9. SEISMIC BEHAVIOUR, VULNERABILITY AND PROBABILITY OF COLLAPSE OF MASONRY BUILDINGS

9.1 Introduction

(D. Liberatore and A. Bernardini)

The first phase of the investigation reported herein is devoted to studying the materials and the elements of the masonry buildings of Catania. Two sample buildings have been chosen, representing as many building types, to be studied in detail. The first building dates back to the nineteenth century, has masonry walls of lavic stones with irregular fabric and horizontal structure consisting of vaults without any tie-rods. The second building was erected in the post-war period, has walls both in lavic stones and in brick masonry, and horizontal structure consisting of tile-lintel floors connected to the masonry through R/C ring beams.

Experimental and numerical investigations have been performed on both these buildings. Different Research Units (R.U.) took part in the numerical investigations, using their own models, developed in recent years. The outcomes of these analyses were used for the calibration of the vulnerability evaluations at medium and large scale.

The large scale evaluations have been performed through the information collected on available databases of the buildings in Catania: the LSU Database (see sub-sect. 5.4) of the overall stock of nearly 26000 buildings of the town and the CONARI Database of nearly 6000 masonry buildings in the historical center. Moreover, 131 additional buildings have been inspected in detail, and their seismic strength evaluated through a method based on the combination of simple mechanical models and experimental knowledge.

Classification into three classes of vulnerability of the identified masonry typologies and probabilistic damage matrices of past Italian earthquakes have been used to forecast damage scenarios for the scenario earthquakes.

9.2 Seismic behaviour of typical masonry buildings: materials and elements

(D. Liberatore, G. Beolchini, L. Binda, L. Gambarotta and G. Magenes)

9.2.1 *Materials*

To erect the masonry buildings of Catania, large use was made of stone quarried from Etnean lavaflows, both historical and recent (Lo Giudice and Novelli, 1997).

The surface layer of the local lavaflow consists of prevalently vitreous scoriae (“sciara”). These scoriae, after being crushed, were used as mortar aggregate (“azolo”). The body of the lavaflow is made of a compact rock with high hardness (6°-7° degree Mohs) and good workability.

Lavic stone was used in different shapes and sizes: non-hewn stones, stones roughly hewn on one face, square-cut stones (“intostoni”, one hand palm wide, 15÷18 cm high, 2÷3 hand palms long, one hand palm being 26 cm; “cannarozzoni”, with the same width and height but longer). The compressive strength of lavic stone is generally greater than 100 MPa. Failure is typically brittle (Cuomo and Badalà, 1998). Besides lavic stone, clay bricks were used since the nineteenth century.

Materials of volcanic origin were used for the aggregate as well. "Azolo", already quoted, is a porous material that, after being crushed, becomes a sharp, grey to black, sand. Another type of aggregate is red “ghiara”. It is the paleosoil under the lavaflow that undergoes a metamorphism process because of high temperature (800÷900°C). Unlike azolo, red ghiara generally has pozzolanic properties and, mixed with lime, becomes a hydraulic mortar with high mechanical characteristics. Mortars with red ghiara are present in most of the historical centre of Catania.

Lime and plaster are the main binders. Caustic lime was obtained by kilning calcareous stones from the Siracusa area. Plaster, together with pumice-stone or bricks, was used to build “real” vaults.

Different types of mortar were used, depending on the purpose and the historical period (Battiato, 1988, Sciuto Patti, 1896). Mortar of lime and azolo is a mixture of slaked lime and azolo, with size between 2 and 4 mm. The volumetric ratio between lime and azolo ranges from 1/3 to 1/2. The quantity of water is that strictly necessary for workability. Mortars of lime and azolo were used up to 1860. Mortar of lime and red ghiara is a mixture of lime and ghiara in the ratio ranging from 1/4 to 1/3. It is a hydraulic mortar with pozzolanic properties and high mechanical properties. Recent tests did not confirm the pozzolanic properties of ghiara; however, this could be ascribed to inhomogeneities of the material due to non uniform thermal fields of lavaflows. Mortar of lime and red ghiara, introduced in 1860, rapidly replaced mortar of lime and azolo, thanks to its smaller lime demand, which was the most expensive material. In the post-war period, this type of mortar was replaced by concrete mortars, because of the difficulty and cost to quarry red ghiara. Plaster mortar is a mixture of water and plaster in the ratio 1/1.

9.2.2 *Elements*

Masonry can be classified according to the material, shape and size of the blocks (Randazzo, 1988). The most frequent types are the following.

- V1) Masonry with irregular fabric (Fig. 9.1). It is made by lavic stones of irregular shape and medium to small size. Its weight ranges from 16000 to 21000 N/m^3.
- V2) Masonry with regular fabric (Figs. 9.2-3). It is the most frequent type and is made by hewn lavic stones of medium size, arranged in more or less regular horizontal courses. The inner and outer facings are connected by transverse “cannarozzoni”. The weight of this type ranges from 17000 to 22000 N/m^3.

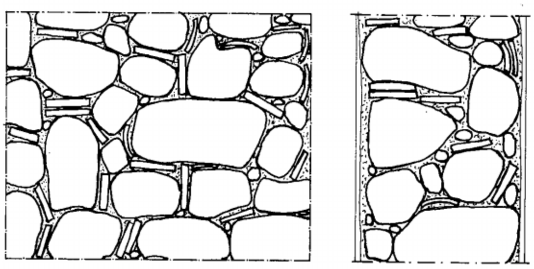


Figure 9.1 - Masonry with irregular fabric (Randazzo, 1988).

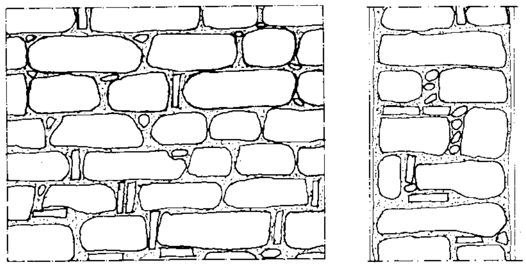


Figure 9.2 - Masonry with regular fabric (Randazzo, 1988).

V3) Masonry of lavic stone blocks. It is made by square-cut stones arranged in horizontal courses. The inner and outer facings are effectively connected by "cannarozzoni". The weight of this type ranges from 19000 to 25000 N/m^3. Masonry of lavic "intostoni" and brick courses, forming walls 26 cm thick, can be included in this type (Fig. 9.4).

V4) Block masonry. It is typical of suburban neighbourhoods and is made by blocks of calcareous stone or concrete, with partial or complete ring beams.

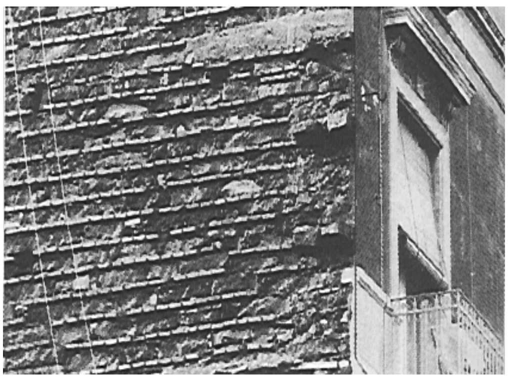


Figure 9.3 - Masonry with regular fabric, with brick courses and jutting blocks for the connection of the facades (Randazzo, 1998)

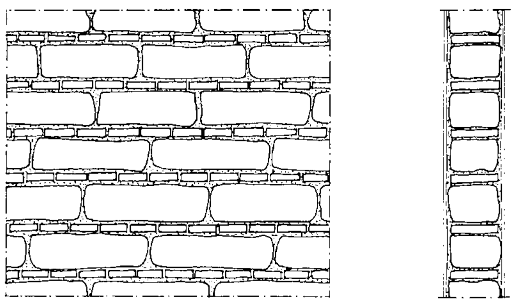


Figure 9.4 - Masonry of lavic "intostoni" and brick courses (Randazzo, 1988).

The horizontal structure can be classified in the following types:

H1) Floors with wooden beams. They were used up to the first half of the nineteenth century.

H2) Vaults. The vaults in pumice-stone and plaster (Fig. 9.5) represents the most frequent type in the historical centre (Arezzo, 1994). Their span length ranges from 4 to 6 m. The key thickness is 8÷12 cm. Sometimes, the fill is replaced by counter-vaults (Fig. 9.6).

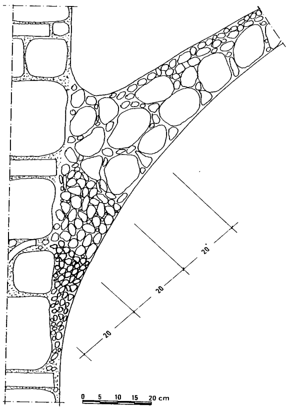


Figure 9.5 - Vault in pumice-stone and plaster (Randazzo, 1988).

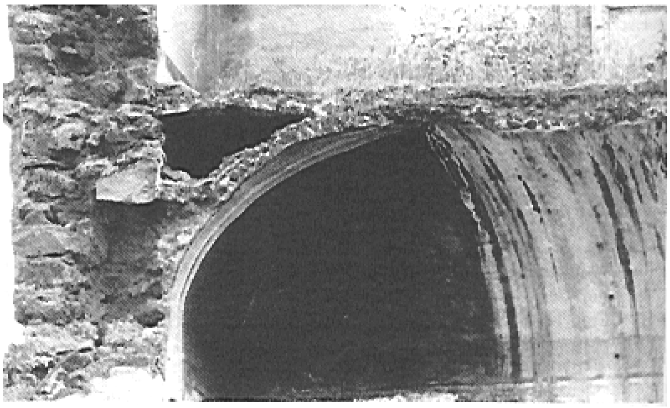


Figure 9.6 - Vault and counter-vault in pumice-stone and plaster (Randazzo, 1988).

H3) Vaults with tie-rods. Most of tie-rods were inserted after the damaging earthquakes of 1818, 1848 and 1908.
H4) Floors with steel beams and vaults. The spacing within the steel beams ranges from 50 to 80 cm. The vaults are made of pumice-stone and plaster, and have flat upper surface (Fig. 9.7).
H5) Tile-lintel floors. The first examples date back to the thirties and the forties.

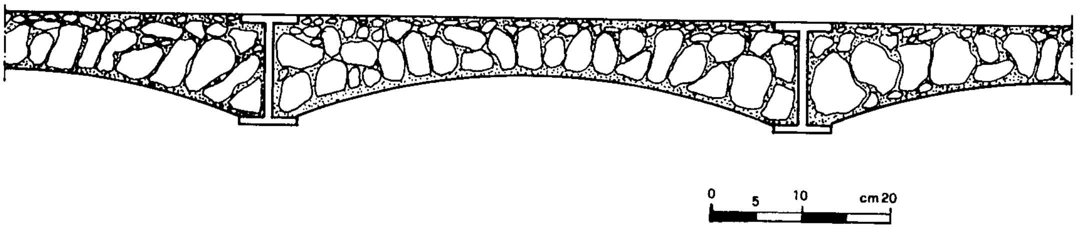


Figure 9.7 - Floor with steel beams and vaults in pumice-stone and plaster (Randazzo, 1988).

