The partial collapse of the ‘Palazzo Edilizia’ in Salerno

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Summary

On the night of June 15 2007, a side of one of the most important buildings of Salerno (the so called ‘Palazzo Edilizia’) collapsed 80 years after its construction. Such a ruinous failure did not cause any casualties only thanks to the circumstance that the collapsed side was that of the living rooms.

The aim of the analysis performed by the authors was mainly the search of the causes of such a ruinous collapse. The paper not only deals with the description of the methodology adopted for the analysis, but also discusses the results in order to trace general criteria that could be useful for the reliability assessment of the existing masonry buildings.

Keywords: masonry; buildings; assessment/repair; forensic.

1. The methodology used in the investigation

On June 29 2007 the authors were nominated technical consultants of the investigate Magistrate. Since then, investigations, surveys and tests were carried out to obtain useful information to understand the causes and dynamics of such a ruinous and apparently unexpected collapse. The main steps of the methodology that was used were the following:

- Historical-kinematic reconstruction of the collapse based on questioning, video-photographic documentation and visual inspections of the ruins.
- Acquisition of technical data necessary to create a model using: documentation found in public offices, documentation on the on-going restoration works in the Varese Bar (ground and underground floor), on-site geometrical surveys (also during the works of debris removal), visual sample inspections, laboratory and on-site tests, analysis of video-photographic documentation.
- Re-calculation.
- Identification of design and construction errors and analysis of their influence on the collapse dynamics.

In cases similar to that under study, it is very common that the collapse is disproportionate to the original cause. Hence, in the analysis, a clear distinction between the triggering cause and the causes of the damage evolution has been made.

2. The building construction

The area where Palazzo Edilizia was built was handed over by the Municipality of Salerno to the
construction Company ‘Società Edilizia Salernitana’ in November 1920. No data about the construction were found in the investigations but it is sure that the building was already completed in the second half of the twenties since the first regulation of condominium, found in the investigations, is dated 1928. A project of 1951 relating to the modification of openings of the Varese Bar at the ground floor on the corner between Verdi street and Trieste promenade was found in the investigations. Moreover in 1955 same modifications were made on the opposite façade in order to have a homogeneous ground floor. At the time of the collapse some restoration works were in progress at the Varese Bar (i.e. in the area affected by the collapse). The works did not regard any intervention on the bearing structures of the building.

3. The collapse

3.1 Depositions on the collapse

The building south-west corner, between Verdi street and Trieste promenade (Figure 1), collapsed (see Figure 2) on the night of June 15 2007, at about 3.30 a.m. In that corner, only the slab of the ground floor, the vertical structures of the underground floor and the foundations were not involved in the collapse. In the collapsed corner living rooms and offices were placed. Just before the collapse, some creaks were heard and plaster detachments were seen. This was why some people of the building called the Fire Brigade. A first team of firemen arrived at the building at 1.30 a.m. and started to evacuate the building and to close the near roadways. During these operations the collapse occurred.

3.2 Depositions on the premonitory signs

As told before, in the collapsed corner, some restoration works at the ground and underground floors were in progress in the Varese Bar. During these works, after removing the plaster, some cracks in the masonry walls were detected mainly near the openings on Verdi street. The site supervisor stopped the restoration works and sent, on May 2 2007, to the Condominium Manager a report in which a proposal for retrofitting intervention was included. It is worth mentioning that, according to the Italian law, the owner of the bearing structures of a building is the whole Condominium; this means that works on the bearing structures have to be approved by the Condominium. The Condominium did not answer the report of the site supervisor until June 13 2007 when one of the tenants saw some large cracks around one of the doors of the Varese Bar (Figures 3 and 4). The tenant called the Condominium Manager who immediately went to Palazzo Edilizia for a survey. The morning before the collapse (May 14 2007), the Condominium Manager made another survey with the site supervisor of the Varese Bar, a Consultant Engineer of the Condominium and the tenant who had seen the cracks the day before. The Condominium consultant suggested only to prop up the slab of the first floor, in the area were cracks were detected, in order to reduce the load on the masonry walls of the ground floor. Those props were not installed since that night the corner collapsed.

3.3 Structural pre-existent symptoms

The minutes, from 1993 to 2007, of the Condominium General Assembly were found. These minutes reveal that since 1993 many cracks in the masonry walls were detected an retrofitting and monitoring intervention were planned. Neither detailed reports on the cracks nor projects of the interventions were found in the investigations.

4. The technical characteristics of the building

The vertical bearing structures of Palazzo Edilizia are mainly tuff masonry walls made of irregular-shaped units. In some internal areas of the building, some R.C. columns were found. It is unknown
whether these columns are of the original structure of the building or they were made during some following modifications. All the floors are made with a composite R.C. beam – masonry block structure.

