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RELIABILITY APPROACHES AFFECTING SUSTAINABILITY OF EXISTING STEEL STRUCTURES

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Abstract. Steel structures are second most numerous in the stock of existing buildings. In contrast to dominating concrete buildings, they are typically lightweight and are more sensitive to alterations in use or loads. While the sustainability principles require to maintain and keep using these structures, structural assessments often indicate insufficient reliability and need for replacements. The submitted contribution shows that the most important reliability considerations affecting the sustainability of existing steel structures consist of specifying (1) appropriate target reliability level, (2) verification methods, and (3) intervention procedures. The study focuses on the first two aspects. (1) Optimum target reliability can be specified by probabilistic optimisation considering sustainability aspects including structural costs, and expected consequences of replacement and of possible failure. It is shown that lower reliability levels might be considered for the assessment of existing structures than for the design of new structures, with benefits for sustainability in construction.

Regarding (2), the most efficient verification methods are based on advanced probabilistic approaches. It is demonstrated that sustainability may be significantly affected by the selection of assessment methods. Advanced reliability approaches commonly reduce assessment requirements by 10–15 %. Sustainability indicators are mostly related to the key aspects (1) and (2). Using the advanced methods may bring a significantly positive contribution to sustainability, particularly when an upgrade of the existing structure is associated with high economic cost and significant environmental impact.

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

1. Introduction

Steel structures are second most numerous in the stock of existing buildings. In contrast to dominating concrete buildings, steel structures are typically lightweight and are more sensitive to changes in use and adjustment of loads. Some of the existing structures are more than 100 years old and protected for their heritage value. While the sustainability principles require to maintain and keep using these structures, structural assessments often indicate insufficient reliability and need for replacements. This situation may be solved by applying advanced reliability assessment methods that mitigate the conservativeness of simplified methods utilised in engineering practice. In agreement with this, the Global Consensus on Sustainability in the Built Environment [1] requires facilitating and rewarding the use of advanced analyses and methods of structural reliability to achieve sustainability in construction.

Applications of advanced assessments of existing structures may contribute to achieving the Sustainable Development Goals (SDGs). In October 2015 the United Nations adopted Resolution 70/1 *Transforming Our World: the 2030 Agenda for Sustainable Development* to balance the three aspects of sustainable development: economic, social, and environmental. Improved assessments of existing structures can particularly help contribute to reach SDG 12 *Ensure sustainable consumption and production patterns.* Relevant targets presented in the resolution include

(a) achieving the sustainable management and efficient use of natural resources by 2030, and

(b) substantially reducing waste through prevention, reduction, recycling and reuse.

The assessment may positively contribute to sustainability in construction, facilitating to keep existing structures in service. The assessment of existing steel bridges may be improved by specifying

- appropriate target reliability level,
- verification methods, and
- intervention procedures.

This study focuses on the first two aspects: (1.) optimum target reliability can be specified based on probabilistic optimisation considering sustainability aspects including structural costs, expected consequences of replacement and of possible failures. Regarding (2.), the most efficient verification methods are based on advanced probabilistic approaches, considering actual load conditions and properties of the structure and related failure consequences. This study investigates benefits of applying advanced methods in the reliability assessment of an existing steel structure, critically comparing the obtained results with those based on the partial factor method for structural design.

Basic variable	X	Dist.	$\mu X/X_k$	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 %
Ground snow (1-year maxima)	q_1	Gum	0.4	50 %
Snow load – time-invariant component	C_0	LN	0.8	20 %
Load effect model uncertainty	θE	LN	1.0	7.5 %

μX – 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.

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

2. Adjusted partial factors

General guidelines for adjusting and updating partial factors are provided by the basic Eurocode EN 1990 [2]. Partial factors may be adjusted considering structure-specific (information about materials, dimensions, permanent actions, system behaviour etc.) and site-specific conditions (e.g. information about variable loads). The assessment values are obtained as fractiles corresponding to probability from generalized values of sensitivity factors and a selected target reliability level. For more details see [3].

3. Probabilistic reliability analysis

A generic limit state function for members of steel structures may be written as follows:

$$g(x) = \theta_R R - \theta_E[G + C_0 q_{ref}], \quad (1)$$

where the notation of the basic (random) variables is as follows:

θ_R and θ_E uncertainties in resistance and load effect models respectively,

R resistance of the cross-section or of a structural member,

G permanent load,

C_0 time-invariant component (e.g shape factor for the roof snow loads), and

q_{ref} time-variant component of the variable load related to a reference period tref (e.g. maxima of the ground snow loads).

