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A Reliability Assessment Methodology for Existing Masonry Structures

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ABSTRACT

Despite the high level of vulnerability of unreinforced masonry structures under applied loads and the importance of their reliability evaluation, there is no formal methodology to assess the reliability of historic masonry structures. Therefore, a step-by-step methodology for assessing the reliability level of historic masonry structures is being developed. To develop an appropriate determinate methodology, estimations of probabilistic models of structural resistance and load effects are required to formulate a limit state function. The stochastic characteristics of construction materials play key roles in the determination of probabilistic models of structural resistance. Codes of practice recommend values as well as the best fit distributions for different material characteristics. As codes are necessarily conservative and are also generally aimed at design or assessment with modern masonry materials, the use of code values for historical structures may lead to inaccurate reliability assessment. Destructive testing of a historic masonry structure or its components in order to get more realistic information regarding the material properties is not recommended as such tests may lead to irreparable damage to these valuable structures. Thus methodologies for estimating the statistical characteristics of historic masonry materials through non-destructive tests as well as suitable probabilistic models are described. Best fit probabilistic models for different load effects are also presented. Finally, target reliability index and failure probability values and different approaches for calculating suitable targets for historic structures are described. Evaluation of the reliability level of historical structures through the recommended procedure would lead to more realistic and accurate levels of reliability estimation without requiring degradation of the historic structure.

KEYWORDS: *masonry, reliability analysis, target reliability, limit state function, probabilistic model*

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INTRODUCTION

Every scientific field involves uncertainty in some way or other. This fact highlights the need for the probabilistic treatment of problems in different fields of study. In structural engineering, deterministic approaches are commonly used in order to avoid complex solutions and calculations. However, more accurate and efficient solutions can be achieved through fully-probabilistic approaches which guarantee comprehensive reliability assessment of structures.

Historic structures are of great importance as they reflect the history and culture of a people and their society. The conservation and maintenance of heritage structures requires considerable care. As these heritage structures were constructed before modern codes and standards were written, current codes will not necessarily provide satisfactorily accurate estimates of loads or resistance when it comes to the evaluation of the historical materials or the structure. This inaccuracy may lead to inefficient and even destructive strengthening modifications. Probabilistic approaches should provide more accurate evaluations and consequently more efficient modifications.

Historical structures are among the most complicated structures in terms of reliability assessment. Different material properties with various stochastic characteristics, complicated configurations, the shortage of available construction documents and limitations in the application of destructive testing techniques are the main concerns and difficulties in reliability assessment of historic structures. To date, considerable research has been directed to structural steel, concrete and timber but only a few studies have focussed on the reliability assessment of masonry structures. Moreover, there are difficulties in applying the methodologies presented by the National Building Code of Canada (NBCC) [18] and Canadian Standards Association (CSA) [6] to the structural evaluation of existing structures in Canada. These difficulties include [1]:

1) In the case of historical structures, there are structural systems, components and materials that are not addressed by the NBCC [18] or CSA S304 [6].
2) Although historical structures were not designed to modern code requirements, they have performed satisfactorily over the years. In addition, a number of structural characteristics such as dead load, strength, deterioration, fatigue and creep cannot be measured directly. While satisfactory performance to date does not mean that a structure will survive the next earthquake, the factors mentioned are considered neither by the NBCC [18] nor CSA S304 [6].
3) Most code clauses are focused on specifying the required and economical percentage of strengthening materials and their related arrangement during the construction process. However, it is often impractical and uneconomical to apply these modifications to an existing structure.

Therefore, alternative requirements need to be defined. As an initial step towards meeting this need for appropriate alternative requirements, the main focus of the proposed research is to develop a step-by-step methodology for assessing the reliability level of historical masonry structures under applied loads without requiring degradation of the historic structure. Evaluation of the reliability level of a historical structure through the recommended procedure should lead to more realistic and accurate levels of reliability estimation.

