Annual reliability requirements for bridges and viaducts

R. de Vries [1,2], R.D.J.M. Steenbergen [1,3], J. Maljaars [1,4]

[1] TNO, Department of Structural Reliability, Delft, the Netherlands

[2] Delft University of Technology, Faculty of Civil Engineering and Geosciences, Delft, the Netherlands

[3] Ghent University, Faculty of Engineering and Architecture, Ghent, Belgium

[4] Eindhoven University of Technology, Department of the Built Environment, Eindhoven, the Netherlands

In the assessment of existing structures, such as bridges and viaducts, reliability requirements are used to decide if a structure is sufficiently safe, even when subjected to degradation. The reliability requirements may be expressed in different ways but should ultimately result in similar reliability performance of the structure. Most of the current assessment rules follow from a maximum allowable failure probability, depending on the reliability class, within a fixed period of time (the reference period: e.g. 15, 30, 50 or 100 years). A reliability requirement expressed using a fixed reference period fits the design of new (to be built) structures, but it is problematic for existing or temporary structures. In the case of existing structures, when for instance the design life has passed, the assessment should be flexible with respect to the expected or desired remaining life. Other factors, such as deterioration or changing loads, may also call for an assessment with a smaller time period since the failure rate could increase with time which makes a reliability requirement for a longer period less useful. Using fixed reference periods therefore could lead to suboptimal solutions where the economy and environment are unnecessarily hurt. Reliability requirements formulated on an annual basis provide a solution to this problem. In this article, reliability analyses of several typical bridges are performed to quantify the requirements in such a way that assessments based on annual reliability result in performance similar to the current practice. Special attention is paid to minimising and highlighting the cases where the annual requirements may lead to a trend that breaks with the current reliability requirements.

Key words: Annual reliability, probabilistic assessment, existing structures, bridges, viaducts, time-dependence, Eurocode, individual risk, durability, fatigue

1 Introduction

1.1 Context

A large number of bridges and viaducts have passed or are close to their intended functional (design) life. The question arises if they can still be safely used for an extended period of time, or if they should be renovated or replaced. Reliability requirements are used to decide if a structure is sufficiently safe and durable. The projected extension of the design life is usually different (often smaller) than the design life used for an entirely new bridge. Therefore, the Eurocode (CEN, 2011) assessment rules that are based on a fixed period of time (e.g. 50 or 100 years) and partly motivated from an economic perspective are not suited for this application. The related costs and environmental impact of reaching the high level of reliability belonging to design reference periods are unnecessary for existing structures. This also holds for temporary structures that typically have shorter design life periods. Flexible reliability requirements are desired in which infrastructure (under degradation and/or increasing loads) may be assessed at any point in time. When structures are constantly monitored, and their reliability is regularly updated, better decisions can be made with regard to repairs and renovation. If the assessment were to be based on annual reliability requirements, this would provide such flexibility. Engineers are not required to perform a reliability calculation every year; instead, they are expected to demonstrate in their analysis that the annual reliability requirement is achieved throughout each year of the projected lifespan. Adopting annual reliability values simplifies requirements by avoiding various combinations of reliability indices and periods. The difference in reliability is immediately evident if expressed using the same 1-year reference period. As a result, a more transparent and durable approach to maintaining infrastructure assets can be put into practice.

In addition, annual reliability values are more consistent with regulations and acceptance criteria related to life safety (e.g. individual risk). In this way, the reliability requirements provide a clear boundary between what is acceptable and what is not – irrespective of the elapsed or remaining lifespan of a structure. Annual reliability requirements avoid situations where the failure probability over a longer fixed period is deemed acceptable but concentrated in just a few years. This can occur in structures experiencing significant deterioration or rapidly increasing loads. Using a lifetime based reliability requirement would lead to the situation in which low failure probabilities in the initial years would compensate for a societally unacceptably high probability of failure at the end of the lifespan. Therefore a shorter reference period, such as 1 year, is needed for these situations.

*1.2 **Objective***

In this article, several time-dependent reliability calculations are performed to obtain insight into the evolution of the annual reliability of structures. Given a certain structural type, the annual reliability can be 'back-calculated' for a certain combination of reliability index and reference period. As a result, appropriate values of the required annual reliability are suggested. In this way, the undesirable situation in which previously designed structures have insufficient reliability according to new requirements can be avoided.

This article considers two types of limit states: the ultimate limit state (excluding loss of structures due to fatigue) (ULS) and the fatigue limit state (FLS). Section 3 treats the ULS, whereas Section 4 treats FLS.

2 Reliability requirements

*2.1 **Eurocode***

EN 1990 (CEN, 2011) recommends reliability values, but they may be changed in the National Annex. Appendix B of EN 1990 provides minimum values for the reliability index β in the ultimate limit state. The reliability index is related to the probability of failure via:

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

where $\Phi(\cdot)$ is the cumulative distribution function of the standard normal distribution (i.e. mean equal to 0 and standard deviation equal to 1) and P_f is the failure probability. Table 1 provides the minimum reliability indices. The Eurocode refers to reliability classes (RC), which are linked to consequence classes (CC). The consequence and reliability classes are directly linked (i.e. the same) in almost all European National Annexes. Furthermore,

Table 1. Recommended minimum values of the reliability index β as per Eurocode

Reliability class	1 year	50 years
RC1	4.2	3.3
RC2	4.7	3.8
RC3	5.2	4.3

bridges assigned to RC3 are commonly required to adopt a minimum reference period of 100 years in the National Annex. This makes the reliability requirement for RC3 more stringent in comparison with the recommended 50-year reference period.

The reliability indices in Table 1 for 1 and 50 years are only approximately compatible if no correlation exists between failure happening in one year and another: $1 - P_{f,50} = (1 - P_{f,1})^{50}$ where $P_{f,y}$ indicates the failure probability for a reference period of y years. This is not a realistic assumption for the static resistance of the structure or its self-weight. Although the actual value of the resistance is indeed uncertain, the same structure is under consideration each year, i.e. the static resistance and self-weight are fully correlated in time. Only the time-dependent part of variable load may be considered to be uncorrelated each year, or every few years. The model uncertainty is also considered to be fully correlated in time. This means that a discrepancy exists between the minimum values of the reliability indices expressed for 50 years and those for 1 year for practical structures.

2.2 ***NEN 8700***

Reliability requirements for existing structures are usually less stringent compared to those for new structures in most European countries. This article refers to the Dutch standard for existing structures NEN 8700 (NEN, 2011), but similar principles may be found in other National standards.

