Probabilistic Framework for Assessment of Existing Bridges Under Abnormal Loads

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ABSTRACT: Reliability-based bridge assessment is an accepted way for assessing bridges in many countries under normal traffic loads. Target reliabilities are either calculated based on available information or specified in codes or guidelines. Adjusted partial factors for resistance and load effects can then show that a structure is fit to carry a specified load, even if code based partial factors may have condemned it, or implied strengthening requirements. Loads that exceed the traffic load models of bridge design codes are becoming more frequent. Examples of such loads are components for wind turbines or components for nuclear power plants. These loads are infrequent and require special provisions for reliability-based assessment. This paper provides a framework for such assessment. In the presented case study, the target reliability is governed by economic optimisation. The resulting utilisation ratio for the assessment based on the adjusted partial factors is significantly lower (by about 23 %) compared to that based on design values, leading to a more favourable analysis for the bridge.

1. INTRODUCTION

Reliability-based bridge assessment is becoming an accepted way of assessing existing bridges in many countries. Procedures for the reliability-based assessment of bridges under normal traffic conditions are well established (Enevoldsen, 2001, 2011; Wiśniewski, Casas & Ghosn, 2012; Cremona & Poulin, 2017; Wisniewski, Casas & Ghosn, 2018; Braml & Kainz, 2022; Melhem & Caprani, 2022). Better data collection and potentially reduced target reliability based on economic optimisation while maintaining human safety requirements enable reduced partial factors when compared to new bridges. This has been shown to enable retainment of some existing bridges, rather than strengthening or replacement (Melhem & Caprani, 2022). However, the implementation of target reliability varies between nations and codes.

On the other hand, the assessment of bridges under controlled abnormal loads, such as permit loads, is not well described in probabilistic terms. The transportation of heavy equipment for nuclear power plants or wind farms, is becoming more frequent as these facilities approach the end of their service lives or are newly established. Loads in excess of 400 t are not uncommon for nuclear equipment and these loads are typically spread out over many axles. A formulation is needed for assessment as the reliability calculation parameters are different than for bridge assessment under normal traffic.

The purpose of this paper is to extend a general formulation provided by Lenner & Sýkora (2016) on the calibration of partial factors for special vehicles on road bridges, extended from the safety concept developed for military traffic (Lenner, 2014). Some typical examples of such abnormal crossings are provided in this work and the partial factor formulation and stochastic models are described. A specific focus is on special transports of occasional abnormal loads, which occur perhaps once in decade(s) when nuclear facilities are refurbished. A discussion of the target reliability and sensitivity factors is provided, followed by a case study and recommendations for future research.

2. PARTIAL FACTOR FORMULATION

The semi-probabilistic format is preferred in current engineering practice and thus it is taken as a basis for the suggested format for reliability verification. It is suggested here that, for the implementation of load and resistance models in the abnormal crossing situation, partial factors (PFs) for each crossing are calibrated for each limit state and mode of failure according to fib (2016), Van der Spuy (2020) and Van der Spuy & Lenner (2021).

The target reliability index, denoted by β_T, is a measure of the probability of failure of a structure for a specified reference period. The probability of failure is denoted by P_f.

$$\beta_T = -\Phi(P_f) \tag{1}$$

Φ in Eq. (1) denotes the standardised normal distribution. β_T is therefore a transformation of the probability of failure, P_f, and a higher β_T implies a lower P_f.

The First Order Reliability Method (FORM) sensitivity factors, denoted by α, indicate the influence of a particular variable on the resulting reliability (Lenner & Sýkora, 2016; van der Spuy & Lenner, 2021). The sensitivity factor for the load is denoted by α_E and the resistance by α_R. It should be noted that the α values are multiplied by 0.4 for accompanying actions (Konig & Hosser, 1981)

The sensitivity factors indicate the direction of the normalised β vector and as a result hold the following relationship, indicated in Eq. (2) (Schneider, 1997; Holicky, 2009).

$$\alpha_E^2 + \alpha_R^2 = 1 \tag{2}$$

The partial factor format is described in *fib* Bulletin 80 (fib, 2016) where the material is denoted as M, the self-weight as G and the imposed load as Q. PFs are denoted by γ.

