SEISMIC RESPONSE OF TYPICAL MASONRY BUILDINGS IN THE COMMUNE OF CATANIA

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SUMMARY

The seismic response is studied of two masonry buildings representing as many building types of Catania. The first building, dating back to the nineteenth century, has structural vaults at all the storeys and is unprovided with tie-rods. The most probable failure mode is overturning of the walls. In order to determine the vulnerability to overturning, two masonry piers are analyzed, subjected to a seismic motion at the base. Afterwards, assuming to prevent overturning (e.g. through tie-rods), the in-plane response is studied. The second building represents those buildings erected during the post-war period. It has tile-lintel floors connected to the masonry through R/C ring beams. The risk of overturning can be neglected. The study of the in-plane response shows high vulnerability due to the inadequacy of resisting elements.

INTRODUCTION

"Project Catania" of the Italian National Group for the Defence against Earthquakes (GNDT) consists in the evaluation of the impact, on the present city of Catania, of an earthquake similar to the destructive earthquake of 1693. To this end, the study of the response of masonry buildings, both in the historical centre and in the more recent neighbourhoods, is of paramount importance.

Two buildings were chosen for detailed analyses, based on a preliminary study of the materials, elements and building types of Catania. The first building was erected in the nineteenth century and represents rather valuable constructions of the historical centre. The second building represents part of those buildings erected in the post-war period. These buildings have been subjected to experimental and numerical investigations by several Research Units of GNDT. In this paper, the investigations of the University of Basilicata are reported.

The outcomes of the analyses have been used to calibrate the vulnerability evaluations at medium and large scale.

BUILDING "A"

2.1 Short description and structural characteristics

The building is part of a larger block which develops around an inner courtyard. The investigation is focused on the wing dating back to the second half of the nineteenth century, which is separated from the older wing by a construction joint (Liberatore *et al.* 1999).

The building has C-shaped plan and three floors. The ground floor, used as shops, has a large entrance hall through which the stairs and the courtyard are accessed. The original plan is changed by some openings made in interior walls. The upper floors, used as flats, present a rather regular layout of resisting elements and large windows on the secondary elevation.

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The perimetrical walls are in masonry of lavic stones with irregular fabric; the interior walls are made by "cannarozzoni" or "intostoni" of square-cut lavic stone (Randazzo 1988). The mortar is made by lime and the so-called red "ghiara", originated by metamorphism of the paleosoil under the lavaflow due to high temperature.

The horizontal structure is made by "real" vaults in pumice-stone and plaster (Arezzo 1994, Randazzo 1988). The fill is replaced by conter-vaults. The key thickness is 10 cm. There are different types of vaults: cross vaults at the ground floor, barrel vault over the entrance hall, cloister vaults at the first and second floors.

In situ tests and laboratory tests were carried out on the masonry. The mechanical properties adopted in the analyses are shown in table 1.

Table 1. Building "A": mechanical properties of masonry adopted in the analyses.

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

2.2 Out-of-plane response of the walls

Since the building is completely unprovided with tie-rods and has thrusting vaults even at the upper storeys, the failure mode which appears most probable is overturning of the perimetrical walls (Liberatore and Spera 1999).

The dynamic response is calculated of two masonry piers, representative for geometry and loading. The piers are modelled as rigid blocks free to rock around their lower outer vertex (Fig. 1). The analyses have been repeated locating the centre of rotation at the height of each storey, in order to check the possibility of partial overturning. The thrust of the vaults has been cautiously calculated assuming the maximum possible inclination for the line of thrust.

The orthogonal walls are taken into account preventing the rotation of the perimetrical wall around the lower inner vertex. After any impact between the perimetrical wall and the orthogonal wall, the kinetic energy of the former is damped out.

The analysis leads to foresee overturning even under the gravity loads only. Therefore, the connection with the orthogonal walls shall be considered in the analysis.

