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Inspections and NDT for the characterization of historical buildings after seismic events: 2012 Emilia earthquake

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ABSTRACT: Following the recent earthquakes that occurred in Emila and Mantua areas (Italy) there was the necessity to inspect the damages, to characterize materials and structural elements and to evaluate the seismic behavior with reference both to the damaged and to the strengthened structures. A wide range of NDT methods were then applied to characterize masonry, timber and R.C. structures and also new test methods (for the measurement of masonry shear characteristics through flat-jack) were experimented, calibrated and applied during in situ operations. It is relevant that there was the possibility to perform the tests on buildings made with similar construction techniques, thus obtaining interesting comparative results.

1 THE EARTHQUAKE IN THE ITALIAN REGULATORY FRAMEWORK

1.1 *Overview*

The assessment of structural safety of existing buildings is crucial in Italy, both because of the high vulnerability of the main part of the heritage, especially due to seismic forces, and because of the historical, architectural and artistic importance of these buildings. This fact was particularly highlighted following the many seismic events that have occurred in Italy over recent decades: Umbria (1997), Molise (2002), L'Aquila (2009) and Emilia (2012) earthquakes.

The last two earthquakes occurred after the update of a national regulatory framework in the structural field, which was organic and well defined, consisting of the 2008 Italian seismic code (D.M. 14/01/2008) currently under review, and the 2011 guidelines concerning the seismic risk of the cultural heritage (D.P.C.M. 09/02/2011). This regulatory framework tracks a clear and well identified path to face the seismic assessment of existing buildings; it can also be useful in assessing their static safety.

The evaluation of structural safety and structural performances of an existing building requires reducing its complexity to a model that can be described from the numerical point of view. Although the experimental and numerical analysis are a powerful tool to aid empirical observation making it more objective, it remains a shared opinion that for existing buildings the current analysis tools, even the most advanced, often prove to be inadequate in explaining the stability of complex structures which have demonstrated it over time.

It is then necessary to pay special attention to understanding the behaviour of the structure comparing the in situ situation to the ever-present need of providing an intuitive justification of the results obtained by numerical modelling; otherwise they may be the mere result of arithmetic calculations, not fully representative of the actual physical behaviour of the building.

To reach full knowledge of an existing building it is essential:

- to reconstruct the fabrication process and the subsequent changes undergone by the building over time, as well as the events that it has experienced;
- to obtain adequately detailed geometric-structural information, specifically identifying the structural concept, the construction details and the cracking and deformation patterns;
- to obtain full knowledge of material characteristics and of the level of degradation based on data available on visual inspections and in situ experimental investigations.

The 2011 guidelines particularly underline the fact that it is not always possible to reach complete structural knowledge of a building. So, a path of knowledge is tracked that can be developed with different levels of detail, depending on the accuracy of the preliminary analysis and that is developed over the following steps:

- survey of the building, cracks and deformation patterns;
- interpretation of the historical evolution of the construction;
- structural identification of the building and of its construction details;
- evaluation of mechanical properties of materials and their deterioration;
- evaluation of soil-foundation relationship.

The knowledge level reached on the basis of this pathway then determines the choice of the confidence factors (F.C.) to be applied in the safety assessment.

1.2 *Interpretation of the historical evolution*

It is appropriate to emphasize the fact that the knowledge of the building resulting from the survey and diagnosis cannot be considered complete without a proper understanding of the historical evolution of the construction. The historical-critical analysis is based on the geometrical survey of the building, on its detailed observation, on the structural, historical and stylistic relationship between its parts and on the analysis of available historical documents. This means that a proper understanding of the past occurrences that, to some extent, can be engraved on the actual state of conservation of the building can be reached.

This allows for the identification of the sequence of construction phases which have determined the structural design of the building, but also the inherent weaknesses that cannot always be detected with diagnostic methods. Therefore, it constitutes an essential support to the structural identification that must then be deepened either by inspection or by experimental methods.

1.3 *Inspection and preliminary analysis*

At the same time, it is necessary to carry out a detailed inspection of the building, which will be reported through specific standardized survey forms that have been developed by the author under recent specific research projects.

Based on this inspection, structural damage is located, identified and photographed and the most significant damage mechanisms triggered by the earthquake are identified. The analysis is carried out in accordance with the specifications listed in the 2011 guidelines, and it aims to reach the "1st Evaluation Level" (LV1).