Two types of roof are present: saddle roofs and flat roofs. Saddle roofs are in chestnut-wood (Fig. 9.8). Under the roof, there are “false” vaults in canes and plaster with wooden frame, without any structural function (Fig. 9.9). Flat roofs are in wood or in steel beams.

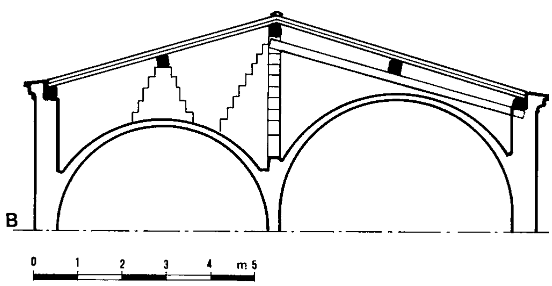


Figure 9.8 - Typical section of a wooden saddle roof (Randazzo, 1988).

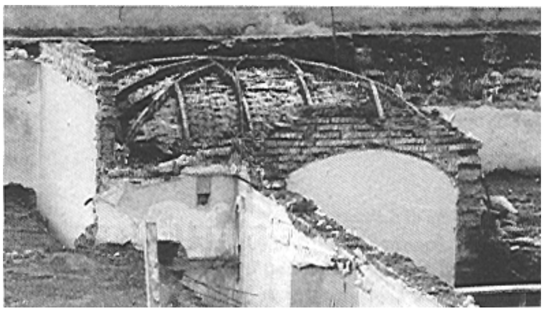


Figure 9.9 - Upper surface of a “false” vault (Randazzo, 1988).

9.3 Morphological and mechanical characteristics of masonries and material properties

(L. Binda, G. Baronio, G. Mirabella Roberti and D. Penazzi)

9.3.1 *Survey on texture and cross-sections of some masonry walls*

Eight buildings were the object of the survey, including the two chosen as case-study for in-depth multidisciplinary research within the Catania Project, thanks to the collaboration between GNDT and the local Authorities.

Since it was impossible to inspect the inside of the walls of these two buildings due to the fact that rendering could not be destroyed, only the texture and inside of the walls of the other six buildings were surveyed.

9.3.1.1 *Texture of the masonry walls*

A complete survey was possible on six sites thanks to the fact that some ruins were available or works were carried out for repair and restoration of the same buildings. Sixteen cross-sections (Table 9.1) were surveyed and studied, first of all according to the bonding technique and to the stone shape and dimensions (Binda, 1999, Binda *et al.*, 1999b).

Table 9.1: Surveyed walls.

Name	Location	Analysed walls
Ca3	Via Vittorio Emanuele	Ca3p1, Ca3p2, Ca3p3
Ca4	Via Consolazione	Ca4p1, Ca4p2
Ca5	Via Consolazione	Ca5p1, Ca5p2
Ca6	Church S. Nicolò	Ca6p1
Ca7	Province Building	Ca7p1, Ca7p2, Ca7p3, Ca7p4, Ca7p5, Ca7p6
Ca8	Via Dusmat	Ca8p1, Ca8p2

Most frequently the stones are simply boasted, but often are more or less regular ashlars. A large use has been found of clay elements, such as bricks in the so-called “intosto” masonries (see figure 9.11a) or roof tiles used as wedges in the external masonry leaves. The mortar joints are irregular or quasi-regular (sub-horizontal) (Fig. 9.10a). Figure 9.10b shows the overall per cent distribution of mortar, stone and voids over the surveyed sections.

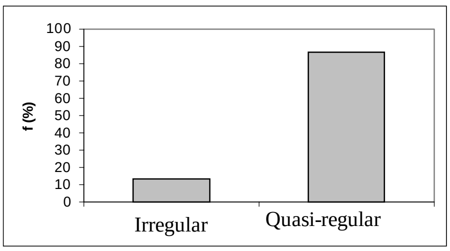


Figure 9.10a - Distribution of horizontal courses.

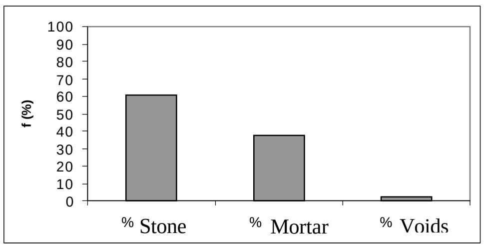


Figure 9.10b - Material percentage.

9.3.1.2 *Cross-sections of the walls*

The surveyed cross-sections were sixteen, but one was discarded due to the non-typical dimension, being 1300 mm thick. In fact the most common wall type seems to be a double-leaf masonry with a fairly good connection between the two leaves and average thickness of 65 cm (Table 9.2).

Five cases belong to the so-called "intosto" typology with an average thickness of 35 cm. The "intosto" masonries were used typically during the last century in Sicily as partitioning walls; the horizontal courses were made alternatively with bricks and volcanic basalt units defined locally as "cannarozzone da intosta" (see also sub-sect. 9.2.2). The ashlars were tooled and fairly regular with a height of 25÷30 cm

Table 9.2: Geometrical parameters of the surveyed sections.

Name	**No. of leaves**			**Elements**		
	1	2	3	Stone %	Mortar %	Voids %
Ca3s1			x	62.70	29.44	7.86
Ca3s2			x	64.04	35.58	0.38
Ca3s3	x			58.66	39.23	2.11
Ca4s1		x		58.66	39.23	2.11
Ca4s2	x			67.58	32.23	0.19
Ca5s1	x			69.77	27.75	2.48
Ca5s2	x			61.24	37.91	0.85
Ca7s1		x		39.82	58.19	1.99
Ca7s2	x			n.r.	n.r.	n.r.
Ca7s3		x		54.80	43.26	1.76
Ca7s4			x	65.52	33.19	1.29
Ca7s5		x		60.22	33.86	5.92
Ca7s6		x		65.15	30.22	0.63
Ca8s1		x		41.46	56.30	2.24
Ca8s2		x		74.81	24.05	1.14

Three walls appeared to be three-leaf walls but in two of them one external leaf had been built in bricks and added after the construction of the wall. The data concerning the surveyed cross-sections are given in table 9.2. From the same table, it can be seen that the void percentage is usually very low (<3%). This information can be very useful when deciding on the repair and strengthening technique to be applied (Binda *et al.*, 1997). Figure 9.11 shows the most representative cross-sections among the surveyed ones.

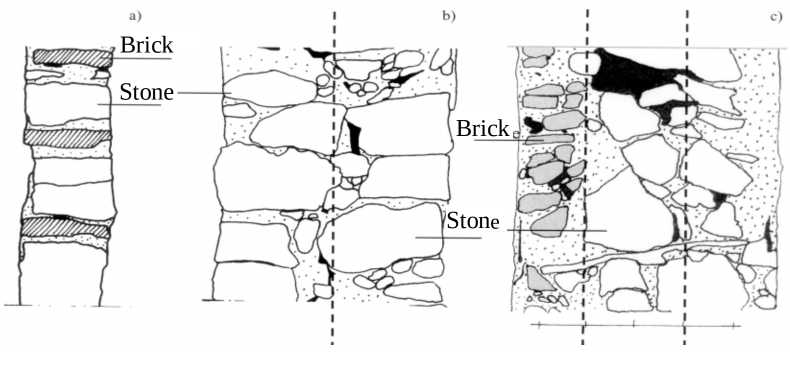


Figure 9.11 - Some types of sections: a) "intosto", b) two-leaves partially connected, c) three-leaves.

9.3.1.3 *Laboratory analyses and tests on materials*

From the walls of the buildings Ca3 and Ca4 some stones and mortars were sampled and characterised through laboratory tests.

9.3.1.3.1 Analysis of mortars

From the building Ca3 two mortars were sampled and identified as Ca3p3.2m and Ca3p5.3m.The chemical analyses allowed to detect the calcareous nature of the binder and a very good level of carbonation. The aggregate is of volcanic origin, siliceous with rare carbonaceous pebbles. The petrographic-mineralogical analysis did not detect any pozzolanic reaction between binder and aggregate as usually expected when dealing with volcanic materials. The bulk density was respectively 1561 and 1623 kg/m^3, quite low if compared to other traditional mortars prepared without volcanic aggregates.

9.3.1.3.2 Physical and mechanical tests on stones

From the ruins of the building situated in via della Consolazione and referred as Ca4 two stone ashlars and one brick were sampled and used for laboratory tests. They were sampled as follows: a) an irregular stone named Ca4.2 from the wall section Ca4s1, a two-leaf well connected wall 68 cm thick, b) a brick called Ca4.1 and an ashlar called Ca4.3 from the single-leaf section, 20 cm thick called Ca4s2.

The thin section analysis under petrographic microscope has shown the following characters:

- Ca4.2: vesicular volcanic rock with rare microphenocrysts of plagioclase in very fine grained groundmass;
- Ca4.3: basaltic volcanic rock with porphyritic structure, containing plagioclase microphenocrysts in glassy groundmass and phenocrysts, mainly of geminate plagioclase, subordinately of pyroxene and olivine.

Eight cylinders (5 from Ca4.2 and 3 from Ca4.3) of 5 cm diameter and 10 cm height were cored from the stones. The cylinders were submitted first to physical tests and later to mechanical compressive and splitting tests; these last were only possible for Ca4.2. From the brick, cubes of 35 mm side were sampled for splitting and compressive tests. Table 9.3 gives the results of the tests.

It is clear that the mechanical and physical characteristics of the volcanic stones can be very different.

Table 9.3: Mechanical and physical characteristics of stones.

	Compressive Strength N/mm^2	Elastic Modulus N/mm^2	Tensile Strength N/mm^2	Capillarity rise coefficient $g/cm^2s^{0.5}$
Ca4.1	4.96	820	0.81	1.788
Ca4.2	34.9	7765	3.88	0.0926
Ca4.3	119.7	13797	/	0.035

9.3.2 *Investigation on materials and masonries of the two reference buildings*

For each of the two buildings examined, a description form was prepared as described in (Binda *et al.*, 1999a). Section surveys and sampling of stones were impossible due to the fact that the buildings are currently in use. Flat-jack tests were allowed, as well as sampling of the mortar from the joints cut for the tests.