Probabilistic models for basic variables given in Table 1 are selected taking into account in situ measurements and data in JCSS Probabilistic Model Code [4] and previous studies [5]. The results of numerous studies indicate that a Gumbel distribution is often an appropriate model for annual maxima. The background report for Eurocodes [6] proposes the generalised values for annual maxima that are adopted here (Table 1).

The statistical parameters for different reference periods are recalculated using general equations for Gumbel distribution. 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 [6]. The model for load effect uncertainty, θ_E is based on the JCSS Probabilistic Model Code [4].

4. Case study – reliability analysis of roof girder of existing steel building

In this section, reliability requirements following from the fixed partial factors (FPF) provided in EN 1990, adjusted partial factors (APF) (Section 2), and probabilistic method (PM) (Section 3) are critically compared. Reliability assessment is performed considering a 10-year remaining service life (equal to a considered reference period). Target reliability index is recommended according to EN 1990 [2]. However, these recommendations 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]. Optimisation of the target reliability for existing structures by implementing cost optimization procedures and criteria for human safety is presented in [8]. 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}. For middle Consequence Class (CC2) $\beta_{up} = 3.3$ and $\beta_0 = 2.8$ are considered [8].

EN 1990 [2] is the basic document that suggests the load combinations and relevant partial factors. The following partial factors are recommended for structural design for permanent loads: $\gamma_G = 1.35$ and $\xi = 0.85$ and for variable loads $\gamma_Q = 1.5$ and $\Psi_0 = 0.5$ (snow). The load combination rule 6.10(a,b) is applied; for the considered load ratios (see below) relationship (6.10b) with a reduced permanent action effect is dominating.

	APF ($\beta_{0,10} = 2.8$)	APF* ($\beta_{0,10} = 2.8$)	APF ($\beta_{up,10} = 3.3$)	APF* ($\beta_{up,10} = 3.3$)	PM ($\beta_{0,10} = 2.8$)	PM ($\beta_{up,10} = 3.3$)
γ_G	1.07	1.10	1.08	1.11	1.12	1.13
γ_Q	1.10	1.40	1.24	1.64	1.42	1.65
γ_{M0}	0.97	0.84	1.00	0.85	0.85	0.86

* Adjusted partial factors calculated with the actual values of the sensitivity factors ($\alpha_R = 0.2$, $\alpha_G = 0.2$, and $\alpha_Q = 0.95$).

Table 2. Comparison of partial factors ($\chi = 0.8$).

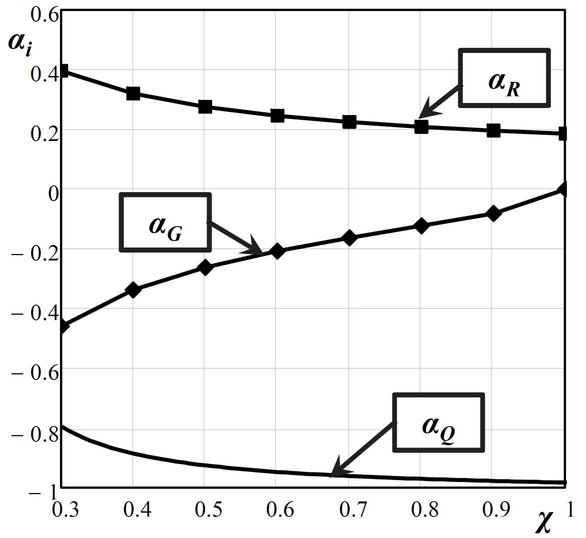

Figure 1. Variation of sensitivity factors with χ.

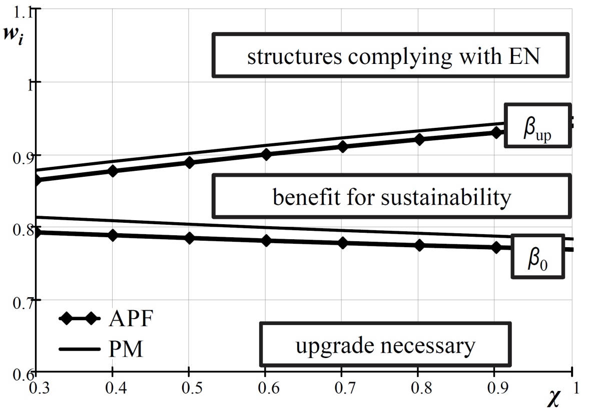

Figure 2. Variation of w_i with χ (APF based on the actual values of sensitivity factors – $\alpha_R = 0.2$, $\alpha_G = 0.2$, and $\alpha_Q = 0.95$).