The detailed and analytical examination of damage mechanisms is essential to identify the macro-elements which activated local failures and kinematic mechanisms with repeatable and predictable damage phenomena. This will lead to the identification of the actual damage level and then to the evaluation of the necessity of seismic repairs or retrofitting.

Concerning the Emilia earthquake, this filing was particularly useful also from the administrative point of view as the laws for the repair of damage caused by the 2012 earthquake (D.L. 06/06/2012) requires the assessment of the level of damage by detailed analysis of the actual situation. The afore-mentioned survey forms have proven to be an effective operational tool to fulfil this request.

The inspection also allows for the definition of the necessity to provide temporary works and the subsequent investigation plan, assessing the operational and technical feasibility, with particular reference to the operational safety, an essential aspect concerning buildings that are damaged by seismic events.

2 ANALYSIS FOR THE CHARACTERIZATION OF MATERIALS

2.1 *Local context*

The professional investigations carried out following the 2012 earthquake have allowed analysing several buildings located in the provinces of Modena and Reggio Emilia. With regards only to historical buildings, we can refer to religious sites like S. Biagio church at Carpi (MO) and S. Maria Assunta church at Fabbrico (MO), civil buildings like Finale Emilia town hall (MO), a private building at Mirandola (MO), a vinegar factory at Novi di Modena, Concordia oratory (MO), Bonasi-Benucci manor at Stuffione (MO) and La Bertusa manor at Novi di Modena.

These buildings feature differing typologies, but all have a significant uniformity of materials and construction techniques. Therefore, it is possible to outline a comprehensive approach to structural diagnosis, in accordance with what has already been developed in other case histories (Armanasco & Foppoli 2014).

In order to reach the full understanding of the structural behaviour of the buildings it is necessary to carefully assess the characteristics of the building elements listed below.

Soil: geometry and materials of foundation works, soil-construction behaviour.

Masonry: geometry, stratigraphy, materials and their characteristics, conservation status, mutual interlocking walls, internal wooden or metal tie elements.

Masonry vaults: in this case too geometry, stratigraphy, materials and their characteristics, conservation status, support methods on bearing structures, efficiency of tie rods.

Wooden structures (ceilings and roofs): geometry, mechanical characteristics, conservation status.

In the following section, the steps necessary to reach the characterization of the above listed elements are described.

2.2 *Masonry*

The bearing structures of historical buildings in the Emila region are made with solid brick masonry; their mortar joints generally exceed 1 cm of thickness and the materials used are locally sourced.

The mortar is made using lime, usually with low binder/aggregates ratio, and fairly unsifted sand or mixed with sandy clay that makes the quality of the walls very poor.

Bricks are typically $5.5 \times 11.5 \times 24.5$ cm and have very variable characteristics due to the materials used for their production. Red-brown bricks usually are produced with heavy ferrous clays and are more porous and therefore not suitable for use on damp walls. Bricks made from siliceous clays, on the other hand, are more compact and therefore more suitable for use in contexts of high humidity.

Masonry thickness usually ranges from 2 to 3 bricks dimension, that means from 24 to 38 cm and attention to masonry texture is usually quite close (Fig. 1): it

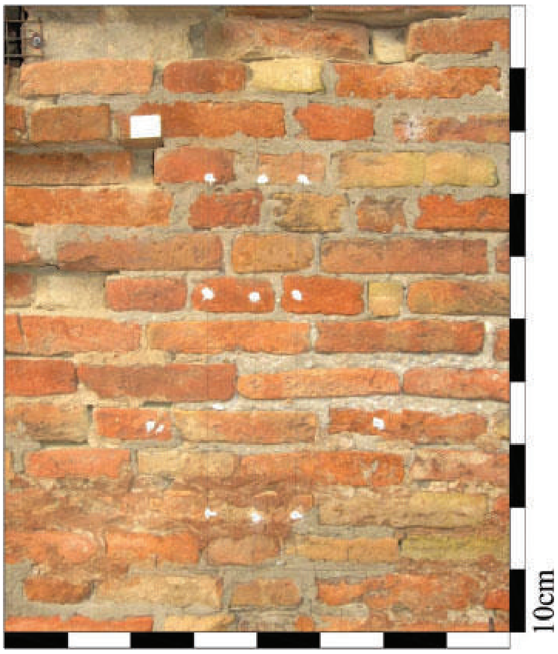

Figure 1. Texture of traditional brick masonry of Emilia region.

