Acta Polytechnica CTU Proceedings **36**:216–223, 2022
https://doi.org/10.14311/APP.2022.36.0216

A STUDY ON THE EFFECTS OF HIDDEN SAFETY WHEN ASSESSING EXISTING STRUCTURES

Max Teichgräber*, Daniel Straub

Technical University Münich, School of Engineering and Design, Engineering Risk Analysis Grou, Theresienstraße 90, 80333 München, Germany

* corresponding author: max.teichgraeber@tum.de

Abstract. In many instances, the safety of existing structures can no longer be demonstrated by standard code-based assessments. Reasons for this include changes in the code, changes in the demands on the structures and deterioration. To address this problem, it is common practice to perform a more detailed assessment utilizing advanced models. In this way, many structures can be shown to comply with safety requirements, even if they cannot be verified by standard assessments. The standard code models are often conservatively biased. This leads to designs which include hidden safety. If the reassessment is performed with more advanced models in lieu of standard models, the hidden safety can vanish. Concurrently, the reduced uncertainty of advanced models may compensate this safety reduction. In this paper we investigate this issue on a hypothetical population of existing bridge structures under traffic. We consider that the standard code model is exchanged by an advanced traffic load simulation.

Keywords: Code based design, existing bridge structures, hidden safety, traffic load.

1. Introduction

Structural design codes aim to provide easy-to-use rules that lead to economical and safe structures [1]. Today's structural design codes are based mainly on the semi-probabilistic partial safety factor (PSF) concept [2–5]. It contains two explicit safety components: The PSFs and the characteristic values. Both components are determined via code calibration [6–8]. The calibration is typically based on standard models and the determined safety components are intended to be used with these standard models. In many cases the standard models are biased in a conservative sense (e.g. [9]). This leads to another implicit safety component: The hidden safety.

The presence of hidden safety is unproblematic, as long as standard models are applied; however, problems may arise if standard models are exchanged by more advanced models. Thereby, two effects are present: First, advanced models are typically not biased; hence, the hidden safety vanishes. Second, advanced models typically include less model uncertainty.

When it comes to the design of new structures, these two effects influence the structural reliability in opposite directions. If one effect dominates, a recalibration of the explicit safety components of the PSF concept is needed. The recalibration ensures that, on average, the standard and the advanced models lead to the same level of safety. We investigate this in detail in [9, 10].

In the assessment of existing structures both effects are present as well. The difference to new designs is that for existing structures the design choices are already made; hence, the advanced models are only used to reassess if a given design is in compliance with the requirements. Whether or not the accepted design is safe enough is the question addressed in this paper.

We perform a case study on a hypothetical population of existing bridge structures under traffic load. We consider an increase of the traffic load since the construction date of the bridges such that they are no longer in compliance with standard code-based assessments. We investigate how the application of advanced traffic load simulation affects the reliability.

2. Design following the partial safety factor concept

We follow the approach and the nomenclature of the Eurocode [11]. The investigations can be transferred to other semi-probabilistic structural design codes.

In a Eurocode design, four different models can be identified (Figure 1): The load model $\mathcal{M}_{L,EC}$, the structural model $\mathcal{M}_{S,EC}$, the material model $\mathcal{M}_{M,EC}$, and the resistance model $\mathcal{M}_{R,EC}$. We use the subscript EC to stress that these are the standard models provided by Eurocode.

The load model and the material model are typically probabilistic models. In order to make the design process deterministic, a characteristic load $l_{k,EC}$ and characteristic material properties $m_{k,EC}$ are determined. To consider the uncertainty of the load model and the material model and to ensure a sufficiently safe structure, the characteristic values are modified by the PSFs γ_f and γ_m. The resulting values are the input of functions $t_{S,EC}$ and $t_{R,EC}$ provided by the structural model and the resistance model. To consider the uncertainty of these models, the outcome is

Load model $\mathcal{M}_{L,EC}$		Material model $\mathcal{M}_{M,EC}$
↓		↓
Characteristic load $l_{k,EC}$		Characteristic material $m_{k,EC}$
↓		↓
Partial safety factor γ_f		Partial safety factor γ_m
↓		↓
Structural model $\mathcal{M}_{S,EC}$		Resistance model $\mathcal{M}_{R,EC}$
↓		↓
Partial safety factor γ_{Sd}		Partial safety factor γ_{Rd}
↓		↓
Design load effect $e_{d,EC}$	$\overset{!}{\leqslant}$	Design resistance $r_{d,EC}$

FIGURE 1. Overview of the Eurocode design appraoch.

