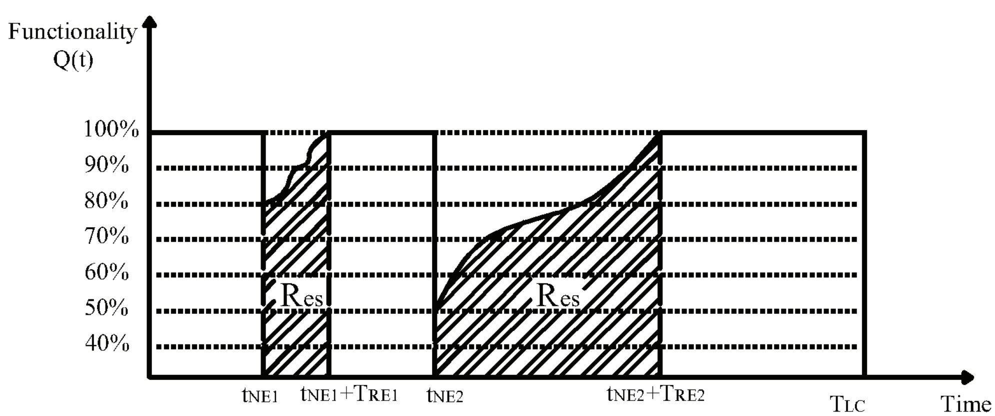


Figure 9-1 Community Resilience

Analytically, Resilience is defined as

$$R(\vec{r}) = \int_{t_{OE}}^{t_{OE}+T_{LC}} Q_{TOT}(t)/T_{LC}\, dt \tag{9.2}$$

where $Q_{TOT}(t)$ is the global functionality of the region considered (Cimellaro et al. 2005, 2006) which is given in Equation (9.3). For example functionality is the combination of all functionalities related to different facilities, lifelines, etc. for the case when physical infrastructures resources and services are considered.

Disaster resilience is often divided between technological units and social systems. On a small scale, when considering critical infrastructures, the focus is mainly on technological aspects. On a greater scale, when considering an entire community, the focus is broadened to include the interplay of multiple systems – human, environmental, and others – which together add up to ensure the functioning of a society.

In order to emphasize the primary role of the human system in community sustainability, the acronym "PEOPLES" (Renschler et al. 2010) has been adopted in order to describe the framework that is built on and expands previous research at MCEER linking several previously identified resilience dimensions (technical, organizational, societal, and economic) and resilience properties (r4: robustness, redundancy, resourcefulness, and rapidity) (Bruneau et al. 2003, Cimellaro et al. 2009). PEOPLES incorporates MCEER's widely accepted definitions of service functionality, its components (assets, services, demographics) and the parameters influencing their integrity and resilience.

Resilience can be considered as a dynamic quantity that changes over time and across space. It can be applied to engineering, economic, social, and institutional infrastructures, and it can use various geographic scales (Figure 9-2).

The first step in order to quantify the resilience performance index (R) is to define the spatial scale (e.g., building, structure, community, city, region. etc.) of the problem of interest (Figure 9-2).

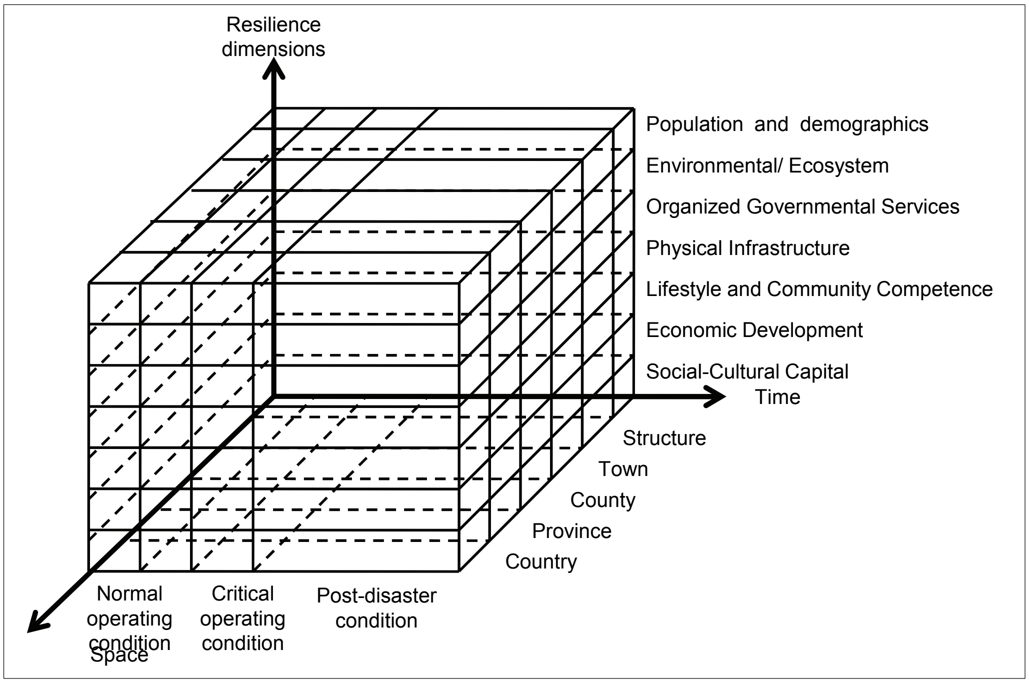


Figure 9-2 Spatial and temporal dimension of Resilience-Based design (RBD) using PEOPLES approach

This selection will define the performance measures to be considered to define the global functionality of the system (Equation (9.3)).

The second step is to define the temporal scale (short term emergency response, long term re-construction phase, midterm reconstruction phase etc.) of the problem of interest (Figure 9-2). The selection of the control period T_{LC} will affect the resilience performance index, therefore when comparing different scenarios the same control period should be considered.

