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A code calibration for heavy special transports on railway bridges

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ABSTRACT: The current paper summarizes a code calibration project for heavy special transports on railway bridges. The basis is the Swedish code for assessing existing bridges, which has a safety format based on partial factors. Reliability-based models have been established for different bridge types and failure modes, covering the most common railway bridges and relations between permanent and variable loads. A sample of special transport vehicles from previous admission applications was considered and modelled with stochastic properties reflecting known axle loads and gross weights. Statistical data for stochastic variables were collected from the literature, but additional input had to be evaluated from recent measurements. The results include safety indices for the present partial factors and a suggestion for a reduced factor for vehicles with a known weight. With a target value of $\beta = 3.7$, applicable to existing bridges, the partial factor for vehicles can be reduced from 1.5 to 1.3.

1. Introduction

When special transports are requested on railways, the load-bearing resistance of the bridges along the line in question has to be assessed. An example of a vehicle is shown in Fig. 1, a 20-axle wagon carrying an electrical transformer. The assessment is often a simplistic load effect verification comparing the influence of the special transport with the specified line category described in, e.g., CEN (2021). If the special transport has a geometry significantly different from the line category vehicle or has a much greater load, the comparison may lead to a decision of insufficient resistance. However, the uncertainties related to the characteristics of a known vehicle are typically smaller than those afflicted with a general assessment for mixed traffic. There is no support to handle this reduced uncertainty in today's codes based on the partial safety method.

A reliability-based code calibration has been performed (Leander, 2022) to investigate the possibility of suggesting a new partial factor specifically for heavy vehicles with known characteristics and is summarised in this paper.

Figure 1: A 20-axle freight wagon for transport of an electrical transformer.

The calibration procedure follows established principles outlined by, e.g., Ditlevsen and Madsen (2007) and Melchers and Beck (2018). In short, two parallel safety formats are analysed, one semi-probabilistic based on partial factors and another, a superior probabilistic format with a limit state equation and statistical distributions for the variables included. Structural dimensions determined with the former giving sufficient safety are then used in the superior format to estimate an associated probability of failure. The calibration is the optimisation procedure to determine partial factors reaching a target value for the probability of failure.

The project's research worked on finding statistics for the stochastic variables relevant to railway bridges. Guidance has been found in, e.g., publications by Vrouwenvelder and Siemes (1987), JCSS (2001), and Sørensen (2002). Variables related to some loads had to be derived based on recent data.

2. SWEDISH CODE FOR ASSESSMENT OF EXISTING BRIDGES

Since 1996 there has been a code in Sweden dedicated to the assessment of existing railway bridges. The prevailing version (Trafikverket, 2023) is in several parts referring to related sections in the Eurocode. However, amendments are incorporated to consider existing structures in contrast to new designs. It relates, e.g., to the evaluation of material properties and loads.

A general load combination for the ultimate limit state is defined as

$$S_{\mathrm{Ed}} = \sum_{i=1}^{n} \psi\gamma_{\mathrm{G},i}\, G_i + \psi\gamma_{\mathrm{Q},1}\, Q_1 + \sum_{j=2}^{4} \psi\gamma_{\mathrm{Q},j}\, Q_j \quad (1)$$

where G is the load effect of the permanent loads $i = 1 \ldots n$, Q_1 is the load effect of the leading variable action to be multiplied with the largest associated $\psi\gamma$ value, and Q_j is load effect of the accompanying variable actions $j = 2 \ldots 4$ to be multiplied with lower associated $\psi\gamma$ values.

The presented calibration was limited to consider the loads self-weight (sw), ballast (bl), and train load (trn). With partial factors from Trafikverket (2023), Eq. (1) can be simplified to

$$S_{\mathrm{Ed}} = 1.2\, G_{\mathrm{sw}} + 1.3\, G_{\mathrm{bt}} + 1.5\, Q_{\mathrm{trn}} \quad (2)$$

The purpose of the calibration was to update the partial factor equal to 1.5 in Eq. (2) for train loads with known characteristics.

