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SEISMIC RELIABILITY OF MASONRY STRUCTURES STRENGTHENED WITH FRP MATERIALS

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ABSTRACT

The problem of the seismic reliability assessment of old masonry wall systems strengthened with fiber reinforced polymers (FRP) is of great practical concern. Recently, the use of these high strength composite materials, like Carbon or Glass fiber composites, has considerably increased in the field of structural repair. The applications in the field of architectural heritage are mainly intended to prevent the loss of the construction (with its frescoes, ornaments, sculptures), as a result of exceptional actions like earthquake.

However, the application of FRP external reinforcements with elastic-brittle behaviour on low ductility structural elements like masonry walls induces a further reduction of the componential ductility, moving the system toward an ideal elastic-brittle behaviour. This involves a careful examination of the brittle components "bundle effect" in order to evaluate the change in the reliability of the structure as a whole.

In this study, a great number of existing ordinary masonry buildings located in Italy are considered and the wall organizations are examined in detail. The selected buildings are sorted within different architectural typologies in order to have a representative sample of structural systems, show structural regularity along the height, and count typically one or two rigid floor levels allowing for proportional load sharing among the structural walls in the elastic range.

The ultimate strength and displacement of wall systems is evaluated by means of a displacement driven non linear analysis (pushover analysis). Then, the structural strengthening with elastic brittle behavior like FRP strips is considered, and the increased shear strength of each masonry wall is provided, computing finally the building force-displacement relationship up to failure.

The statistical analysis of the population's performance points out with strong evidence the conflicting effect of strength and brittleness of FRP materials in enhancing the seismic reliability of complex low strength stone and masonry structures.

As this preliminary study shows, the effective safety increment of the repaired system is seriously limited by its reduced ductility. The presented results seem to predict that very small reliability increments can be produced even applying huge FRP reinforcement ratios, so that for a class of buildings characterized by very low ductility, the strengthening with FRP is simply unfeasible.

INTRODUCTION

In this paper the problem of the seismic reliability assessment of ordinary masonry wall systems strengthened with fiber reinforced polymers (FRP) is considered. Recently, the use of these high strength composite materials, like Carbon or Glass fiber composites, has considerably increased in the field of structural repair of both recent and historical masonry buildings. The applications in the field of architectural heritage are mainly intended to prevent the loss of the

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construction (with its frescoes, ornaments, sculptures), as a result of exceptional actions like earthquake. In case of such events, the net of reinforcing elements must allow the structure to do not collapse.

However, the application of FRP external reinforcements with elastic-brittle behavior on low ductility structural elements like masonry walls induces a further reduction of the componential ductility, (Tinazzi *et al*., 2000, Tumialan *et al.,* 2002, Bonfiglioli *et al.* 2003) moving the system toward an ideal elastic-brittle behavior. This involves a careful examination of the brittle components "bundle effect" in order to evaluate the restoration effect on the reliability of the system. In fact, the simple evaluation of the safety margin of each structural element does not lead directly to a reliability judgment on the structure as a whole (Melchers 1999). As is well known, in case of brittle elements, the collapse load of the system can be significantly lower than the sum of the single structural element collapse loads (Daniels, 1945).

MASONRY WALL SYSTEMS

In this study, a number of 39 existing ordinary masonry buildings are considered and the relative structural systems are examined in detail. The selected buildings are sorted within different architectural typologies in order to have a representative sample of structural systems in terms of age, material quality, building technique and dimensions. All of the selected examples show structural regularity along the height, and count typically one or two floor levels only, in order to consider an equivalent one-floor shear wall system. Finally, the assumption of rigid floor panels allows to account a uniform load shearing among the structural walls in the elastic range.

