Probabilistic updating in the reliability assessment of industrial heritage structures

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Assessment of existing structures should be based on the actual as-built conditions concerning geometry, material properties, loading and environmental conditions. A crucial step of the assessment is the updating of prior information by newly obtained measurements which can be conducted by a Bayesian approach. Updating of probabilistic distributions of basic variables or direct updating of failure probability can be used effectively. The application of theoretical principles is illustrated by the case study of a historic factory built in 1900s.

Key words: Bayesian approach, existing structures, industrial heritage, probabilistic updating, reliability assessment

1 Introduction

Numerous factories, warehouses, power plants and other industrial buildings have been registered worldwide as industrial cultural heritage. Such structures are mostly of significant architectural, historic, technological or societal value [TICCIH, 2003]. Protection (including adaptations and re-use) of these structures is an important issue, positively contributing to the sustainable development of urban areas by:

- Preservation of cultural values - the heritage value of the structure commonly originates from its uniqueness, quality of craft execution, relationship with an important event or person, urban context, importance as a landmark etc.;
- Recycling of potential resources and avoiding wasting energy;
- Facilitating the economic regeneration of regions in decline.

However, insufficient attention seems to be paid to systematic recognizing, declaring and protecting the industrial heritage in most countries. This is an alarming situation as the lack of attention and awareness of the industrial structures may gradually lead to their extinction. When out of use the industrial heritage buildings are degrading and often turning into ruins. Re-use and adaptation to hotels, museums, residential parks, commercial centres etc. help protect cities' cultural heritage [Läuferts, Mavunganidze, 2009].

Some of these structures are long-span roof or long-span floor structures, flexible for future use. The protection of the industrial heritage is a multidisciplinary topic including historical, architectural, civil engineering and ecological aspects. In 1978 the International Committee on the Conservation of the Industrial Heritage (TICCIH) was founded to study, protect, conserve and explain remains of industrialisation.

It has been recognised that many heritage structures do not fulfil requirements of present codes of practice. Decisions about adequate construction interventions should be based on the complex assessment of a structure. Minimisation of construction interventions is required in rehabilitation and upgrades, but sufficient reliability should also be guaranteed. Application of simplified procedures used for design of new structures may lead to expensive repairs and losses of the heritage value.

That is why a general probabilistic procedure is thus proposed here to improve the reliability assessment of industrial heritage buildings particularly with respect to:

- Better description of uncertainties related to the assessment and
- Facilitating inclusion of results of inspections and tests and the satisfactory past performance of a structure.

Moreover the outcomes of the probabilistic assessment can be used in a risk-informed decision concerning safety measures as recommended by the Joint Committee on Structural Safety [JCSS, 2001b].

2 General aspects of assessment

Re-use and adaptation of the industrial structures require assessment of structural reliability. In contrast to the design of new structures, the assessment of existing structures often relies on the subjective judgement of the investigating engineer [Caspeele, Taerwe, 2014]. Insufficient attention has been paid by experts to specific issues of the reliability assessment of such structures though the methodology was available already in 1980's [Diamantidis, 1987]. The following differences between the assessment and design of new structures should be carefully considered:

- Societal and cultural aspects - loss of the heritage value,
- Economic aspects - additional costs of measures to enhance reliability of a heritage building in comparison with a new structure (at a design stage cost of such measures is much lower than the cost of strengthening),
- Principles of the sustainable development   waste reduction and recycling of materials (these aspects may be more significant in case of the assessment),
- Lack of information for the assessment – limited number of tests restricted by required costs, even though very important due to variability of mechanical properties and changes that may have occurred during the working life of a structure (including effect of deterioration and damage).

Significant uncertainties related to actual material properties and structural conditions usually need to be considered in the reliability assessment. In design codes a limited number of safety factors is intended to cover all possible design situations. Therefore, verifications based on semi-probabilistic design procedures (partial factor method or load resistance factor design format) may be too conservative. Application of commonly used design procedures may thus lead to expensive repairs and losses of the heritage value. It follows that the use of the semi-probabilistic design procedures may not be an appropriate approach.

It has been recognised that assessment of existing structures is a structure-specific task that is difficult to codify. In accordance with [EN 1990, 2002] and [ISO 13822, 2010] a general probabilistic procedure is thus accepted here to enhance the reliability assessment of the industrial heritage buildings. The procedure facilitates inclusion of results of inspections, testing and consideration of the satisfactory past performance.