Regarding the collapse, the most important critical aspects of the structural conceptual design are the following:

- very few vertical diaphragms;
- weakening of the collapsed corner due to the presence of large openings;
- lack of tying systems;
- eccentric width reduction of the masonry walls from the lower to the upper floor;
- ground floor masonry piers narrower than those of the other floors (because of the larger openings at the ground floor).

Moreover the following critical aspects relevant to the geometry and the arrangement of the masonry units have been found:

- irregular shape and dimension of the masonry units;
- large thickness of mortar joints;
- absence of transversal masonry units.

4.1 Quality of structural elements and materials

When dealing with masonry behaviour, it is true that tests on masonry panel are preferable to those on units and mortar but it is also true that a large number of tests is, in general, needed in order to get statistically reliable results. This is why in the case under study, taking into account the general irregularity of the masonry, these tests were not performed because of the relevant costs, time and invasiveness on the building. Hence the most important part of the tests made in the investigations, regards those on tuff units. Numerical analyses, performed also in an ‘optimistic’ scenario (see paragraph 5), have shown that the collapse probability was so high to make further tests unnecessary with regards to the individuation of the causes of the collapse.
In this paragraph only the test results on tuff units are reported.

Two different groups of tuff units were selected. The first was composed by 12 units taken in-situ from non-collapsed walls in the area of the collapse; from each unit 2 samples were taken. The second group was composed by 21 units taken from the ruins; from each unit of the second group 4 samples were taken. All the samples were taken according to [1], they were cubic with a side of 70 mm ± 5 mm. Before testing the unit weight was measured. The first set of the tests was performed on the first 12 samples of the first group; they were conditioned at 20 °C and 60% relative humidity for 18 days. On each sample 4 strain gauges were applied before the test. The compressive tests were performed applying the load along the in-situ vertical direction. The second set of the tests was performed on the samples of the second group and on the remaining 12 samples of the first group. In particular, from the four samples taken form each unit of the second group, two were conditioned at 20 °C and 60% relative humidity for 18 days and two were conditioned at 70 °C for 36 hours. This means that, regarding the second group, 42 sample were in ‘natural conditions’ and 42 in ‘dry conditions’. The remaining 12 samples of the first group were conditioned at 20 °C and 60% relative humidity for 18 days.

The aim of the first set of the tests was only to assess the deformation characteristics of the units. For this purpose the load velocity was sensibly lower than that given in [1] and applied to the second set of the tests. This means that the compressive strength obtained in the first set of the tests was not used in the re-calculations. Comparing the results of the first and second set of the tests it was possible to evaluate the increase of compressive strength due to the slow application of the testing load. The aim of the second set of the tests was to have statistically reliable results and to evaluate the influence of water content on the compressive strength. It was noted that the ‘natural’ water content produces a 5% increase of the average unit weight and a 21% reduction of the average compressive strength. From the second set of the tests it was noted that the compressive strength of masonry was not influenced by the collapse. According to what previously highlighted, only data (unit weight and compressive strength) of the samples of the second group in ‘natural’ conditions were used for the re-calculations (Table 1).

To assess the masonry compressive strength the approach proposed by [2] has been used. This procedure starts from the evaluation of the normalised mean compressive strength of a masonry unit \( f_b \). [2] gives two different unit categories that, regarding natural stone units, are defined in [3]. Hence, according to the procedure given by [1], in the case under study \( f_b = 2,94 \) MPa for category I and \( f_b = 3,24 \) MPa for Category II have been obtained. The second step to assess the masonry compressive strength is to evaluate mortar compressive strength. In the case under study the mortar seemed to be, at naked eye, of very bad quality. It was not possible to take mortar samples because the mortar immediately tended to crumble even at the simple contact with the hands. This is why an ‘optimistic’ value of 2,50 MPa for the compressive strength of masonry mortar \( f_m \) has been assumed. This value is equal to that given in the Italian National Annex of [2]. In particular the value of pozzolanic mortar has been assumed, since that kind of mortar was widespread in Salerno during the construction period of the building. The characteristic compressive strength of masonry \( f_k \) has been evaluated according to [2] and it is equal to 1,26 MPa and 1,35 MPa for category I and II respectively. It is worth noting that the evaluated masonry compressive strength of the masonry is ‘optimistic’ not only because of the assumed strength of mortar but also of the hypothesis, not fulfilled in the case under study, that the masonry walls of Palazzo Edilizia were built accordingly to what indicated in [2].

| Table 1: Results of the tests on the samples of the second group in ‘natural’ conditions |
|---------------------------------|-----------------|-----------------|
|                                 | Unit weight     | Compressive strength |
| Mean value                      | 11,70 kN/m³     | 4,28 MPa         |
| Standard deviation              | 0,64 kN/m³      | 1,25 MPa         |
| Coefficient of variation        | 0,05            | 0,29             |

