Probabilistic Background of Eurocode Ship Impact Forces and a Remaining-Life-Based Reduction Rule for Existing Bridges

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Ship impact on bridges represents a highly complex load case due to its dynamic and probabilistic nature. Experimental research gained momentum in the mid-20th century, with seminal studies laying the foundation for understanding of impact mechanics. The key influencing parameters include the vessel's mass, geometry, stiffness, and velocity. This variability makes it difficult to derive representative impact forces for design and analysis. The load models in the Eurocode have been derived from probabilistic analyses; however, verifications are typically carried out deterministically. This paper reconstructs the probabilistic framework underlying the Eurocode ship impact provisions and clarifies the link between collision models, impact force distributions and return periods. Building on this framework, a rational approach is presented that allows ship impact loads for existing bridges to be adapted depending on the remaining working life of the structure. The proposed approach is also, in principle, applicable to other types of accidental actions. The method enables impact-action levels to be related to a finite remaining assessment period while preserving the nominal cumulative exceedance probability assigned to the reference action, thereby fostering more efficient and sustainable use of resources.

Keywords: impact; accidental actions; structural reliability; probabilistic assessment; existing bridges; existing structures; dynamic load

1. Introduction

Ship impact on bridge piers represents a challenging accidental load case characterised by pronounced dynamic effects and considerable uncertainties in both action magnitude and probability of occurrence. While the kinetic energy of a vessel can be estimated relatively easily, translating this energy into representative design loads is difficult because the force time history depends strongly on vessel geometry, stiffness, mass distribution and impact conditions. In addition, ship impacts are rare – accidental – events that are often dominated by human error, which makes reliable estimation of occurrence probabilities particularly difficult.

Modern design provisions therefore combine simplified mechanical impact models with probabilistic concepts to derive representative design loads. In Europe, the governing framework is provided by Eurocode EN 1991-1-7, where ship impact forces are derived using probabilistic considerations linking collision models with mechanical load models. Despite the practical relevance of these provisions, their probabilistic background and underlying assumptions are rarely discussed in the international scientific literature. As a result, the origin and interpretation of the prescribed design values remain unclear.

At the same time, the assessment of existing bridges is becoming increasingly important, as large parts of the bridge stock were designed according to older standards and infrastructure owners aim to extend working life wherever possible. In this context, assessment methods are needed that allow accidental actions, such as ship impact, to be treated consistently when the remaining working life of the structure is shorter than the design working life assumed for new bridges.

Several studies and recommendations address reliability levels and partial factors for existing structures, including (Allaix et al., 2016; fib, 2024; JCSS, 2001). These documents provide guidance on minimum target reliabilities based on criteria of economic optimisation and human safety. However, they address persistent design situations and are largely focused on buildings. They do not provide a directly applicable procedure for adapting accidental action levels, such as vessel impact loads, to the reduced remaining working life of existing bridges.

Against this background, the paper has two main objectives. First, it provides a structured reconstruction of the probabilistic basis of the Eurocode ship impact load model, linking collision probabilities, impact force distributions and nominal return periods. Underlying principles are highlighted, verified and compared with their counterparts in the American code (AASHTO, 2009). Second, building on this clarified background, a rational approach is presented for consistent adaptation of ship impact loads for existing bridges as a function of their remaining working life. The approach is intended as a practical reassessment rule for existing infrastructure.

The proposed approach supports a differentiated treatment of existing bridges and may contribute to more efficient and sustainable infrastructure management. By relating accidental action levels to the remaining working life, it allows available resources for assessment, strengthening and replacement to be allocated more effectively. Although the paper focuses on ship impact on inland waterways, the underlying concept may also be applicable, in principle, to other accidental actions for which design values are expressed in terms of nominal occurrence probabilities or return periods.

2. Background

The purpose of this section is to provide a more detailed technical and historical context for the ship impact load case, forming the basis for the load adaptation for existing bridges presented in the subsequent sections.

2.1. Characteristics of the ship impact load case

Dynamics. Ship impact is a dynamic load case. Determining the energy involved is straightforward, as this simply corresponds to the kinetic energy of the ship: $E = 1/2 \cdot mv^2$. This holds at least in the case of frontal impacts, assuming the vessel comes to a complete stop (*EN 1991-1-7*, 2025). However, it is much more difficult to determine representative forces. This is because the impact force acts dynamically on the bridge; it is a function of time. The force-time history and above all the maximum of this force are highly individual and depend in particular on the geometry and the stiffness and mass distribution of the ship's hull (Song & Wang, 2019). Nevertheless, the maximum impact force alone is probably not very meaningful. A maximum force that only acts for a tenth of a second will hardly have any damaging effect on a solid pier (AASHTO, 2009). A somehow defined averaged value of the force history is often considered more meaningful (Svensson, 2009).

Probabilistics. Most ship impacts can be attributed to human error, which renders risk assessments and the definition of design loads based on causal analysis particularly complex. Consequently, a more promising approach is to base such assessments on empirical evidence and systematic analysis of observed incidents. Nevertheless, this approach faces inherent limitations, as ship impacts constitute a relatively rare load case (Proske, 2008). The majority of bridges are unlikely to experience a catastrophic collision during their working life. Due to this infrequency, the estimation of ship impact probabilities remains highly uncertain, as historical data are typically sparse – particularly when considering individual structures or site-specific conditions, cf. (Proske, 2018).

To address this, analyses often focus on broader geographic and temporal scopes – such as entire rivers or countries – where a sufficient number of past events provide a representative basis for probabilistic assessments. These broader assessments are then refined for individual river sections or specific bridges using adaptation factors. Such factors account for local and site-specific conditions, including river geometry, bridge clearance profiles, or the presence of protective measures like dolphins. Risk-reducing measures, such as transit restrictions (e.g., daytime-only navigation) or the presence of a second ship pilot, can also be incorporated into these refinements (AASHTO, 2009; Knott & Winters, 2018).

2.2. Historical development

Ship impacts on bridges – particularly on their piers – have been a recurring issue throughout history (Proske, 2008). However, systematic research into the forces generated by such events and the development of standardised regulations to address this load case have emerged only in recent decades.

