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Ultimate and fatigue limit states of existing steel railway bridges

LRFD with historical steel products and connection types

The assessment of the load-bearing capacity and fatigue strength of existing railway bridges has been playing an increasingly important role in the infrastructure management of railway operators for several years now. Currently, many bridge structures have been in operation longer than it was foreseen during their planning and construction. In addition, the axle loads on many lines, as well as the demands on the reliability of the verification results, have steadily increased. As the material properties and construction techniques in existing structures differ to some extent from nowadays structures, e.g., riveting instead of welding, it is important to provide engineers and operators with recommendations for the assessment of existing steel bridges. This article summarises the studies conducted as part of a research project initiated by Deutsche Bahn Netz AG for possible updates to DB RiL 805, which is used for the verification of railway bridges in the Deutsche Bahn (DB) network. The studies concerned the transition of verification concepts against static and fatigue loads used in the past to limit state verifications with partial safety factors in accordance with the Eurocodes. While initially related to an upgrade of a specific operator's design recommendation, the findings in this article are of more general nature and could form the basis for similar developments of recommendations for the assessment of existing, riveted structures independently throughout Europe.

Keywords railway bridges; fatigue; riveted connections; damage equivalent factors; partial safety factors

1 Introduction

1.1 Motivation

In recent years, a large number of existing railway bridges have reached their originally intended service life. At the same time, in many cases the traffic volumes have grown more strongly than predicted in the original design of the bridge structures. For reasons of economy, but also of preservation of historic structures, replacement of these bridge structures is often not desired. This raises the question of further usability of many structures, in particular regarding the residual service life against fatigue and corrosion as well as the residual load-bearing capacity against increased axle loads or faster bridge-crossing. The use of materials that can only be welded to a limited extent and of joining techniques that are no longer common today, such as riveting, requires an in-depth examination of the special properties of these structures and materials to obtain accurate predictions of structural safety in the ultimate limit state (ULS) and fatigue limit state. Periodic assessment of the load capacity and operational safety of existing infrastructure buildings has become an efficient tool for infrastructure management and maintenance resource planning. The structural assessment is usually conducted under consideration of bridge-specific data: on the one hand, the actual condition of the structure determined in the course of inspections, and on the other hand, the actual traffic loads in a cumulative sense of annual volumes and those currently expected as maximum values. Various recommendations for assessment of existing bridges have been published since the 1990s by the various operators of bridge structures for different reasons. The key problem is certainly the specification of the criteria and the mechanical properties to be applied for the assessment of historical steel products and connection types, which are not common anymore in nowadays design and construction. The most significant motivations from an engineering point of view for release of adequate recommendations are thus as follows:

- Meet the growing importance of preservation of existing structures.
- Standardisation of the approaches applied for structural assessment.
- Achieve the most precise possible prediction of the remaining service life of bridge structures.

For the assessment of railway bridges in Germany, the so-called recommendation DB RiL 805 published by Deutsche Bahn (DB) is most commonly applied, currently in the version RiL 805:2012 [1]. Among other things, this recommendation contains specifications on the verification of the structural safety and of the fatigue strength for structures made of historical steel products and riveted connections. Currently, RiL 805 is being updated, with a majority of the work completed by the end of 2021 (see also [2]). During a study funded by DB, the fundamentals for the revision and updating of the structural assessment of steel railway bridges, especially riveted ones, were developed in the years 2017 to 2020. The authors of this article dealt specifically with the aspects summarised in Section 1.2. This article summarises the applied methodologies, some excerpts of the collected data on historical steel products used for the studies and some representative results. Other parts of the recommended updates for

RiL 805:2012 have already been published and documented in [3, 4]. In dealing with these aspects, the authors built on various articles initiated and prepared by Prof. Dr. Richard Greiner during his tenure as Chair of Steel and Shell Structures at Graz University of Technology in cooperation with the second author of this article [5–8].

1.2 Intended upgrade of DB recommendation RiL 805

Modules 103 and 201 – as well as associated annexes – of the recommendation RiL 805 of DB Netz AG contain the essential regulations for verification of the ultimate and fatigue limit states of steel bridges. The research project concerned with the possible updating and reviewing the possibility of harmonization with Eurocode 3 procedures of these modules is summarised below:

1. Review of the applicable partial safety factors to perform verifications in the ULS, in general, using the design rules in Eurocode 3, with selective adjustments to account for the historic, riveted construction method.
2. Adaption of the normalised fatigue and residual service-life check with damage equivalent factors to the Eurocode concept and use of the damage equivalent factors specified therein for prediction of the damage accumulation in the future.
3. Update and introduction of a fatigue class catalogue for riveted structural details for a more differentiated assessment of the fatigue strength, as well as calibration of the partial safety factors for the fatigue strength in the fatigue verification of riveted structural members.
4. Extension of the time period for fatigue damage accumulation in the past from 1996 up to the year 2020 and calculation of the applicable damage equivalent factors for the extended time range in the past.

2 Partial safety factors in the ultimate limit state

2.1 Outline of the present structural safety concept in DB RiL 805

The assessment of existing railway bridges according to RiL 805 is structured in four stages. Stages 1 and 2 allow for an assessment by estimating or roughly determining the load capacity in the ULS. In stages 3 and 4, a more precise determination of the load capacity in the ULS applies, whereby stage 4 involves a measurement-based structural analysis. The design checks for the ULS in stages 3 and 4 are regulated in RiL 805 module 201. Those design checks already apply the partial safety factor concept. The changeover from the verification concept with permissible stresses to the limit state concept had already been conducted in anticipation of the introduction of the Eurocodes to ensure the compatibility of the codes [1]. The currently applicable version RiL 805:2012 [1] covers the limit states of the plastic cross-sectional load capacity, the resistance of bolts and rivets against shear failure, the resistance of the hole-bearing resistance in the base material, the resistance of welds as well as the buckling resistance, with the latter still based on DIN 18800. The buckling curves, the reduction factors κ and the formulas for the verification of the buckling load are given in RiL 805:2012 Module 201 Annex 3. The applicable material properties of the (historical) material products to be used in steel bridge construction can be taken from RiL 805:2012 Module 103 Table 1 and are printed also here in Tab. 1 for convenience. For the base material, a distinction is made between different material products, with an additional distinction being made between the manufacturing period for mild steel produced around the turn of the 19th century. Concerning the partial safety factors to be used for the load-bearing capacities, RiL 805:2012 also differentiates between the material products and specifies a partial safety factor γ_M for the determination of the load capacity of each material. The applicable values in RiL 805:2012 are also printed in Tab. 1. A differentiation of the partial safety factor according to the limit state category has not been made so far because the verifications are essentially based on the yield strength f_y – even for verifications of the net cross-sectional resistance or the weld seams – and thus do not make use of the nominal values of the tensile strength, for which a higher partial safety factor $\gamma_{M2} = 1.25$ is used in the verification criteria according to EN 1993-1-1 [9].

The material parameters and partial safety factors summarised in Tab. 1 were derived from a combination of experience and statistical evaluations of relatively small datasets available to the company DB. There was no consistent adherence to the reliability requirements of EN 1990 [10], especially about the compilation and evaluation of representative test data.

Tab. 1 Nominal material strength properties and partial safety factors in RiL 805:2012 for validation of the load-bearing capacity

Steel grade	Yield strength $f_{y,k}$ [N/mm^2]	Tensile strength $f_{u,k}$ [N/mm^2]	Young's modulus [N/mm^2]	Shear modulus [N/mm^2]	Partial safety factor
Puddle steel, mild steel before 1900	220	320	200.000	77.000	1.20
Mild steel after 1900	235	335	210.000	81.000	1.15
St 37	240	360	210.000	81.000	1.10
St 48	312	480	210.000	81.000	1.10
St 52	360	510	210.000	81.000	1.10

Therefore, the calibration of partial safety factors in accordance with the methods and requirements of EN 1990 is considered as a relevant contribution to the revision of RiL 805. The statistical parameters of the material and geometric properties of existing riveted structures required for this objective were found in agreement with material test results on historical steel products available from the literature as presented in Section 2.3.2.

2.2 Methodology for partial safety factor calibration

As part of the research project on the possible update and EC3 harmonization of RiL 805, the verification concepts and the reliabilities of the verifications are to be generally aligned with those of the Eurocodes. On the one hand, the transition to the meanwhile familiar verification concepts in the Eurocodes contributes to the simpler feasibility of the structural assessment of existing structures. Furthermore and more importantly, the differentiation of the partial safety factors with the limit state category makes the safety margins more comparable. The calibration of the partial safety factors was conducted in accordance with the specifications in EN 1990 [10] Annex C.7. The procedure for calibrating the design values according to the first-order reliability method (FORM) is described in the aforementioned section. In particular, the method offers the possibility to dispense with an explicit probabilistic analysis. Instead, the proportional safety margins on the action and resistance sides are generally determined using the importance factors α_R and α_E as a projection of the total safety margin onto the respective marginal distributions. Thus, it is possible to consider the action and resistance sides of the design problem separately and to calibrate the respective partial safety factors. The concept is illustrated schematically in Fig. 1. The concept of separation of the safety margin with reference to a FORM analysis is described in more detail in [11].