3. LIMIT STATE EQUATIONS

A general limit state equation for the probabilistic format is defined as

$$g = R - S \quad (3)$$

where R is the resistance and S is the load effect. A negative limit state defines a region of failure and the probability of failure is determined as

$$P_{\mathrm{f}} = P\left[g \leq 0\right] \quad (4)$$

The associated reliability index $\beta = -\Phi^{-1}\left(P_{\mathrm{f}}\right)$ is used in this paper as a measure of safety, where $\Phi^{-1}\left(\cdot\right)$ is the inverse of the standardized normal distribution.

The stochastic variables considered were described by distribution functions and statistical moments. The mean values are throughout the paper related to the nominal values, the characteristic values used for loads and material properties, or dimensions specified on design drawings. This relation can be described as

$$\mu_Y = k\, y_{\mathrm{nom}} \quad (5)$$

where μ_Y is the mean value of the stochastic variable Y, k is a bias factor, and y_{nom} is the nominal value used for the variable Y in conventional (code based) assessments.

3.1. Loads

Splitting the load effect S in Eq. (3) into the separate loads give

$$S = C_{\mathrm{sw}}\, G_{\mathrm{sw}} + C_{\mathrm{bl}}\, G_{\mathrm{bl}} + C_{\mathrm{trn}}\, D\, Q_{\mathrm{trn}} \quad (6)$$

with G_{sw} and G_{bl} representing the self-weight of the structure and ballast, respectively, Q_{trn} represents the train load, and D is a dynamic factor. The C variables in Eq. (6) are model uncertainties associated with the calculation of load effect in structural analyses. These were given lognormal distributions in the calibration with a mean value of unity and a coefficient of variance (CoV) equal to 0.05. This agrees with model uncertainties suggested by Vrouwenvelder and Siemes (1987) and Vejdirektoratet (2004) for permanent loads. For train loads in general, a higher CoV could be motivated. However, when the specific vehicle is known, the same CoV as for permanent loads was judged appropriate.

3.1.1. Permanent loads

The self-weights of the structure and ballast depend on the material density and volume. For concrete, three variables were considered, as presented in the first rows of Table 1. The references for the statistical properties are shown in the same table. For steel members, the density and the cross-section area were modelled with uncertainties, also stated in Table 1.

Table 1: Stochastic variables for the load effect. The sources refer to [1] JCSS (2001), [2] Nowak et al. (2011), [3] Alpsten (1972), [4] Vrouwenvelder and Siemes (1987).

Variable	Dist.	Mean	CoV	Source
Concrete				
- Density	N	γ_c	0.04	[1]
- Width	N	$1.01\, b_{nom}$	0.04	[2]
- Height	N	$0.99\, h_{nom}$	0.04	[2]
Steel				
- Density	N	γ_s	0.01	[1]
- Area	N	$0.99 A_{nom}$	0.03	[3]
Ballast				
- Density	N	$0.94\, \gamma_{bl}$	0.04	
- Area	N	A_{nom}	0.05	
Train load				
F_{trn}	Wbl	$0.95\, F_{nom}$	0.02	
D	LN	$0.85\, D_{nom}$	0.10	
Model uncertianties				
C_{sw}	LN	1	0.05	[4]
C_{bl}	LN	1	0.05	
C_{trn}	LN	1	0.05	

The ballast is the gravel between the track and the bridge. The dimensions of this layer can differ substantially between bridges. Trafikverket (2023) states that a density of 18 kN/m^3 should be used and that the thickness should be determined upon assessment. Therefore, the calibration assumed that the ballast dimensions are fairly well considered in assessments. The parameters for ballast in Table 1 stem from a mean value of 17 kN/m^3 and the same CoV as for concrete density, and general uncertainty of the ballast cross-section area of 5 %.

3.1.2. Train loads

The trains were considered as moving axle loads with specified intermediate distances. The operator must specify these properties for an admittance application regarding a special transport. This means the uncertainties can be limited to a particular vehicle and do not have to cover various types. Moreover, the vehicle's presence is given and, typically, also the number of passages over the considered bridge. Nevertheless, the actual values of the axle loads are afflicted with uncertainty. The operator likely states maximum or high fractile values in the admittance process to not compromise safety.

