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# DAMAGE EVALUATION AND REHABILITATION OF THE MONTORIO MEDIEVAL TOWER AFTER THE SEPTEMBER 14TH, 2003 EARTHQUAKE

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**Abstract**

On September 14th, 2003, a moderate earthquake struck the Bolognese Apennines, with the epicenter near Monghidoro (30 km far from Bologna, Italy). The seismic event, felt in a sufficiently large area, showed an inhomogeneous damage distribution, due both to site effects and building different vulnerability.

The paper deals with the evaluation of the seismic input (in general and specifically) and its effects on Masonry CUltural Heritage Structures (MCUHESs): in fact, several among them, mainly churches and ancient monuments, were subjected to relevant damage, including the medieval Montorio Tower, matter of this paper, not far from the epicenter. Some of the authors, involved in the on-site Civil Defense investigations, carried out a detailed survey on the above told building (declared unsafe), which showed heavy and spread damage to structural elements, including vertical walls and wooden floors, with one MCS Intensity level more than the pattern suggested by macroseismic data.

After a detailed analysis of its structural characteristics, the Montorio Tower post-seismic rehabilitation (which must avoid a possible conflict between specific conservation criteria and antiseismic requirements) is discussed.

# INTRODUCTION

## 1.1 A brief information on the September14$^{th}$, 2003 seismic event

A moderate earthquake struck the Bolognese Appennines on September 14$^{th}$, 2003 (local time 23:43; Ml 5.0, Mw 5.3; MCS Intensity VII). The epicenter (Lat. 44.220 N, Long. 11.360 E) was identified in the vicinity of Monghidoro (Fig. 1), a small municipality located 30 km South of Bologna (Italy). Other neighbouring hit cities were Loiano, Castel del Rio, Monzuno, San Benedetto Val di Sambro, Monterenzio and Firenzuola.
Due to the focal depth (about 20 km, strike-slip mechanism), the earthquake, originated by the N-S compressive movement interesting the external zone of the Northern Appennines, was felt in a sufficiently large area and caused an inhomogeneous damage distribution, due to site effects and various structural vulnerability. Table 1 shows the most important historical seismic events, both those close to the 2003 epicentral area (in particular 1725, 1781 and 1869) and those at the bordering Mugello seismogenetic zone (1542, 1864, 1919 and 1960).

## 1.2 Seismic input

The Italian territory has been reclassified a few years ago [Presidente del Consiglio dei Ministri, 2003a]. About one half of the municipalities encircled by the MCS Intensity VII were not considered as seismic zones in the old classification (Fig. 2). The seismic classification relies upon the standard probabilistic approach [Cornell, 1968; Bender and Perkins, 1987] that is acceptable, as a general indication of the hazard in terms of probability of excedance of an acceleration value, but that has been proven to be not fully satisfactory in several instances. Case studies indicate the limits of the currently used methodologies, deeply rooted in engineering practice, based on a probabilistic approach. The Probabilistic Seismic Hazard Analysis (PSHA) supplies indications that can be useful but are not sufficiently reliable to characterize seismic hazard: recent examples Kobe (17.1.1995), Bhuj (26.1.2001), Boumerdes (21.5.2003) and Bam (26.12.2003) events. The analysis of global seismicity shows that a single Gutenberg-Richter (GR) law is not universally valid. The multiscale seismicity model [Molchan et al., 1997] implies that the Gutenberg-Richter law can describe adequately only the set of earthquakes with dimensions that are small with respect to the elements of the zonation. This condition, fully satisfied in the study of global seismicity made by Gutenberg and Richter, has been violated in many subsequent investigations. Such a violation has given rise to the concept of Characteristic Earthquake (CE) in opposition to the Self-Organized Criticality (SOC) paradigm. In other words, the problem with PSHA is that its data are inadequate and its logic defective [e.g. Castanos and Lomnitz, 2002, Wang et al., 2003]. Furthermore for the protection of the cultural heritage, the concept of return period is of little value; in fact, such kind of patrimony, which must be handed down intact to posterity as far as possible, cannot be exposed to the roulette of the probabilistic approach.