again modified by PSFs γ_{Sd} and γ_{Rd}. Eventually, the design values $e_{d,EC}$ and $r_{d,EC}$ are obtained. A design is sufficient, if the design values fulfill the following inequality:

$$e_{d,EC} \leq r_{d,EC} \tag{1}$$

$$\Leftrightarrow \quad \gamma_f \cdot t_{S,EC}(l_{k,EC} \cdot \gamma_{Sd}) \leq \frac{t_{R,EC}\left(\frac{m_{k,EC}}{\gamma_m}\right)}{\gamma_{Rd}}$$

For the sake of simplicity, the Eurocode merges the partial safety factors of the action and the resistance side:

$$\gamma_F = \gamma_f \times \gamma_{Sd} \tag{2}$$

$$\gamma_M = \gamma_m \times \gamma_{Rd} \tag{3}$$

In the subsequent case study we assume $t_{S,EC}$ and $t_{R,EC}$ to be linear functions through the origin. Then Equation 1 simplifies to:

$$\gamma_F \cdot t_{S,EC}(l_{k,EC}) \leq \frac{t_{R,EC}\left(m_{k,EC}\right)}{\gamma_M} \tag{4}$$

If the design is chosen optimally in regard to resource consumption, equality holds in Equation 4. From this, $\mathcal{P}_{EC}$ is derived:

$$\mathcal{P}_{EC} = \frac{\gamma_F \cdot \gamma_M \cdot t_{S,EC}(l_{k,EC})}{t_{R,EC}\left(m_{k,EC}\right)} \tag{5}$$

$\mathcal{P}_{EC}$ can be interpreted as the design choice following the PSF concept (e.g. the cross section area of a truss).

The corresponding probability of failure can be determined via the following limit state function (LSF):

$$g = \mathcal{P}_{EC} \cdot t_R(M) - t_S(L) \tag{6}$$

Note that in Equation 6 t_R, t_S, M and L are not subscripted with EC, hence, do not originate from the Eurocode. M and L are the "true" – or as we call them: purely aleatoric [9] – distributions of the load and the material property. t_R and t_S represent the true relationship between the load and the material property to the load effect and the resistance. Generally, these are unknown and have to be estimated carefully.

The probability of failure $\Pr(F)$ results from

$$\Pr(F) = \int_{\Omega_F} f_{M,L}(m,l)\,\mathrm{d}m\,\mathrm{d}l \tag{7}$$

where $f_{M,L}(m,l)$ is the joint probability density function (PDF) of M and L and $\Omega_F = \{m,l \mid g < 0\}$ is the failure domain.

3. Case study: Existing bridge structures under traffic load

In the following we perform a case study on a hypothetical population of existing bridge structures under traffic load.

3.1. Setup and assumptions

We choose the following setup of the case study:

- $n = 10\,000$: Number of samples of the population of bridge structures. This is needed, since some of the subsequent calculations are sample-based. Apart from the statistical error originating from the sampling, the results are not effected by n.
- We choose the material property M to follow a lognormal distribution:

  $$M \sim \mathcal{LN} \qquad \mathrm{E}[M] = 1 \qquad \mathrm{c.o.v.}[M] = 0.1 \tag{8}$$

  The coefficient of variation of 0.1 is in the typical range of material properties (e.g. concrete compression strength) [12]. Some material properties may have higher (e.g. timber bending strength) or lower coefficients of variation (e.g. structural steel yielding strength). Therefore, we vary the coefficient of variation between 0.05 and 0.3 in a sensitivity analysis.
- We choose the annual maximum of the traffic load L to follow a standardized Gumbel distribution:

  $$L \sim \mathcal{G} \qquad \mathrm{E}[L] = 1 \qquad \mathrm{c.o.v.}[L] = 0.08 \tag{9}$$

  The choice is based on simulated traffic following the approach of [13]: Local traffic data from three locations of German highways with light, medium and heavy traffic is analyzed. On this bases, traffic streams (sequences of vehicles) are generated applying numerical traffic simulation. The simulated traffic load is applied on a portfolio of bridges. The portfolio includes bridges with 1-3 spans, each with 10, 15, or 20 m. On each bridge we apply either light, medium and heavy traffic. Hence, the portfolio consists of $3 \cdot 3 \cdot 3 = 27$ bridges. For each bridge a structural analysis is carried out and the resulting 100-year time histories of the internal forces are evaluated at all locations that are needed for the design. The annual block maxima of the time histories are taken to fit generalized extreme values distributions via the Maximum-Likelihood-Method. The shape parameters of the fitted generalized extreme value distributions are close to 0, in which case the generalized extreme value distribution converges to the Gumbel distribution. The sample mean of the coefficient of variation of the field moments is 0.06, of the supporting moments 0.08 and of the shear forces at the supports 0.08. This justifies the choice of $\mathrm{c.o.v}[L] = 0.08$. Note that L is the purely aleatoric distribution; hence, $\mathrm{c.o.v}[L] = 0.08$ only covers aleatoric uncertainties. Epistemic uncertainties are added via the error in the estimation of the characteristic load (see Θ_{EC} and Θ_{adv} below).