3 Principles of probabilistic analysis

Probabilistic methods may be useful for the assessment of existing structures where appropriate data can be obtained [JCSS, 2001b], [Holicky, Sykora, 2012]. Uncertainties that can be greater than in structural design (such as the statistical uncertainty due to a limited amount of test data or uncertainties related to inspection and test procedures, inaccessible members and connections where construction details cannot be inspected and verified) can be adequately described by such methods [Ellingwood, 1996]. On the contrary, some of the uncertainties reflected (often implicitly) in the load and resistance factors (modelling approximations, deviations from specified dimensions and strengths resulting from the differing quality of materials and construction practices) may be less than in new construction, particularly when in-situ measurements are taken. It might be expected that a ‘‘good’’ quality existing structure would have reduced variability when compared to a ‘‘generic’’ structure considering at a design phase [Val, Stewart, 2002].

3.1 ***Specification of models for basic variables***

Models for basic variables should be adjusted to the actual situation and state of a structure and verified by inspection and testing. The following principles should be taken into account:

- Material properties should be considered according to the actual state of a structure verified by destructive or non-destructive testing. It may often be appropriate to combine limited new information with prior information. Bayesian techniques provide a consistent basis for this updating; details are provided e.g. in [ISO 12491, 1997] and in documents of the Joint Committee on Structural Safety JCSS, [JCSS, 2001], [JCSS, 2001b]. Prior information may be found in normative documents (for example in the Czech National Annex to [ISO 13822, 2010] where characteristics of different historic materials are provided), scientific literature, reports of producers etc.
- When significant deterioration is observed, an appropriate deterioration model should be used to predict changes in structural parameters due to foreseen environmental conditions, structural loading, maintenance practices and past exposures, based on theoretical or experimental investigation, inspection and experience.
- Dimensions of structural members should be determined by measurements. When the original design documentation is available and no changes in dimensions exist, nominal dimensions given in the documentation may be used.
- Load characteristics should be introduced considering the values corresponding to the actual situation. For structures with significant permanent actions, the actual geometry should be verified by measurements and weight densities should be obtained from tests.

- Model uncertainties should be considered in the same way as at a design stage unless previous structural behaviour (especially damage) indicates otherwise. In some cases model factors, coefficients and other design assumptions can be established from measurements.

It follows that the reliability verification of a heritage building should be backed up by inspection including compilation of appropriate data. Evaluation of prior information and its updating using newly obtained measurements may be a crucial step of the assessment.

3.2 ***Probabilistic updating***

The failure probability, related to the period from the assessment to the end of a working life t_D, can be obtained from a general probabilistic relationship

$$p_f(t_D) = P\{\min Z[\mathbf{X}(\tau)] < 0 \text{ for } 0 < \tau < t_D\} = P\{F(t_D)\}, \tag{1}$$

in which,

$Z(\cdot)$	is the limit state function;
$\mathbf{X}(\cdot)$	is the vector of basic variables including model uncertainties, resistance, permanent and variable actions;
$F(t_D)$	is the failure in the interval $(0,t_D)$.

When additional new information I related to structural conditions is available, the failure probability may be updated according to [ISO 13822, 2010] as follows

$$p_f''(t_D \,|\, I) = P\{F(t_D) \cap I\} \;/\; P(I). \tag{2}$$

The information should be selected to maximise correlation between the events $\{F\}$ and $\{I\}$. Strong correlation improves the posterior estimate of failure probability whilst weak correlation yields nearly the same estimates as based on Eq. (1) [Ellingwood, 1996]. The new information may be based on:

1. Inspections that can for instance provide data for the updating of a deterioration model,
2. Material tests and in-situ measurements that can be taken to improve models of material or geometry properties,

3. Consideration of the satisfactory past performance,
4. Intensity of proof loading,
5. Static and dynamic response to controlled loading.

In the first two cases the new information is usually applied in the direct updating of prior distributions of relevant basic variables that are commonly based on experience from assessments of similar structures, long-term material production, findings in literature or engineering judgement. The third case may be very important for the industrial heritage buildings as described in detail below. The fourth case is essentially similar to the third one. In the fifth case the known structural response to controlled loading can lead to a reduction of the resistance model uncertainties.

Note that it can be important to consider the satisfactory past performance (the third case) for instance for a structure originally used as a factory that is to be used as a museum or gallery. Such a structure may have resisted to loads much greater than those expected for an intended use.