The orthogonal walls are made by "intostoni", which are square-cut lavic stones with size 25×60×15 cm^3; in the analyses, the connection has been assumed given by the insertion for half length of one "intostone" every five in the perimetrical wall. The thickness of the wall orthogonal to pier 2, always greater than 50 cm, permits 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".

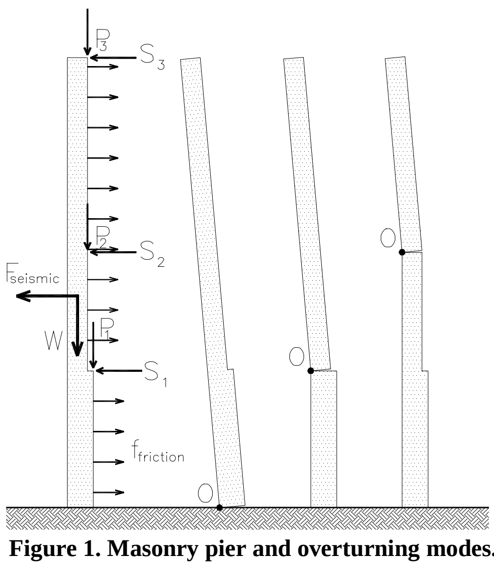

Figure 1. Masonry pier and overturning modes.

The connection between the perimetrical wall and the orthogonal wall has been modelled through forces corresponding to the courses of "intostoni" inserted into the perimetrical wall. During the rocking of the perimetrical wall, these forces have sign opposite to the velocity. They are calculated neglecting cohesion and adopting a friction coefficient between mortar and block equal to 0.5. The connection with the orthogonal walls prevents overturning under the gravity loads for both the piers.

An artificial accelerogram with 0.48 *g* PGA is 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 to 0.35 *g*.

Overturning of pier 1 as a whole occurs under the accelerogram with 0.48 *g* PGA. 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, to prevent overturning, it is sufficient to eliminate the thrust at the uppermost storey. A residual rotation is present in any case.

Overturning of the two uppermost storeys of pier 2 occurs under the accelerogram with 0.48 *g* PGA. 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.

2.3 In-plane response of the walls

The in-plane response is studied for some walls, assumed as isolated, and for the whole building.

The analyses have been performed through the program Mas3D, based on a no-tension panel element (Braga *et al.* 1997). Mas3D is able to model even very complex buildings, with zones of different height, different constraint conditions between floors and walls, offset floors and varying load along the walls.

To model the unilateral restraint at the base, vertical springs are used, quasi-rigid in compression and non reacting in tension. The loads applied by the vaults to the walls are determined accounting for the actual shape of the vaults: they are concentrated at the corners for the cross vaults, and spread along the length for the cloister vaults. Since the present implementation of Mas3D assumes rigid floors, it was not possibile to account for the deformability of the vaults.

The distribution of the seismic loads along the height is determined concentrating the dead and live loads of the floors at their height, and the weight of the masonry into the nodes of the model of the wall.

2.3.1 Analysis of isolated walls

The walls analyzed have been chosen to cover the different situations occuring in the building (Fig. 2):

- wall A, corresponding to the main facade, is rather squat and nearly symmetric; it is regular as for thickness, mechanical properties of masonry, size and layout of the openings, apart for the central opening at the ground floor, wider and higher than the others;
- wall B has small openings but high global aspect ratio (height divided by width) and small thickness (36 cm at the ground floor and 30 cm at the upper floors);
- wall C has aspect ratio nearly equal to 1, but is strongly asymmetric as for size and layout of the openings, thickness (ranging from 30 cm to 57 cm even within the same storey), seismic forces originated by the intermediate floors which are present only in some parts;
- wall D, corresponding to the secondary facade, is regular, with thickness close to that of wall A, but has greater aspect ratio, both global and of the individual panels.