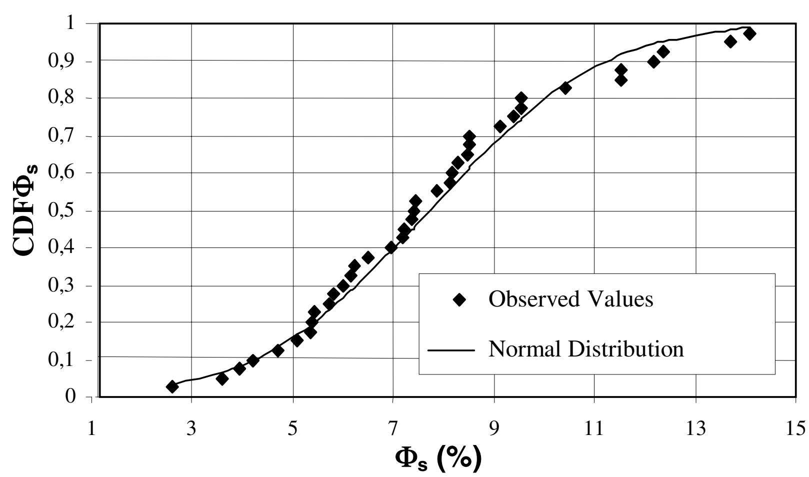

Figure 1: Cumulative distribution of Φ_s

In order to show the main representative characteristics of the considered systems, the following geometrical factors are defined:

– *area factor (Φ_s):* ratio between the resisting walls area at the basement and the basement area;

– *eccentricity factor (Φ_e):* ratio between the eccentricity between the stiffness and the mass centres at the first floor and the basement area square root.

The statistical cumulative distributions of Φ_s and Φ_e are shown in Fig.s 1 and 2. It can be observed from the figures that the selected examples typically represent the field of the Italian traditional masonry buildings.

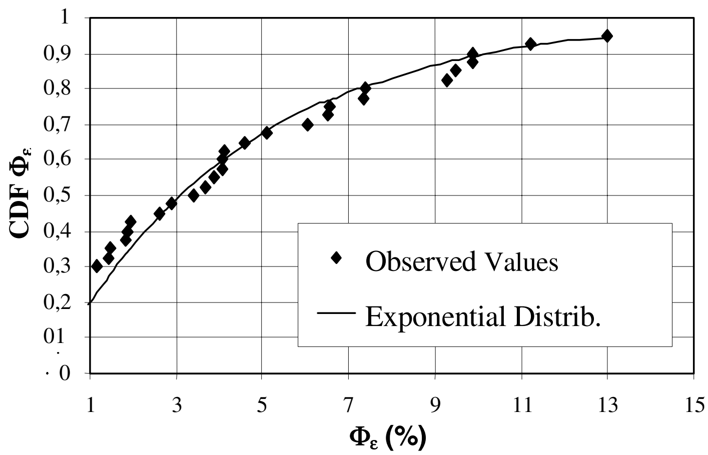

Figure 2: Cumulative distribution of Φ_e

STRENGTH DISTRIBUTIONS OF STRUCTURAL ARRANGEMENTS

The ultimate strength and displacement of each structural system is evaluated by means of a displacement driven nonlinear analysis (pushover analysis), accounting for a single direction displacement at a time and the coupled rotation due to the effective and accidental eccentricities,. Also, the performed pushover analysis allows to compute the building force-displacement relationships, where the gravity centre of the first floor is selected as a control point for the displacement, and the total shear force at the basement is the output force.

The constitutive model of the *k-th* shear wall is computed in accordance with the POR method (Braga, 1977). The strength R_k (Benedetti and Ceccoli, 1997) and the stiffness K_k are derived as follows:

$$R_k = A_k \frac{\sqrt{\left(1-\frac{\sigma_{vk}}{f_{ck}}\right)\left(1+\frac{\sigma_{vk}}{f_{tk}}\right)}}{\frac{1}{f_{ck}}+\frac{1}{f_{tk}}}, \qquad K_\mathrm{k} = \frac{A_k G_k}{1.2 h_k}\left(1+\frac{1}{6}\left(\frac{h_k}{b_k}\right)^2\right)^{-1}, \qquad (1, 2)$$

where, for the *k-th* wall, σ_{vk} is the vertical normal stress, f_{ck} and f_{tk} the compressive and tensile strengths, A_k the resistant area, G_k the elastic shear modulus, h_k and b_k respectively the height and the length in the considered direction. The maximum elastic displacement d_k is computed as ratio between R_k and K_k; the ultimate displacement is computed assuming a standard ductility $\eta_k = 1.5$.