We alter the coefficient of variation between 0.05 and 0.3 in a subsequent sensitivity analysis. A coefficient of variation of 0.3 is rather high in the context of traffic loads. By this, we ensure to also cover design situations with high uncertainty (e.g. due to rerouting of roads).

- As already mentioned, we assume that the functions $t_{S,EC}$ (translates the loads to the load effects) and $t_{R,EC}$ (translates the load property to the resistance) are both linear and trough the origin. We further assume that both functions are equal to the true functions t_S and t_R. With this assumptions the functions $t_{S,EC}$, t_S, $t_{R,EC}$ and t_R cancel each other out within the LSF (Equation 6).
- We assume a traffic load increase since the construction of the bridge by a factor of $f_{inc} = 1.3$. This value is not on the bases of any data but should be interpreted as a hypothetical increase. In an actual application case, this value must be derived respectively. We vary f_{inc} between 1 and 2 in a subsequent sensitivity analysis.
- Following the traffic load model of the Eurocode [14], we define the characteristic traffic load as the 99.9% quantile of L (1000 year return period which corresponds to the probability of exceedance of 5% in 50 years). We model the error in the estimation of the characteristic value via a relative error:

$$\Theta_{adv} = \frac{l_k}{l_{k,adv}} \tag{10}$$

$$\Theta_{EC} = \frac{l_k}{l_{k,EC}} \tag{11}$$

where l_k is the characteristic value of the purely aleatoric traffic load, $l_{k,adv}$ is the estimation of l_k by means of advanced traffic load modeling and $l_{k,EC}$ is the estimation of l_k following the Eurocode.

We choose Θ_{adv} and Θ_{EC} to follow lognormal distributions:

$$\Theta_{adv} \sim \mathcal{LN} \quad \mathrm{E}[\Theta_{adv}] = 1 \qquad \mathrm{c.o.v}[\Theta_{adv}] = 0.1 \tag{12}$$

$$\Theta_{EC} \sim \mathcal{LN} \quad \mathrm{E}[\Theta_{EC}] = 0.7 \quad \mathrm{c.o.v}[\Theta_{EC}] = 0.2 \tag{13}$$

The choices of the distribution-parameters are justified as follows:

▷ $\mathrm{E}[\Theta_{adv}] = 1$: Since the simulation is based on real data, it is reasonable to assume that it estimates the characteristic value without a bias.

▷ $\mathrm{c.o.v}[\Theta_{adv}] = 0.1$: The coefficient of variation comprises three different uncertainties: The statistical uncertainty due to a limited simulation time, the uncertainty of the chosen distribution type and the uncertainty of incomplete modeling (e.g. construction sites on the highway may change the traffic load, but are not included in the simulation). The statistical uncertainty is estimated via the multivariate normal distribution of the parameter estimates of the fitted extreme value distributions of the inner forces [15]. The resulting coefficients of variation due to statistical uncertainty are in the range of 0.02-0.04. We – unfortunately – lack any data to estimate the uncertainty of the two other sources; hence, the overall uncertainty of $\mathrm{c.o.v}[\Theta_{adv}] = 0.1$ is based on our subjective assessment. We alter $\mathrm{c.o.v}[\Theta_{adv}]$ between 0.05 and 0.2 in a subsequent sensitivity analysis.

▷ $\mathrm{E}[\Theta_{EC}] = 0.7$: Within the portfolio of bridge structures the sample mean of the relative error is 0.76 regarding field moments, 0.70 regarding supporting moments and 0.79 regarding the shear forces at the supports. We apply a value of 0.7 and alter $\mathrm{E}[\Theta_{EC}] =$ between 0.4 and 1.0 in a subsequent sensitivity analysis.

▷ $\mathrm{c.o.v}[\Theta_{EC}] = 0.2$: Θ_{EC} can be rewritten as $\Theta_{EC} = \frac{l_k}{l_{k,adv}} \cdot \frac{l_{k,adv}}{l_{k,EC}} = \Theta_{adv} \cdot \frac{l_{k,adv}}{l_{k,EC}}$. The distribution parameters of Θ_{adv} is already known/estimated. The distribution of $\frac{l_{k,adv}}{l_{k,EC}}$ is also lognormal. The mean coefficient of variation of $\frac{l_{k,adv}}{l_{k,EC}}$ is 0.17 regarding field moments, 0.13 regarding supporting moments and 0.14 regarding the shear forces at the supports. We follow the biggest value of 0.17. This results in a a coefficient of variation of Θ_{EC} of approximately 0.2. We alter $\mathrm{c.o.v}[\Theta_{EC}]$ between 0.1 and 0.3 in a subsequent sensitivity analysis.