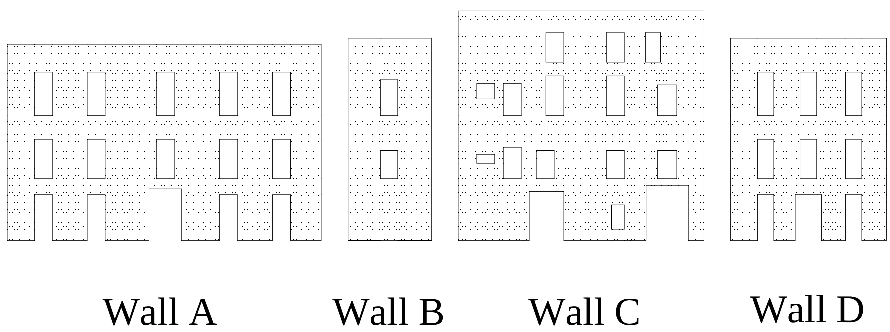

Figure 2. Walls extracted from building "A".

Static seismic loads are applied to each wall, according to the Italian regulations. Each wall is subjected to the seismic loads corresponding to the dead and live loads directly applied to the wall itself. It is worth to mention that this assumption is optimistic, because in the reality the wall is loaded also by the seismic loads originated by the gravity loads applied to the orthogonal walls.

The total gravity load W, the lateral strength H and the ratio H/W for the two possible signs of the seismic loads are reported in table 2. These results suggest some remarks:

- the largest lateral strength is attained by wall A, which is regular and squatter than the others; the lateral strength reaches 90% of that demanded for the 1st Italian seismic category (40% of the total gravity load);
- the greater aspect ratio of wall D, compared to wall A, reduces its lateral strength under the 2nd Italian seismic category (28% of the total gravity load);
- again, the high aspect ratio of wall B causes its lateral strength be less than that demanded for the 2nd category; its failure occurs because crushing at one vertex of the base;
- in constrast to the other walls, the asymmetry of wall C yields lateral strengths which are different for the two possible signs of the seismic action; in the case of loads from left to right (positive), collapse is triggered by crushing of the panel at the right end of the ground floor; in the case of loads from right to left (negative), the lateral strength is higher.

Table 2. Lateral strength of the walls extracted from building “A”.

Wall	*W* (kN)	Seismic loads with positive sign		Seismic loads with negative sign	
		H (kN)	*H*/*W* (%)	*H* (kN)	*H*/*W* (%)
A	4145	1459	35.2	1459	35.2
B	814	208	25.6	208	25.6
C	3202	1026	32.0	1178	36.8
D	1965	503	25.6	503	25.6

The failure mode of walls A and D presents a drift localization at the uppermost storey. On the contrary, the failure mode of wall B is characterized by a drift nearly constant along the height. Finally, wall C exhibits localization of drift at some piers; therefore the response of the wall is conditioned by some elements.

2.3.2 Analysis of the whole building

The lateral strength of the whole building is calculated for the two principal directions, named X and Y, and for the two possible signs (+ and –). In fact, the asymmetry of the structure yields to different responses for loads applied with opposite sign. *H* and *H*/*W* are reported in table 3. The total gravity load of the building is *W* = 21221 kN.

Table 3. Lateral strength of building “A”.

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

As expected, the ratios *H*/*W* for the whole building are significantly lower to those calculated for the isolated walls. As explained above, this occurs because the walls parallel to the direction assumed for the seismic loads are loaded – under the same gravity load – also by the seismic loads originated by the gravity loads applied to the orthogonal walls. Since the horizontal structure consists of cross vaults or cloister vaults, this effect explains why *H/W* halves in the average.

The displaced plots of the floors (Fig. 3), suggest the following remarks. At the first two storeys, where the vaults are nearly at the same height, only a slight torsion can be noticed, for seismic loads applied in the Y direction. At the upper storeys, where significant offsets are present, the situation is dangerous for different reasons:

- at the third and, where present, at the fourth storey, very strong localizations of drift occur, to confirm what already found for some of the isolated walls;
- in the X direction, the left wing of the building is more deformable than the remaining part; as a consequence, dangerous poundings (+X) and separations (–X) may occur, yielding to the collapse of the vaults;
- in the Y direction, torsion is more pronounced for the stairs.