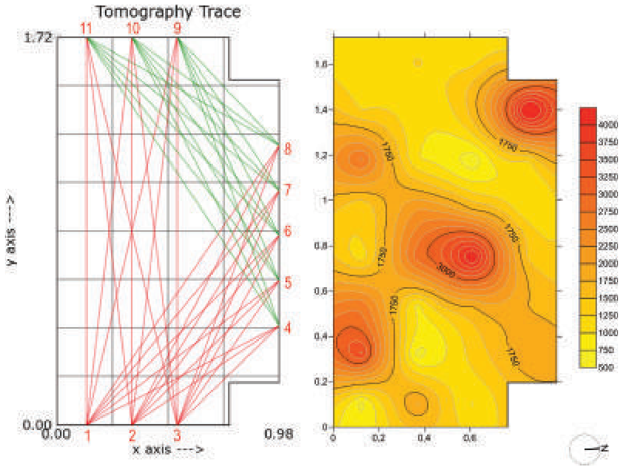

Figure 2. Sonic tomography of the horizontal section of a masonry pillar.

presents good transverse connections, except in the case in which the walls have been built by increasing the thickness in several subsequent stages. Less care is found in clamping the corners of the walls, however.

The masonries were tested to evaluate their nature and stratigraphy, and in addition to determine their state of stress and mechanical characteristics.

The foundation structures were inspected by direct prospection through narrow excavations or by sub-vertical drillings aimed at identifying the materials, the height and width of the foundation plane and the geometry of the masonry. To improve the information gained through the description of the extracted material surveys with video-endoscope were carried out: they made it possible to detect and to size the discontinuities and cavities of the masonry foundation. The masonry of the bearing walls was analyzed with similar techniques.

The compressive stress within masonry was estimated using flat jack measurements (ASTM C 1196-09), the deformability properties were measured with two flat jacks (ASTM C 1197-09). This last test allows also, in certain conditions, to measure the maximum compressive strength.

The 2011 guidelines explicitly state that only direct measurements performed through slightly destructive tests (such as flat jacks) can provide the mechanical parameters necessary to characterize the masonry, in particular in terms of resistance. Non-destructive techniques (such as pulse velocity measurements) only make it possible to assess the homogeneity of the mechanical parameters, but they do not provide a reliable quantitative estimation of the relevant mechanical characteristics necessary for the structural assessment.

Therefore, if compatible with the prevailing conservative requirements, the impact on the building caused by the slightly destructive testing techniques can be accepted. This impact must obviously be limited as much as possible by extending the results of direct measurements with the use of indirect techniques. This is why, during the tests, the number and the invasiveness of the investigation was reduced by correlating the results of the flat jack tests with the measurements of the sonic pulse velocity.

These tests have been carried out determining the longitudinal sonic pulse velocity by transparency between points located on opposite sides of a testing section. The measurements were also carried out where flat jack tests had already been performed, matching the velocity values resulting from the former test and the elastic modulus values provided by the latter test.

Some of the tests carried out were then processed using tomographic methods in order to evaluate the homogeneity of the masonry. In this case, the velocity measurements were repeated according to a dense grid of emission/reception points and according to a large number of paths; they were then processed obtaining tomograms showing the distribution of the velocity of longitudinal sonic pulses (Fig. 2).

2.3 *Wood*

Ceilings and roofs of the analysed buildings are made with wooden beams. The species most used locally in the past were chestnut, cherry and poplar (only rarely oak and conifer).

The inspection of wooden elements was carried out following criteria and procedures defined in accordance with Italian standard (UNI 11119) in order to obtain the in situ classification of wood according to its resistant quality. The visual analysis made it possible to define the state of conservation of such structures and to identify the areas obviously degraded and the areas of possible degradation, that need to be subjected to further investigation by instrumental analysis.

Where signs of abnormalities were identified such as cracks, fissures, dry rot, wood-boring insect bore-dust and torsions, manual tools (hammer and punch)

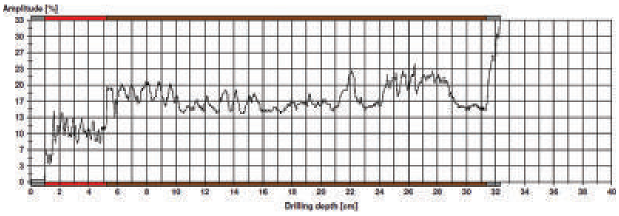

Figure 3. Drilling diagram taken from test on a wooden beam.

were used alongside visual investigation. In this way, it was possible to establish those areas needing further instrumental analysis with the aim of quantifying and positioning any potential areas of internal decay of the timber elements within the structure.

