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Orginal Article

Comparison of approaches to reliability verification of existing steel structures

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Abstract: Many existing steel structures are exposed to degradation due to corrosion or fatigue and to increasing loads. Their reliability assessment is then needed. The key question is whether a particular structure can be preserved '*as it is*', or needs to be strengthened, or whether it needs to be replaced. Unnecessary replacements of existing structures may be avoided and the remaining service life of existing steel structures may be authorized by: using advanced reliability verification techniques, optimizing target reliability, and obtaining data for a specific site or structure. In this contribution, the application of advanced reliability approaches is illustrated by the assessment of an existing steel structure. The case study demonstrates that such approaches may significantly improve assessment and allow to increase the load-bearing capacity of the structure (in the case under investigation by 10 to 20%). Improvements in reliability assessment are attributed to the use of an optimal target reliability level, case-specific statistical parameters and probabilistic distributions of the basic variables, and adjusted partial factors.

Keywords: Existing structures, adjusted partial factors, probabilistic approaches, reliability

1. Introduction

Existing structures represent a large volume of structures and they can be exposed to degradation due to corrosion or fatigue or to increasing loads. Their reliability assessment is then needed. The key question is whether a particular structure can be preserved 'as it is', needs to be strengthened, or whether it needs to be replaced. Unnecessary replacements of existing structures may be avoided and the remaining service life of existing steel structures may be authorized by:

- Using advanced reliability verification techniques – the main focus of this study;
- Optimizing target reliability;
- Obtaining data for a specific site or structure [1, 2].

At present, existing structures are mostly verified using simplified procedures based on the partial factor method commonly applied in the design of new structures according to actual codes. Such assessments are often conservative for existing structures and may lead to expensive upgrades [3, 4, 5]. A more realistic verification of the actual performance of existing structures can be achieved by using:

- Adjusted partial factors where the assessment values are obtained as fractiles of updated probabilistic distributions corresponding to probability defined based on sensitivity factor and a selected target reliability level. General guidelines for adjusting partial factors are provided by the basic Eurocode EN 1990 [6] and the international standard ISO 2394 [7].
- Probabilistic methods consider all basic variables describing loads and resistances as random variables using appropriate probabilistic models based on available experimental data. However, their applications require additional calculations and special experience. Further information about probabilistic analysis, reliability management and utilisiation of monitoring can be found in [8 – 11].

The submitted study is aimed at improvements methods of reliability assessment gained by applying advanced procedures in reliability assessment of an existing steel structure; a comparison with results obtained by application of the partial factor method recommended for structural design is provided. The analysis is carried out for the snow load as the leading variable action. Further, optimizing target reliability for existing buildings is briefly discussed briefly.

2. Adjusted partial factors

As the first advanced approach applied in the case study in Section 4, partial factors are adjusted considering structure-specific (information about materials, dimensions, permanent actions, system behaviour etc.) and site-specific (e.g. information about variable loads) conditions [9]. The assessment values are obtained as fractiles corresponding to probability from generalized values of sensitivity factors and a selected target reliability level. Adjusted partial factors are one of the basic approaches to assessment of existing structures introduced in the draft prEN 1990:2022, providing the basis for structural design and assessment of existing structures. This is why it is important to investigate and critically compare this approach with the partial factor method using the values of partial factors recommended for design (hereafter "fixed partial factors").

Assuming a lognormal resistance given as the product of resistance model uncertainty θ_R, geometrical property a, and steel yield strength f_y, the partial factor for resistance R of generic steel members could be obtained as:

$$\gamma_M = R_k / R_d \approx [\exp(-1.645 V_{fy})] / [\mu_{\theta R}\, \mu_a\ \exp(-\alpha_R\, \beta\, \sqrt{(V_{\theta R}^2 + V_a^2 + V_{fy}^2)})] \quad (1)$$

where V is coefficient of variation; $\mu_{\theta R}$ is the mean value of resistance model uncertainty θ_R; μ_a is systematic deviation of random values of the geometric characteristics from its nominal value, expressed as the ratio of mean to nominal value; and α_R is the sensitivity factor of the FORM (First Order Reliability Method) for resistance; and β is the target reliability index. The subscripts "*k*" and "*d*" denote characteristic and design (assessment) values, respectively.