9.3.2.1 *Ca1 Building: mortar analyses*

A sample taken from the Ca1 building was analysed: the mortar appeared to be very consistent and of reddish colour. Visual inspection shows red pebbles, calcite pebbles and fragment of basaltic stone (Fig. 9.12). The specimen was named Ca1 p2m and was sampled from the CTJ1D test location. The chemical analysis reported in table 9.4 shows the calcareous nature of the binder, a high level of carbonation and the siliceous nature of the aggregate. The soluble silica content is very high compared to a traditional mortar based on hydrated lime.

The petrographic mineralogical analyses confirmed the results of the chemical one. The presence of a high percentage of soluble silica can be due to a pozzolanic reaction between the binder and the volcanic aggregate particularly the high porous one; in fact reaction borders can be seen around some volcanic aggregates (Fig. 9.13) (Baronio et *al.*, 1994).

9.3.2.2 *Flat-jack tests*

Three double flat-jack tests were carried out on the masonries of the Ca1 and Ca2 buildings: respectively two at Ca1 (CTJ1D, CTJ2D) and one at Ca2 (CTJ3D).

The stress-strain curves obtained are reported in figure 9.14a,b,c. The following maximum stress values were reached, which can be considered around the 70% of the real peak stresses, during the tests: 2.27, 2.24, 2.59 N/mm^2 respectively for CTJ1D, CTJ2D and CTJ3D. These values and the stress-strain behaviour detected indicate by experience of the authors a good masonry (Binda *et al.*, 1999c). The weakest behaviour was definetely shown by CTJ2D which was carried out on a wall added after the construction of the building. Table 9.5 also gives other calculated mechanical parameters.

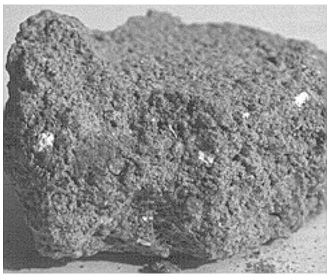


Figure 9.12 - Mortar sample from Ca1 building.

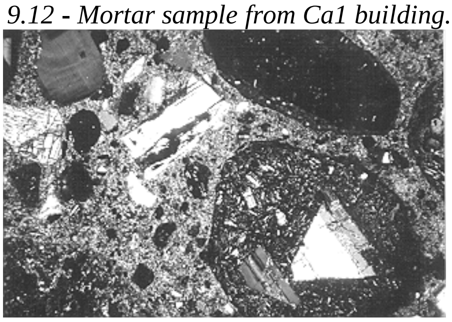


Figure 9.13 - Thin section of Ca1 p2m with reaction layers.

	%
(SiO_2)	46.59
Al_2O_3	18.62
(Fe_2O_3)	6.48
CaO	13.38
MgO	2.74
NaO	2.12
KO	1.94
(SO_3)	0.36
Ignition Loss	7.61
(CO_2)	7.30
Chloride	0.034

Table 9.4: Chemical analysis of the sample Ca1 p2m

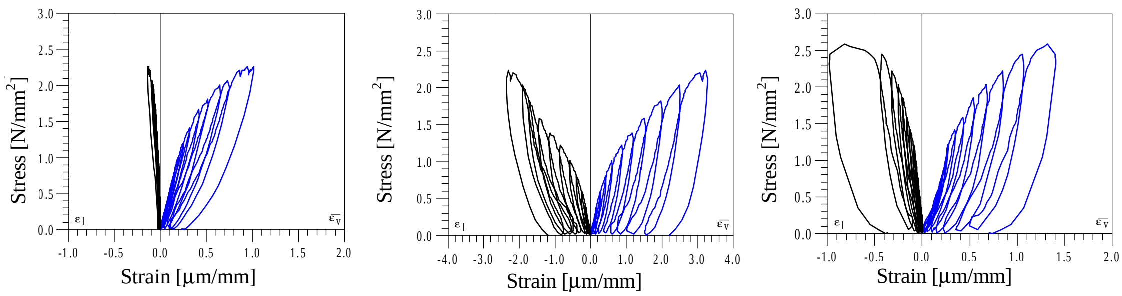


Figure 9.14a,b,c - Stress-strain curves obtained from CTJ1D, CTJ2D, CTJ3d, respectively.

Table 9.5: Flat-jack results.

Name of the Test	max σ applied [N/mm²]	E sec. [N/mm²]		Δεl/Δεv	
		Load interval [N/mm²]		Load interval [N/mm²]	
		0.4÷1	1.2÷1.8	0.4÷1	1.2÷1.8
CTJ1D via Verdi	2.27	5200	2300	0.15	0.13
CTJ2D via Verdi	2.24	1400	550	(1.01)	(0.81)
CTJ3D via Martoglio	2.59	4700	2400	0.38	0.43

9.4 Seismic behaviour of typical masonry buildings: numerical calculation of the lateral strength

(D. Liberatore, G. Beolchini, C. Braggio, A. Brencich, S. Lagomarsino, L. Gambarotta, G. Magenes and G. Spera)

On the basis of the study of masonry types and building types, two sample buildings have been chosen for the detailed numerical analyses.

The first building, hereafter referred to as building “A”, dates back to the nineteenth century and is representative most buildings in the historical centre. The masonry is in lavic stone with irregular fabric. The horizontal structure consists of vaults. The second building, referred to as building “B”, was erected in the fifties and represents part of the masonry buildings erected in the post-war period. The peripheral walls have irregular fabric and some internal walls are in solid brickwork. The horizontal structure is made by tile-lintel floors.

The mechanical properties adopted in the analyses for the lavic stone masonry and, where present, the brick masonry, are shown in table 9.6.

Table 9.6: Mechanical characteristics of masonry adopted in the analyses.

	Lavic stone masonry	Brick masonry
Modulus of elasticity E	1500 MPa	1600 MPa
Shear modulus G	150 MPa	300 MPa
Specific weight γ	19 kN/m^3	17 kN/m^3
Compressive strength f_u	2.4 MPa	6.0 MPa
Characteristic shear strength τ_k	0.13 MPa	0.16 MPa
Mortar-block friction coefficient μ	0.5	0.5
Cohesion c	0.2 MPa	0.15 MPa
Block tensile strength f_{bt}	2 MPa	1 MPa

9.4.1 *Building “A”*

9.4.1.1 *General description*

The building is a part of a more complex block which develops around an internal courtyard. The plan has a C-shape, and the number of storeys above the ground is three (Fig. 9.15). The building is representative of rather valuable constructions of the Catania historical centre, and is made of two main bodies, built in different periods. The investigations were focused on the more recent wing, built around the second half of the nineteenth century, which is separated from the older wing by a construction joint. Its overall structural layout is very similar to that of the older wing.

The ground floor, mostly occupied by shops, has a wide entrance hall through which the stairs and the internal courtyard are accessed. The original plan layout was apparently kept, with the exception of the corner room and an adjacent room, where some internal walls had been removed and replaced by round arches. The upper storeys, used as residences, have two apartments per storey, with a rather regular layout. The building had been subjected to renovations in which intermediate floors at the first and second storey were built.

9.4.1.2 *Structural system*

The load-bearing walls are made of irregular stonework. The internal walls are made of “cannarozzoni” or “intostoni” of square-cut lavic stone (see sub-sect. 9.2.2). The mortar is made of lime and red “ghiara”.

The ceilings are made of “real” vaults in pumice-stone and plaster. The thickness at the keystone is 10 cm. The filling above the vaults is replaced by counter-vaults, built with the same technique as the main vaults. Several types of vaults are present: cross vaults at the ground floor, barrel vaults above the entrance hall, cloister vaults at the second and third floor. The floors in correspondence of service rooms and attics are made of steel beams at 70 cm spacing with small vaults in pumice-stone and plaster.

The first storey presents several intermediate floors at a height of 2.40 m above the main floor, made of steel beams with hollow flat blocks, covered by a concrete slab reinforced with welded wire mesh. The steel beams are embedded in the load-bearing walls for a length of about 15 cm.

The construction of intermediate floors made it necessary to modify the existing openings and to create new ones. An intermediate floor at the second storey was built in one room only, at the boundary with the adjacent building.

The stairwell has a heavy steel structure with marble facing. The service room at the second storey, adjacent to a boundary wall, has a smaller room (built subsequently to the construction of the main building), which juts out above the internal courtyard, and is carried by two massive R/C cantilever beams.

The roofing is made of bent clay tiles, carried by a wooden structure. Under the roof, “false” vaults in canes and plaster with wooden frame are present.

The building was affected by the 1990 earthquake, so that at present many vaults are cracked.