The first theoretical investigation into ship collisions was carried out by Vladimir Minorsky in 1959 (Minorsky, 1959). His work primarily addressed ship-to-ship collisions, focusing on energy dissipation and damage assessment, rather than interactions with rigid structures like bridge piers. This study laid the foundation for understanding collision mechanics but left ship-bridge interactions unexplored.

Progress was made in 1976 when Gehard Woisin conducted experimental research in Hamburg, Germany, on seagoing ship hull impacts against rigid objects (Woisin, 1976). These tests clarified the mechanics by directly measuring force–time histories during impact. From the results, Woisin derived an empirical relationship that expresses the peak impact force as a function of the ship's deadweight tonnage (DWT). He also highlighted the strongly transient nature of the impact event: the force maximum occurs only briefly and may therefore overstate the sustained demand on a structure. To address this, Woisin introduced the mean force over the collision duration, F_m, and showed that it can be taken, in first approximation, as about half the peak value, $F_m \approx 0.5F_{max}$ (Svensson, 2009). Similar experiments, specifically involving barges and inland waterway ships, were later performed by Meier-Dörnberg (Meier-Dörnberg, 1983).

In 1980, a cargo ship struck a bridge pier of the Sunshine Skyway Bridge in Florida during a storm. The impact caused 366 meters of the bridge to collapse, resulting in 35 fatalities. This accident was the starting point for a more intensive scientific debate on the subject of ship impacts in the USA, which ultimately led to the *Guide Specification and Commentary for Vessel Collision Design of Highway Bridges* (AASHTO, 1991) in 1991. The 2nd edition of this standard followed in 2009 (AASHTO, 2009); in the meantime, the regulations on vessel impacts have been incorporated into the general bridge construction standard *AASHTO-LRFD Bridge Design Specifications* (AASHTO, 2010).

In Germany, although pioneering experimental research on ship impacts had been conducted, standardized regulations were slow to emerge (Kunz, 1990; Grob & Hajdin, 1995; Kunz, 2006). Earlier editions of DIN 1072, including the 1985 version (Deutsches Institut für Normung, 1985), delegated responsibility for determining ship impact loads to local authorities. An exception was the Rhine waterway, which had specific impact load regulations from 1974 (BMV, 1974) with a static equivalent impact force of 30 MN against bridge piers.

The first comprehensive German standard addressing ship impacts was (DIN, 2003), which was later replaced by (*EN 1991-1-7*, 2025). The ship impact provisions of (DIN, 2003) have been incorporated into the European standard in practically identical form. These standards introduced general load assumptions and procedures for ship-bridge interactions. Unlike earlier empirical approaches, the design values for ship impacts in these standards were derived using probabilistic modelling, reflecting the long-standing tradition of risk-based methodologies in maritime engineering (Kunz, 2006).

A comprehensive presentation of code regulations of the ship impact load case has so far been largely absent from the international literature. Hence, the underlying background and original reasoning have become difficult to trace over time (AASHTO, 2009). At the same time, it should be noted that some relevant contributions on this topic do exist in German, in particular by the second author of this paper (Kunz, 2006, 2013, 2024).

2.3. Challenges of the present

Nowadays, the question of how to deal with existing bridges is increasingly coming to the fore - especially those that have been designed according to outdated standards. There is a great desire to preserve existing structures without massive interventions. There are good reasons for this in terms of economic efficiency, climate protection and monument protection (Vrouwenvelder et al., 2024).

Hence, a method is needed to select a consistent ship impact force level for a defined remaining assessment period. The method that will be presented here was originally developed by the *German Federal Waterways Engineering and Research Institute* (BAW, 2013). To maintain clarity and focus, the subsequent discussion is limited to impacts involving inland waterway vessels.

3. Impact load on new bridges according to Eurocode

3.1. Regulations

When assessing the impact of a ship on a pier, both frontal and lateral impact must be considered, though they do not act simultaneously. Indicative values for the forces to be applied can be found in (*EN 1991-1-7*, 2025), as illustrated in Figure 1. The forces to be applied are based on the displacement mass of the ships in traffic, which in turn depends on the waterway class.

CEMT class	Displacement mass m [t]	Frontal impact force F_{dx} [kN]	Lateral impact force F_{dy} [kN]
I	200 to 400	2000	1000
II	400 to 650	3000	1500
III	650 to 1000	4000	2000
IV	1000 to 1500	5000	2500
Va	1500 to 3000	8000	3500
Vb	3000 to 6000	10000	4000
VIa	3000 to 6000	10000	4000
VIb	6000 to 12000	14000	5000
VIc	10000 to 18000	17000	8000
VII	14000 to 27000	20000	10000

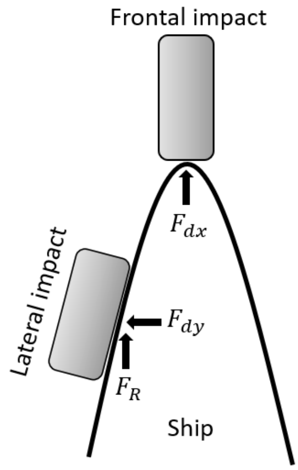

Figure 1: Indicative values for the dynamic forces due to ship impact on inland waterways, according to (*EN 1991-1-7*, 2025), table revised

These impact loads were determined using a probabilistic approach, taking into account prevailing distributions for travel paths, ship sizes, speeds, and other relevant factors (Kunz, 2006). The specified values apply only to locations that are not considered accident blackspots.

The provided forces F_{dx} and F_{dy} include the effect of hydrodynamic added mass, i.e. the influence of the surrounding water on the moving vessel. In (*EN 1991-1-7*, 2025), F_{dx} and F_{dy} are termed “dynamic” impact forces. This terminology should be interpreted with some care. What is meant is that the values represent forces derived from the dynamic collision process of the striking vessel, more precisely the maximum transient force during the impact event, but they do not by themselves constitute a complete time-dependent action model for the impacted bridge.

In fact, (*EN 1991-1-7*, 2025) states that, in the absence of a dynamic analysis of the impacted structure, the tabulated values should be multiplied by an appropriate dynamic amplification factor, since the values include the dynamic effects in the colliding object, but not in the structure. The recommended factors are 1.3 for frontal impact and 1.7 for lateral impact. A genuine dynamic analysis of the bridge requires an explicit load–time history, which is only introduced later in the form of half-sine or trapezoidal pulses.