Data from weighted iron ore wagons were analysed to determine relevant statistical properties of axle loads for certain vehicles. The data was collected as a part of an investigation to increase the axle loads on the iron ore line in northern Sweden to 32,5 metric tonnes. The bogie loads of 25 660 wagons of type Fanooo, see Fig. 2, were recorded and made available for statistical curve fitting.

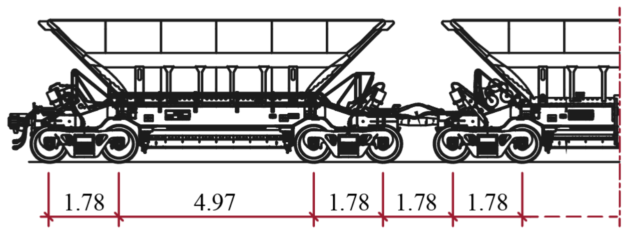


Figure 2: Two connected iron ore wagons of type Fanoo. The axle distances are given in meters.

The two axle loads in each bogie were assumed the same, rendering a sample size of 51 320 values for the curve fitting. The data is shown as a histogram in Fig. 3 together with a Weibull distribution showing the best fit of several tested distributions. The target weight of 32.5 tonnes is also indicated in the figure. The mean value was found to be 0.95 times the target value F_{nom}, and the CoV was determined to be 0.02.

Regarding the dynamic factor D in Eq. (6), the statistical background of the model in codes such as the Eurocodes (CEN, 2010) and Trafikverket (2023) is unclear. A literature review by Leander (2022) suggests a lognormal distribution, a CoV of 0.10 and a mean value associated with a nominal value equal to the 95 per cent fractile ($\mu_D =$

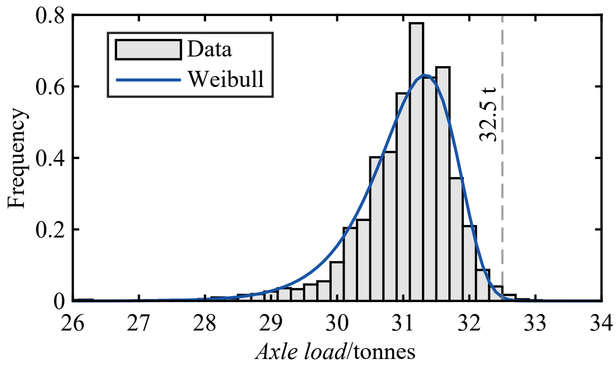


Figure 3: Axle loads for fully loaded iron ore wagons with a target weight of 32.5 tonnes.

$0.85\,D_{\mathrm{nom}}$). The nominal value was calculated as

$$D_{\mathrm{nom}} = 1 + \varphi' + 0.5\,\varphi'' \tag{7}$$

where φ' considers the bridge's dynamic behaviour with a perfect track, and φ'' is a factor considering rail irregularities. Furter details can be found in Annex C of the Eurocode EN 1991-2 (CEN, 2010).

3.2. Concrete bridges

The resistance R in Eq. (3) depends on bridge type and failure mode. For bridges with load-bearing slabs or beams in concrete, failures due to bending moment and shear force were verified. Considering the former, the resistance was formulated as (Nowak et al., 2011)

$$M_{\mathrm{R,c}} = C_{\mathrm{M,c}} A_{\mathrm{s}}\, f_{\mathrm{y}} \left(d - \kappa \frac{A_{\mathrm{s}}\, f_{\mathrm{y}}}{b\, f_{\mathrm{c}}} \right) \tag{8}$$

where $C_{\mathrm{M,c}}$ is the model uncertainty related to the resistance, and κ is a factor considering the stress-strain relation for concrete. All other variables in Eq. (8) can be found in the governing codes.

The effective depth of the cross-section d was used as the design variable for the calibration. The reinforcement content A_{s} was assigned to 0.5 % of the effective area of concrete bd. The statistical properties of the stochastic variables are listed in Table 2.

Regarding failure due to shear force, models for verification can differ between governing codes. For the assessment of existing bridges, Trafikverket (2023) refers to the Eurocode and the dated

Table 2: Stochastic variables for concrete resistance. The sources refer to [1] Nowak et al. (2011), [2] Ditlevsen and Madsen (2007), [3] JCSS (2001), [4] Vrouwenvelder and Siemes (1987), [5] Braml et al. (2009).