The first step for calibration of the resistance-sided partial safety factors for ULS design of steel structures using formulas from EN 1993-1-1 [9] (differentiation of γ_{M0}, γ_{M1}, γ_{M2}) was the generation of samples for the strength and stiffness parameters by means of Monte Carlo simulations. These samples were used to evaluate representative limit states covering the scope of application of the partial safety factors γ_{M0}, γ_{M1} and γ_{M2}. This results in stochastic input parameters and a sample for the structural resistance that can be visualised as a histogram (see Fig. 1). The expected value μ_R and standard deviation σ_R of the sample are also shown schematically. The design quantile R_d, which fulfils the safety margin according to EN 1990, can thus be determined directly with the help of the importance factor α_R and the statistical moments of the sample (expected value μ_R and standard deviation σ_R) for the structural resistance in the ULS as follows:

$$R_d = \mu_R - \alpha_R \cdot \beta \cdot \sigma_R \qquad (1)$$

The calibration of the respective partial safety factors was conducted essentially according to Eq. (2) from a direct comparison of the nominal resistance R_{nom} with the design quantile R_d, wherein R_d was determined from the sample for the structural resistance according to Eq. (1):

$$\gamma_M{}^* = \frac{R_{nom}}{R_d} \qquad (2)$$

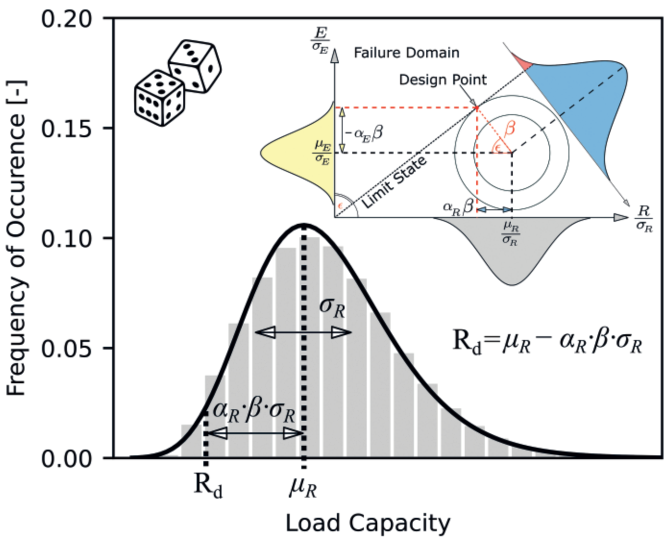


Fig. 1 Determination of the design value for the load-bearing capacity from a sample according to EN 1990 using importance factors

2.3 Historical steel and ferrous materials

2.3.1 Classification and use with railway bridges

Between the end of the 19th and the middle of the 20th century, the steel industry experienced a period of rapid technological developments in manufacturing processes, some of which were short-lived and replaced fast by more efficient processes. The competing development and use of the processes are indicative of the availability of steel products which differ at least in their production and therefore usually also in their mechanical properties. The distinction of the material properties according to the production process in Tab. 1 can be assigned to this background. The history of the technological development in the steel industry is documented in several contributions (see, e.g., [3, 11, 12]) and is revisited here only briefly to bring the conducted studies on different historical steel products in context with the historical development of steel production techniques.

Whereas between 1785 and 1860 the majority of available steel products are still produced by the puddling process, it was replaced by newer processes shortly after. This was, in particular, due to the development of the Bessemer process as the first process for producing mild steel. In the years after, the Thomas process and Siemens-Martin process were developed and used for many years due to their advantageous production technology [12]. The Siemens-Martin process prevailed due to better mechanical-technological properties and was the leading process for the production of mild steel after 1910 [13].

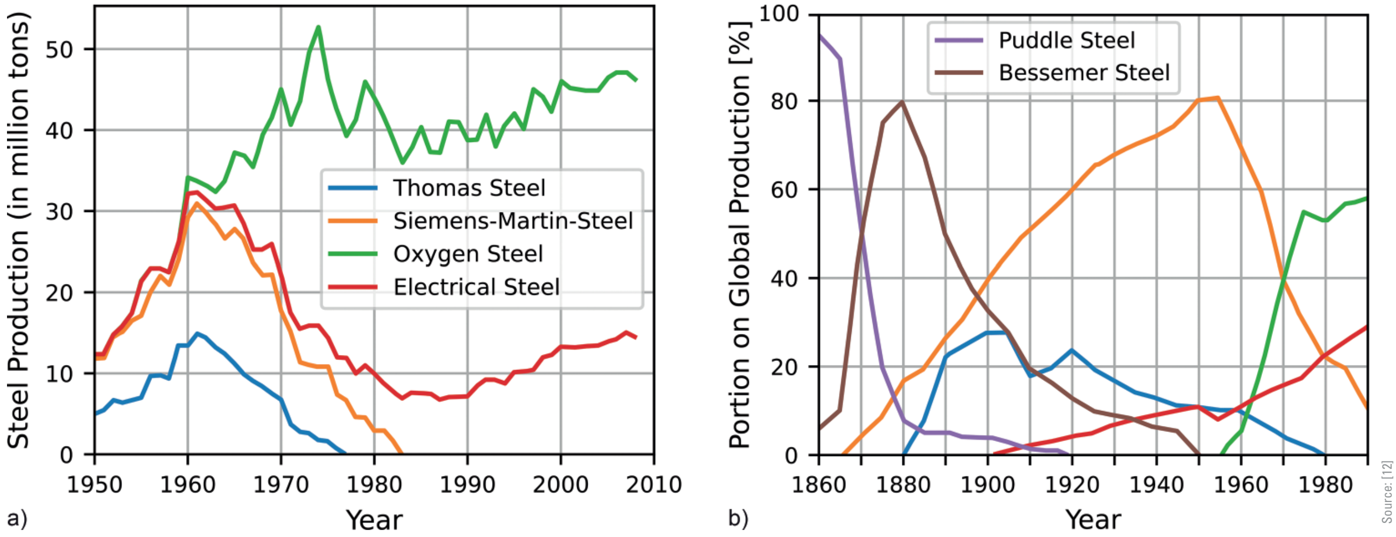


Fig. 2 a) Steel production in Germany; b) international steel production.

From the mid-1950s onwards, the proportional production volumes through the Siemens-Martin process declined due to the emergence of the oxygen blowing process, which significantly improved the material properties of converter steels. The process was developed to operational maturity at VÖEST in Linz and was first used for large-scale industrial production of oxygen steel in 1950 [12]. In the following years after 1950, the oxygen blowing process (oxygen steel) and the electric arc process became increasingly popular. In 1979, only 9.9 % of the steel produced in Germany was still made using the Siemens–Martin process. The shares of oxygen steel and steel from electric arc furnaces became much more important and were expanded after the shutdown of the last Siemens–Martin furnace in Western Europe in 1993. In 2005, 31.1 % of the steel production was provided by the electric arc steelmaking process and 68.9 % by the oxygen steelmaking process [14]. A further shift towards the electric process can be expected in the future. Fig. 2 illustrates the national and international development trends in steel production techniques.

2.3.2 Statistical database

The calibration of the partial safety factors for the representative limit states requires the availability of statistical data on the material properties. For this reason, a comprehensive and consistent database on the material properties of historical steel products has been compiled. Starting with data on puddle steel, this database covers historical steel products from the end of the 19th century to the present day. In the context of this publication, it is only possible to show some excerpts from the database used for the determination of the stochastic distributions for calibration of the partial safety factors. The data and filter criteria for data selection are described in more detail in [11, 15].

The data in Tab. 2 contain statistical information for the material test results for the yield strength R_{eH} and the tensile strength R_m of puddle steel and mild steel coupons. More precisely, the data in Tab. 2 document the expected value E of the mechanical properties, the coefficient of variation (CoV) and, if available, the boundaries of the test data obtained. The data originate from an extensive collection of material test results from the period 1850 to 1920 at the MFPA Leipzig [16]. For puddle steel, the dataset was extended later with material test results from the Federal Institute for Material Testing (BAM) in Germany [13]. In this dataset, the mean value of the tensile strength turns out to be somewhat lower [12], with a larger standard deviation at the same time (E = 361 MPa, σ = 31 MPa). Those more conservative statistical parameters were used to calibrate the partial safety factors for puddle steel. The data on material test results for mild steel in Tab. 2 include samples from the various production processes for mild steel at the turn of the century (Bessemer, Thomas, Siemens-Martin process). In [15], the

Tab. 2 Statistical data on material strength properties of historical steel products

Steel grade	R_{Eh} [N/mm^2]				R_m [N/mm^2]			
	E	CoV	Min	Max	E	CoV	Min	Max
Puddle steel	235	0.105	177	307	371	0.058	305	426
Mild steel	268	0.152	192	422	393	0.094	326	513
St 37	276	0.065	N.A.	N.A.	397	0.05	N.A.	N.A.
St 52	397	0.05	N.A.	N.A.	535	0.04	N.A.	N.A.

data are broken down even more finely according to the production processes for mild steel and were used to calculate individual partial safety factors for each of the three mentioned production processes for mild steel. By this, the partial safety factor obtained from the overall stochastic distribution for mild steel products according to Tab. 2 turned out to be conservative.