Accordingly, in practical design, the tabulated Eurocode values are best understood as dynamically derived design forces that are commonly used as equivalent design loads for the bridge. If no explicit time-dependent structural analysis is carried out, the above dynamic amplification factors provide a means of approximately accounting for the structural dynamic response.

As an alternative to the tabulated values shown in Figure 1, (*EN 1991-1-7*, 2025) provides equations for calculating impact forces, allowing for the consideration of specific impact speeds and exact mass. The provisions referred to as "Advanced ship impact analysis for inland waterways" in (*EN 1991-1-7*, 2025) are derived from the kinetic energy of the ship. If no site-specific information is available a design speed of 3 m/s, increased by the flow velocity, is proposed. For the mass, the use of the mean value of the mass for the relevant ship class, see Figure 1, is recommended.

For elastic deformations (if $E_{def} \leq 0.21$ MNm), the dynamic impact force is calculated according to (*EN 1991-1-7*, 2025), with E expressed in [MNm] and F in [MN]:

$$F_{dyn,el} = 10.95 \cdot \sqrt{E_{def}} \tag{1}$$

For plastic deformations (if $E_{def} > 0.21$ MNm), the dynamic impact force is calculated according to (*EN 1991-1-7*, 2025):

$$F_{dyn,pl} = 5.0 \cdot \sqrt{1 + 0.128 \cdot E_{def}} \tag{2}$$

In the case of a frontal impact, the deformation energy E_{def} corresponds to the kinetic energy of the vessel at the impact:

$$E_{kin} = \frac{m_{eff}}{2} v^2 = \frac{c \cdot m}{2} v^2 \tag{3}$$

whereas in the case of lateral impact with an angle $\alpha < 45°$, an impact with sliding friction is assumed and the deformation energy with (*EN 1991-1-7*, 2025):

$$E_{def} = E_{kin}(1 - \cos\alpha) \tag{4}$$

may be used. The factor for the hydrodynamic mass c should be set to 1.1 for the frontal impact and 1.4 for the lateral impact. The mechanical model is based on (Meier-Dörnberg, 1983).

Figure 2 illustrates the dynamic impact force, calculated using Equations (1), (2), and (3), as a function of mass and impact velocity. A transition between the elastic and plastic ranges is evident at $F_{dyn} = 5$ MN, where a kink can be observed. The criterion for elastic impact, $E_{def} \leq$ 0.21 MNm, previously introduced, can also be expressed in terms of force. The equivalence of these two criteria becomes clear when the energy-based condition, $E_{def} = 0.21$ MNm, is substituted into Equation (1):

$$F_{dyn,el} = 10.95 \cdot \sqrt{0.21} = 5.0 \text{ MN} \tag{5}$$

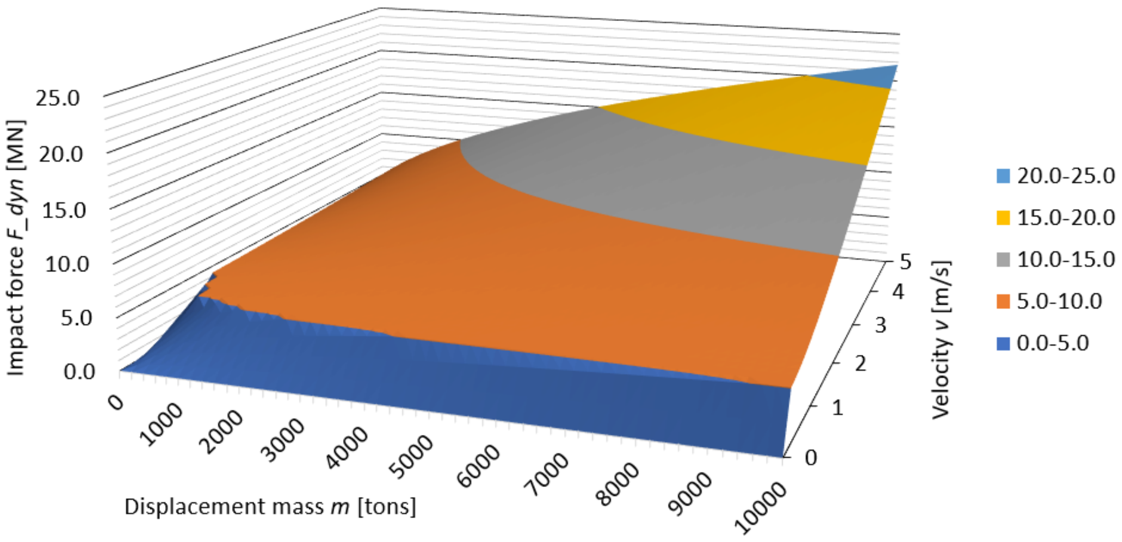

Figure 2: Dynamic impact force according to the equations in section C.6.2 "Advanced ship impact analysis for inland waterways" in (*EN 1991-1-7*, 2025). An elastic impact is assumed for $F_{dyn} < 5$ MN and a plastic impact for $F_{dyn} > 5$ MN.

Figure 3 compares the tabulated forces from Figure 1 with the dynamic impact forces calculated using Equations (1), (2), and (3) for frontal impacts. The diagram is based on the following assumptions: Since ship masses are provided in class-based ranges, the respective mean values were used. For the calculated values, a velocity of 3 m/s was assumed, as recommended by Eurocode, and the hydrodynamic mass factor was set to 1.1.

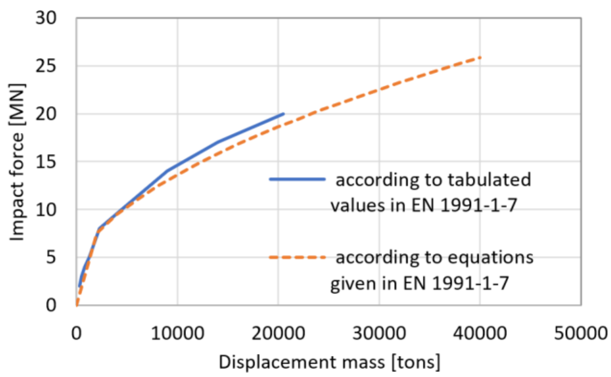

Figure 3: Dynamic impact forces for inland waterways according to (*EN 1991-1-7*, 2025): Comparison of tabulated values with values resulting from equations

It can be seen that the tabulated impact forces, given here in Figure 1, are almost identical to those resulting from the equations in section C.6.2 of (*EN 1991-1-7*, 2025). However, the equations can be used to perform more precise calculations, as the impact speed, mass of the ship and the factor for the hydrodynamic mass can be individually selected.

