Accounting for the Effect of In-Situ Tests and Inspections on the Performance Assessment of Existing Buildings

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ABSTRACT:
One of the most challenging aspects in the study of existing buildings is the evaluation of the influence of modeling uncertainties on seismic performance. Modern codes seem to synthesise it in a deterministic, Confidence Factor although the effect of its application on structural reliability is not explicit. In this paper a Bayesian framework is adopted in order to both characterise the uncertainties playing a major role in influencing the seismic performance of existing buildings and also to update their joint probability distribution, taking into account for the results of tests and inspections. An efficient method is used to estimate the probability of failure of a case study structure, an Italian pre-seismic code building, for which a small number of tests is available. Moreover, a method for considering the heterogeneity (measurement errors) of tests and inspections is proposed. The main purpose of this paper is to study the effect of the number of tests and inspections on the reliability assessment of existing buildings.

Keywords: Existing Buildings, Performance Assessment, Bayesian Updating, In-situ Tests, Confidence Factor.

1. INTRODUCTION

The main feature distinguishing the assessment of existing buildings from that of new construction is the significant amount of modeling uncertainties involved in the assessment. Recent European codes, such as Italian NTC2008 (CS.LL.PP., 2008) and Eurocode 8 (CEN, 2003), consider the overall effect of uncertainties defining a so-called Confidence Factor, whose value depends on the knowledge level achieved by performing tests and inspections on the existing structure. Dividing the mean value of material resistances by the code-specified confidence factor is intended to guarantee a certain level of conservatism and to take into account for all the involved uncertainties. Nevertheless, with the emerging of concepts such as performance based design and life-cycle cost, an increasing attention has been posed on the effect of confidence factor in guaranteeing a specific seismic reliability. As argued by some researchers (Monti and Alessandri, 2008; Jalayer et al., 2009), this would be possible only through a probabilistic approach. In fact, the application of confidence factor appears to be a deterministic way for addressing an inherently probabilistic problem and the logical connection between the numerical value of this factor and the level of knowledge needs to be further elaborated. Moreover, as it can be observed from Tab. 1.1., the confidence factor is independent from the outcome and spatial configuration of tests and inspections. The Bayesian framework for statistical inference seems to be particularly suitable for taking into account the results of tests and inspections, leading to both updated probability distributions for structural modeling parameters and also to updated structural reliability estimation, as a function of data collected on-field. Jalayer et al. (2010a) have already demonstrated the application of advanced simulation methods in updating both the uncertain parameter and structural reliability, resulting in a robust estimate of probability of failure. Moreover, an efficient method has been proposed by Jalayer et al. (2010b), which uses of a small set of structural analyses (in the order of 10-30) to yield the estimate of the "robust" structural reliability, i.e. calculated by considering all possible structural models and their relative plausibilities (Au and Beck, 2002). In this paper, these concepts are applied to a case study, in order to provide a probabilistic framework for considering modeling uncertainties, with particular emphasis on the uncertain parameters playing a major role in influencing seismic performance and on the effect of the number of tests and inspections on the structural reliability.

Table 1.1. Recommended minimum requirements of inspections and tests for different KL

Knowledge Level	**FC value**	**Inspections of reinforcement details (% structural elements)**	**Testing of Materials (sample/floor)**
Limited	1.35	20	1
Extended	1.20	50	2
Comprehensive	1	80	3

1.1 The Structural Performance Parameter and the Structural Reliability

The structural performance parameter is defined herein in terms of a critical demand to capacity ratio, denoted by Y and the considered limit state is the severe damage limit state, as defined by the Italian code. According to this, the critical threshold for the structure coincides with the onset of a chord rotation larger than the $3/4^{th}$ of the corresponding ultimate chord rotation in the first member. In this context, the failure probability can be calculated as the probability that Y assumes values major than one. In the case of static non-linear analysis, the Capacity Spectrum Method by Fajfar (CSM) (Fajfar, 1999) is used to obtain the critical demand to capacity Y. Moreover, particular attention has been posed on shear, since existing buildings are generally characterized by insufficient shear reinforcement details and fragile failure modes are expected to dominate. At the onset of the limit state, the shear demand to capacity ratio is also calculated for all the structural components, then the critical performance parameter $Y_{critical}$ is taken as the maximum between the global ratio $Y_{flexural}$, obtained by CSM, and the critical shear component ratio Y_{shear}.