Regarding the mechanical properties of structural steel St 37 and St 52 from the second half of the 19th century, an extensive set of material test documents exists from the German Railway company DB. Despite intensive literature research, however, the statistical data are limited to the distribution of the yield strength. The statistics for the tensile strength were therefore developed from the statistics for the yield strength according to FprEN 1993-1-1 [17] and are explained in more detail in [15].

According to the procedure outline in Section 2.2, the statistical data on the material properties mentioned above were used to provide the mechanical properties as stochastic variables to the Monte Carlo simulations. In the case of puddle steel, the scatter of the material parameters can be represented by the normal distribution. In the case of mild steel specimens, this also applies in principle, but the lognormal distribution might be more suitable to reproduce the probability density function [16]. In a separate study, the difference in the partial safety factors for mild steel obtained from using the normal or the lognormal distribution turned out to be negligible and the lognormal distribution was used finally [15]. The mechanical properties of St 37 and St 52 structural steel products were also modelled using the lognormal distribution.

The statistical scatter of geometric parameters, which is finally reflected in the cross-sectional values, was modelled according to the data in [18]. The expected value of the cross-sectional area and the moment of inertia were therefore set to 99 % of the nominal values with coefficients of variation of 3 %.

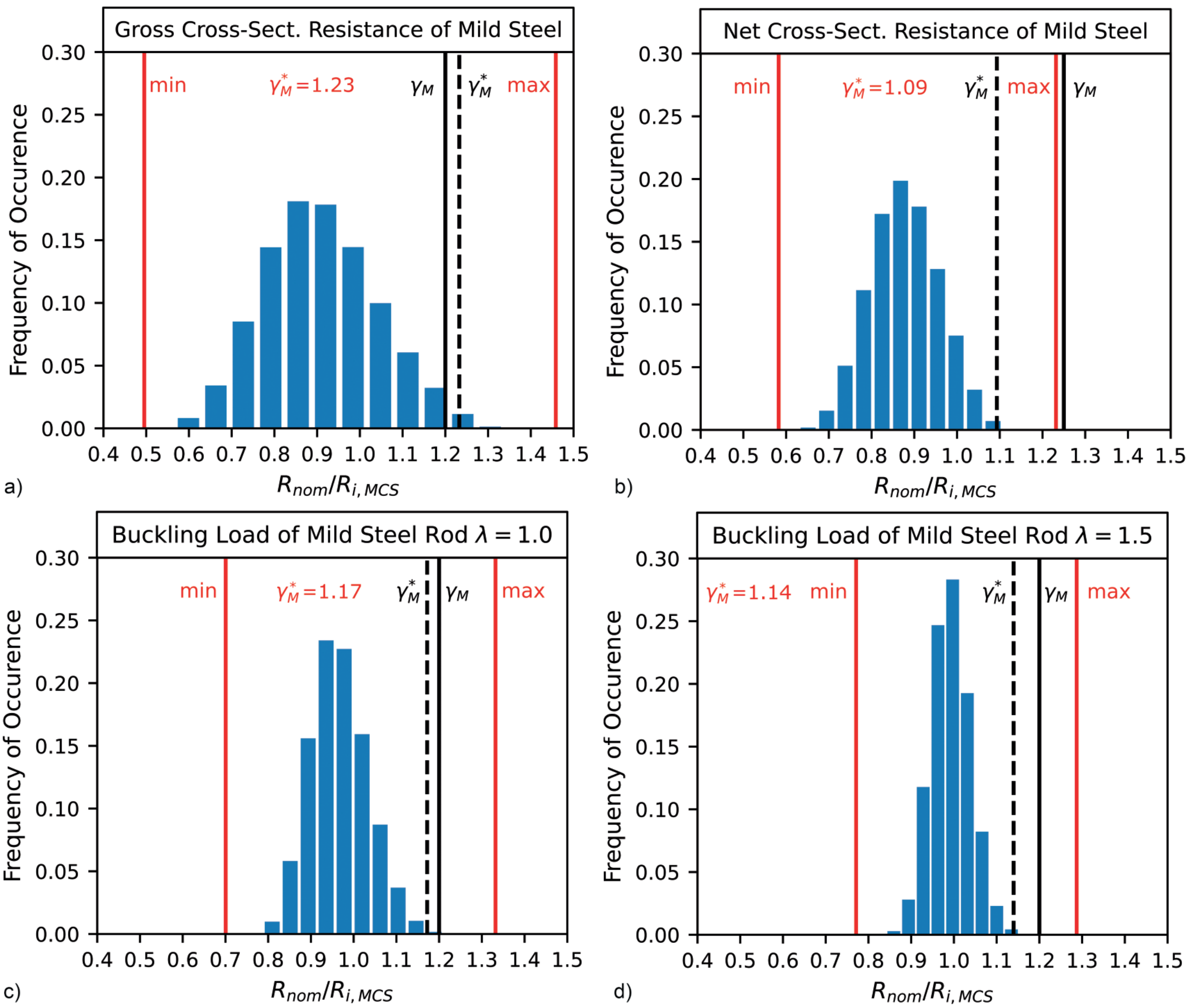


Fig. 3 Results of the partial safety factor calibration for mild steel in the limit state of a) gross cross-sectional load capacity, b) net cross-sectional load capacity, and c, d) buckling for different slenderness ratios

2.4 Representative results

Fig. 3 shows some exemplary results from the calibration of the partial safety factors for mild steel. The results in Fig. 3 cover all three representative limit state categories according to EN 1993-1-1 [9]. More precisely, the histograms show the quotient of the nominal resistance value and the resistance values sampled through Monte-Carlo simulations, respectively. This corresponds to a visualisation of the evaluation procedure according to Eq. (2). Therefore, the histogram data formally represent partial safety factors for different safety levels. Via the position of the design quantile R_d, the position of the nominally required partial safety factor γ_M^* for compliance with the required safety margin is clearly defined and indicated in the diagrams. This form of result visualisation was chosen because it allows comparing the nominally required partial safety factor and the specified value γ_M graphically. As the distribution densities in the tails are low and therefore hard to see, the interval limits are illustrated as well to indicate the scatter ranges of the samples. In the evaluation of the limit state of the net cross-sectional resistance with mild steel, it happened that the specified and generally accepted value 1.25 for the partial safety factor γ_{M2} was even outside the interval of the sample (Fig. 3b). In addition, it is worth mentioning that the pre-factor 0.9 in the limit state equation for the net cross-sectional resistance in tension was meant to avoid brittle tensile fracture, but is likely to be omitted in future editions of EN 1993-1-1, at least to a large extent. This factor was therefore neglected as well in the calibration of the partial safety factors for historical steel products.

A special characteristic of the buckling limit state is the variability of the parameter sensitivity with the member slenderness. The parameter sensitivity of the ultimate load-bearing capacity focuses from the material parameters to the geometric properties as the slenderness ratio increases. For this reason, the buckling limit state was analysed for three different slenderness ratios between 0.5 and 1.5 ($\bar{\lambda}$ = 0.5; 1.0; 1.5). The corresponding results in Fig. 3c,d for slenderness ratio $\bar{\lambda}$ = 1.0 and 1.5 indicate that γ_M^* decreases with increasing slenderness ratio. This observation is due to the shift of sensitivity to the geometric parameters with increasing slenderness ratio in combination with a lower scatter of the geometric parameters compared to the scatter of the material parameters.

2.5 Safety elements for the ultimate limit state verification of historical steel products

The proposals for the partial safety factors for limit state verification of historical steel products in accordance with the limit states in EN 1993-1-1 [9] are given in Tab. 3. The parameters provided thus allow to differentiate between the cross-sectional resistance in the gross section and the net section. The buckling limit state is covered separately as well with the partial safety factor γ_{M1}. The given mechanical properties are characteristic values in the sense of EN 1993-1-1. As already discussed in context with the data in Tab. 2, the designation mild steel refers here to the various processes for mild steel production at the turn of the 19th century (Bessemer, Thomas, Siemens-Martin process).

3 Fatigue checks with the damage equivalent factor concept

3.1 Present procedure in DB RiL 805

In addition to the ultimate limit state, the verification of the structural safety and residual service-life of existing bridges requires to assess the fatigue limit state as well. Typically, the damage equivalent factor concept applies for this purpose, for new structures as well as for existing ones. However, very often the concepts for the verification of the fatigue limit state in the current versions of recommendations for assessment of existing structures do not yet apply the limit state design concept with partial safety factors. While the proposal for the fatigue verification concept in the revised version of RiL 805 is a Eurocode-compliant concept, the concept specified therein is a concept for new structures and needs some adaptions for application to existing structures. Furthermore, the proposal aims to preserve compliance with the existing verification criteria. Therefore, the current concept for the fatigue verification in RiL 805 is presented briefly first (see [19] for more details), followed by the approach for conversion of the current concept to a Eurocode-compliant concept. The explanations emphasise the database and evaluation methodo-

Tab. 3 Proposal of characteristic material strength properties and partial safety factors for historical steel products

Steel grade	Yield strength $f_{y,k}$	Tensile strength $f_{u,k}$	Young's modulus	Shear modulus	Partial safety factors		
	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	γ_{M0}	γ_{M1}	γ_{M2}
Puddle steel	220	320	200.000	77.000	1.25	1.25	1.25
Mild steel	240	335	210.000	81.000	1.20	1.20	1.25
St 37	240	360	210.000	81.000	1.05	1.10	1.25
St 48	312	480	210.000	81.000	1.05	1.10	1.25
St 52	360	510	210.000	81.000	1.05	1.10	1.25

logy for development of the fatigue class catalogue for riveted connections.