It should be noted that some national annexes to (*EN 1991-1-7*, 2025) provide more precise impact velocities. Specifically, the German National Annex (DIN EN 1991-1-7/NA,

2019) specifies velocities that increase with higher CEMT classes. Additional adjustment factors are also specified, particularly the reduction of impact force depending on the distance of the pier from the navigation channel. A comparable provision exists in the AASHTO code, where the design impact speed is reduced for piers located farther from the navigation channel.

3.2. Comparison Eurocode – AASHTO

A direct comparison between the impact forces specified for inland vessels in Eurocode and those prescribed by the AASHTO code (AASHTO, 2009) is not straightforward. The two frameworks differ in terms of vessel types, navigation conditions and underlying assumptions, and, most importantly, in their verification formats. Nevertheless, some meaningful comparisons can be made. Table 1 indicates that the Eurocode generally specifies higher impact forces, particularly for larger barge formations. One reason is that, in the AASHTO guidelines, the mass of barges coupled in parallel is neglected in the impact force calculation, as these barges are assumed to quickly break away from the formation upon impact (AASHTO, 2009).

Eurocode			AASHTO
CEMT class - barge tow description	Displacement mass	Impact force	Impact force
	[t]	[kN]	[kN]
Class Vb - tow + 2 barges (2 x 1)	3000-6000	10000	8229
Class VIa - tow + 2 barges (1 x 2)	3000-6000	10000	10097
Class VIb - tow + 4 barges (2 x 2)	6000-12000	14000	10097
Class VIc - tow + 6 barges (2 x 3)	10000-18000	17000	11943
Class VII - tow + 9 barges (3 x 3)	14000-27000	20000	11943

Table 1: Comparison of impact forces of “barges” in (AASHTO, 2009) with impact force for “dynamic forces due to ship impact on inland waterways” in (*EN 1991-1-7*, 2025)

Figure 4 shows the impact force of inland vessels according to the equations in Eurocode, which are based on the Meier-Dörnberg investigations (Meier-Dörnberg, 1983), and AASHTO. The factor for the hydrodynamic mass was set to 1.1 for Eurocode as for AASHTO. In both diagrams, a kink can be recognised at 5 and 6 MN respectively, which separates the elastic from the plastic range. It can also be clearly seen that the values from the Eurocode equations are considerably higher in the plastic range.

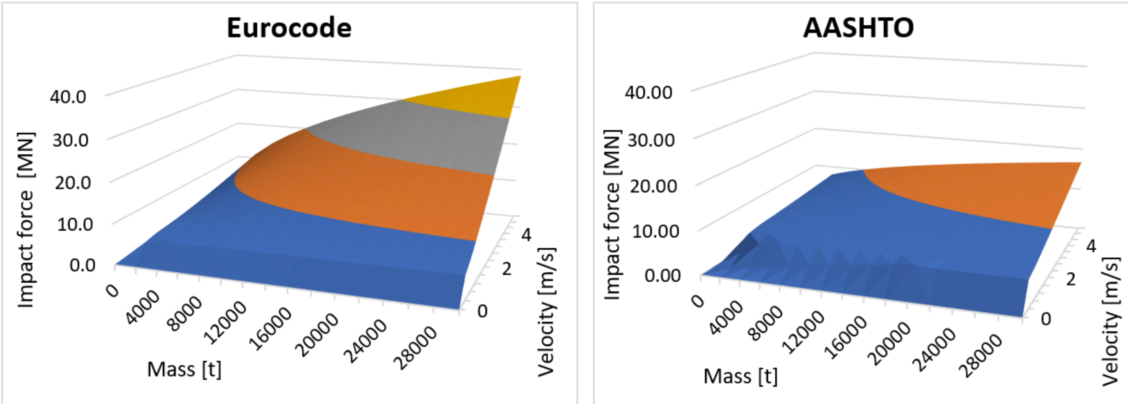

Figure 4: Comparison of impact force for "dynamic forces due to ship impact on inland waterways" in (*EN 1991-1-7*, 2025) with impact forces for "barges" in (AASHTO, 2009)

It should be noted that the forces specified in the AASHTO provisions are derived from a time-averaged impact force, whereas the Eurocode design force corresponds to the maximum transient force during the impact event, cf. Section 2.1. In addition, the Eurocode tabulated forces generally should be multiplied by a dynamic amplification factor if no explicit dynamic analysis of the impacted structure is performed. EN 1991-1-7 recommends amplification factors of 1.3 for frontal impact and 1.7 for lateral impact. This further increases the difference between forces obtained according to the Eurocode and those obtained according to AASHTO.

A further important difference concerns the verification format. Although the Eurocode approach is based on probabilistic considerations, as discussed in Sections 3.5 and 3.6, these are not applied directly by the user in the structural verification. Instead, the designer is provided with representative design actions for the verification of the structure, as described in Section 3.1. By contrast, AASHTO formulates the ship impact problem more explicitly as a risk problem. The annual frequency of bridge collapse is estimated as the product of several factors, including vessel traffic frequency, aberrancy probability, geometric probability of collision, probability of collapse given collision, and adjustment factors accounting for waterway conditions and protective measures. The resulting annual frequency of collapse is then compared with acceptance criteria, which depend on the importance of the bridge. Thus, under the AASHTO framework, the user performs a probabilistic assessment for the individual bridge under consideration.

This difference is important for the interpretation of the two approaches. AASHTO aims to estimate the annual collapse frequency of a particular bridge exposed to vessel impact. This requires not only an action model, but also assumptions regarding the conditional probability of collapse given impact. In (AASHTO, 2009), the probability of collapse is estimated using a highly simplified procedure. Its background lies in older Japanese research on ship-ship collisions (Fujii & Shiobara, 1978), rather than in direct studies of ship impact on bridges. The resulting collapse probability should therefore be understood as a pragmatic approximation, not as mechanically rigorous estimates of bridge failure probability.

The Eurocode approach avoids this additional step by prescribing nominal impact forces on the load side. It therefore separates the derivation of the accidental action from the verification of structural resistance. This distinction is central to the present paper, which is concerned with the action side of the ship impact problem and not with the calculation of bridge-specific collapse probabilities.