9.3.2 Layer's Model

The general framework at the community level is described by the following equations where for each dimension a performance indicator and /or functionality is defined which is combined with other functionality dimensions as follows

$$Q_{TOT}(t) = Q_{TOT}(Q_P, Q_E, Q_O, Q_{Ph}, Q_L, Q_E, Q_S) \tag{9.3}$$

where Q_{TOT}=global functionality; Qx=functionality of each of the seven dimensions according to PEOPLES framework. Within each dimension functionality is defined as a combination of functionality of the respective subsystems, for example the functionality of the physical infrastructures is defined as follows

$$Q_{Ph}(t) = Q_{Ph}\left(Q_{Hosp}, Q_{Ele}, Q_{Road}, Q_{Water}, \ldots\right) \tag{9.4}$$

This list of functionality terms that can be inserted within the physical infrastructure block is not complete, and additional terms can be eventually added, such as functionality of schools, stations, fire stations, oil and natural gas systems, nuclear facilities, emergency centers, communication towers/antennae, etc. Once the geographic scale and the global functionality Q_{TOT} is defined, it is possible to plot its value over the region of interest in a contour plot at a given instant of time *t*, so time-dependent functionality maps of the region can be obtained. When also the temporal scale is defined through the control time T_{LC}, then the resilience contour map of the region of interest can be plotted. The Resilience maps will vary in space from point to point, but it will be time independent and described by Equation (9.2). Finally, the community resilience index is given by the double integral over time and space as follows:

$$R_{com} = \int_{A_C} R(\vec{r})/A_C dr = \int_{A_C} \int_{t_{OE}}^{t_{OE}+T_{LC}} Q_{TOT}(t)/(A_C T_{LC})\,dtdr \tag{9.5}$$

For each dimension, a contour plot can be determined and combined using a layered approach as the one shown in Figure 9-3. Then a radar graph can be plotted and the area will define the final value of the resilience score (Figure 9-4) for the region of interest. This will allow identifying gaps as well as priority actions, which will enter in the decision loop shown in Figure 9-5.

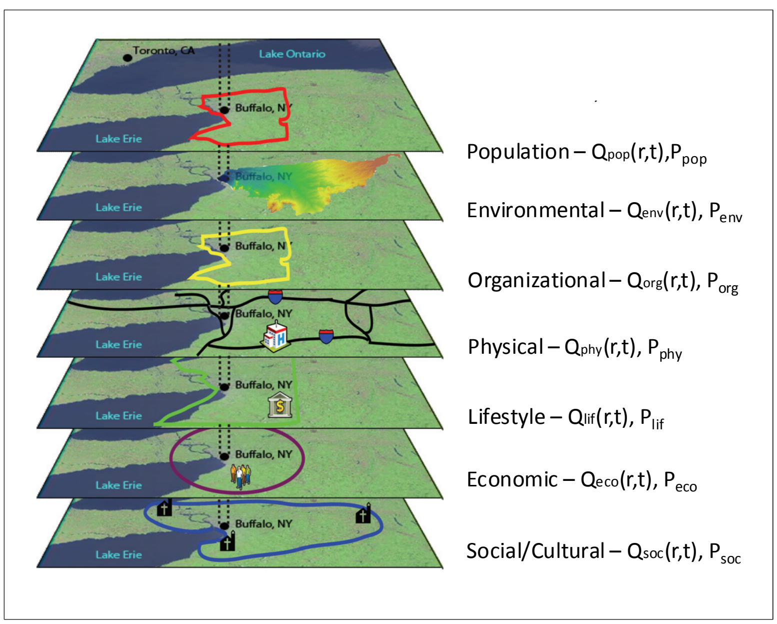


Figure 9-3 Layer model of PEOPLES (Cimellaro et al, 2011)

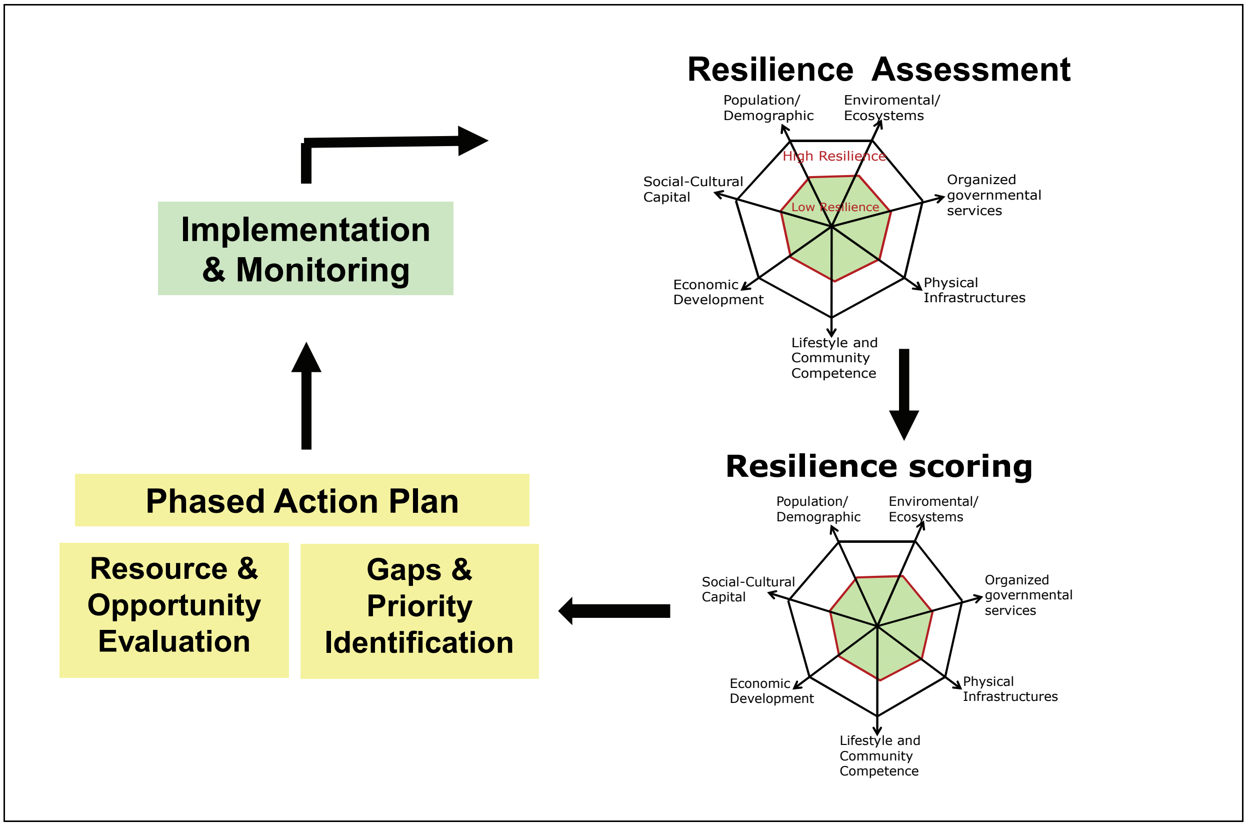


Figure 9-4 Resilience scoring using PEOPLES methodology

In summary, a schematic step-by-step procedure of the MCEER methodology described in Figure 9-5 is as follows:

1. Define extreme event scenarios (e.g., PSHA, ground motion selection)
2. Define the system model
3. Evaluate the response of the model
4. Compute different performance measures (e.g., losses, recovery time, functionality, resilience)
5. Identify remedial mitigation actions (e.g., advanced technologies) and/or resilience actions (e.g., resourcefulness, redundancy, etc.)