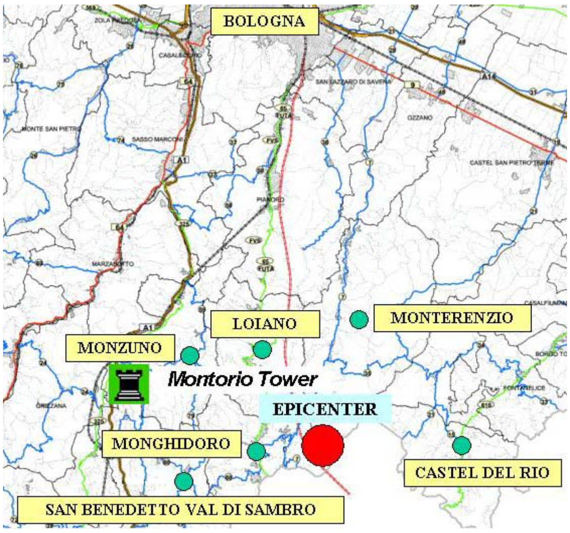



**Figure 1: Epicentral area**

An innovative deterministic procedure has been developed and widely applied [e.g. Panza et al., 2001] that supplies realistic time histories from which it is possible to retrieve peak values for ground displacement, velocity and design acceleration at bedrock level, in correspondence of earthquake scenarios. The procedure is particularly suitable for the optimum definition of the characteristics of the modern antiseismic devices, when the accelerometric data available are not representative of the possible scenario earthquakes and when non-linear dynamic analysis is necessary. A proper evaluation of the seismic hazard, and of the seismic ground motion due to an earthquake, can be accomplished by following a deterministic or scenario-based approach, coupled with engineering judgment. This approach allows us to incorporate all available information collected in a geological, seismotectonic and geotechnical database of the site of interest as well as advanced physical modeling techniques to provide a reliable and robust basis for the development of a deterministic design basis for cultural heritage and civil infrastructures in general [Field et al. 2000; Panza et al., 2001]. The scenario-based approach removes the ambiguity in the results of PSHA. The deterministic methodology (Fig. 3) is strictly based on observable facts and data and complemented by physical modeling techniques, which can be submitted to a formalized validation process. By sensitivity analysis, knowledge gaps related to lack of data can be dealt with easily due to the limited amount of scenarios to be investigated.
It is worth noting that the reference earthquake for the Montorio Tower (Vergato 1869, see Table 1) was not cited in the official seismological reports, but recognized by some of the authors investigating both local archive sources and stratigraphy evidences. Therefore, also the need of an “eco-historical” approach, based on the historical seismography analysis and the study of Local Seismic Culture, is recommended [Pierotti, 2001, 2003].