Assuming a normal distribution of the permanent load effect given as the product of load effect model uncertainty θ_E and permanent action g, the partial factor for the permanent load could be calculated as:

$$\gamma_G = G_d / G_k \approx 1 - \alpha_E \beta \sqrt{(V_{\theta E}^2 + V_g^2)} \tag{2}$$

where G_d is the design value of permanent action effect; G_k is the characteristic value of permanent action effect; and α_E is the sensitivity factor of the FORM method for load effects.

Expression (2) assumes that tshe characteristic value of the permanent load effect corresponds to its mean value, the permanent load effect is normally distributed, unity characteristic value is considered for unbiased load effect model uncertainty and that a nominal value of a geometrical property corresponds to its mean.

In general, the partial factor for the variable load could be obtained as [5]:

$$\gamma_Q = Q_d / Q_k = F_{Q,tref}^{-1}[\Phi(-\alpha_E\beta), t_{ref}] / Q_k \tag{3}$$

where Q_d is the design value of variable action effect; Q_k is the characteristic value of variable action effect; and $F_{Q,tref}^{-1}$ is the inverse cumulative distribution function of maxima of the variable load Q_{tref} during a reference period t_{ref}, for which a target reliability index β is specified.

A generic model for variable load effects may be written as follows:

$$Q_{tref} = \theta_E C_0 q_{tref} \tag{4}$$

where θ_E is load effect model uncertainty; C_0 is the time-invariant component of variable load; q_{tref} is the time-variant component of variable load.

When a Gumbel distributed time-variant component is a dominating source of variability (commonly for climatic and imposed loads – see Section 3.2 for further details), the partial factor γ_Q can then be estimated as:

$$\gamma_Q = \mu_{Qtref} \times \{1 - V_{Q,tref}[0.45 + 0.78\ln(-\ln \Phi(-\alpha_E b_t))]\} \tag{5}$$

where $\mu_{Q,tref}$ is mean of maxima of variable load effect (relatively to its characteristic value) and $V_{Q,tref}$ is its coefficient of variation related to t_{ref}:

$$\mu_{Q,tref} \approx \mu_{\theta E} \times \mu_{C0} \times \mu_{q,tref} \tag{6}$$

$$V_{Q,tref} \approx \sqrt{(V_{\theta E}^2 + V_{C0}^2 + V_{q,tref}^2)} \tag{7}$$

3. Probabilistic reliability analysis

The second advanced approach compared in Section 4 with the fixed partial factors is the probabilistic method. In contrast to adjusted partial factors, the probabilistic approach requires no assumptions on sensitivity factor values. In the general case limit state function for steel structural members may be written as follows:

$$g(\boldsymbol{x}) = \theta_R R - \theta_E [G + C_0 \mu_{tref}] \tag{8}$$

where θ_R and θ_E are random variables characterizing the uncertainty in resistance and load effect models respectively, R is a random variable characterizing the resistance of the cross-section or of structural member, G is a random variable characterizing the permanent load, C_0 is the time-invariant component (e.g. shape factor), q_{tref} is the time-variant component of the variable load.

3.1. Resistance

For checks of the ultimate limit states of steel structures, resistance models are often based on yield strength. Reducing the uncertainty in yield strength by additional measurements on the existing structure thus often has a positive effect on the quality of the assessment. During design, the coefficient of variation of the yield strength is in range 5-8% [10], [11]. In the assessment of existing structures, it is possible to measure material and geometrical properties of steel members that may considerably vary for different steel grades, profiles and production processes adopted by various producers. For existing steel structures, the main source of uncertainty is a within-batch (within-rolling) variability. Based on these assumptions, $\mu_{fy} / f_{\text{yk}} = 1.09$ and $V_{\text{fy}} = 5\%$ are adopted in this study as representative values for the assessment. The variability of the geometric characteristics for steel structures is small compared to variability of members from other construction materials, the coefficient of variation is 2-5% [10], [11]. When dimensions are verified in-situ, unbiased values and a lower coefficient of variation can be considered [12], $V_{\text{geo}} = 3\%$ is taken for further analysis. In this study, a resistance model of the cross section under bending (sufficiently braced to restrain instability; thus without buckling effects) is adopted with the following statistical parameters $\mu_{\theta R} = 1.1$ and $V_{\theta\text{R}} = 5\%$ [13]. It is important to note that the resistance model is based on steel yield strength; resistance models based on steel ultimate strength would have different model uncertainty characteristics.