According to the regulations in the current version of RiL 805, a fatigue check must be conducted in addition to the verification of the ULS for existing railway bridges made of steel older than 60 years. In accordance with the fatigue check in the Eurocodes, the concept in RiL 805 applies the damage equivalent factor concept for consideration of the load spectrum in the past and *S-N* curves for consideration of the fatigue strength of the materials. However, instead of checking a limit state, the fatigue verification itself is finally conducted by calculation of a residual service lifetime.

The calculation of the residual service lifetime applies essentially three steps. First, the present concept in RiL 805:2012 requires calculation of the so-called fatigue-relevant utilisation parameter $\beta_{D,UIC}$. This parameter can be calculated from the permissible stress range (zul $\Delta\sigma_{Be,\kappa}$) at two million load cycles, the dynamic amplification factor Φ and the maximum stress range (max $\Delta\sigma_{UIC}$) from crossing of load model LM71 according to EN 1991-2 [20]. The formula is given below:

$$\beta_{D,UIC} = \frac{\text{zul}\Delta\sigma_{Be,\kappa}}{\Phi \cdot \max \Delta\sigma_{UIC}} \tag{3}$$

In the second step, a basic value for the normalised fatigue damage accumulation $D_{past,1876}$ for the period within the years 1876 and 1996 is determined by means of the parameter $\beta_{D,UIC}$ and the damage equivalent factor α, which depends on the respective influence line and the associated span of the actual structure:

$$D_{past,1876} = \alpha \cdot \left(\frac{1}{\beta_{D,UIC}}\right)^5 \tag{4}$$

To account for the actual service life in the past and the traffic conditions of the respective bridge, the base value $D_{past,1876}$ for the fatigue damage accumulation in the past is corrected by the factors ρ_i. With this the normalised damage accumulation D_{past} in the past is obtained, considering the actual service time in the past and the annual gross tonnage per track, as well as the number of tracks and the train speed on the track with the factors ρ_1 to ρ_4. In the present concept, the fatigue damage accumulation in the past is used as well to determine the expected annual fatigue damage accumulation in the future applying a fixed percentage of 2.5% per year. Finally, the residual service-life can be calculated assuming the fatigue limit state at a damage accumulation value equals one as follows:

$$R = \frac{1 - D_{past}}{0.01 + D_{fut}} - A \leq 50 \text{ years} \tag{5}$$

with $D_{fut} = 0.025 \cdot D_{past}$

and A = year at time of analysis – 1996

3.2 Development and refreshment of a fatigue class catalogue for riveted connections

The fatigue strength of materials is typically described with *S-N* curves. Those curves represent the number of cycles to failure N as a function of the stress range and can be defined typically through (multi-)linear curves when both axes are in the logarithmic scale. In this format, *S-N* curves with a single slope can be provided with very few parameters, such as the permissible stress range at a defined number of cycles to failure and the corresponding slope. Modern design codes typically apply this format to provide the fatigue strength of structural details in a fatigue class catalogue.

In RiL 805:2012, however, the permissible stress range (zul $\Delta\sigma_{Be,\kappa}$) for the fatigue limit state is provided differently. Instead of using a fatigue class catalogue to differentiate between different types of connections, the permissible stress range has so far been provided through a table differentiating between the material, the fatigue class category and the stress ratio κ. In some cases, the permissible stress ranges differ significantly depending on the material. Tab. 4 shows an excerpt from Tab. 4 and 5 in RiL 805:2012 Module 201 for the determination of fatigue strengths.

As RiL 805 only applies the damage equivalent factor concept for verification of the fatigue limit state, it is in principle sufficient for description of the fatigue resistance to provide the permissible stress range at a certain number of cycles to failure. However, a differentiation between the connection types is not possible so far. As the assessment of the service life of riveted connections was at times regulated very differently internationally anyway [5], ÖBB initiated the development of the recommendation ONR 24008 [21] for evaluation of the reliability of existing road and railway bridges. The necessary research work was conducted at Graz University of Technology and included both fatigue tests on riveted bridge girders and a comprehensive literature study on fatigue tests on riveted connections, as well as the statistical evaluation and interpretation of this data. Within this framework, the most important internationally available data on fatigue tests on riveted details were compiled, classified into fatigue classes and statistically evaluated [5–8]. From the results, a fatigue class catalogue for riveted connections emerged, which allows to assess the fatigue behaviour of riveted connections more appropriately and, in the case of some standard details, allows a more favourable evaluation of the service life [5]. The results

Tab. 4 Permissible stress ranges for historical steel products per structural detail in RiL 805:2012 ($\kappa = 0$)

Structural detail	Puddle steel and mild steel before 1900	Mild steel St 37, St 48, St 52 etc.
With hole	74 [N/mm²]	127 [N/mm²]
Riveted	58 [N/mm²]	100 [N/mm²]

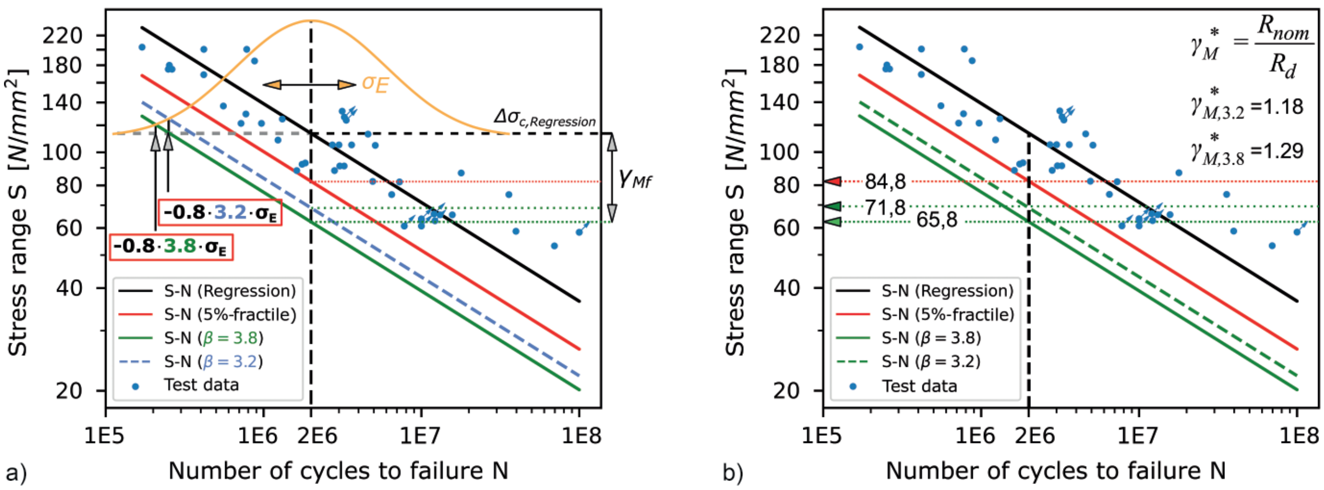


Fig. 4 a) Methodology for determination of design *S-N*-curves and partial safety factors and b) representative results for fatigue class category 1 (fatigue class catalogue see Tab. 5)

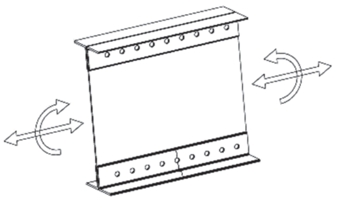
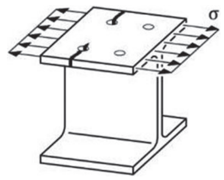
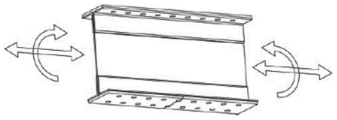
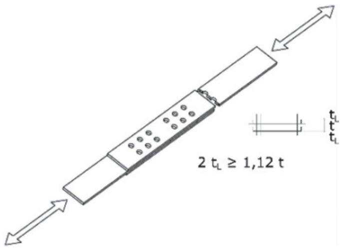
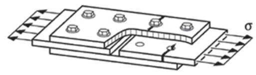
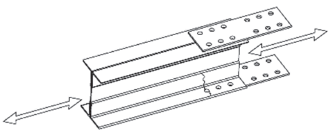
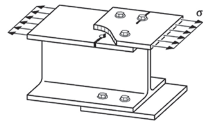
are documented in [5, 6] and have already become part of some recommendations in use (e.g., [21, 22]). It is recommendable to establish the differentiation of the fatigue strength according to the member type and loading conditions as proposed in [5–8] as well in the new edition of RiL 805 in the form of a fatigue class catalogue allowing for a more appropriate assessment of the fatigue strength. Within the updating efforts for RiL 805, the existing database was refreshed through a further literature study on fatigue tests on riveted connections published after 2007 and re-evaluated for specification of the fatigue class catalogue. The findings of the additional literature study and the evaluation methodology for the fatigue test data are reviewed in the sequel.