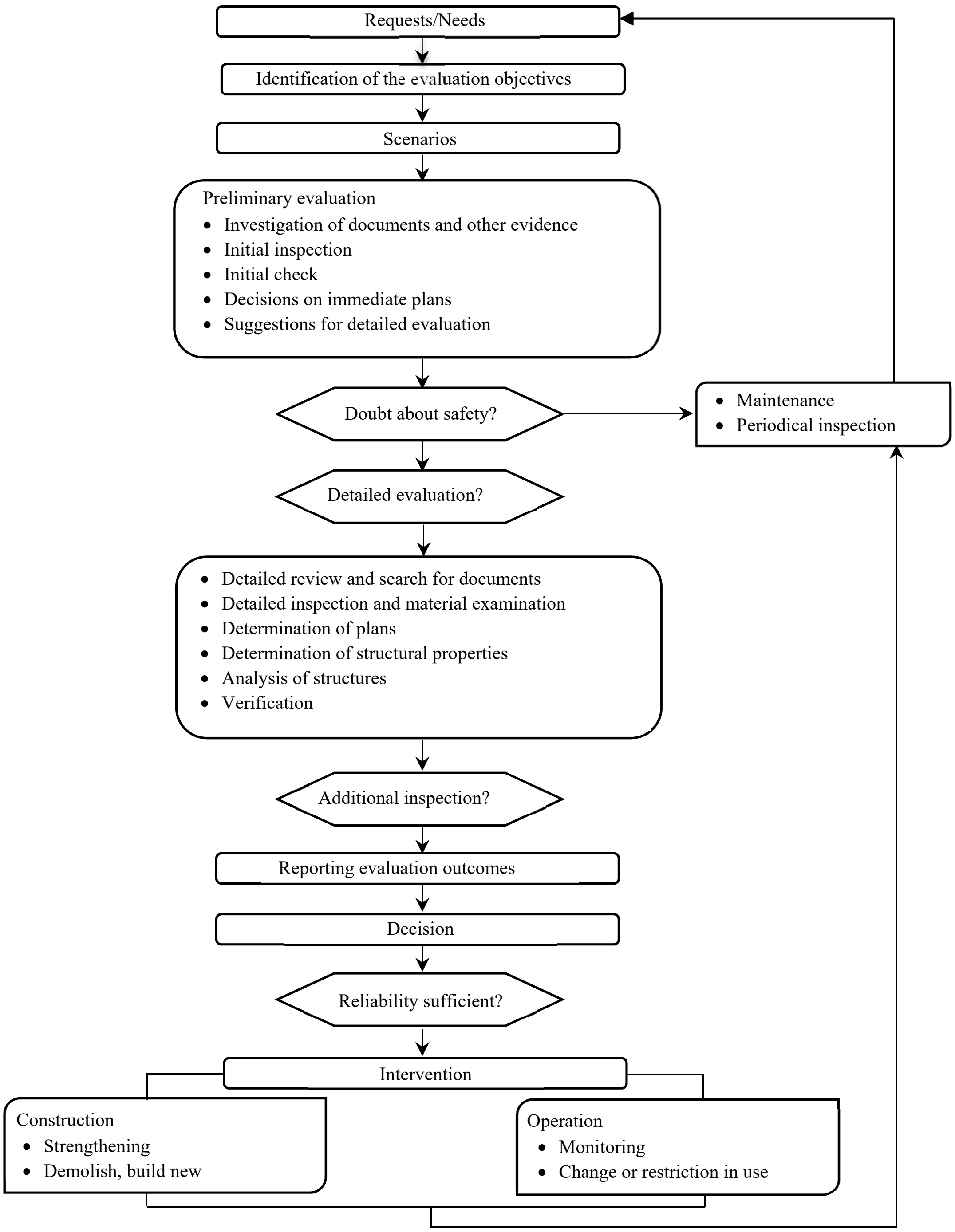

Figure 1: Flowchart for general reliability assessment procedure of existing structure.

STRUCTURAL RELIABILITY ANALYSIS

Reliability assessment is a procedure for evaluating the reliability of an existing building during a performance period, based on information gathered about the building, from material properties to structural layout. In other words, reliability assessment includes estimation of the level of

reliability of a structure considering certain limit states during its service life. Figure 1 shows the flowchart for a general procedure for assessing the reliability of existing structures [15].
Reliability analysis of a structure involves the calculation of its probability of failure under the applied loads. That probability can be determined by following the procedure below:

I) The random variables (X) which influence the structural behaviour including structural resistance (R), load effect (L), model uncertainty (θ) have to be identified. Then, the probabilistic characteristics (mean, Coefficient of Variation (CoV), distribution type) of these random variables have to be determined.