3.2. Action effects

Structures may be exposed to the effects of permanent loads, imposed loads, climatic (snow, wind, etc.) actions, differential settlements, water and earth pressures, earthquakes, accidental actions etc. The following analysis is focused on two key load types for structures – permanent loads and snow loads. Considering the snow load is the only variable action, the fixed and adjusted partial factors are applied according to the load combination rule 6.10(a,b) in EN 1990 [6]. According to this rule, either design value of the snow load effect, $\gamma_Q\, Q_k$, is combined with a reduced design value of permanent action effect, $\xi\, \gamma_G\, G_k$, or the design value $\gamma_G\, G_{\text{k}}$ is combined with a combination value $\psi_0\, \gamma_Q\, Q_k$; the maximum total load effect of the two is then considered. In the probabilistic approach, maxima of the snow load effect related to a reference period adopted for the reliability analysis are considered.

Permanent loads are caused by the self-weight of structural and non-structural members connected to the structure. The permanent loads may be commonly described by the normal distribution with the unbiased mean and coefficient of variation 3-10% [11]. To simplify the following analysis, the permanent load is assumed here to be a single-source, unbiased with respect to a nominal value and with the coefficient of variation of 5% (considering the possibility of measurements during the assessment).

Description of ground snow loads is typically based on sufficiently long records of annual maxima. The results of numerous studies indicate that a Gumbel distribution is often an appropriate model for annual maxima as also recommended in ISO 4355:2013 [14] and

EN 1991-1-3:2003 [15] for snow loads on structures. The background document [16] for EN 1991-1-3 suggested that a Weibull distribution provided the best fit to local measurements; Sadovsky [17] considered a flexible three-parameter GEV distribution for annual maxima of ground snow loads. It is noted that statistical uncertainty may be significant for the three-parameter distribution particularly when records span over short period only or when the records are affected by measurement uncertainty (e.g. for snow depth measurements) [18 – 20]. It may then be preferred to apply the two parameter distributions such as Gumbel, Weibull or lognormal accounting for generally good experience with these in particular climates.

The averaged values of the statistical parameters for describing the distribution of annual maxima of the snow load are adopted. Mean and coefficient of variation for annual maxima are taken equal to 0.4 Q_k and 50%, correspondently. These statistical characteristics can be objectively compared to similar data obtained in countries with a similar climate. The background report for Eurocodes [11] proposes the generalised values for annual maxima that are similar to those adopted here. Statistical parameters of Gumbel distribution for different reference periods (Table 1) are obtained as follows:

$$\mu_{Q,tref} = \mu_{Q,1}\,[1 + 0.78\,\ln(T)\,V_{\mathrm{Q},1}] \tag{9}$$

$$\sigma_{Q,tref} = \sigma_{Q,1} \tag{10}$$

where $\sigma_{Q,tref}$ and $\sigma_{Q,1}$ are standard deviation for snow load related to t_{ref}.

Snow load on the roof is obtained from the ground snow load by using shape, thermal and exposure factors. Uncertainties related to these coefficients are described here by the time-invariant coefficient C_0 according to [11].

In accordance with the generally accepted practice, load effect model uncertainty is described here by a unity mean and coefficient of variation of 7.5% [10].

The probabilistic models considered in the case study are presented in Table 1.

Table 1. Probabilistic models of basic variables considered in the case study

Basic variable	X	Dist.	μ_X / X_k	V_X
Yield strength	f_y	LN	1.09	5%
Geometry	a	N	1.0	3%
Resistance model uncertainty	θ_R	LN	1.15	6%
Permanent load	G	N	1.0	5%
Snow (1-year maxima)	q_1	Gum	0.4	50%
Snow (10-year maxima)	q_{10}	Gum	0.76	26%
Snow load – time-invariant component	C_0	LN	0.8	20%
Load effect model uncertainty	θ_E	LN	1.0	7.5%

μ_X – mean, V_X – coefficient of variation, N – normal distribution, LN – lognormal distribution with the lower bound at the origin, Gum – Gumbel distribution (max. values), X_k – characteristic value of basic variable.

4. Case study of steel beam

In this section, reliability requirements following from the fixed partial factors (*FPF*) provided in EN 1990 (values recommended for design), adjusted partial factors (*APF*)

(Section 2), and probabilistic method (*PM*) (Section 3) are critically compared. Reliability of the existing structure needs to be verified. Structural survey reveals no defects affecting structural reliability at the Ultimate Limit States. A particular focus of the case study is a steel beam – roof girder exposed to the dominant effect of snow load. Previous studies indicated low reliability of such structural members when they are verified considering the probabilistic models adopted in design [21-23]. The steel beam is fully laterally-restrained and stability issues do not affect structural reliability.