Three more publications with relevant data on the fatigue behaviour of riveted connections were found as a supplement to the existing database considering references up to the year 2007 [5–8]. The content of those three references [23–25] is revisited briefly. In reference [23], Mayorga et al. report on fatigue tests conducted on specimens of three riveted bridges in Portugal and France made of puddle steel. This contribution provides a considerable number of fatigue test results focusing on riveted connections of tensile rods with fishplate-like connections, using a single or multiple rivets along the loading direction. Bassetti et al. [24] report on fatigue tests on large components available from the dismantling of a bridge over the Hinterrhein river in Switzerland. Four riveted cross girders made of mild steel were taken from the bridge and subjected to a fatigue test in a four-point bending setup. Although the bridge had already been in service before for 91 years between 1902 and 1993, Bassetti et al. considered the accumulated fatigue damage in the parts to be presumably negligible. This is an important aspect for compliance with the data in [5–8]. Gocál et al. [25] report on six fatigue tests on riveted connections designed to represent the connection of the cross girder to the main girder as commonly built in steel railway bridges. A fourth publication [26] on fatigue tests on riveted members conducted recently at the Graz University of Technology was of less relevance in this context because it focused more on the fracture mechanical behaviour, and the members tested had presumably accumulated a considerable fatigue damage.

The evaluation of the fatigue data for specification of the fatigue class catalogue goes back to the concept described in Section 2.2. The objective of the evaluation is the determination of the design *S-N* curves, i.e., *S-N* curves at a certain safety margin. For application within the damage equivalent factor concept for riveted connections, those *S-N* curves are described entirely with the fatigue strength at two million cycles and the slope of the *S-N* curves, respectively, and the applicable partial safety factors γ_{Mf}. The first step of the methodology conducts a linear regression analysis on the test results from the fatigue tests on a logarithmic scale on both axes. Fig. 4 shows an example of the regression line on the results of fatigue class category 1, which is illustrated in Tab. 5. The goodness-of-fit of the regression line can be expressed globally by a scalar error term of the actual test data against the regression line and locally as a distribution with standard deviation σ_E around the regression line (see Fig. 4a). Using the standard deviation of the error term σ_E, it is possible to determine the design *S-N* curves and the design quantiles for the fatigue strength at 2 million cycles in accordance with the applicable importance factor α_R and reliability index β in EN 1990 [10] as explained in Section 2.2. The determination of the *S-N* curves corresponding to the 5 % fractile and the design levels β equal to 3.2 and 3.8 is shown schematically in Fig. 4a. Eq. (2) allows to calibrate the applicable partial safety factors at different safety levels by use of the fatigue strength values at 2 million cycles in those *S-N* curves in comparison to a nominal fatigue strength at 2 million load cycles. This is illustrated in Fig. 4b.

In some fatigue tests, the test was stopped before a fatigue crack was detected. As no failure was observed and the tolerable number of cycles is therefore unknown, these so-called run-outs are to be regarded as a special type of data in the statistical sense. In contrast to the tests with fatigue cracking, only a minimum number of admissible load cycles is determined in the case of run-outs. This re-

Tab. 5 Proposal for classification of fatigue details for the fatigue service-life check in the draft for the revised RiL 805

Fatigue strength	Structural detail/ Fatigue class	Description and examples	Comparable fatigue class in prEN 1993-1-9 [32]
$\Delta\sigma_C$ 90 $m = 5$	Category 1 	Continuous connection of flange angles and web plates in built-up girders; $\Delta\sigma$ at the centre of the rivet hole	$\Delta\sigma_C = 90$ $m_1 = 5$ 
	Category 2 	Continuous connection between cover plates and flange angles in built-up girders; $\Delta\sigma$ at the centre of the rivet hole	
$\Delta\sigma_C$ 90 $m = 5$	Category 3  	Symmetrical joint with splice plates Middle plates in two-shear connections are to be verified with $\Delta\sigma_C = 90$ $\Delta\sigma_C = 80$ applies for the splice plates themselves, so no verification is required when $2t_L > 1.12$ t.	$\Delta\sigma_C = 90$ $m_1 = 5$ 
$\Delta\sigma_C$ 80 $m = 5$	Category 4 	One-sided shear joint with gusset plates	$\Delta\sigma_C = 80$ $m_1 = 5$ 
	–	All other types of riveted connections with failures in the plates, conservatively	–

quires a suitable methodology for determining the regression curve between the stress range and the number of cycles to failure, which is explained in [27]. This procedure was used for development of the fatigue class catalogue for riveted connections [5, 6], but has found broad acceptance meanwhile, such as for application in, e.g., [28, 29]. The regression analysis is based on the maximum likelihood method and enables the determination of those regression parameters that describe the dataset with the greatest likelihood. This likelihood is composed of contributions from each individual test result, whereby a clear distinction is to be made between specimens with fatigue failure and run-outs. All test data with fatigue failure contribute to the likelihood estimator with their probability densities of the goodness-of-fit around the regression line, whereas the run-outs contribute with their exceedance probabilities and thus a cumulative probability density. According to this method, the error terms of the regression to the results from cracked samples are minimised. At the same time, the exceedance probabilities of the prediction from the regression line are maximised compared to the test results of the run-outs. For illustration purposes, the different contributions depending on the outcome of the test are illustrated in Fig. 5. The contribution of a cracked sample to the likelihood via the probability density is shown schematically in red in the distribution of the standard error (yellow). The contribution of a run-out via the exceedance probability is shown in blue over the distribution of the standard error.

The statistical evaluation results in the proposal for a fatigue class catalogue for riveted components as shown in Tab. 5. The proposed fatigue resistance parameters compare well with the currently intended parameters for comparable bolted connections in the upcoming revised version of EN 1993-1-9 [30]. The fatigue resistance parameters of bolted connections had been re-evaluated in a recent study [31], which formed the basis for the revision in prEN 1993-1-9 [32]. The fatigue resistance parameters intended for [32] are given as well in Tab. 5 for convenience.

Finally, a further, important aspect needs to be considered with respect to fatigue class definitions. The values

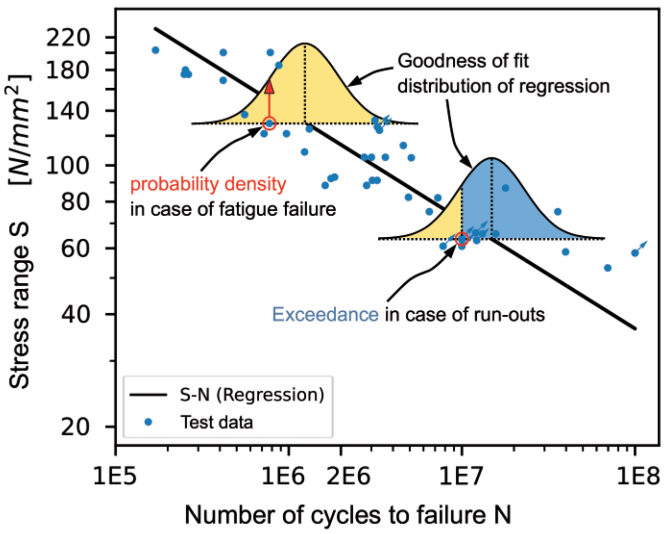


Fig. 5 Regression analysis using the maximum likelihood method for adequate consideration of run-outs

given in Tab. 5, considering also the findings in [31] for 'virgin' details, are considered to be valid without further modification for stress ratios equal or somewhat greater than 0.5 (with values higher than 0.75 being atypical or rather not connected with large fatigue loading). The proposed fatigue classes may thus conservatively be applied without explicit consideration of stress levels if these are positive. This proposal corresponds to the general fatigue verification concepts in EN 1993-1-9 [30] or the ECCS regulations [33] and those for riveted connections such as in, e.g., SIA269/3 [22]. This is justified by the fact that a stress ratio of approximately 0.5 is implied to be 'covered' by the chosen design fatigue classes, as is also described for bolted details in [31].

In many cases of riveted bridge details, the stress ratio will indeed not be known and thus the use of the fatigue resistances of Tab. 5 can be seen as cautious and good practice. For stress ratios lower than 0.5, this approach thus implies an additional safety margin by expectation. As the assessment of the actual mean stress level in complex bridge structures is highly uncertain, this additional safety margin is usually accepted. However, for very low stress ratios, the fatigue classes proposed in Tab. 5 might lead to smaller residual lifetimes of the structures respectively higher fatigue damage accumulation values in comparison with the former fatigue strength values WII and WIII in RiL 805:2012. The mentioned recent study [31] for reassessment of the fatigue classes for bolted connections confirmed similarities between the fatigue resistance of riveted and bolted connections and found similar parameters for the *S-N* curves as proposed here for riveted structural details. A substantial influence of the stress ratio on the fatigue strength has been confirmed therein as well.

Therefore, and in case where the assessing engineer is sufficiently confident that the actual stress ratio can be determined with sufficient reliability, an upgrade of the applicable fatigue classes is proposed, as shown in Tab. 6, to

Tab. 6 Modified reference fatigue resistance values in case of favourable and reliable mean stress levels

valid for $\kappa \geq +0.5$	$\kappa = 0.0$
$\Delta\sigma_C = 90$ ($m = 5$)	$\Delta\sigma_C = 112$ ($m = 5$)
$\Delta\sigma_C = 80$ ($m = 5$)	$\Delta\sigma_C = 100$ ($m = 5$)

compensate for the mean stress level influence. This means that the fatigue classes of Tab. 5 ($\Delta\sigma_c$ = 90 or 80 MPa) are modified as shown in Tab. 6 in the range between $\kappa = 0.0$ and 0.5. Intermediate values can be interpolated linearly.