According to what previously highlighted.
5. Re-calculations

Re-calculations were performed with reference to the element that, according also to depositions, triggered the collapse: the second masonry pier from left on the Verdi street façade (figure 5). Numerical analyses were performed according to the approaches proposed by [2]. In paragraph 5.1 a simplified approach has been used; in this approach the mean stress on the transversal horizontal section of the masonry pier has been evaluated. In paragraphs 5.2 and 5.3 the results of linear and nonlinear finite element analyses are reported. The aim of different analyses is both to catch detailed aspects of structural behaviour and to highlight that each improvement in the analysis implies a reduction of the safety factors. Re-calculations in paragraphs 5.1, 5.2, 5.3 are performed according to the limit state design method while in paragraph 5.4 a probabilistic approach is showed.

5.1 Simplified analysis

In this paragraph the structural verifications for axial load of the masonry pier showed in figure 5 according to the limit state design method of [2] are reported. The following assumptions have been made:

- only the part of the pier from the ground floor to the roof has been considered;
- actions on the wall have been evaluated for ‘influence areas’;
- axial load is centred in the longitudinal direction (parallel to Verdi street);
- at the bottom of each floor the axial load is centred also in the transversal direction.

It is worth noting that last assumption is extremely ‘optimistic’ since it assumes that floors have the capacity to catch the thrusts generated by the load deviation in the transversal direction. According to the authors, taking into account that, in the case under study, the longitudinal reinforcement of the floor slabs is not often sufficiently anchored in the walls, this assumption seems to be unreliable.

Only gravitational loads have been considered. Characteristic values of permanent loads have been assumed as the mean values (according to [4]) of those evaluated by the tests; only in very few cases they have been taken from the technical standards. Concerning variable loads, only the part relevant to sustained loads ([5]) has been assumed in the analysis, in order to have a more realistic scenario with reference to the collapse diagnosis.

This means that the characteristic value of variable load is the 5% superior fractile of a random ‘normal’ variable with mean value equal to 0.54 kPa and a standard deviation of 0.19 kPa ([6]). On the roof and on the balconies a nil value of the variable loads has been assumed.

Structural dimensions were measured on-site or, thanks to the similarity of the facades, obtained from the integer part of the building.

The design compressive strength of masonry $f_d$ has been evaluated according to [2] applying the partial factors $\gamma_o$ given by the Italian National Annex of the Eurocode (2.7 and 3.0 for category I and II respectively). According to paragraph 4.1 the following values have been obtained: $f_d = 0.47$ MPa and $f_d = 0.45$ MPa for category I and II respectively. At these values the ‘capacity reduction factor’ ([2]) that takes account of load eccentricity has been applied.
Re-calculations have been performed in two different scenarios: floor load transversally centred on the underlying wall, floor load transversal eccentricity equal to 10 cm from the inner face of the underlying wall. Moreover in order to have an ‘optimistic’ evaluation of the safety factor, the maximum strength between category I and II has been assumed. The results show that structural verifications are not fulfilled at the bottom and at the top of walls of the ground, first, second floor. The minimum value of the safety factor is 0.56 at the bottom of the wall of the ground floor.

Even if coming from a simplified analysis, the very low safety factor clearly shows the structural risk of that wall before the collapse.

5.2 Linear finite element analysis

The most important (and extremely ‘optimistic’) assumption of the simplified analysis is to consider only the mean stress value in the transversal horizontal section of the wall pier. Probably the triggering cause of the first failure was the stress concentration near openings. To catch numerically this aspect (even in an ‘optimistic’ scenario) a linear finite element analysis of the wall pier has been performed (i.e. ‘Model 7’) by using the finite element software ‘Midas/Gen 7.21’ (figure 6). It has been assumed a homogeneous linear-elastic isotropic material for masonry. In the model, the bottom of the ground floor wall is vertically and horizontally fixed, at each floor the wall is transversally fixed (‘optimistic’ assumption according to what mentioned in paragraph 5.1) and the borders of the wall are longitudinally fixed. The same actions of the simplified analysis have been applied. The floor load has been considered transversally centred on the underlying wall. A secant modulus of elasticity $E = 27$ GPa (corresponding to a concrete with compressive cylinder strength $f_{ck} = 12$ MPa according to [7]) and a Poisson’s ratio $\nu = 0.1$ have been assumed for R.C. lintels and ring beams. According to [2] $E = 1.35$ GPa (masonry units of category II) and $\nu = 0.25$ have been assumed for masonry.