The satisfactory past performance of a structure during a period t_{SP} (e.g. from completion of the structure till the time of assessment) may be included in the reliability analysis considering the conditional failure probability $p_f''(t_D \mid t_{SP})$ that a structure will fail during a working life t_D given that it has survived the period t_{SP}. This probability may be estimated in several ways. When the load to which the structure has been exposed during the period t_{SP} is known with negligible uncertainties, the resistance or a joint distribution of time-invariant variables may be truncated (a lower bound is set to the value of the load). Using the bounded distribution, the conditional (updated) probability $p_f''(t_D \mid t_{SP})$ can be estimated. This approach, similar to the updating for proof load testing [JCSS, 2001b], is illustrated elsewhere [Diamantidis, Holicky *et al.*, 2012]. More generally, the updated failure probability may be determined using the following relationship

$$p_f''(t_D \mid t_{SP}) = \mathrm{P}\{F(t_D) \cap F'(t_{SP})\} \;/\; \mathrm{P}\{F'(t_{SP})\}, \tag{3}$$

where F' is the complementary event to the failure.

The updated probability can be determined by standard techniques for reliability analysis such as FORM/SORM (First/Second Order Reliability Methods) or importance sampling (see the case study below).

Reliability verification can be based on either of the following (equivalent) relationships

$$p_f''(t_D \mid I) < p_t\,, \quad \beta''(t_D \mid I) = -\Phi^{-1}[p_f''(t_D \mid I)] \geq \beta_t\,, \tag{4}$$

in which

p_t	is the target failure probability;
Φ^{-1}	is the inverse cumulative distribution function of the standardised normal variable;
β_t	is the target reliability index.

3.3 ***Target reliability***

The target reliability level can be taken as the level of reliability implied by acceptance criteria defined in proven and accepted design codes. The target level should be stated together with clearly defined limit state functions and specific models of basic variables. [ISO 2394, 1998] provides examples of the target reliability indices for the anticipated life-time period, related to different relative costs of safety measures and failure consequences, see Table 1.

Table 1: Target reliability index (life-time, examples) in accordance with ISO 2394

Relative costs of	Consequences of failure			
safety measures	Small	Some	Moderate	Great
High	0	1.5	2.3	3.1
Moderate	1.3	2.3	3.1	3.8
Low	2.3	3.1	3.8	4.3

Considering the indicative values given in Table 1, the target reliability index for existing structures usually decreases as it takes relatively more effort to increase the reliability level than at the design stage of a new structure [Vrouwenvelder, 2002]. For instance, in case of an existing structure, one may move from class ''moderate'' to ''high''.

Depending on particular conditions the consequences of structural failure may include [Janssens, O'Dwyer *et al.*, 2012]:

- Cost of repair or replacement,
- Economic losses due to malfunction,
- Societal consequences (costs of injuries and casualties),
- Losses of the cultural and heritage values,

- Unfavourable environmental effects (CO_2 emissions, energy use, release of dangerous substances),
- Psychological effects (loss of reputation).

In common cases an investigated structure or its member is associated with failure consequences given in Table 1 using expert judgement. Some guidance on the classification can be obtained from [EN 1990, 2002] where examples of civil engineering works for three Consequence Classes are provided. However, the inconsistency in classes of failure consequences (four in [ISO 2394, 1998] and three in [EN 1990, 2002]) may somewhat complicate the judgement. Additional information can be obtained from [Vrouwenvelder, 2002], [Diamantidis, Bazzurro, 2007], [Steenbergen, Vrouwenvelder, 2010], [Vrouwenvelder, Scholten, 2010], [JCSS, 2001] and [Sykora, Diamantidis *et al.*, 2014].
Upgrade costs normally consist of:

- Costs related to surveys, design and directly related to structural upgrades;
  and possibly also of:
- Losses of the heritage value,
- Economic losses due to business interruption,
- Replacement of users etc.

A limited guidance on the assessment of relative costs of safety measures is provided in the committee approved draft of the revision of ISO 2394 (not publicly available yet). Therein the relative cost of improving life safety is classified with respect to the ratio between the costs C_1 related to safety measure and the costs C_0 of design and construction costs. The following indicative values may be considered for different relative costs of improving life safety:

- High: $C_1 / C_0 = 0.1$,
- Normal: $C_1 / C_0 = 0.01$,
- Low: $C_1 / C_0 = 0.001$.

For the industrial heritage buildings moderate consequences of failure and moderate costs of safety measures can often be assumed. In this case [ISO 2394, 1998] indicates $\beta_t = 3.1$. It is worth noting that other standards such as [EN 1990, 2002] and [ISO 13822, 2010] provide different target reliability levels, classified only with respect to the failure consequences. However, the costs of safety measures may become an important aspect in case of the industrial heritage structures.

