EVALUATION AND DESIGN CRITERIA FOR RESTORING AND RETROFITTING DAMAGED MASONRY BUILDINGS

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SUMMARY

The paper has the aim to suggest some criteria for the seismic evaluation and design of the restoring and retrofitting intervention to be carried out on the masonry buildings damaged by the Umbria – Marche earthquake. The reference codes are the national seismic regulations and the specific seismic regulations endorsed by the technical committee instituted for the post-earthquake construction activities. Starting from these bases a flow chart is proposed to perform the following actions: (i) assessment of the building resistance to vertical and seismic loads prior to the earthquake, (ii) identification of the possible vulnerability sources to be eliminated, and (iii) evaluation of the building resistance after the interventions.

The work has been devoted both to masonry and reinforced concrete buildings; in the present paper only masonry structures will be analysed, which are the greatest fraction of the damaged buildings.

The proposed procedure is devoted primarily to professional applications, so approximated practical calculations were preferred to more rigorous scientific approaches. The indications from existing regulations and from other relevant research works have been synthesised to make them applicable in the practising activity.

INTRODUCTION

The general strategy to be followed for the post-earthquake restoring interventions has been established by the reconstruction law in Umbria and Marche (1998) and is based on the "seismic improvement" concept. It is one of the two options considered in the national seismic code (1996) for the existing buildings and is defined as "a series of structural interventions aiming to increase the seismic resistance of the building, without substantial modification of the original structural behaviour". The other option is the seismic retrofitting, defined as "a series of structural interventions aiming to give to the building the same resistance of a new one, also allowing a substantial modification of the structural behaviour ".

Generally only local interventions are needed for the seismic improvement and often the design is based mainly on qualitative procedures, also if the Instructions to the code (1997) specify that "an estimate of the increase of safety and of the final safety level have to be carried out, also in a simplified way".

The 1998 recommendations for the restoring and retrofitting procedures enforced by the Umbria-Marche Technical Committee contain a substantial innovation with respect to the national seismic regulations. In fact, also for the seismic improvement category a quantitative measure of the security level is required. Furthermore for all buildings the achievement of a minimum security level (65% of the security level of a new building) and a quantitative measurement of the security level before the interventions (before the seismic damages) are required. The latter information is useful for the calculation of a cost/benefit indicator.

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Crossing the damage level observed on the building with the vulnerability level, obtained by checking both the structural elements and the global seismic resisting system, one obtains a list of the most likely interventions to be conducted. In any case, some “minimum” interventions must be carried out.

Another innovation is that the site amplification effect of the seismic action has been considered in the design by means of a coefficient established for most of the localities on the basis of a wide rapid microzonation, carried out with the funds available for the reconstruction.

2. STRUCTURAL VERIFICATIONS

The proposed methodology starts both from the damages evaluation and the vulnerability analysis. In this paper we are concerned especially with the vulnerability analysis, referring the reader to other works in this session for the damage evaluation.

The vulnerability analysis should be suitably supported by structural models capable to simulate the structural behaviour under the seismic action: in particular the various collapse behaviours the structure could undergone should be correctly reproduced. The models should also allow a measure of the building safety against each collapse behaviour. Therefore both the global response of the structure and the local failure mechanisms (such as out-of-plane wall overturning and bending or the ties effectiveness) have to be checked.

2.1 Local behaviour

Separations between structural components like: sliding of the roof supporting beams, slipping of the floor beams from the walls, out of plane overturning of the walls, falling of chimneys and parapets, are likely to occur when restraints and connections are lacking.

Other local failure mechanisms like wall overturning under thrusting roofs or wall bending collapses are likely due to lack of resistance of the masonry

For the verifications against these kinds of collapses one may use many procedures, belonging both to recent and previous recommendations and to a large number of research works on the topic.

2.2 Global behaviour

When all the local mechanisms have been checked and partial failures are avoided for the reference seismic action, also the effectiveness of the global behaviour must be assured.

With reference to the whole wall-floors system, for the masonry buildings, the so called ’box-like’ behaviour guarantees the best exploitation of the available structural resources.

Various methods depending on the particular situation (structural regularity, floor number, type of restrains) can be selected for the in-plane resistance estimation. The following checks should be conducted: (i) shear resistance for in-plan actions and (ii) axial-bending resistance for in-plane actions.