2. METHODOLOGY

2.1. Characterizing the Uncertainties

Generally speaking, the sources of uncertainty involved in the assessment of the seismic performance of existing buildings can be grouped as: (i) uncertainties in the load conditions, such as the ground motion record-to-record variability; (ii) modeling uncertainties related to the finite element modeling of the structure and uncertainties in component capacity models, (iii) uncertainties in the structural modeling parameters. This work is focused on the uncertainties in the structural modeling parameters which reflect the level of knowledge about the existing structure. This type of uncertainty is believed to be implicitly addressed by the application of confidence factor. In particular, two kinds of structural modeling uncertainties are taken into account: the uncertainty in the mechanical properties of materials (e.g., concrete compressive strength and steel yielding strength) and the uncertainty in structural construction details, also known as “structural defects”, since possible deviations from the original configurations are mostly taken into account in those cases leading to undesirable effects. In the present work, particular attention is focused on those construction details (the stirrup spacing and diameter) which seem to mainly affect the shear behavior of the structure. Prior probability distributions for the uncertainties reflect the state of knowledge before performing any test and inspection on the structure. These probability distributions can be defined from qualitative information coming from expert judgment or based on ignorance in the extreme case (Jalayer et al., 2010a). The parameters identifying the prior probability distributions for the material mechanical properties have been gathered from typical values of the post world-war II constructions in Italy (Verderame et al. 2001a,b). Tab. 2.1. illustrates the parameters used to define the lognormal prior probability distributions for mechanical properties and the uniform probability distributions for the construction details. The uncertainties in the structural modeling parameters are then propagated, using efficient simulation-based reliability methods, in order to obtain a probability distribution for the global structural performance given the code-specified seismic spectrum. It is assumed that the material properties are homogeneous for each floor (reflecting the usual construction time-line of the structure). As far as it regards the construction details, it is assumed that the corresponding uncertainties are systematic for the whole structure.

Table 2.1. Prior distributions and homogeneity assumptions (COV: coefficient of variation)

Uncertainties	Prior distribution	type
concrete compressive strength f_c	LN(median: 165 kg/cm^2, COV: 0.15)	Systematic over floor
Steel yielding strength f_y	LN(median: 3200 kg/cm^2,COV: 0.08)	Systematic over floor
Stirrup spacing	Uniform PDF; interval: [20cm, 35cm]	Systematic
Stirrup diameter	Uniform PMF; values: {6mm,8mm}	Systematic

In the following, the procedure of updating the probability distributions for the material properties is outlined, using the concrete compressive strength as an example (the procedure is identical for the steel yield strength). Let f_c denote the median concrete strength over the floor (the characteristic value used in the structural model) and D denote the set of available data for the concrete strength. The updated probability distribution for f_c , given D, can be calculated by the Bayes' theorem:

$$p(f_c|D) = \frac{p(D|f_c) \cdot p(f_c)}{\int p(D|f_c) \cdot p(f_c) \cdot df_c} \tag{2.1}$$