Yet none of aforementioned standards explicitly takes into account the heritage value of a structure. To the best knowledge of the authors, the only model accounting for the heritage value is a simple empirical relationship proposed by [Schueremans, Van Gemert, 2004]:

$$p_t = S_c\, t_D\, A_c\, C_f \;/\; (n_p\, W) \times 10^{-4} \tag{5}$$

in which

S_c is the social criterion factor (recommended value for listed historic buildings 0.05);

A_c is the activity factor (recommended value for buildings 3);

C_f is the economical factor (5 for a moderate consequences, recommended values: 10 for not serious and 1 for serious consequences of failure);

n_p is the number of endangered persons (the most favourable and unfavourable estimates $n_{p,min} = 1$ and $n_{p,max} = 10$, respectively, are considered for significant risk of injury or fatalities - a middle class of consequences [Trbojevic, 2009]);

W is the warning factor (unity for a sudden failure without previous warning).

Considering these indicative data, lower and upper estimates of the target reliability level are obtained from Eq. (5)

$$\begin{aligned} p_{t,max} &= 0.05 \times 50 \times 3 \times 5 \;/\; (1 \times 0.3) \times 10^{-4} \approx 3.8 \times 10^{-3};\ \beta_{t,min} = 2.7, \\ p_{t,min} &= 0.05 \times 50 \times 3 \times 5 \;/\; (10 \times 0.3) \times 10^{-4} \approx 3.8 \times 10^{-4};\ \beta_{t,max} = 3.4. \end{aligned} \tag{6}$$

It appears that the target reliability is within the broad range from 2.7 to 3.4. The value recommended in [ISO 2394, 1998] is approximately in the middle of this range.
It is interesting to indicate the target reliability level for a structure with the same characteristics (regarding the inputs to Eq. (6)), but having no heritage value. For a structure not listed as a historic building, the factor $S_c = 1$ might be assumed. Then, Eq. (5) yields

$$\begin{aligned} p_{t,max} &= 1 \times 50 \times 3 \times 5 \;/\; (1 \times 0.3) \times 10^{-4} \approx 7.6 \times 10^{-2};\ \beta_{t,min} = 1.4, \\ p_{t,min} &= 1 \times 50 \times 3 \times 5 \;/\; (10 \times 0.3) \times 10^{-4} \approx 7.6 \times 10^{-3};\ \beta_{t,max} = 2.4. \end{aligned} \tag{7}$$

It follows from Eqs. (6) and (7) that the target reliability index for the heritage building should be greater (by about one) than that for a similar structure not listed as a historic building. Whether this is an adequate increase of reliability is a complex question that should be addressed in a separate study. In such an investigation it should be taken into account that an increase in the target reliability may potentially result in losses of the heritage values. More detailed information on the procedures for the assessment of the target reliabilities for existing structures is provided by [Diamantidis, Bazzurro, 2007], [Steenbergen, Vrouwenvelder, 2010], [Vrouwenvelder, Scholten, 2010], [Bigaj-van Vliet, Vrouwenvelder, 2013] and [Sykora, Diamantidis *et al.*, 2014].

4 Case study

The proposed procedure is applied in a case study of the reliability assessment of a former factory for boiler production; another example of the probabilistic assessment of a cast-iron heritage structure was provided by [Markova, Holicky *et al.*, 2015]. The factory (Figure 1) was built in 1900s. A reconversion has been conducted to adjust the building for use as headquarters of a publishing house. The anticipated working life is 50 years. Characteristics of the resistance and permanent action are specified considering results of on-site surveys and original design documentation. Effects of degradation are negligible. An assessment by the partial factor method reveals that the critical structural member is a cast-iron truss girder supporting the roof (Figures 1 and 2). The girder is statically determinate. Suction due to wind pressure, causing buckling of a long-span lower chord of the girder, was identified as the most unfavourable load case; axial and shear forces need not to be taken into account. The following analysis is considerably simplified to illustrate key steps of the probabilistic updating rather than to describe case-specific details. The purpose of the case study is two-fold:

1. To show the development of the probabilistic model for cast-iron strength using non-destructive and destructive tests,
2. To illustrate consideration of the satisfactory past performance.