2.1. Materials

The PF for materials is given by Eq. (3).

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

where γ_{Rd} is a PF accounting for model uncertainty in the resistance model and geometrical deviations. γ_m is the reliability-based PF for the resistance and is defined in Eq. (4) as

$$\gamma_m = \frac{R_k}{R_d} \tag{4}$$

where R_k is the characteristic resistance and R_d is the design value of the resistance.

For materials the characteristic resistance, R_k, is the 5 % fractile (fib, 2016).

From Model Code 2010 (fib 2010) the concrete and reinforcement are modelled with a Gaussian distribution so that Eq. (4) can be expressed as Eq. (5).

$$\gamma_m = \frac{R_k}{R_d} = \frac{1-1.645\times COV_R}{1-\alpha_R\beta_T\times COV_R} \tag{5}$$

where COV_R denotes the coefficient of variation of the material model, taken as 0.15 for concrete and 0.05 for reinforcement. γ_{Rd} is taken as expressed in Eq. (6).

$$\gamma_{Rd} = \frac{1}{1-\alpha_R\times\beta\times COV_R} \tag{6}$$

In-situ measurements, sampling and testing can result in smaller COV values and smaller PFs.

2.2. Loading

The two load cases considered in this contribution are the self-weight and other permanent actions, G, and the traffic load, Q.

A single model uncertainty, denoted by γ_{Ed}, applies to both the self-weight and traffic load effects.

2.2.1. Self-weight

The PF for self-weight is given by Eq. (7).

$$\gamma_G = \gamma_{Ed}\gamma_g \tag{7}$$

where γ_g is the reliability-based PF defined as:

$$\gamma_g = \frac{G_d}{G_k} \tag{8}$$

In accordance with *fib* Bulletin 80 (fib, 2016), the self-weight can be modelled with a Gaussian distribution. For permanent actions, Eq. (8) reduces to Eq. (9).

$$\gamma_g = 1 - \alpha_E \beta_T COV_G \tag{9}$$

where COV_G denotes the coefficient of variation of the self-weight model, taken as 0.05 (fib 2010). For the self-weight, the model uncertainty γ_{Ed} is expressed in Eq. (10).

$$\gamma_{Ed} = 1 - \alpha_E \beta_T COV_G \tag{10}$$

2.2.2. Traffic load

For abnormal (or special) vehicles it is accepted to take the characteristic value of the load effect as the mean value and to derive an associated load effect Q from model uncertainty, static load effects and a potential dynamic amplification. Further, the load effect Q is considered as a log-normally distributed random variable (Lenner & Sýkora, 2016). The PF is therefore expressed as:

$$\gamma_Q = \frac{Q_d}{Q_k} = \frac{\mu_Q \times exp(-\alpha_E \beta_T \times COV_Q)}{\mu_Q} \tag{11}$$

$$= exp\left(-\alpha_E \beta_T \times COV_Q\right)$$

where μ_Q is the mean value and COV_Q denotes the coefficient of variation of the total traffic load effects due to an abnormal (special) vehicle, refer to (Lenner & Sýkora, 2016).

3. SENSITIVITY FACTOR AND LOAD RATIO

The sensitivity factors are functions of the resulting standard deviations of the resistance and the loading. Therefore, they change in any design or assessment situation and may be calculated iteratively (Konig & Hosser, 1981).

Considering bending limit state of a reinforced concrete beam as a dominant failure mode for the simplification in this scenario, the limit state function is given by Eq. (12).

$$g = M_R - M_E = M_R - (M_G + M_Q) \tag{12}$$

where M_R is the bending resistance of the beam and M_E denotes the characteristic variable action split into M_G and M_Q components, permanent and variable loads respectively.