In the previous formulation, it is assumed that the coefficient of variation of the concrete strength is fixed (e.g., C.O.V.=0.15) and that $p(f_c)$ represents the prior probability distribution for concrete strength. If the gathered data are assumed to be independent, the likelihood function $p(D|f_c)$ can be calculated as:

$$p(D|f_c) = \prod_{i=1}^{N} p(D_i|f_c) \tag{2.2}$$

where N is the number of data. In order to deal with the measurement error associated with observation D_i, the likelihood function $p(D_i|f_c)$ can be expanded with respect to the exact value d using the total probability theorem:

$$p(D_i|f_c) = \int p(D_i|d) \cdot p(d|f_c)\ dd \tag{2.3}$$

In this work, it is assumed that the destructive drilled-core tests and steel tension test have no measurement error associated with them. Meanwhile, for the ultrasonic tests, it is assumed that the measurements are un-biased and are modeled by a lognormal distribution; that is, $D = d \cdot \varepsilon$, where ε has unit median and a standard deviation (of the natural logarithm) equal to σ. Moreover, in order to take into account for the uncertainty in the estimation of the standard deviation σ for the probability distribution $p(D_i|d)$, it can be further expanded with respect to σ:

$$p(D_i|f_c) = \iint p(D_i|d,\sigma) \cdot p(\sigma) \cdot p(d|f_c)\ dd\ d\sigma \tag{2.4}$$

With regard to the stirrup spacing, it is assumed that the probability of not observing the presence of defect in a member (that is to say that the design prescription is confirmed by inspection) is equal to f, the probability distribution for f can be updated using the inspection results by Bayes' formula:

$$p(f|D) = \frac{p(D|f) \cdot p(f)}{\sum_f p(D|f) \cdot p(f)} \tag{2.5}$$

The prior distribution for f denoted by $p(f)$, in absence of information, can be assumed uniform between 0 and 1, otherwise, expert judgment can be employed to limit the upper and lower bounds and/or use a more informative form of the prior distribution. In particular, if the inspections indicate that, out of n cases, n_d of them demonstrate a defect, the likelihood function $p(D|f)$ can be calculated using binomial distribution:

$$p(D|f) = \binom{n}{n_d} \cdot (1-f)^{n_d} \cdot f^{n-n_d} \tag{2.6}$$

With regard to the stirrup diameter, the same approach presented above for the stirrup spacing is adopted with the difference that *f* denoted the probability of inspecting diameter ϕ6 and *1-f* denotes the probability of inspecting diameter ϕ8 (only two possibilities are considered).

2.2. Efficient Method for Structural Reliability

The efficient method proposed in (Jalayer et al., 2010b) is used to obtain a robust estimate of probability of failure for the structure. In fact, as demonstrated by the authors, this method employs a small set of structural analyses in order to estimate the posterior probability distributions for median and standard deviation (of the natural logarithm) of the structural performance parameter *Y*. More specifically, the robust failure probability is estimated by calculating the posterior expected value and standard deviation of the probability of failure, based on the posterior probability distributions resulting from updating the structural model. Assuming χ to be the set of parameters describing the distribution of probability of the structural performance parameter *Y* (e.g. the median η_Y and standard deviation β_Y of a lognormal distribution), the probability of failure, given χ, is denoted by $P(F|\chi)$. The expected value, or robust estimate, for the probability of failure, given a set *Y* of structural performance parameters, calculated for a small sample of structural models ($Y = \{Y_1, Y_2, \dots Y_n\}$ is the vector of n different realizations of the structural performance parameter), can be expressed as:

$$E[P(F|Y)] = \iint P(F|\eta_Y, \beta_Y) \cdot p(\eta_Y, \beta_Y\,|Y)\, d\eta_Y\, d\beta_Y \qquad (2.7)$$

In the previous formulation $P(F|\eta_Y, \beta_Y)$ is the structural reliability, or the probability of failure, and $p(\eta_Y, \beta_Y\,|Y)$ is the posterior probability distribution for the set of parameters χ, given the set of *Y* values, obtained from bayesian inference assuming both median and standard deviation for *Y* unknown, as reported in (Box and Tiao, 1992).