In addition to this conceptual distinction between the two verification formats, it is important to clarify the scope of the impact scenario considered here. The regulations discussed above refer to the impact of a vessel's bow on a bridge pier. Both codes also contain provisions for impacts on the bridge deck; however, such impacts can only occur through parts of the vessel above the hull, such as the deckhouse or other superstructures, and therefore generally result in significantly lower forces. This aspect is not discussed further in the present paper.

The following subsections outline the derivation of the impact loads specified in the Eurocode. These loads are determined through a combination of mechanical-experimental investigations and probabilistic analyses. In the present paper, "new bridges" denotes bridges in the design stage, assessed for their full design working life, in contrast to existing bridges assessed for a reduced remaining working life. The systematic derivation of impact loads for new bridges also forms the basis for the methodology introduced later, which focuses on reducing impact loads for bridges with a shorter remaining working life.

3.3. Theoretical background for ship impact

Probabilistic considerations are fundamental to any risk-based assessment. Designing a structure to resist every conceivable impact – regardless of how improbable – would be neither practical nor economically viable. A consistent derivation of impact loads associated with a defined probability of occurrence therefore forms the cornerstone of a rational risk-based design approach. This is achieved by linking a load model with a collision model (Kunz, 2013), allowing ship impact forces to be determined for specified occurrence probabilities. The key outcomes of this methodology have been incorporated into Eurocode (*EN 1991-1-7*, 2025). The present paper provides the first comprehensive presentation of this concept, thereby filling an important gap in the literature.

3.4. Load model

The Eurocode load models and the corresponding impact forces, presented in section 3.1, are derived from experimental findings on the load-deformation behaviour of vessels discussed in section 2.2. The general validity of these relationships was later confirmed by numerical (Biehl et al., 2007; Sha & Hao, 2012; Song & Wang, 2019; Wang & Morgenthal, 2018), as well as empirically-analytical (Pedersen et al., 2020) analyses. Since the Eurocode load model was briefly outlined in section 3.1 and will not be further detailed here.

3.5. Collision model

The total annual impact rate λ, see Figure 5; cf. (Kunz, 2013), can be expressed as:

$$\lambda = \sum_{i=1}^{n} N_i \int_{s_0}^{s_1} \nu(s) P_{1,i}(s) P_{2,i}(s)\, ds \qquad (6)$$

where
N_i = annual number of passages of vessel class i,

s = longitudinal coordinate along the navigational path.

The integration limits s_0 and s_1 define the length of the approach and passage zone where aberrant motion can result in a collision. The route-related aberrancy rate per unit distance is defined as:

$$\nu = \frac{d\Lambda(s)}{ds} \tag{7}$$

It represents the local probability density of course deviations caused by human or technical failures (e.g., rudder malfunction). This factor cannot be determined analytically and is therefore usually estimated on the basis of accident or failure statistics. The geometric hit probability is defined as:

$$P_{1,i}(s) = G_{\varphi_i}\left(\varphi_{2,i}(s)\right) - G_{\varphi_i}\left(\varphi_{1,i}(s)\right) \tag{8}$$

It represents the probability that the course deviation φ falls within the angular range that would result in a collision with the pier under consideration. Here, G_{φ_i} denotes the cumulative distribution function (CDF) of the course deviation φ, while g_φ in Figure 5 is the corresponding probability density function (PDF). The non-avoidance probability is defined as:

$$P_{2,i}(s) = 1 - G_{S_i}(s) \tag{9}$$

where $G_{S_i}(s)$ is the CDF of the stopping distance S. The term $1 - G_S(s)$ thus expresses that an impact occurs when the available distance to the pier is smaller than the required stopping distance. The non-avoidance probability is primarily influenced by the stopping behaviour of the ships.

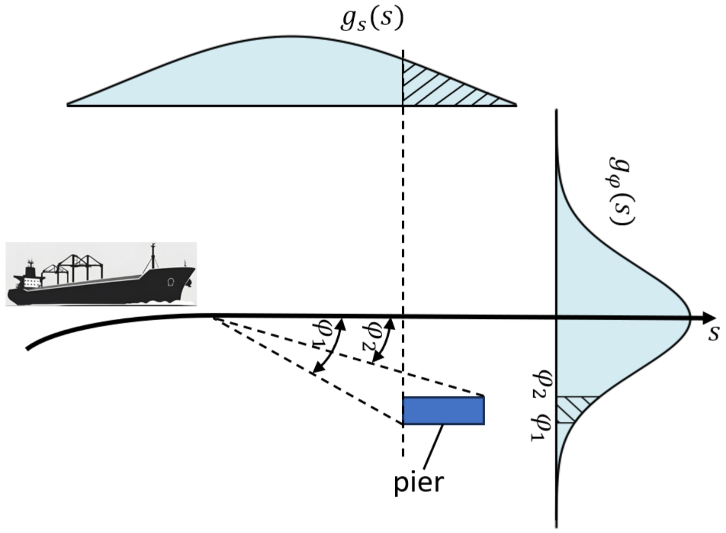

Figure 5: Collision model for waterways

3.6. Linking the load model with the collision model

The collision model introduced in section 3.5 provides the annual rate λ at which vessels impact a bridge pier. To derive action levels for structural verification, this collision rate must be combined with a probabilistic model of the force generated by an individual impact.

The occurrence of ship impacts is represented by a homogeneous Poisson process with a constant annual collision rate λ cf. (Kunz, 2013). Each collision is assigned an impact-force value F, described by the cumulative distribution function

$$G_F(F^*) = \mathbb{P}(F \leq F^*) \ , \tag{10}$$

where F^* denotes a specified force threshold. The probability that the force generated by an individual collision exceeds this threshold is therefore

$$p_{exc}(F^*) = \mathbb{P}(F > F^*) = 1 - G_F(F^*) \ . \tag{11}$$

The impact-force values associated with individual collisions are assumed to be independent and identically distributed and independent of the collision times. Under these assumptions, the collision process may be interpreted as a stationary marked Poisson process.