NEN 8700 considers reliability levels related to human safety (leading to the minimum reliability requirements), but also economic optimization so that structures may be profited of for a longer period of time as compared to the design life. The minimum reliability values that follow from a human safety perspective are developed by considering the annual allowable individual risk. The individual risk is the probability of death of a person at a certain location or undertaking a certain action. This metric is often expressed on an annual basis. The values are obtained by relating the fatality rates estimated for each consequence class to the assumed maximum individual risk of 10^{-5} per year. The reasoning and elaboration may be found in Vrouwenvelder and Scholten (2010) and in Appendix 1 of the background document to NEN 8700 (TNO, 2011). Under the assumption of uncorrelated random variables (resistance, load, etc.) in time, the yearly values are related in NEN 8700 to a reference period of 15 years. The minimum reliability index values are listed in Table 2. However, the assumption of uncorrelated random variables in time is too simplistic for the same reason described in Section 2.1. Therefore, the values for 15 years in Table 2 should be viewed as indicative.

Table 2. NEN 8700 lower limits from a human safety perspective

Consequence class	1 year	15 years
CC1b	2.3	1.1
CC2	3.4	2.5
CC3	4.0	3.3

In light of the economic optimization of structures, NEN 8700 prescribes higher minimum reliability values in some cases than those when only human safety would be considered. Table 3 provides the minimum values of the reliability index in NEN 8700. It states that the remaining lifespan for renovation and disapproval must be chosen with a minimum of 15 years, except for CC1a. However, a remaining lifespan of 30 years, and thus adoption of a 30-year reference period, is recommended. Note that requiring the same reliability index (or probability of failure) over a longer period is more stringent – as the structure is required to last a longer period of time with the same probability of failure.

The reference period used to determine the characteristic value of the variable actions should be chosen equal to the remaining lifespan (and thus with a minimum of 15 years). Essentially, in case the projected remaining lifespan is smaller than 15 years, the assessment should proceed as if it were 15 years. If the remaining lifespan is larger than 15 years, the characteristic values of the time-variant loads should be based on a reference period equal to the remaining lifespan.

Table 3. Minimum values of the reliability index β as per NEN 8700

Conseq.	Reference	Renovation		Disapproval	
class	period [yr]	*wn*	*wd*	*wn*	*wd*
CC1a	1	2.8	1.8	1.8	0.8
CC1b	15	2.8	1.8	1.8	1.1
CC2	15	3.3 (3.1)	2.5	2.5	2.5
CC3	15	3.8 (3.6)	3.3 (2.6)	3.3	3.3

Table 3 gives two categories: the *disapproval* level is used to decide whether or not an existing structure meets the minimum requirements and the *renovation* level applies to the design of structural upgrading measures in case they are needed. Two subcategories are used: *wd* for structures where the wind dominates the loading (i.e. leading) and *wn* is for structures where this is not the case (i.e. accompanying). Requirements have been lowered for cases where the wind load is leading to bring their reliability performance in line with existing structures. The values in parentheses apply to structures for which a permit was granted on the basis of the Dutch Building Decree (Bouwbesluit) 2003 or before.

2.3 ROK/RBK

The commonly used reliability requirements of Rijkswaterstaat, the Dutch directorate-general for public works and water management, are listed in Table 4 (RWS, 2013). The *minimum* level is only used for a limited number of structures in the underlying road network. The *usage* level is introduced to assess existing structures built before 2012. The new level is used for existing structures built after 2012 and new to-be-built structures (in line with the Eurocode and the Dutch National Annex for structures in CC3).

Table 4. Minimum values of the reliability index β as per ROK/RBK requirements (RWS, 2013)

Level	Reliability index	Reference period [yr]
Minimum (disapproval structures secondary roads)	2.5	15
Usage	3.3	30
New	4.3	100

3 Ultimate limit state reliability

3.1 Traffic load

The statistical description of the resistance and actions on a bridge is based on report R1814 (TNO, 2012). The report provides the equivalent uniformly distributed load (EUDL) acting on the span of the bridge and the respective return period. The current article considers a single-span bridge with a span of 50 m and a *Slow* + *Fast* lane configuration (Table 5). In a *Slow* lane, the traffic is mainly made up of trucks, whereas in a *Fast* lane the traffic is primarily cars, vans, and occasionally trucks.

The values in the table followed from an analytical model where the bridge span was subdivided into fields, each with a certain probability of the presence of a truck. The model was calibrated by means of a traffic simulation that made use of data from WIM measurements on a motorway in The Netherlands (motorway A16) (TNO, 2012). The

Table 5. Equivalent uniformly distributed load q_{EUDL} for a span of 50 m

Return period	Rate λ [1/yr]	Fast [kN/m]	Slow [kN/m]	Slow + Fast [kN/m]	Slow + Slow [kN/m]
1 day	365	22	33	34	39
10 days	36.5	34	38	40	47
30 days	12.2	36	39	43	50
365 days	1	40	42	50	55
12500 years	8.00×10^{-5}	47	49	75	78
76500 years	1.31×10^{-5}	49	50	78	80

values provided here are also valid for similar European motorways with heavy traffic. The methodology followed hereafter is generally valid and can be applied to any traffic measurement database. An exceedance rate plot for the different configurations is obtained by plotting the average exceedance rate against the equivalent distributed load (Figure 1).

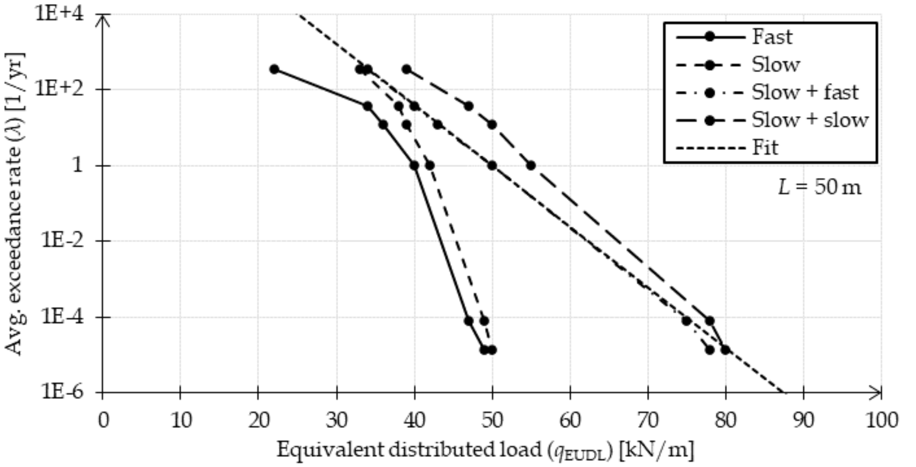

Figure 1. Exceedance rate plots for the different configurations on a bridge span of 50 m