A target reliability index is recommended according to EN 1990 [6]: β_t = 3.8 for a reference period of 50 years. However, target reliabilities are intended to be used primarily for the design of members of new structures. In general, lower reliability levels can be accepted for existing structures in comparison to structural design as follows from the general principles of structural reliability provided in ISO 2394:2015 [7] and prEN 1990-2 [24]. The two standards suggest that besides failure consequences, the target levels should also be differentiated with respect to relative cost of safety measures that is often much higher for the existing structure than for a structure being designed. There are a number of studies in the field of optimization reliability levels [25] – [28]. Optimizing target reliability for existing structures by implementing cost optimization procedures and criteria for human safety is presented in *fib* Bulletin 80 [5]. According to [5] two reliability levels are recommended – the minimum level below which the structure is considered unreliable and should be upgraded - reliability index β_0; and the target level indicating an optimum upgrade strategy - β_{up}. The reliability indexes β_0 and β_{up} are presented as a function of the collapsed area due to the failure structural member and a reference period (it is assumed that a reference period is equal to the remaining service life). In considered case the collapsed area is smaller than 100 m^2. Reliability assessment should verify whether or not the structure can remain in service for next 10 years, it means reference period t_{ref} is equal 10 years. For middle Consequence Class (CC2), β_{up} = 3.3 and β_0 = 2.8 are obtained according to [5] for reference period t_{ref} equal 10 years.

EN 1990 [6] is the basic document that suggests the load combinations (such as those in equations (6.10) or (6.10a,b) therein) and relevant partial factors. The following partial factors are recommended for structural design for permanent loads: γ_G = 1.35 and ξ = 0.85 and for variable loads γ_Q = 1.5 and ψ_0 = 0.5 (snow). The load combination rule 6.10(a,b) according to [6].

Using the adjusted partial factors and the probabilistic method (FORM), partial factors are derived to provide for the adopted target reliability index. The values of the partial factors are presented in Table 2.

Table 2. Comparison of partial factors (load ratio χ = 0.8)

	APF ($\beta_{0,10}$ = 2.8)	*APF* ($\beta_{up,10}$ = 3.3)	*APF** ($\beta_{0,10}$ = 2.8)	*APF** ($\beta_{up,10}$ = 3.3)	*PM* ($\beta_{0,10}$ = 2.8)	*PM* ($\beta_{up,10}$ = 3.3)
γ_G	1.07	1.08	1.05	1.06	1.05	1.09
γ_Q	1.10	1.24	1.40	1.64	1.42	1.65
γ_{M0}	0.97	1.00	0.84	0.85	0.85	0.86

* Adjusted partial factors calculated with the actual values (according to Figure 2) of the sensitivity factors (α_R = 0.2, α_G = 0.2, and α_Q = 0.95).

The geometrical characteristic of a cross-section W_i, such as section module, required to satisfy the limit state in accordance with a particular approach to reliability verification (hereinafter referred to as a requirement) is calculated from the limit state function:

$$g(\boldsymbol{x}) = W f_{yk} / \gamma_{M0} - [\gamma_G G_k + \gamma_Q\ C_0\ Q_k] \tag{11}$$

To cover a wide range of load combinations, load ratio χ is introduced. The load ratio χ denotes the ratio of characteristic variable loads to the total characteristic load given as:

$$\chi = Q_k / (G_k + Q_k) \tag{12}$$

In most practical cases for steel beams the load ratio may vary within the interval from 0.3 (for example, a steel beam with a reinforced concrete deck) up to 0.8 (lightweight steel roofing for industrial halls) [29].

Figure 1 displays variation of $w_i = W_i / W_{EN}$ with χ where W_{EN} is the reference value based on the partial factors recommended in Eurocodes for structural design. When $w_i <$ 1, the reliability requirements according to approach "*i*" are lower than those according to Eurocodes for structural design.

