A meaningful case of structural assessment following the collapse of a historical masonry building

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Abstract

The partial collapse of the so-called 'Palazzo Edilizia', which occurred in Salerno (Italy) on the night of June 15, 2007, is a meaningful case since it has revealed that 'hidden' structural defects can be the cause of ruinous and apparently unpredictable failures. The paper presents the investigations carried out by the technical consultants of the Criminal Investigate Magistrate. The purpose of the investigations was not only to determine the causes of such unexpected failure but also to give the Investigate Magistrate sufficient data in order to identify the parties responsible.
Keywords: masonry, buildings, failures, assessment, forensic engineering.

1 Introduction

The technical consultants of the Investigate Magistrate have followed the methodology proposed by Palmisano *et al.* [1] that draws inspiration from that proposed by Fib [2].

From a general point of view, the procedure adopted in investigations following collapses is similar to that used in structural assessment of existing constructions. This is why this article aims also to trace a path that can be followed for the reliability assessment of existing masonry buildings.

Regarding the case study, as it will be described in the following, visual inspections and depositions have made immediately clear that the primary cause of the collapse was the huge intrinsic structural vulnerability of the building. This is why, in each step of the investigations, the technical consultants of the Investigate Magistrate have started from ordinary procedures and then they have decided, if necessary, to adopt more advanced techniques. Moreover, when it has

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not been possible to have reliable evaluation of the parameters necessary for the analyses, ‘optimistic’ values have been assumed. The aim of such approach is to show that the collapsed side, even in an ‘optimistic’ scenario, had a safety factor significantly lower than the limit given by technical standards.

In order to quantify the safety factor of the collapsed side, the technical consultants of the Investigate Magistrate have decided to make reference to the Eurocodes that are both the technical standards adopted by the members of the European Union and one of the most advanced codes of practice in the world.

2 Background

In the investigations no data about the construction were found but, from historical documents, it is sure that the building (fig. 1) was built between 1920 and 1928. A project of 1951 relating to the enlargement of openings of the Varese Bar at the ground floor on the corner between Verdi Street and Trieste Promenade was found in the investigations. In 1955, the 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 ground floor in 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.

Figure 1: Picture of the north façade.

3 Collapse

The building south-west corner, between Verdi Street and Trieste Promenade, collapsed (fig. 2) on the night of June 15, 2007, at about 3.30 a.m. On 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. Just before the collapse, some creaks were heard and plaster detachments were seen.

The building collapsed during evacuation operations. Such a ruinous failure did not cause any casualties only because the collapsed side was that of the living rooms.

As previously mentioned, 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 that included a proposal for retrofitting intervention. It is worth mentioning that, according to 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 two openings of the Varese Bar on Verdi Street (fig. 3). The tenant called the Condominium Manager who immediately went to Palazzo Edilizia for a survey. The morning before the collapse (June 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 cracked masonry pier of the ground floor. Those props have not been installed since that night the corner collapsed.

Figure 2: Picture of the south-west corner (between Verdi Street and Trieste Promenade) before (left) and after (right) the collapse.

4 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 had been detected and the Condominium General Assembly planned retrofitting and monitoring intervention. Neither detailed reports on the cracks nor projects of the interventions were found in the investigations. Moreover, since the first survey (June 29, 2007), the technical consultants of the Investigate Magistrate observed many symptoms of compressive ‘over-stressing’, such as vertical cracks, also in areas very far from the collapsed corner and, hence, not attributable to the collapse.

Figure 3: Cracks around two openings of the Varese Bar (Verdi Street, June 13, 2007).

5 Structural description

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 [3], 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; fig. 4).

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, and absence of transversal bondstones.

The technical consultants of the Investigate Magistrate have used the Load Path Method to analyse, from a conceptual point of view, all the above-mentioned critical aspects. In fact, born as a method to design strut-and-tie models in reinforced concrete structures, the Load Path Method was introduced by Schlaich *et al*. [4], developed mainly by Palmisano and Vitone [5–9] and extended to masonry structures mainly by Palmisano [10–20] and their colleagues. For the sake of brevity, the analyses performed by using the Load Path Method are not reported in this paper.

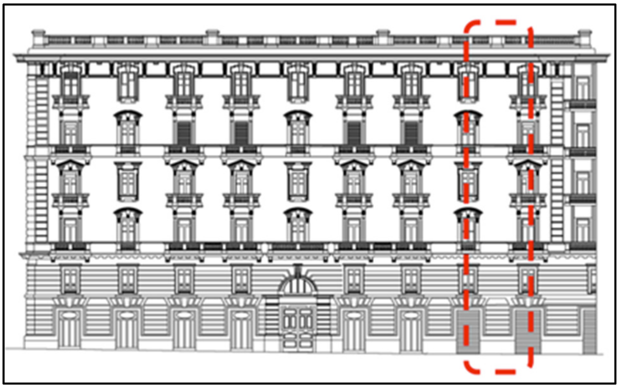


Figure 4: CAD reconstruction of the Verdi Street façade with the indication of the masonry pier of the numerical analyses.