3. NUMERICAL EXAMPLE

3.1. Case Study Structure

The considered case study is a scholastic building built in the sixties and designed for gravity loads only, located in Avellino (Italy), seismic zone II according to the Italian seismic guidelines (OPCM 3519, 2006). The structure is composed of three stories and a semi-embedded storey, and its foundation lies on soil type B according to EC 8 classification. The structure is irregular both in elevation and in plane, columns have all rectangular sections variable from 40x55 cm^2 at the first level to 30x30cm^2 at the last one and beams are rectangular too, with a cross section area variable from 30x80cm^2 to 30x50 cm^2, violating the *strong-column-weak-beam* principle because inadequate code at the time of construction. For the structure in question original design graphics and notes have been gathered; from them, it has been inferred that steel rebars are of type Aq40 and concrete has a minimum resistance of 165 kg/cm^2 (RDL, 1939). The finite element model of the structure assumes the non-linear behavior concentrated in plastic hinges at the elements end, modeled as by coupling in series an elastic element and a rigid-plastic one. The rigidity of this latter is defined by the moment-rotation diagram obtained from the study of the section at hinge location. The widely adopted Mander-Priestly (Mander et al.,1988) constitutive law has been considered for the unconfined concrete, while an elastic-perfectly plastic behavior has been adopted for steel. The plastic hinge diagram is composed of four phases, namely, rigid, cracked, post-yielding and post-peak, and the yielding rotation takes into account not only the flexural deformation but also the shear one and the bar slip, according to (CS.LL.PP., 2008). In the present work, the outcomes of in-situ tests and inspections performed on the structure, are used. These consist of two destructive tests and eight ultrasonic measurements for each floor and five inspections on reinforcement details for the whole building. Tab. 3.1. shows a summary of abovementioned tests and inspections.
Regarding the steel strength, only one test is available, resulting in a yielding strength equal to 2962 kg/cm^2. These values are used to subsequently update prior distributions for uncertain parameters, then uncertainties are propagated using simulations to estimate structural reliability.

Table 3.1. Destructive (drilled core) and non-destructive (ultrasonic) tests on concrete and inspections on details

Concrete - 1st Floor			Concrete - 2nd Floor			Concrete - 3rd Floor			Reinforcement details		
element	***fc_{ultras}***	***fc_{destr}***	***element***	***fc_{ultras}***	***fc_{destr}***	***element***	***fc_{ultras}***	***fc_{destr}***	***test***	***diameter***	***spacing***
beam	252,5	311,0	beam	237,0	-	column	149,7	-	n.1	$\phi 8$	20
column	265,9	-	column	239,7	-	beam	216,5	169,0	n.2	$\phi 8$	20
column	248,5	-	column	132,9	-	beam	224,9	-	n.3	$\phi 8$	20
column	237,1	236,0	column	170,6	-	column	225,3	-	n.4	-	20
beam	257,9	-	beam	249,4	246,0	column	202,5	-	n.5	-	20
column	225,2	-	column	240,7	-	column	153,4	-	-	-	-
column	223,0	-	column	138,1	145,0	column	190,5	-	-	-	-
column	299,7	-	column	162,1	-	column	196,3	202,0	-	-	-

Figures Fig.3.1.a,b show how the prior distributions for concrete compressive strength and stirrup spacing are updated considering the increasing in knowledge achieved through tests and inspections.