In the areas of excessive superficial softening, hollow sounds on tapping, detachment of wooden elements and in the positions not directly suitable for inspection (like the insertions of the beams within the walls) the analysis was carried out with the use of a specific penetrometer (Resistograph model) which measures the resistance of the timber to drilling as correlated to the density of the wood (Rinn 1994). The detected density variations were plotted by dendrograms (Fig. 3) allowing for the location of the presence of cracks and lesions and the areas where the wood is decomposed or decomposing because of dry rot.

2.4 *Reinforced concrete*

The analysis also concerned some modern buildings built in the 70s of the last century and damaged by the recent earthquake. In these cases, all the bearing structures are made both with masonry and reinforced concrete. The masonry is made with perforated bricks, set in place with vertical holes that can be analysed with techniques similar to the ones described above for the traditional masonry.

The reinforced concrete structures were analysed with the following methods:

- visual inspection;
- survey of the reinforcements of the R. C. structures by electromagnetic covermeter to locate the reinforcement bars in concrete and their orientation as well as to measure the cover thickness and to estimate the bar size;
- evaluation of the carbonation depth in concrete with a solution of phenolphthalein;
- tests to determine the rebound number;
- ultrasonic pulse velocity measurements;
- evaluation of compressive strength with combined methods (SonReb);
- coring, sampling and compression tests;
- sampling and tensile tests on rebars.

It is not, however, the argument of this paper to provide further information about investigation methods of modern buildings, for which we refer to other references (Foppoli 2015).

3 DATA ANALYSIS

3.1 *Criteria of data processing*

The following mechanical properties, derived from experimental data of the numerous tests carried out on masonry, have been tabulated:

- f_m = compressive strength;
- τ = shear strength;
- E = Young's modulus;
- ν = Poisson's coefficient;
- G = shear modulus;
- v = sonic pulse velocity.

Dealing with flat jack tests, the standards provide information about the precision of the test methods: measurements of compressive stress shows the coefficient of variation as great as 20% and exhibits no inherent bias; measurements of deformability characteristics show variations between tests as great as 24% (still less than the results of destructive tests conducted on prisms) and over estimate the average compressive modulus of the masonry up to 15%. The technical literature (Rossi 1994) (Gregorczyk & Lourenco 2000) (Jurina 2007) specifies that the calibration campaigns made in the 90s, referring to properly conducted tests, have provided indications of greater precision and states a systematic overestimate of the compression strength up to 15%. Dealing with the testing method it is also useful to underline that, for all the tested walls, the increase of load during tests with flat jacks was limited to the maximum value of 3.6 MPa. Due to the non-homogeneity of the ancient masonry, over this value the rupture of the jack might happen. This means that it was not possible to detect values of f_m above this limit.

The numerous tests carried out in situ in Emilia allowed the comparison of the results in order to reach the assessment of the mechanical properties of the masonry typical of the area. These characteristics were then compared with the data provided by the current Italian seismic code.

The (average) reference values of the mechanical minimum and maximum parameters, below denoted by the letter a, for different types of masonry (referred to the conditions of proper texture, poor quality mortar, weak connections within leaves, and no consolidation) are deduced from the tables quoted within the 2008 seismic code.

These values have to be multiplied by the correction factors (k) defined by the standards themselves taking into account the possibility of variation in the above defined conditions, that means good quality mortar, good quality connections and so on. In the following considerations, we don't regard the cases of weak core and consolidations with injections or reinforced plaster, which have never been observed in the buildings making the subject of this test campaign: this means that such multiplication coefficients vary from 1 to 1.5. Finally, it is necessary to apply a reductive confidence factor (F.C.) varying from 1 to 1.35, depending on the level of knowledge reached.

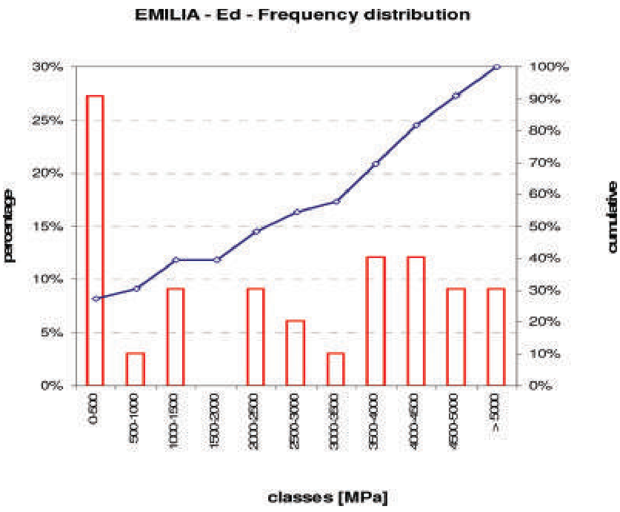

Figure 4. Emilia masonry: Young's modulus frequency distribution – percentage and cumulative frequency.