2.3 The Italian national regulations framework

In the past years the ’POR’ method was the most widely used. It is based on the assumption that (i) the static-equivalent seismic forces are transmitted to the various walls by the in-plane rigid floors and (ii) the possible collapse of the walls is mainly due to the shear forces.

This method was introduced by the regulations issued after one of the biggest earthquake that struck an Italian region in the recent years [Basilicata-Irpinia, 1981] and was considered applicable only for buildings in which the above mentioned requirements (i) and (ii) were met. In particular the failure criterion was considered applicable for low-rise buildings with stiff and resistant walls above the openings. For buildings without stiff floors the seismic shear was shared among the walls according to the weights supported by them, for high-rise buildings frame models were recommended. In the same law, the local mechanisms due to out-of-plane bending were checked with reference to a tensile-resisting section, and to maximum ‘characteristic compressive and tensile stresses specified on the basis of a gross description of the masonry elements and mortars. In the subsequent years, up today in some zones, this method has been widely used to design post-earthquake intervention on existing building. In the same period an officially recognised calculation method for new masonry buildings was unavailable and the seismic resistance of these buildings has been checked only on the basis of ‘geometrical’ rules (distance between walls, position of openings, wall thickness, etc.) .

In the 1987 a decree was issued for the masonry building in non seismic areas, introducing a complete method to check the resistance of these buildings on the basis of calculations and material characteristics. In this decree the eccentricity due to the vertical loads is taken into account in the verifications and lead to a reduction of the resisting area of the section, both in the axial-bending and in the shear verifications. The masonry failure domain is described trough a Coulomb-like criterion.

In the last issue of the seismic regulations (1996) the limit state method in seismic checks was allowed together with the pre-existing allowable stress method. Furthermore, the applicability of the 1987 decree on masonry structures was extended to seismic zones, closing the pre-existing code gap which allowed calculation checks in seismic zones only for existing buildings.

This complex code evolution and the innovation introduced just before the 1997 earthquake by the extension of the 1987 decree to the seismic zones, has produced uncertainties in the professionals involved in the reconstruction. For this reason the National Seismic Survey and the Building Office of Perugia have published a volume [De Sortis et al., 1998] containing suggestion and examples to apply old and new regulations in a coherent way, taking also into account alternative approaches developed in the research field. The volume examined both masonry and reinforced concrete buildings, considering old and new regulations. In the following, for shortness, reference will be made only to the 1996 national and to the 1981 Basilicata-Irpinia seismic regulations.

3. CRITERIA FOR THE STRUCTURAL VERIFICATIONS

The proposed verifications criteria are based on the following hypotheses:

1. Limit states approach, using partial safety factors both for the applied loads and for the material strength.

2. Selection of the strength parameters by the structural engineer based on the typology, the quality and the preservation conditions of the masonry, if experimental tests are not carried out (recommended for important buildings).

3. Possibility to apply both the 1987 degree and the 1981 (Basilicata-Irpinia) regulations. In the second case, for the existing buildings, material safety factor is equal to 1. In any case, the structural model shall be representative of the actual behaviour of the structure.

4. The total element forces to be introduced in the verifications are: $\alpha_p' \pm \gamma_e\, \alpha$ where the load safety factor $\gamma_e = 1$ for the masonry buildings, α are the seismic forces and α_p' are the vertical forces obtained following the combination: $\alpha_p' = \gamma_G\, G_k + \gamma_Q\, [(Q_{1k} + \Sigma(\psi_{0i}\, Q_{ik})]$, where G_k is the characteristic value due to the permanent loads, Q_{1k} is the statistical value due to the main variable load, Q_{ik} are the statistical values due to the other variable loads and ψ_{0i} are the respective combination factors (0 for the wind and 0.7 for the other loads), γ_G equal 1 or 1.4, γ_Q equals 0 or 1.5. Due to the typical behaviour of the masonry, not all the possible combinations obtainable with the previous expression are needed, so, for the sake of simplicity, it is suggested that only the two combinations leading respectively to the maximum axial force ($\gamma_G = 1.4$ and $\gamma_Q = 1.5$) and to the maximum eccentricity (minimum axial force, $\gamma_G = 1$ and $\gamma_Q = 0$) can be considered.