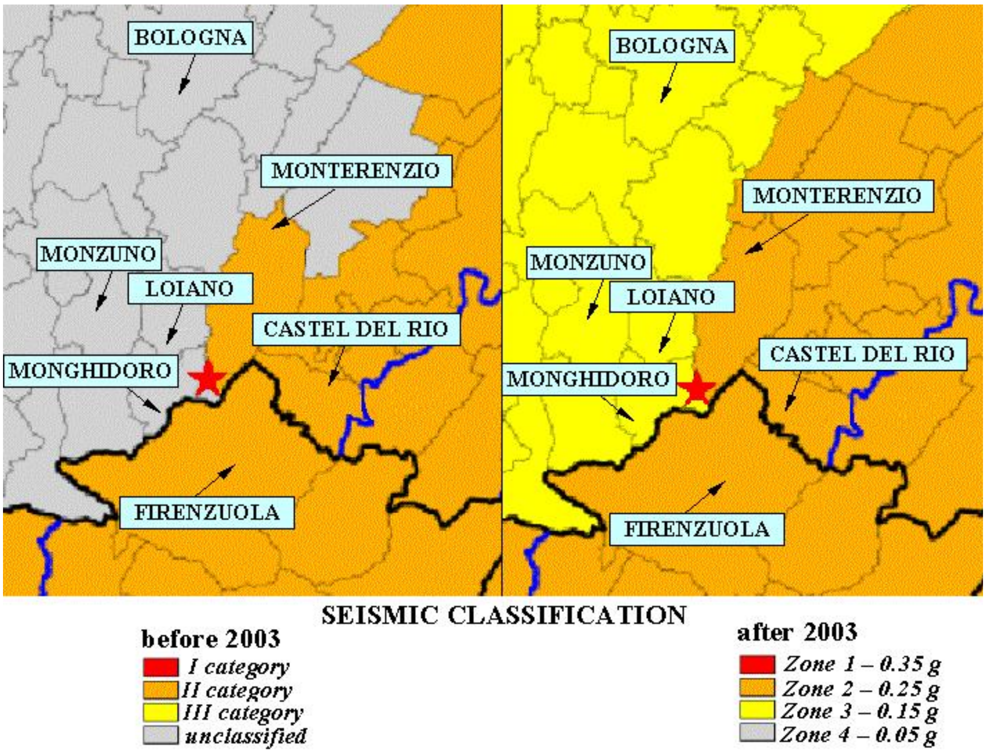



**Figure 2: Seismic classification**

**Table 1: Historical seismic events in the Bolognese Appennines**

| DATE | | | TIME | | | EPICENTER | Lat | Lon | $I_{max}$ | $I_0$ | M |
|---|---|---|---|---|---|---|---|---|---|---|---|
| year | month | day | hr | min | sec | | | | | | |
| 1365 | 07 | 25 | 18 | - | - | Bologna | 44 500 | 11 330 | VII-VIII | VI-VII | 4.6 |
| 1455 | 02 | 06 | - | - | - | Bolognese | 44 400 | 11 250 | - | VII-VIII | 5.1 |
| 1470 | 04 | 11 | - | - | - | Appennino Bolognese | 44 161 | 11 037 | VIII | VII | 4.8 |
| 1505 | 01 | 03 | 02 | - | - | Bologna | 44 480 | 11 250 | VII | VII | 4.8 |
| 1542 | 06 | 13 | 02 | 15 | - | Mugello | 44 000 | 11 380 | IX | IX | 6.0 |
| 1584 | 09 | 10 | 20 | 30 | - | App. Tosco-Emiliano | 43 870 | 12 000 | IX | IX | 6.0 |
| 1725 | 10 | 29 | 17 | 40 | - | App. Tosco-Emiliano | 44 200 | 11 570 | VIII | VIII | 5.6 |
| 1771 | 08 | 13 | - | - | - | Camugnano | 44 167 | 11 167 | - | VI | 4.3 |
| 1779 | 06 | 04 | 07 | - | - | Bolognese | 44 450 | 11 520 | VII | VI-VII | 4.6 |
| 1781 | 04 | 04 | 21 | 20 | - | Emilia Romagna | 44 230 | 11 780 | IX-X | IX-X | 5.9 |
| 1843 | 10 | 25 | 03 | 22 | - | Vernio | 44 072 | 11 144 | VII-VIII | VII | 4.8 |
| 1864 | 12 | 11 | 17 | 40 | - | Mugello | 44 042 | 11 282 | VII | VII | 4.8 |
| 1869 | 06 | 25 | - | - | - | Vergato | 44 314 | 11 116 | VII-VIII | VII-VIII | 5.1 |
| 1874 | 10 | 07 | - | - | - | Imolese | 44 164 | 11 579 | VII | VII | 4.8 |
| 1878 | 11 | 09 | 17 | 49 | - | Castel del Rio | 44 250 | 11 500 | - | VII | 4.8 |
| 1881 | 01 | 24 | 16 | 04 | - | Bolognese | 44 320 | 11 350 | VII | VI-VII | 4.6 |
| 1919 | 06 | 29 | 15 | 06 | 13 | Mugello | 43 950 | 11 480 | IX | IX | 6.0 |
| 1929 | 04 | 20 | 01 | 09 | 46 | Bolognese | 44 470 | 11 130 | VIII | VII | 4.8 |
| 1929 | 07 | 18 | 21 | 02 | - | Mugello | 43 988 | 11 507 | VII | VI-VII | 4.6 |
| 1939 | 02 | 11 | 11 | 17 | - | Marradi | 44 002 | 11 431 | VII | VII | 4.8 |
| 1949 | 03 | 09 | 04 | 16 | 30 | Firenzuola | 44 100 | 11 383 | - | VI | 4.3 |
| 1953 | 02 | 13 | 16 | 29 | 45 | Casaglia | 44 033 | 11 517 | - | VI | 4.3 |
| 1959 | 03 | 24 | 10 | 24 | - | Fiorentino | 43 698 | 11 296 | VII | VI-VII | 4.6 |
| 1960 | 10 | 29 | 00 | 09 | - | Mugello | 43 981 | 11 403 | VII | VII | 4.8 |
| 1962 | 05 | 11 | 01 | 05 | 31 | Camugnano | 44 200 | 11 167 | - | VI | 4.3 |
| 2003 | 09 | 14 | 23 | 43 | 00 | Appennino Bolognese | 44 220 | 11 360 | VII | VII | 5.3 |
| 2006 | 03 | 27 | 08 | 04 | 00 | Appennino Bolognese | 44 210 | 11 040 | V | V | 3.8 |