The load ratio, κ, is a function of the characteristic permanent load G_k and characteristic variable load Q_k

$$\kappa = G_k / (G_k + Q_k) \tag{13}$$

Using FORM analysis, and if the values for load and resistance are known, the relationship given by Eq. (13) can be examined. The sensitivity factors for resistance, permanent loading and variable loading are obtained directly. However, to alleviate the need to run a FORM analysis for a simple specification of a partial factor in the assessment situation, the values can either be approximated by the values used in design, or taken approximately from Lenner & Sýkora (2016). For an assessment scenario of a given controlled load, the load ratio can be calculated and an appropriate sensitivity factor for both resistance and loading can be obtained from Figure 1. Note, the condition specified in (Lenner & Sýkora, 2016) must be met, that is either controlled crossing with no dynamics or unrestricted movement of the special load, but both with a maximum COV of static loading equal to 0.05. An approach how to reflect a level of control of crossing through model uncertainty and possibly dynamic amplification factor was proposed by Lenner & Sýkora (2016).

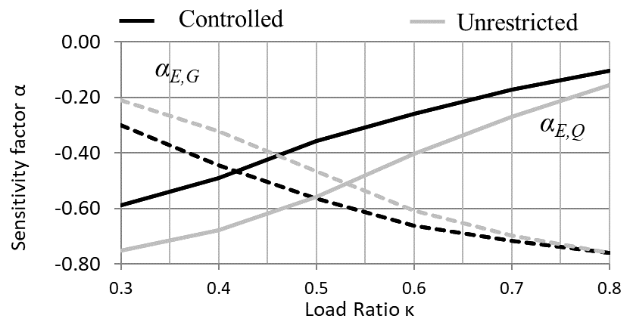

Figure 1: Sensitivity factors for variable loads

As an example, for a load ratio of 0.6 and controlled movement of the abnormal load, that is slow speed along the centreline (or statically most advantageous line), the sensitivity factors for the loading E are obtained as $\alpha_{E,Q} =$ -0.65 for the variable action and $\alpha_{E,Q}$= -0.25 for the permanent load which yields an overall α_E = -0.70 and per Eq. (2) corresponding α_R = 0.71.

Should the conditions for the abnormal loads not be met, FORM analysis must be used to arrive at an appropriate value for the given situation.

4. TARGET RELIABILITY

In the engineering practice, the target reliability levels β_T used in the assessment of existing bridges is commonly associated with the failure of a structural component. In this study it is assumed that the reliability index is related to the failure of a key structural member; for secondary structural members lower target reliability can reflect lower failure consequences.

Target reliabilities for assessment are given in international and national standards; some of them providing lower β_T-levels in comparison to the design levels (Orcesi, A. et al. 2023). The main factors affecting the β_T-levels include:

1. Failure consequences, C_f, in some cases with the explicit consideration of the type of failure (ductile or brittle, system behavior)
2. Cost of safety measures, C_{safe}
3. Reference period, t_{ref}

The consequences related to failure due to the crossing of an abnormal load are expected to be similar to those associated with persistent design situations. In principle, failure consequences should cover potential societal consequences such as injuries and fatalities, cost of repair, upgrade or replacement, economic losses and potential societal consequences caused by bridge malfunction (possibly including the losses due to damage on detours), and other consequences such as unfavourable environmental or psychological effects.

In engineering practice, estimates of C_f are commonly unavailable. To provide a first insight, it might be assumed that by adopting a certain design target level, some failure consequences are implicitly considered to be associated with the relevant failure mode. For instance, a 50-year target reliability index $\beta_{T,50}$ = 3.8 might be considered to correspond to annual $\beta_{T,1}$ = 4.2 for reinforced concrete structures according to the draft *fib* MC 2020 (*fib*, 2023). The latter value is estimated to be associated with the ratio of failure consequences to structural cost C_f / C_{str} ≈ 1-4 (Steenbergen, Rózsás & Vrouwenvelder, 2018). The structural cost (cost of a new bridge) C_{str} can commonly be well estimated for a given type of the structural system, span, and width of the bridge.