5. A system of static loads equivalent to the seismic action has to be applied to the building’s floors. The horizontal load at the “i” level has the expression:

$$F_i = K_{hi}\, W_i \tag{1}$$

where $K_{hi} = C\, R\, \varepsilon\, \beta\, \gamma\, I$, $W_i = G_i + s\, Q_i$, C is a coefficient depending on the Italian seismic zonation ($C = C_{ref} = 0.1$ for the high seismicity, 0.07 for the medium and 0.04 for the low), R depends on the natural period of the structure (for masonry usually equals 1), ε is called “foundation coefficient” and substantially depends on possible local amplification problems (supplied for many sites in the two regions on the basis of a rapid seismic microzonation with values in the range 1 to 2), β is called “structural behaviour coefficient” and takes into account the lower ductility of the masonry with respect to r.c. and the different check method, assuming a value of 4, I is the importance factor (1 for the residential buildings), γ is a force distribution factor roughly corresponding to a first-mode of vibration, W_i is the "seismic weight" that is the weight of the mass excited by the earthquake, G_i is the self weight of the floor, of the masonry and of the non structural elements pertaining to it, Q_i is the total variable load on the floor and s is a reduction coefficient (usually equals 0.33 for residential buildings). It is worthwhile to note that the load combination for the calculation of α_p' is different from that for the calculation of α.

6. In accordance to the 1998 Umbria-Marche restoring recommendations, the following steps are required: a) the building must resist to a minimum seismic action calculated with the (1) where $C = 0.65\ C_{ref}$; b) the resistance of the building must be increased with respect to the pre-earthquake situation. For the latter demonstration, the engineer has to calculate the value of C that substituted in the (1) leads to a load system corresponding to the failure of the building before and after the interventions. The possible failures to be considered, as a minimum, are: (i) out-of-plan overturning, (ii) out-of-plane axial-bending collapse, (iii) in-plane shear failure, (iv) in-plane axial-bending collapse, (v) failure of ties and anchorage.

4. OUT-OF-PLANE FAILURE

Usually this check is conducted on a single vertical strip of wall considering the following static forces: (i) a distributed load equal to the wall's self-weight multiplied by βC, (ii) concentrated loads transmitted to the wall by the floors, if they are not effectively connected to the transversal walls, multiplied by βC.

4.1 Overturning

The following steps must be performed: (i) selecting the possible overturning configurations (whole wall or some portions of it, see Fig. 1); (ii) writing the overturning moment with respect to the unknown value of C; (iii) writing the stabilising moment (all terms are known); (iv) resolving, with respect to C, the expression in which the overturning and the stabilising moment are equated. The procedure can be followed both for the pre-earthquake and for the post-intervention configurations, changing the applied loads and the overturning mechanisms. The contribution of the ties to the stabilising moment calculation can be considered as the yielding load (axial or of the anchorage), for the pre-earthquake situation, and as the minimum value leading to value of C not lower than that pertaining to the other failure mechanisms, for the post-intervention.

4.2 Axial force and bending

The structural configuration for the pre-earthquake and for the post-intervention verifications could be different. In the first case, for example, if the ties, or other suitable connecting devices, are completely absent, a cantilever clamped at the foundation level should be the reference scheme. After the intervention, having designed suitable connections at the floor level, the reference scheme could be a continuous beam or several simple supported beams. A more sophisticated scheme, such as a plate restrained by the floors and by the transversal walls, could be also selected, alternatively simplified rules to assess the 'equivalent span' can be used.

To express the final safety the following steps should be done: (i) selecting the resisting scheme (see Fig. 2); (ii) writing the maximum compressive or tensile stress with the formula:

$$\sigma = N/A \pm M/W$$

with respect to the unknown value of C (N and M are functions of C); (iii) resolving, with respect to C, the expressions in which the maximum compressive stress is set equal to the limit compressive strength or the maximum tensile stress is set equal to the limit tensile strength (lacking experimental data, the limit tensile strength is selected equal to the characteristic limit shear strength). When using the 1987 decree the procedure is more complicated because a non-linear relation exists between the geometrical reference parameters and the seismic action, but this topic is not described here.