II) A limit state function involving the variables from stage I has to be determined in order to calculate the probability of failure. In structural engineering, the formulation of ultimate limit states is commonly in the basic form:

$$g(R, L) = R - L \tag{1}$$

where R is the resistance and L is a load effect. The safe and failure performances are conventionally defined as:

- $g(R, L) < 0 \rightarrow Failure$ (2)
- $g(R, L) = 0 \rightarrow Critical\ Situation\ (Considerd\ Safe)$ (3)
- $g(R, L) > 0 \rightarrow Safe$ (4)

III) The probability of failure (P_f) or reliability index (β) are calculated. In general, R and L are functions of time and as a result, P_f is also a function of time. However, due to the complexity of calculating P_f, a pre-set reference period (t_l) (design service life) is considered for reliability analysis. This leads to transformation of the probability density functions into time-invariant probability density functions. Thus the probability of failure becomes

$$P_f = P(g(R, L) < 0) = P(R < L) = \iint_{r<l} f_{R,L}(r, l)drdl, \text{ for } t_l \tag{5}$$

where f denotes the probability density function, see Figure 2. Consequently, the probability of survival (R_f) is

$$R_f = 1 - P_f \tag{6}$$

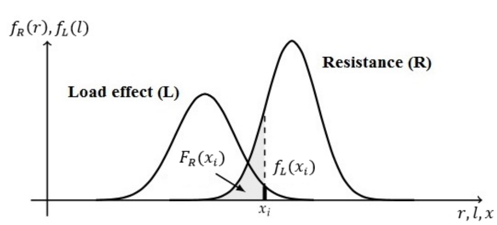

Figure 2: Load effect and resistance probability density functions.

IV) A target failure probability (p_{fT}) or target reliability index (β_T) are determined. The safety assessment of structures necessitates the definition of target safety levels as decision criteria. Codes of practice present different values as target probabilities of failure, e.g. $P_{fT} = 5.10^{-4}$ according to Eurocode [10]. Regarding historical structures, the necessity of meeting the target probability of failure values denoted in codes of practice is still a controversial issue. This is due to that fact that these values are required for new structures, not historical ones with specific criteria and requirements. Several studies have advocated widening of the discussion and the need to develop a more accurate target probability of failure for historic structures. For example, Schueremans [20] considered other influential parameters e.g. cost factor and reassigning the social factor to propose a comprehensive formula to calculate the nominal target failure probabilities for a historical structure

$$P_{fT} = \frac{10^{-4} S_c t_L A_c C_f}{n_p W} \tag{7}$$

where (S_c) is social criterion factor, (t_L) is residual service life/years, (A_c) is an activity factor, (C_f) denotes the economic factor, (n_p) denotes the number of endangered lives and (W) is a warning factor. Moreover, GruSiBau [13] categorizes failure consequences based on defined consequence classes. Reliability indices were determined for each class and consequence for a 50-year observation period, see Table1. Diamantidis [7] also presented final tentative target reliability values as listed in Table 2. JCSS [16] developed a different approach to account for the risk to both human life and investment. Through this approach, the target reliability links to the relative cost of enhancing the reliability of the structure, as shown in Table 3.

Table 1: Target reliability and consequences class [13].

Consequence Class	Ultimate Limit State (ULS)	Serviceability Limit State (SLS)
1	Safe for human life Small economic influence	Small economic impact Small interference with use
	$\beta = 4.2$	$\beta = 2.5$
2	Dangerous for human life Significant economic influence	Significant economic influence Considerable interference with use
	$\beta = 4.7$	$\beta = 3.0$
3	Very dangerous for human life Large economic influence	Large economic influence Large interference with use
	$\beta = 5.2$	$\beta = 3.5$

Table 2: Tentative target reliability values β_T (P_{fT}) [7].