Using the adjusted partial factors and the probabilistic method (the First Order Reliability Method FORM), partial factors are derived to provide for the target reliability index. 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. The load ratio may vary within the interval from nearly 0 (underground structures, foundations) up to nearly 1 (local effects on crane girders). For steel structures, $0.5 \leq \chi \leq 1$ is expected [9]. The values of the partial factors are presented in Table 2 for $\chi = 0.8$.

The main deficiency of the APF is that the generalised sensitivity factors are applied. More precisely, it is possible to determine the values of the partial factors using the actual values of the sensitivity factors obtained by FORM. Figure 1 displays variation of the sensitivity factors with the ratio χ. For adjusted partial factors the sensitivity factor could be recommended $\alpha_E =$ -0.95 for the snow load, $\alpha_E =$ -0.2 for the permanent load and $\alpha_R = 0.2$ for resistance. The results the APF and PM become close when using these values of the sensitivity factors.

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

$$g(x) = W f_{yk}/\gamma_{M0} - [\gamma_G G_k + \gamma_Q C_0 S_k]. \qquad (2)$$

Figure 2 displays variation of the standardised ratio $w_i = W_i/W_{EN}$ with χ, where W_{EN} is the reference value based on the partial factors recommended in Eurocodes for structural design. When $w_i < 0$, the reliability requirements according to approach "i" are lower than those according to Eurocodes for structural design.

Figure 2 shows that the adjusted partial factors (APF) and probabilistic method PM lead to the reliability requirements 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 that reduces 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 sustainability 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 Czech standards valid before Eurocodes has been introduced, ratio w_i is expected to range approximately from 0.75 (χ close to unity) to 0.85 (χ close to 0.3) when the roof snow load is the leading variable action. These estimates are based on the results of detailed reliability analysis of existing steel roofs exposed to snow loads in the Czech Republic [10]. Such low w_i-values are attributed to increased design roof snow loads as introduced by Eurocodes.

5. Discussion on appropriate sustainability indicator for assessment of existing structures

This example provides first insights into the sustainability benefits possibly gained by applying advanced reliability methods and considering the target reliability levels optimised for existing structures. The fundamental decision in reliability assessments – whether the existing structure can be used without upgrade or upgrade is needed – is analysed focusing on the roof girder investigated in Section 4. Various sustainability indicators have been proposed to quantify the effects of decisions about structures on sustainability. For instance, Müller et al. proposed a simplified measure – building material sustainability potential (BMSP) [11]. To focus on the main aspects and allow for analysing a range of the assessment situations of practical relevance, this simple indicator is considered:

$$BMSP = P \times SL/EI, \tag{3}$$

where

P performance;

SL service life; and

EI environmental impact.

It is assumed that the existing structure under consideration fully provides its function if an Ultimate Limit State (ULS) criterion is fulfilled, $P = 100\,\%$. When the ULS condition is violated, the structure should be closed, P = 0 %. Service life is measured in years with a reference level considered here as 50 years, and then SL = 100 %. If a service life is estimated as 25 years, then SL reduces to 50 %. Very small environmental impact is assumed when the existing structure is continuously used without upgrade, $EI \approx 0$, while EI increases proportionally with the level of strengthening, $EI > 0$. When comparing two alternatives, $BMSP_A > BMSP_B$ should indicate alternative A being preferable.

However, it can be argued whether BMSP is an appropriate indicator for comparing decision alternatives about existing structures since the environmental impact is close to zero for a "no upgrade" alternative and BMSP converges to infinity. To further illustrate the need for modification of BMSP, let us assume that:

- Benefit ($P \times SL$) and environmental impact related to maintenance and upgrade (EI) can be both expressed in monetary terms; typically the former as a gain and the latter as a loss.
- The benefit from using the structure, ($P_0 \times SL_0$), may be much larger than the environmental impact EI_0 that is now related to maintenance only; as an example ($P_0 \times SL_0$) = 100 units and $EI_0 = 1$ unit; the total gain from using the structure is thus 99 units over service life.
- In the case of upgrade, performance level may be retained, service life may be doubled and likewise related benefit, $P_0 \times SL_{up} = 200$ units. The upgrade may have significant environmental impact; for instance 10-times increased compared to "no upgrade", $EI_{up} = 10EI_0 = 10$ units. The total gain is then 190 units, nearly doubled in comparison to "no upgrade".

In this example, BMSP for the structure "as it is" and upgraded would be:

$$BMSP_0 = 100/1 = 100 > BMSP_{up} = 200/10 = 20$$

and the "no upgrade" strategy should be preferred. However, a comparison of the total gains clearly points to the opposite.

