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ESTIMATING DESIGN RESISTANCE OF WROUGHT BALCONY GIRDERS

Abstract

The contribution is focused on reliability of balcony girders of a Czech national heritage monument. As preliminary reliability assessment suggests insufficient resistance, a series of non-destructive tests supplemented by a single tensile test are performed and evaluated by the statistical methods. Values of material properties, recommended in standards for historic materials, seem to be overly conservative and it is advised to specify properties of historic metallic materials by tests.

Keywords

Bayesian updating, Brinell hardness test, heritage structures, homogeneity, material properties, reliability assessment, statistical approach, wrought steel.

1 INTRODUCTION

Load-bearing structures of numerous heritage buildings are made of historic metallic materials. Particularly in the 19th and early 20th century wrought steel and cast iron became popular construction materials [1]. It has been recognised that such structures often fail to fulfil requirements of present codes of practice [2,3]. Decisions about adequate construction interventions should be based on the complex assessment of a structure considering actual material properties, environmental influences and satisfactory past performance [4]. A key step of this assessment is modelling of resistance of load-bearing members [5].

The submitted contribution is focused on reliability assessment of the balcony girders of the Estates Theatre in Prague under rehabilitation, one of the oldest theatres in Europe, listed as a Czech national heritage monument (Fig. 1). The girders were fabricated in the 19th century; a type of the metallic material is unknown.

Preliminary reliability assessment, based on conservative recommendations of current standards for existing structures, reveals that resistance of the steel girders is insufficient. Reliability analysis of heritage structures has to treat numerous uncertainties related to lack of information about material properties, construction procedures, structural system behaviour etc. Focusing on the first aspect, a number of destructive tests (DT) that are needed to gain credible information on material properties is mostly limited by the requirements on cultural heritage value protection. This is why a very limited number of DTs only is commonly supplemented with a series of non-destructive tests (NDTs).

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Fig. 1: A view of the balcony of the Estates Theatre in Prague

The information given in Annex D of EN 1990 [6] provides a first insight into specification of a minimum number of tests. When coefficient of variation of the material property under consideration, V_X, or its conservative estimate is known, a characteristic value of the material property, X_k, can be assessed from one test result only. In case of unknown V_X, no prior knowledge is available and at least three tests are needed.

For the balconies under study, a cultural heritage protection authority has approved to take only one specimen for destructive testing. The submitted contribution illustrates how a characteristic value and partial factor for material properties can be estimated under such conditions. One destructive tensile test is supplemented by:

- Non-destructive hardness Brinell tests to verify homogeneity of a material across several balcony girders
- Chemical analysis to confirm a type of the material
- Prior information based on previous experience with historical steel materials.

Characteristic value and partial factor are then estimated in accordance with the principles of EN 1990 [6], ISO 13822 [7] and the Czech standard for assessment of existing structures – CSN 73 0038 [8].

Note that the CIB guide [9] for the structural rehabilitation of heritage buildings indicates that a key issue of historic steel structures is corrosion. This has been addressed in the case study as well; however, information on this is beyond the scope of the submitted contribution.