Costs of Safety	SLS (Permanent)	ULS - Failure Consequences		
		Low	Moderate	Significant
High	1.0 (0.2)	2.8 (3.10^{-3})	3.3 (5.10^{-4})	3.8 (7.10^{-5})
Moderate	1.5 (7.10^{-2})	3.3 (5.10^{-4})	3.8 (7.10^{-5})	4.3 (8.10^{-6})
Low	2.0 (2.10^{-2})	3.8 (7.10^{-5})	4.3 (8.10^{-6})	4.8 (8.10^{-7})

Table 3: Target reliability for a 50-year observation period [16].

Relative Cost of Enhancing Reliability	Failure Consequences		
	Minor	Average	Major
	e.g. agricultural building	e.g. residential building	e.g. high-rise building
Large	$\beta = 1.7$	$\beta = 2.0$	$\beta = 2.6$
	$P_f \approx 5.10^{-2}$	$P_f \approx 3.10^{-2}$	$P_f \approx 5.10^{-3}$
Medium	$\beta = 2.6$	$\beta = 3.2$	$\beta = 3.5$
	$P_f \approx 5.10^{-3}$	$P_f \approx 7.10^{-4}$	$P_f \approx 3.10^{-4}$
Small	$\beta = 3.2$	$\beta = 3.5$	$\beta = 3.8$
	$P_f \approx 7.10^{-4}$	$P_f \approx 3.10^{-4}$	$P_f \approx 10^{-5}$

STOCHASTIC MODELING OF APPLIED LOAD

To be able to calculate the probability of failure and the reliability index accurately, stochastic modeling of the applied loads needs to be as realistic as stochastic modeling of the resistance. Therefore, stochastic models of the most common applied loads on historical structures are presented. Generally, loads vary over time and space and, therefore, can be expressed as random variables. It is complex and inefficient to assess the load stochastically for every design case. Therefore, general stochastic models for different loads have been derived to be used in the estimation of the probability of failure and the reliability index.

Dead Loads

Dead loads are assumed to be constant over the service life. However, this assumption is uncertain. Dimensional tolerances and the uncertainty of the unit weight of materials are the main cause of dead load uncertainty. Moreover, the process of converting the dead load into load effects leads to uncertainty. A summary of the statistical parameters for dead loads is presented in Table 4.

Table 4: Summary of statistical parameters for dead load.

Dead Load	Bias	COV	Distribution Type
Ellingwood et al. [8]	1.05	0.1	-
Brehm [3]	1.0	0.06	Normal/Log-normal
Bartlett et al. [2]	1.0	0.10	Normal

Live Load

Total live load is the sum of contributions categorized as permanent live load and transient live load [11]. Permanent live load is that component of live load which remains constant for a period of time but is still removable (e. g. furnishings). Transient live load is associated with a short or instantaneous duration (e. g. a group of people). The recommendations of different authors regarding the stochastic characteristics of live load are summarized in Table 5. The statistical distribution of total live load has been mostly reported to be a Gumbel distribution, whereas the Weibull distribution has proven to be the best fit distribution for point-in-time live load.

Table 5: Summary of recommended stochastic parameters of live load.

Reference	Classification	Mean	CoV	Bias
Permanent Live Load				
CIB [5]	Office	2.64	0.19	-
	Residential	1.73	0.20	-
	Classroom	1.63	0.12	-
Rackwitz [19]	Office	1.81	0.20	-
	Residential	1.52	0.29	-
	Classroom	2.65	0.36	-
Bartlett et al. [2]	50-year max live load	-	0.17	0.9
	Transformation to load effect	-	0.206	1.0
Glowienka [12]	Office	2.51	0.37	-
	Residential	1.81	0.28	-
	Classroom	3.61	0.22	-
Brehm [3]	Residential	-	0.2	1.1
Point-in-time Live Load				
Bartlett et al. [2]		-	0.674	0.273

Wind Load

Wind loads are one of the main horizontal loads on structures. In structural analysis, wind loads refer to the stresses or forces on members due to applied wind. Various studies have examined the uncertain nature of wind load and estimated its related stochastic parameters, but as wind load parameters are strongly dependent on the geographical region of the structures, only values reported for three different parts of Canada are summarized here in Table 6 [2]. It should be noted that Gumbel distribution is usually recommended for wind loads. However, this distribution usually misses the upper limit. Kasperski [17] suggested the Weibull distribution instead as a better fit for wind load and to prevent ignorance of the upper limit.