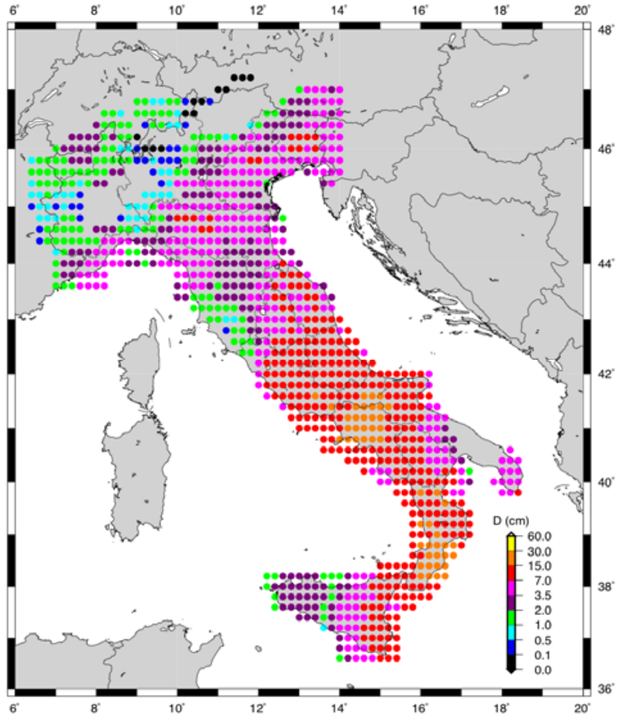



**Figure 3: Horizontal Peak displacement (D) distribution at the national scale, obtained as a result of advanced deterministic zonation**

### 1.3 Design codes and requirements

In the Italian scientific knowledge regarding the restoration of historical centers and MCUHESs located in seismic areas, it is well-established that antiseismic interventions must harmonize protection and conservation. The original approach of Antonino Giuffrè [Giuffrè, 1988, 1991, 1993], dealing with construction techniques and mechanical characteristics, seismic vulnerability and damage mechanisms, protection and rehabilitation, has been largely developed, in particular learning from periodical earthquake lessons [examples in Doglioni, 1994, 2000]. Furthermore, the antiseismic design set of rules has been recently updated in Italy, taking into account both the Eurocodes and research [Modena, 2004, Tomaževič, 2004]. The principal reference for the restoration design of the Montorio Tower is the new Italian seismic code [Presidente del Consiglio dei Ministri, 2003a, b, 2005]. In particular, it contains all the requirements for seismic retrofitting or improving existing MCUHESs. Specific criteria are also indicated in the guidelines produced by the Italian Ministry of Cultural Heritage.

In order to avoid a possible conflict between the conservation requirements prescribed for MCUHESs (integrity, compatibility, reversibility and durability) and the antiseismic rehabilitation, the philosophical approach can be summarized in these following simple statements:

a) because the MCUHESs rehabilitation problems are much more difficult to solve than those related to modern reinforced concrete or steel structures, interventions can derogate from the antiseismic design criteria foreseen for ordinary buildings;

b) in relation to the state limit analysis, the intervention must be defined as a “controlled structural improvement”, i.e. accepting an antiseismic protection level lower than required, in order to reduce invasivity;

c) for each limit state, the improvement effectiveness must be quantified, evaluating the PGA levels which generate the local collapse mechanisms, before and after the intervention;

d) because the MCUHESs characteristics (history, material properties, construction details, quality of connections, state of integrity and maintenance, etc.) are frequently not well known, detailed survey, damage assessment and diagnostic campaigns must be carried out, in order to reach a knowledge level as deeper as possible [Binda 2004]; moreover, each MCUHES is different: therefore, it is necessary to undertake the rehabilitation design in a specific way, use of standardized procedures being not possible;

e) the observance of the "regola dell'arte", i.e. the unwritten construction rules for masonry elaborated by architects and bricklayers in centuries of work practice, is fundamental for protection (good overall static and dynamic behavior), conservation (durability in after years) and restoration (avoiding irreversible mistakes); the use of modern techniques and materials can be very useful to reduce seismic vulnerability, but it must be philologically correct, compatible and mechanically effective.

## 2. THE MONTORIO TOWER

### 2.1 Site description, construction history and building characteristics

Site and building description, damage survey and analysis are fully described in the ENEA report quoted in bibliography [Carpani et al. 2004].