- Following Eurocode [11], we define the characteristic material property as the 5% quantile of M. We assume that $m_{k,EC}$ exactly meets m_k, hence, that no estimation error in the determination of the characteristic value is present.
- Following Eurocode [11] we choose the PSF $\gamma_F = 1.35$. We further set $\gamma_M = 1.1$.

3.2. Numerical investigations

Our main objective is to investigate the probability of failure of the bridges within the hypothetical population designed by Eurocode and the change of the probability of failure when more advanced traffic load models are used. We investigate how parameter changes in the setup influence the mean probability of failure and the acceptance rate of reassessed bridge structures.

Our secondary objective is to understand how the probability of failure would change if the advances models would have been used to design the bridges.

We only cover this issue for the setup defined in section 3.1 and do not investigate parameter changes within the setup.

By rearrange Equation 10 and 11, we determine the distribution of the characteristic traffic load according to Eurocode and according to advanced traffic load modeling by solving Equation 11 and 10 for $l_{k,EC}$ and $l_{k,adv}$. The resulting distributions are:

$$L_{k,EC} \sim \mathcal{LN} \quad \mu_{\ln L_{k,EC}} = \ln(l_k) - \mu_{\ln \Theta_{EC}}$$
$$\sigma_{\ln L_{k,EC}} = \sigma_{\ln \Theta_{EC}} \tag{14}$$
$$L_{k,adv} \sim \mathcal{LN} \quad \mu_{\ln L_{k,adv}} = \ln(f_{inc} \cdot l_k) - \mu_{\ln \Theta_{adv}}$$
$$\sigma_{\ln L_{k,adv}} = \sigma_{\ln \Theta_{EC}} \tag{15}$$

where $\mu_{\ln}$ and $\sigma_{\ln}$ are the location and the scale parameter of the respective lognormal distribution.

We sample $l_{k,EC,i}$ and $l_{k,adv,i}$ from $L_{k,EC}$ and $L_{k,adv}$ simulating the estimation of the characteristic traffic load of $i = 1, ..., 10\,000$ bridges. We sample from $L_{k,EC}$ and $L_{k,adv}$ independently.

Given one sample $l_{k,i}$ (either from the characteristic value of the Eurocode or the advanced traffic load model) the probability of failure is calculated as:

$$\Pr(F) = \int_{\Omega_L} F_M\left(\frac{f_{inc} \cdot l \cdot m_k}{\gamma_M \cdot \gamma_F \cdot l_{k,i}}\right) \cdot f_{inc} \cdot f_L(l)\, \mathrm{d}l \tag{16}$$

First, we calculate the probability of failure for the case of $f_{inc} = 1$. From this our secondary objective (how the probability of failure changes if the advances models would have been used to design the bridge) can be studied. In a next step, we increase the traffic load $f_{inc} > 1$ to investigate how the increase in the load changes the probability of failure and how a reassessment using advanced traffic load modeling affects the probability of failure).

The reassessment decision is based on the following train of thought: Advanced traffic load modeling may estimate a characteristic traffic load that is lower than the characteristic traffic load according to Eurocode (even when an increased traffic load is present). If this lower characteristic value would be used to design the bridge, it would result in smaller resistances; therefore, the reassessment according to advanced traffic load modeling would deem the existing Eurocode design to be sufficient. From this, the following acceptance/rejection rule is derived:

$$l_{k,adv,i} \leq l_{k,EC,i} \Rightarrow \text{accept} \tag{17}$$
$$l_{k,adv,i} > l_{k,EC,i} \Rightarrow \text{reject} \tag{18}$$

3.3. Results and Discussion

Figure 2 shows the probabilities of failure (annual reference period) of the bridges designed by Eurocode loaded with the original traffic load ($f_{inc} = 1$). The mean annual probability of failure is $\mathrm{E}[\Pr(F \mid \text{EC-Design}, f_{inc} = 1)] = 4.9 \cdot 10^{-8}$. This probability can be interpreted as the target probability of failure,