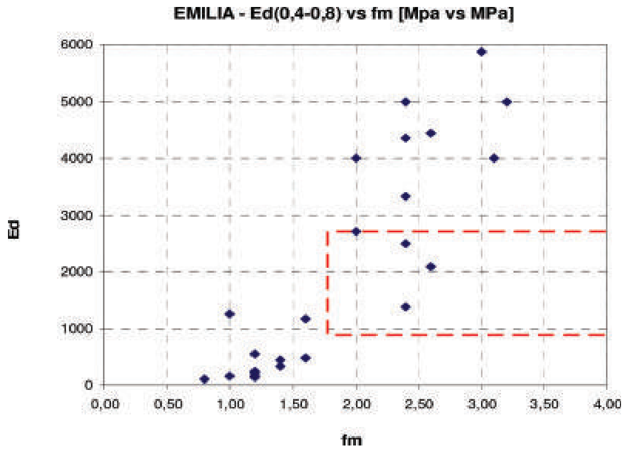

Figure 5. Emilia masonry: Young's modulus vs. compressive strength – results of in-situ tests compared with the range expected by the code (red dashed line).

In conclusion, the mechanical characteristics provided by the seismic code may vary within limits which are determined according to the formula (1).

$$a \lim = a \cdot k \div FC \tag{1}$$

3.2 *Results of processing*

The collected data were then tabulated and plotted to deduce significant correlations.

Young's modulus was calculated with different methods:

- secant modulus value calculated during the first load cycle (hereafter called deformability modulus) in the range from 0.4 to 0.8 MPa = E_d(0.4–0.8);
- secant modulus value calculated during the first reload cycle (elastic modulus) in the range from 0.4 to 0.8 MPa = E_e(0.4–0.8).

This allows us to point out the inelastic behaviour of masonry during the first load cycle and the elastic behaviour of masonry during the following un-loading and re-loading cycles with the consequent higher level of elastic modulus.

The linear correlations among these modulus values resulted in a good quality relationship ($R^2 > 0.93$). That's why all subsequent considerations will be carried out only with reference to the modulus E_d(0.4–0.8), but they can, in any case, certainly be extended to the other values. It is also useful to note that all the stress levels measured within the masonry are lower than 0.8 MPa.

The frequency distribution diagram of the values E_d is plotted on figure 4. It clearly highlights how the values collected through in situ analysis are scattered, with an average of 2672 MPa; 30% of such values are under the level 1000 MPa (very soft masonry) and 30% are over the level 3500 MPa (very stiff masonry).

The diagram that correlates E_d and f_m (Fig. 5) highlights that the relationship between the two parameters is not so significant ($R^2 = 0.78$). In this diagram the range of variation of the (average) reference values provided by the seismic code was plotted with red dashed lines: it is possible to observe that in most cases the measured values are outside the range expected by the code. This is more significant as it shows that many of the analysed brick masonry walls are less resistant than the values expected by the standards, and that there are also walls with stiffness greater than the value expected from the same tables.

Poisson's coefficient values are rather too scattered, within limits 0.03 and 0.79 (average value 0.26). Relationship between Poisson's coefficient and Young's modulus resulted in poor quality ones ($R^2 < 0.20$).

The values of the elastic modulus E_e were also correlated with the propagation velocity of sonic waves v; this relationship is not very effective working with interpolations both of first an second degree. All the sonic tests were carried out repeating the measurements on a grid; it is significant to observe that the single values thus obtained showed a very strong variability, even in very close positions, presumably because of they are strongly affected also by the local masonry texture. It is evident that, within masonry with such low thickness, a single brick could be placed throughout the entire masonry, thus providing local velocity values much higher compared with the case of two bricks arranged with poorly compacted vertical joints.

3.3 *Shear properties of masonry*

The testing methods currently available to study the shear properties of masonry, meaning the diagonal compression tests (ASTM E519-81) or the shear compression tests (Sheppard 1985), requires the application of horizontal load though cylindrical jacks and, if applied to in-situ panels, usually result in high level destruction, so they cannot be practically used for the analysis of existing buildings that are not already badly damaged.