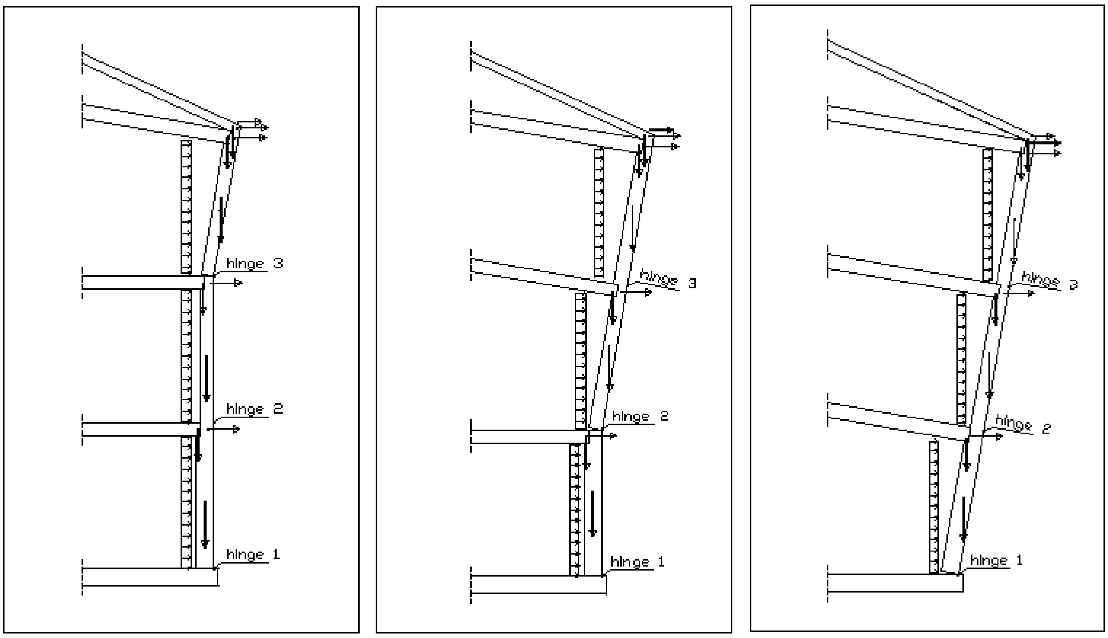

Figure 1: Selection of the possible overturning configurations

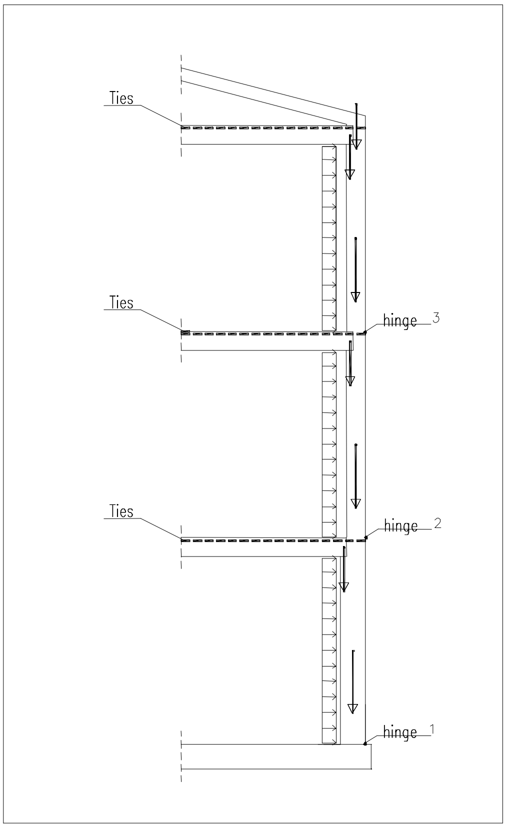

Figure 2: Selection of the resisting scheme

5. IN-PLANE FAILURE

The in-plane forces should be evaluated with reference to the complete structural system, taking into account only the effective connections due to the floors and applying the seismic loads to the centre of mass of each floor.

Rigid floors or in-plane flexible floors can be considered according to characteristics of the structure. In the rigid-floors option, the seismic loads are distributed among the walls proportionally to their stiffness; in the flexible floors option, the seismic loads are distributed proportionally to the influence area of each wall taking into account the seismic forces transmitted by the restrained walls.

Shear failure or axial-bending failure are the possible mechanisms considered. The shear failure is typical of low rise buildings or complex aggregation of buildings with squat walls connected by the so called 'rigid floor strips', that is portions of masonry with high stiffness and resistance in the vertical plane. The rigid floor strips delimit portion of walls in which shear-type behaviour and shear failure occurs. The axial-bending failure is typical of high rise building (towers, bell towers) or buildings with peculiarities (weakness of the floor strips, low thickness of the masonry); in this case the behaviour of the buildings can be assimilate to independent vertical cantilevers and axial-bending failure occurs.