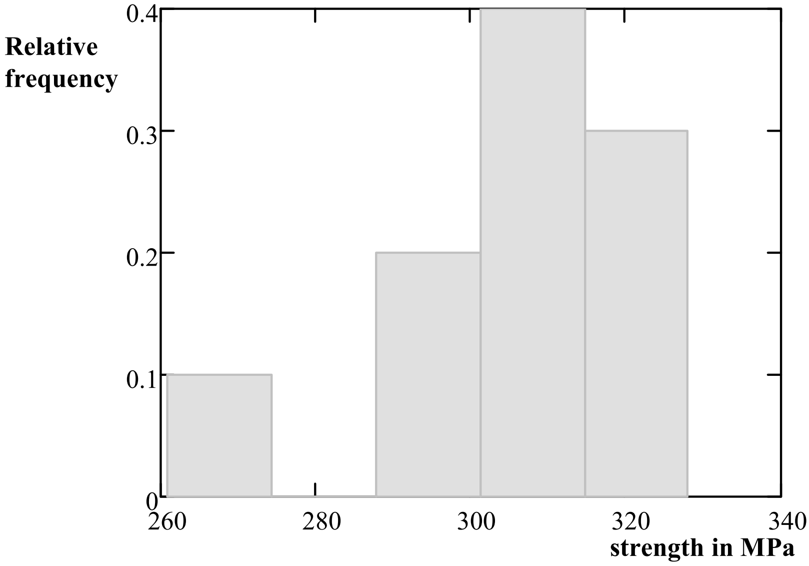

Fig. 2: Histogram of strengths based on NDTs

2 VERIFICATION OF MATERIAL HOMOGENEITY

Results of Brinell hardness tests are taken into account to verify homogeneity of the material. Measurements are taken at ten locations; eight hardness tests are carried out at each of the locations. Fig. 2 displays the histogram of strengths based on NDTs. A conversion factor is applied to make the NDTs estimates consistent with the tensile strength obtained by DT. Grubb's test [10] indicates that the sample likely contains no outlier and extreme observations can result from random variability. This is why the wrought steel is considered as homogenous across all the inspected girders.

3 INPUT DATA AND BASIC ASSUMPTIONS

Tensile test (Fig. 3) leads to the following basic material properties:

- Yield strength: f_y = 275 MPa obtained for strain of 0.2%
- Ultimate strength: f_u = 304 MPa
- Ductility ε_u = 5.1%
- Modulus of elasticity E = 127 GPa.

These values well correspond to the general information provided by the report of the European Joint Research Centre JRC [11] where the following ranges are indicated for wrought steel: $f_y \approx$ 220-310 MPa; $f_u \approx$ 280-400 MPa and $\varepsilon_u \approx$ 5 20%. These observations are also in agreement with an experience gained in the Czech Republic - structures constructed before 1894 were mostly made from wrought steel or cast iron, CSN 73 0038 [8]. Tab. 1 provides an overview of information about properties of historical steels. This evidence thus clearly suggests that the material can be classified as wrought steel. To support this conclusion, metallurgical analysis and chemical composition investigation were carried out and confirmed that the analysed material of the girder is wrought steel [12].

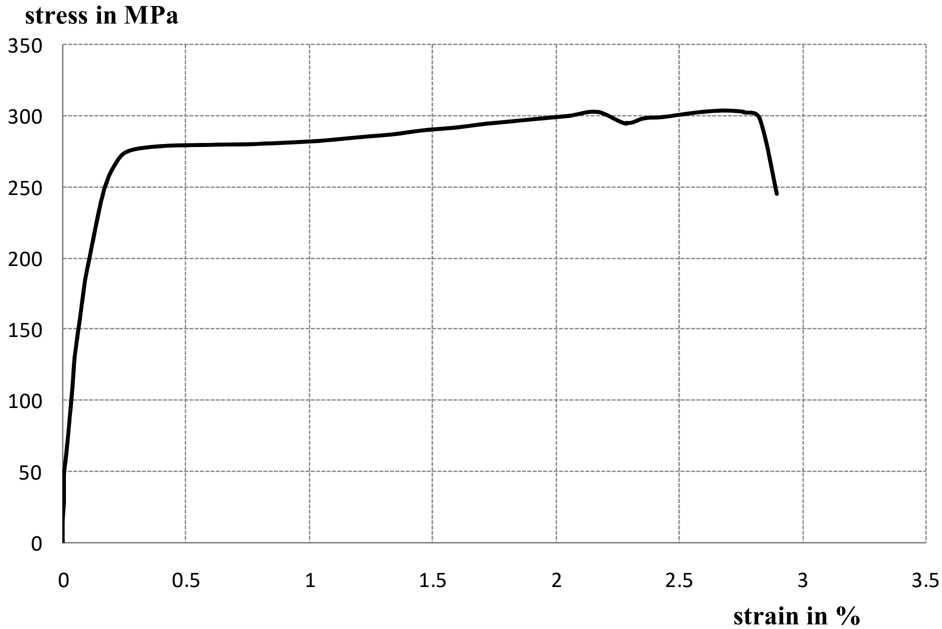

Fig. 3: Stress and strain diagram of wrought steel of the beam

Material strength based on NDTs exceeds the strength based on a tensile test by about 50%, which is common for historic steels [13]. This is why the NDTs estimates are hereafter considered as indicative only and wrought steel properties are assessed on the basis of the tensile test and general experience with historical structures.