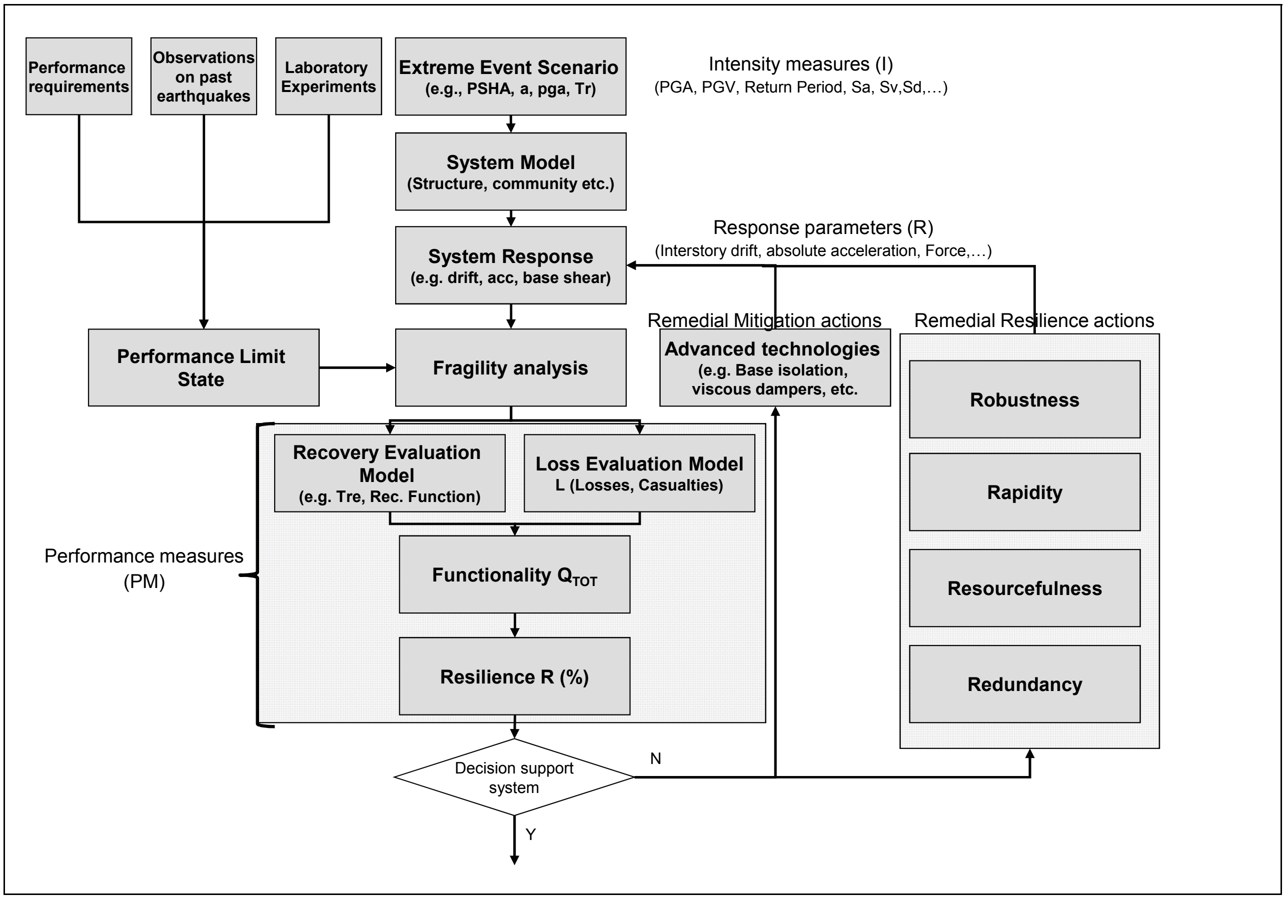


Figure 9-5 MCEER methodology for Resilience-based design (RBD) based on control (feedback loop) approach

This design approach has analogies with the feedback loop taken from control theory. The same framework can be used for a region as well as a single structure (e.g., hospital). In this case, the functionality Q_H reduces the functionality of a single hospital Q_H which can be evaluated with the procedure described in Cimellaro et al. (2011) for example, so the resilience index of a single hospital can be evaluated as follows

$$R_{HU}\left(\vec{r}\right)=\int_{t_{OE}}^{t_{OE}+T_{LC}} Q_{HU}\left(t\right)/T_{LC}\,dt \tag{9.6}$$

However, in Resilience-Based Design (RBD), the hospital or in general a building is part of a community, therefore its functionality cannot be considered independent of the rest of the system (e.g., a housing unit without water and electricity or without roads connecting it to the rest of the world cannot be considered functional even if it does not have any structural damage). This example in very simple terms distinguishes RBD from PBD, where PBD would consider the house ok, while in RBD the house would not satisfy the requirements.

9.3.3 Uncertainties of Resilience-Based Design

Either a deterministic or probabilistic approach can be used within the RBD with preference to the latter approach when a particular level of confidence of achieving performance objective is of interest. The MCEER methodology also has a probabilistic approach which is more general even if the information provided to the public (e.g., decision makers, politicians, etc.) should be deterministic, because it is more simple and easy to understand. The main equation of RBD based on the total probability theorem is as follows (Cimellaro et al., 2006, 2010)

$$\bar{\mathrm{R}} = \int_{T_{RE}}\int_{L}\int_{DM}\int_{R}\int_{i^*} r_i \cdot P(T_{RE}/Q)\,P(Q/PM)\,P(PM/R)\,P(R/I)\,P\left(I_{T_{LC}} > i^*\right) dI \cdot dR \cdot dPM \cdot dQ \cdot dT_{RE}; \qquad (9.7)$$

9.3.4 Resilience Performance Levels

The objective of Performance Based Seismic Engineering (PBSE) is to design, construct and maintain facilities with better damage control. A comprehensive document has been prepared by the SEAOC Vision 2000 Committee (1995) that includes interim recommendations. The performance design objectives couple expected or desired performance levels with levels of seismic hazard as illustrated by the Performance Design Objective Matrix shown in Figure 9-6.

Earthquake Design Level		Earthquake Performance Level			
		Fully Operational	Operational	Life Safe	Near Collapse
	Frequent (43 years)	Basic Objective	Unacceptable	Unacceptable	Unacceptable
	Occasional (72 years)	Essential/Hazardous Objective	Basic Objective	Unacceptable	Unacceptable
	Rare (475 years)	Safety Critical Objective	Essential/Hazardous Objective	Basic Objective	Unacceptable
	Very Rare (975 years)	Not Feasible	Safety Critical Objective	Essential/Hazardous Objective	Basic Objective

Figure 9-6 Recommended seismic performance objectives for buildings (SEAOC Vision, 2000)

Performance-Based design levels focus on the performances a building can hold during the shaking and are associated to engineering demand parameters such as deformations (Figure 9-7).