The probability that the force threshold F^* is exceeded at least once within an exposure period t is consequently

$$P_{exc}(F^*, t) = 1 - \exp\{-\lambda t[1 - G_F(F^*)]\} \ . \tag{12}$$

The return period $T_R(F^*)$ is defined as the mean time interval between successive exceedances of the force threshold F^*:

$$T_R(F^*) = \frac{1}{\lambda[1 - G_F(F^*)]} \ . \tag{13}$$

Rearranging Equation (13) gives

$$\lambda T_R(F^*) = \frac{1}{1 - G_F(F^*)} \ . \tag{14}$$

Substitution of Equation (13) into Equation (12) yields the direct relationship between the return period and the probability of at least one exceedance during an exposure period t:

$$P_{exc}(F^*, t) = 1 - \exp\left[-\frac{t}{T_R(F^*)}\right] \ . \tag{15}$$

Equations (13)–(15) concern action exceedances and their return periods. They do not represent the probability of structural failure.

Conversely, for a known collision rate λ and a specified return period T_R, the corresponding impact force threshold may be obtained from

$$F^* = G_F^{-1}\left(1 - \frac{1}{\lambda T_R}\right) \ . \tag{16}$$

Equation (16) establishes the link between the collision model, the impact force distribution, and an action level associated with a specified return period. It also provides the basis for adapting the impact force level to a reduced remaining working life, as discussed in Section 4.

Calculations performed by the *Federal Waterways Engineering and Research Institute* (BAW) for an existing bridge waterway system resulted in the following collision rates for piers of typical bridges: $\lambda_{frontal}$ = 0.0115 [1/a] for frontal impacts, and $\lambda_{lateral}$ = 0.00276 [1/a] for lateral impacts (Kunz, 2013). The distribution of the impact loads $G_F(F^*)$ was determined by recording the existing ship traffic and using Equations (1) and (2) as mechanical model and treating ship mass, velocity, impact angle, and a "severity factor" as random variables. In this context the term "severity factor" is used as a practical adjustment factor between theoretical risk models and empirical reality, especially to account for hard-to-model influences like human error, unknown interactions, or unpredictable externalities. As the underlying calibration data are not reproduced here, the resulting force distributions are treated as published reference distributions from Kunz (2013).

Figure 6 (left) shows the frequency distribution of impact forces for waterway class Vb, which represents about 70 % of Germany's inland waterways. Analytical probability density functions were fitted to the histogram, then converted into cumulative distribution functions $G_F(F^*)$. Substituting these into Equation (16) yields the corresponding one-sided, dimensionless impact-load distribution functions, shown in Figure 6 (right).

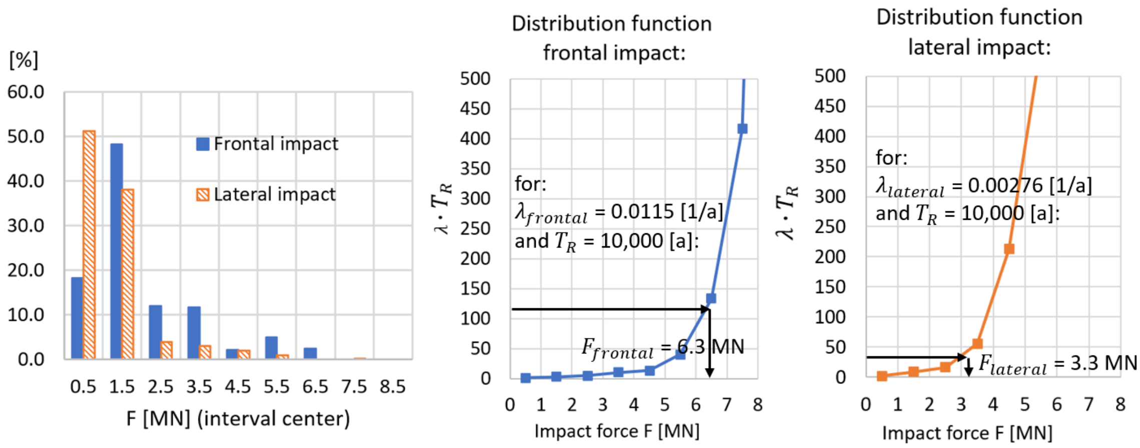

Figure 6: Left: Frequency distribution for ship impact forces for waterway class Vb, frontal impact and lateral impact, according to (Kunz, 2013). Right: Impact load distributions for frontal impact and lateral impact, based on (Kunz, 2013)

In simple terms: for a given annual impact rate λ and an associated distribution of impact forces, the force (frontal or lateral) whose threshold is exceeded, on average, once per return period T_R can be determined using Equation (16). For a reference value of T_R = 10,000 years (*EN 1991-1-7*, 2025), the calculated values are:

- for frontal impact: $F_{frontal}$ = 6.3 MN
- for lateral impact: $F_{lateral}$ = 3.3 MN

A comparison with the tabulated values in the Eurocode (*EN 1991-1-7*, 2025) for waterway class Vb, as shown in Figure 1, reveals that the stipulated impact loads are higher, at 10 MN and 4 MN, respectively. One reason for this difference is the expectation of increasing ship traffic loads in the future, which should be considered in the design of new bridges. In addition, empirical adjustments, the need for geographical generalisation, and a tendency toward conservative rounding likely also contributed to the higher specified loads.

The formulation above assumes that the collision rate λ and the conditional force distribution $G_F(F)$ remain constant during the period under consideration. If substantial changes in vessel traffic, vessel dimensions, navigation conditions, water levels, protective measures, or navigation technology are expected, the collision rate and impact-force distribution should be reassessed using updated, preferably site-specific information.

4. Load reduction for existing bridges

4.1. General rationale and scope

The assessment of existing structures differs fundamentally from the design of new structures. Existing structures have a known construction history and an observable condition, but measures required to increase their safety are generally more costly, disruptive, and resource-intensive than equivalent measures incorporated during the design and construction of a new structure. Consequently, a differentiated treatment of existing and new structures is widely recognised in standards and the scientific literature (Diamantidis & Sýkora, 2019; Vrouwenvelder & Scholten, 2010).

Standards and background documents recognise that the assessment of existing structures may require a more differentiated treatment than the design of new structures. EN 1990 establishes the general requirement for an appropriate level of reliability over the intended working life (EN 1990, 2023), while ISO 13822 relates the greater differentiation applicable to existing structures to economic, social, and sustainability considerations (ISO, 2010). The recent Eurocode reliability background report likewise identifies reliability updating and reduced target levels as characteristic issues in the assessment of existing structures (Vrouwenvelder et al., 2024).