5.1 Quality of the masonry

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 account of 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, 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. Before testing, the unit weight was measured. Compressive tests were performed on samples conditioned either in natural or in dry conditions. Strain gauges were applied on some samples in order to assess the deformation characteristics.

To assess the masonry compressive strength the approach proposed by EN 1996-1-1 [21] has been used. This procedure starts from the evaluation of the normalised mean compressive strength of a masonry unit f_b. EN 1996-1-1 [21] gives two different unit categories. 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 EN 1996-1-1 [21]. The characteristic compressive strength of masonry f_k has been evaluated according to EN 1996-1-1 [21] 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 because of the hypothesis, not fulfilled in the case under study, that the masonry walls of Palazzo Edilizia were built accordingly to what indicated in EN 1996-1-1 [21].

6 Numerical analyses

Numerical analyses were performed with reference to the element that, according also to depositions, triggered the collapse: the second masonry pier from right on the Verdi Street façade (fig. 4). Numerical analyses were performed according to the approaches proposed by EN 1996-1-1 [21]. As discussed in the following paragraphs, four different approaches have been used: (a) simplified analysis, (b) linear finite element analysis, (c) nonlinear finite element analysis, (d) simplified probabilistic analysis.

Numerical analyses according to approaches (a), (b), (c) were performed according to the limit state design method and the aim of these 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. The simplified probabilistic analysis was performed in order to immediately understand and quantify the risk level of the collapsed side.

6.1 Simplified analysis

This paragraph reports the structural verifications for axial load of the masonry pier showed in fig. 4 according to the limit state design method of EN 1996-1-1 [21].

In this approach the mean stress on the transversal horizontal section of the masonry pier has been evaluated. 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';

- the 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 the last assumption implies that floors have the capacity to catch the thrusts generated by load deviation in the transversal direction. Taking into account that, in the case under study, the longitudinal reinforcement of the floor slabs was not often sufficiently anchored in the walls, this assumption seems to be extremely 'optimistic'.

The design compressive strength of masonry f_d has been evaluated, according to EN 1996-1-1 [21], applying the partial factors γ_m given by the Italian National Annex of the Eurocode (2.7 and 3.0 for category I and II respectively). By using the characteristic compressive strength of masonry previously calculated, the following values have been obtained: f_d = 0.47 MPa and f_d = 0.45 MPa for categories I and II respectively. At these values the 'capacity reduction factor' [21] that takes account of load eccentricity has been applied.

Numerical analyses 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. In order to have an 'optimistic' evaluation of the safety factor, the maximum strength between categories 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.

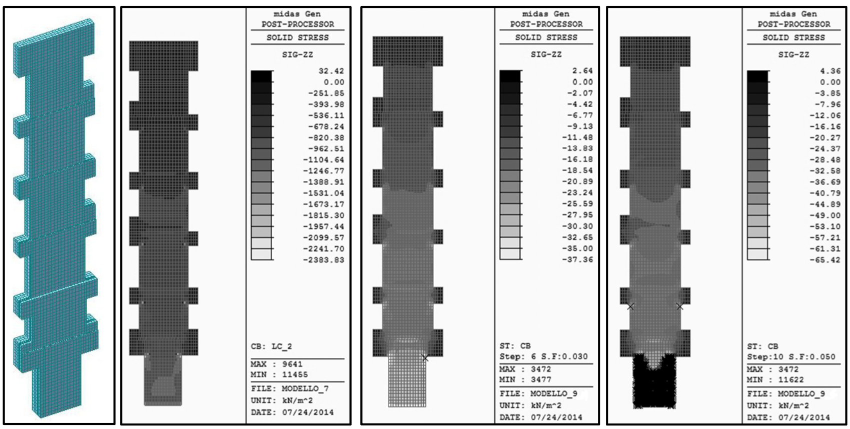
6.2 Linear finite element analysis

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' (fig. 5(a)). The domain has been subdivided into a regular mesh using the finite element 'solid' (eight or six nodes). 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) and the vertical 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 EN 1992-1-1 [22]) and a Poisson's ratio ν = 0.1 have been assumed for R.C. lintels and ring beams. According to EN 1996-1-1 [21], E = 1.35 GPa and ν = 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 what mentioned in the previous paragraph, 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 (fig. 5(b)). These peaks are not negligible and they determine an important reduction (from 0.56 to 0.26) of the global safety factor evaluated by the simplified analysis, increasing, from a numerical point of view, the structural risk of that wall before the collapse.