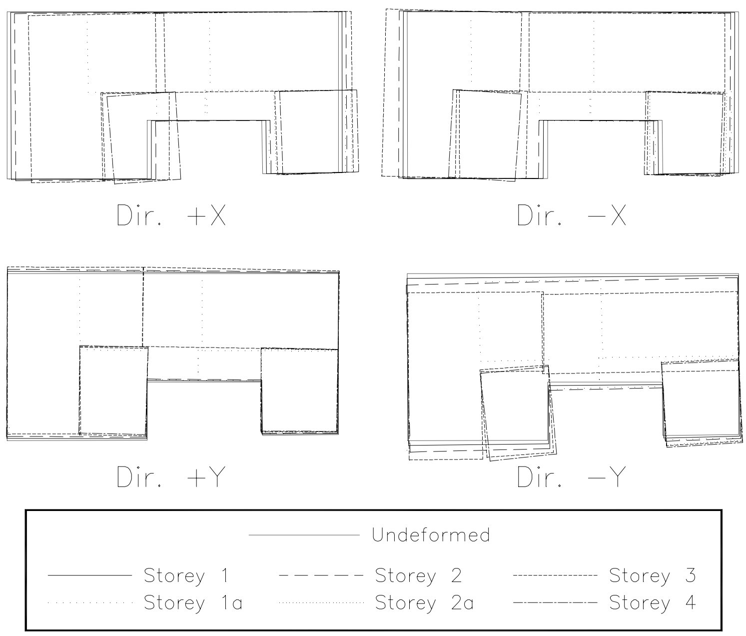

Figure 3. Displaced plots of the floors.

BUILDING "B"

3.1 Short description and structural characteristics

The structure, dating back to the fifties, has L-shaped plan with 5 floors (Liberatore *et al.* 1999). The building is part of a larger block. The ground floor has a large entrance hall which gives access to an inner courtyard.

Two types of masonry are present: masonry in lavic stone with irregular fabric and mortar of lime and "azolo" (Randazzo 1988) with thickness ranging from 55 to 80 cm for the perimetrical walls; brick masonry with two or more wythes and thickness ranging from 25 to 30 cm, adopted both for partitions and perimetrical walls in limited zones; brick masonry with single wythe and thickness ranging from 12 to 20 cm for partitions. The mechanical properties adopted in the analyses are reported in table 4.

Table 4. Building "B": mechanical properties of masonry adopted in the analyses.

	Masonry in lavic stone	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

The survey of the building showed that some walls orthogonal to the corridors are offset with respect to the walls below. In particular, the wall which separates the flats relies upon the floor over the entrance hall, near its midspan.

Tile-lintel floors are present, with joists spanning in both orthogonal directions, and thickness nearly equal to 25 cm. The floors are connected to the masonry through R/C ring beams. The balconies consist of cantilever R/C slabs.

The R/C ring beams prevent overturning of the walls. Therefore, the study is focused on the in-plane response. In particular, an interior wall in brick masonry and the whole building are analyzed.

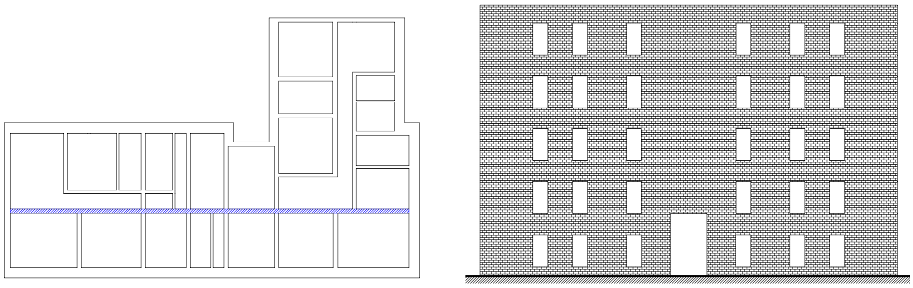

Figure 4. Interior wall in brick masonry.