Furthermore and independently of whether the modification in Tab. 6 is taken into account or not, in case of loading cycles that include compressive stresses, the applicable stress range for the fatigue damage analysis may be reduced, using the well-known procedure in EN 1993-1-9: the applicable stress range can be reduced eventually to the sum of the portion in the tensile stress domain and 60% of the portion in the compressive stress domain, as is done also in [22, 30, 31].

Regarding the applicable partial safety factor γ_{Mf} for fatigue verifications, it is intended to distinguish between inspectable and non-inspectable structural details. The proposed partial safety factors can be found in Tab. 7. This proposal is based on the results of the statistical evaluation of all fatigue class categories, as shown, for example, in Fig. 4b for fatigue class category 1. The target reliability level β of the proposed partial safety factors is 3.2 in this case; this level is by intention lower than the typical level equals 3.8 for new buildings and can be considered sufficient for the assessment of existing structures and the accuracy requirement of the damage equivalent factor concept.

3.3 Conversion to a Eurocode-compliant format

Newer standards and codes, and in particular the Eurocodes, conduct the fatigue verification of fatigue-prone structural details through the damage equivalent factor concept. Compared to the present concept in RiL 805, there are no fundamental differences, but some formal differences: basically there are adequate equivalents for the previous damage equivalent and correction factors α and ρ_i in RiL 805:2012 with the factors λ in the Eurocodes. Application of the damage equivalent factor concept as given in the Eurocodes to existing bridges, however, needs some adaptions. The adaptions are mainly related to the fact that existing bridges have accumulated fatigue damage in the past, but also do so in the future. In

Tab. 7 Partial safety factors for the fatigue service-life check

Inspectable	Yes	No
γ_{Mf}	1.0	1.15

addition, the fatigue verification in RiL 805 was conducted so far by calculation of a remaining service lifetime and some compliance of the new concept with the previous one was desired. The proposal for the new concept is presented in the sequel and illustrates how a Eurocode-compliant format enables to consider for damage accumulation in the past and in the future and how this format preserves compliance with the existing verification concept.

From a general point of view, e.g., for a new structure or if an overall value of λ is known, the damage or damage accumulation in a fatigue-prone structural detail can be determined with the following relationship according to Eurocode 3 [30, 34]:

$$D=\left(\frac{\gamma_{\mathrm{Ff}}\cdot\Phi_2\cdot\Delta\sigma_{\mathrm{p}}\cdot\lambda}{\frac{\Delta\sigma_{\mathrm{c}}}{\gamma_{\mathrm{Mf}}}}\right)^{\mathrm{m}} \tag{6}$$

In this concept, the damage equivalent factor is now designated with λ instead of α. The damage equivalent factor is again made up of a span-dependent basic factor λ_1 and correction factors to take into account the traffic volume, the actual service life, and the number of tracks, as has been done in RiL 805 so far with the correction factors. The proposed fatigue check in the draft for the new edition of RiL 805 is to be conducted on this basis through the verification of the normalised fatigue damage accumulation and the residual service life determined from the fatigue damage. The total fatigue damage accumulation is calculated additively from the fatigue damage accumulation D_{2020} in the past up to the year 2020 inclusively, and an annual damage accumulation D_{fut} in the future. The calculation of the damage accumulation parameters and the total damage are defined as follows:

$$D_{2020}=\left(\frac{\lambda_{\mathrm{past}}\cdot\Phi_2\cdot\Delta\sigma_{\mathrm{LM71}}}{\frac{\Delta\sigma_{\mathrm{c}}}{\gamma_{\mathrm{Mf}}}}\right)^{5} \tag{7}$$

$$D_{\mathrm{fut}}=\frac{1}{100}\cdot\left(\frac{\lambda_{\mathrm{fut}}\cdot\Phi_2\cdot\Delta\sigma_{\mathrm{LM71}}}{\frac{\Delta\sigma_{\mathrm{c}}}{\gamma_{\mathrm{Mf}}}}\right)^{5} \tag{8}$$

$$D=D_{2020}+a\cdot D_{\mathrm{fut}} \tag{9}$$

with a = year at time of analysis 'minus' (–) 2020

$$\lambda_{\mathrm{past}}=\lambda_{1,\mathrm{past}}\cdot\lambda_2\cdot\lambda_{3,\mathrm{past}}\cdot\lambda_4$$

$$\lambda_{\mathrm{fut}}=\lambda_1\cdot\lambda_2\cdot\lambda_4$$

The proposed revision for the fatigue check intends to use separate damage equivalent factors for the damage accumulation in the past and the one in the future. The factor λ_{past} is supposed to account for the damage accumulation in the past and has been determined under consideration of load spectra and the corresponding fatigue damage accumulation since 1880 for main and secondary lines. The damage accumulation due to nowadays and future traffic is supposed to be considered with the factor λ_{fut}. According to the terminology in EN 1993-2 [34], the correction factors λ_2 and λ_4 consider the annual traffic volume and the number of lanes, respectively. The extension of the damage accumulation period in the past from the year 1996 in RiL 805:2012 to the year 2020 is significant because this allows grasping the fatigue damage processes in the past with fixed values tabulated in RiL 805 and adopts the damage equivalent factors of EN 1993-2 for determination of the damage accumulation processes in the future. Since the year of transition between damage accumulation periods in the past and in the future is proposed to be shifted from 1996 to 2020 in the course of the revision, it is necessary to re-determine the basic factor $\lambda_{1,\mathrm{past}}$ for damage accumulation in the past and the service life coefficient $\lambda_{3,\mathrm{past}}$ for the period between the reference year 1880 in the past and the year 2020. The methodology for determining these two parameters and some representative results are discussed in Sections 3.4.2 and 3.4.3.

Whereas the calculation of the residual service-life RND in RiL 805:2012 applied a fixed ratio of the fatigue damage accumulated in the past for estimation of the annual damage accumulation in the future, the latter can be computed explicitly with the proposed concept. With this, the residual service lifetime is to be computed with the following relationship (with $\mathrm{YT}_{\mathrm{Analysis}}$ as the year at the time of the analysis):

$$\mathrm{RND}=\frac{1-D}{D_{\mathrm{fut}}}-\left(\mathrm{YT}_{\mathrm{Analysis}}-2020\right) \tag{10}$$

The calculated damage index D and the remaining service life RND in years are decisive whether normal inspections are sufficient or whether proof of safe service life intervals or further measures are required. A corresponding decision matrix, which determines, for example, at which damage levels a more precise verification on the basis of fracture mechanics is required or the inspection intervals should be condensed, is currently under discussion and supposed to be part of the new edition of RiL 805.

3.4 Update of the damage equivalent factors λ for past damage accumulation

3.4.1 Applicable traffic load spectra

The damage equivalent factor concept is a very efficient approach for verification of the fatigue limit state of fatigue-prone components and structural details. More precisely, it is not necessary to simulate the passage of the expected traffic queue and to perform a damage accumulation analysis. Instead, only the stress range due to load model LM71 is to be calculated and scaled with the applicable damage equivalent factors to determine the damage accumulation from the expected traffic spectrum with a single-level stress spectrum. The concept applies in the first instance the normalised damage equivalent factor λ_1, which is supposed to consider the damage accumulation due to a specified load spectrum as a function of the influence line and span length of the structure. The factor is called normalised because it is calibrated and given for a fixed volume of annual traffic loads. This damage equivalent factor was last determined in 1996 for RiL 805:2012 (see [35]) and needed to be re-evaluated for two reasons. On the one hand, the damage equivalent factors are to be updated from time to time, especially if the load spectra change, but also if the underlying time period is adapted. On the other hand, the annual damage accumulation in the future is supposed to be calculated explicitly in the proposed new version of RiL 805 instead of as a crude ratio of the total damage accumulation in the past. As the calibration of the damage equivalent factors depends more on the slope of the *S-N* curve than on the fatigue strength of the structural detail itself, and as the slope of the *S-N* curves for riveted connections is similar to those EN 1993-2 [34] operates with in terms of damage equivalent factors, it appears reasonable, if potentially conservative for shorter periods of planned lifetime extension, to use the damage equivalent factors in EN 1993-2 for calculation of the annual damage accumulation in riveted connections in the future. This is a particularly sensible approach if no object-specific traffic data is known or planned to be determined. In order to link the damage accumulation analysis for existing bridges to the damage equivalent factors available in EN 1993-2 for new structures, it was necessary to split the damage accumulation processes in the past from that in the future and to grasp the damage accumulation in existing bridges in the past by adequate damage equivalent factors. For this purpose, the year 2020 was defined as break between the damage accumulation periods in the past and in the future. As this corresponds to an extension of the damage accumulation period in the past in RiL 805:2012 from 1996 to 2020, the normalised damage equivalent factor for damage accumulation in the past $\lambda_{1,\text{past}}$ and the corresponding factor for accounting of the actual service life $\lambda_{3,\text{past}}$ had to be recalculated.