Variable	Dist.	Mean	CoV	Source
General				
b	N	$1.01\,b_{\mathrm{nom}}$	0.04	[1]
d	N	$0.99\,d_{\mathrm{nom}}$	0.04	[1]
$f_{\mathrm{y}}, f_{\mathrm{yw}}$	LN	$1.13\,f_{\mathrm{y,nom}}$	0.03	[1]
Bending moment				
A_{s}	Det	-	-	
f_{c}	LN	$1.20\,f_{\mathrm{c,nom}}$	0.14	[1]
κ	LN	0.55	0.05	[2]
$C_{\mathrm{M,c}}$	LN	1.20	0.15	[3]
Shear force				
A_{sw}	Det	-	-	
ρ	Det	-	-	
f_{ct}	LN	$1{,}43\,f_{\mathrm{ct,nom}}$	0,20	[4]
$\cot\theta$	Det	1,2	-	[5]
$C_{\mathrm{V,c}}$	LN	1,4	0,25	[3]

Swedish handbook for concrete structures (Boverket, 2004). For structures with stirrups the resistance can be calculated as (CEN, 2008)

$$V_{\mathrm{R,c(a)}} = C_{\mathrm{V,c}} \frac{A_{\mathrm{sw}}}{s} 0.9\, d\, f_{\mathrm{yw}} \cot\theta \tag{9}$$

where $C_{\mathrm{V,c}}$ is the model uncertainty related to the resistance. The equation describes the reinforcement resistance. The associated verification of the compressed concrete should be considered in a design situation, but it is rarely decisive why it was disregarded in the calibration.

Initial investigations showed insufficient reliability when Eq. (9) was used. Therefore, the verification model from Boverket (2004) was also included, formulated as

$$V_{\mathrm{R,c(b)}} = C_{\mathrm{V,c}}\, 0.3\, \xi\, (1 + 50\rho)\, f_{\mathrm{ct}}\, b\, d \tag{10}$$

This equation describes the resistance of a concrete member without stirrups considering the bending moment reinforcement content ρ and the tensile strength of concrete f_{ct}. The statistical properties of the stochastic variables are listed in Table 2.

3.3. Steel bridges

Bridges with steel beams were verified as if they were carrying the whole load, regardless of whether they had a concrete slab. The difference is the load effect, where concrete slab bridges have to sustain the self-weight of the slab and ballast, in contrast, to open deck bridges, where the sleepers are resting directly on the steel beams.

Considering an elastic verification and cross-section class 3, the moment resistance can be calculated as

$$M_{R,s} = C_{M,s} W_{el} f_y \tag{11}$$

where $C_{M,s}$ is the model uncertainty related to the resistance. The statistical properties of the stochastic variables were derived based on data from Alpsten (1972). Results for several material qualities are presented in Leander (2022), but herein, values only for steel with $f_{y,nom} = 260$ MPa are considered and listed in Table 3.

Table 3: Stochastic variables for steel resistance. The sources refer to [1] Alpsten (1972), [2] JCSS (2001).

Variabel	Dist.	Mean	CoV	Source
W_{el}	N	$0,99\,W_{el,nom}$	0,03	[1]
A_w	N	$0,99\,A_{w,nom}$	0,03	[1]
f_y	LN	$1,17\,f_{y,nom}$	0,09	[1]
$C_{M,s}$	LN	1	0,05	[2]
$C_{V,s}$	LN	1	0,05	[2]

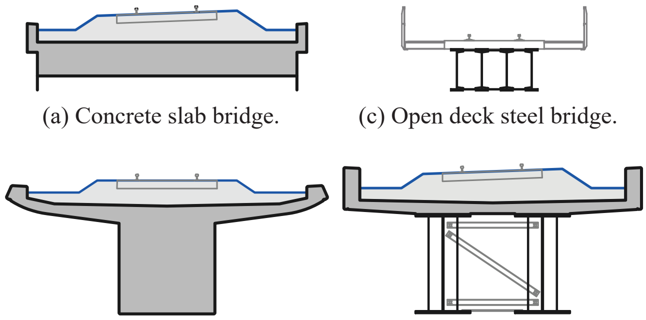
The shear force was verified using the elastic resistance and neglecting web buckling as

$$V_{R,s} = C_{V,s} \frac{A_w f_y}{\sqrt{3}} \tag{12}$$

where $C_{V,s}$ is the model uncertainty related to the resistance. The statistical properties of the stochastic variables are listed in Table 3.