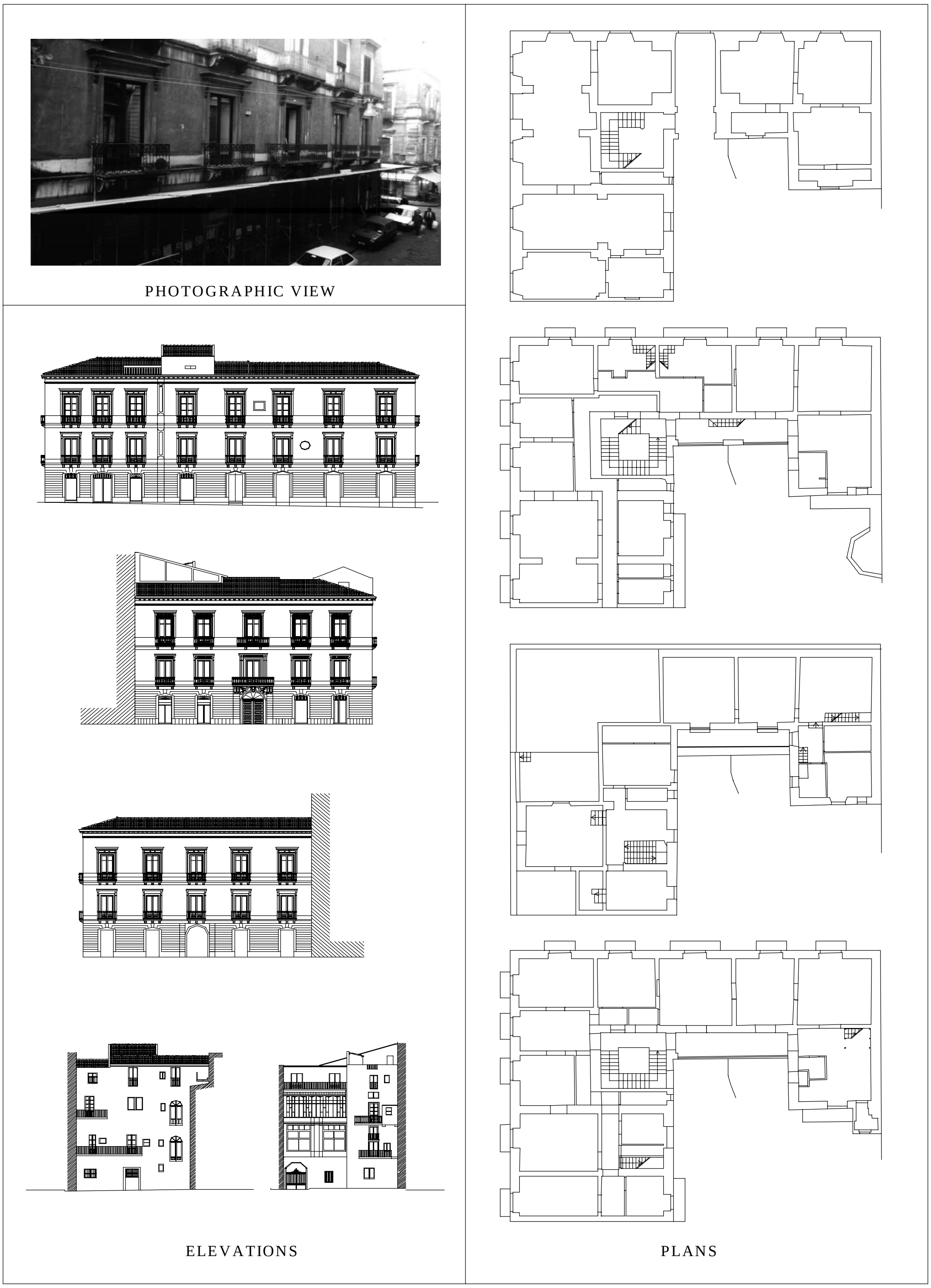


Figure 9.15 **-** *Photographic view, plans and elevations of building "A".*

9.4.1.3 *Numerical analyses*

Building “A” was analysed with different methods. Dynamic analyses wereperformed to study the out-of-plane response of walls, while static analyses were made to study the in-plane response.

Since the building is completely without tie-rods and has thrusting vaults even at the upper storeys, the failure mode which appears most probable is overturning of the peripheral walls (Liberatore and Spera, 1999). The out-of-plane response was studied by the R.U. University of Basilicata, to determine the possibility of partial or global collapse under the scenario earthquake, combined with gravity loads and with the thrust of the vaults. The dynamic response of two masonry piers is calculated, with the aim to assess the vulnerability to overturning. The piers are representative for geometry and loading. They are modelled as rigid blocks free to rock around the lower outer vertex (Fig. 9.16). The analyses have been repeated at each storey to check the possibility of overturning around centres of rotation at the height of the different storeys. The thrust of the vaults has been conservatively calculated assuming the maximum possible inclination for the line of thrust.

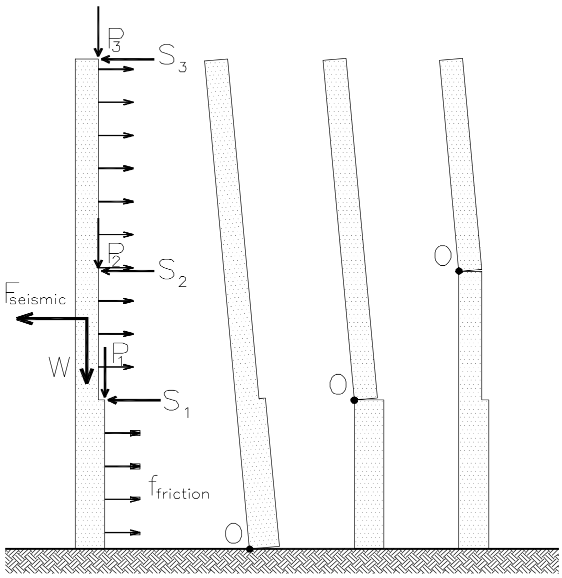


Figure 9.16 - Masonry piers and overturning modes.

In the presence of gravity loads only, the out-of-plane collapse of peripheral walls is prevented, despite the thrust of the vaults, by the orthogonal walls which intersect the peripheral walls. The orthogonal walls are made of “intostoni”. The connection assumed in the analyses is that resulting from the insertion for half length of one “intostone” every five in the peripheral wall. The thickness of the wall orthogonal to pier 2, always greater than 50 cm, allows to consider two adjacent “intostoni”. On the contrary, the thickness of the wall orthogonal to pier 2, ranging from 25 to 30 cm, leads to consider a single “intostone”. The connection between the peripheral wall and the orthogonal wall has been modelled through forces corresponding to the courses of "intostoni" inserted into the peripheral wall. These forces are calculated neglecting cohesion and adopting a friction coefficient of 0.5.

If the seismic action is also considered, the orthogonal walls have been assumed to prevent the rotation around the lower inner vertex and to damp out the kinetic energy following the impact of the peripheral wall against the orthogonal wall itself.

An artificial accelerogram with PGA = 0.48 *g* has been used in the analyses. It has been generated for the foundation soil of the building, consisting of lava. In order to assess the influence of intensity, the same accelerogram is used scaling the PGA to 0.40 *g* and 0.35 *g*.

Overturning of pier 1 as a whole occurs with PGA = 0.48 *g*, even considering the connection with the orthogonal wall. Eliminating the thrust at the two uppermost storeys prevents overturning, even though a residual rotation is present at the end of the seismic action. Overturning occurs even reducing the PGA to 0.40 *g* or to 0.35 *g*. In both these cases, however, it is sufficient to eliminate the thrust only at the uppermost storey to prevent overturning. A residual rotation is present in any case.

Overturning of the two uppermost storeys of pier 2 occurs under PGA = 0.48 *g*. Eliminating the thrust at the uppermost storey prevents overturning. A residual rotation is present. Overturning occurs even reducing the PGA to 0.40 *g* or to 0.35 *g*. In both these cases, eliminating the thrust at the uppermost storey prevents overturning, as well as residual rotation.

Table 9.7:Calculated global strength (total base shear) of the walls of building "A"; (·): residual value.

Wall	Research Unit	*W* (kN)	*H* (kN)	*H*/*W* (%)
A	Basilicata	4145	1459	35.2
	Genova	3575	1673	46.8
	Pavia[1]	3764	1110	29.5
	Pavia[2]	3764	960	25.5
B	Basilicata	814	208	25.6
	Genova	772	244 (150)	31.6 (19.4)
	Pavia	780	198	25.4
C	Basilicata[3]	3202	1026	32.0
	Basilicata[4]	3202	1178	36.8
	Genova[3]	3170	1104	34.8
D	Basilicata	1965	503	25.6
	Genova	1753	617 (450)	35.2 (25.7)
	Pavia[1]	1903	467	24.5
	Pavia[2]	1903	403	21.2

[1] *Seismic loads distribution according to the R.U. of Genova.*
[2] *Seismic loads distribution according to the R.U. of Basilicata.*
[3] *Seismic loads in positive direction.*
[4] *Seismic loads in negative direction.*

Assuming that out-of-plane collapses are prevented by proper means (e.g. ties), the in-plane response of four walls was studied by the R.U. Universities of Basilicata, Genova, and Pavia. These analyses reported values of *H/W* (ratio between the maximum base shear and the total gravity load) between 21% and 47% (Table 9.7). If the latter upper value is excluded, since was obtained by the R.U. of Genova for one wall only, the maximum value is 37%. The results of the R.U. of Genova give also information on the residual resistance in the post-peak response. It should be pointed out that the in-plane analyses on single walls are optimistic, in the sense that each wall is supposed to be subjected to the seismic loads associated to the masses and gravity loads carried by the wall itself, while in reality it will be also subjected to the seismic loads coming from the masses carried by orthogonal walls. The analysis of the whole building, considering only in-plane response of walls, was carried out by the R.U. of Basilicata, and gave values of *H*/*W* between 11% and 18% (Table 9.8). These results would indicate the collapse of the building under the average scenario seismic excitation for the city of Catania, where the PGA values range mostly between 0.25 *g* and 0.35 *g*.

Table 9.8: Global strength (total base shear) of building "A" (R.U. of Basilicata, *W* = 21 221 kN).

Direction and sign	*H* (kN)	*H/W* (%)	Direction and sign	*H* (kN)	*H/W* (%)
+X	3056	14.4	–X	3395	16.0
+Y	2377	11.2	–Y	3735	17.6

9.4.2 *Building "B"*

9.4.2.1 *General description*

The building was erected in 1952, and has an L-shaped plan (Fig. 9.17). The stairwell is located at the inner corner between the two wings, which extend along the sides of the block and end against the adjacent buildings. On the longer wing of the building, a wide carriageable entrance hall gives access to the internal courtyard. The typical storey plan has two apartments with a rather regular internal layout, consisting of a central corridor which gives access, on both sides, to the rooms.

The ground floor, which is at a height of 60 cm above the ground level, differs from the typical storey because of the entrance hall. Also the fourth, uppermost storey differs from the others, especially on the shorter wing of the building, where a peripheral wall sets back to allow the creation of a small terrace.