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

Figure 5: Models of the FEAs: axonometric view of Models 7 and 9 (a), axial vertical stresses of Model 7 (b), yielding points of Model 9 at 3% of ULS load (c), yielding points of Model 9 at 5% of ULS load (d).

6.3 Nonlinear finite element analysis

The linear elastic analysis, showed in the previous paragraph, does not consider the capacity of the masonry to redistribute 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 ϕ_d have been assumed equal to 0.018 MPa and 8.42° respectively, according to paragraph 3.6.2 of EN 1996-1-1

[21]. It is worth noting that the assumed parameters for masonry lead to a significant decrease of masonry compressive strength when compared to that previously evaluated. Even if given by EN 1996-1-1 [21], the assumed parameters are proposed 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 and not the safety factor.

The results of the nonlinear analysis (figs 5(c) and (d)) 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). This means that the masonry under study has no capacity to redistribute the internal stresses.

6.4 Probabilistic analysis

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

As previously mentioned, to evaluate masonry compressive strength, the approach of EN 1996-1-1 [21] has been adopted using the test results and assuming a compressive strength of masonry mortar f_m = 2.50 MPa. According to EN 1990 [23], 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 loads have been considered. For permanent and variable loads a normal density function has been assumed [23]. The mean values 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 EN 1990 [23]. The statistical distribution of the unit weight of the tuff blocks has been experimentally evaluated assuming a normal density function. On the roof and on the balconies a nil value of the variable loads has been assumed.

Along the same assumptions and with the same approach of the simplified analysis, 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 EN 1990 [23], the probability of failure at the ultimate limit state of strength should be less than 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 analysis 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 obtained in the simplified analysis.

Comparing this simplified probabilistic approach with the simplified analysis, 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 analyses, 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%.

7 Causes of the collapse

According to the results of the investigations, it is possible to give the following answers:

- The whole building at the moment of the collapse was characterised by a huge intrinsic vulnerability to gravitational loads with many symptoms of compressive over-stressing.
- The collapse was originated in the masonry pier in fig. 4.
- At the moment of the collapse the demand of axial resistance due to gravitational loads was sensibly higher than the capacity of the masonry pier in fig. 4. This is the primary cause of the collapse.
- The causes of the propagation of the failure were the (i) very low capacity of the masonry pier to redistribute the internal stresses and (ii) the lack of building robustness both for general conception and for detailing [3].
- 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. In fact, it is possible, that before the collapse, the masonry pier in fig. 4, in a 'desperate search for help' was using every element (also a non-structural one such as the mortar plaster) as a temporary support.

Regarding the responsibilities, the following aspects have to be highlighted.

The inhabitants of Palazzo Edilizia knew that the building had important structural vulnerabilities (see structural pre-existent symptoms). This is why the Condominium General Assembly should have had performed a general structural assessment of the whole building before authorizing any kind of intervention (also those not regarding structural elements). Moreover the Assembly did not respond promptly to the proposal for retrofitting intervention of the site supervisor of Varese Bar and this behaviour give time to the mechanism, probably activated by the plaster removal, to evolve into a disastrous collapse.

8 Conclusions

Recent Italian collapses, such as that of Palazzo Edilizia, have revealed that hidden structural defects can be the cause of ruinous and apparently unpredictable failures. Moreover the most recent Italian experience on vulnerability assessment has been showing that these cannot be treated as extraordinary cases. The wide-spreading of such hidden weaknesses within existing buildings is, first of all, the consequence of the less conservative and hazardous design/construction approach which was the practice during the economic growth period, in order to save time and minimise construction costs.

Secondly, in many cases, enlargement interventions to existing buildings have been made without complying with the regulations in force.

It follows that there is the necessity of defining national guidelines in order to give a common direction to each local Italian government to activate a procedure for structural vulnerability assessment at the territorial scale.

References

[1] Palmisano, F., Vitone, A., Vitone, C. & Vitone, V., Collapse of the Giotto Avenue Building in Foggia. *Structural Engineering International*, 17(2), pp. 166-171, 2007.

[2] Fédération Internationale Du Béton (Fib), *Bulletin 17. Management, maintenance and strengthening of concrete structures*, Sprint-Digital-Druck: Stuttgart, 2002.

[3] Palmisano, F., Mitigation of progressive collapse by the activation of the elasto-plastic catenary behaviour of R.C. slab structures. *The Open Construction & Building Technology Journal*, 8, pp. 122-131, 2014.

[4] Schlaich, J., Schafer, K. & Jennewein, M., Toward A Consistent Design of Structural Concrete", *PCI Journal*, 32 (3), 1987, 74-150.

[5] Palmisano, F., Form and structure in the harmonious complexity of the building process: from conceptual design to detailing in some reinforced concrete works. *Structural Concrete*, 6 (3), pp. 122-130, 2005.

[6] Palmisano, F., Vitone, A. & Vitone, C., A first approach to optimum design of cable supported bridges using load path method. *Structural Engineering International*, 18(4), pp. 412-420, 2008.