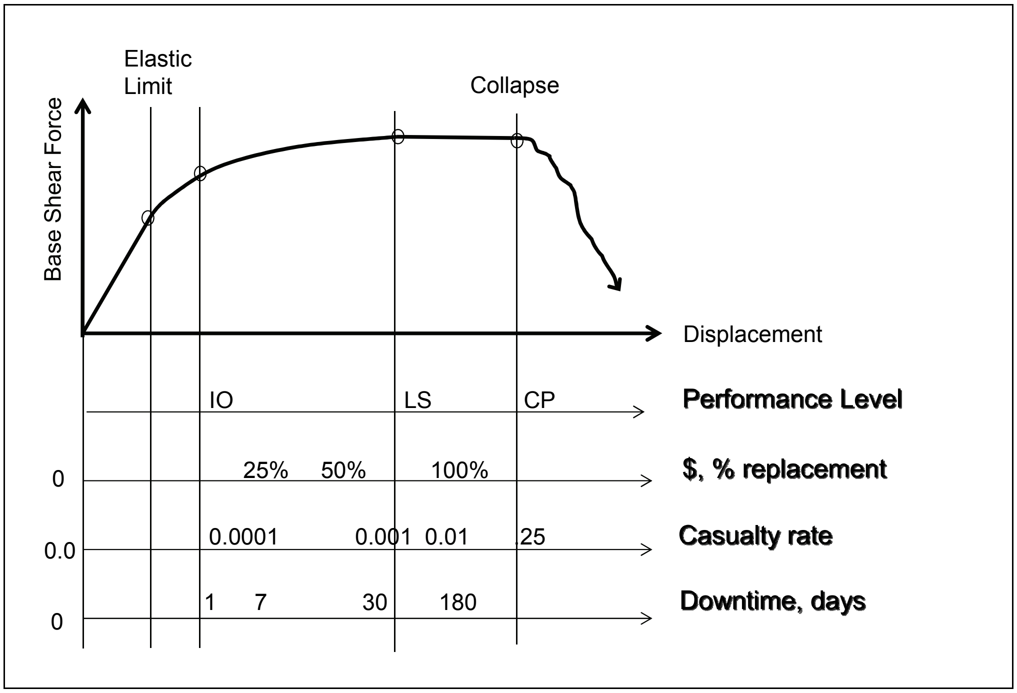


Figure 9-7 Performance levels of PEER methodology

More recently, SPUR (2009), which is the San Francisco Planning and Urban Research Association, introduced other definitions of performance levels for physical infrastructures based on recovery target states which take into account the safety as well as the recovery time. Five performance measures for buildings have been identified:

1. Safe and Operational
2. Safe and usable during repair
3. Safe and usable after repair
4. Safe but not repairable
5. Unsafe

In Japan, there was a trial to find the desired performance goals in downtime and bridge performance from public interviews (Figure 9-8). This information will be a useful guideline to define the boundaries of the different resilience performance levels.

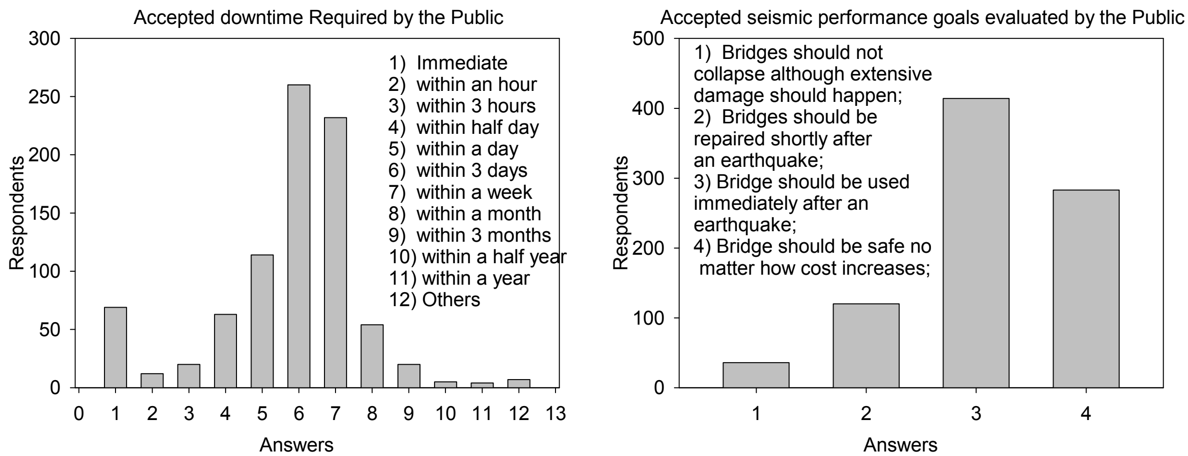


Figure 9-8 (a) Accepted downtime required by the public; (b) Accepted seismic performance goals evaluated by the public (Yokohama National University, 2011)

The new proposed Resilience Performance Levels (RPL) focus on building performance after the earthquake stops, recognizing the importance of the temporal dimension (Recovery time T_{RE}) in the assessment of the RPLs of structures and communities in general. A 2-D performance domain consisting of Performance Levels PL(i, j), defined by the combination of Functionality (index j) and recovery time (index i) is proposed. By accounting for the effect of the temporal dimension, a 3-dimensional performance matrix (Figure 9-9) can be visualized as a set of predefined joined performance domains ("masks") for different seismic intensity levels, IM and different RPLs.

		Recovery Time		
		Short term (Emergency)	Midterm	Long term (Reconstruction)
Functionality Performance Levels	Fully Operational (Q1)	Basic Objective	Unacceptable	Unacceptable
	Operational (Q2)	Essential Objective	Basic Objective	Unacceptable
	Partially Operational (Q3)	Critical Objective	Essential Objective	Basic Objective
	Near not Operational (Q4)	Not feasible	Critical Objective	Basic Objective

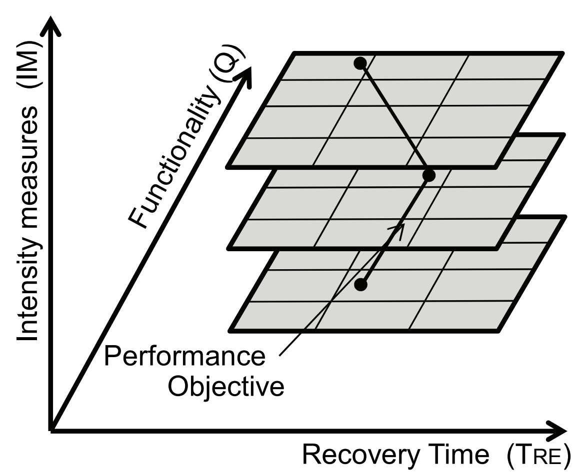


Figure 9-9 Three-dimensional Resilience Performance objectives matrix for structures, communities, systems etc.