Based on these arguments, it is thus proposed to modify the sustainability indicator for decision making about existing structures when benefits and losses can be expressed in the same units:

$$SI = P \times SL - (EI + C). \tag{4}$$

The term in brackets denotes the expected losses. Newly introduced cost C should cover all expenses related to maintenance and possible upgrade. When the owner (particularly the society) saves financial resources, these may be utilised to implement measures positively contributing to sustainability.

Focusing on the girder analysed in Section 4, Figure 3 displays variation of sustainability indicator SI, benefit expressed as SL, and losses C with ratio w that is the property of the girder "as it is".

The trends of SI, SL, and C are estimated on the basis of the assumptions discussed below. As a reference level, the maximum benefit is assumed to be related to service life of 50 years, $SL_{max} = 100$ units. No distinction between performance levels is made – the girder just needs to comply with reliability requirements and then the building can be fully used; P is thus disregarded hereafter.

Assumptions related to losses, C:

(1.) Based on the detailed analysis in [9], upgrade cost is assumed to correspond to about 40 % of the total benefit, 40 units, out of which 50 % is fixed cost independent of w (costs of surveys, assessment, administration and management, economic losses due to business interruption or replacement of users, etc.). Maintenance is disregarded for simplification. Environmental impact was ignored in [9].

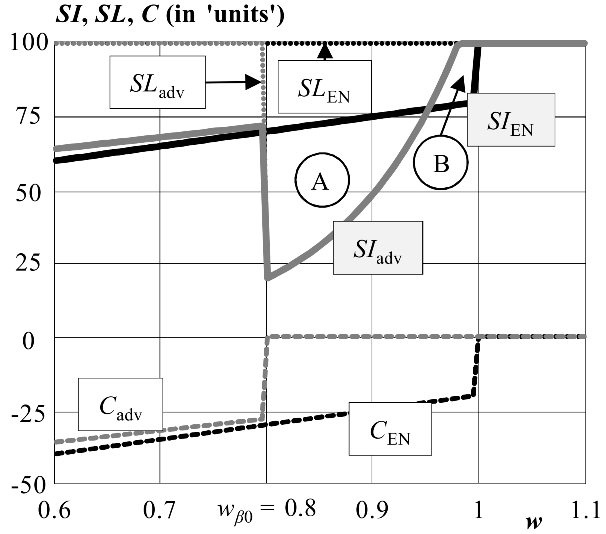

Figure 3. Variation of sustainability indicator SI, benefit SL, and loss C with ratio w (neglected EI).

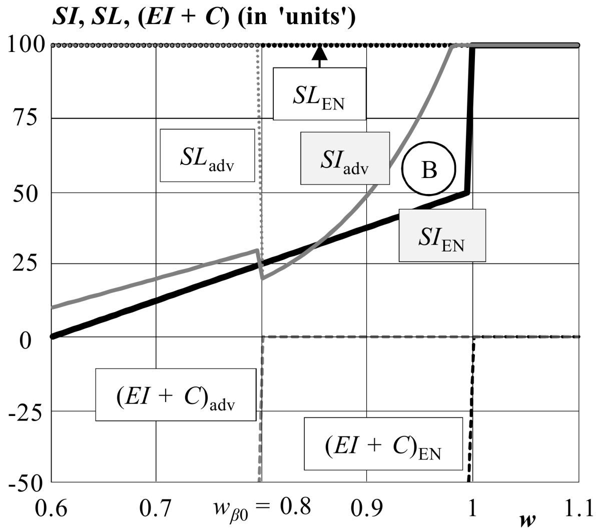

Figure 4. Variation of sustainability indicator SI, benefit SL, and losses $(EI+C)$ with ratio w (including EI).

(2.) Focusing initially on application of the partial factor method according to Eurocodes ("EN"), upgrade is needed for $w < 1$ while no structural intervention takes place otherwise. Full upgrade is assumed to be associated with $w = 0.6$; upgrade cost then linearly decreases with increasing w and drops to zero for $w = 1$ when the existing girder complies with the EN requirements.

(3.) For $0.6 \leq w < w_{\beta 0} = 0.8$ (Figure 2), similar assumptions for upgrade cost apply when the advanced methods are used. Reliability of the girder is below β_0 and an upgrade is necessary. As the optimum upgrade level is slightly below to that required by EN, $w_{adv,upgrade} \approx$ 0.9–0.95 (Figure 2), $C_{adv}(w)$ is slightly lower than $C_{EN}(w)$. For $w \geq w_{\beta 0}$, no upgrade is needed.