4.1 Updating of the strength of cast iron

Dissimilar to present construction materials, prior information for historic materials (Section 3.1) may not be available. For instance cast-iron strengths vary in a wide range depending on the production process and producers.
That is why models for properties of historic materials need to be solely based on measurements and standard Bayesian updating (combining prior information with test results) [JCSS, 2001b] can hardly be performed. However, the technique of Bayesian

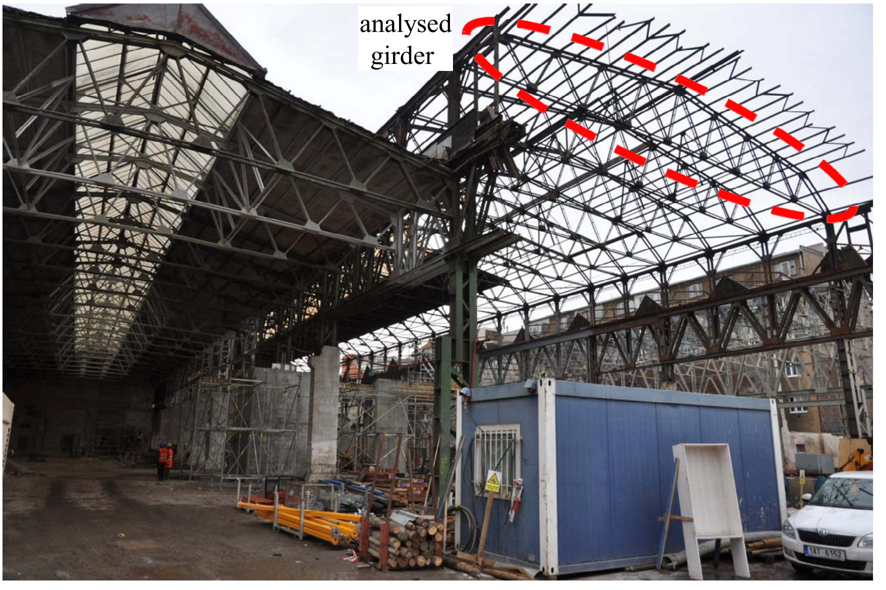

Figure 1: Former factory for boiler production in Prague – Karlin under reconversion

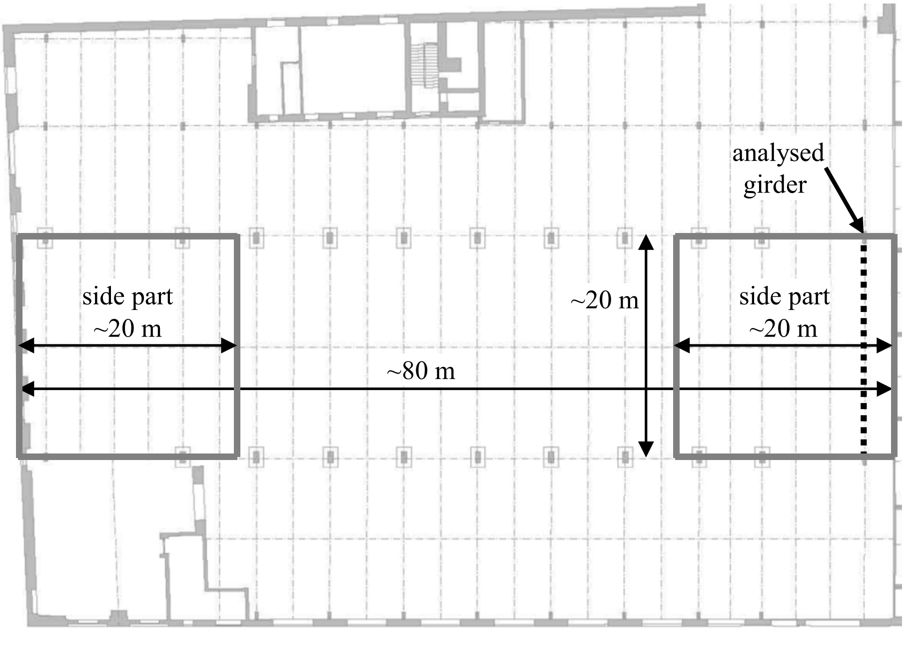

Figure 2: Schematic plan view

updating can be efficiently applied when combining results of non-destructive (affected by a measurement uncertainty) and destructive (deemed to be associated with negligible measurement uncertainty for metallic materials, [Sykora, Holicky, 2013]) testing.

In the beginning of the analysis Brinell hardness tests were performed at ten locations of the structure. Using a relationship based on long-term experience with the test method, results were converted to equivalents of outcomes of tensile tests. Statistical characteristics are obtained by the method of moments

$$m_0' = 385 \text{ MPa},\ s_0' = 20.5 \text{ MPa},\ v_0' = s_0' \,/\, m_0' = 0.053,\ n' = 10,\ \xi' = n - 1, \tag{8}$$

in which

m_0' is the point estimate of the population mean,

s_0' is the point estimate of the standard deviation,

v_0' is the coefficient of variation,

n' is the sample size,

ξ' is the number of degrees of freedom for the standard deviation.
(note that the common symbol ν is not used to avoid confusion with the symbol for a coefficient of variation).