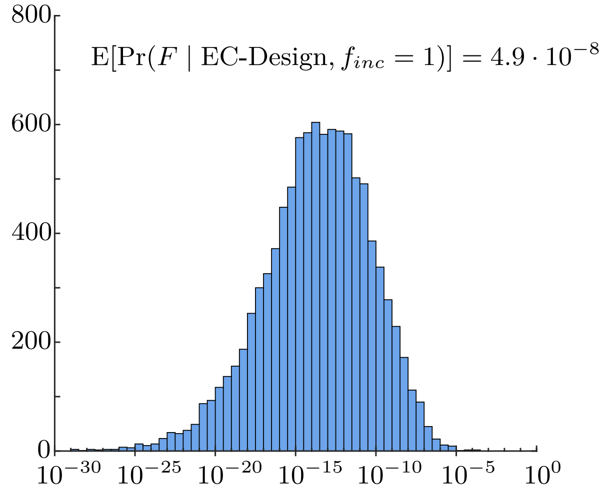

Figure 2. Histogram of the probability of failure $\Pr(F \mid \text{EC-Design}, f_{inc} = 1)$ of bridge structures designed following Eurocode.

since the original design criteria reaches this probability of failure on average and these designs were accepted by society. The probability is rather low compared to the common range of probabilities of failures of structures or compared to the target probability of failure defined in the Eurocode of $8.5 \cdot 10^{-6}$. The reasons for the deviation are as follows: We do not consider model uncertainties of the structural model, the material model and the resistance model. Moreover, we applied only traffic load on the bridge structures. If also other loads would be considered (with a higher coefficient of variation compared to the coefficient of variation of the traffic load), the probability of failure might be higher. We do these simplifications to keep the issue simple. This way the effects hidden safety are more isolated and more straightforward to study. In case of an actual application these simplifications should not be conducted.

Due to these simplifications the absolute value of the probability of failure may not be very meaningful. However, a relative comparison to the probabilities of failure of the bridges designed or accepted/rejected by advanced traffic load models is valid, since the probabilities of failure are calculated on the same basis.

Figure 3 shows the probabilities of failure that would result if the bridges (loaded with the original traffic load $f_{inc} = 1$) would have been designed using advanced traffic load modeling. The resulting mean probability of failure $\mathrm{E}[\Pr(F \mid \text{adv-Design}, f_{inc} = 1)] = 3.1 \cdot 10^{-7}$ is below the target of $4.9 \cdot 10^{-8}$. This shows, that the negative effect on the structural reliability of the lost hidden safety due to advanced traffic load modeling is stronger than the positive effect of the reduced model uncertainty. An additional safety factor would be needed if advanced traffic load models were to be used for bridge design. Here, we do not conduct a calibration of such a factor. A detailed guide is

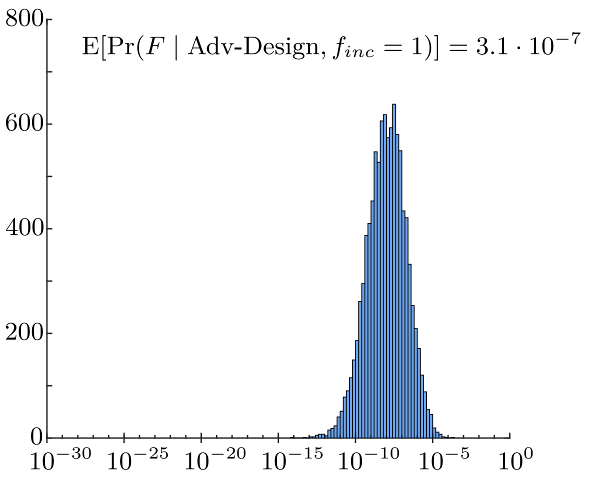

FIGURE 3. Histogram of the probability of failure $\Pr(F \mid \text{Adv-Design}, f_{inc} = 1)$ of bridge structures designed following advanced traffic load modeling.

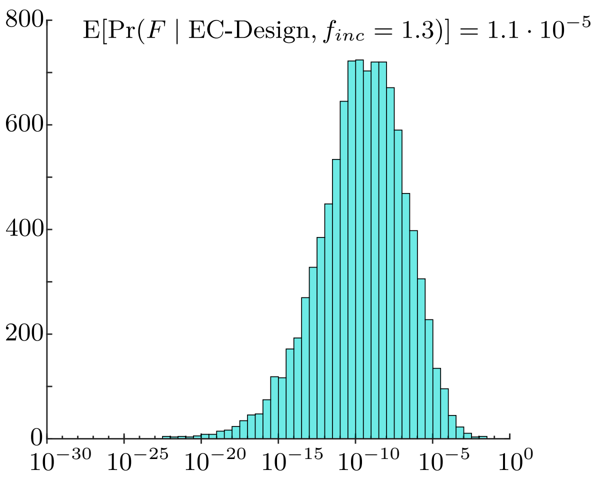

FIGURE 4. Histogram of the probability of failure $\Pr(F \mid \text{EC-Design}, f_{inc} = 1.3)$ of bridge structures designed following Eurocode loaded by a 30 % higher traffic load.

given [9].