A fit to the exceedance rate plot is produced for the Slow + Fast configuration using an exponential distribution (describing the tail of a Gumbel distribution), see Figure 1:

$$\lambda(q) = \exp(a - bq) \tag{2}$$

with the parameters

$a = 18.5, \qquad b = 0.37$

For a return period of 50 years, the corresponding Gumbel extreme value distribution (i.e. the maximum in 50 years) parameters are:

$$\mu = \frac{a - \ln\frac{1}{50}}{b} = 60.6 \text{ kN/m} \qquad \text{(location)}$$
$$\beta = 1 / b = 2.7 \qquad \text{(scale)}$$

3.2 ***Self-weight dominated (concrete bridge)***

3.2.1 Description

In the case studied, the self-weight is thought to contribute significantly to the total load acting on the bridge. This is a typical situation for a concrete bridge. A ratio of 70/30 between dead and live load is assumed for the selected span of 50 m.

The limit state function, indicating bending failure when $Z < 0$, is expressed in line with JCSS (2001) and report R1814 (TNO, 2012):

$$Z = R - S = R - (m_G G + m_T T) \qquad (3)$$

The capacity to resist loading R is modelled using a lognormal distribution. The self-weight acting on the bridge G is modelled using a normal distribution, and the traffic load T is modelled using a Gumbel distribution to describe the maximum in the considered reference period. Normally distributed model factors m_G and m_T are introduced acting on the self-weight and traffic load, respectively. These factors are assumed to be fully correlated (constant) in time. Other actions, such as snow, wind and temperature, are not considered.

The mean values of R and G have been chosen such that the lifetime-based reliability requirement is met (for example, the reliability index is 3.8 for a reference period of 50 years), respecting the 0.70/0.30 dead to live load ratio. Table 6 provides an overview of the random variables of the limit state function. The variation coefficients align with JCSS (2001) and report R1814 (TNO, 2012).

The failure probability P_f is defined as:

$$P_f = P(Z \leq 0) \qquad (4)$$

The failure probability for a 50-year reference period is obtained using Eq. (3) with T_{50}.

Table 6. Overview of random variables for concrete bridge

Symbol	Description	Distribution	Mean	Coeff. of variation [-]
R	Resistance (capacity)	Lognormal	(varies)	0.10
m_G	Model uncertainty for G	Normal	1.0	0.07
G	Self-weight	Normal	120 kN/m (70%)	0.07
m_T	Model uncertainty for T	Normal	1.04	0.17
T_1	Traffic load, max. 1 year (μ_T = 50.0, β_T = 2.7)	Gumbel	51.5 kN/m (30%)	0.067
T_{50}	Traffic load, max. 50 years (μ_T = 60.6, β_T = 2.7)	Gumbel	62.1 kN/m	0.056

The reliability is determined with the first-order reliability method (FORM) using the limit state function of Eq. (3) and the random variables of Table 6. The calculated influence coefficients following from the FORM analysis for 50 years are α_R = 0.78 and α_S = -0.62. These values are close to the defaults given in the Eurocode for a reference period of 50 years ($\alpha_R = 0.8$ and $\alpha_S = -0.7$).

The failure probability shows a time-dependent behaviour in these 50 years: each year contributes to the 50-year failure probability. The time dependence can be made visible through either statistical simulation or a number of repeated FORM analyses with increasing reference periods.

In this article, the conditional failure probability is used: this means that for the 2nd and the following years, the assessment only considers the structures that have survived up to that moment in time. For the 1st year, the conditional and unconditional failure probability are the same.

The conditional failure probability may be calculated as follows:

$$P_{\mathrm{f,\,cond},i} = \frac{P_{\mathrm{f},i} - P_{\mathrm{f},i-1}}{1 - P_{\mathrm{f},i-1}} \tag{5}$$

where $P_{\mathrm{f},i}$ is the cumulative failure probability up to and including the year i. In the first year, no conditionality holds and thus $P_{\mathrm{f,\,cond},i} = P_{\mathrm{f},1}$.

3.2.2 *Eurocode*

Figure 2 shows the annual reliability, defined in Section 3.2.1, as a function of time. Use is made of the Eurocode reliability requirements (Section 2.1). The top graph displays the development of the cumulative probability of failure (e.g. the results of the FORM analyses) and the bottom graph shows the annual conditional probability as obtained from Eq. (5). From the results, it becomes clear that the smallest annual reliability (conditional failure probability) is observed at the beginning of the lifespan of the structure. Thus, the probability of failure of a concrete bridge is larger in the first years compared to the later years of its life. Note that the assessment is made under the assumptions of no deterioration of the resistance and no traffic load increase in time. Additional figures displaying the development of the failure probability and failure rate with time may be found in TNO (2018).

Due to the correlation in time, the annual reliability calculated here is lower than the annual reliability indices provided in Table 1. The reliability values in Table 1 correspond to the average since for small failure probabilities it holds that $P_{\mathrm{f},1} \approx \frac{1}{50} P_{\mathrm{f},50}$. However, this average value it not directly useful because a disproportionate distribution of the failure probability over the lifespan may result in societally unacceptable low reliability at some point in time (Section 1.1). This is not typically a concern given the high (economically motivated) reliability values for new structures. But, for a consistent framework, the lowest annual reliability value is the most appropriate for assessing new and existing structures.

3.2.3 Other requirements

In case of assessment of existing structures, often the structure under consideration survived its entire original design life. Bridges are sometimes reassessed earlier in response to changing traffic intensity and updated insights with respect to the resistance. Typically, a bridge will first be assessed using the disapproval level. If it fails to meet the safety criterium, then the bridge should be taken out of service or mitigating measures should be taken. As an alternative, renovation of the bridge may be considered.

In the assessment of concrete structures and in absence of degradation, the elapsed service life is normally not considered explicitly in the current guidelines and standard. Following this standard practice, the structure is reassessed using the statistical properties of the structure as if it were to be built again. The fact that the structure has not failed in the elapsed service life (proven strength) is thus not considered, which is a conservative approach.

With regard to the ROK/RBK requirements, similar considerations hold with respect to the elapsed service life. In the same way as for the Eurocode, the lowest annual reliability has been calculated for the NEN 8700 and ROK/RBK requirements. Table 7 gives the results.