Existing recommendations mainly address structural reliability, partial factors, and persistent design situations. For example, Allaix et al. (2016) derive target reliability levels and partial-factor methods for existing concrete structures from economic optimisation and individual-risk considerations. However, these approaches do not provide a directly applicable procedure for adapting the nominal level of *accidental* actions, such as ship impact, to a finite remaining working life.

For accidental actions, the duration of exposure is directly relevant. A bridge that is intended to remain in operation for a limited additional period is exposed to the hazard for less time than a new bridge designed for its full working life. This provides the rationale for relating the accidental-action level to the defined remaining assessment period, while all structural safety requirements remain applicable.

The method proposed below is confined to the action side of the assessment. It adjusts the return period and corresponding ship-impact force so that the nominal cumulative exceedance probability of the selected action level is preserved over the relevant exposure period. It does not determine the failure probability of the bridge. Structural condition, resistance, deterioration, dynamic response, robustness, and failure consequences must therefore be considered separately. The remaining working life is understood as the authorised period of continued operation covered by the assessment; continued operation beyond this period requires a renewed assessment.

4.2. Adaptation of the return period to the remaining working life

The impact-force levels specified for new bridges are associated with a reference return period and a corresponding design working life. For existing bridges, however, the intended period of continued operation may be considerably shorter than the design working life assumed for a new structure. This raises the question of whether the same impact-force level should be applied irrespective of the duration for which the bridge remains exposed to ship traffic.

The proposed approach addresses this question by adjusting the return period of the ship impact force to the remaining working life of the bridge. The nominal probability of exceeding the corresponding impact force level during the considered period remains equal to that associated with a new bridge over its full design working life. The approach is a practical and rational procedure to adjust accidental forces of existing structures with reduced working life.

Let F_{new} denote the reference impact force level for a new bridge. This force is associated with the return period $T_{R,new}$ and the design working life t_d. For an existing bridge, F_{rem} denotes the impact force level assigned to the remaining working life t_{rem}, with the corresponding return period $T_{R,rem}$.

The proposed condition is

$$P_{exc}(F_{rem}, t_{rem}) = P_{exc}(F_{new}, t_d) \quad . \tag{17}$$

Thus, the nominal probability that the respective impact-force level is exceeded at least once is kept constant over the relevant exposure period.

Using Equation (15), Equation (17) can be written as

$$1 - exp\left(-\frac{t_{rem}}{T_{R,rem}}\right) = 1 - exp\left(-\frac{t_d}{T_{R,new}}\right) \quad . \tag{18}$$

Since the exponential function is one-to-one, it follows that

$$\frac{t_{rem}}{T_{R,rem}} = \frac{t_d}{T_{R,new}} \quad , \tag{19}$$

and therefore

$$T_{R,rem} = T_{R,new} \frac{t_{rem}}{t_d} \quad . \tag{20}$$

Equation (20) shows that the return period assigned to the existing bridge is proportional to its remaining working life.

For ship impact, (*EN 1991-1-7*, 2025) and (DIN EN 1991-1-7/NA, 2019) refer to a nominal return period of $T_{R,new}$ = 10,000 years, which corresponds to an annual exceedance probability of 10^{-4}. Taking a design working life of t_d = 100 years, an existing bridge with a remaining working life of 50 years is assigned a return period of

$$T_{R,rem} = 10{,}000\frac{50}{100} = 5{,}000 \text{ years} \quad . \tag{21}$$

The corresponding probabilities of at least one exceedance are identical:

$$1 - exp\left(-\frac{100}{10{,}000}\right) = 1 - exp\left(-\frac{50}{5{,}000}\right) \approx 0.00995 \quad . \tag{22}$$

The adjustment therefore preserves the cumulative exceedance probability of the accidental action over the respective exposure period. The force corresponding to $T_{R,rem}$ must subsequently be obtained from the impact-force distribution.

The term remaining working life is used here to denote a defined period of intended continued operation for which the bridge is reassessed. Continued operation beyond this period requires a renewed assessment. The derivation assumes that the annual collision rate and the conditional impact-force distribution remain stationary over the periods being compared. Where substantial changes in vessel traffic, vessel dimensions, navigation conditions, protective measures, or other relevant circumstances are expected, the ship impact hazard should be recalculated using updated information.

4.3. Derivation of remaining-life-dependent ship impact forces

Equation (20) defines the return period $T_{R,rem}$ associated with a specified remaining working life t_{rem}. The corresponding ship impact force is determined from the collision rate and the probabilistic impact-force distribution introduced in Section 3.6.

For an annual collision rate λ and a cumulative distribution function $G_F(F)$ of the force generated by an individual collision, Equation (16) gives the force threshold associated with a specified return period:

$$F(T_R) = G_F^{-1}(1 - \frac{1}{\lambda T_R}) \quad . \tag{23}$$

Accordingly, the impact-force level assigned to the remaining working life of an existing bridge is

$$F_{rem} = G_F^{-1}(1 - \frac{1}{\lambda T_{R,rem}}) \quad . \tag{24}$$

Substituting Equation (20) gives

$$F_{rem} = G_F^{-1}(1 - \frac{t_d}{\lambda T_{R,new} t_{rem}}) \ . \quad (25)$$

Equation (25) shows that the impact force is not reduced in direct proportion to the remaining working life. The remaining working life first determines the adjusted return period according to Equation (20). The corresponding force is then obtained from the generally nonlinear impact-force distribution $G_F(F)$.

For practical application, the resulting force may be expressed relative to the reference force for a new bridge:

$$\eta_F(t_{rem}) = \frac{F_{rem}}{F_{new}} \ , \quad (26)$$

where

$$F_{new} = G_F^{-1}(1 - \frac{1}{\lambda T_{R,new}}) \ . \quad (27)$$

The factor η_F represents the ratio between the impact force associated with the remaining working life and the force associated with the reference return period for a new bridge.

The procedure was originally introduced by the German Federal Waterways Engineering and Research Institute (BAW) for the assessment of existing bridges. Using the annual collision rates and impact-force distributions presented in Section 3.6, the return periods and corresponding forces listed in Table 2 are obtained.