Remark: The resulting probabilities of failure of the advanced designs with an increased load ($f_{inc} > 1$) would be exactly the same as in Figure 3, as long as the load increase is also considered within the design. f_{inc} would appear twice and cancel each other out: Once within the design (a factor applied on $l_{k,adv,i}$) and once as a factor applied on the load L.

Figure 4 shows how the probabilities of failure of the bridges designed according to Eurocode change if the traffic load is increased by 30 % ($f_{inc} = 1.3$). The increased traffic load raises the mean probability of failure from $4.9 \cdot 10^{-8}$ to $1.1 \cdot 10^{-5}$.

Figure 5 subdivides the histogram of Figure 4 into accepted/rejected cases (according to advanced traffic load modeling). 68.8 % of the bridge structures are accepted and 31.1 % are rejected. The mean probability of failure of the accepted bridges

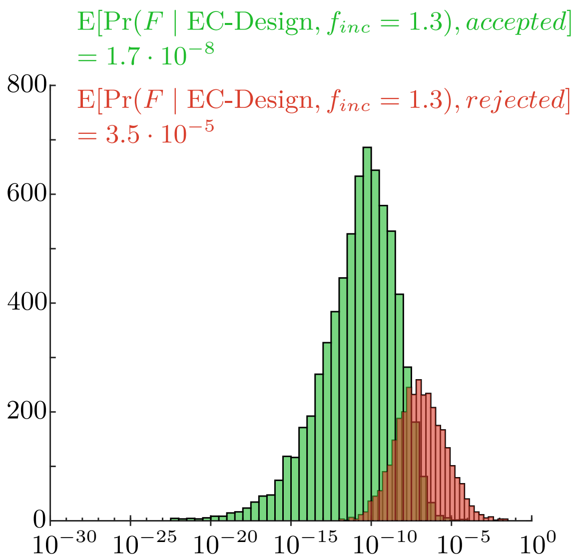

FIGURE 5. Histogram of the probability of failure $\Pr(F \mid \text{EC-Design}, f_{inc} = 1.3)$ of bridge structures designed following Eurocode loaded by an 30 % increased traffic load divided into accepted (green) and rejected (red) bridges according to advanced traffic load modeling.

$\mathrm{E}[\Pr(F \mid \text{EC-Design}, f_{inc} = 1.3), \text{accepted}] = 1.7 \cdot 10^{-8}$ is smaller than the target probability of failure of $4.87 \cdot 10^{-8}$; hence, the reassessment leads to sufficiently safe structures.

Therefore, the use of advanced models for reassessment of existing structures appears justified without further adjustment of the safety factors. This is in contrast to the design of new structures. The deifference can be understood by comparing the histogram of the probability of failure of the accepted bridge structures (green histogram in Figure 5) to the histogram of the probability of failure of bridge structures designed by advanced traffic load modeling (blue histogram in Figure 3): Following Equation 17 a structure is accepted if $L_{k,adv} \leq L_{k,EC}$. If equality would holds, the accepted Eurocode design and the advanced design are equal, hence both histograms would coincide. The greater the difference $L_{k,EC} - L_{k,adv}$, the more hidden safety remains, which decreases the probability of failure in the reassessment case. We conclude that the application of advanced models to reassess a given structure with increased load

- retains parts of the hidden safety of accepted structures;
- decreases the probability of failure by reducing uncertainty.

Figure 6 shows the changes in the mean probability of failure and the acceptance rate changes, when altering selected parameters of the case study one at a time.

The reassessment of bridge structures should be critically considered, if the mean probability of failure of the accepted structures is greater than the mean