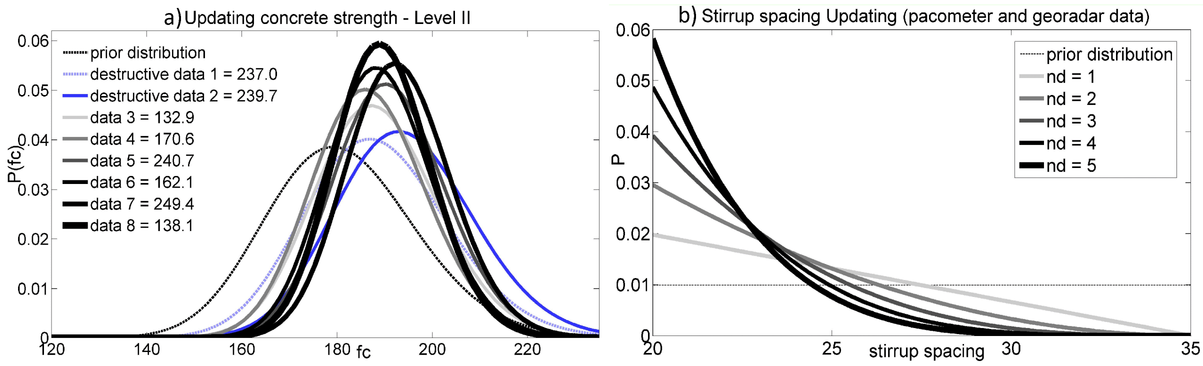

Figure 3.1. Bayesian updating procedure for concrete compressive strength of level I (a) and stirrup spacing (b)

The error in the ultrasonic measurement has been calculated from the values of resistance obtained in those elements for which both drilled-core tests and ultrasonic tests where available. The probability distribution can be obtained using the Bayesian updating procedure:

$$p(\sigma|D/d) = c \cdot p(D/d|\sigma) \cdot p(\sigma) \quad (3.1)$$

Where c is the normalization constant, and $D/d{:}\{D_i/d_i\}$ is the ratio of the ultrasonic to drilled-core measurements performed at the same point within the structure. Assuming unbiased ultrasonic measurements and constant σ over all d values, the statistics of the logarithm of D/d are:

$$median(D/d) = 1; \;\; std(ln(D/d)\,) = std(ln(D|d)) = \sigma \quad (3.2)$$

Than the likelihood function $p(D/d|\,\sigma)$ is calculated as the product of lognormal probability distribution with unit-median and standard deviation of the natural logarithm equal to σ. The prior probability distribution $p(\sigma)$ is assumed to be uniform. This leads to the distribution $p(\sigma|D/d)$ shown in Fig. 3.2. (this distribution is denoted simply as $p(\sigma)$ since it is going to be substituted in Eqn. 2.4.):

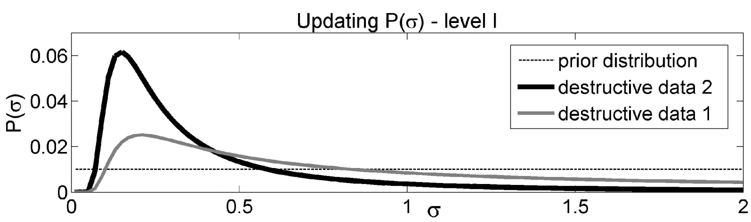

Figure 3.2. Probability distribution for standard deviation

3.2. Calculating Structural Reliability

The structural fragility is estimated employing the above mentioned Efficient Bayesian method, based on structural performance parameter Y for a set of 20 Monte Carlo realizations of the structural model, generated from the updated probability distributions of the uncertain parameters.
Three kinds of analysis are performed: the first set of analyses referred to as "type A", considers only the uncertainties in the material properties, assuming the nominal values for structural details. It is composed of ten levels of updating: starting from the prior distribution of material uncertainties, the structural reliability is obtained at each level by performing 20 simulations. At each level, both the updated probability distributions for the material properties and the updated structural reliability are obtained by considering one more test result with respect to the previous level. The second set of analyses, called "type B", is performed similar to that of "type A" but considering only the uncertainties in the construction details, assuming that the material properties are equal to their nominal values from the original design documents. Type A analyses are based on structural model realizations that are generated based on the probability distributions similar to those shown in Fig. 3.1.a and the type B analyses are based on model realizations that are generated based on the probability distributions similar to those shown in Fig. 3.1.b. The third set of analyses, named "type C" considers both uncertainties in material properties and construction details.