#### *2.1.1 Site description*

The Montorio Tower (Fig. 4) lays directly on the top of a sandstone rocky cliff, overhanging on three sides and anciently suitable for defensive purposes. This kind of topographic position can emphasize seismic shaking and the geological configuration is particularly exposed to landslides and atmospheric erosion; in fact, significant damage is documented after the 1869 (Vergato) seismic event and several retaining interventions were necessary since the beginning of the XX century.

**Figure 4: The Montorio Tower**

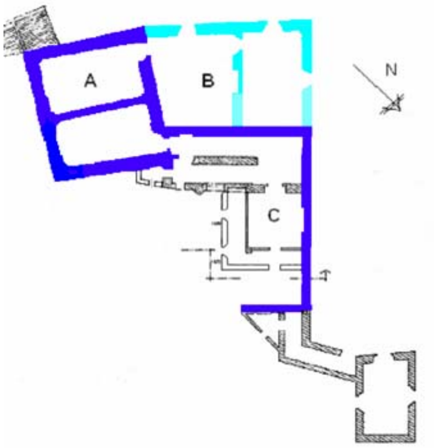



**Figure 5: Chronological growth**

#### *2.1.2 Construction history*

The whole architectonic complex grew around an original XII-XIII century nucleus, identified by the tower itself, the defense wall and the shelter inner court (Fig. 5, part A). The enlargement B (two vaulted rooms) belongs to the XVI century and the sectors C were added in the 1700 first decades. The monument, a relevant example of genuine defense architecture, has been protected by the Ministry of

Cultural Heritage since 1910. If the tower presence is certified by the first known cadastral documents (1235), historical considerations make suppose the existence of a Longobard fortification system in the Bolognese Appennines area before the X century. In order to stiff the building, two S-W buttresses (Fig. 6) were realized in 1570-80, while the E corner strengthening was completed during 1900, together with other minor modifications. Furthermore, after the above told 1869 Vergato earthquake damage, the tower was lowered of one level in 1870.

### *2.1.3 Structural description*

The Tower is a five-level construction (cellar, raised level, mezzanine, first floor and garret), with a typical two-roomed space divided by a central wall. External dimensions in plan are about 10.6 x 9.3 m and foundations lay directly on the rock (Fig. 7). The vertical walls (made by local sandstone and river pebbles) generally show regular horizontal stone-lines and bigger squared elements on the corners (Fig. 8), but somewhere the masonry fabric is inhomogeneous and presents discontinuity due to the subsequent additions (Figs. 9-10); the thickness is about 1 m; pronounced tapering with height is evident and an out of plumb has been measured in the S-E front. The horizontal floors (Fig. 12) are mainly made by single or double wooden truss frames, covered by a light layer on pressed sand or debris. In correspondence of the first level, three steel ties have been found. The sixteenth-century addition (cellar, two floors and garret) leans against the N-W tower front and the shelter inner court wall, but the lack of transversal connections is evident. The stone-masonry (similar to the tower's one) reveals minor thickness (0.6-0.7 m) and, externally, consistent out of plumbs. The raised floor rooms are covered by pavilion vaults and the cellar (Fig. 11) shows an interesting barrel vault (a one-leaf brick top with reinforcing vertical brick lines and a background arch). Generally, the old roofs are made by light double wooden frames, including top truss, midst pitch trusses, sloping joists, wooden layer and tiles. Even if string-courses are absent, longitudinal wooden trusses are located at their place, leaning directly on the top of the perimeter walls, with the aim to distribute the static loads. In correspondence of the transversal walls, diagonal struts support the longitudinal roof frame. Moreover, the covering structure elements, which can be responsible of pushing effects, show an overall deterioration and often marked inflexions (Fig. 12).