3.2 Interior wall in brick masonry

The interior wall in brick masonry is one of the main resisting elements in the longer direction, named X (Fig. 4). It has thickness 30 cm with two wythes at the first four storeys, thickness 16 cm with single wythe at the uppermost storey. The weight of the wall, summed to the gravity loads applied by the floors, is W = 3661 kN.

The masonry piers between the openings are rather wide, especially at the ends of the wall; the spandrels are the weak element of the wall even though they are relatively squat.

The wall is regular apart from the central opening at the ground floor, which is very wide and high. The weight of the masonry over this opening is supported by a R/C beam.

Because of the weakness of the spandrels, the modelling of the R/C ring beams is of paramount importance, since they can exert a significant coupling between the different piers.

Initially, the wall without the ring beams is analyzed, in order to quantify their contribution to the lateral strength through the subsequent analyses.

The ring beams are modelled with elastic beam elements. Two different analyses have been carried out, adopting for the modulus of elasticity the values E = 20 000 and 4000 MPa, corresponding to the hypotheses of uncracked section and totally cracked section, respectively. The beam elements which represent the ring beams are connected to the panel elements which represent the masonry at the nodes of the model. Therefore, the ring beams can deform freely also in the zones corresponding to the piers. To account for the stiffening effect exterted by the piers, the wall is also studied assuming infinite stiffness for the portions of the ring beams corresponding to the piers, through the adoption of rigid arms.

3.2.1 Wall without ring beams

The wall without ring beams has lateral strength 1406 kN, equal to the 38.4% of the total gravity load, which is slightly smaller than that for the 1st seismic category. The deformation localizes to the spandrels, especially for the uppermost storey. The interstorey drifts are reported in table 5, together with the global drift.

Stresses are concentrated to the lee-side of the wall. However, neither crushing or shear failures occur. This is to be ascribed to the good mechanical characteristics of the masonry and to the noticeable width of the piers, especially those at the ends of the wall.

Table 5. Interstorey drift and global drift for the model without ring beams.

Storey	1	2	3	4	5	global
Drift (%)	0.16	0.10	0.10	0.08	1.20	0.31

3.2.2 Wall with elastic ring beams

The introduction of elastic ring beams increases the lateral strength to 2577 kN (70.4% of the total gravity load) for modulus of elasticity of the ring beams E = 20 000 MPa, and to 2402 kN (65.6% of the total gravity load) for E = 4000 MPa. However, in both cases, shear failure of a panel occurs when the base shear reaches the 56.0% of the total gravity load, corresponding to a lateral top displacement of 5 cm.

The wall with elastic ring beams is able to reach higher deformations, thanks to the more uniform distribution of displacements along the height. The deformed shape at the last step shows a strong reduction of the localization at the uppermost storey (Table 6).

Table 6. Interstorey drift and global drift for the model with elastic ring beams.

Storey	1	2	3	4	5	global
Drift (%) (E = 20 000 MPa)	0.77	0.57	0.58	0.61	1.59	0.81
Drift (%) (E = 4000 MPa)	0.67	0.52	0.56	0.52	0.79	0.61

3.2.3 Wall with elastic ring beams provided with rigid arms

The introduction of rigid arms in the portions of ring beams corresponding to the piers does not yield significant increases of the lateral strength. In fact, this reaches 2460 kN (67.2% of the total gravity load) for E = 20 000 MPa and 2402 kN (65.6% of the total gravity load) for E = 4000 MPa. However, in both cases, shear failure of one panel occurs, at H/W = 60.8% in the first case and at H/W = 57.6% in the second case. The lateral top displacement corresponding to the shear failure is nearly 4 cm.