The calibration of the damage equivalent factors in the present version of RiL 805 is based on the load spectrum as described and published in [35]. Therein, six load spectra were defined, which represent the traffic composition on the lines of the DB Netz AG in eight periods between 1876 and 1995 fictitiously. A distinction was made between main lines and secondary lines, which were further subdivided into lines used mainly for freight, passenger and mixed traffic, respectively. References [19, 35] are given for further details on the load spectra in the past. For re-evaluation of the damage equivalent factors, the same load spectra as in [35] were used for the period between 1880 and 1996. Those spectra were extended by further load spectra between the last damage accumulation analysis in 1996 up to the proposed reference year 2020. For this, the proposed load spectrum in [35] for the time after 1996 was adopted and applied up to the year 2012 inclusively. The subsequent time period between 2013 and 2020 and beyond was covered with the load spectrum in EN 1991-2 Annex D [20]. The traffic composition on mixed traffic lines was assumed according to Table D.1 and that one on freight traffic lines according to Table D.2. The load spectrum on passenger traffic lines was specified on the base of Table D.1 as well, whereby 90 % of the annual traffic volume goes back to passenger traffic and 10 % to freight. Finally, the set of damage equivalent factors was augmented with those for suburban railway traffic. Similar with the load spectra on main and secondary lines, this load spectrum had been analysed already before up to the year 1996 in [19] and needed to be re-evaluated up to the year 2020. All load spectra were specified in close agreement with DB Netz AG and the German Federal Railway Authority (EBA).

3.4.2 Methodology for determination of the damage equivalent factors λ

The determination of the damage equivalent factors makes use of *S-N* curves and the linear damage accumulation hypothesis of Palmgren-Miner to convert the load spectrum into a damage-equivalent single-level stress spectrum. For this purpose, the inclination of the underlying *S-N* curves is of significant importance. For the fatigue-prone connection types considered here (mainly riveted ones, and generally non-welded connections), a constant gradient of $m = 5$ can be assumed, as shown in section 3.2. Therefore, the formulas shown hereafter apply this value for the inclination of the *S-N* curve. The damage accumulation for a specific load spectrum is thus determined as follows:

$$D_\text{d} = \frac{1}{2 \cdot 10^6} \cdot \frac{1}{\Delta\sigma_\text{c}^5} \sum n_\text{Ei} \cdot \Delta\sigma_\text{i}^5 \tag{11}$$

The concept of damage equivalence implies that this fatigue damage D_d is represented equivalently through a single-level stress spectrum due to crossing of load model LM71 after scaling with the damage equivalent factor λ and the dynamic amplification factor:

$$D_\text{d,equ} = \frac{1}{2 \cdot 10^6} \cdot \frac{1}{\Delta\sigma_\text{c}^5} \cdot 2 \cdot 10^6 \cdot \left(\lambda \cdot \Phi_2 \cdot \Delta\sigma_\text{p}\right)^5 \tag{12}$$

From this, the explicit expression for calculating the basic value λ_1 of the damage equivalent factor can be developed:

$$\lambda_1 = \frac{1}{\sqrt[5]{2 \cdot 10^6}} \cdot \frac{1}{\Phi_2 \cdot \Delta\sigma_\text{p}} \sqrt[5]{\sum n_\text{Ei} \cdot \Delta\sigma_\text{i}^5} \tag{13}$$

The damage equivalent factors $\lambda_{1,\text{past}}$ for the load spectra of the different railway lines (freight, passenger, mixed traffic on main and secondary lines, respectively; suburban lines) were calculated according to Eq. (13). The spectrum of stress ranges $\Delta\sigma_\text{i}$ was obtained from a Rain-

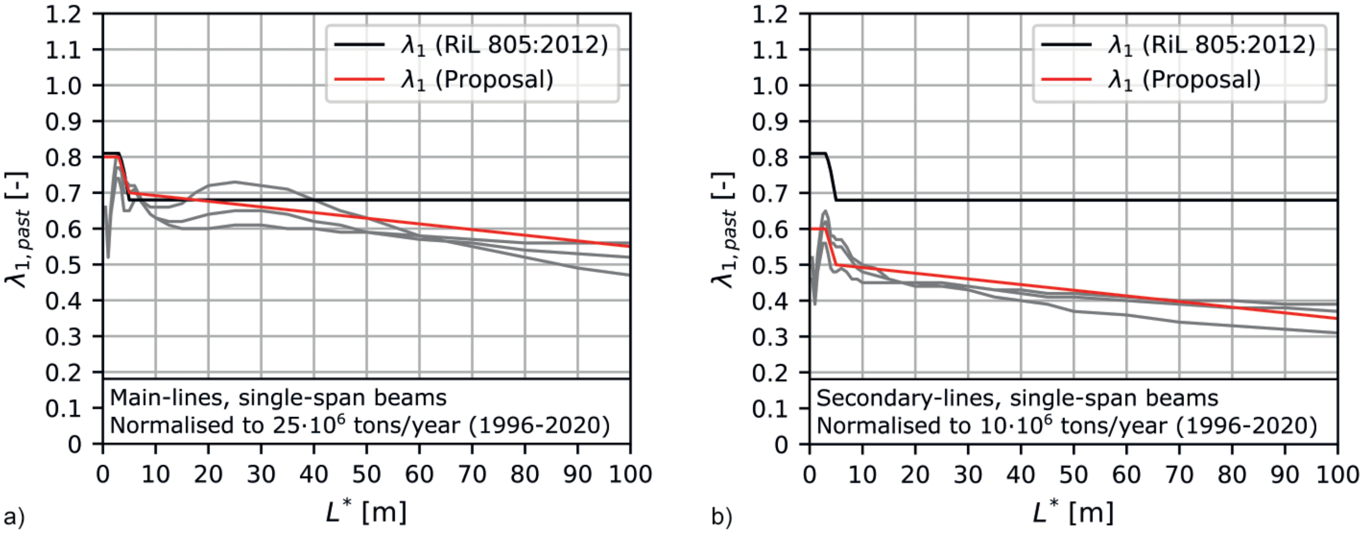


Fig. 6 Representative results of the calculated damage equivalence factors for a) main and b) secondary traffic routes as well as proposals for reappraisal of the basic value $\lambda_{1,\text{past}}$ of the damage equivalence factors

flow counting [36] analysis on the stress time signal from traffic simulations in accordance with the load spectra presented in Section 3.4.1. This signal consists of a composition of the stress signals of the load models for individual trains, which were determined in advance by a crossing simulation using influence lines. The full time signal was composed of the individual signals of about 85 different trains in total using the relative frequencies of occurrence for specification of a random traffic series.

3.4.3 Representative results

In the course of study of potential updates to RiL 805, the basic value of the damage equivalent factors $\lambda_{1,\text{past}}$ was re-determined keeping the reference year 1880 for the begin of the damage accumulation period in the past but extending it up to the year 2020 inclusively. In accordance with the distinction between single-span and two-span beams in RiL 805, the analysis was conducted as well for both system types. Both static systems were analysed systematically for the seven load spectra mentioned in Section 3.4.1 (passenger, freight and mixed traffic on main and secondary lines, respectively; suburban traffic) and all spans between 2 and 100 m. Fig. 6 and 7 show representative results for the damage equivalent factors $\lambda_{1,\text{past}}$ for cyclic bending effects in midspan of single-span beams on main and secondary lines and the suburban railway line along with the effective length of the influence line. The grey lines show the raw damage equivalent factors calculated for the traffic spectra under investigation, which are freight, passenger and mixed traffic spectra for the results shown in Fig. 6a,b. The calculations show that the damage equivalent factors used so far in RiL 805:2012 (in the form of α, converted to λ_1 for comparability) are not fundamentally unconservative, even approximately 25 years after their computation in [35]. Therefore, the update does not require any fundamental change to the previous values. For secondary lines and suburban railway lines, however, an adjustment of the

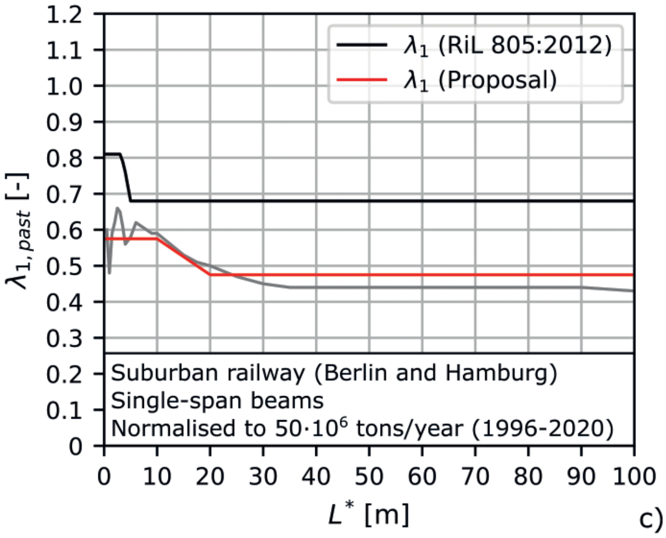


Fig. 7 Damage equivalence factor $\lambda_{1,\text{past}}$ for suburban railway traffic (Berlin and Hamburg)

present lines is recommended due to the composition of the load spectra and different normalisation of the traffic volume. Fig. 6b and 7 show adequate proposals.