Table 6: Summary of recommended stochastic parameters of wind load.

Load Type		Bias	CoV	Distribution Type
50-year maximum velocity	Regina	1.039	0.081	Gumbel
	Rivière-du-Loup	1.054	0.112	Gumbel
	Halifax	1.049	0.103	Gumbel
Point-in-time velocity	Regina	0.156	0.716	Weibull
	Rivière-du-Loup	0.064	1.149	Weibull
	Halifax	0.084	1.001	Weibull
Transformation to load effect		0.680	0.220	Log-normal

Snow Load

Snow loads are prevalent in mountainous and cold regions all over the word. In order to calculate the actual snow load on a roof accurately, the difference in the quantity of snow or rain being accumulated and that of the snow or rain being revoked by wind, melting or evaporation should be calculated [9]. As snow load and consequently its probabilistic parameters depend on the location of a structure (like wind load), here only the stochastic parameters reported for Canada are presented, see Table 7.

Table 7: Summary of recommended stochastic parameters of snow load.

Load Type	Bias	CoV	Distribution Type
50-year maximum depth	1.100	0.200	Gumbel
Point-in-time depth	0196	0.822	Weibull
Density	1.000	0.170	Normal
Transformation to load effect	0.600	0.420	Log-normal

STOCHASTIC MODELING OF MATERIAL PROPERTIES

The strength of the materials currently forming a structure has a major influence on the structural resistance; therefore, investigation of the materials' properties is a key factor in reliability assessment. In masonry, units and mortars are recognized as the two main categories of materials. However, various materials having different characteristics were used to build historical structures, with varying levels of workmanship, both of which in turn complicate material analysis. Additionally, there are frequently no construction documents (even as-built documents in modern construction often fail to provide enough information). Applying destructive techniques to determine the properties of the construction materials in situ may result in damage to the structure's fabric or even structural instability. Thus, another technique should be identified and used to assess the characteristics of the materials. All of these information shortages and concerns make analysis and understanding of the behaviour of a historical structure more complicated, but such assessment is necessary to avoid an inappropriate intervention (strengthening scheme). Therefore, behavioural analysis of historical structures without applying destructive techniques is of great interest. Thus one objective of the current work was to find a non-destructive procedure to determine the material properties of historic structures including modulus of elasticity, compressive strength, shear strength and tensile strength and their related variability.

Modulus of elasticity

Loading tests or non-destructive dynamic tests can be applied to estimate the modulus of elasticity. For historical structures it is suggested to apply non-destructive tests to prevent potential damage to these structures. Among non-destructive tests, the Ultrasonic Pulse Method (UPM) has been recognized as an efficient technique to estimate the modulus of elasticity [4].

Compressive strength

There are some expressions estimating the compressive strength of masonry based on the compressive strength of its units and mortar [21]. A general form of these expressions is:

$$f_m' = \mathrm{K}.f_b^{\alpha}.f_{mo}^{\beta} + C \tag{8}$$

where f_m', f_b and f_{mo} represent the compressive strength of the masonry, and the mean values of the unit and mortar compressive strengths, respectively. K, α and β are coefficients and C is a constant value. It is worth mentioning that in order to obtain the compressive strength of the units and mortar, construction documents and/or the results of the material testing are required, which may not be readily available and/or possible for historical structures. In addition, since in a single historical structure there may be a variety of materials possessing different properties, it may not be sensible to apply the above expression in such a structure. Estimation of the compressive strength of the masonry using the modulus of elasticity and the following expression has been suggested as an alternative.

$$E = a.f_m^{'} \tag{9}$$

where E is modulus of elasticity, f_m' is compressive strength of masonry and a is a coefficient of different values assigned by different authors or standards and codes [21]. Since this formula is based on modern materials rather than the historical ones, it is predictable that there should be an error in the estimated value of compressive strength. However, using a non-destructive test multiple times and deriving therefrom a mean strength and the associated variability will provide more accurate values of that strength and variability than trying to estimate the means and CoVs of the compressive strengths of the mortars and the units. Thus, the variability of the materials can be measured using a non-destructive UPM without imposing any damage to the structure under consideration. Numerous articles suggest different relations between E and f_m' which only differ only in their coefficients, here referred to as (k) [21]. For a concrete masonry structure, the coefficient of f_m' has been widely reported as 700 to 900. For clay masonry, the range is wider, being from around 600 to 1000.