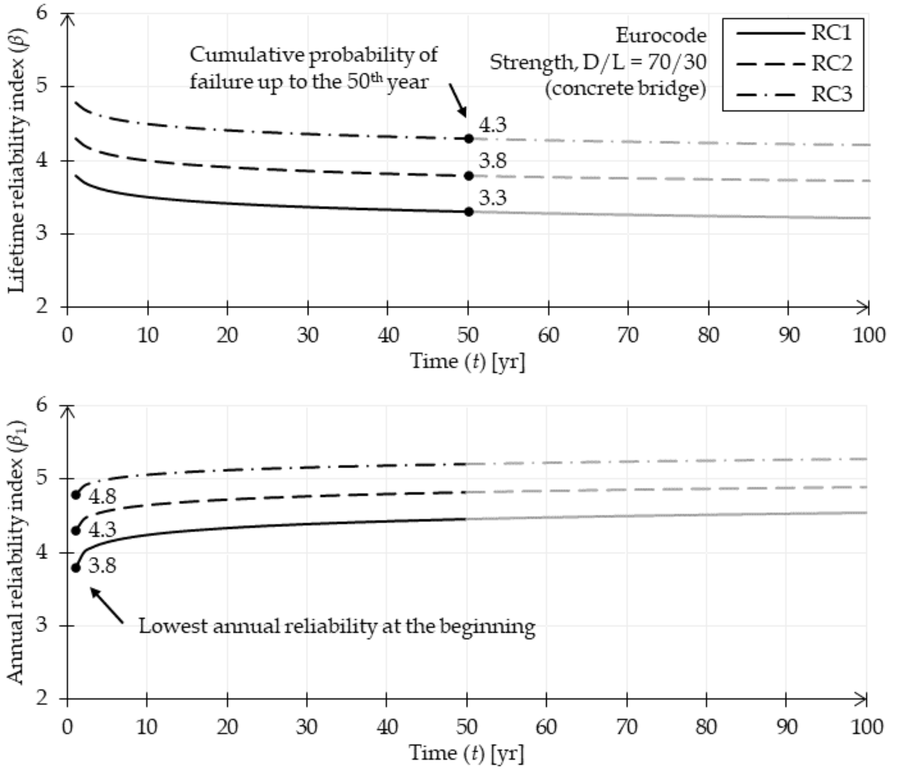

Figure 2. Lifetime reliability index (calculated from the cumulative failure probability) and annual reliability index as a function of time

Table 7. Minimum yearly reliability indices NEN 8700 and ROK/RBK; concrete bridge

Category	Reliability class / level	Reliability index (β)
NEN 8700 Disapproval	CC1b	2.1
	CC2	2.8
	CC3	3.7
NEN 8700 Renovation	CC1b	3.1
	CC2	3.7
	CC3	4.1
ROK/RBK	Minimum	2.8
	Usage	3.7
	New	4.9

3.3 *Variable load dominated (steel bridge)*

3.3.1 Description

In the case of a steel bridge, the self-weight is thought to contribute less to the total load. How much exactly depends on the length of bridge spans. A ratio of 30/70 between dead and live load is used here and is considered to be representative for a single-span bridge with 50 m span length. The limit state function and description of the random variables is the same as for the concrete bridge, but in this case the mean value of the self-weight G is 22.0 kN/m to satisfy the D/L = 30/70 ratio.

Note that the coefficient of variation used for the steel bridge is the same as that of the concrete bridge considered earlier. This does not reflect reality, as steel is known to show significantly less variation in strength. More common values are 0.07 and 0.08 according to JCSS (2001) and Gajot et al. (2003). Moreover, the distribution is truncated at the nominal value of the yield strength due to the delivery test issued by the steel supplier. This implies that the distribution differs from a lognormal distribution, where the difference is especially significant in the important lower tail. However, the coefficient of variation and the distribution type have been kept the same because the results are not expected to change significantly. After all, the mean value of the resistance is calibrated to give the desired value for the lifetime (e.g. β = 3.8 for RC2, etc.).

3.3.2 Eurocode

The assessment is performed in the same way as for the self-weight-dominated case (Section 3.2.2) and Figure 3 provides the results. Comparing Figure 3 with Figure 2, it appears that the yearly reliability is more constant. This was expected because the variable traffic load, which is dominant in this case, is not correlated in time. As a result, the minimum annual reliability indices are closer to the values in Table 1.

3.3.3 Other requirements

Similar probabilistic calculations have been performed for the other reliability requirements (NEN 8700 disapproval, NEN 8700 renovation and ROK/RBK). The same effect is found as demonstrated with the Eurocode requirements; the lowest annual reliability is slightly higher compared to the previous concrete bridge case.

4 Fatigue limit state reliability

4.1 Description

The fatigue limit state (FLS) is different in nature from the ultimate limit state (ULS) as considered in the previous section. Instead of examining an extreme load situation within a period of time, in case of fatigue, the entire history of action events is important. This deterioration process is typical for steel bridges and the current section studies its influence on the annual reliability. Note that deterioration through corrosion is not considered here.

Similar to the previous section, the objective is to find the minimum annual reliability indices for a structural design according to the current requirement of the reliability for a 50- or 100-year period. This allows for selecting new annual reliability requirements that do not break with the current trend.

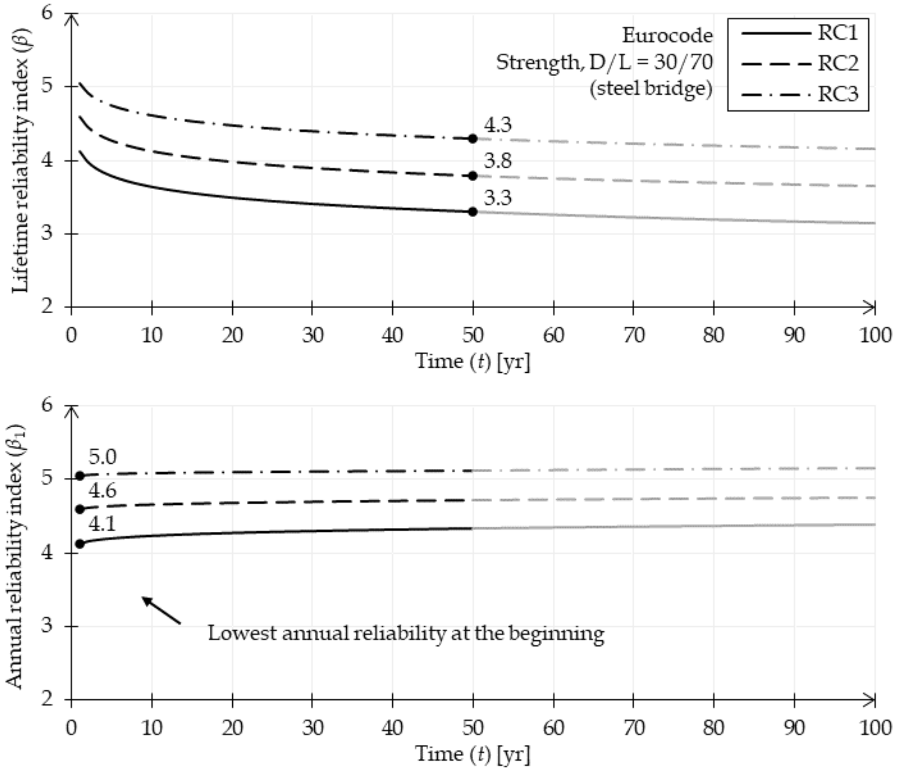

Figure 3. Lifetime reliability index (calculated from the cumulative failure probability) and annual reliability index as a function of time