3 CHARACTERISTIC VALUE

A two-parameter lognormal distribution [10] provides commonly an appropriate model for strengths of metallic materials including historic steels [3]. A characteristic value is then estimated as follows [6]:

$$X_{\mathrm{k}} = \exp(m_{\ln X} - k_{\mathrm{n}}\, s_{\ln X}) \approx \exp(m_{\ln X} - k_{\mathrm{n}}\, V_X) \quad (1)$$

where $m_{\ln X} = \sum_i \ln(X_i) / n$ (for $i = 1..n$ and number of tests n) and standard deviation $s_{\ln X}$ corresponds approximately to coefficient of variation V_X.

CSN 73 0038 [8] recommends for cast iron strength a coefficient of variation in the range $V_{f\mathrm{u}} \approx 0.1\text{-}0.15$. As variability of wrought steel strength is commonly lower than that of cast iron [3], the middle value of this interval, $V_{f\mathrm{u}} = 0.125$, is deemed to provide a reasonably conservative estimate. The same value is taken into account for yield strength of wrought steel, $V_{f\mathrm{y}} = 0.125$.

Tab. 1: Material properties of historical steels

Material	**Chemical composition**	**Use**	**Material properties (f in MPa, modulus of elasticity E in GPa)**	**Ref.**
Cast iron	C ≈ 2.0-4.0%, Mn ≈ 0.2-1.2%, Si ≈ 0.3-3.0%, S ≤ 1.2%, P ≤ 1.0%		$f_u \approx 90\text{-}135$; $\varepsilon_u \approx 0\%$	[11]
Wrought Steel	C ≤ 0.8%, Mn ≤ 0.4%, S ≤ 0.04%, P ≤ 0.6%	load bearing structures	$f_y \approx 220\text{-}310$; $f_u \approx 280\text{-}400$; $\varepsilon_u \approx 5\text{-}20\%$	
Wrought steel (produced before 1900)	NA		tensile strength $f_{yd} = 180$	
Cast iron	NA	all structural members except for columns	design value of tensile strength 30, design value of compressive strength 65, $E = 100$	[8]
Cast iron	NA	columns	design tensile strength 45, design compressive strength 100, $E = 100$	
White iron of very good quality, completely fibrous	S ≈ 0.25-0.5%, P ≈ 1.5-2%	bridges, truss girders	$f_u \approx 330\text{-}360$; $\varepsilon_u \approx 6\text{-}9\%$	
White iron of ordinary quality, half-granular, half-fibrous	S ≈ 0.25-0.5%, P ≈ 2-2.5%	girders, angle, T-profile	$f_u \approx 250\text{-}320$; $\varepsilon_u \approx 4\text{-}5\%$	[14]
Grey cast iron of high quality	C < 0.3%	bridges, truss roof girders	$f_u \approx 330\text{-}500$; $\varepsilon_u \approx 20\text{-}31\%$	
Grey cast iron (Germany)	NA	columns	$f_u \approx 111\text{-}125$ (448-462 compressive); $E \approx 96\text{-}111$	[15]
Grey cast iron (UK)	NA	buildings	$f_u \approx 75\text{-}160$ with mean 124 (compressive 750); $E \approx 91$	[16,17]
Cast iron (UK)	NA		$f_u \approx 124$ (compressive 590-780); $E \approx 66\text{-}93$	

Following the guidance of Annex D of EN 1990 [6] for "known V_X" – see Equation (1), the characteristic values of wrought steel strengths are estimated on the basis of one tensile test as follows:

$$f_{yk} \approx \exp[\ln(275) - 2.31 \times 0.125] = 206 \text{ MPa}$$
$$f_{uk} \approx \exp[\ln(304) - 2.31 \times 0.125] = 228 \text{ MPa} \tag{2}$$

5 PARTIAL FACTORS AND DESIGN VALUES

Whereas the estimate of a characteristic value may be based on a limited number of tests, the partial factor is commonly based on previous general experience with reliability assessments of steel structures and with uncertainties in modelling, material properties and geometry variables [6]. CSN 73 0038 [8] provides the following relationship:

$$\gamma_M = \exp(-1.645 V_X) / \exp(-\alpha_R \beta V_R) \tag{3}$$

where $\alpha_R = 0.8$ denotes the sensitivity factor for resistance and $\beta = 3.8$ the target reliability index [6,7], and V_R is coefficient of variation of resistance. The target level corresponds to moderate failure consequences, taking into account the effect of cultural heritage protection aspects [3].