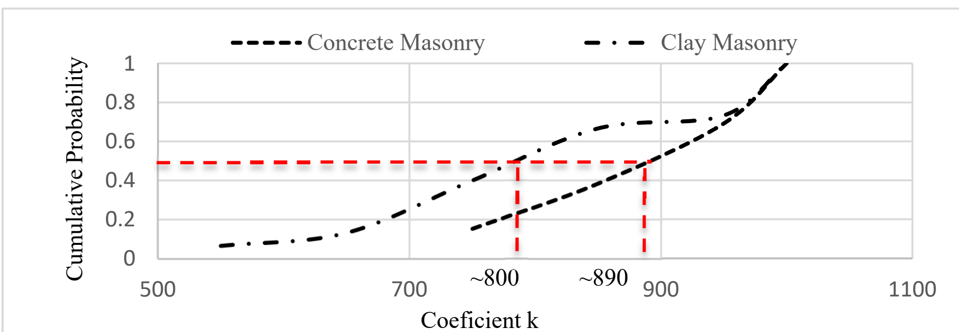

Figure 3: Cumulative probability of coefficient k [21].

As can be inferred from Figure 3, the 50 % probable coefficients are 800 and 890 for clay and concrete masonry, respectively. Values higher than the 50 % probable coefficients (i.e., 800 for clay and 890 for concrete masonry structures) result in more conservative values for compressive strength, while values lower than the 50 % probable coefficients lead to higher values of compressive strength. Thus, one can estimate the compressive strength of a historical masonry structure using non-destructive tests and the relation between E and f_m'. In the case of stone masonry, these coefficients have not been reported so far. Thus there is a need for future research on this matter to aid in the determination of the reliability and safety of historic stone structures. Since stone structures contain units of different size, different source materials and different topologies it is highly likely that more than one coefficient will be required to estimate the strength of these structures based on the measurement of modulus of elasticity.

However, as the accuracy of URM in the estimation of the compressive strength of the historical masonry structures has been described qualitatively in previous studies, it is recommended to evaluate it by several experiments in future to validate the applicability of the proposed procedure.

Shear Strength

In-situ tests can be performed using flatjacks to estimate shear strength. Flatjack tests involve insertion of thin stainless-steel bladders into slots in the masonry. The bladders are inflated to apply force on the masonry. By pressurizing the masonry horizontally between two flatjacks and monitoring the resultant deformation, the shear strength of the masonry can be estimated [14]. Individual flatjack tests have a minor destructive effect on the masonry and therefore the method can be useful for historical structures. However, estimation of strength variability is arguably destructive as that would require making a large number of slots. Moreover, the flatjack method has not shown good accuracy in cases of deep and multi-wythe and multi-layer structures as the bladders cannot penetrate the masonry sufficiently to get the desired information. Therefore, more work also needs to be done in area of shear strength estimation of existing masonry.

Tensile Strength

In order to estimate tensile strength, the bond-wrench test can be used, although it is semi-destructive. In this method, prisms are removed to be tested and then the units replaced in their original location. However, as the tensile strength of masonry has been proved to be negligible, it is simpler to consider the tensile strength as zero in the calculation of masonry resistance.

CONCLUSION

Historic structures are of great importance and should be preserved for future generations. Despite the vulnerability of historic structures to possible applied loads and the need to preserve them, there is still no formal methodology to evaluate the reliability of these valuable structures. For reliability assessment the structural resistance minus the load effect needs to be determined, requiring the estimation of probabilistic models of structural resistance and applied loads. The statistical properties of the materials play key roles in the determination of probabilistic models of structural resistance. Values of target reliability index and failure probability recommended for historic structures and the best fit probabilistic models for different loads have been presented. Possible methods for estimating the stochastic characteristics of historic masonry materials by non-destructive tests are presented. The methodology would provide a ground by which a more accurate and reasonable reliability level would be estimated using less interference imposed to cultural heritage. Consequently, more accurate reliability analysis would lead to determination of more efficient upgrading and strengthening level which would minimize the destruction of the originality of a historical structure in terms of both materials and architecture.