Then, the structural strengthening with elastic brittle behaviour like FRP strips is considered, and an increased shear strength of each masonry wall is accounted. For each wall, different levels of shear reinforcement are applied in order to obtain a final strength increment of the 10, 20, 30, 40, 50% of the original strength. On the other side, for each building, a different number of shear wall are strengthened in order to repair the 25, 50, 75 and 100% of the resisting shear walls. In Fig. 3, a schematic representation of the constitutive model assumed for un-reinforced masonry walls (URM), and externally strengthened masonry walls (ESM) is reported.

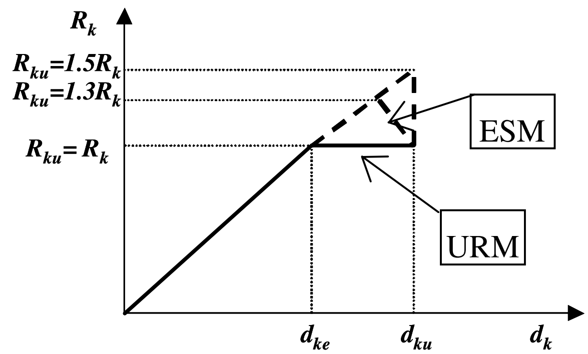

Figure 3: Assumed constitutive models for URM and ESM walls

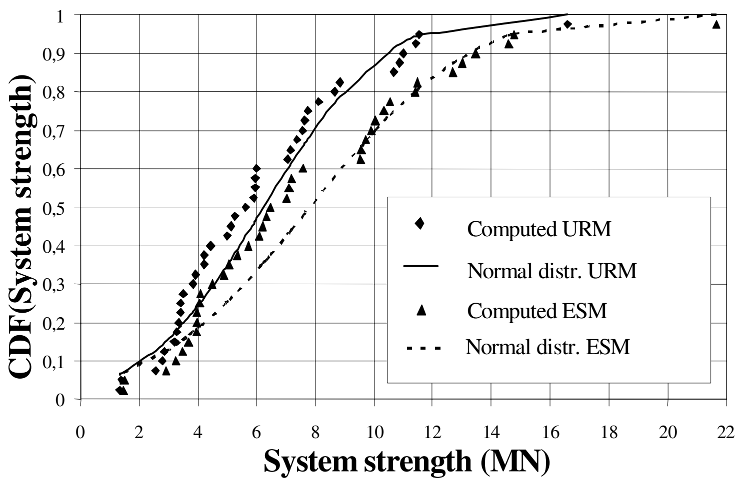

Figure 4: Cumulative distribution of the URM and ESM building strengths

Making use of the pushover analysis, the overall structural response is computed for the considered examples (original and strengthened systems) and the obtained results are reported in Fig. 4 in terms of statistical cumulative distributions. The choice of the probability distributions is founded on the best fit on computed values and is verified by means of the standard χ^2 test (Taylor 1986, Lewis 1994); the computed values frequencies agree with the Normal theoretical distribution at a level of significance $P(\chi^2 \geq \chi_0^2) < 5\%$.

As is well known, when a finite number of structural elements are composed in parallel, the componential ductility degree influences the potential benefit offered by the active redundancy of the system. In fact, only the ideal plastic system provides the largest reliability achievable through redundancy, which renders the total strength of the system equal to the sum of all the component strengths (Gollwitzer and Rackwitz, 1990).

The considered systems redundancy is evaluated in terms of total strength loss or *bundle effect*. For URM and ESM systems, by denoting with R_{sys} the overall system strength, the bundle effect coefficient (BE) is computed as follows:

$$BE = \frac{\sum R_k - R_{sys}}{\sum R_k}, \quad (3)$$

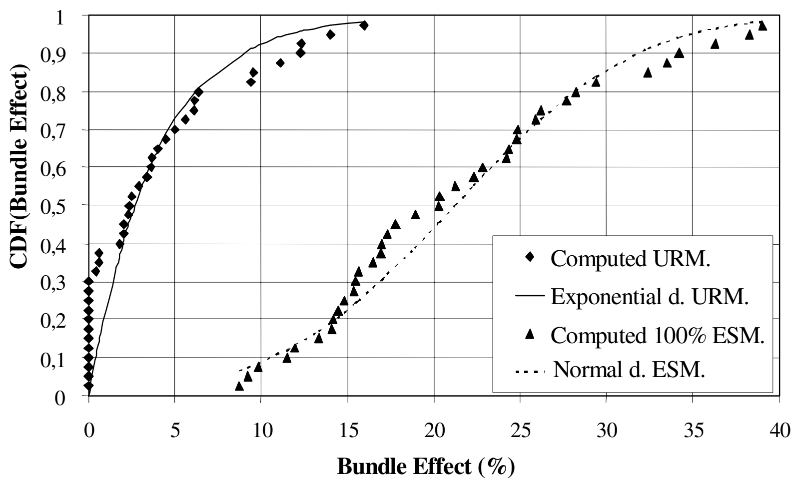

Figure 5: Cumulative distribution of the strength loss for URM and ESM wall systems

The obtained results are shown in Figure 5. Thanks to the even low ductility of the masonry walls, the original structural systems exhibit a limited strength reduction, with mean value smaller than 3% and maximum value around 15%. As expected, a not negligible strength reduction can be observed for fully strengthened system, with mean value around 20% and maximum value 40%. This sensible increase of the bundle effect is due to the FRP reinforcement that activates a more brittle behaviour of the reinforced walls and consequently reduces the overall system ductility (Figure 6).

By a careful examination of the obtained bundle effect for the original systems, two masonry building main classes can be defined:

Class I: structural systems with BE ≤ 5 %

Class II: structural systems with BE > 5 %.

This classification results to be very effective for design purposes. In order to evaluate the strength deterioration effect, an Efficiency Index (EI) is then introduced:

$$EI = \frac{\Delta R_{sys}}{\Delta R_{sys} + BE}, \tag{4}$$

where the reinforced system strength increment ΔR_{sys} is given as follows:

$$\Delta R_{sys} = \frac{R_{sys,FRP} - R_{sys}}{R_{sys}}. \tag{5}$$

The EI index is always bounded between 0 (null strength increment) and 1 (null strength loss); when EI ≤ 0.5, the strength increment is equal or less than the strength loss and, in this case, this repair technique cannot be considered efficient enough for the structural repair.

In Figure 7, the obtained EI values are plotted for building class I and II. As can be drawn from the figure, the building class I raises sensibly higher values of EI. In fact, the 75% of the structural systems attain EI > 0.5 in class I while only the 38% in class II. This seems to address a possible design criterion able to select the optimal repair technique for URM structural systems.

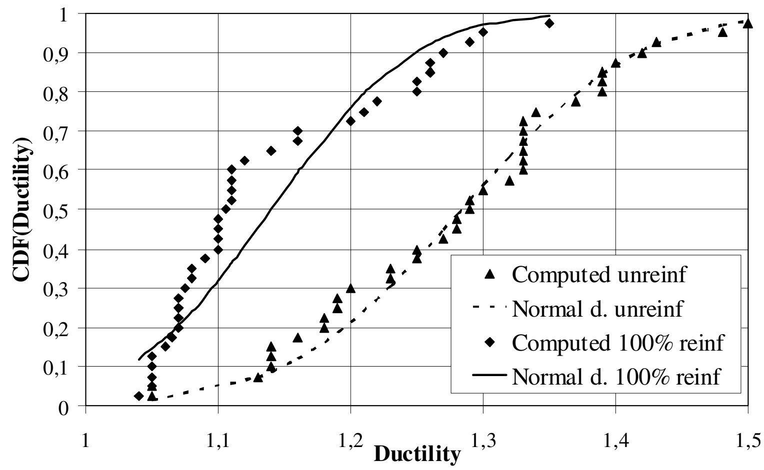

Figure 6: Cumulative distribution of the overall ductility for URM and ESM wall systems