For the reference case of a new bridge, a design working life of 100 years and a nominal return period of 10,000 years are adopted. As shown in section 3.6, the distributions give impact forces of approximately 6.3 MN for frontal impact and 3.3 MN for lateral impact. For example, a remaining working life of 50 years corresponds, according to Equation (20), to a return period of 5,000 years. The associated forces obtained from the same distributions are approximately 5.75 MN for frontal impact and 2.3 MN for lateral impact. The resulting force ratios are therefore

$$\eta_{F,frontal} = \frac{5.75}{6.3} \approx 0.91$$

and

$$\eta_{F,lateral} = \frac{2.3}{3.3} \approx 0.70 \ .$$

The substantially different reduction factors result from the different shapes of the frontal and lateral impact-force distributions. The return period is reduced by the same proportion for both impact directions, whereas the corresponding force reduction is governed by the respective distribution.

The impact-force distributions used in Table 2 were derived for representative conditions on German inland waterways. They do not reproduce the tabulated Eurocode forces exactly. For waterway class Vb, the forces obtained from the distributions for a return period of 10,000 years are 6.3 MN and 3.3 MN, whereas EN 1991-1-7 specifies tabulated values of 10 MN and 4 MN for frontal and lateral impact, respectively.

For application to the Eurocode action values, the relative force factors from Equation (26) may be applied to the corresponding reference forces:

$$F_{EC,rem} = \eta_F(t_{rem})\, F_{EC,new}\ , \tag{28}$$

where $F_{EC,new}$ is the Eurocode impact force applicable to a new bridge and $F_{EC,rem}$ is the adapted force for the existing bridge. This transfer assumes that the relative variation of the relevant force quantiles with return period, as obtained from the representative distributions, is also applicable to the more generalised Eurocode action model.

In the BAW guideline (BAW, 2013), a pragmatic lower bound of 40% of the reference lateral impact force was imposed. Additionally, we recommend applying adjustment factors only for remaining working lives of at least 10 years, as regulations for shorter time spans are not practical.

t_{rem} [a]	T_R [a]	$\lambda_{frontal} \cdot T_R$ [-]	$F_{frontal}$ [MN]	$\eta_{F,frontal}$ [%]	$\lambda_{lateral} \cdot T_R$ [-]	$F_{lateral}$ [MN]	$\eta_{F,lateral}$ [%]
100	10000	115	6.3	100	27.6	3.3	100
75	7500	86	6.0	95	20.7	2.9	88
50	5000	58	5.75	91	13.8	2.3	70
25	2500	29	5.3	84	6.9	1.2	36
10	1000	12	4.4	70	2.8	0.63	19

Table 2: Dynamic frontal and lateral impact loads as a function of the remaining working life, cf. (Kunz, 2013)

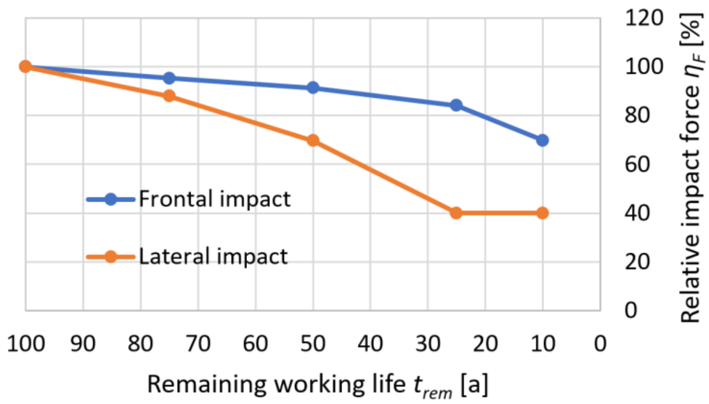

Figure 7: Proposal for reduction of vessel impact forces, adopted from (BAW, 2013)

The adapted force remains subject to all other provisions governing the mechanical representation of ship impact. In particular, dynamic amplification factors, load application areas, impact directions, accompanying actions, and robustness requirements remain unaffected. The procedure modifies only the reference level of the ship impact force as a function of the defined remaining working life. It does not account for the resistance, deterioration state, structural response, or failure consequences of the bridge, which must be considered separately in the structural verification. In principle, the procedure is also applicable to accidental actions other than ship impact.

5. Conclusions

This study has revisited the derivation of ship impact loads as defined in the Eurocode and examined their historical development, with particular emphasis on probabilistic aspects. The analysis confirms that the Eurocode approach – linking a mechanical load model with a probabilistic collision model – provides a consistent and rational framework for assessing accidental ship impacts on bridges. The presentation of this concept in English helps to close an existing gap in the literature.

Building on this foundation, a consistent method for the assessment of existing bridges has been presented. By preserving the nominal cumulative exceedance probability of the selected impact force over the relevant exposure period, the proposed procedure provides a consistent hazard-side basis for reassessing existing bridges with a finite remaining working life. The approach was developed for the ship-impact load case. The concept may also be applicable to other accidental actions, provided that occurrence can be represented by an appropriate stochastic event model and that the action intensity can be related to a return period.

The life-dependent reduction concept ensures that reliability remains consistent with the intended service duration of the structure while supporting a more efficient and sustainable management of infrastructure assets. It allows engineers and authorities to allocate resources strategically, maintaining overall network safety. In practical terms, the following aspects should be considered when reassessing existing bridges for ship impact:

- Site-specific conditions: Incorporate local factors such as waterway geometry, pier location, and vessel traffic composition, as e.g. specified in the German National Annex.
- Remaining working life: Adapt design impact loads in accordance with the bridge's residual working life.
- Probabilistic reassessment: Where significant changes in navigation traffic, safety technology, such as automated navigation systems, a probabilistic recalculation should be performed using updated data.
- Accurate resistance data: Use measured or verified structural properties rather than nominal design values to minimise uncertainty.
- Design and enhancement of robustness: Ensure that structural robustness is explicitly considered in both design and rehabilitation measures.

Future research should continue to build on these points, further refining reliability-based assessment methods and promoting their straightforward practical implementation in bridge engineering.

Acknowledgement

This article is based on results with reference to the research project carried out at the request of the Federal Ministry for Transport, represented by the Federal Highway and Transport Research

Institute, under research project No. 15.0638/2017/FRB. The authors are solely responsible for the content.

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