**Figure 6: S-W buttress**

**Figure 7: Wall on sandstone**

**Figure 8: Vertical walls**


**Figure 9: The Montorio Tower façade**

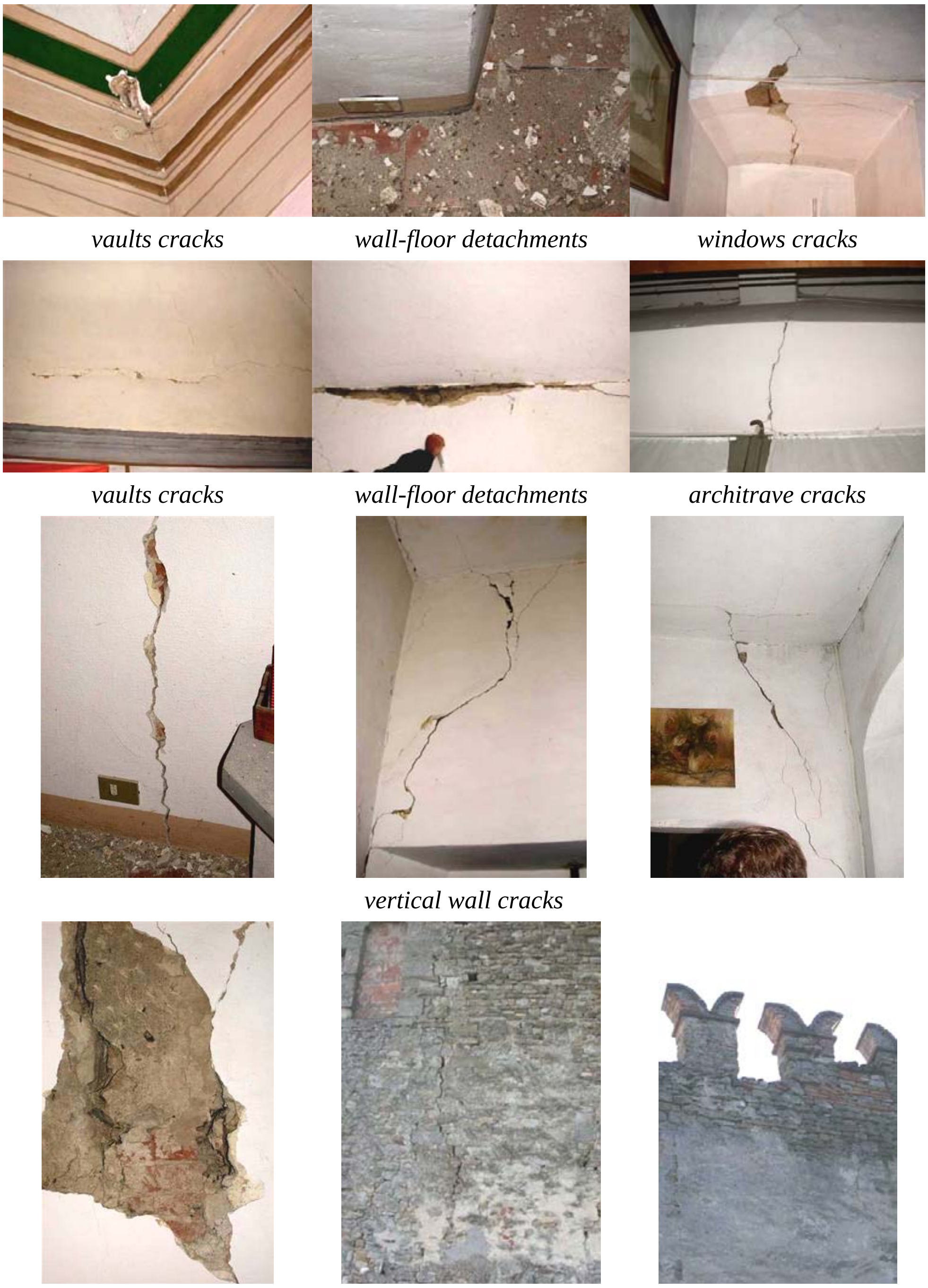

**Figure 10: Structural discontinuity**


**Figure 11: The cellar vault**

### *2.1.4 Earthquake damage patterns*

Most of the earthquake damage patterns are due to the following reasons (Fig. 13): lack of connections between different structural elements; ineffectiveness or absence of horizontal steel ties; presence of inhomogeneous masonry sectors (closed openings, niches, etc.); adoption of vulnerable construction typologies (one-leaf masonry vaults, floors with low stiffness, etc.); evidence of static defects existing before the seismic event (out of plumbs, detachments, previous cracks, etc.); generally deteriorated conditions due to scarce maintenance. In any case, the earthquake has been surely responsible of significant outwards overturning of the walls (first-mode mechanism due to out-of-plane dynamic forces).