It can be considered that resistance of a steel load bearing member R is linearly dependent on its strength X, geometrical properties geo (e.g. sectional areas for failure modes related to compressive or shear forces; in the study under consideration sectional modulus for flexural resistance) and resistance model uncertainty ξ:

$$R = \xi \times geo \times X \quad (4)$$

Coefficient of variation of resistance – see Equation (3) – can be then estimated as follows:

$$V_R \approx \sqrt{(V_X^2 + V_{\mathrm{geo}}^2 + V_\xi^2)} \quad (5)$$

Tab. 2 provides an overview of coefficients of variation for historic metallic materials [8] and justification of the values adopted herein.

Using Equations (3) and (5), partial factors for yield and ultimate strengths become:

$$V_R \approx \sqrt{(0.125^2 + 0.05^2 + 0.05^2)} = 0.144$$
$$\gamma_M \approx \exp(-1.645 \times 0.125) / \exp(-0.8 \times 3.8 \times 0.144) = 1.26 \quad (6)$$

and the design values are obtained from the characteristic values in Equation (2) as follows:

$$f_{\mathrm{yd}} \approx f_{\mathrm{yk}} / \gamma_M = 206 / 1.26 = 163 \text{ MPa}$$
$$f_{\mathrm{ud}} \approx f_{\mathrm{uk}} / \gamma_M = 228 / 1.26 = 181 \text{ MPa} \quad (7)$$

It is interesting to observe that CSN 73 0038 [8] indicates a design value of yield strength of wrought steel for structures constructed before 1900 as $f_{\mathrm{yd}} \approx 180$ MPa. Design values for cast iron are, however, much lower (30-45 MPa).

Tab. 2: Coefficients of variation for historic metallic materials

Symbol	Coefficient of variation according to [8]	Adopted value	Justification
V_X	0.10 – 0.15	**0.125**	The recommended range is deemed to provide conservative estimates for homogenous, high quality wrought steel [3]. In the absence of structure-specific experimental data, a middle value of the interval is taken into account.
V_{geo}	0.05 – 0.10	**0.05**	Dimensions are verified in-situ; the lower bound is thus considered.
V_ξ	0.05 – 0.10	**0.05**	The lower bound applies for flexural and shear resistance of steel girders [18,19]. The adopted model for ξ is deemed to be somewhat conservative as: - Equation (3) is based on the assumption of an unbiased model and yield (not ultimate) strength is to be applied in reliability analysis - Reliability is not affected by the loss of stability.

Annex D of EN 1990 [6] allows estimating a design value directly from one test – such estimates are by about 10% lower than those given in Equation (7) as was shown by the previous study [12]. However, EN 1990 [6] generally recommends estimating a design value on the basis on the ratio of a characteristic value and partial factor and thus the values given in Equation (7) are further compared with the results of a fully probabilistic reliability analysis (Section 6).

6 BAYESIAN UPDATING

When specifying material properties, it is often appropriate to combine limited new information with prior information [2]. Bayesian techniques provide a consistent basis for such updating; details are provided e.g. in ISO 12491 [20] and in the documents of the Joint Committee on Structural Safety JCSS [21,22]. Prior information may be found in normative documents – for example in CSN 73 0038 [8] (the Czech National Annex to ISO 13822 [7]) where characteristics of different historic materials are provided, in scientific literature, reports of producers etc. New information for updating can 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.

To simplify the following text, the updating is focused on yield strengths, f_y, only. The procedure introduced in ISO 12491 [20] and applied in the reliability analysis of an existing steel structure [23] is adopted; for details see the references. Updating, based on the second type of new information, relies on the following assumptions:

- Prior information: assuming a lognormal distribution, the information given in Tab. 1 can be well represented by the probabilistic model with μ_{fy}' = 265 MPa and V_{fy}' = 0.125. The range 220-310 MPa then covers approximately a 75% confidence interval of f_y; the estimate of a design value, f_{yd} = 180 MPa, corresponds to a 1.2‰ fractile of f_y which is commonly used for design resistances when a target reliability index of 3.8 is taken into account.
- According to the JCSS Probabilistic Model Code [21], prior information on modern structural steels may be relatively strong and a hypothetical sample size is $n' \approx 50$. For wrought steel, such information is deemed to be weaker and $n' = 5$ is considered. For concrete compressive strength, ISO 2394 [24] indicates a prior number of degrees of freedom for the prior standard deviation $\nu' = 5$ while the JCSS Probabilistic Model Code [21] suggests $\nu' = 10$. The latter estimate is deemed to be representative for wrought steel.
- New information is conveyed by the tensile test result - f_y = 275 MPa. Test uncertainty is neglected as its coefficient of variation is negligible for practical applications, being less than 1% [25].