From the observation of the most frequent building typologies and of the damages occurred in the Umbria and Marche regions, one could conclude that shear failure is prevalent. For this reason, in the following only shear failure verifications will be considered in detail.

5.1 Rigid floors

In the rigid-floors option, the seismic loads are distributed between the various walls proportionally to their stiffness (Fig. 3):

$$K = \frac{G \cdot A}{1.2 \cdot h} \cdot \frac{1}{1 + \frac{1}{1.2} \cdot \frac{G}{E} \cdot \left(\frac{h}{b}\right)^2}$$

where G and E are the masonry shear and Young's moduli, A is the area of the cross section of the wall, b is the wall base, h is the wall height. Lacking experimental tests, the following relation could be selected: E/G = 6 and G = 1100 τ_k, τ_k is the masonry characteristic shear strength (ranging from 2 ton/sqm for irregular stone masonry to 18 ton/sqm for clay bricks masonry). Also if the consistence of physical properties like the shear resistance for the masonry is questionable, due to the complex internal behaviour, never the less some experimental results show that the conventional values of the shear 'characteristic' strengths considered in the 1981 law are generally on the safe side, as it can be seen from the Fig. 4 [Di Pasquale et al., 1999]. For the compressive stress the same consideration holds, with the exception of bad quality irregular masonry repaired with cement injections or grouted bars, which experimentally show values lower than those indicated by the law.

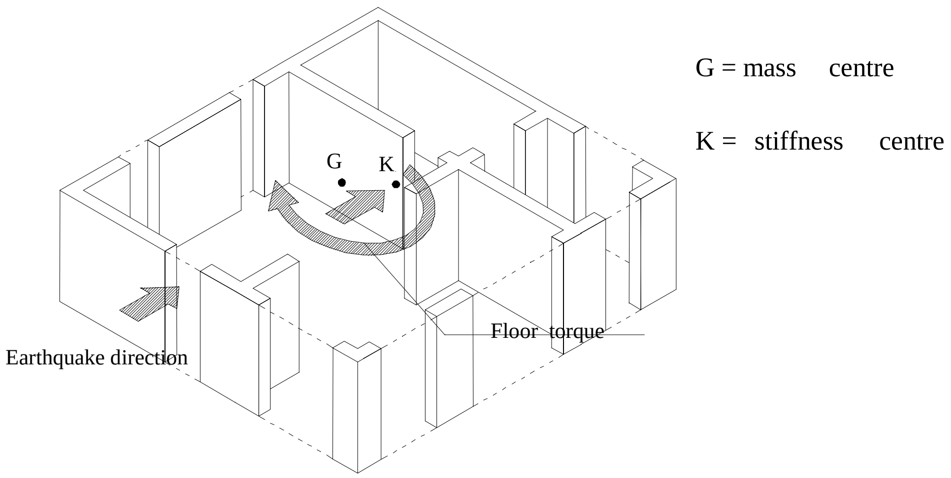

Figure 3: Distribution of the seismic forces with the rigid floors option

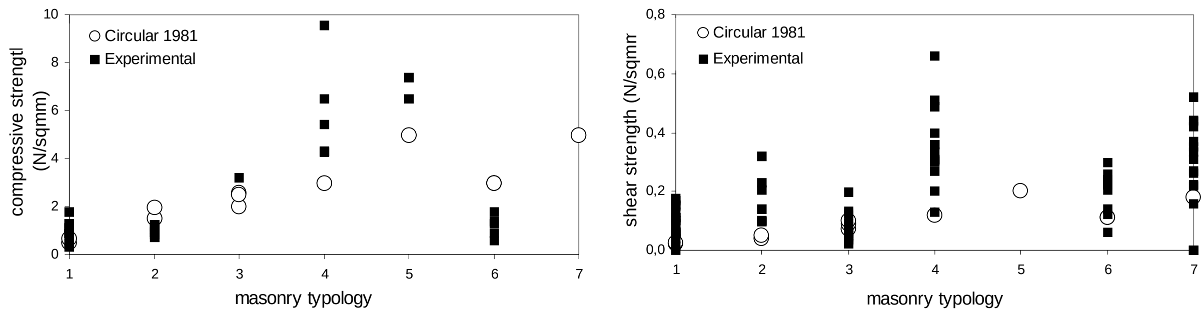

Figure 4: Experimental vs. suggested values of ultimate stresses for various masonry typologies