9.3.5 Examples of Applications of RBD

One of the requirements of RBD is to achieve redundancy in a given community, which is a complex system of interacting sub-systems where both the performance objectives of individual structures and the performance objectives of the entire community can be measured. In a selected region, resilience can be improved by increasing the number of hospitals in the region advocating redundancy. However, one of the aspects that should be taken into account in RBD which is not taken into account in the current practice is that, for example, if the region is affected by the same earthquake hazard is it resilient to build the new hospital with the same retrofit technique of the already existing hospital? Selecting different retrofit strategies for the new hospital might enhance the probability of survival of at least one hospital in case of an extreme event. Following the same concept, an example of design that has followed the current practice of PBD is the C.A.S.E. project realized after 2009 L'Aquila earthquake in Italy. It allowed several new housing units with the same type of retrofit consisting of a base isolation system realized with friction pendulum bearings. If, in a very rare scenario, the next earthquake was characterized by low frequency content, probably all these new housing units would collapse, because they do not have diversity in performance. When looking at all housing units as a system, they are not redundant, because resilience has not been considered in the design process.

9.4 Concluding Remarks

After the L'Aquila earthquake and the many recent extreme events worldwide, the international community became aware that resilience could be the key to describing earthquake engineering performance.

A new more general design methodology is developed in this chapter called "Resilience-Based Design" (RBD), which can be considered as an extension of PBD, which is only a part of the total "design effort". The goal of RBD is to make individual structures and communities as "Resilient" as possible, developing technologies and actions that allows each structure and/or community to regain its function as promptly as possible. The definition of community resilience combines information from technical and organizational fields, from seismology and earthquake engineering to social science and economy. Thus, it is clear that many assumptions and interpretations are made during the study of seismic resilience, but the final goal is to integrate the information from these fields in a unique function that achieve results that are unbiased by uninformed intuitions or preconceived notions of how large or how small the risk is. The goal of this chapter is to provide a quantitative definition of resilience in a rational way through the use of an analytical function that may fit both technical and organizational issues. The fundamental concepts of community resilience are analyzed and a common frame of reference is established. However, even if the performance evaluation of an individual structure is the engineers` goal, the level of performance required should not be determined individually, but on a community basis. A Resilience Performance level matrix combining functionality and recovery time as performance levels and performance objectives at increasing levels of seismic intensity has been presented. The resilience-based approach is illustrated in detail in some examples, showing its efficiency and better description with respect to PBD. However, it is important to mention that the assumptions that are made for the case presented are only representative to illustrate the definitions; for other problems, users calculating resilience should focus on the assumptions that most influence the problem at hand.

SECTION 10

CONCLUSIONS

10.1 Summary and Conclusions

MCEER researchers have collected a substantial amount of data from field investigations in the area affected by L'Aquila earthquake. In this earthquake more than other, the catastrophic level of damage to masonry buildings and infrastructures was largely caused by soil amplification factors.

Investors observed that the majority of reinforced concrete buildings performed well structurally, but many nonstructural components collapsed, such as partition and infill walls.

The information presented in this report expands on multi-hazard studies to better understand the similarities and differences between various potential hazards such as hurricanes, earthquakes and terrorist attacks.

10.2 Rebuilding Strategies

L'Aquila downtown must be recovered to become a safer place where the built environment is made resilient to future disasters of any kind. This is a key issue to keep in mind while revitalizing the area, which will necessarily call for some significant changes. A seismic and geological map of the area as well as of the infrastructure and materials in place before the earthquake will guide decisions on where to intervene. Before starting reconstruction interventions, it is important to map such characteristics using available data and, when necessary, new techniques. Reconstruction interventions, however, should also consider the subjective aspects of revitalization: the value of the memories of places, which are the sources people's attachment to them. Structural changes should be discussed with the population to make clear both reconstruction benefits as well as potential physical changes.

Some building strategies are proposed to guide the rebuilding process in the affected region as follows.

1. An extensive public information program should be initiated to keep the public as informed as possible on the rebuilding effort, trying to track the funding that will be used for the reconstruction to minimize the personal interests of construction companies and illegal activities;
2. Owners of damaged properties who want to build as quickly as possible or without meeting the necessary seismic resistant standards should be educated with constant workshops and seminars aimed at teaching design professionals and construction technicians about more seismic resistant building practices.
3. The entire region affected by the earthquake should be divided into zones and nonprofit organizations should be established to administer and manage the rebuilding process. This strategy is aimed at addressing public concerns that financial resources being allocated to the recovery effort be appropriately spent and monitored. These organizations are composed of representatives from government, business and industry, academic institutions, professional associations, and property owners.

10.3 The CASE Project

Different plans for constructing new houses in L'Aquila have been realized in the post-earthquake scenario. Below is the list of the four main plans:

1. C.A.S.E. (Complessi Antisismici Sostenibili ed Ecocompatibili)
2. M.A.P. (Moduli Abitativi Provvisori)
3. M.A.P.- Frazioni (Moduli Abitativi Provvisori Frazioni del Comune di L'Aquila)
4. M.U.S.P. (Moduli ad Uso Scolastico Provvisori)

Among all, the biggest investment by the government was realized in the C.A.S.E. project; therefore a brief explanation is provided.

C.A.S.E stands for Complessi Antisismici Sostenibili Ecocompatibili (Anti-seismic, Sustainable and Environment-Friendly Buildings), that is, the plan for the design and construction of new housing and neighborhoods in L'Aquila (Figure 10-1). The C.A.S.E. Housing Project was for people whose homes were destroyed or left unfit to live in by the earthquake, of type E or F, or in the red area of the Municipality of L'Aquila. The complex provided accommodation for more than 15,000 people.

Construction took about 5 months and a total of 185 buildings were delivered, grouped in 5 buildings for each of the lots (a total of 4,500 flats). This was an extraordinary demonstration of engineering management for the earthquake engineering community throughout the world. The pursuit of these objectives was based on the construction of large base isolated plates and the subsequent assembling of pre-fabricated three-story living units, made of different materials like laminated wood, pre-stressed concrete, bricks or thermal insulated metal.