In contrast, the cost of safety measures and the reference period are likely significantly different for persistent design situations and the crossing of an abnormal load. The cost of safety measures is the additional cost to increase reliability for new bridges. In the case of an abnormal load crossing, reliability of the bridge can be ensured by rerouting the road transport, using other means of transport, ad hoc monitoring of the bridge or minimal strengthening of the bridge. Inevitably, cost C_{safe} is likely significantly different when compared to the persistent load situation.

In the standards, the target reliability is commonly related to annual or 50-year reference periods. While the former might be considered as the basis for permits covering multiple crossings over some longer periods, generally abnormal load transports are associated with much shorter reference periods.

Considering the difference in the cost of safety and the reference period for abnormal loads the standardised values seem to poorly account for the particularities of such crossings. To establish appropriate target levels for such situations, the general guidance provided by ISO 2394:2015 is followed. The standard prescribes two fundamental concepts as the basis for specification of the target reliabilities namely economic optimisation and minimum levels required for acceptable human safety.

In the context of risk-informed decision making, economic optimisation can substantiate authorizing a once-off crossing. In Eq. (13) (Lenner & Sýkora, 2016) propose a simplified approach to estimate minimum target level for a crossing.

$$\beta_{T,\mathrm{eco}} \approx -\Phi^{-1}(C_{\mathrm{safe}} / C_{\mathrm{f}}) \qquad (13)$$

It is argued that this approach might also be applied for multiple crossings.

Further to economic optimisation, ISO 2394 indicates that human safety levels should be adhered to. Assuming normal traffic being restricted during a crossing, only the safety of persons involved in the transport (the driver and possibly crew) is endangered. These persons are assumed to be repeatedly exposed to the risk associated with these specific crossings. Higher risk exposure is normally compensated as it is a common practice in various industries such as mining, power production or the shipping industry (Terwel, Boot & Nelisse, 2014). Significant risk compensations are then provided to members of rescue or army corps. The acceptance of related risks and decisions on appropriate compensations is the task of a company responsible for the transport and detailed discussion is beyond the scope of this study.

To provide some indication, it might be considered that a broadly accepted value of the individual lethal accident rate is 10^{-4} per year for workers in most types of industries, with a doubled value for users of motor vehicles. The rate of 10^{-3} per year might be accepted for miners and in military operations, but it is deemed unacceptably high. For further details see (Sykora et al., 2014)

Following *fib* Bulletin 80 and Faber, Sørensen & Vrouwenvelder (2015), maximum acceptable failure probability can be obtained as the ratio of the acceptable lethal rate (2×10^{-4} per year) and of conditional probability of fatality given structural failure (considering here a representative value of 0.05). This leads to a maximum failure probability $2\times10^{-4} / 0.05 = 0.004$ per year, corresponding to $\beta_{\mathrm{human\_safety,1y}} = 2.65$.

Note that Lenner & Sýkora (2016) discussed the human safety requirements when normal traffic is allowed to cross a bridge along with special transports. Specification of human safety criteria can be improved by using the Life Quality Index approach provided in ISO 2394 that considers both costs of safety measures and failure consequences; an example of the application in the assessment of existing structures is provided in Sykora et al. (2016).

5. CASE STUDY

A case study where a nuclear turbine must be transported across a bridge is considered. The vehicle mass is 380 t including the payload and the trailer. The load, shown in Figure 2, is uniformly spread over 18 axles spaced at 1.5 m.

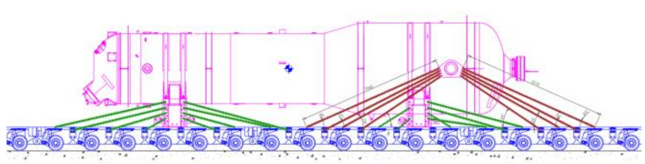

Figure 2: Abnormal vehicle

The bridge investigated by Skokandić & Mandić Ivanković (2022) is considered in the case study. The bridge is a three-span structure with spans of 9 m, 15 m and 9 m respectively. The cross section is a solid slab with a trafficable width of 7 m and a total depth of 0.6 m. The concrete class is C30/37 and the characteristic reinforcement strength is 500 MPa. The deck is reinforced with 2450 mm^2/m in the bottom layer with a cover to reinforcement of 50 mm.