4. CALIBRATION CASES

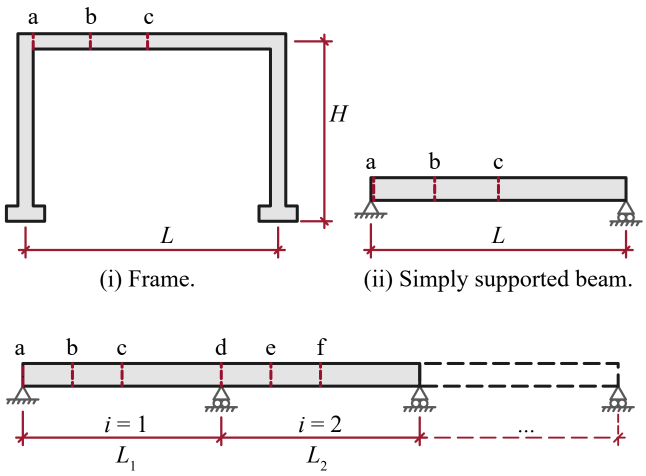
This process aimed to calibrate the assessment of individual bridges for specific vehicles. It required covering a range of bridge types and their failure modes, different ratios between permanent and variable loads, and various load configurations for a range of statical systems. The bridge types covered were concrete frame and beam bridges, steel bridges with concrete slabs, and open deck steel bridges. Examples of cross-sections are shown in Fig. 4, depicting actual bridges in the Swedish bridge stock.



(a) Concrete slab bridge. (c) Open deck steel bridge.
(b) Concrete beam bridge. (d) Steel bridge with concrete slab.

Figure 4: Examples of bridge types considered.

The statical systems considered are shown in Fig. 5. Concrete bridges were modelled as frames, simply supported and continuous beams up to 11 spans. Steel bridges were modelled as simply supported and continuous beams up to 17 spans. In total, 54 different bridge cases were considered. All of them are described by Leander (2022).



(i) Frame. (ii) Simply supported beam.
(iii) Continuous beam.

Figure 5: Examples of statical systems considered. The letters a-f indicate the locations verified.

An assembly of 22 vehicles from previous admittance applications was considered to cover a representative range of possible characteristics for future special transports. It ranged from heavy electrical transformer transports, as in Fig. 1, to more frequently occurring timber transports, as in Fig. 6.

All 22 vehicles are described by Leander (2022). The nominal values of the axle loads were taken from the data sheets of the vehicles. The uncertainty of these values was then described by the F_{trn} variable in Table 1.

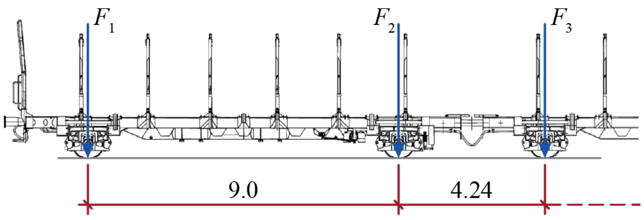


Figure 6: Two connected timber transport wagons of type Laaps, with nominal axle loads of 25 tonnes and axle distances given in meters.

5. RESULTS

The calibration process comprised the following main steps:

1. structural analyses for each bridge case and vehicle;
2. determination of design variables to fulfill the partial factor safety format;
3. stochastic analysis, using the design variables, to determine the reliability index;
4. comparison with target reliability and updating of the partial factors. If needed, starting over from Step 2.

Due to space limitation all steps cannot be reviewed in detail. Selected results from steps 2 and 3 are presented.

5.1. Stochastic analysis

The stochastic analyses (Step 2) were initially performed using the first-order reliability method (FORM) and Monte Carlo simulation. Comparisons of the outcome showed good agreement, suggesting that FORM was sufficient for the calibration process. Moreover, it has the advantage of giving the sensitivity factors (α factors) as an intermediate step in the calculation.