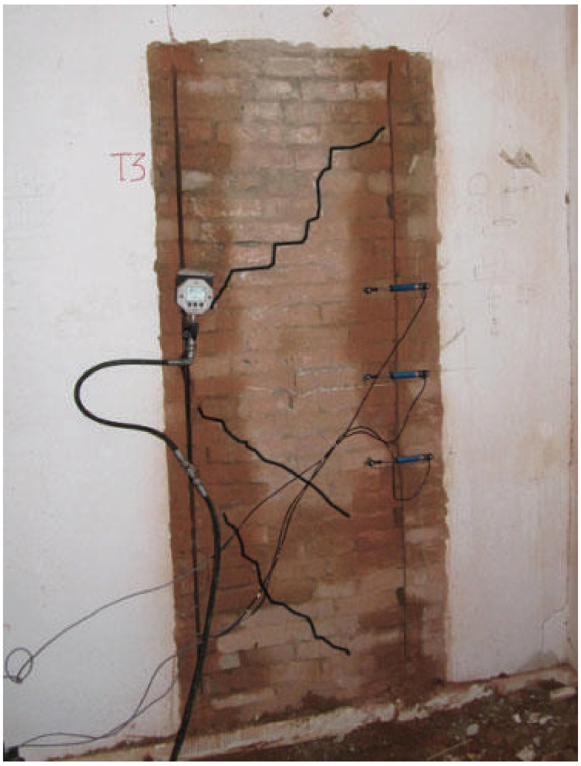

Figure 6. Masonry panel subjected to Flat Jack – Shear Compression Test: the typical diagonal cracks are underlined.

A new test method suitable for the determination of shear masonry properties was recently developed by the author (Foppoli & Pulcini 2016), based on the use of flat jacks that, causing only slight destruction, allows a significant reduction of the impact of the tests on the buildings and then is applicable to a wide range of cases.

Such a test method was calibrated in the laboratory on masonry panels with texture, materials and characteristics similar to the ones of the masonry of historical buildings in northern Italy; a good correlation was found between the results obtained by the traditional tests and those obtained by the new Flat Jack - Shear Compression Test (FJ-SCT). The test method was also used on site in the study context and provided positive results regarding the efficiency and effectiveness of the developed procedures.

The test consists in creating two cuts through the masonry to be analysed, 160–200 cm in length and 8–10 mm in thickness. In one of these cuts, a vertically arranged flat jack is inserted and the opposite cut is equipped with displacement transducers to measure the horizontal deformations of the masonry. In this way the test pattern identifies two half panels placed one above the other, with squared shape and 60–80 cm per side, which are simultaneously subjected to the shear load.

Firstly, a standard flat jack measurement is performed in order to determine the compressive stress within masonry. Through the vertically arranged flat jack a horizontal load is then applied to the test sample and the pressure is increased up to inducing the development of diagonal cracks in the two half panels above and below the jack itself. The shape of the diagonal cracks (Fig. 6) confirms the correctness of the shear failure mechanism developed within the masonry.

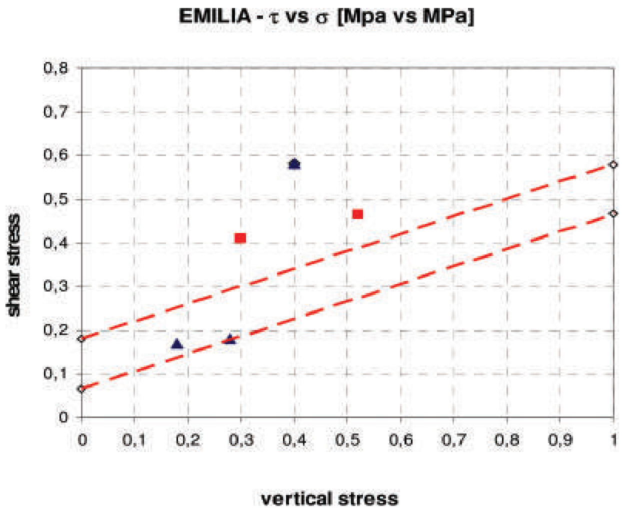

Figure 7. Coulomb’s relationship between compressive stress and shear strength.

The test is then carried out until the detection of appreciable horizontal displacements: all the tests carried out have shown that with a horizontal displacement of 8–10 mm, the masonry is completely cracked.