The coefficient $\lambda_{3,\text{past}}$ takes into account the year of construction and the actual service life and represents by this the damage accumulation rate due to the load spectra over time. This damage accumulation rate is not constant over time because the load spectra have varied in the past. It is therefore important to provide this parameter. As $\lambda_{3,\text{past}}$ correlates directly with the basic factor $\lambda_{1,\text{past}}$, the parameter $\lambda_{3,\text{past}}$ requires re-evaluation as well. The proposal for updating the service life coefficient $\lambda_{3,\text{past}}$ for damage accumulation in the past on main and secondary lines, respectively, and suburban railway traffic is shown in Fig. 8.

Fig. 8 also shows representative results for the intermediate damage accumulation on single-span and two-span girders with spans of 5, 10, 20 and 40 m for the load spec-

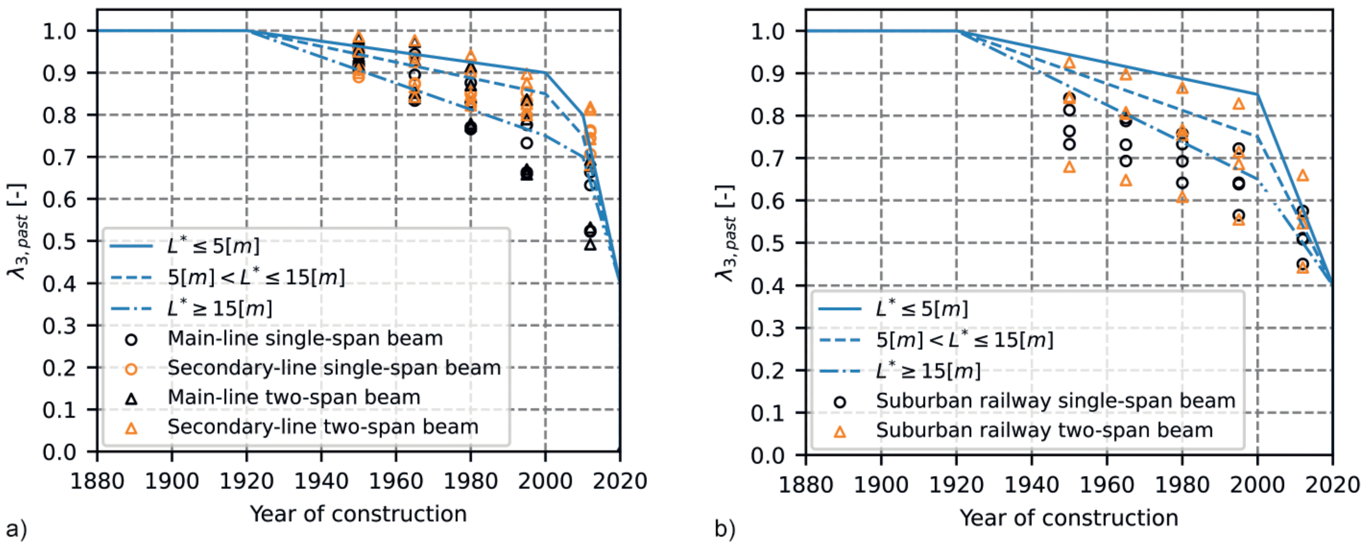


Fig. 8 Damage equivalence factors $\lambda_{3,past}$ for consideration of the service time on a) main/secondary lines and b) suburban railway lines

trum of passenger traffic on main and secondary lines (Fig. 8a) as well as for suburban railway traffic (Fig. 8b). The damage equivalent factors are typically a little lower with higher span lengths; therefore, the proposal varies with the respective span length. For the linear interpolation of $\lambda_{3,past}$, the anchor points of the proposed lines are given additionally in Tab. 8 and 9.

4 Summary of possible updates in modules 103 and 201 in RiL 805

The planned updates in module 201 of RiL 805 affect the verification of the ultimate and fatigue limit states. Partial safety factors have been calibrated for historical steel products to allow verification of the ULS against static loads in analogy to the verification concepts in EN 1993-1-1 [9] with differentiation between the different types of the ULS. The proposed partial safety factors are given in Tab. 3. Regarding the material properties, there are only some minor changes, which are also included in Tab. 3.

The fatigue verification concept according to RiL 805 is converted into the concept according to Eurocode and applies partly the parameters in EN 1993-2 [34]. The planned actualisations of the recommendation include the introduction of a fatigue class catalogue for riveted connections, which is given in Tab. 5, as well as the use of partial safety factors for the fatigue strength given in the fatigue class catalogue. The correlation of the fatigue strength with the mean stress level is to be considered through an optional upgrade of the fatigue strength and the applicable stress range instead of giving the fatigue strength values explicitly in a table. The underlying time period for assessment of the damage accumulation in the past was extended up to the year 2020. The corresponding damage equivalent factors were re-calculated and changed to the terminology in EN 1993-2. For the calculation of the annual damage in the future, the damage equivalent factors in EN 1993-2 can be used. With the damage in the past and the annual damage in the future, it is still possible to determine the residual service life according to the fatigue limit state. The residual service life and the accumulated fatigue damage are decisive whether further fracture mechanical analyses for proof of safe operating time intervals apply or whether even further measures are required.

Tab. 8 Significant values for the service-life correction factor $\lambda_{3,past}$ on main/secondary traffic routes in Fig. 8a

L^*	1880	1920	1940	1960	1980	2000	2010	2020	2021
$L^* \leq 5\ m$	1,0	1.0	0.975	0.95	0.925	0.9	0.8	0.4	0
$5\ m < L^* \leq 15\ m$	1.0	1.0	0.9625	0.925	0.8875	0.85	0.75	0.4	0
$L^* > 15\ m$	1.0	1.0	0.9375	0.875	0.8125	0.75	0.7	0.4	0

Tab. 9 Significant values for the service-life correction factor $\lambda_{3,past}$ on suburban traffic routes in Fig. 8b

L^*	1880	1920	1940	1960	1980	2000	2010	2020	2021
$L^* \leq 5\ m$	1.0	1.0	0.9625	0.925	0.8875	0.85	0.625	0.4	0
$5\ m < L^* \leq 15\ m$	1.0	1.0	0.9375	0.875	0.8125	0.75	0.575	0.4	0
$L^* > 15\ m$	1.0	1.0	0.9125	0.825	0.7375	0.65	0.525	0.4	0

5 Summary and conclusion

The recommendation DB RiL 805 used for the verification of railway bridges in the DB network in Germany was subjected to an actualisation in recent years. The intention of the update mainly refers to the adaptation of the verification concepts to those in the Eurocodes. This article reports exclusively on contributions to the update of the verifications for existing steel bridges. In the last majorly updated version of RiL 805 from 2012, the ULS checks are already conducted according to the semi-probabilistic concept with limit states and partial safety factors, so that no fundamental changes are planned for the ULS checks. However, in contrast to the verification concepts in EN 1993-1-1 [9], the differentiation of the partial safety factors on the resistance side according to the type of the ULS, i.e., cross-sectional resistance in gross/net section, buckling, was missing up to now. Therefore, partial safety factors were calibrated in accordance with EN 1990 [10] in order to harmonise the verification concept with EN 1993-1-1, and in particular to equalise the safety margins between the different types of the ULS. In accordance with the differentiation of the mechanical material properties in RiL 805, the calibration was also conducted individually for the various historical steel products in order to consider the respective mechanical properties and their statistical characteristics appropriately.

Regarding the fatigue verification in RiL 805, the changeover to the damage equivalent factor concept according to EN 1993-2 [34] is proposed. For this purpose, an existing fatigue class catalogue [5] for riveted components was updated and partial safety factors for the fatigue strengths specified therein were calibrated. This was done in accordance with the safety requirements in EN 1990.

Furthermore, the underlying time periods for calculation of the fatigue damage accumulation were adapted such that an explicit calculation of the annual damage accumulation in future periods is enabled and that the damage equivalent factors in EN 1993-2 can be used for this. The year 2020 was defined as the reference year up to which the damage in the past is being calculated, and the damage equivalent factors were recalculated and updated up to this year. Thus, all prerequisites are given to carry out the fatigue verification for existing riveted bridges with the damage equivalent factor concept in accordance with the concept in EN 1993-2. For this purpose, the draft of the new version of RiL 805 now provides a fatigue class catalogue for a more differentiated assessment of the fatigue strength of riveted connections and updated damage equivalent factors for efficient determination of the accumulated fatigue damage.

In the opinion of the authors, the adaptations presented here fulfil the goal of upgrading RiL 805 to become an assessment recommendation that is more compatible with Eurocode 3-type verifications, as currently used for new structures. A corresponding upgrade ensures direct compatibility with the verifications and reliability requirements of the technical building regulations of the Eurocodes that also apply to new buildings. Together with the other contributions presented in [2–4] regarding the verification of the safe operating time interval and the choice of materials, RiL 805 would thus be modernised in a way that matches the task of this recommendation: enabling the longest possible and safe operation of existing railway bridges.

Currently, parts of these proposals are being included in the RiL 805 revision, while the previous format of, e.g., the fatigue verification was maintained due to the currently still great familiarity of most railway engineers with its format. Regardless of this only partial implementation, the authors believe that the results and procedures for the reassessment of partial factors, fatigue classes and damage equivalent factors are suitable for implementation in standards and codes of practice, e.g., at European or other national levels.

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