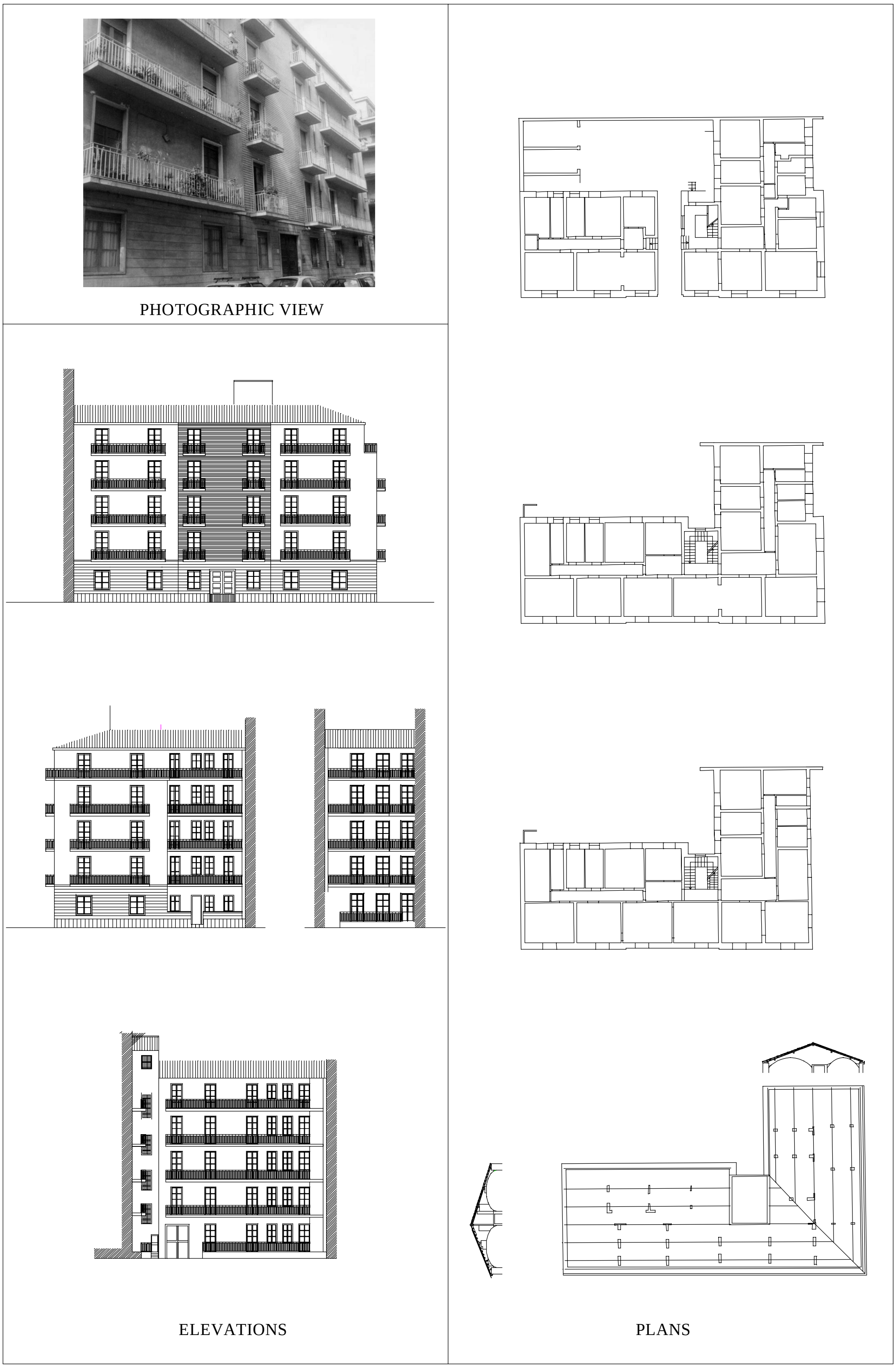


Figure 9.17 - Photographic view, plans and elevations of building "B".

9.4.2.2 *Structural system*

The peripheral walls consist of masonry with irregular fabric made of lavic stone and mortar (apparently the traditional mortar of lime and “azolo”), and have an average thickness of 60÷70 cm. The peripheral walls facing the internal courtyard have constant thickness along the whole height of the building, while the walls facing the street decrease their thickness above the second floor. On the shorter wing, the external wall at the last floor, which sets back with respect to the lower storeys, has a smaller thickness and is presumably made of solid brickwork. The openings on the external walls are French windows with rolling-shutter box. The central corridors are delimited on both sides by solid brickwork walls, with a thickness of approximately 25÷30 cm at the ground storey and first storey, of 18÷25 cm at the second an third storey, of 16 cm at the last storey. Orthogonally to the corridors, several walls delimit the rooms, with a thickness generally ranging between 30 and 40 cm, with the exception of offset walls, which have a thickness of 16 cm.

The tile-lintel floors have a depth of about 20 cm and are connected to the walls by means of R/C ring beams having a depth of about 30 cm. The joists span in both orthogonal directions. The balconies consist of cantilever slabs.

The ceilings at the last storey generally consist of “false” vaults, and the roofing is made of Sicilian curved tiles with a wooden structure.

The stairwell is made of reinforced concrete, with three flights and a landing per floor. The lift is positioned inside the stairwell.

Apparently there is no sign of existing damage in the building. However, the structure had been subjected to the earthquake of 13/12/1990, after which the building was subjected to some repairs which do not allow at present to identify the consequences of that event.

9.4.2.3 Numerical analyses

Building “B”, characterized by a good regularity in plan and in elevation, was subjected to many types of analysis, using simplified design models, static macro-element models, static finite element models, and dynamic macro-element models. In particular, analyses were made on an internal brick masonry wall, parallel to the longer wing, with the purpose of comparing the results obtained by the different models. Also, static three-dimensional analyses of the building were made using simplified and macro-element methods.

The three-dimensional analyses gave values of H/W ranging between 8% and 22% (Table 9.9). On average, the highest values were obtained by the POR methods (H/W = 19.5%), applied in this study by the R.U. of the University of L’Aquila, and widely used in the current practice. The tendency to an “optimistic” estimate of strength is thus confirmed for these methods. However, the use of the POR90 method, which takes into account the flexural failure of piers, gives a reduction of about 40% with respect to the original POR approach (H/W = 12.3%). The static macro-element models of the Basilicata and Pavia Universitiesgive mean values of H/W = 15.8% and H/W=13.1%, respectively, which would indicate a collapse of the building under the scenario earthquake.

Table 9.9: Global strength (total base shear) of building "B".

		Direction X		Direction Y	
Model	R. U.	H (kN)	H/W %	H (kN)	H/W %
Elastic R/C ring beams (E = 20 000 MPa)	Basilicata[1]	6991	20.8	5915	17.6
	Basilicata[2]	5376	16.0	5376	16.0
Elastic R/C ring beams (E = 4000 MPa)	Basilicata[1]	5915	17.6	2689	8.0
	Basilicata[2]	4840	14.4	5376	16.0
Free torsion of floors, walls in Y dir. only	Pavia[3]			1258	8.6
Free torsion of floors, all walls considered	Pavia[3]			1932	13.1
Torsion restrained, walls in Y dir. only	Pavia[3]			2112	14.4
Torsion restrained, all walls considered	Pavia[3]			2385	16.2
POR, all walls, height = interstorey	L'Aquila[4]	6824	21.6	6097	19.3
POR, walls // to seis. action, height = interstorey	L'Aquila[4]	6318	20.0	5560	17.6
POR, all walls, height=height of openings	L'Aquila[4]	6824	21.6	5276	16.7
POR90, all walls, height = interstorey	L'Aquila[4]	3096	9.8	3791	12.0
POR90, walls // to seis. act., height = interstorey	L'Aquila[4]	2875	9.1	3475	11.0
POR90, all walls, height=height of open.	L'Aquila[4]	5023	15.9	4960	15.7

[1] Seismic loads in positive direction, W = 33 606 kN.
[2] Seismic loads in negative direction, W = 33 606 kN.
[3] The model considers only the part of the building on the longer wing, delimited by the access to the internal courtyard; seismic loads in positive direction, W = 14 710 kN.
[4] W = 31 592 kN.

The analyses on the internal wall, which involved also the R.U. of University of Genova, are not directly applicable for the evaluation of the seismic strength of the building, but are of special interest since they represent an in-depth comparison among different models developed in recent years. Unlike the three-dimensional analyses, substantial differences were reported among the different models. In particular, the highest values of strength were found by the R.U. of Basilicata, followed, in order, by the R.U. of L'Aquila, of Genova and of Pavia (Table 9.10). Without entering into details, it seems that the differences are due, in part, to the different ways of modelling the coupling elements (in particular masonry spandrel beams). It is interesting to point out that these methods had predicted with a good approximation the response of a two-storey masonry building prototype tested at the University of Pavia some years ago, within a research funded by GNDT (AA.VV., 1995). Since the prototype had only two storeys, the role of the coupling elements was secondary in the final determination of the collapse mechanism, which was mainly governed by the strength of piers. In the internal wall of building "B", which has a higher number storeys, the coupling given by masonry spandrels and R/C ring beams plays an important role on the overall response and on the collapse mechanism. It must be remarked that there is almost no experimental reference available on the response of unreinforced masonry spandrel beams subjected to seismic loading. The analyses presented herein show that experimental studies on the subject would be of great interest for the evaluation of the seismic response of buildings higher than two storey.

To conclude the comments regarding the comparison between different methods of analysis, the good agreement between the three-dimensional analyses of the R.U. of Basilicata and Pavia could be explained with the absence of spandrel beams in the peripheral walls of the building. The only coupling element is thus represented by the R/C ring beams, for which comparable models were used.

Table 9.10: Global strength of the internal wall of building "B"; (·): residual value.

Model	R. U.	H (kN)	H/W (%)
Without ring beams	Basilicata[1]	1406	38.4
	Genova[2]	993 (700)	25.3 (17.8)
	Pavia[3]	656	19.7
With elastic ring beams (E = 20 000 MPa)	Basilicata[1]	2050	56.0
	Genova[2]	1492	38.0
	Pavia[3]	1227	36.9
With elastic ring beams (E = 4000 MPa)	Basilicata[1]	2050	56.0
	Genova[2,4]	1263	32.2
	Pavia[3]	848	25.5
Elastic ring beams (E = 20 000 MPa) with rigid arms	Basilicata[1]	2226	60.8
Elastic ring beams (E = 4000 MPa) with rigid arms	Basilicata[1]	2109	57.6
Elasto-plastic ring beams (E = 4000 MPa)	Pavia[3]	674	20.3
Limit analysis (overturning of cantilever walls)	Genova[2]	598	15.2
POR, pier height = interstorey height	L'Aquila[5]	1502	46.0
POR, pier height = height of openings	L'Aquila[5]	1630	49.9
POR90, pier height = interstorey height	L'Aquila[5]	1394	42.7

[1] W = 3661 kN [2] W = 3928 kN [3] W = 3327 kN [4] E = 5000 MPa [5] W = 3266 kN.

An interesting study on the internal wall is represented by the dynamic analyses carried out by the R.U. of University of Genova by means of a macro-element model. The available displacement ductility was evaluated as 3.6, and the corresponding force reduction factor (behaviour factor) equal to 2.8. Since the maximum spectral acceleration for the scenario earthquake falls in the period range T = 0.2÷0.4 s – where the natural period of the considered building is expected to fall – and reaches values from 0.7 g to 1.0 g, the design acceleration, calculated dividing the spectral acceleration by the behaviour factor, will have a value ranging between 0.25 g e 0.36 g. In the dynamic response, therefore, the available ductility of the wall is compensated by the dynamic amplification. If this result is extended to the whole building, the collapse of the structure would be foreseen under the scenario earthquake. It can be finally remarked that the behaviour factor calculated by the dynamic analysis of the internal wall is higher than the value of 1.5 adopted by Eurocode 8 for “good” unreinforced masonry construction. This discrepancy shows how the problems regarding the evaluation of the behaviour factor for unreinforced masonry buildings are still open, calling for further in-depth numerical and experimental investigation to evaluate the influence exerted by the numerous mechanical and geometric parameters.

9.5 **Vulnerability and probability of collapse for classes of masonry buildings**

(P. Arezzo, A. Bernardini, R. Gori, E. Muneratti, C. Paggiarin, O. Parisi, G. Zuccaro)

9.5.1 ***The LSU and CONARI Databases***

In the years 1996-1999 the buildings of Catania have been systematicallysurveyed as a part of the activities co-ordinated by GNDT (Gruppo Nazionale per la Difesa dai Terremoti) and sponsored by the public funds for the so-called *Lavori Socialmente Utili* (LSU, or "Socially useful work") Project.