[7] Palmisano, F., Alicino, G. & Vitone, A., Nonlinear analysis of R.C. discontinuity regions by using the bi-directional evolutionary structural optimization method. *Proc. of the OPT-I, An International Conference on Engineering and Applied Sciences Optimization*, M.G. Karlaftis, N.D. Lagaros, M. Papadrakakis eds., pp. 749-758, 2014.

[8] Mezzina, M., Palmisano, F. & Raffaele, D., The design of r.c. bridge deck subjected to horizontal actions by strut-and-tie model. *Bridge Maintenance, Safety, Management and Life-Cycle Optimization* (proc. of the 5th International Conference on Bridge Maintenance, Safety and Management – IABMAS 2010, Philadelphia, Pennsylvania, USA, July 11-15 2010), edited by D.M. Frangopol, R. Sause, C.S. Jusko, Taylor & Francis Group: London, UK, pp. 2390-2397, 2010.

[9] Mezzina, M., Palmisano, F. & Raffaele, D., Designing simply supported R.C. bridge decks subjected to in-plane actions: Strut-and-Tie Model approach. *Journal of Earthquake Engineering*, 16(4), pp. 496-514, 2010.

[10] Palmisano, F., Vitone, A. & Vitone, C., Load path method in the interpretation of the masonry vault behaviour. *Structural Studies, Repairs and Maintenance of Heritage Architecture IX (Advances in Architecture Series 20)*, C.A. Brebbia, A. Torpiano eds., WIT-press: Southampton, UK, pp. 155-167, 2005.

[11] Palmisano, F., Interpretation of the behaviour of masonry arches and domes by simple models. *Structural Studies, Repairs and Maintenance of Heritage*

Architecture XIII (WIT Transactions on the Built Environment 131), C.A. Brebbia ed., WITpress: Southampton, UK, pp. 233-244, 2013.

[12] Palmisano, F., Assessment of masonry arches and domes by simple models. *International Journal of Structural Engineering*, 5(1), pp. 63-75, 2014.

[13] Palmisano, F., Rapid diagnosis of crack patterns of masonry buildings subjected to landslide-induced settlements by using the Load Path Method, *International Journal of Architectural Heritage*, published on-line on the 31st September 2014, DOI 10.1080/15583058.2014.996922, in print.

[14] Palmisano, F. & Elia, A., Masonry buildings subjected to foundation settlements due to landslide: a preliminary study on the interpretation of structural behaviour using load path method. *Structural Studies, Repairs and Maintenance of Heritage Architecture XI (WIT Transactions on the Built Environment 109)*, C.A. Brebbia ed., WITpress: Southampton, UK, pp. 141-150, 2009.

[15] Palmisano, F. & Totaro, A., Load path method in the interpretation of dome behaviour. *Structures & Architecture (proc. of the 1st International Conference on Structures & Architecture, Guimarães, Portugal, 21-23 July 2010)*, P.J.S. Cruz ed., CRC Press/Balkema: Leiden, The Netherlands, pp. 1826-1833, 2010.

[16] Palmisano, F. & Elia, A., Analysis of the structural behaviour of masonry buildings subjected to landslide by using the Load Path Method, *International Journal of Earth Sciences and Engineering*, 6(1), pp. 39-49, 2013.

[17] Palmisano, F. & Elia, A., Structural behaviour of masonry buildings subjected to landslide: load path method approach. *Structures and Architecture: New concepts, applications and challenges (proc. of the 2nd International Conference on Structures & Architecture, Guimarães, Portugal, 24-26 July 2013)*, P.J.S. Cruz ed., CRC Press/Balkema: Leiden, The Netherlands, pp. 888-895, 2013.

[18] Palmisano, F. & Elia A., Behaviour of masonry buildings subjected to landslide-induced settlements. *International Journal of Structural Engineering*, 5(2), pp. 93-114, 2014.

[19] Palmisano, F. & Elia A., Assessment of masonry buildings subjected to landslide by using the Load Path Method. *International Journal of Civil Engineering*, 12(2), pp. 312-330, 2014.

[20] Palmisano, F. & Elia A., Shape optimization of strut-and-tie models in masonry buildings subjected to landslide-induced settlements. *Engineering Structures*, 84, pp. 223-232, 2015.

[21] European Committee for Standardization (CEN), *EN 1996-1-1:2005 Eurocode 6 – Design of masonry structures – Part 1-1: General rules for reinforced and unreinforced masonry structures*, 2005.

[22] European Committee for Standardization (CEN), *EN 1992-1-1:2004 Eurocode 2. Part. 1-1 Design of concrete structures. General rules and rules for buildings*, 2004.

[23] European Committee for Standardization (CEN), *EN 1990:2005 Eurocode. Basis of structural design,* 2005.