The procedure is well known and it has been implemented in various commercial software products, with several improvements. In the following, the simplest procedure will be described. The following steps should be done: (i) evaluation for each wall i of the maximum shear force $T_{ui} = A_i \tau_k \sqrt{1+\frac{\sigma_{oi}}{1.5\tau_k}}$, where σ_{0i} is the mean compressive stress (usually the worst combination is that leading to the minimum vertical load); (ii) distribution between the various walls of the seismic forces (depending on the unknown value of C); (iii) by equating the applied seismic force with the maximum shear force for the worst case, one obtains the required value of C. In this procedure the ductility of the walls has been neglected. Usually the consideration of the masonry ductility (limited to 1.5 – 2) does not changes very much the calculated C value.

5.2 Deformable floors

In the deformable-floors option, the seismic loads are distributed proportionally to the influence area of each wall (Fig. 5). In this case, the stiffness of each wall does not influence the seismic forces distribution. The global failure of the structure is attained when one of the walls is subjected to its maximum shear strength; it is obvious that, in this case, the masonry ductility cannot be exploited. The calculation procedure is similar to that described for the rigid floors option, without distributing the forces between the walls proportionally to their stiffness.

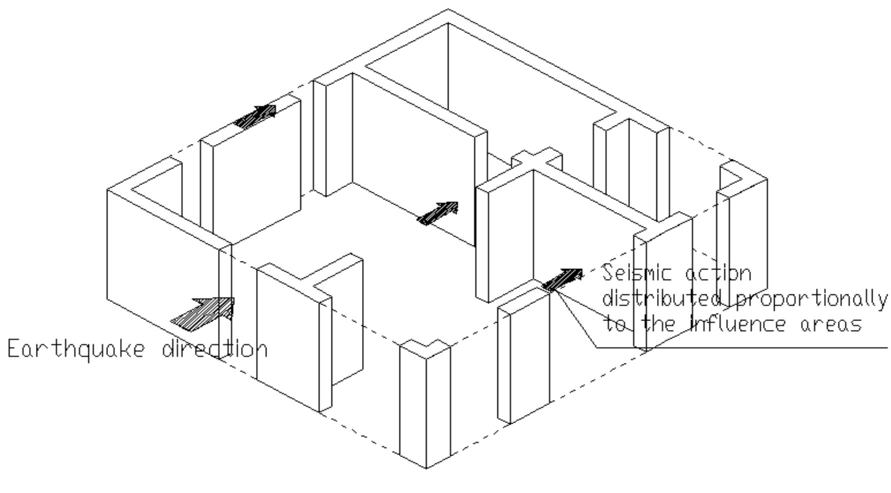

Figure 5: Distribution of the seismic forces with the deformable floors option

6. OTHER VERIFICATIONS

Further verifications are needed to assure the seismic performance of the building The maximum axial force a steel tie can exert is $T = f_y A$, where f_y is the tensile yielding strength and A is the anchorage cross section area. This force should be utilised during the out-of-plane verifications of the walls.

Also the foundations safety should be evaluated. To this purpose, the national (1996) regulations criteria should be followed, reminding that, at the present, only allowable stresses verifications are applicable for the foundations. For this reason, the above calculated seismic actions must be divided by two and the vertical loads must be calculated with γ_g, γ_p and γ_q equal to one.

7. CONCLUSIONS

A verification procedure for the masonry building has been proposed, which starts with the identification of the most likely failure mechanisms and is completed by the numerical evaluation of the limit value of the seismic action activating each of them. In the complete work, the approach of the 1981 Basilicata-Irpinia post-earthquake regulations was compared to that of the 1987 decree and to other methods proposed by some researchers. By some numerical applications, a good agreement of the results between the two regulations was observed, with the exception of the check of the walls with low axial loads, where an important section cracking is indicated by the 1987 decree. For shortness, here only the 1981 regulations approach has been reported.

The use of the calculus has been sometime considered questionable for ancient structures, which have complex behaviours and do not have clear properties proper of engineered constructions (e.g. strength parameters, plane sections and so on), but the identification of the most critical failure mechanisms is, in any case, a valuable tool for the designer to detect the most important vulnerability sources and the ranking of the most effective intervention. Furthermore, the security levels assessed are more homogeneous between different cases, keeping uniform the risk, a scope which is also supported by the attention paid to the site effects.

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