As of September, 1999 the data of about 12.900 masonry buildings have been recorded in the LSU database (see sub-sect. 5.4). It is likely that the masonry buildings surveyed by LSU project will total around 17.900, so that the current database contains about 72% of the final records (Table 9.11).

Table 9.11: LSU-Catania Database

	Masonry buildings	RC buildings	Total
Records (22-09-99)	12899	6608	19507
Unrecorded (estimate)	5000	1600	6600
Total (estimate)	17900	8200	26100

The database describes each building by means of 15 parameters of the "first level" GNDT form concerning: type of floors and walls; number of stories, maximum and minimum height (but not the average built surface and then the volume); place, location, year of construction, following interventions, state of conservation of plasters and use. Furthermore, 3 of the 11 parameters of "second level" GNDT form for masonry buildings have been evaluated.

Since on January 1999, the database contained only 8097 masonry buildings placed in the suburban sections, a second database has been considered, created by CONARI Company on behalf of Catania municipality in 1989-1990, in view of a restoration plan of the historical town. This second database concerns about 6000 masonry buildings and collects records similar to those of LSU database, and additionally the total volume of the building (see also sub-sect. 5.3).

In Tables 9.12 and 9.13 a well-founded correlation is proposed between definitions and codes of the two databases, corroborated by the observed frequencies. The more evident anomaly concerns the considerable frequency of type G (squared stone masonry) in LSU database, while it is considered in CONARI database as class B, together with the roughly hewn stone masonry (declared with a quality lower than class C (*listata* or *intosta* masonry).

The survey observations seem to indicate the actual non-existence of square-cut and regular stones masonry across the entire wall thickness, with the exception of calcareous stones (*tufo*) masonry used in the last post-war in suburban areas, especially for building additions and as an alternative to concrete blocks masonry. This is the reason why in the following analyses type G erected before 1945 (G1) will be grouped with type C, while the most recent buildings (G2) will be grouped with the concrete blocks masonry type (H or I) . It seems likely that some LSU surveyors used C and G codes for classing V1 and V2-V3 types (see § 9.2) respectively. This could justify the low values of the frequencies of LSU A, E, B, F codes, compared to CONARI A code. A reasonable criterion to solve the problem has been assumed by using the LSU field "State of plasters" to subdivide buildings classified as C in two groups: C1 with a poor conservation state; C2 with a good conservation state.

Table 9.12: Masonry types according to GNDT/LSU and CONARI codes, listed following increasing values of tensile strength

LSU				**CONARI**		
Code	Description	τ_k suggested (*) (MPa)	Relative Freq. (%)	Code	Description	Relative Freq. (%)
A	Double leaf masonry	0.04	0.0	A	Masonry with irregular fabric	31.1
E	Rounded stones	0.04	2.5			
B	Double leaf masonry with transverse connect.s	0.04	0.3			
F	Rounded stones with transverse connections	0.04	0.6			
C	Roughly hewn lavic stones	0.07	66.1	B	Square cut stones or roughly squared stone	52.0
D	Roughly hewn lavic stones with transverse connections	0.07	6.9	C	"*Listata*" (regular fabric with brick courses)	6.6
G	Calcareous stone blocks, squared stones	0.10	20.0	E	Blocks	1.9
H	Heavy concrete blocks	0.10	0.5			
M	Hollow clay bricks	0.10	0.1			
I	Light concrete blocks	0.15	1.9			
T	Mixed		0.4	H	Mixed	1.9
L	Solid or hollowed clay bricks	0.18	0.8	D	Clay bricks	6.5
O	Reinforced concrete	0.2		F	R.c. frame	

(*) Martinelli, 1998

Concerning the horizontal structures, a higher coherence can be found in Tab. 9.13, by comparing the two databases, taking into account the larger frequency of vault structures in the historical centre. The Table shows the scarce diffusion of tie-rods, the prevalence of floors with steel beams together with the traditional type of lavic stones or *pomice* vaults, mostly in the historical centre. The modern light reinforced concrete floors are very widespread, particularly in the suburban city sections, often on masonry structures of poor quality.

In addition, a number of buildings classified as having vaults (F) in LSU database have the indication that tie-rods or RC ring beams are present at all the storeys. For these buildings (F3) the following analyses have been carried out grouping them as them in G code.

Table 9.13: Floors types according to GNDT/LSU and CONARI codes, listed following increasing values of confinement effects on masonry

LSU			**CONARI**		
Code	Description	Relative Frequency (%)	Code	Description	Relative Frequency (%)
F	Vaults without tie-rods	14.4	N	Stone vaults	28.9
			O	Clay brick vaults	
A	Wooden floors	2.1	A	Wooden floors	20.2
H	Mixed vaults – plain floors	5.4			
G	Vaults with tie-rods	0.5	NP, OP	Vaults with tie-rods	2.3
I	Mixed vaults - floors with tie-rods	0.1			
C	Steel beams and vaults or tiles	47.3	L	Mixed floors (steel beams and vaults)	32.5
B	Wooden floors with tie-rods	0.4	AP	Wooden floors with tie-rods	1.0
D	Steel beams and vaults or tiles with tie-rods	0.5	LP	Mixed floors with tie-rods	
E	Solid or light RC slabs	29.3	B	Reinforced concrete floors	15.1

9.5.2 Definition of a significant sample

In the Spring of 1999 a sample of about 100 masonry buildings of Catania has been selected to control the reliability of the information collected in the LSU database.

The LSU database itself gives important criteria which allow to improve the significance of the sample, preserving the relative frequency of wall and floor types, age and number of stories. Moreover, taking into account that at that time the historical centre was not yet included in the LSU database, the sample has been enlarged with buildings identified by means of the CONARI database, the catalogue of buildings carried out with the co-ordination of A. Barbera, University of Catania (BARBERA database), the catalogue of the minor buildings in the *Picanello* quarter carried out by E. Pagello (Pagello, 1990) (PAGELLO database). Finally the building blocks A and B described in § 9.4 have been added to the sample.

Finally, taking into account that in several cases the building blocks have been further subdivided into homogeneous buildings, due to their lack of geometric homogeneity, a sample of 135 buildings has been assembled. In Figs 9.18 e 9.19 the sample is compared with the LSU database, from the point of view of the frequencies of number of stories and age of the building (year of construction). It must be noted that the first choice of using the CONARI, BARBERA and PAGELLO databases has produced a sample of buildings fairly older in comparison with the reality described by LSU database. On the contrary, the frequencies corresponding to the number of storeys are substantially similar, even if with a slight over-estimation in the sample of the tallest buildings.

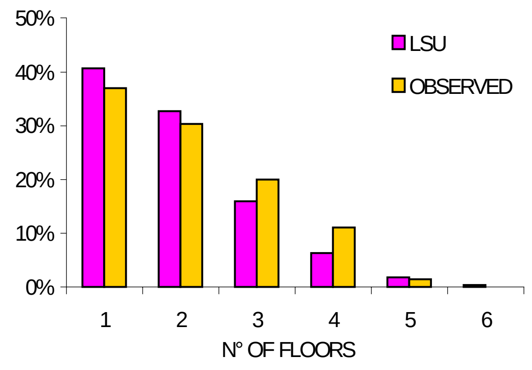


Fig. 9.18 - Relative frequencies of number of stories in the of LSU – 22.09.99 database (12000 buildings) and in the observed sample (135 buildings)

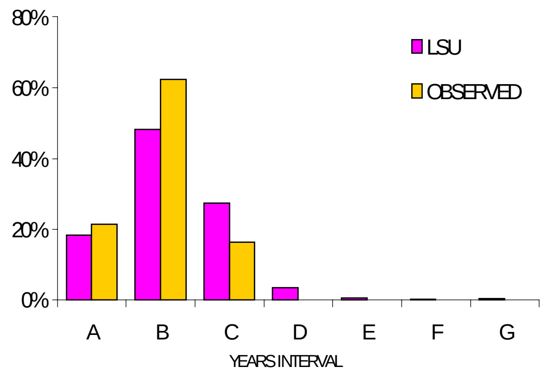


Fig. 9.19 - Relative frequencies of construction age in LSU database and in the observed sample (A: <1919; B: 1919–1945; C: 1946–1960; D: 1961–1971; E: 1972–1975; F: 1976–1980; G: after 1980)

9.5.3 *Vulnerability analysis according to VULNUS methodology*

The VULNUS procedure (Bernardini and al., 1989) is based on a vulnerability model of masonry buildings, that depend on the following parameters:

I_1 : ratio of in-plane shear strength of the walls system to total weight;

I_2 : ratio of out-of-plane flexural strength of the most critical external wall to total weight, evaluated by summing the resistance of vertical (I2’) and horizontal (I2’’) strips;

I_3 : weighted sum of the scores of seven partial vulnerability factors;

A : mean absolute acceleration response of the building;

a : uncertainty factor depending through a fuzzy relation on I3.

The output Vu = f(I1, I2, A, a) is the Probability of collapse or damage ≥ degree D4 (EMS98 : European Macro-seismic Scale 1998). The analysis can be performed for a building (Vu) or for a group of buildings (Vg).

Upper bounds, lower bounds and mean “white probabilities” of the Cumulative PDF F(Vu) or F(Vg) , as well of the corresponding Expectations E[Vu] or E[Vg], can be calculated according to the Theory of Random Sets (Bernardini, 1999) from the obtained fuzzy sets.

Shaking table tests on masonry buildings models (Benedetti and Pezzoli, 1996) show that in the highly damaged state A is nearly equal to PGA.

For example Figures 9.20-21 gives E[Vg] for the blocks A and B (see sub-sect. 9.4), each one subdived into two interacting buildings. For both blocks, the following characteristics of the walls have been assumed: average compression strength = 3 MPa (justified by results of flat jacks tests), density = 2300 kg/m^3, average tensile strength = 0.20 MPa (1/15 of compression strength, justified by the good quality of the mortars).

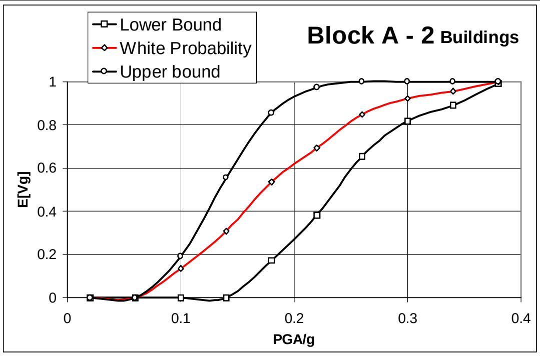


Figure 9.20 - Building block A of via Verdi / via Capuana: expected value of vulnerability, average for the two interacting parts.