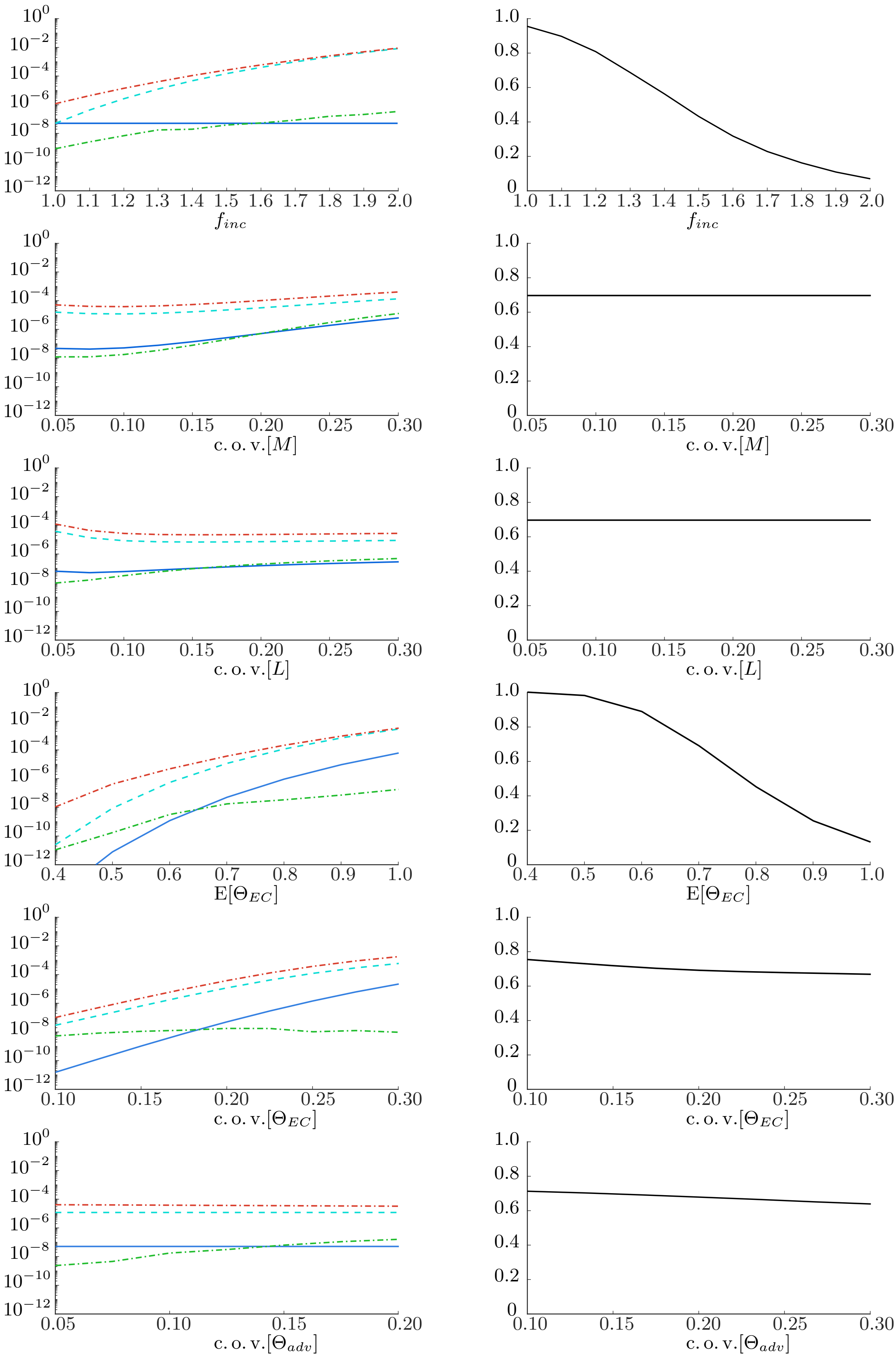

Figure 6. Mean probability of failure (left) and acceptance rate (right) of bridge structures altering setup parameters one at a time. The blue lines represent the mean probability of failure when the original load is applied (target probability of failure), the teal dashed lines represent the mean probability of failure when the increased load is applied and the dash doted green and red lines represent the mean probability of failure of the accepted and the rejected bridge structures.

probability of failure of the bridge structures under the original load (target probability of failure). This is the case if

- the load strongly increases ($f_{inc} > 1.6$). However, this case is not critical, since the target probability of failure is not exceeded by much and the acceptance rate simultaneously drops very low.
- the coefficient of variation of the material property is rather large ($\mathrm{c.o.v}[M] > 0.2$). This is also not critical, since the target probability of failure is only exceeded marginally.
- the coefficient of variation of the traffic load is rather large ($\mathrm{c.o.v}[M] > 0.15$). Again, this is not critical, since the target probability of failure is only exceeded marginally.
- the estimation of the characteristic traffic load according to Eurocode is very biased ($\mathrm{E}[\Theta_{EC}] < 0.65$). This is also not concerning, since in this case the Eurocode design includes high amounts of hidden safety, which leads to very low probabilities of failure. This hidden safety can serve as a reserve against increased traffic loads.
- the estimation of the characteristic traffic load according to Eurocode is associated with low uncertainty ($\mathrm{c.o.v}[\Theta_{EC}] < 0.18$). In this case the coefficient of variation in the estimation of the characteristic value of the Eurocode model and the advanced traffic load model match closely. Hence, the reassessment does not reduce the uncertainty significantly, but reduces the bias (hidden safety). This case can be critical and should be prevented; however, this case is rare, since the uncertainty of advanced models is typically much smaller than the uncertainty of standard models.
- the estimation of the characteristic traffic load according to advanced models is relatively uncertain ($\mathrm{c.o.v}[\Theta_{adv}] > 0.18$). As in the previous bullet point the coefficient of variation in the estimation of the characteristic value of the Eurocode model and the advanced traffic load model are close. This case can also be critical and should be prevented; however, again is rather rare.