5.1. Assessment according to Eurocode design partial factors

The sagging moment in the centre span is considered as a critical scenario, with the permanent action due to self-weight G_k = 171 kNm/m and the load effect due to the specified abnormal vehicle Q_k = 265 kNm/m. If the design load is considered using PFs of 1.35 for both G and Q according to EN 1990, E_d = 589 kNm/m is obtained.

The design resistance, using PFs of 1.15 for reinforcement and 1.5 for concrete is obtained as R_d = 568 kNm/m. Therefore, by using the design values for assessment $E_d > R_d$ and the transport cannot be authorised to cross the bridge. The utilisation ratio is E_d / R_d = 1.04.

5.2. Probabilistic based assessment

5.2.1. Target reliability

According to Eq. (13) for specification of an economically optimum target reliability level, it is necessary to determine the cost of failure and the cost of measures to ensure safe passage. For the transportation of a turbine, the following approximate cost are obtained from authorities:

- The construction cost of the bridge under investigation, in the present value, is estimated as €500 000.
- If a cost ratio of 4 is assumed, then the cost of failure is €2 million.
- For delays in the transport due to failure, an additional cost of €16 million is incurred to the operator and thus to the society due to the shortage of power production, giving a total cost of failure, C_f, totals €18 million.
- It is estimated that it would cost €20 000 for temporary supports to allow the load to cross safely.

According to Eq. (13) $\beta_{T,\mathrm{eco}}$ is calculated as 3.05. As this value exceeds that of the 2.65 for human safety it governs as the target reliability.

5.2.2. Sensitivity factors

The load ratio, κ, for this particular assessment situation is calculated as 0.39. From Figure 1, the following sensitivity factors are obtained:

$\alpha_{E,G}$ = -0.32
$\alpha_{E,Q}$ = -0.67

which leads to an overall α_E of -0.74 and according to Eq. (2) α_R is determined as 0.67.

5.2.3. Partial factors

For the self-weight, from Eqs. (7), (9), and (10), the PF, γ_G , is determined as 1.07. For the abnormal load, the PF γ_Q, is calculated as 1.15 from Eq. (11).

The PFs for resistance are calculated according to Eqs. (5) and (6). For the concrete resistance, the partial factor γ_C, is calculated as 1.16. This accounts for accurate measurements of the concrete which allows that the COV for the model uncertainty can be reduced to 0.08. The sensitivity factor for model uncertainty is multiplied by 0.4 as a non-dominating resistance variable (fib, 2016), $\alpha_R = 0.4 \times 0.67 = 0.27$. For steel the partial factor is calculated as 1.07.

5.2.4. Comparison between load and resistance

By using the PFs from section 5.2.3, the assessment load effect is calculated as E_{assess} = 488 kNm/m and the assessment resistance as R_{assess} = 608 kNm/m. From $E_{\mathrm{assess}} < R_{\mathrm{assess}}$ it can be concluded that the abnormal load can safely cross the bridge if probability based assessment is applied, rather than using design values. In comparison to the latter, the utilisation ratio decreases by about 23 % to $E_{\mathrm{assess}} / R_{\mathrm{assess}}$ = 0.80.

6. CONCLUSIONS AND FUTURE RESEARCH

The transport of abnormal vehicles is becoming more frequent due to the construction of new facilities like wind farms and restoration of nuclear power plants. This paper provides a framework for the reliability-based assessment of bridges exposed to abnormal traffic loads. Recommendations are made for the FORM sensitivity factors based on the load ratio and target reliability values based on cost optimisation and human safety requirements. In a case study, the target reliability is governed by economic optimisation, $\beta_{\mathrm{eco}} \approx 3.05$. The resulting utilisation ratio for the assessment is significantly lower (by about 23 %) compared to that based on design values, leading to a more favourable analysis for the bridge.

This study is applicable to single crossings and further work is required to determine the effect of multiple crossings.

7. ACKNOWLEDGEMENTS

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