They lie on seismically isolated bases (20×60m), i.e. bases supported by columns on which friction pendulum bearings are installed so that the platforms are isolated from the ground. A total of 7,368 isolators have been used. The plates were designed knowing neither the local soil properties, nor the weight and plan distribution and structure of the buildings, therefore a conservative thickness of 500mm was selected. The seismic isolation system was designed using a response spectrum with 1,000 yrs return period that corresponds to a displacement demand of about 250mm for soil type E.

Figure 10-1 C.A.S.E. house module

The friction pendulum bearings were selected in order to achieve a vibration period of 3.29 sec and an equivalent damping ratio of 20.1% (Figure 10-2). Friction pendulum bearings were selected, because at the moment of the design, the total weight to be set on top of the plates was unknown and this system is the only type of bearing that can be designed without knowing the total mass on top of the plates (Christopoulos and Filiatrault, 2006).

Figure 10-2 Friction pendulum bearing

According to engineers working at the site during construction, the entire seismic isolation system and the reinforced concrete plates were over dimensioned, which resulted in resources being diverted that could have been used for the reconstruction of the historical center of L'Aquila.

Many criticisms by expert seismic engineers have occurred regarding the reliability of these devices. Indeed, static and dynamic friction coefficients are not fixed parameters, but they change with time increasing their values. These considerations have been validated by the dynamic tests on site of 11 buildings. Two hydraulic actuators were used to move the upper plate with respect to the lower one with displacements of 150-200mm at a speed of 150-200mm/s. Results of dynamic tests are that the friction coefficient already increased a few months after the construction. Therefore, it is clear that the maximum base shear at the building level is larger than the one used to design the buildings. From these considerations, it can be concluded that there will be a high chance that when needed (approximately 200 yrs from now), there will not be seismic isolation unless a proper maintenance plan is set up. This maintenance plan will increase the total costs of the seismic isolation system; therefore, from an economical point of view, rubber bearings would have been a more reliable and economical alternative. Furthermore, the rubber bearings would have been able to take care of the vertical component of the earthquake that was quite intensive during April 6th earthquake.

During field observations in the construction site of the C.A.S.E. project, several problems with construction details of the nonstructural components were noticed such as the installation of a metal railing (Figure 10-3) perpendicular to the RC plate that would create an obstacle to its movement during the earthquake, resulting in the possible development of dangerous torsional site effects.

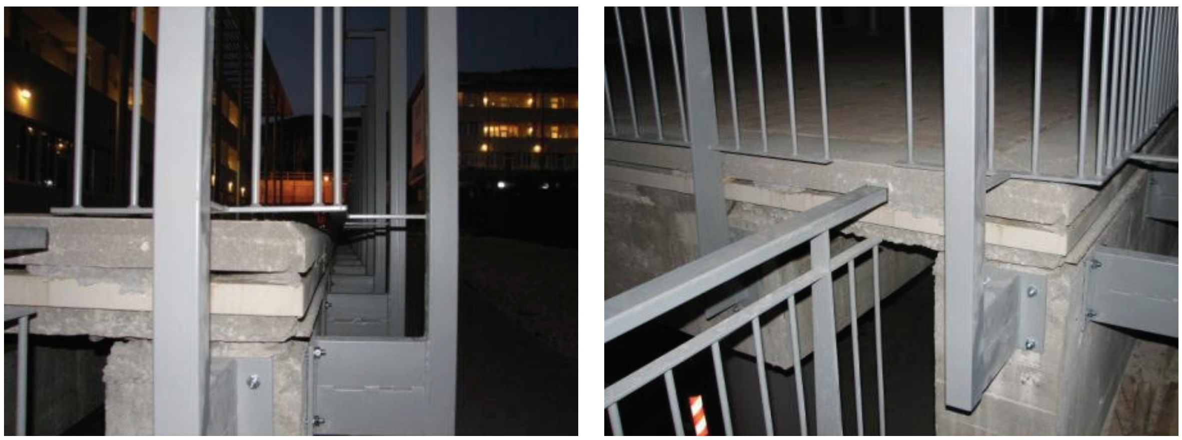


Figure 10-3 Problems in the construction details of the nonstructural components

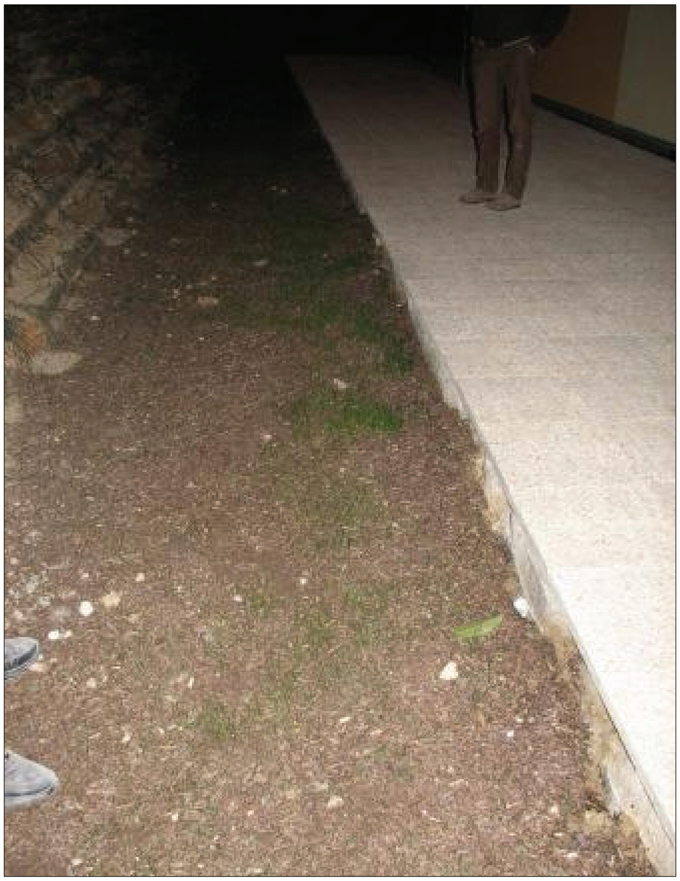


Figure 10-4 Gap filled with ground soil hindering the lateral movements of the plate

On one side of a realized plate (Figure 10-4), the gap between the plate and the ground wall was filled with soil, also hindering the lateral movement of the plate in this case. These two examples show the potential dangers of miscommunication between the building designers and the companies installing the nonstructural components.