Assumptions related to benefit, SL:

(4.) For $w < 1$, EN requires upgrading. The upgrade is assumed to provide for a service life of 50 years. When $w \geq 1$, the existing girder meets the EN requirements and it is again assumed to have a service life of 50 years.

(5.) For $0.6 \leq w < w_{\beta 0}$, SL_{adv} is also 100 units as upgrade provides for a 50-year service life. For $w = w_{\beta 0}$ the girder exactly complies with β_0 for $t_{ref} = 10$ years (Section 4) and thus a 10-year service life is guaranteed, yielding $SL_{adv}(w_{\beta 0}) = 20$ units. When w increases above $w_{\beta 0}$, t_{ref} can be increased to comply with the β_0-requirement. For $w = 0.98$, a 50-year service life is reached, $SL_{adv}(w \geq 0.98) = 100$ units.

Comparison of SI-values – evaluated according to Equation (4) and plotted in Figure 3 – indicates that:

- For $0.6 \leq w < w_{\beta 0}$ and $w \geq 1$, the use of advanced methods has a small effect on the SI-values.
- For $w_{\beta 0} \leq w < 0.96$ (marked in Figure 3 as "A"), the use of advanced methods seems to lead to lower SI-values as a service life lower than 50 years would be authorised. In this case with a relatively low upgrade cost, upgrading seems beneficial.
- For $0.92 \leq w < 1$ (marked as "B"), the advanced methods provide benefit as upgrade is unnecessary while the EN-based assessment indicate otherwise.

In [9] no account for environmental impact was taken. During upgrade, environmental impact may cover for instance material consumption, transportation of materials and equipment and related fuel consumption and emissions etc. To illustrate the effect of environmental impact, let us consider that $(EI + C)$ for full upgrade is 2.5-times higher than in the previous case, $(EI + C) = 100$. All other assumptions remain unchanged. Figure 4 portrays variation of SI, SL, and $(EI + C)$ with w. It appears that increased upgrade cost $(EI + C)$ significantly change the obtained SI-values:

- While for $0.6 \leq w < w_{\beta 0}$ there is again a small difference between SI_{adv} and SI_{EN}, for $w_{\beta 0} \leq w < 1$ area "A" nearly vanishes and area "B" remains – the use of the advanced methods is beneficial.
- For $w \geq 1$, there is again no difference between using the advanced methods or EN.

It is emphasised that the example presented in this section is intentionally simplified focusing on the main aspects of decision making and implications for sustainability. Situations when decision making may be more complex include:

- For very low resistance ($w < w_{\beta 0}$), the girder is considered unreliable and decision should be made whether it should be replaced or should be upgraded; the example indicates that environmental impact plays a significant role in this decision making.

- For low but possibly acceptable resistance ($w_{\beta 0} < w < 1$), the girder may be considered reliable with a reduced service live. It should be then decided whether the girder can be preserved, upgraded or replaced by a new structure.
- Even for sufficient resistance ($w \geq 1$), upgrading or replacement may be considered to reach a longer service life of the structure. However, this decision should be made with caution as useful service life of buildings is mostly affected by a number of factors causing obsolescence that are beyond the control of civil engineers (economic, functional or technological obsolescence, failure to meet legal requirements) [12].

Large investments in the attempt to achieve long service life from the reliability perspective may then be in vain. Within further research, the obtained results will be verified considering a wide range of factors to quantify the overall sustainability impact of various assessment strategies by a full probabilistic approach as proposed by Webb and Ayyub [13]. Also, the use of surveys results should generally reduce uncertainties in basic variables and increase reliability estimates for existing structures, making it possible to avoid or minimise structural interventions.

6. Conclusions

While the sustainability considerations require to maintain and keep using existing structures, structural assessments often indicate insufficient reliability levels and need for replacements. The submitted contribution investigates how this situation may be solved by applying advanced reliability assessment methods. It is demonstrated how the advanced methods may bring a significant positive contribution to sustainability, particularly when an upgrade of the existing structure is associated with high economic cost and large environmental impact. It is newly proposed to modify the sustainability indicator for decision making about existing structures considering associated benefits and losses expressed in the same units.

Case studies show that the application of advanced probabilistic approaches reduces the assessment requirements by 20–25 % when the minimum reliability level is accepted, and by 5–10 % when the optimum reliability level for upgrades is considered. It is demonstrated that the application of advanced reliability methods may allow continued use of existing structures when conservative methods of structural design may indicate needs for upgrading.

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