The limit of this test technique depends on the thickness of the masonry which should be reasonably limited to make the samples representative of the previous size. The standard states a generic reference to the “thickness of the wall type being tested” without further information related to the maximum thickness of the sample. The validity range of the calibrations performed until now covers a side/thickness ratio from a minimum of 2.5; that means, with reference to the above dimensional limits, it refers to a thickness from 24 to 32 cm, in line with the masonry tested in the context of Emilia.

Figure 7 shows the results of tests carried out and the reference to Coulomb’s relationship (2) defined on the basis of the limit values derived from the code, plotted with red dashed lines.

$$\tau = \tau_{0} + \sigma \cdot \tan\phi \tag{2}$$

The experimental on site data provides some shear values much higher than those indicated by the seismic code; however, that fact can be deduced also from an analysis of the literature data (Milosevic et al. 2013). It therefore seems reasonable to deduce that the data provided by the standards prove to be very conservative with regard to the shear strength of the masonry types covered by this test campaign.

3.4 *Summary of the characteristic of brick masonries in the Emilia area*

Based on the previously expressed considerations, it is possible to summarise that it was not possible to

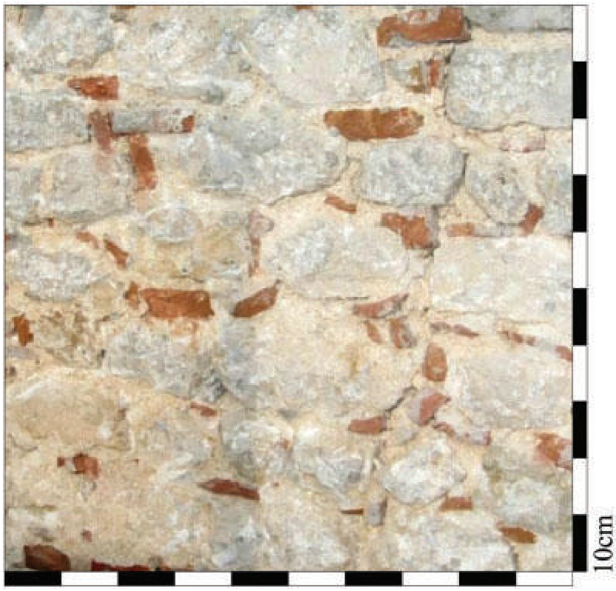

Figure 8. Texture of traditional brick masonry of L'Aquila historical centre.

obtain reliable linear correlations between the mechanical parameters which characterize the masonry (E, ν, f_m).

Scatter of the detected values appears much greater than conceivable on the basis of the tables provided by the code, and this, in many cases, is not in favour of safety.

Furthermore it was not possible to obtain reliable linear correlations between the Young's modulus and the velocity of sonic pulses due to the limited thickness of the analysed masonry.

4 COMPARISON WITH THE MECHANICAL DATA OF HISTORICAL MASONRY IN L'AQUILA

4.1 *Local context*

After the 2009 L'Aquila earthquake a lot of structures located in the area of the seismic crater were tested; also in this case, the investigations had involved mainly historical buildings of different typologies, mostly located in the historic centre: Chiarino palace, Antinori palace, S. Giuseppe neighbourhood, Giuliani neighbourhood, S. Maria Picenze cloister, and a private building placed at Casamaina (AQ).

The analysed historical buildings have masonry made with substantially similar characteristics: irregularly shaped limestone elements set in place with inaccurate techniques (Fig. 8). The most relevant category is described in the standard as "rubble stone disordered masonry", with elements irregular in shape, size, lito-type and material as further specified by other authors.

L'Aquila buildings were mainly tested with flat jacks; the data obtained was analysed using criteria similar to those already mentioned above.

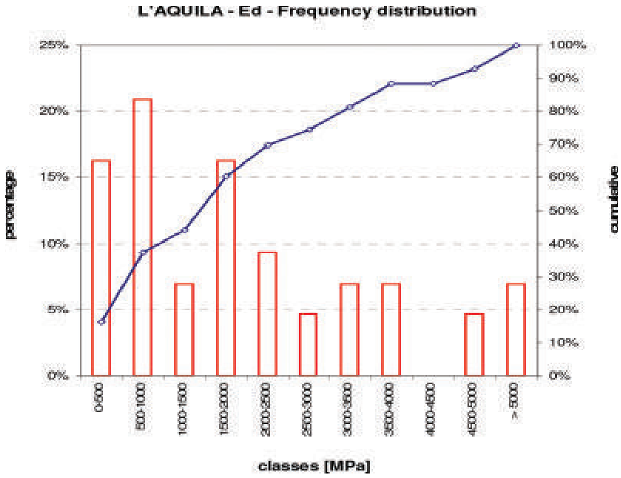

Figure 9. L'Aquila masonry: Young's modulus frequency distribution – percentage and cumulative frequency.