The results of the analysis show that:

- the maximum compressive stress at the bottom of the ground floor wall is 1247 kPa with a safety factor equal to 0.37 (considering that, according to paragraph 5.1, $f_d = 0.47$ MPa);
- the maximum compressive stress of the wall is at the intersection with the lintel of the opening at the ground floor and it is equal to 1771 kPa with a safety factor equal to 0.26.

The linear finite element analysis has permitted to evaluate the stress peaks due to the deviations of loads because of the architectonical characteristics of the wall (figure 7). These peaks are not negligible and they determine an important reduction (from 0.56 to 0.26) of the global safety factor, increasing, from a numerical point of view, the structural risk of that wall before the collapse.

5.3 Nonlinear finite element analysis

The linear elastic analysis, showed in the previous paragraph, does not consider the capacity of the masonry structure to redistribute the internal stresses; hence it could underestimate the safety factor.

To understand this aspect a nonlinear finite element analysis of the wall pier has been performed (i.e. ‘Model 9’) by using the finite element software ‘Midas/Gen 7.21’. Model 9 is equal to Model 7. The only difference is that, for masonry, a nonlinear Mohr-Coulomb constitutive law has been assumed. The design cohesion $c_d$ and the internal friction angle $\phi_i$ have been assumed equal to 0.018 MPa and 8.42° respectively, according to paragraph 3.6.2 of [2]. It is worth noting that the assumed parameters for masonry lead to a significant decrease of masonry compressive strength when compared to that evaluated in paragraph 5.1. Even if given by [2], the assumed parameters are given in that code only to evaluate masonry resistance to shear loading and not to evaluate axial resistance. It follows that this kind of analysis has validity only to highlight the damage propagation mode.
The results of the nonlinear analysis (figures 8 and 9) show that the first yielding points are at the intersection of the wall with the lintel of the opening at the ground floor (3% of the ultimate limit state design load). Then the yielding points immediately spread all over the ground floor panel (ultimate load equal to the 5% of the ultimate limit state design load).

5.4 Probabilistic analysis

In this paragraph, the probability of failure of the masonry pier under study is evaluated.

As mentioned in paragraph 4.1, to evaluate masonry compressive strength the approach of [2] has been adopted using the test results of the 2nd set of the 2nd group and assuming a compressive strength of masonry mortar $f_m = 2.50 \text{ MPa}$. According to [4] a normal density function for masonry compressive strength has been assumed and from the tests the mean value and standard deviation have been calculated (equal to 1.336 MPa and 0.272 MPa respectively). Only gravitational actions have been considered. For permanent loads, with the only exception of the unit weight of the tuff blocks, a normal density function has been assumed ([4]). The mean value measured on-site have been adopted; only in very few cases the values given by the technical standards have been assumed. Standard deviations have been evaluated according to [4]. The statistical distribution of the unit weight of the tuff blocks has been experimentally evaluated as shown in paragraph 4.1 assuming a normal density function. According to what mentioned in paragraph 5.1, a normal density function with mean value 0.54 kPa and standard deviation 0.19 kPa has been assumed for the variable loads. On the roof and on the balconies a nil value of the variable loads has been assumed.

In the same assumptions and with the same approach of paragraph 5.1 it has been possible to evaluate the probability of failure at the top and at the bottom of the wall of each floor.

According to [4], the probability of failure at the ultimate limit state of strength should be less that 0.007% for a design service life of 50 years. In the case under study this limit is ‘optimistic’ since
the life of the building at the time of the collapse was about 80 years. The results of the probabilistic re-calculations show that this limit is not fulfilled at the walls of the ground, first, second floor. The lowest value of the probability of failure is equal to 0.41% at the bottom of the ground floor wall.

These results are perfectly coherent with those of paragraph 5.1.

Comparing this simplified probabilistic approach with the simplified analysis showed in paragraph 5.1, it is possible to correlate the safety factor with the probability of failure. By using this correlation it is possible to evaluate that the probability of failure correspondent to the safety factor calculated by linear analysis (0.26) is equal to 48%.

Hence, according to what obtained by the re-calculations, it is possible to conclude that, even in an ‘optimistic’ scenario, the probability of failure of the masonry pier under study in the design service life (50 years) was surely more than 48%.

6. Concluding remarks

According to what mentioned in paragraph 5, the main cause of the collapse was that the demand of axial resistance due to gravitational loads was sensibly higher than the capacity of the second masonry pier from left on the Verdi street façade.

The causes of the propagation of the failure were the following:

- very low capacity of the masonry pier to redistribute the internal stresses;
- lack of building robustness both for general conception and for detailing.

Finally it is worth adding that, in a scenario of a very high probability of failure, it is possible that the removal of the mortar plaster on the ground floor wall during the restoration works in the Varese bar was the triggering cause of the collapse.

7. References