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REFERENCES

[1] Allen, D. E. (1991). "Limit states criteria for structural evaluation of existing buildings." Canadian Journal of Civil Engineering, 18(6), 995–1004. https://doi.org/10.1139/l91-122.

[2] Bartlett, F. M., Hong, H. P., & Zhou, W. (2003). "Load factor calibration for the proposed 2005 edition of the National Building Code of Canada: Companion-action load combinations." Canadian Journal of Civil Engineering, 30(2), 440–448. https://doi.org/10.1139/l02-086.

[3] Brehm, E. (2011). "Reliability of unreinforced masonry bracing walls-probabilistic approach and optimized target values." Technische Universität Darmstadt, Darmstadt.

[4] Brozovsky, J. (2013). "Use of non-destructive ultrasonic pulse testing methods in evaluation of brick parameters." *Proc., the 3rd International Conference on Sustainable Construction Materials & Technologies (SCMT'13)*: 1-10.

[5] CIB W81. (1989). Actions on Structures, Live Load in Buildings, Rpt. No. 116, Rotterdam.

[6] CSA. (2014). Design of Masonry Structures, Standard S304, Canadian Standards Association, Ontario, Canada.

[7] Diamantidis D. (2001). "Assessment of Existing Structures.", Joint Committee of Structural Safety JCSS, Rilem Publications S.A.R.L., The publishing Company of Rilem, 159 pages.

[8] Ellingwood, B.R., Galambos, T.V., MacGregor, J.G., and Cornell, C.A. (1980). "Development of a probability based load criterion for American National Standard A58." NBS Special Publication 577, National Bureau of Standards, U.S. Department of Commerce, Washington, D.C.

[9] Ellingwood, B. and O'Rourke, M. (1985). "Probabilistic models of snow loads on structures." *Structural Safety*, **2**: 291–299.

[10] EN1990. (2002). *Basis for design of structures*, Eurocode, European Standard.

[11] Ferry Borges, J. & Castanheta, M. (1971). "Structural Safety." Laboratorio Nacional de Engenharia Civil, Lisbon.

[12] Glowienka, S. (2007). "Zuverlässigkeit von Mauerwerkswänden aus großformatigen Steinen." doctoral thesis, Institut für Massivbau, T. U. Darmstadt, Darmstadt.

[13] GruSiBau. (1981). "Grundlagen zur Festlegung von Sicherheitsanforderungen für bauliche Anlagen (GruSiBau)." NA Bau, Beuth Verlag, Berlin.

[14] Harvey Jr., Donald. and Schuller, Michael P. (2010). "Nondestructive Evaluation: Structural Performance of Masonry." APT Bulletin,Vol. 41, No. 2/3 (2010), pp. 1-8.

[15] ISO13822. (2003). *Bases for design of structures - Assessment of existing structures*, International Standards Organization.

[16] JCSS. (2001). *Probabilistic model code*, Zurich: Joint Committee on Structural Safety.

[17] Kasperski, M. (2000). "Festlegung und Normung von Entwurfswindlasten." Habilitation, Ruhr-Universität Bochum.

[18] NBCC. (1995). *National Building Code of Canada,* Institute for Research in Construction, National Research Council of Canada", Ottawa, ON, Canada.

[19] Rackwitz, R. (1996). "Einwirkungen auf Bauwerke." in: Der Ingenieurbau: Tragwerkszuverlässigkeit, Einwirkungen. Ernst & Sohn, Berlin.

[20] Schueremans L. (2001). "Probabilistic evaluation of structural unreinforced masonry." Ph. D. Thesis, KULeuven, Belgium.

[21] Seyedain Boroujeni, Setare. and Shrive, Nigel. (2016). "Study of compressive strength of historical masonry structures for reliability assessment." *Proc., the 16th International Brick and Block Masonry Conference,* Padova, Italy.