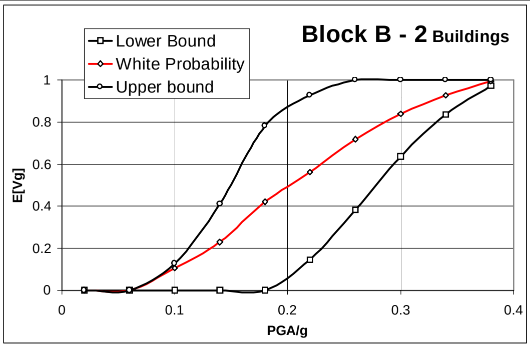


Figure 9.21 - Building block B of via Martoglio: expected value of vulnerability, average for the two interacting parts.

*Walls mechanical characteristics and interaction forces between floors and walls*The numerical values assumed for the main mechanical parameters required by VULNUS analysis code for vertical and horizontal structures are given, respectively, in Tables 9.14 and 9.15.

It must be observed that such values represent just reasonable hypotheses based on the experimental tests carried out by flat jacks technique discussed in § 9.3.2.2 and similar tests carried out on Catania Cathedral (Leone, 1995), and also on tests described in (Sciuto Patti, 1896). Nevertheless, the uncertainties linked to such values are taken into account in the analysis by means of the fuzzy representation of the vulnerability measures.

The choice of the values to be assumed for the active confinement forces on the walls, corresponding to the various floor types, is particularly difficult. In the case of plane floors they have been assumed substantially proportional to the vertical support reactions multiplied by friction coefficients varying between 0.3 and 0.6.

Table 9.14: Average strengths and densities of masonry types in the sample

	LSU Code	Compression strength (MPa)	Tensile strength (MPa)	Specific density (kg/m^3)
Irregular fabric of rubble lavic stones	A, E	1.2	0.07	1800
Irregular fabric reinforced by transverse *cannarozzoni* and/or clay bricks .	B, F, C1	2	0.12	2000
Quasi-regular fabric of roughly hewn lavic stones with nearly horizontal mortar joints	C2, G1	3	0.20	2300
Quasi-regular fabric of roughly hewn lavic stones reinforced by layers of clay bricks	D	4	0.22	2200
Regular fabric of concrete blocks or calcareous *tufo* hewn stones	H, I, G2	4	0.20	1700

As it regards the vaults the thrusting effect due to vertical loads should be taken into account, as well to the vertical components of acceleration, uniformly distributed on the boundary walls for *padiglione* vaults, substantially concentrated and absorbed by transverse walls for *crociera* vaults.

This consideration could suggest assuming negative values of confinement forces.

A careful observation of the geometry of Catania vaults made of pumice-stone, almost semicircular, seems to suggest a substantial balancing of positive and negative effects on confinement, justifying then values close to 0.

Table 9.15: Average confinement forces and unit weight of floor types in the sample

	LSU Code	Confinement on walls orthogonal to the beam direction (kN/m)	Confinement on walls parallel to beam direction (kN/m)	Unit weight (kN/m^2)
padiglione vaults on thin shoulders, without tie-rods	F1	- 1	- 1	3 - 4.5
padiglione or *crociera* vaults on thick shoulders, without tie-rods	F2	0.5	0.5	3 - 6
Wood beams without tie-rods	A	2	0.5	1.5 - 3
Steel beams and vaults	C	6	1	3 - 4.5
Solid or lighted RC slabs	E	20	10	3 - 6
Mixed vaults- plain floors	H	Average values weighted with their relative areas are assumed		
Floors with tie-rods	I, B, D	The procedure evaluates separately the contribution of the tie-rods in the two principal directions (15 kN) for each building and adds it to the corresponding forces of the various typologies.		

9.5.4 *Preliminary ordering of the buildings into 3 vulnerability classes*

A first classification of the buildings, significant for the analysis of seismic vulnerability, may be done by calculating, for each building, a parameter called "MSK class" of domain (A, B, C), according to the rule of combination of the qualities of vertical and horizontal structures, shown in Table 9.16. The usual criterion of considering three classes of decreasing vulnerability in the macro-seismic MSK (or EMS98) scale is assumed.

Table 9.16: Hypothesis of classification of masonry buildings into three vulnerability classes

STRUCTURES	**Horizontal**	Vaults	Wooden floors	Mixed vaults / steel & vaults floors	Vaults or mixed vaults / floors with tie-rods	Steel beams and vaults or tiles	Wooden floors with tie-rods	Steel beams & vaults or tiles with tie-rods	RC slabs
Vertical	LSU Code	F, F1, F2	A	H	G, I	C	B	D	E
Irregular fabric of rubble lavic stones, low or fair quality mortar	A,E	**A**	**A**	**A**	**A**	**A**	**A**	**A**	**A**
Irregular fabric reinforced by *cannarozzoni* and/or clay bricks, low or fair quality mortar	B, F, C1	**A**	**A**	**A**	**B**	**B**	**B**	**B**	**B**
Quasi-regular fabric or roughly hewn lavic stones with nearly horizontal mortar joints, low or fair quality mortar	C2, G1	**A**	**A**	**B**	**B**	**B**	**B**	**B**	**C**
Mixed walls of medium quality	T	**A**	**A**	**B**	**B**	**B**	**B**	**B**	**C**
Quasi-regular fabric or roughly hewn lavic stones reinforced by layers of clay bricks, fair quality mortar	D	**A**	**A**	**B**	**B**	**B**	**B**	**B**	**C**
Regular fabric of concrete blocks or calcareous tufo hewn stones, fair quality mortar	H, I, M, G2	**A**	**A**	**B**	**B**	**B**	**B**	**B**	**C**
Regular fabric of solid or low hollo-wed clay bricks, good quality mortar	L	**A**	**B**	**B**	**B**	**B**	**B**	**C**	**C**

The list ordered by decreasing vulnerability, shown in Table 9.16, is based on considerations concerning the wall resistance (according to data of Table 9.12, and further assumptions resumed in § 9.5.2), and, as regards the horizontal structures, on considerations concerning the positive effect of confinement forces and of tie-rods (when they are present), and the negative effect of the dead load. It must be noted that such a classification of the national GNDT codes has a valid meaning just for the specific application to Catania, by interpreting in the best reasonable way the criteria adopted by LSU surveyors.

The resulting classification is shown in Figure 9.22, where the large differences of the relative frequencies in the LSU database and in the observed sample appear clearly. In Figure 9.23 an homogeneous comparison is shown for 81 buildings respectively as recorded in LSU database and observed in the survey. It seems that such a considerable difference could be mainly due to inconsistent recording of the structural types in LSU database, as documented in the survey, particularly for the horizontal structures.

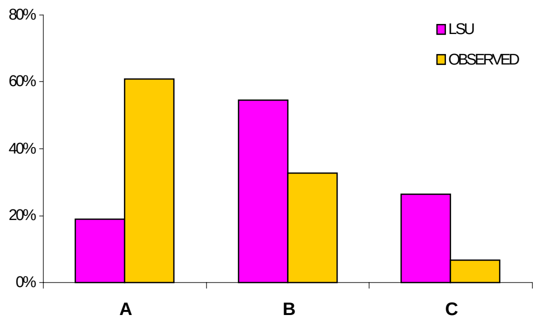


Fig. 9.22 - Relative frequencies of MSK classes in LSU database (12000 buildings) and in the observed sample (135 buildings).

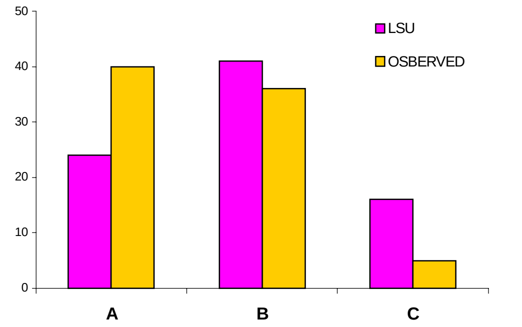


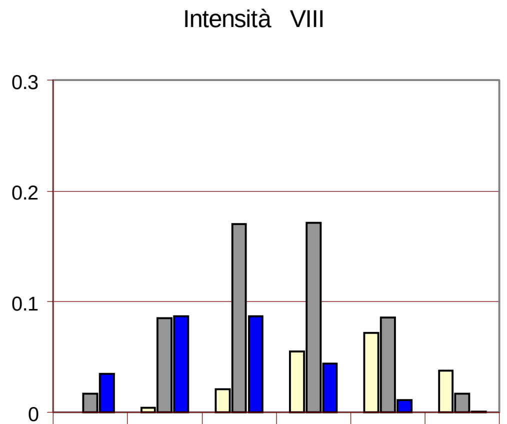
Fig. 9.23 - MSK Classification of a sample of 81 buildings, according to characteristics assumed by LSU teams and those actually observed.

9.5.5 *Vulnerability analyses*

The evaluation of the expected vulnerability for the classes A, B and C in Catania has been computed by using the Damage Probability Matrices (DPM) calibrated during the Irpinia earthquake in the 1980 (Braga et al. 1982). The study has been carried out on the masonry buildings only, in agreement with the scope of the present contribution. At the moment, specific DPMs for the Catania buildings are not available. The comparison between the typological characteristic of the building structures of both areas (Catania, Irpinia) brings out basic differences in the mechanical characteristics of the material used either the vertical structures (harder volcanic stones and some good mortar in Catania against rough stones with worse mortar in Irpinia), or in the horizontal structures (vaults more diffuse in Catania and wooden floors in Irpinia). However the application of the DPM ’80 is justified by the good agreement in the number of storeys and the age of the buildings, as well as by the behaviour of the masonry buildings without tie-rods.

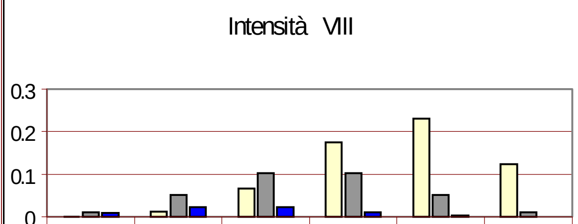
Hence, the following damage evaluation should be considered as overestimated both for the quality of the buildings, generally more reliable in Catania, and for the topographic site effects, more critical in Irpinia. This damage overestimation due to the Irpinia DPM analysis is confirmed by the study carried out in another volcanic area, the Vesuvian villages, where comparison with other vulnerability analyses, has been performed. However, it has been shown, in the Vesuvian case, that the vulnerability index analysis (Benedetti and Petrini, 1984) underestimates the number of collapses so that an averaging between both evaluations should be considered.

In Figures 9.24 and 9.25 the damage distributions (ratio of the frequencies to the total number of buildings), for the overall building stock (LSU database:12000 buildings) and for two different MSK intensities, are illustrated and compared with the percentage of damage derived by using the sample of the 135 buildings.