3.3. Analysis Results

The results are presented, for each type of analysis performed, in terms of Cumulative Distribution Function for the structural performance parameter Y, which represents the structural reliability and is equal to one minus the failure probability outlined in Section 1.1. In order to interpret the influence of increasing test results on structural reliability, the structural performance parameter corresponding to a fixed probability, for example the 95%, is chosen as a measure of the global performance of the structure. In Fig.3.3.a the results for type A analysis are shown. It is observed that median and standard deviation of the fragility curves plotted for the structural performance parameter $Y_{flexural}$, tend to reduce progressively as a result of increasing number of tests. More over it can be observed that the reduction of median and standard deviation in levels 1 and 2, in which destructive tests are considered, is more pronounced with respect to the reduction of median and standard deviation in Levels 3 to 10, in which the non-destructive ultrasonic tests are considered. This is to be expected given the error of measurement associated with the non-destructive tests.

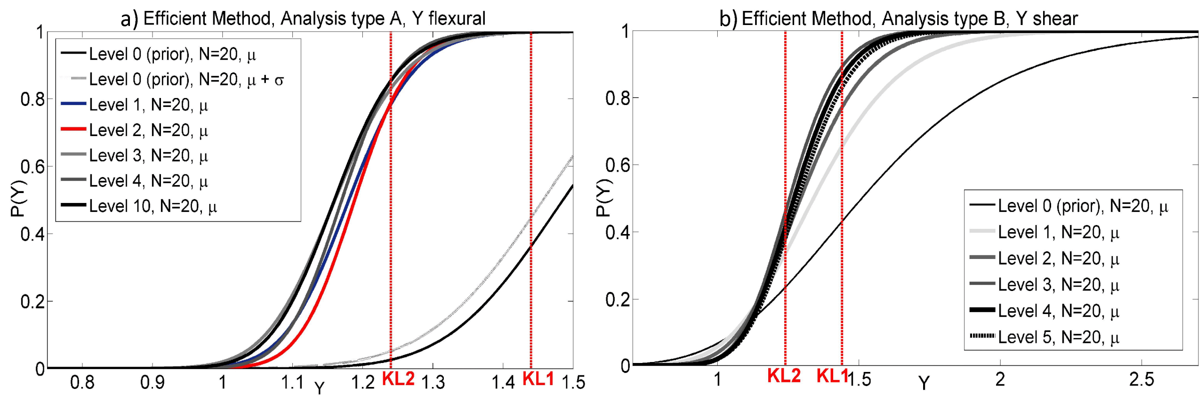

Figure 3.3. Fragility curves for analysis type A (a) B (b)

Considering the fragility curves coming from type B analysis, reported in figure Fig.3.3.b. in terms of Y_{shear}, it can be observed that the influence of inspections on reinforcement details, in terms of the median and the standard deviation of the fragility curves, is bigger than those corresponding to material strengths (even if the number of inspections performed for construction details is less than the number of tests performed for material properties). Moreover, focusing on levels 3 and higher, in which updating procedure on stirrup diameter stops, the fragility curves for levels from 4 to 5, where only updating of stirrup spacing is performed, tend to remain the same as of level 3 or even translate to worst values of Y. This can be explained considering that, unlike stirrup diameter, where a larger diameter is detected (positive effect), the inspections on stirrup spacing tend to verify larger values and tend to affect in a negative way the shear performance of the structure. This aspect is also remarked in Tab. 3.2., which shows the results for the performed analysis in terms of median η and standard deviation σ of the mean fragility curve and of the abovementioned index $Y^{0.95}$.

Table 3.2. Results of the performed analysis

Level	Type A, $Y_{flexural}$			Type B, Y_{shear}			Type C, $Y_{critical}$		
	η	σ	$Y^{0.95}$	η	σ	$Y^{0.95}$	η	σ	$Y^{0.95}$
0 (prior)	1.49	0.087	1.73	1.51	0.270	2.38	1.57	0.170	2.09
1	1.18	0.064	1.31	1.34	0.179	1.82	1.38	0.138	1.75
2	1.19	0.063	1.30	1.30	0.144	1.65	1.33	0.124	1.65
3	1.16	0.068	1.30	1.25	0.112	1.51	1.42	0.175	1.92
4	1.17	0.054	1.29	1.27	0.113	1.54	1.39	0.152	1.80
5	1.19	0.058	1.32	1.28	0.118	1.57	1.37	0.154	1.78
6	1.18	0.060	1.32	-	-	-	1.38	0.141	1.75
7	1.17	0.051	1.29	-	-	-	1.38	0.114	1.67
8	1.18	0.060	1.32	-	-	-	1.34	0.101	1.60
9	1.17	0.062	1.30	-	-	-	1.40	0.156	1.83
10	1.16	0.063	1.29	-	-	-	1.35	0.172	1.81