The stiffness of the wall is significantly affected by the rigid arms, and the displacement halve in the average.

Again, the distribution of deformations along the height is rather uniform, with a considerable reduction of the localization at the uppermost storey, compared to the wall without ring beams (Table 7). The deformation of the spandrels is reduced as well, leading to a better global behaviour of the wall.

The stress localization at the lee-side is even more pronounced. The piers at the windward side are almost at rest, while those at the lee-side carry the whole base shear without crushing or shear failures, even though the forces have strong eccentricity.

Table 7. Interstorey drift and global drift for the model with elastic ring beams provided with rigid arms.

Storey	1	2	3	4	5	global
Drift (%) (E = 20 000 MPa)	0.42	0.22	0.26	0.28	0.78	0.39
Drift (%) (E = 4000 MPa)	0.42	0.23	0.28	0.32	0.95	0.43

3.3 Analysis of the whole building

The whole building is analyzed assumed elastic ring beams with two values of the modulus of elasticity (E = 20000 and 4000 MPa).

The modelling is carried out similarly to building “A”.

The gravity loads applied by the floors to the walls have been determined assuming a system of joists spanning in both orthogonal directions. It has been assumed that the walls which are offset with respect to the walls below do not carry gravity loads. The load of the floor corresponding to the entrance hall has been determined assuming an unidirectional system of joists, orthogonal to the direction of the entrance. The total gravity load is W = 33 606 kN.

Table 8. Lateral strength of building “B”.

	Ring beams with E = 20 000 MPa		Ring beams with E = 4000 MPa	
Direction and sign	H (kN)	H/W (%)	H (kN)	H/W (%)
+X	6991	20.8	5915	17.6
–X	5376	16.0	4840	14.4
+Y	5915	17.6	2689	8.0
–Y	5376	16.0	5376	16.0

The main results are the following.

a) *Lateral strength*. It has been determined for the two principal directions X and Y and for the two signs of the seismic loads (+ and –). In fact, the asymmetry of the structure leads to different responses when changing the sign of the seismic forces. The results are reported in table 8, for the two values adopted for the modulus of elasticity of the ring beams. It can be noticed that the lateral strength is rather small.
b) *Floor displacements*. The torsional deformations are small and uniformly spread along the height. This is confirmed by the interstorey drifts and by the shear-displacement diagrams.
c) *Displaced plots of the walls*. For each wall, the deformations are spread along the height.
d) *Forces and stresses*. Element forces and stresses at the last step show a good global behaviour without strong localizations.

In conclusion, the small values of lateral strength are not to be ascribed to a bad plan layout, but rather to a general inadequacy of the resisting elements.

CONCLUSIONS

The seismic response of two masonry buildings representing as many building types of Catania is determined through numerical models.

The vulnerability of the perimetrical walls to overturn is particularly pronounced for those buildings erected in the nineteenth century, which have small thickness of the walls, even at the ground floor, thrusting vaults even at the upper storeys, and are often unprovided with tie-rods. The good connection between the perimetrical walls and the orthogonal walls, given by the so-called "intostoni" (square-cut lavic blocks), prevents overturning under the gravity loads but is inadequate for the destructive earthquake expected in the territory of Catania. A significant improvement could be achieved by eliminating the thrust at the uppermost storeys.

Regarding the in-plane response of masonry, some conclusions can be drawn based on the analyses:

- both the buildings studied have small lateral strength, typically below 20% of the total gravity load;
- the response of isolated walls and, in some cases, of the whole structure is conditioned by deformation localization, especially at the uppermost storey which is less loaded and often characterized by small thickness of masonry;
- the offsets of the horizontal structures of building "A" may lead to poundings or separations; as a consequence, the vaults may collapse;
- the R/C ring beams of building "B" lead to significant increases of the lateral strength; however, it is worth to mention the uncertainties on the actual behaviour of the ring beams, with particular reference to the yielding of reinforcement, cracking of concrete and slidings between ring beams and masonry.

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