SECTION 11

REFERENCES

Ameri, G., Augliera, P., Bindi, D., D'Alema, E., Ladina, C., Lovati, S., Luzi, L., Marzorati, S., Massa, M., Pacor, F., and Puglia, R., 2009. "Strong-motion parameters of the Mw=6.3 Abruzzo (Central Italy) earthquake," INGV Report.

Antonacci E, Beolchini G., Di Fabio F. and Gattulli V. (2001). "Retrofitting effects on the dynamic behavior of the basilica S. Maria of Collemaggio", *10 CNEM*, (CD ROM), 6-9 Ottobre, Siviglia, Spain, 2001.

Antonacci E, Beolchini G., Di Fabio F. and Gattulli V. (2001). "The dynamic behavior of the basilica S. Maria of Collemaggio", *2nd International Congress on Studies in Ancient Structures*, Istanbul, Turkey, 2001.

APAT (2006). Carta Geologica d'Italia alla scala 1:50.000 - Foglio n. 359 "L'Aquila". S.EL.CA. Firenze.

Baggio, C., Bernardini, A., Colozza, R., Colozza, L., Coppari, S., Bella, M. D., Pasquale, G. D., Dolce, M., Goretti, A., Martinelli, A., Orsini, G., Papa, F., and Zuccaro, G. (2000). "Manuale per la compilazione della scheda di livello di Rilevamento Danno, Pronto Intervento e Agibilita` per edifici ordinari nell`emergenza Post-sismica." Servizio Sismico Nazionale e Gruppo Nazionale per la Difesa dai Terremoti.

Blumetti, A.M., Filippo, M.D, Zaffiro, P., Marsan, P., and Toro, B., (2002). "Seismic hazard of the city of L'Aquila (Abruzzo — Central Italy): new data from geological, morphotectonic and gravity prospecting analysis," *Studi Geologici Camerti,* **1**:7–18.

Bruneau, M., Chang, S., Eguchi, R., G. Lee, O'Rourke, T., Reinhorn, A. M., Shinozuka, M., Tierney, K., Wallace, W., and Winterfelt, D. v. (2003). "A framework to Quantitatively Assess and Enhance the Seismic Resilience of Communities." *Earthquake Spectra*, 19(4), 733-752.

Cartapati, E. (2000). L'intervento di miglioramento antisismico delle navate della Basilica di Collemaggio a L'Aquila, in: V. De Vecchi (Hg.), Il patrimonio storico architettonico ed il terremoto. La diagnosi e gli interventi di recupero. *Atti della giornata di studi. L'Aquila 10 novembre 1988* (2000), 155-164.

Chang, S., and Shinozuka, M. (2004). "Measuring Improvements in the Disaster Resilience of Communities." *Earthquake Spectra*, 20(3), 739-755.

Christopoulos, C., and Filiatrault, A. (2006). "Principle of Passive Supplemental Damping and Seismic Isolation." IUSS Press, University of Pavia, Italy.

Ciampi, V., and Samuelli Ferretti, A. (1990). Energy dissipation in buildings using special bracing system. *Proc. of 9th European Conference on Earthquake Engineering, Mosca, **3**:9-18.*

Ciampi, V., Marioni, A., (1991). “New types of energy dissipating devices for seismic protection of bridges” *3rd World Congress on Joint Sealing and Bearing Systems for Concrete Structures*, Toronto, Canada.

Cimellaro, G. P., Reinhorn, A. M., and Bruneau, M. (2005). "Seismic Resilience of a Health care facility." *Proceedings of the 2005 ANCER Annual Meeting*, Session III, paper N3, November 10-13, Jeju, Korea.

Cimellaro, G. P., Reinhorn, A. M., and Bruneau, M. (2006). "Quantification of seismic resilience." *Proceedings of the 8th National Conference of Earthquake Engineering*, paper 1094, April 18-22, 2006, San Francisco, California.

Cimellaro, G. P., Fumo, C., Reinhorn, A. M., and Bruneau, M. (2009). "Quantification of Seismic Resilience of Health care facilities " *MCEER Technical Report-MCEER-09-0009*, Multidisciplinary Center for Earthquake Engineering Research, Buffalo, NY.

Cimellaro, G. P., Reinhorn, A. M., and De_Stefano, A. "Considerations regarding the retrofit intervention of Santa Maria di Collemaggio basilica in L’Aquila following 2009 italian earthquake." *Proceedings of Structures Congress 2010 (SEI 2010)*, May 12-14, 2010, Orlando, Florida.

Cimellaro, G. P., Soong, T. T., and Reinhorn, A. M. (2009). "Integrated Design of Controlled Linear Structural Systems." *Journal of Structural Engineering, ASCE*, 135(7), 853-862.

Cimellaro, G. P., Lavan, O., and Reinhorn, A. M. (2009). "Design of Passive systems for controlled inelastic structures." *Earthquake Engineering & Structural Dynamics*, 38(6), 783-804.

Cornell, A., and Krawinkler, H. (2000). "Progress and challenges in seismic performance assessment " *Peer News*, April 2000, 3(2).

De Luca, G., Marcucci, S., Milana, G., and Sano, T., 2005. “Evidence of Low-Frequency Amplification in the City of L’Aquila, Central Italy, through a Multidisciplinary Approach Including Strong- and Weak- Motion Data, Ambient Noise, and Numerical Modeling,” *Bulletin of the Seismological Society of America,* **95**(4):1469-1481.

Devoti R., Riguzzi F., Cuffaro M., and Doglioni C., 2008. “New GPS constraints on the kinematics of the Apennines subduction,” *Earth and Planetary Science Letters,* **273**:117-130.

EEFIT, 2009. Rossetto, T., Peiris, N., Alarcon, J., So, E., Sargeant, S., Sword-Daniels, V., Libberton, C., Verrucci, E., Del Re, D., and Free, M.,“The L’Aquila, Italy Earthquake of 6 April 2009. A preliminary file report by EEFIT.”

EERI, 2009. “Learning from Earthquakes,” EERI Special Earthquake Report – June 2009.

Encyclopedia Britannica, 2009. "Alpine orogeny." Encyclopedia Britannica Online. 15 Aug. 2009 http://www.britannica.com/EBchecked/topic/17310/Alpine-orogeny.

Filiatrault A, Cherry S. Seismic design spectra for friction damped structures. *Journal of Structural Engineering (ASCE)* 1990; 116:1334–1355.

GEER, 2009. Stewart, J.P., “Preliminary Report on the Seismological and Geotechnical Aspects of the April 6 2009 L'Aquila Earthquake in Central Italy,” Geotechnical Earthquake Engineering Reconnaissance.