4.2 *Linear damage model*

To evaluate the annual reliability of a generalized steel bridge that is subject to fatigue, use is made of a so-called S-N curve for the resistance and a stress range histogram for the traffic load. The first describes the maximum number of cycles allowed N_i as a function of the stress range S_i , while the latter describes the number of cycles to which the structure is subjected n_i for the same stress range. Using Miner's rule, the normalized cumulative damage caused by the load is calculated via:

$$D = \sum_i \frac{n_i}{N_i} \tag{6}$$

Failure is deemed to occur when the damage D is larger than a critical value D_{cr} . The variable D_{cr} is a random variable in the current study to represent the variability in the (variable amplitude) fatigue life. Following JCSS (2011), a lognormal distribution is used with a mean value of m_{Dcr} = 1 and a variation coefficient of V_{Dcr} = 0.3. The following limit state function applies:

$$z = D_{cr} - D = D_{cr} - m_T \sum_i \frac{n_i}{N_i} \tag{7}$$

where m_T is the model uncertainty of the load effect and accounts for the uncertainty in modelling choices regarding the probabilistic damage calculation such as the adoption of (linear) Miner's rule, the assumption of a Rayleigh distribution for the shape of the stress histogram, the distribution functions for n_i and N_i and so on.

4.3 *S-N curve*

The S-N curve describes the number of cycles to failure N_i for each stress range S_i . Use is made of the Eurocode definition for the characteristic S-N curve (see EN 1993-1-9; CEN, 2012). It consists of two linear relations when viewed on log-log scale; one for stress ranges larger than the constant amplitude fatigue limit (slope m_1 = 3) and one for stress range values that are smaller (slope m_2 = 5). The detail category is based on the stress range value for $N_C = 2 \cdot 10^6$ cycles, the constant amplitude fatigue limit (or knee-point) is located at $N_D = 5 \cdot 10^6$ cycles and the cut-off limit, below which the contribution to the fatigue damage is ignored, at $N_L = 10^8$ cycles. The left graph of Figure 4 shows an S-N curve based on fitting experimental data to the Eurocode definition. The right graph shows the probability density function of the damage as it is accumulated during the considered period. Note that the right graph is not an input, but a result of the combination of fatigue load and resistance, evaluated through Eq. (6).

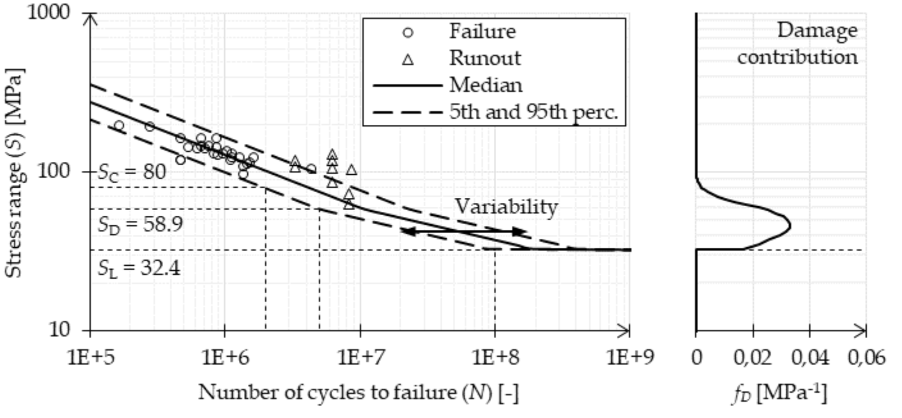

Figure 4. S-N curve used in the probabilistic analysis (left). Resulting distribution of accumulated damage in the probabilistic analysis (right)

The characteristic S-N curve represents a lower bound and, similar to other strength parameters, it is assumed to correspond to the 5th fractile of the distribution. It is assumed that $\log_{10}(N)$ is normal distributed with a standard deviation of 0.2 (DNV GL, 2016). The shape of the S-N curve is considered to be fixed; a random realization may be viewed as a random translation of the curve in the horizontal direction, as indicated in Figure 4. The median S-N curve can be obtained with this information, and the description is now suitable for a probabilistic calculation. Note that this is a simplified probabilistic model of the S-N curve, see Leonetti (2017), Leonetti (2020) and Baptista (2017) for more realistic models, but this simplified model is sufficient for the current assessment because it is aimed for a relative comparison only between a 50 years reliability index and an annual value.

4.4 *Stress range histogram*

The load exerted on a bridge continuously varies. This variability is expressed through the use of a stress range histogram. It expresses the expected or measured number of stress reversals n_i for each stress range S_i . A histogram may be calculated from the combination of expected traffic intensity and structural properties (e.g. influence lines) for the design of new structures. The load histogram usually follows a generic description, for example the Rayleigh probability density function.

In the assessment of an existing structure, the load histogram may be obtained from calculations or from measurements. In the latter case, the strains are measured using gauges at certain (critical) locations, which can be converted into stresses. Such measurements result in a time-history of the stress. To be useful for fatigue assessment, the number of stress reversals and their stress range need to be determined. The rainflow-counting method is commonly used for this conversion.

The left graph of Figure 5 shows a measured load histogram and the more generic histogram based on the Rayleigh distribution function as a fit to the data. Note that a mismatch between the fit and the measured data for the few cycles with a high range and for the large number of cycles with a very low range is not important, as their summed contribution to the damage is negligible. The generic Rayleigh histogram is used instead of the measured one in the following calculations. The right graph of Figure 5 shows the same probability density function of the damage as the right graph of Figure 4 but on a linear scale.