**Figure 12: floors (left) and roofs (right)**



*vaults cracks* — *wall-floor detachments* — *windows cracks*

*vaults cracks* — *wall-floor detachments* — *architrave cracks*

*vertical wall cracks*

*inhomogeneous masonry* — *fractured merlon*

**Figure 13: Principal earthquake damage patterns**

## 3. THE MONTORIO TOWER REHABILITATION

The seismic classification, consistent with the average seismic load, cannot deal neither with the seismogenetic potential (historical seismicity and seismotectonics) nor with the local effects that can cause an increment of one degree $I_{MCS}$. The geotechnical properties of local soils and the quality of buildings can have worsened in the past centuries, therefore it could be necessary to actualize the Intensity values of historical events. The new classification gives for the site of interest an anchoring value of 0.15g. At the same site, the upper bound of the deterministic peak values, that take into account the seismogenetic potential, are consistent with this value. Therefore 0.15g will be used as stringent first order guideline for the Montorio Tower rehabilitation. More detailed ad hoc studies will allow to take realistically into account local soil conditions, avoiding the use of convolutive methods that proved to be quite unreliable [e.g. Field et al., 2000; Panza et al., 2001].

The identification of structure knowledge levels (geometry, construction details, properties of materials) and calculation procedures (limit state structural analyses), are clearly reported by the already mentioned new Italian seismic code. Due to the complexity of the existing ancient MCUHESs (often characterized by the reciprocal lack of connections of vertical masonry elements and between them and their floors), the Italian code foresees the analysis of local damage mechanisms, in particular regarding the out-of-plane collapse of single panels, but also the overturning of entire vertical walls (if monolithic), as shown by many earthquake damage patterns. Due to the almost-zero tensile masonry resistance, structure geometry and constraint effectiveness are crucial factors for the possible origin of labilities under dynamic loads. Then, a kinematic approach is necessary, identifying the horizontal force which moves the structural element from its static equilibrium [see Giuffrè, 1993]. In addition, structural aggregates must be closely studied, because often they cannot be separated into smaller blocks, in order to simplify the structural analysis. Due to the above mentioned concepts, both diagnostic/dynamic characterization experimental campaigns and numerical analysis have to be planned in the next months, as foreseen for similar constructions [Indirli et al., 2001, 2006].