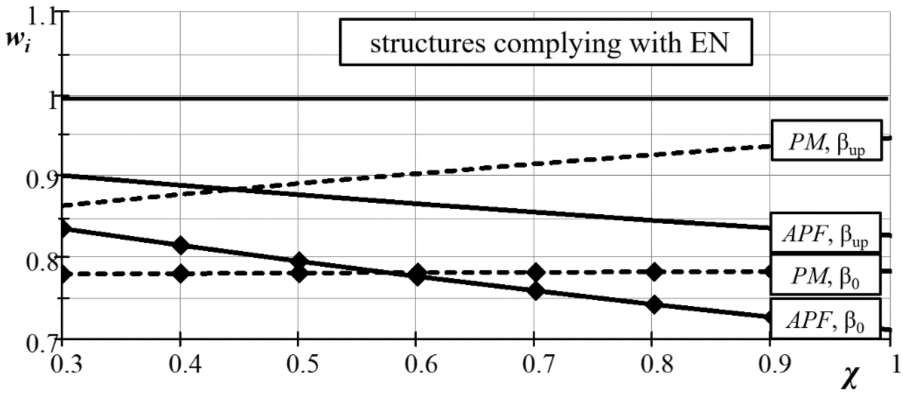

Fig. 1. Variation of w_i with χ (adjusted partial factors based on generalised values of sensitivity factors)

Figure 1 shows that an adopted target reliability level has a significant influence on the reliability requirement. For both adjusted partial factors (*APF*) and probabilistic method (*PM*) the lowest requirements are related to the β_0-level while the requirements based on β_{up} are between the β_0- and Eurocode requirements. Further, the *APF* and *PM* lead to different requirements. The main difference in these two methods is due to the assignment of generalised sensitivity factors α for the *APF*. It follows from Figure 1 that the generalised α-values may lead to unconservative requirements for $\chi > 0.6$ (β_0-requirement) and for $\chi >$ 0.45 (β_{up}-requirement).

It is possible to determine the sensitivity factors using FORM to eliminate this deficiency of the *APF*. Figure 2 displays variation of the FORM sensitivity factors with the ratio χ. It appears that for steel structures, the dominant influence on reliability can be attributed to variability of the load effect.

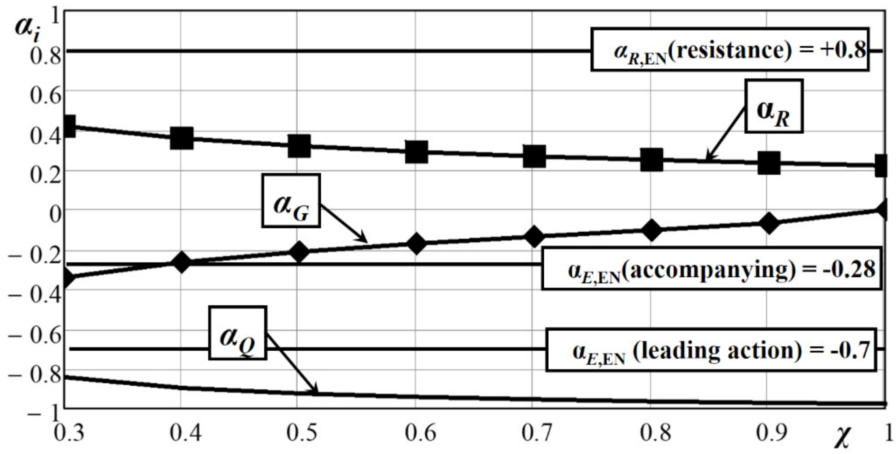

Fig. 2. Variation of sensitivity factors with load ratio χ

Figure 3 displays variation of w_i with χ for adjusted partial factors with the sensitivity factor α_E = -0.95 for the snow load and α_R = -0.2 for resistance. Using these α-factors, partial factor for snow load $\gamma_Q \approx 1.65$ and for resistance $\gamma_M \approx 0.85$ can be considered to comply with the β_0-requirement (Table 2). Similar reliability can be achieved by considering a commonly accepted $\gamma_M = 1.0$ along with reduced $\gamma_Q \approx 1.4$.

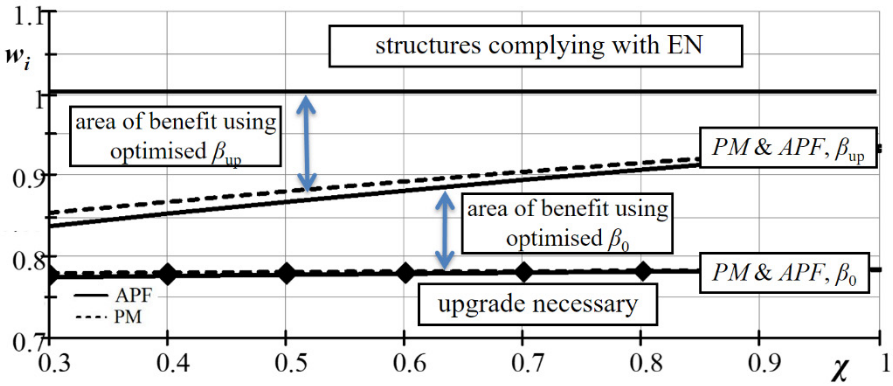

Fig. 3. Variation of w_i with load ratio χ (adjusted partial factors based on updated sensitivity factors)