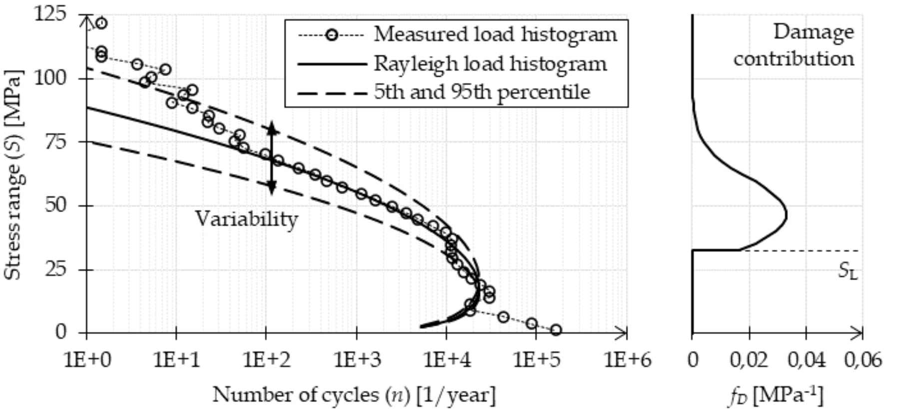

Figure 5. Load histogram used in the probabilistic analysis (left). Resulting distribution of accumulated damage in the probabilistic analysis (right)

In accordance with ISO 2394 (ISO, 2015), the load histogram is subject to uncertainty. In contrast to the statistical description of the S-N curve, variability in the stress range *S* instead of the number of cycles n is accounted for. A variation coefficient of 0.1 is used for new structures (where a calculated histogram is used) based on Maljaars (2021) and 0.02 for existing structures (where a measured histogram is used). As in case of the resistance, the model for the fatigue load is a simplification of reality (Hashemi, 2018) but this simplified model is sufficient for the current assessment because it is aimed for a relative comparison only between a 50-year reliability index and an annual value. The 5th and 95th percentiles, as displayed in Figure 5, correspond to a coefficient of variation of 0.1.

4.5 *Probabilistic analysis*

The time-dependent probabilistic analysis can be performed by making use of the statistical description of the S-N curve, the load histogram and the other random variables. Table 8 provides an overview of the parameters used in the probabilistic model. The detail category strength value S_C is adjusted such that the reliability requirement (for example $\beta = 3.8$ in $t = 50$ years) is exactly met.

Table 8. Overview of parameters for the probabilistic fatigue analysis

Symbol	Description	Distribution	Mean	Coefficient of variation
N	S-N curve (resistance), number of cycles	Lognormal	(Follows from detail)	0.46 ($\sigma_{\log_{10}(N)} = 0.2$)
S	Stress range load histogram	Lognormal	(Follows from load histogram)	0.10 (new) 0.02 (existing)
D_{cr}	Damage model variability	Lognormal	1	0.30
m_T	Model uncertainty for traffic load	Normal	1.04	0.17

The mean value and coefficient of variation of the model uncertainty for the traffic load effect m_T are equal to the values used for the ULS analysis. The effect of the load is not exactly the same as in the ULS case (damage accumulation instead of maximum load), but similar considerations hold with respect to trend, dynamics of the vehicle and bridge, etc. By adopting the same mean value and coefficient of variation, the influence of the model uncertainty in present both the ULS and FLS results, which will be compared later.

A Monte Carlo simulation is performed to calculate the annual reliability for each year. Random realizations are taken from the distributions of the S-N curve, load histogram, critical damage D_{cr} and load effect uncertainty m_T in each simulation (a fictional bridge).

The damage in one year is calculated from the load histogram realization. From this value the total number of years to failure (lifespan) is determined. The probability of failure in that year is increased and the simulation continues. In this way, using many realizations, the annual failure rate is determined over the entire period considered (100 years).

4.6 *Results*

4.6.1 Eurocode

The FLS results for the Eurocode requirements shown in Figure 6 reveal that the development of the reliability with time is very different from the ULS case. The analysis is performed using coefficient of variation value 0.1 (new structures) for the stress range following from load histogram *S*. At the beginning of the lifespan the reliability is high and it subsequently decreases. The speed of deterioration depends on the input variables, but the trend is typical for fatigue due to the gradual increase of damage (and thus a gradual decrease of resistance) with time.

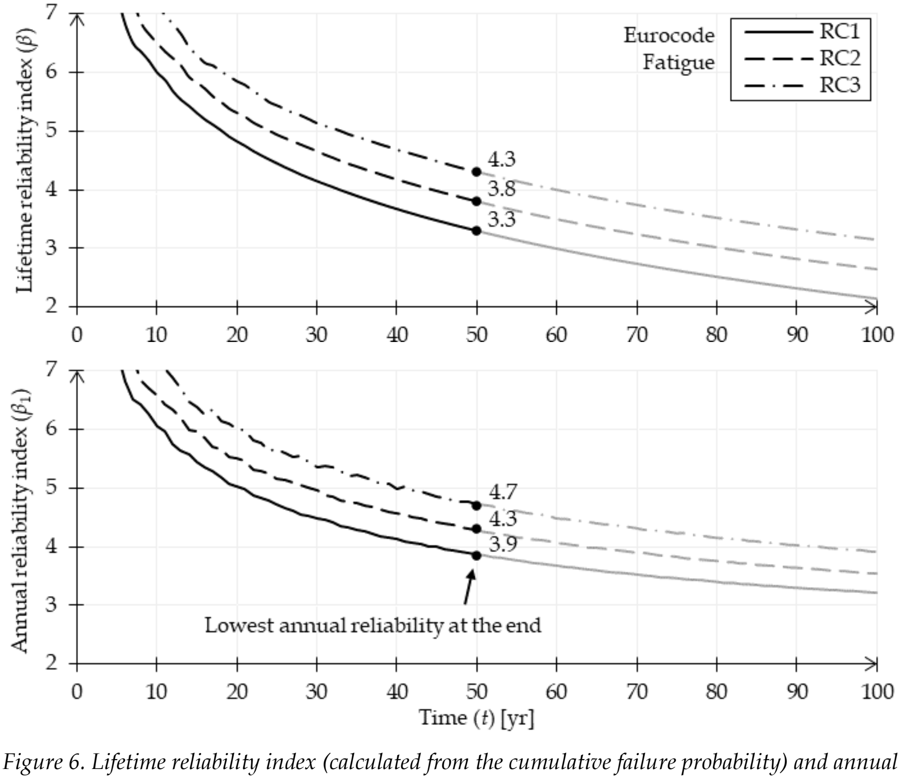

Figure 6. Lifetime reliability index (calculated from the cumulative failure probability) and annual reliability index as a function of time

Note that the Eurocode allows the use of a less strict reliability requirements for FLS as compared to ULS under some conditions (e.g. β = between 1.5 - 3.8 for RC2) depending on inspectability, repairability, and tolerance to damage. Structures that are inspected will therefore not display the high level of reliability in the first years as shown here, but rather a stepped trend of reliability will emerge (Maljaars & Vrouwenvelder, 2014). The current simulation adopted the simple case of a non-inspected structure with significant consequences of failure, for which the minimum required reliability should be in line with those of the ULS.