Ghasemi, H., Cooper, J., Imbsen, R., Piskin, H., Inal, F., and Tiras, A. (2000). "The November 1999 Duzce Earthquake: Post-Earthquake Investigation of the Structures on the TEM", Publication No, FHWA-RD-00-146. http://www.tfhrc.gov/structur/00-146.pdf

Gluck, N., Reinhorn, A.M., Gluck, J., and Levy, R. (1996), "*ASCE/Journal of Structural Engineering*, 122(12), 1394-1399.

Gruppo di lavoro CPTI, 2004. Catalogo Parametrico dei Terremoti Italiani, versione 2004 (CPTI04), INGV, Bologna <http://emidius.mi.ingv.it/CPTI04/>.

Gvirtzman, Z., and Nur, A., 2001. "Residual topography, lithospheric structure and sunken slabs in the central Mediterranean," *Earth and Planetary Science Letters,* **187**:163-174.

INGV, 2009. *Instituto Nationale di Geofisica e Vulcanologia* online report http://portale.ingv.it/primo-piano-1/news-archive/2009-news/l-aquila-seismic-sequence-april-2009/.

Joyner W., and D. Boore (1981): Peak horizontal acceleration and velocity from strong-motion records including records from the 1979 Imperial Valley, California, earthquake. Bull. Seism. Soc. Am., vol. 71, no. 3, pp. 2011-2038.

Kamagata, K., Miyajima, K., and Seto, K. (1996). "Optimal design of damping devices for vibration control of parallel structures." *Proc., 3rd Int. Conf. on Motion and Vibration Control (MOVIC)*, Vol. 2, Chiba, Japan, 334–339.

Maccallini, R., *Terremoti in Aquila*, Rivista Abruzzese, XXX, 9, Teramo, 1915.

Marioni, A. (1997). "Development of a New Type of Hysteretic Damper for the Seismic Protection of Bridges*", Proc. Fourth World Congress on Joint Sealants and Bearing Systems for Concrete Structures*, SP-164, Vol. 2, American Concrete Institute, pp. 955- 976.

Marioni, A. (2000). "Behaviour of Large Base Isolated Prestressed Concrete Bridges During the Recent Exceptional Earthquakes in Turkey". http://192.107.65.2/GLIS/HTML/gn/turchi/g5turchi.htm.

Notizie riportate sulle "Cronache Aquilane" di Buzio di Ranallo nei manoscritti di Anton Ludovico Antinori *Catalogo dei Terremoti Italiani 1000-1980*, n. 992.

Papanikolaou, I.D., Roberts, G.P., and Michetti, A.M., 2005. "Fault scarps and deformation rates in Lazio-Abruzzo, Central Italy: Comparison between geological fault slip-rate and GPS data," *Tectonophysics,* **408**:147-176.

Paolacci, F., Ciampi, V. & De Angelis, M. (1998). "Optimal design of viscoelastic bracing systems for earthquake protection of buildings." *Proc. of XI ECEE*, Paris, September 1998.

Pekcan, G., Mander, J. B., and Chen, S. S. (1999). "Fundamental considerations for the design of non-linear viscous dampers." *Earthquake Engineering & Structural Dynamics*, 28, 1405-1425.

QUEST, 2009. "Rapporto sugli effetti del terremoto aquilano del 6 aprile 2009," INGV Report <http://www.mi.ingv.it/eq/090406/quest.html>.

Renschler, C., Frazier, A., Arendt, L., Cimellaro, G. P., Reinhorn, A. M., and Bruneau, M. "Developing the "PEOPLES" resilience framework for defining and measuring disaster resilience at the community scale." *Proceedings of the 9th US National and 10th Canadian*

Conference on Earthquake Engineering (9USN/10CCEE), Toronto, Canada, July 25-29, 2010.

Renschler, C., Frazier, A., Arendt, L., Cimellaro, G. P., Reinhorn, A. M., and Bruneau, M. (2010). "Framework for Defining and Measuring Resilience at the Community Scale: The PEOPLES Resilience Framework." *MCEER Technical Report –MCEER-10-006, pp. 91*, University at Buffalo (SUNY), The State University of New York, Buffalo, New York.

Roberts, G.P., Michetti, A.M., Cowie, P., Morewood, N.C., and Papanikolaou, I., 2002. "Fault slip-rate variations during crustal-scale strain localisation, central Italy," *Geophysical Research Letters*, **29**:9.1-9.4.

Rosenbaum, G., Listera, G.S. and C., Duboza. 2001. "Relative motions of Africa, Iberia and Europe during Alpine orogeny," *Tectonophysics,* **359**:117-129.

Rovida, A., Castelli, V., Camassi, R., and Stucchi, M., 2009. "Terremoti storici nell'area colpita dagli eventi sismici dell'aprile 2009," INGV Report <http://www.mi.ingv.it/eq/090406/storia.html>.

Roussis, P. C., Constantinou M. C., Erdik, M., Duruka, E. and Dicleli, M. 2003, "Assessment of Performances of Seismic Isolation System of Bolu Viaduct", *ASCE Journal of Bridge Engineering*, Vol. 8, No. 4, pp. 182-190.

Salvi, S., Stramondo, S., Hunstad, I., Chini, M., Atzori, S., Moro, M., Tolomei, C., Bignami, C., Trasatti, E., Antonioli, A., Bruniri, C.A., Kyriakopoulos, C., and Saroli, M., 2009. "Measurement and modeling of co-seismic deformation during the L'Aquila earthquake. Preliminary results", INGV Report http://portale.ingv.it/primo-piano-1/news-archive/2009-news/april-6-earthquake/Report_DInSAR_INGV_Aquila_Eq.pdf.

Scalabrino, F., 2009. "HISTORY: The rich history of L'Aquila," available online at www.wantedinrome.com/articles.

Stucchi et al. 2007. "DBMI04, il database delle osservazioni macrosismiche dei terremoti italiani utilizzate per la compilazione del catalogo parametrico CPTI04," Quaderni di Geofisica, Vol. 49, pp.38 <http://emidius.mi.ingv.it/DBMI04/>.

Symans, M. D., and Constantiou, M. C. (1998). "Passive fluid viscous damping systems for seismic energy dissipation." *ISET J. Earthquake Technology, Paper No. 382*, 35(4), 185-206.

Trombetti T, Silvestri S (2004): Added viscous dampers in shear-type structures: the effectiveness of mass proportional damping. *Journal of Earthquake Engineering*, **8** (2), 275–313.

Vittori, G., Stato dell'Aquila degli Abruzzi nei grandi periodi sismici del 1315, 1349 e 1461-62, in Bollettino della Società Abruzzese di Storia patria, VIII, L'Aternina, Aquila, 1896.

Working Group ITACA, 2008. Data Base of the Italian strong motion data: http://itaca.mi.ingv.it.

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