The results of the *APF* and *PM* become close when using the actual values of the sensitivity factors. Figure 2 shows that the adjusted partial factors method (*APF*) and probabilistic method *PM* lead to the reliability requirements being lower than *EN*. The decrease in requirements is attributed to the use of the lower target reliability level for existing structure β_0 (lower than in *EN*) and case-specific probabilistic distributions for basic variables (measured statistical parameters of this distributions) that reduce the conservativeness of fixed partial factors. In contrast, the requirements for upgrades according to *APF* and *PM* (considering β_{up}) are close to those based on *EN*. The area between the curves for assessment (β_0) and upgrade (β_{up}) in Figure 2 is associated with the situations when the application of the advanced methods is expected to provide assessment benefit. In these situations, *EN* assessment requires an upgrade with economic and environmental impacts while the advanced methods authorise a continued use of the structure '*as it is'*.

For the structures designed according to the old standards valid before Eurocodes has been introduced and for which the snow load is the dominating variable load (e.g. wind load is comparably smaller), ratio w_i is expected to range approximately:

- From 0.8 when the snow load is the leading action in the load combination (χ close to 1.0)
- To 0.9 when the permanent load is the leading action (χ close to 0.3).

Such low w_i - values are attributed to increased design roof snow loads as introduced by Eurocodes. Similar observations were made for structures designed according to past Czech standards [30].

5. Discussion

The presented study provides a first insight into the performance of various approaches to reliability verifications of existing steel structures. Development and wider use of the adjusted partial factors seem to be reasonable considering the balance between demands on the input information, computational complexity, and achieved improvements in reliability assessments. Besides certain limitations of this method, it remains to define the target reliability levels for existing structures.

It seems that the most critical aspects in the application of the adjusted partial factors method is the setting of the sensitivity factor. It is generally accepted that a $\alpha_R = 0.8$ is suitable for the resistance model, and this value is recommended in EN 1990. This value seems reasonable for reinforced concrete, masonry and possibly for timber structures, for which the variability of the basic variables included in the resistance model is relatively large in comparison to uncertainties in the total load effect. For the steel structures, the variability of resistance is small compared to the variability of loads and it seems advisable to revise the recommended values of the sensitivity factors.

The probabilistic approach providing a reference level to simplified approaches requires further investigations as well. In particular, the review of available information shows incomplete empirical evidence to unambiguously justify the statistical parameters of variable load effects.

Modelling of degradation processes due to corrosion and fatigue seems to be another important challenge for further improvement of reliability studies. Special attention should be paid to reliability assessments of existing structures after fire exposure [31].

Cases with a single variable action are considered as a special issue of the reliability theory – combination of several variable actions is beyond the scope of this contribution. Previous studies revealed that the combination factors accepted in Eurocodes are often conservative and lower reliability levels were commonly obtained for the structures exposed to a single variable action compared to structures exposed to the effects of several variable actions [32].

6. Conclusions

Application of advanced probabilistic approaches allows reducing assessment requirements which the structure is considered unreliable and should be upgraded. This is attributed to the use of a particular target reliability level, case-specific statistical parameters and probabilistic distributions of the basic variables, and adjustment of partial factors. The application of fully probabilistic methods requires additional calculations and special experience while the adjusted partial factor method is easier to use. Nevertheless, the main conservatism of the latter method remains in the use of the generalized sensitivity factors.

The presented case study – detailed reliability analysis of an existing steel beam demonstrates that the dominant influence on reliability can be attributed to the variability of the variable load effect. The sensitivity factor α_E exceeding -0.9 for the dominating load and α_R smaller than -0.2 for resistance are obtained. It appears that reliability requirements for the minimum level below which the beam is considered unreliable and should be upgraded can be decreased by about 20%. Requirements for an optimum upgrade strategy of the beam might then be by about 10% in comparison to the design requirements.

For all the methods, the input data include the probabilistic models of basic variables. However, further studies and standardisation of such models is important and demanding task. As uncertainty in the load effects has the largest impact on reliability of steel structures, it is necessary to focus subsequent studies on a description of the models for loads and load effect model uncertainty. Within future research, the target reliability levels for existing structures should be specified.

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