4.6.2 NEN 8700 disapproval

In the approval/disapproval of an existing bridge subject to fatigue, the elapsed service life should always be taken into account because sustained damage should be accounted for.

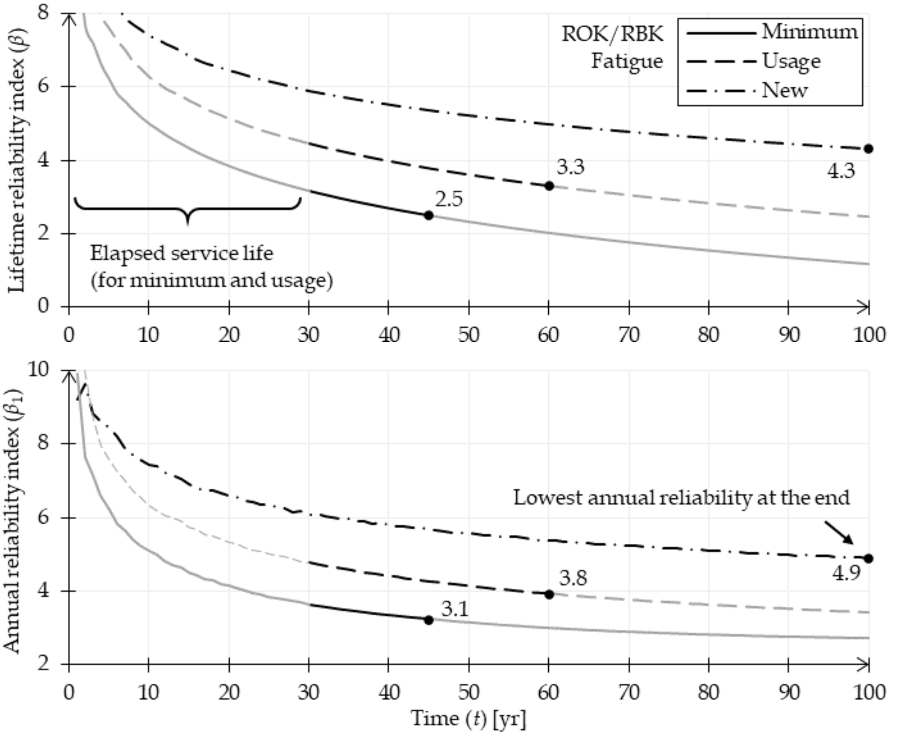

Figure 7. Lifetime reliability index (calculated from the cumulative failure probability) and annual reliability index as a function of time

Heavy traffic has significantly increased in intensity in the last decades. A bridge that has experienced 30 years of today's traffic intensity is assumed here, meaning that the actual age is much higher than 30 years. The (extended) service life is chosen as the minimum value specified in NEN 8700, which is 15 years. The variation coefficient of the model uncertainty m_S is taken as 0.02 because it is assumed that the stress range histogram follows from measurements. Figure 7 provides the reliability analysis results for the NEN 8700 disapproval requirements.

4.6.3 ROK/RBK

The results of the probabilistic fatigue analysis for the ROK/RBK requirements are shown in Figure 8. A variation coefficient of the load histogram model uncertainty of 0.02 is used for the Minimum and Usage level, for the New level it is taken as 0.1.

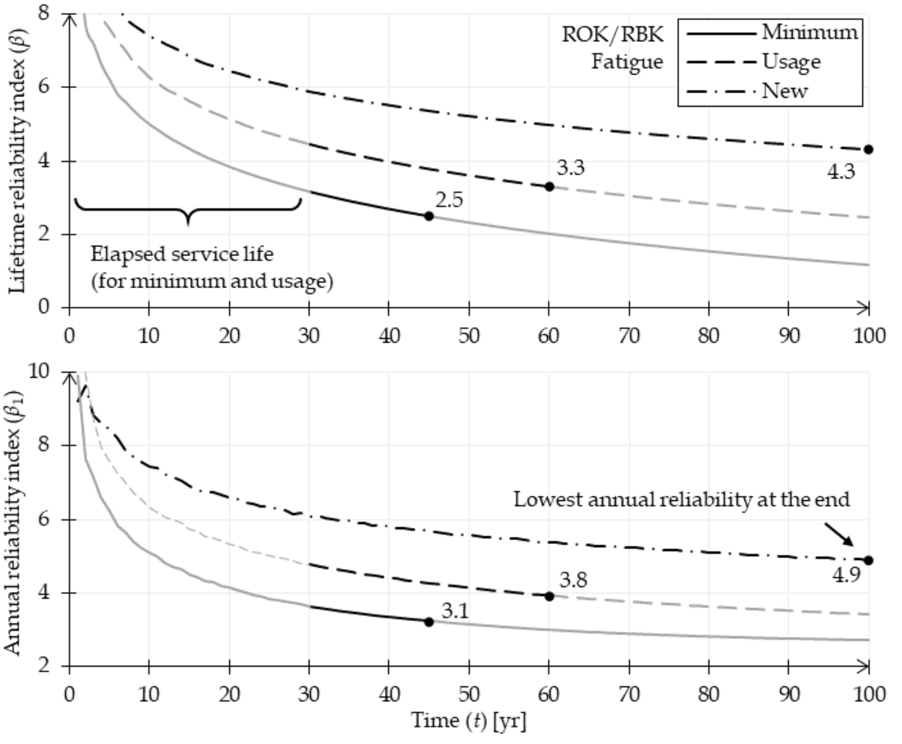

Figure 8. Lifetime reliability index (calculated from the cumulative failure probability) and annual reliability index as a function of time

5 Influence coefficients and partial factors

5.1 *Description*

In this section, a generic case is used to study the effect of the reference period on influence coefficients and partial factors. The section has a more theoretical nature and therefore values are chosen for a typical case for both buildings and bridges, and are slightly different from the values given in the previous chapters. However, the same results (change of influence coefficients, but no change of design point) were observed for the cases studied in the previous sections. The generic nature of the typical case makes the change in influence coefficients depending on the reference period clear, without case-specific details. Additional results, including the change of reliability class, may be found in TNO (2019) and show the same pattern regarding the influence coefficients and the design point.