An example of α factors is shown in Fig. 7 for one of the concrete frame bridge cases and the load from an electrical transformer. The sensitivity factors show the influence of the uncertainties of each stochastic variable. The variables with red bars in the figure have a negative impact on reliability, while the variables with blue bars have a positive. Considering the bending moment, the model uncertainty of the resistance $C_{\text{M,c}}$ has the most significant influence, before the uncertainty of the effective height d and the yield stress of the reinforcement f_{y}. On the negative side, the dynamic factor D and the model uncertainties of the train load C_{trn} and the self-weight C_{sw} have noticeable influences on the reliability. The uncertainty of the trains' axle loads F_{trn} has a modest impact.

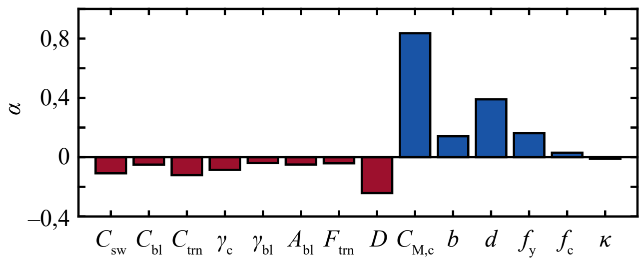


Figure 7: Sensitivity factors for bending moment evaluated for a concrete frame bridge, a span length of 15.7 m, loaded by an electrical transformer Q48+Q49.

An example of one of the steel bridge cases and a train with timber wagons are shown in Fig. 8. Considering the bending moment, the uncertainty of the yield stress of the material f_{y} has the most significant influence before the model uncertainty of the resistance $C_{\text{M,s}}$. On the negative side, the model uncertainty of the self-weight C_{sw}, concrete area A_{c}, and dynamic factor D significantly influences the reliability. As for the concrete case, the uncertainty of the trains' axle loads F_{trn} has a modest influence.

Reliability indices associated with the existing partial factor of 1.5 on train loads are shown in Fig. 9 for all concrete bridges considering the bending moment. All 22 vehicles were included but are represented in the figure with the mean value. The β values vary between 4.2 and 4.8, with a mean value of 4.5. The reliability index decreases with increasing span length, and a review of the results shows that increasing self-weight also negatively influences reliability. The trends are consistent between the statical systems.

The reliability indices in Fig. 10 represent the concrete bridges considering shear force with the

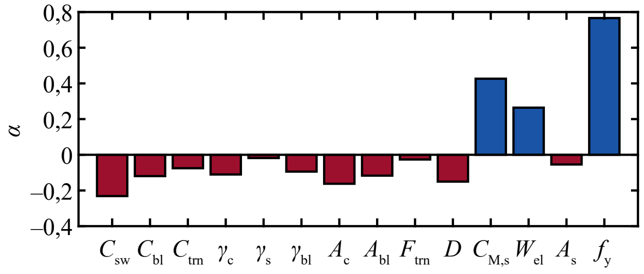


Figure 8: Sensitivity factors for bending moment evaluated for a simply supported steel bridge with a concrete deck, a span length of 22 m, loaded by a timber transport of type Laaps.

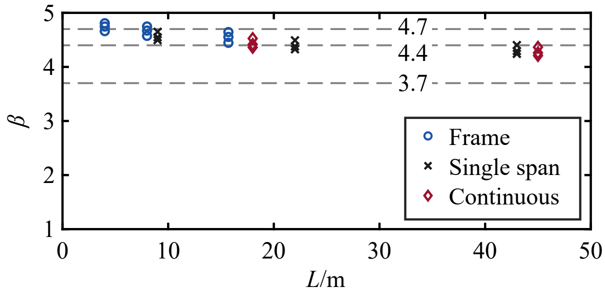


Figure 9: Reliability index over the span length for 24 concrete bridge cases considering bending moment.

Eurocode model, Eq. (9). The β values vary between 3.4 and 3.9, with a mean value of 3.7. The reliability level is distinctly lower than for the bending moment.