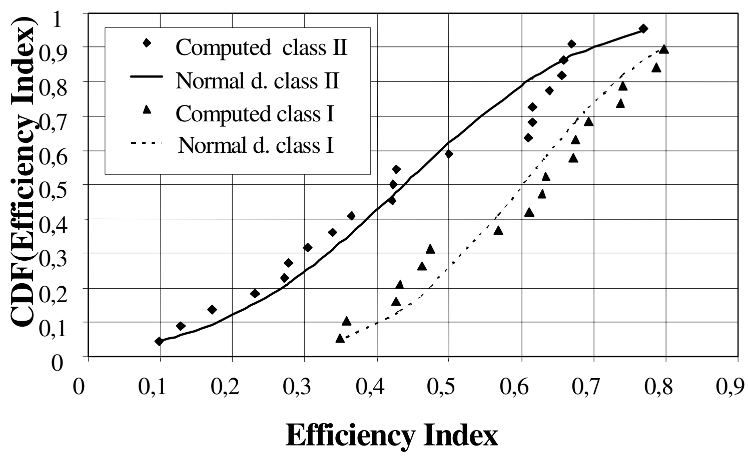

Figure 7: Cumulative distribution of the Efficiency Index for ESM wall systems of both classes.

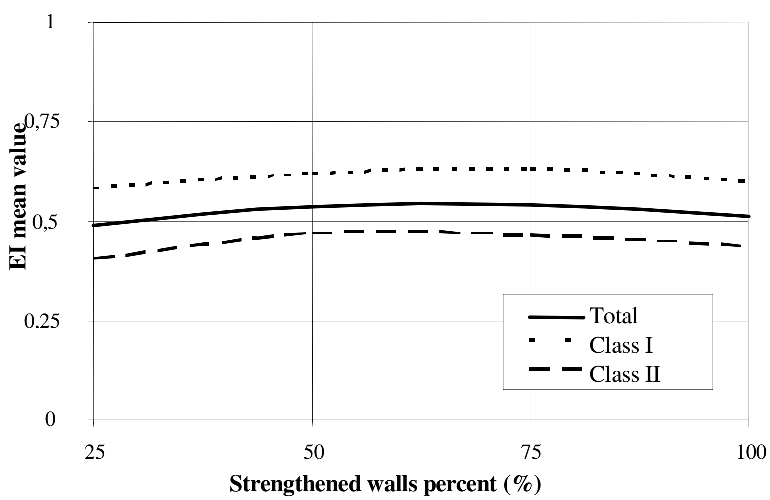

Figure 8: Variability of the Efficiency Index mean value with the ratio of ESM walls

With reference to Fig. 8, it is interesting to observe that the efficiency index mean value of both the building class I and II does not present appreciable variations increasing the percent of the strengthened walls. Since the EI mean value does not further increase when the strengthened wall percent exceeds 50%, it is reasonable to assess an optimal strengthening percent in order to balance the cost to benefit ratio.

SEISMIC FORCE DISTRIBUTIONS

The seismic force evaluation is conducted making use of the design seismic response spectrum as defined by OPMC 3274, 2003. A medium soil quality (class B) and a standard structural damping (5%) are adopted in this analysis.

For sake of generality, three different Italian seismic zones are considered, with different values of horizontal peak ground accelerations, and respectively:

Zone I: PGA = 0.35 g, *Zone II*: PGA = 0.25 g, *Zone III*: PGA = 0.15 g

where g is the gravitational acceleration.

In the classified seismic zones, the assumed PGA held a probability of 10% to be overcome in 50 years; i.e. to be overcome once in 500 years. This seismic level is generally accepted by the international technical community in order to prevent the overall or even partial collapse of the structure. (Bertero and Bertero 2002, Trombetti *et al.* 2003).

For the considered examples, a behaviour factor q=1 is always assumed. The q factor has been computed making use of the performed pushover analysis and following the N2 method proposed by Fajfar (2000, 2003). In this framework, the nonlinear constitutive relation of each structural system (single degree of freedom or equivalent system) is transformed into a pseudo acceleration-displacement relation that can be compared with the elastic seismic acceleration spectrum expressed as a function of the spectral displacement. Finally, once the vibration period T^* is defined for the structural system, the q factor is given by the ratio between the elastic spectral acceleration S_{ae} evaluated in T^* and the maximum pseudo-acceleration S_{ay} of the structural system: By denoting with m_{sys} the total mass of the system, we have:

$$q = \frac{S_{ae}(T^*)}{S_{ay}} = S_{ae}(T^*)\frac{m_{sys}}{R_{sys}}. \qquad (6)$$

For the examined structural systems, $T^* < 0.1$ s is always found, since the masonry low-rise buildings belong to the field of very stiff structures.