Overall, the reassessment of bridge trough advanced modeling in most cases leads to sufficiently safe structures. It is only critical, if $\mathrm{c.o.v}[\Theta_{EC}]$ and $\mathrm{c.o.v}[\Theta_{adv}]$ are very similar. However, the conduced sensitivity analysis only varies one parameter at a time. A case in which multiple parameters are varied simultaneously may lead to a more critical outcome.

4. Conclusion

We investigated the effects of hidden safety if advanced traffic load models replace the standard Eurocode traffic load model. In the design case, the loss of hidden safety increases the probability of failure more than the reduced uncertainty of advanced models decreases it; therefore, advanced traffic load models should not be applied to design new structures without recalibrating the partial safety factors. The reassessment case showed less critical results. In the vast majority of considered cases the accepted bridge structures that are assessed as compliant are sufficiently safe and no recalibration is needed. In this case the hidden safety is only reduced proportional to the load increase and not fully erased (in contrast to the design case).

References

[1] O. Ditlevsen, H. Madsen. *Structural Reliability Methods*. Wiley New York, 1996.

[2] B. Ellingwood, T. Galambos, J. MacGregor. *Development of a Probability Based Load Criterion for American National Standard A58: Building Code Requirements for Minimum Design Loads in Buildings and Other Structures*. Department of Commerce, National Bureau of Standards, 1980.

[3] T. Galambos, B. Ellingwood, J. MacGregor, A. Cornell. Probability Based Load Criteria: Assessment of Current Design Practice. *Journal of the Structural Division* **108**(5):959–977, 1982. https://doi.org/10.1061/JSDEAG.0005958.

[4] B. Ellingwood, J. MacGregor, T. Galambos, A. Cornell. Probability Based Load Criteria: Load Factors and Load Combinations. *Journal of the Structural Division* **108**(5):978–997, 1982. https://doi.org/10.1061/JSDEAG.0005959.

[5] Deutsches Institut für Normung. *GruSiBau: Grundlagen zur Festlegung von Sicherheitsanforderungen für bauliche Anlagen [GruSiBau: Basic principles for the definition of safety requirements for structures]*. Beuth Verlag, 1981.

[6] H. Madsen, S. Krenk, N. C. Lind. *Methods of Structural Safety*. Courier Corporation, 2006.

[7] M. Faber, J. Sørensen. Reliability-Based Code Calibration: The JCSS Approach. *Proceedings of the 9th International Conference on Applications of Statistics and Probability in Civil Engineering* **9**:927–935, 2003.

[8] M. Baravalle. *Risk and Reliability Based Calibration of Structural Design Codes*. Ph.D. thesis, Norwegian University of Science and Technology, 2017.

[9] M. Teichgräber, J. Köhler, D. Straub. Hidden safety in structural design codes. *Engineering Structures* **257**:114017, 2022. https://doi.org/10.1016/j.engstruct.2022.114017.

[10] M. Teichgräber, J. Köhler, D. Straub. Über den Umgang mit versteckten Sicherheiten - Eine Fallstudie am Windlastmodell des Eurocode [on dealing with hidden safeties - a case study on the wind load model of the eurocode]. *Baustatik – Baupraxis 14* 2020.

[11] CEN. Eurocode 0: Basis of structural design, 2002.

[12] Joint Committee on Structural Safety. *The JCSS probabilistic model code*. 2001.

[13] M. Nowak, O. Fischer. Site-specific load models for road bridges – an important tool for advanced evaluation strategies in bridge reassessment. *Beton- und Stahlbetonbau* **112**(12):804–814. https://doi.org/10.1002/best.201700064.

[14] CEN. Eurocode 1: Actions on structures, 2005.

[15] S. Coles, J. Bawa, L. Trenner, P. Dorazio. *An Introduction to Statistical Modeling of Extreme Values*. Springer, 2001.