A lognormal distribution with the origin at zero (hereafter simply "lognormal distribution") is considered as an appropriate probabilistic model for the cast-iron strength f. The probability density function based on the point estimates m_0' and s_0' is plotted in Figure 3.

Long-term experience with the Brinell method indicates that a particular measurement f_{0i}' is affected by an unbiased measurement uncertainty ε (mean $\mu_\varepsilon = 1$) with a standard deviation $\sigma_\varepsilon = 0.15$. An actual (true) value of the strength is estimated as the product of a test result and the measurement uncertainty, $f_i' = \varepsilon_i f_{0i}'$. To account for ε the point estimates of the sample characteristics are modified as follows ([Holicky, 2009], [Holicky, 2013])

$$m' \approx \mu_\varepsilon m_0' = 385 \text{ MPa},\ s' \approx m'\sqrt{(V_\varepsilon^2 + v_0'^2 + V_\varepsilon^2 v_0'^2)} = 61.3 \text{ MPa},\ v' = s' \,/\, m' \approx 0.16, \tag{9}$$

in which $V_\varepsilon = \sigma_\varepsilon \,/\, \mu_\varepsilon$ is the coefficient of variation of the measurement uncertainty.

The measurement uncertainty significantly affects estimated variability of f as also demonstrated in Figure 3. The model including ε is now expressing uncertainty of the

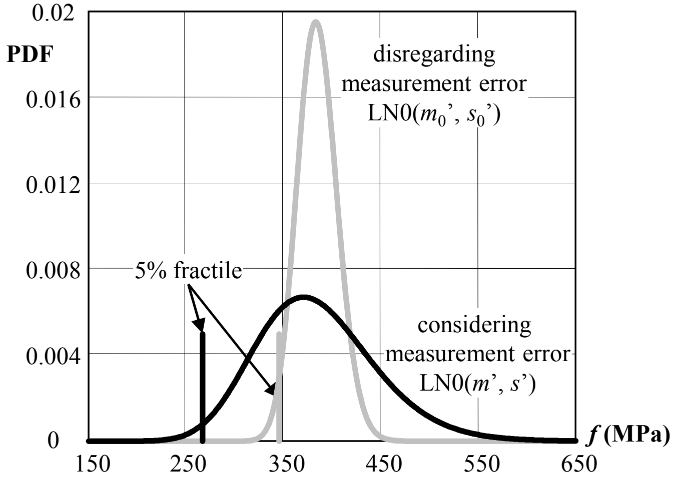

Figure 3: Probability density function of the strength of cast iron based on results of the Brinell tests

analyst in prediction of the cast-iron strength rather than true variability of f that is increased by the variability of ε. The probability density function of the cast-iron strength then corresponds to a greater dispersion and the estimate of a 5% fractile (commonly the characteristic value, [EN 1990, 2002]) considerably decreases. The effect on the design value is even more substantial.

To verify results of the non-destructive testing and improve the material model, three samples were cut from parts of the structure that were replaced due to needs of the intended use. Tensile strengths are as follows: **f** = {371, 351, 418} (in MPa). The following statistical characteristics are obtained by the method of moments

$$m = 380 \text{ MPa}, s = 34.4 \text{ MPa}, n = 3, \xi = 2. \tag{10}$$

According to [ISO 12491, 1997] and [JCSS, 2001b] the updated characteristics (combining prior information   non-destructive measurements here   and results from tensile tests) can be estimated as

$$n'' = n' + n = 13; \xi'' = \xi' + \xi + \delta(n') = 12; m'' = (n'm' + nm) / n'' = 384 \text{ MPa};$$
$$s'' = \sqrt{\{[(\xi'(s')^2 + n'(m')^2 + \xi s^2 + nm^2) - n''(m'')^2] / \xi''\}} = 55.0 \text{ MPa}, \tag{11}$$

where $\delta(n') = 0$ for $n' = 0$ and $\delta(n') = 1$ otherwise.