It is to note that the seismic force cumulative distributions are described by Gumbel and Normal distributions depending on the selected seismic zone. Again, the choice of the probability distributions is founded on the best fit on computed values and is verified by means of the standard χ^2 test, and the computed values frequencies agree with the selected theoretical distribution at a level of significance $P(\chi^2 \geq \chi_0^2) < 5\%$. Figure 9 resumes the seismic force values obtained for the considered examples.

Once the probability density functions f of the resistance R and load S random variables are known for a considered structural system, the failure probability of the system can be derived as follows (Melchers, 1999):

$$p_f = P(R - S \leq 0) = \int_{-\infty}^{\infty} F_R(x) f_S(x) dx \quad , \tag{7}$$

where:

$$F_R(x) = P\left(R \leq x\right) = \int_{-\infty}^{x} f_R(y) dy \quad . \tag{8}$$

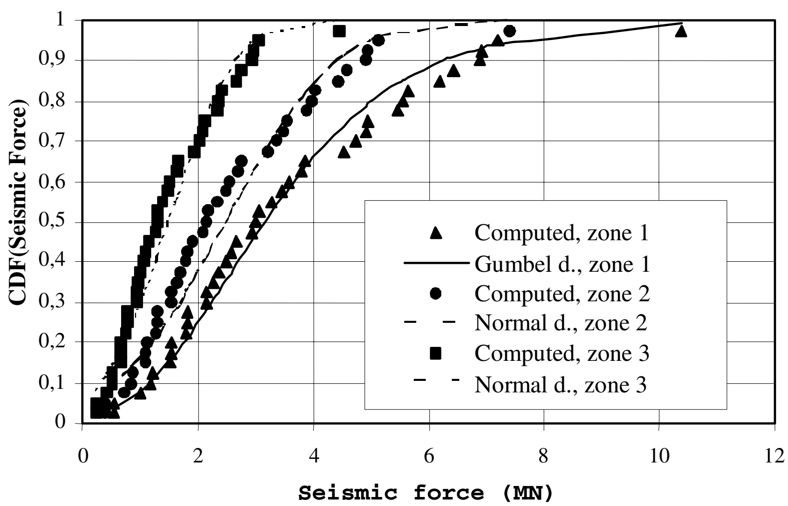

Figure 9: Cumulative distribution of the seismic force reliability index evaluation

If a safety margin function is defined as $Z = R - S$, and R and S are both normal random variables, Eq. (7) can be written as:

$$p_f = P(Z \leq 0) = \Phi\left(-\frac{\mu_Z}{\sigma_Z}\right) = \Phi\left(-\beta_C\right) \tag{9}$$

where Φ is the standard normal distribution function (zero mean and unit variance); μ_Z, σ_Z^2 are respectively the mean value and the variance of the safety margin random variable and β_C is the Cornell Reliability Index (or Safety Index), implicitly defined by Eq. (10). When R and S are both normal random variables:

$$\beta_C = \frac{\mu_Z}{\sigma_Z} = \frac{\mu_R - \mu_S}{\sqrt{\sigma_R^2 + \sigma_S^2}} \,, \tag{10}$$

otherwise, the approximation $\beta_{HL} = -\Phi^{-1}(p_f)$ can be properly assumed (Madsen et Al., 1986).

In the present paper, the β coefficient is adopted to assess the safety conditions of the analysed masonry buildings and to compare the performances of the original structural systems with the strengthened systems. In Figure 10, the probability density functions of the system strength and the seismic force for the considered examples are reported. It is interesting to observe that the strength probability distributions of the original and fully (100%) reinforced masonry systems differ of around 25% in the mean value.