4.2 *Results of processing*

In this case the correlation among the deformability and elastic modulus values is less evident and appears to have little significance ($R^2 < 0,54$).

Detected compressive stress values were on average higher than the values obtained from the tests performed in Emilia (0.44 MPa vs 0.38 MPa) and 10% of the measured values are higher than 0.8 MPa. In any case, the further processing is still performed with reference to the same modulus range 0.4–0.8 MPa.

The frequency distribution diagram of the values E_d is plotted on figure 9 it highlights how the modulus values are less scattered than in the other case, with an average of 2248 MPa: about 40% of such values is under the level 1000 MPa (very soft masonry) and only less than 20% is over the level 3500 MPa (very stiff masonry).

The diagram that correlates E_d ed f_m (Fig. 10) highlights that also in this case the relationship between the two parameters is of little significance. In this diagram, the range of variation of the (average) reference values provided by the seismic code was also plotted with red dashed lines: it is possible to observe that in this case there is a better match with the values measured on site, although there are still a number of values placed outside the range defined by the code.

For a number of tests the values of Young's modulus are also in this case higher than the ones provided from the tables. Poisson's coefficient values are more scattered than the values of Emilia masonry, with an average value 0.55; relationship between Poisson's coefficient and Young's modulus proves to be weak ($R^2 < 0.10$).

4.3 *Comparison of the data acquired in the two territories*

It may be noted that the two masonry types analysed, despite their relevant differences (brick and stonework masonry), provide very scattered values of the modulus E_d(0.4–0.8), but quite close average

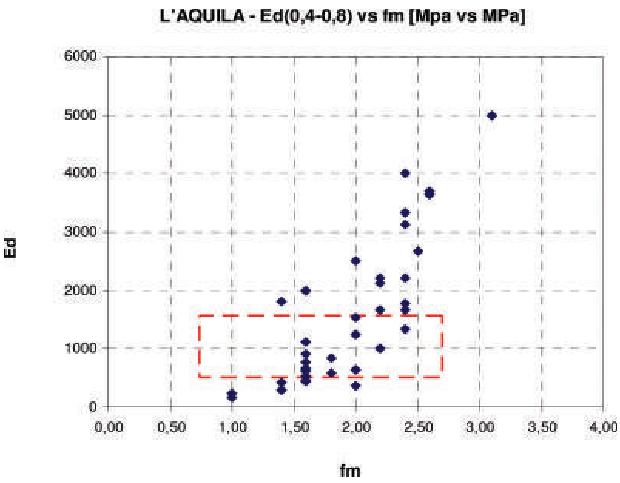

Figure 10. L'Aquila masonry: Young's modulus vs. compressive strength – results of in-situ tests compared with the range expected by the code (red dashed line).

values (respectively 2672 and 2248 MPa); both of them show a bad relationship among their mechanical characteristic values E_d , ν and f_m.

The mechanical characteristics of L'Aquila masonry lie significantly within the limits defined by the technical code, while the Emilia brick masonries provide values which, in most cases, are outside of these limits.

It is interesting to note that, despite their great variability, the average values of ratio E_d/f_m is 972 for the former typology and 938 for the latter typology: these values are very close to the ratio 1000 defined in the code. The Poisson's coefficient is also very scattered with average values of 0.26 in the first case and 0.55 in the second case.

5 CONCLUSIONS

The post-processing of the data taken from analysis on many buildings damaged during Emilia 2012 and L'Aquila 2009 earthquakes allow us to draw some conclusions. Masonry mechanical characteristics detected on the many tests performed produce a great scatter among the results and experimental parameter have not good quality correlations among them.

The comparison of the data related to L'Aquila stonework masonry and those related to Emilia solid brick masonry is significant: it shows that despite the necessary simplifications, the Italian code adequately captures the mechanical characteristics of the rubble stonework masonry, but does not appear sufficiently accurate in defining the mechanical properties of solid brick masonry.

This comes from the fact that the current standard divides stonework masonry in many typologies, depending on the texture, while ranks the brick masonry in a single typology, presumably because there are not evident differences in the texture of these walls. However, it was experimentally observed that, even with correctly built walls, the mechanical properties of brick masonry are strongly influenced by the quality of the mortar and by the joint thickness: this variability should be taken into account by the seismic code to obtain a classification closer to the real characteristics of the masonry.

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