The updated standard deviation is lower than that based on the Brinell method, however it is still considerably greater than the standard deviation obtained from tensile tests. It could thus be accepted to develop the model of f using the tensile tests only. However, the increase of the standard deviation due to the measurement uncertainty is compensated by a considerable increase of the degrees of freedom ($\xi = 2$ and $\xi'' = 12$) that positively affects the left tail of the distribution. This can be demonstrated for instance by the estimates of the characteristic value f_k in accordance with [EN 1990, 2002]. The greatest estimate is obtained for the updated distribution, $f_k = 282$ MPa; when considering either non-destructive or destructive tests a lower estimate by about 15-20 MPa is obtained. Note that the difference becomes more significant for design values (commonly ~1‰ fractile). Supplementary information on the updating of distributions can be found in [Ang, Tang, 2007, JCSS, 2001, JCSS, 2001b].
It is emphasised that the applied simplified technique is not the only procedure to combine data affected by different uncertainties. A more advanced procedure based on the Bayesian approach was developed by [Sykora, 2014]; an alternative procedure based on the likelihood representation of uncertainties was proposed by [Sankararaman, Mahadevan, 2011].

4.2 Updating of the failure probability of the girder

A verification by the partial factor method reveals that the reliability of the girder is insufficient as the actual resistance is by 10% lower than required by Eurocodes for new structures (considering the updated strength of cast iron). The probabilistic reliability analysis is based on the limit state function Z(·) for the member exposed to buckling

$$Z(\mathbf{X}) = K_R \chi A f - K_E [G + W], \qquad (12)$$

in which

χ is the nominal buckling reduction factor,

A is the deterministic cross-section area.

Notation and probabilistic models of the basic variables X (statistically independent) are given in Table 2, following recommendations of JCSS [Vrouwenvelder, 1997], [JCSS, 2001]. The variability of the buckling reduction factor is covered by the uncertainty in the model resistance; the variability of the cross-section area obtained from in-situ measurements is negligible.

Table 2: Models of basic variables

Variable	Sym.	Distribution	μ_X / x_k	V_X
Cast-iron strength (updated)	f	Lognormal	1.36	0.14
Permanent load effect	G	Normal	1	0.05
Wind pressure (50-y. maxima)	W	Gumbel	0.7	0.35
Effect of the survived load	S	Normal	1.4	0.1
Resistance uncertainty	K_R	Lognormal	1	0.1
Load effect uncertainty	K_E	Lognormal	1	0.1

x_k = characteristic value

Using the FORM method and Eq. (1) the reliability index $\beta \approx 2.8$ is lower than the accepted target reliability level $\beta_t = 3.1$. The reliability is then updated considering the satisfactory past performance to improve this estimate. Available measurements from a neighbouring meteorological station reveal that in 2007 the structure was exposed to an extraordinary wind storm causing a wind pressure S exceeding 1.4-times the characteristic value. Based on an expert judgement uncertainties in the survived load effect are described by a normal distribution with the mean equal to the observed value and coefficient of variation 0.1. Given the survival of the load S, the updated reliability index $\beta''(t_D \mid S) \approx 3.1$ follows from the conditional failure probability based on Eq. (3)

$$p_f''(t_D \mid S) = \mathrm{P}\{[K_R\chi A f - K_E(G+W) < 0] \cap [K_R\chi A f - K_E(G+S) > 0]\} / \mathrm{P}\{K_R\chi A f - K_E(G+S) > 0\}. \quad (13)$$

Note that the present conditions of the girder are assumed to be the same as those at the time of exposure to the load S. It is emphasised that the information on previous loads should be always considered carefully and associated with relevant uncertainty.

4.3 System reliability

The roof is supported by 24 identical truss girders out of which six on each side part are exposed to maximum effects of a wind pressure (Figure 2). Thus reliability of the series system of these girders (N = 12) is analysed. The girders are assumed to have the same reliability, $\beta''(t_D \mid S) \approx 3.1$ and $P_f''(t_D \mid S) \approx 1.1 \times 10^{-3}$. Note that in this section measures concerning reliability of any of the girders (component reliability) are denoted simply β and P_f.

Uni-modal bounds on the failure probability of the system $P_{f,sys}$ (based on failure probabilities of individual girders) are wide

$$P_f = 1.1 \times 10^{-3} \leq P_{f,sys} < N\,P_f = 0.011 \; (2.3 < \beta_{sys} \leq 3.1). \tag{14}$$

To improve these estimates the failure probability of a series system consisting of components with equal reliability indices and equal correlation coefficients is obtained as [Dunnett, Sobel, 1955], [Vrouwenvelder, 2006]

$$P_{f,sys} = 1 - \int [1 - \Phi(-\beta^*)]^N \varphi(u)\mathrm{d}u, \tag{15}$$

in which

Φ is the cumulative distribution function of the standardised normal variable;

β^* $= (\beta + u\sqrt{\rho}) / \sqrt{(1 - \rho)}$ where ρ is the FORM coefficient of correlation between the limit state functions of girders;

φ is the probability density function of the standardised normal variable.