	% D0	% D1	% D2	% D3	% D4	% D5
A	0	0.004	0.021	0.055	0.072	0.038
B	0.017	0.085	0.17	0.171	0.086	0.017
C	0.035	0.087	0.087	0.044	0.011	0.001

a)



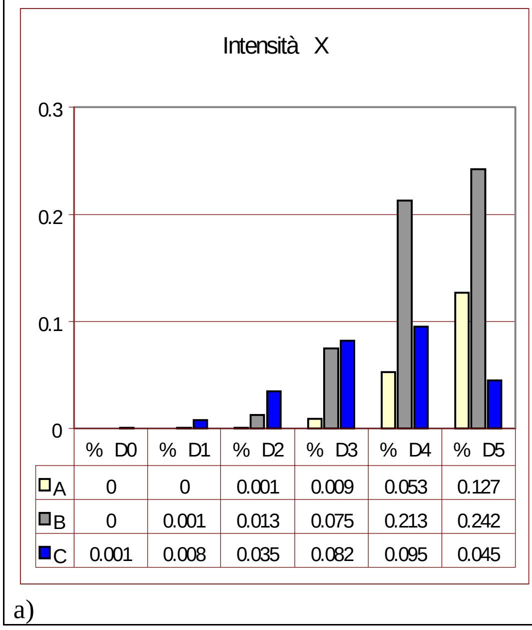
	%D0	%D1	%D2	%D3	%D4	%D5
A	0.001	0.012	0.066	0.174	0.231	0.123
B	0.01	0.051	0.102	0.102	0.051	0.01
C	0.009	0.022	0.022	0.011	0.003	0

b)

Fig. 9.24 - Comparison between the percentage distribution of damage for intensity VIII computed on the LSU data (a) and the 135 buildings sample (b). (D0: no damage; D1: slight; D2:moderate; D3: substantial to heavy; D4:Very Heavy; D5: Destruction

The expected vulnerability forecasted by VULNUS methodology, for the corresponding observed samples, is shown in Figs. 9.26 for the three above defined classes of buildings.

Comparison of the results of the two methodologies is possible taking into account that Vg is the probability of damage $D \geq D_4$ and the correlation between macro-seimic intensities and PGA (or better Equivalent PGA) values. For Italian earthquakes it can be suggested (Guagenti and Petrini, 1989): $\ln(PGA/g) = 0.602\ I - 7.073$; i.e. $PGA/g = 0.35$ for I = X ; $PGA/g = 0.10$ for I = VIII.



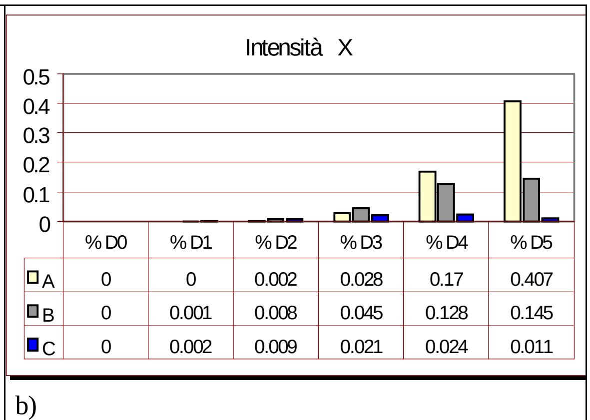
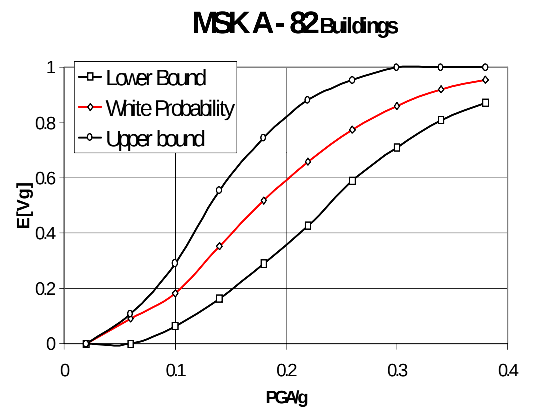
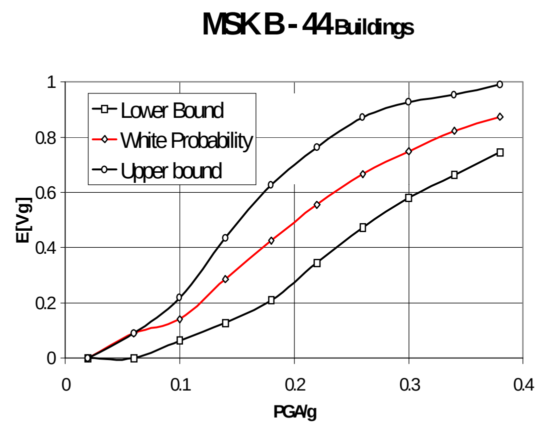


Fig. 9.25 - Comparison between the percentage distribution of damage for intensity X computed on the LSU data (a) and the 135 buildings sample (b). (D0: no damage; D1: slight; D2:moderate; D3: substantial to heavy; D4:Very Heavy; D5: Destruction





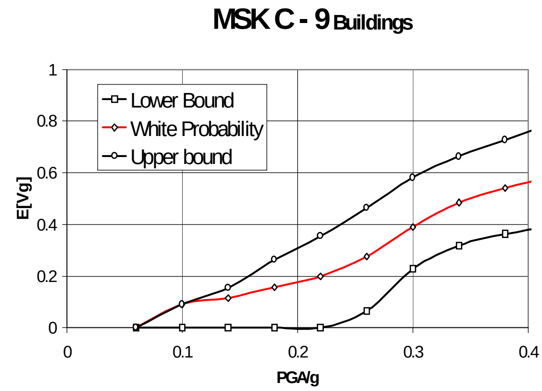


Figure 9.26 - Bounds and central ("White") value of the expected vulnerability, as a function of PGA/g, for the three assumed classes of buildings. The mean absolute acceleration response of the building has been assumed equal to PGA (Bernardini, 1999).

9.5.6 *Damage scenarios of masonry building stock for the reference earthquakes*

The considerable uncertainty related to the definition of vulnerability classes, computation of the relative frequencies from the partially incomplete LSU Database, lack of specific DPM for the Catania area suggest great caution in forecasting damage scenarios for the masonry building stock under the reference earthquakes.

In Figure 9.27 the calculated upper and lower bounds of Vg are shown for the overall sample of 135 buildings: for the higher intensity (mean PGA/g = 0.30 in the town) Vg should be in the range [0.60, 0.90], while Figure 9.25 gives 0.925 (more specifically 0.322 for D4, 0.563 for D5) for the sample, 0.775 for the LSU database; for the lower intensity (mean PGA/g in the town nearly equal to 0.20) Vg is in the range [0.25, 0.70], while Figure 9.24 (IMSK = VIII) gives 0.418 for the sample, 0.225 for the LSU database. This second scenario does not seem to be consistent with the recorded value of IMCS = VII for the 1818 event, see sub-sect. 1.2.

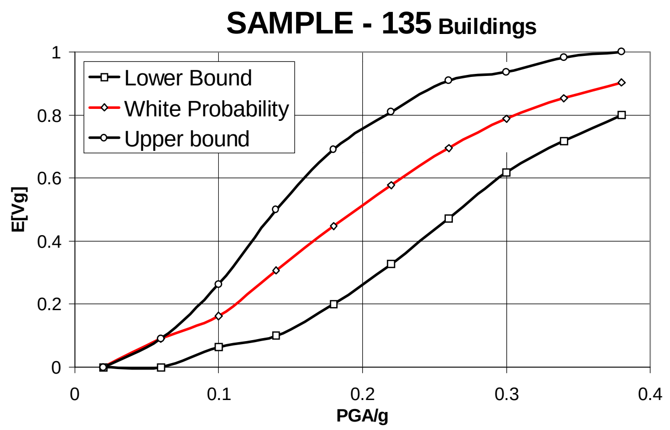


Figure 9.27 - Bounds and central ("White") value of the expected vulnerability, as a function of PGA/g, for the total sample of 135 surveyed buildings. The mean absolute acceleration response of the building has been assumed equal to PGA (Bernardini, 1999).

9.6 Conclusions

(D. Liberatore, A. Bernardini, G. Beolchini, L. Binda, L. Gambarotta, G. Magenes and G. Zuccaro)

The experimental investigation carried out on the Catania masonry buildings allowed to apply with apparent success the approach proposed by the authors. The cross-section survey informs on the morphology of the walls, on the number of leaves they consist of, on the type of connection between the leaves and on the presence and distribution of voids. This is important information for the structural analysis and for an appropriate choice of strengthening. The laboratory tests are useful to characterise the component materials, mortars, bricks and stones in order to choose the new materials for repair. The flat-jack test is at present the only on site mechanical test to give quantitative information on the local state of stress and on the masonry mechanical behaviour.

The numerical analyses carried out on the two sample buildings showed a high seismic vulnerability and foresee their collapse under the scenario earthquake.

Building "A", erected in the nineteenth century and representative of most of the historical centre, undergoes overturning of the peripheral walls under the combined action of the thrust of the vaults and the seismic motion at the base. Further analyses, carried out supposing to prevent overturning (e.g. through ties), showed a seismic capacity, expressed by the ratio H/W between the lateral strength and the total weight, ranging between 11% and 18%, that is significantly lower than that demanded by the scenario earthquake, with PGA ranging between 0.25 g and 0.35 g.

As for building "B", representing part of the masonry buildings erected in the post-war period, and provided at each storey with tile-lintel floors connected to the masonry through R/C ring beams, the risk of out-of-plane collapse of the walls can be neglected. The analyses of the in-plane response show that H/W ranges between 8% and 22%, quite similar, on the average, to building "A". Dynamic analyses on an internal wall in brick masonry show that the ductility resources are compensated by the spectral amplification of the scenario earthquake, thus confirming the collapse foreseen by the static analyses.

Preliminary estimates of expected damage to the masonry building stock have been evaluated through classification of the buildings in three groups of increasing vulnerability, and two independent methodologies based respectively on statistically evaluated Irpinia DPM and VULNUS procedure. Both methods confirm a very high percentage (from 60 to 90%) of collapsed or heavily damaged (in any case unusable in the post-event emergency) buildings for the Level I earthquake, while the uncertainty is greater for the Level II earthquake: for the suggested values of PGA the corresponding range is [25, 70] %, while a lower range from 8 to 25 % is consistent with the suggested macro-seismic intensity. The damage forecast for the level I and II scenario earthquakes are quite consistent with the global, simplified damage predisctions (and the corresponding GIS maps) illustrated in sect. 11 (Faccioli et al.).

Completion and verification of the LSU database, evaluation of volume distributions, calibration of the vulnerability classes (taking into account other relevant parameters, such as number of storeys, age, etc.) and of the Damage Probability Matrices are required to validate this preliminary results for the overall area and to specify damage distributions in the different city sections.

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