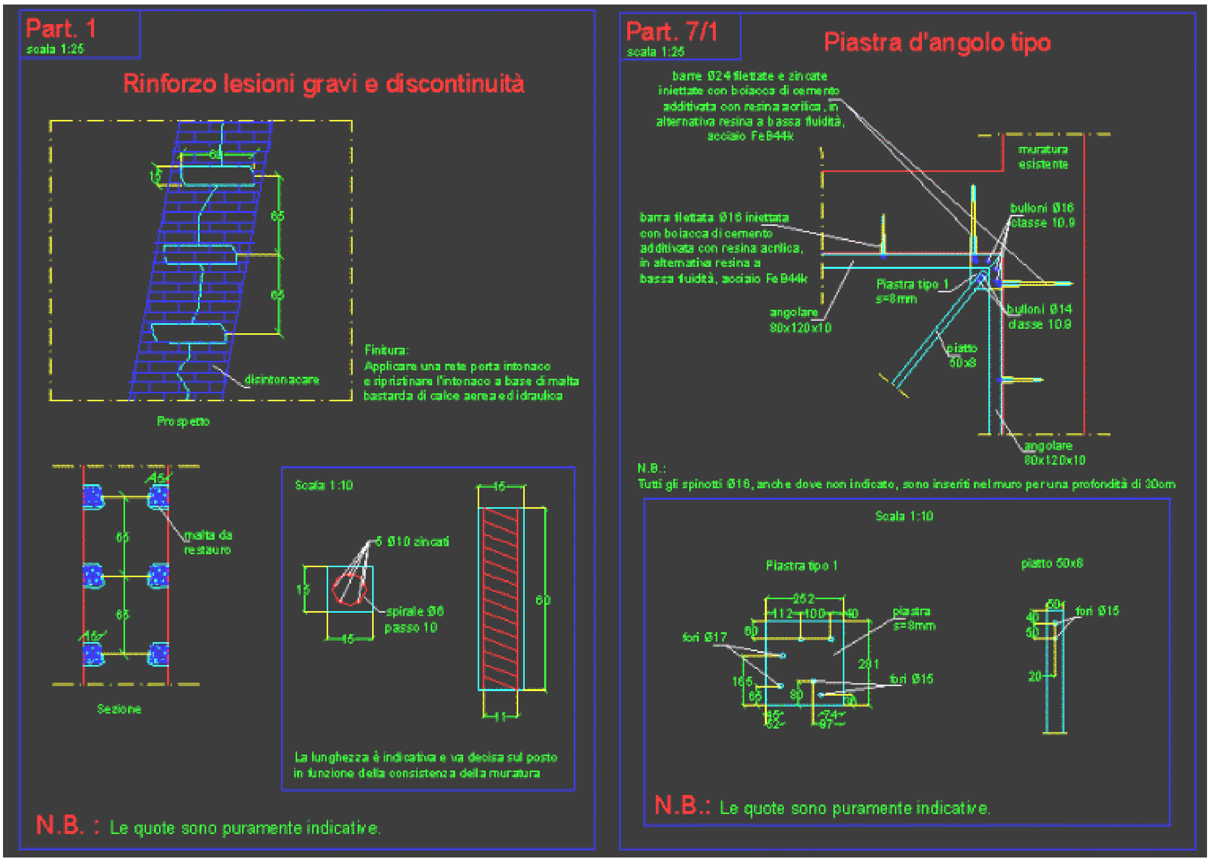



**Figure 14: some intervention details**

Getting into the description of the Montorio Tower rehabilitation, it is necessary to underline that the presence of earthquake serious damage makes necessary to put urgently the structure in safety conditions, in order to avoid a possible future deterioration until failure. To this purpose, public reconstruction funds will be available just enough to cover the essential prompt interventions, which have been thought (together by ENEA and Prof. Rodolfo Antonucci of Ancona University) to be permanent and propaedeutic to further implementations.

Firstly, a widespread introduction of steel ties at each level (in both the horizontal directions) is foreseen, with the aim to improve the structural box behavior and stop the overturning danger; in fact, this remedy is very powerful, as demonstrated by several shaking table tests on steel tied stone-masonry prototypes [Antonucci 2005]. A second aspect regards the punctual heavy cracks stitching by the insertion in the masonry of a series of galvanized iron bars (Fig. 14 left) astride the lesion and then plunged in compatible mortar, in order to keep the reading of the crack still visible; generalized mortar injections will be avoided; in case of widespread cracks - the most sited above the windows - only localized injections of compatible mortar will be carried out. The vaults will be repaired and thickened, if necessary, by layers of thin bricks, because the rotation at springings will be blocked by the above said steel ties.

All the floors will be generally provided by adequate stiffness, implementing the wooden trusses and replacing the deteriorated ones, with a particular attention to the anchoring system to the vertical walls (Fig. 14 right). The top stone-masonry, bearing the roof, will be completely reconstructed in a traditional way, due to its very degraded conditions. At the same time, a steel string-course will be provided, in order to connect the garret floor, stiffened as best as possible, and avoid pushing effects. The roof primary wooden trusses will be kept and reinforced, while the secondary ones, almost rotten, must be replaced. Finally, the cover existing tiles will be integrated with new ones.

## 4. CONCLUSIONS

The innovative deterministic procedure we have applied supplies realistic time histories from which it is possible to retrieve peak values for ground displacement (e.g. Fig. 3), velocity and design acceleration at bedrock level. The same can be done for earthquake scenarios. The procedure is particularly suitable for the best possible definition of the characteristics of the modern antiseismic devices to be used in the rehabilitation, since it allows to properly estimate the local soil effects, thus defining a very realistic seismic input. Should it be required the non-linear analysis of structures, realistic time series are available as well.

The rehabilitation design of ancient MCUHESs, as the Montorio Tower object of this work, is often complicated and a standardized approach is not allowed. The new Italian antiseismic code finally provides clear rules and requirements to restore this kind of structures, and confirms the possibility to derogate from the antiseismic design criteria foreseen for ordinary buildings (reinforced concrete or steel constructions). The intervention must be defined as a “controlled structural improvement”, accepting an antiseismic protection level lower than required, in order to reduce the conflict between strengthening and conservation, but, for each limit state, the improvement effectiveness must be quantified.

The execution of diagnostic and dynamic experimental campaigns has to be encouraged, because they are fundamental, in order to increase, as best as possible, the knowledge level of the structure (geometry, construction details, properties of materials) and implement the numerical models. The results of the structural analysis, together with other considerations coming from the study of local damage mechanisms and regarding structural aggregates, will bring into focus the final intervention. In any case, the observance of the “regola dell’arte” (i.e. the unwritten construction rules for masonry elaborated by architects and bricklayers in centuries of work practice), will guide the rehabilitation approach. The use of modern techniques and materials can be foreseen in order to reduce the seismic vulnerability, but it must be philologically correct, compatible and mechanically effective.

At the moment, public reconstruction funds will be available just enough to cover the essential prompt interventions on the Montorio Tower, to put urgently the structure in safety conditions, in order to avoid a possible future deterioration till failure. At the same time, the first works have been thought to be permanent and propaedeutic to further implementations, supported by appropriate experimental campaigns and numerical analyses.

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