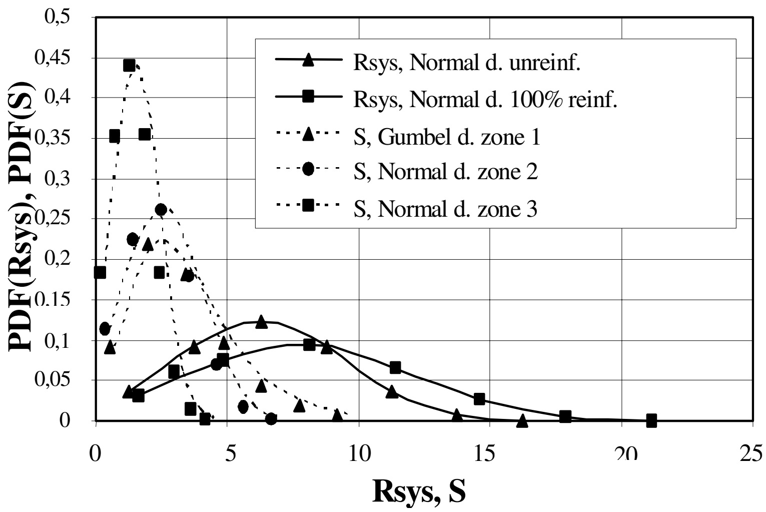

Figure 10: Probability density function of system strengths (R_{sys}) and seismic forces (S)

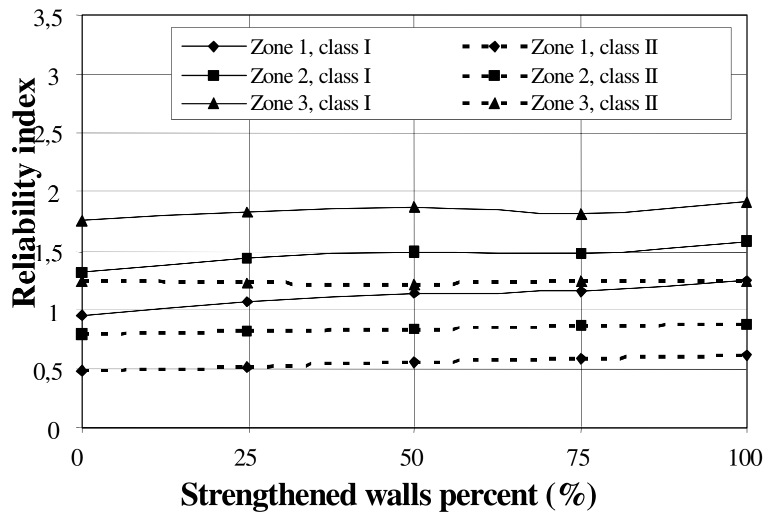

Figure 11: Reliability index for building class I and II

The Reliability Index is then computed making use of Eq.(11) or (12) for building class I and II, original and reinforced with FRP, subjected to seismic actions of Zones 1, 2 and 3. The obtained β values are reported in Figure 11 as a function of the strengthened wall ratio.

Two main observation can be drawn form the figure:

a) For increasing strengthening ratio, the β coefficient increase is relatively small for building class I and very small for building class II; this trend highlights the efficiency lack already detected by means of the efficiency index analysis previously exposed.
b) For any considered seismic zone, $\beta < 2$ ($p_f > 10^{-2}$) for building class I, $\beta < 1.5$ ($p_f > 6\ 10^{-2}$) for building class II, which mean unacceptable reliability values for ultimate limit state condition (requested $\beta \approx 3$, $p_f \approx 10^{-3}$).

Alternately, a different representation of the reliability problem can be given by subdividing the results in the old buildings and new buildings classes. After careful recognition the year 1950 has been used as a subdivision mark. However no definite trend in the response of the two classes with respect to the strengthening ratio can be inferred.

This is probably due to the casual belonging of new and ancient buildings to fragility class I and II.

CONCLUSIONS

The presented results bring to very interesting considerations. A first question arises concerning the real possibility to improve the structural safety of masonry shear walls systems by means of FRP applications. As this preliminary study shows, the effective safety increment of the repaired system is seriously limited by its reduced ductility. Since ductility is a great safety source for seismic applications, any loss of ductility may involves severe consequences in terms of structural safety. The observed trend of the reliability index gives warning on the real effectiveness of this strengthening technique for this kind of application. In fact, the presented results seem to predict that very small reliability index increments can be produced even applying higher reinforcement ratios. The two identified building classes potentially address the design solutions: in fact, class II buildings are characterized by a rather brittle overall behavior and do not seem to be suitable at all for FRP strengthening techniques.

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