The vertical lines in previous Fig. 3.3.a. and Fig. 3.3.b. represent the performance parameter corresponding to the code-based confidence factor approach for knowledge levels I and II. The number of available tests seems to reach quantities required for a knowledge level II. From Fig.3.3.a it can be seen that the value of Y obtained for the second knowledge level corresponds to a probability that ranges between 0.77, at simulation level 2, and 0.85 at simulation level 10. As it can be seen from Fig.3.3.b, on the other hand, these probabilities become even smaller. It can be argued that code prescriptions are able to take into account for the uncertainties in material properties, rather than for the ones in structural reinforcement details and their influence on the shear performance of the structure.

4. CONCLUSIONS

The objective of this work is to discuss a framework to account for the influence of the outcome and the number of tests and inspections on the global seismic performance of existing reinforced concrete structures, expressed in terms of the structural reliability. The Bayesian probabilistic framework is used in order to incorporate the effect of various tests and inspections on the seismic performance of the structure. This framework has proven to be particularly suitable to this end since it is able to take into account all of the information available (or the lack of it) for the existing building. In order to make probability-based performance assessments for the existing structure, an Efficient simulation-based method is employed, which is able to estimate the probability of failure based on a relatively small number of structural analyses. As case-study, an existing building is used for which both test and inspections results are available. The tests results are divided into destructive drilled-core tests and non-destructive ultrasonic tests. It is shown how the error in the measurement associated with the non-destructive test results can be calibrated with respect to the drilled core test results (at the locations for which both drilled core and ultrasonic tests were available) using the Bayesian inference. Moreover, it is demonstrated how the error in the measurement can be incorporated inside the Bayesian inference procedure for updating the probability distribution for material property in question. As far as it regards the inspections, both pacometer and georadar tests are employed which lead to information about the diameter and the spacing of shear stirrups, respectively. It is demonstrated how such information can be incorporated inside the Bayesian framework for updating the probability distributions for the construction details. In order to choose a structural performance parameter, particular attention is focused on shear behavior and the results are expressed in terms of critical structural performance index. Structural reliability is expressed in terms of the fragility curves obtained by (a) updating only material properties, (b) updating only shear-related construction details and (c) updating both material properties

and construction details. In order to study the effect of the number and the outcome of tests and inspections on the global performance of the structure, three statistical parameters are used, namely, the median and the coefficient of variation for the fragility curves and the structural performance parameter with the 95% probability of not being exceeded.
It is observed that, for the case-study structure, shear almost always governs the structural performance as expected for substandard structures. Hence, the division of results into flexural and shear performance parameters is useful for studying the effect of various test and inspection results. Given the significant error of measurement associated with ultrasonic results, the effect of number and outcome of ultrasonic tests on the structural performance seems less pronounced with respect to the drilled core tests that are more accurate (in this work, no measurement error is associated with them). Regarding the construction details, stirrup diameter seems to have a larger impact on global performance than stirrup spacing. Overall, given that the shear behavior is predominant, the influence of the number and the outcome of inspections related to shear reinforcement is more significant with respect to that of the (destructive and non-destructive) tests performed on material properties. This aspect appears remarkable since actual code prescriptions seem to emphasize knowledge about material strength rather than inspections on reinforcement details. The framework proposed in this work is suitable for establishing the number of tests and inspections, in a case-specific manner, guaranteeing a prescribed margin of safety for the global performance of the structure.

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