The coefficient ρ is obtained as

$$\rho = \sum_i (a_i^2 \rho_i), \tag{16}$$

in which

a_i is the FORM sensitivity factor of a random variable included in the limit state function (12);

ρ_i is the FORM correlation coefficient describing dependence amongst realisations of a basic variable X_i for different girders.

To assess the system failure probability (15) it is necessary to estimate the correlation coefficients ρ_i:

- The model uncertainties ($a_{KR} = 0.33$, $a_{KE} = -0.33$) are considered to be affected by systematic influences (common to all the girders) rather than by local factors (possibly deviating for individual girders) and full correlation is thus assumed, $\rho_{KR} = \rho_{KE} = 1$.
- The cast-iron strength ($a_f = 0.45$) is an important variable dominantly affected by the measurement uncertainty ε. This is demonstrated by a decrease of V_f from 0.14 to 0.06 when ε is neglected. The contribution of ε to V_f can thus be estimated as $\sqrt{(0.14^2}$ -

0.06^2) = 0.13. The measurement uncertainty is assumed independent amongst the girders and so for the cast-iron strength, $\rho_f = 0$ is conservatively taken into account.

- The permanent action ($a_G = -0.05$) is an insignificant variable and its correlation coefficient affects the system failure probability negligibly; $\rho_G = 0$ is accepted.
- The effect of the wind action ($a_W = -0.76$) dominates reliability of the girders. Considering an extreme storm causing a 50-year maximum of the wind pressure, load effects on six girders within each of the side parts (Figure 2) are likely fully correlated, $\rho_W = 1$. However, small deviations in the maximum load effect might occur between the side parts. Based on the expert judgement this can be expressed as "high or very high degree of dependence" and thus $\rho_W \approx 0.9$. Accepting this value for correlations amongst any of the girders yields conservative results.

Considering the estimates of ρ_i-values (with $\rho_W = 1$), Eq. (16) leads to $\rho = 0.79$ and then the system failure probability is obtained from Eq. (15), $P_{\mathrm{f,sys}} \approx 0.0057$ ($\beta_{\mathrm{sys}} \approx 2.5$). For the alternative value $\rho_W = 0.9$ the reliability index decreases by about 0.05.
The predicted reliability is thus still lower than the target reliability index 3.1. In general five options can now be considered:

1. To improve information on variables significantly affecting structural reliability by further inspections, tests or applying more advanced theoretical models - in this case additional destructive tests of the cast-iron strength could be performed and/or more advanced models for both resistance and wind action effect could be applied.
2. To strengthen the member, for example by reducing the buckling length of the lower chord of the truss girders by appropriate bracing.
3. To propose an adequate limit on the imposed action - irrelevant for the roof girders, however.
4. To accept a shorter remaining working life (such as 15 years [Vrouwenvelder, Scholten, 2010], [Steenbergen, Vrouwenvelder, 2010]) and after that re-assess the girders - using 15-year maxima of the wind pressure the updated reliability index $\beta''(15\text{ y.} \mid S) \approx 3.4$ is obtained from Eq. (13) and β_{sys} increases from 2.5 to 2.9.
5. To derive an optimum target reliability following the principles provided by [ISO 2394, 1998].

Other options may include start monitoring of the structure or critical members, replacement of members with insufficient reliability, decommissioning or dismantlement.

5 Concluding remarks

Reliability verifications of the industrial heritage buildings should be backed up by inspection including collection of appropriate data. Assessments based on simplified conservative procedures used for structural design may lead to expensive repairs and losses of the heritage value. Probabilistic methods can thus better describe uncertainties and take into account results of inspections and tests as well as satisfactory past performance. Numerical example reveals that it may be important to consider measurement uncertainties related to non-destructive techniques. Direct updating of the failure probability can be effectively performed by the FORM/SORM methods and may improve reliability assessment. The reliability appraisal can be performed by implementing appropriate, cost-optimal target levels for existing structures.

Acknowledgements

The paper is a tribute to Ton Vrouwenvelder for his essential contributions to developments of theory and promotion of practical applications of probabilistic assessment of existing structures. The study is an outcome of the research project NAKI DF12P01OVV040 "Assessment of safety and working life of industrial heritage buildings", supported by the Ministry of Culture of the Czech Republic.

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