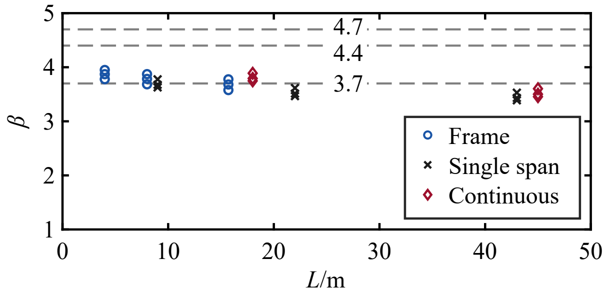


Figure 10: Reliability index over the span length for 24 concrete bridge cases considering the Eurocode shear force verification model, Eq. (9).

Reliability indices for the steel bridges with concrete slabs are shown in Fig. 11, considering bending moments. The β values vary between 4.0 and 4.6, with a mean value of 4.2.

Sensitivity factors and reliability indices for all

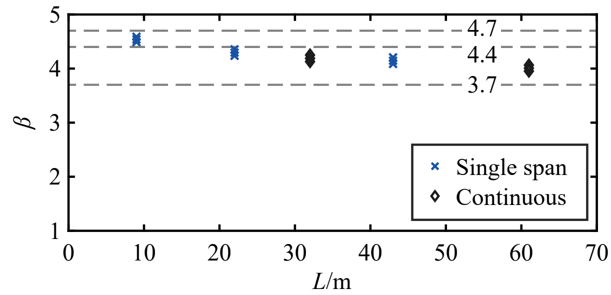


Figure 11: Reliability index over the span length for 15 stel bridges with concrete slabs considering the bending moment verification model, Eq. (11).

bridge cases and verification models are presented in detail by Leander (2022).

5.2. *Calibration*

For a decision on partial factors, target reliability needs to be assigned. The standard ISO 2394 (ISO, 2015) was used as a reference in this respect. For consequence class 4, tentative target values are listed as $\beta_T = 3.7$, 4.4 and 4.7, associated with high, medium and small costs of safety measures, respectively. The partial factor for train load has been varied to estimate the value required to reach each safety level.

The results for concrete bridges considering bending moment are shown in Fig. 12 as the reliability index β over the partial factor for train load $\psi\gamma_{trn}$. The curve represents the average of all concrete bridges and all 22 vehicles. A value of $\psi\gamma_{trn} = 1$ gives a reliability index of $\beta = 3.8$. With a target of $\beta_T = 4.4$ and 4.7, the corresponding partial factors should be 1.43 and 1.66, respectively. A summary of the factor values over all considered cases is shown in Table 4.

Table 4: Partial factor $\psi\gamma$ for train load associated with different target reliability indices.

	Target β_T		
Verification	3.7	4.4	4.7
Concrete, bending (8)	1	1.43	1.66
Concrete, shear (9)	1.52	2.20	2.53
Concrete, shear (10)	1	1.47	1.84
Steel, bending (11)	1.25	1.58	1.73
Steel, shear (12)	1.24	1.48	1.60

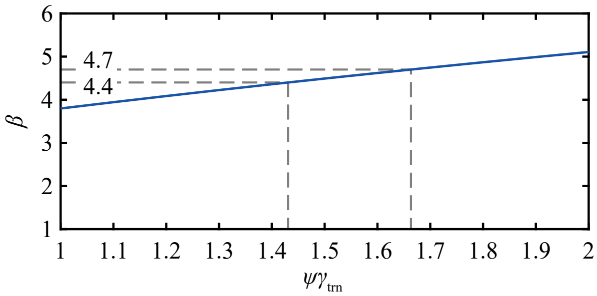


Figure 12: Reliability index β over the partial factor for train load $\psi\gamma_{\mathrm{trn}}$, valid for concrete bridges under bending moment.

6. CONCLUSIONS

The reliability-based calibration of the partial factor for heavy special transports rendered the following conclusions:

- The reliability index β varies between bridge types and failure modes, with values above four in general. The variation between different special transports is small.
- The lowest reliability was reached for shear force of concrete bridges with stirrups, giving values low as $\beta = 3.4$.
- If a target value of $\beta_{\mathrm{T}} = 3.7$ is accepted, the partial factor for the train load can be reduced from 1.5 to 1.3. It is on the condition that the shear resistance of concrete bridges with stirrups is not decisive.

ACKNOWLEDGEMENTS

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