The updated statistical characteristics are as follows:

$$\mu_{fy}'' = 267 \text{ MPa}, \; V_{fy}'' = 0.120, \; n'' = 6, \; \nu'' = 11 \tag{8}$$

Fig. 4 depicts the prior and updated distribution functions of yield strength with the updated characteristic value, f_{yk} = 210 MPa, that nearly equals to that given in Equation (2). The results in Equation (8) and Fig. 4 indicate that the effect of updating is negligible and there is a small difference between the prior and updated characteristics.

The strength of updating can be fully appreciated when estimating a design value of flexural resistance. The following probabilistic model is considered:

$$R / W = \xi \, geo \, f_y'' \tag{9}$$

where ξ and geo are lognormal and normal variables, respectively, with unity means and the coefficients of variation given in Tab. 2, W is a section modulus and f_y'' is the updated distribution of yield strength.

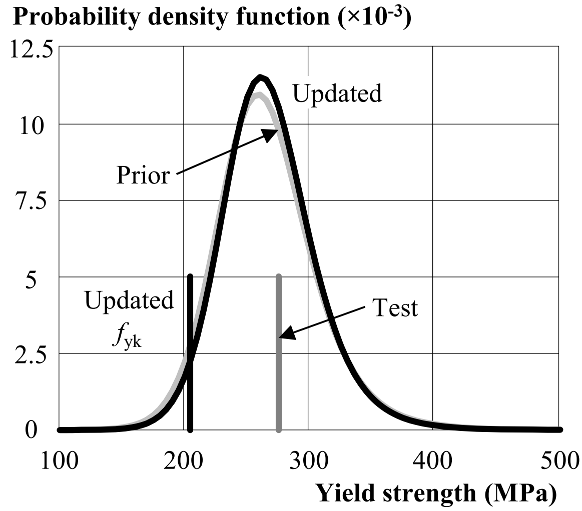

Fig. 4: Prior and updated probability density functions of yield strength

The design value R_d / W = 154 MPa is obtained from Equation (9) as a 1.2‰ fractile of the distribution of R / W. This is slightly lower than the estimate provided in Equation (7) that is based on the statistically very strong assumption of known coefficient of variation.

The partial factor, γ_M = 1.36, exceeds the value of 1.26 given in Equation (6) as statistical uncertainties are fully taken into account in the probabilistic analysis. It is noted that the statistical characteristics of ξ and *geo* based on the recommendations of CSN 73 0038 [8] can be improved considering available experimental data:

- Model uncertainty for yielding flexural resistance of steel beams without the loss of stability is associated with mean value of 1.10 and coefficient of variation of 0.05 [18,19].
- Variability of section moduli of hot-rolled steel beams can be expressed by a normal distribution with unity mean and coefficient of variation of 0.025 [21,26].

The improved probabilistic models leads to an increased design value, R_d / W = 172 MPa, and reduced partial factor, γ_M = 1.22.

7 CONCLUSIONS

The presented study reveals that the reliability assessment of heritage structures is a complex issue. Numerous uncertainties affecting estimated resistance can be treated by statistical approaches and a semi-probabilistic verification method that is suitable for practical applications.

The case study, focused on wrought steel balconies of a heritage building, indicates that:

1. Brinell hardness tests can be used to verify the homogeneity of historic steel materials. However, such tests should always be supplemented by tensile tests to provide credible information on which a material model for reliability verification can be established.
2. Assessment of historic iron or steel structures may be based on a very low number of destructive tests only if:
   - Homogeneity of the material is verified by non-destructive tests and no doubts about non-homogeneity or local damage exist.
   - Metallurgical and chemical composition analyses convincingly indicate the type of a material for which sufficient prior information is available.

   The presented example of having one destructive test only is inevitably associated with large statistical uncertainties and the cooperation with reliability experts is recommended in such cases.

3. Values of material properties, recommended in current standards, seem to be overly conservative and therefore, it is advised to specify properties of historic metallic materials by tests. In the presented case study, the design value of material strength based on measurements exceeds the recommended value given in CSN 73 0038 by about three times.

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