5.2 *Definition of a typical case*

A non-deteriorating structure in ULS subjected to both a permanent and variable load is considered in the following probabilistic analysis example. It is analysed for a reference period of 50 years and a reference period of 1 year, where the latter considers the first year of the lifespan. The limit state is described by the following expression:

$$Z = \theta_R R - (\theta_G G + \theta_Q C_{0Q} Q) \tag{8}$$

where Q is equal to either Q_1 or Q_{50}, corresponding to the maximum variable load over a period of 1 or 50 years, respectively. An overview of the random variables is presented in Table 9. The mean value of the resistance is chosen as unity. The other values are chosen to be representative of a common situation, where:

- the reliability index for the 50-year reference period β_{50} = 3.8; and
- the influence coefficients are approximately α_R = 0.8 and α_E = -0.7.

The ratio between the characteristic variable and the total load, $\chi_k = \dfrac{Q_k}{G_k + Q_k}$, is approximately equal to 0.4, where G_k = 0.33 is equal to the mean value of the permanent load and Q_k = 0.213 relates to the 0.98 fractile of the annual maximum distribution. The variable load parameters of the 50-year maximum distribution have been derived by shifting the annual maximum distribution.

Table 9. Overview of random variables

X	Description	Distribution	Mean	CoV
R	Resistance	Lognormal	1	0.10
G	Permanent load (self-weight)	Normal	0.33	0.07
Q_1	Variable load (annual maximum)	Gumbel-max	0.14	0.20
Q_{50}	Variable load (50-year maximum)	Gumbel-max	0.22	0.12
C_{0Q}	Time-invariant component of the variable load	Lognormal	1	0.07
θ_R	Model uncertainty resistance	Lognormal	1	0.075
θ_G	Model and statistical uncertainty permanent load	Lognormal	1	0.05
θ_Q	Model and statistical uncertainty variable load	Lognormal	1	0.10

5.3 **Results**

5.3.1 Numerical values

The results of the probabilistic analysis for the 50-year reference period are:

β_{50} = 3.80

α_R = 0.75, α_E = -0.66

$\gamma_{\theta R}\gamma_R$ = 1.21, $\gamma_{\theta G}\gamma_G$ = 1.09, $\gamma_{\theta Q}\gamma_{C0Q}\gamma_Q$ =1.59

where $\alpha_R = \sqrt{0.446^2 + 0.594^2}$ and $\alpha_E = -\sqrt{1-\alpha_R^2}$ are the influence coefficients of the resistance terms $\theta_R R$ and load effect terms $\theta_G G + \theta_Q C_{0Q} Q$, respectively.

The analysis with the 1-year reference period (the first year) results in:

β_1 = 4.62

α_R = 0.67, α_E = -0.75

$\gamma_{\theta R}\gamma_R$ = 1.25, $\gamma_{\theta G}\gamma_G$ = 1.10, $\gamma_{\theta Q}\gamma_{C0Q}\gamma_Q$ = 1.49

where $\alpha_R = \sqrt{0.394^2 + 0.525^2}$ and $\alpha_E = -\sqrt{1-\alpha_R^2}$. A comparison of the results obtained for both 50 years and annually is provided in Table 10. Two columns are presented for each reference period: the first contains the influence coefficients, and the second holds the design values. The values can be combined for the resistance and the load effect, as shown in the bottom two rows.

Table 10. Comparison of results obtained for 50 years and annually (first year)

X	50 years		Annual	
	α	X_d	α	X_d
θ_R	0.446	0.879	0.394	0.870
R	0.594	0.795	0.525	0.782
θ_G	-0.152	1.03	-0.139	1.03
G	-0.202	0.348	-0.184	0.350
θ_Q	-0.290	1.11	-0.246	1.11
C_{0Q}	-0.203	1.05	-0.173	1.05
Q	-0.509	0.292	-0.652	0.272
Combined values				
R	0.75	0.70	0.67	0.68
E	-0.66	0.70	-0.75	0.68

Section 2 showed that the RC2 annual reliability varies between the 50-year reference period value (3.8) and the uncorrelated in time value (4.7) depending on the influence of the time-variant component. In this case, it is closer to the latter because of the considerable variability of Q. But it should be noted that the variability is quite moderate compared to common values for imposed loading and climatic actions.

The analysis results show that the influence coefficient of the resistance is lower for a 1-year as compared to a 50-year reference period, whereas the opposite is true for the load effect. Generalizing this effect leads to the change from:

$\alpha_R = 0.8, \quad \alpha_E = -0.7$

for the design life case to:

$\alpha_R = 0.7, \quad \alpha_E = -0.8$

for the annual reliability case, approximately.

5.3.2 Partial factors

Comparing the partial factors for both reference periods indicates that the difference is small. The largest change is observed for the variable load (1.49 to 1.59), but it is offset slightly by the increase for the permanent load (1.09 to 1.10). In the light of approximations in the semi-probabilistic format, this is still deemed acceptable. But, if necessary, the partial factors of the loads can be recalibrated.

In Figure 9, the real space of the random variables (X-space) is shown, including the origins and design points. This figure explains how the reliability index and influence coefficients change significantly for the annual reliability analysis, whilst the partial factors remain very similar. The reason for this is that the design point moves only very slightly; the largest change is the shift of the origin. The coordinates of the design point are provided in the last two rows of Table 10.

5.3.3 Influence time-variant component

The FORM origin of the 1-year distribution shifts upwards in the graph in case the influence of the time-variant component of the variable load is smaller and the annual reliability is closer to the 50-year value (e.g. the self-weight dominated case; Section 3.2). In addition, the design points move slightly further apart, but the partial factors remain similar. In this case, also the influence coefficients remain similar – in contrast to the more common case in which the time-variant component has a larger influence.

6 Proposed annual reliability requirements

6.1 ***Eurocode***

Based on the results in Sections 3 and 4, annual reliability indices are selected to avoid a trend break with current regulations. In this way, a design based on a current reliability requirement (with a fixed reference period) will be equal to the design based on an annual reliability requirement. To achieve this, the minimum annual reliability values from Figures 2-6 and 9-12 are collected for each reliability class, see Table 11.

To simplify the requirements, the minimum of all values is adopted per reliability class (Table 12). This only implies a small trend break with current regulations (for the case ULS with D/L = 30/70) but ultimately leads to more consistent reliability performance on an annual basis.

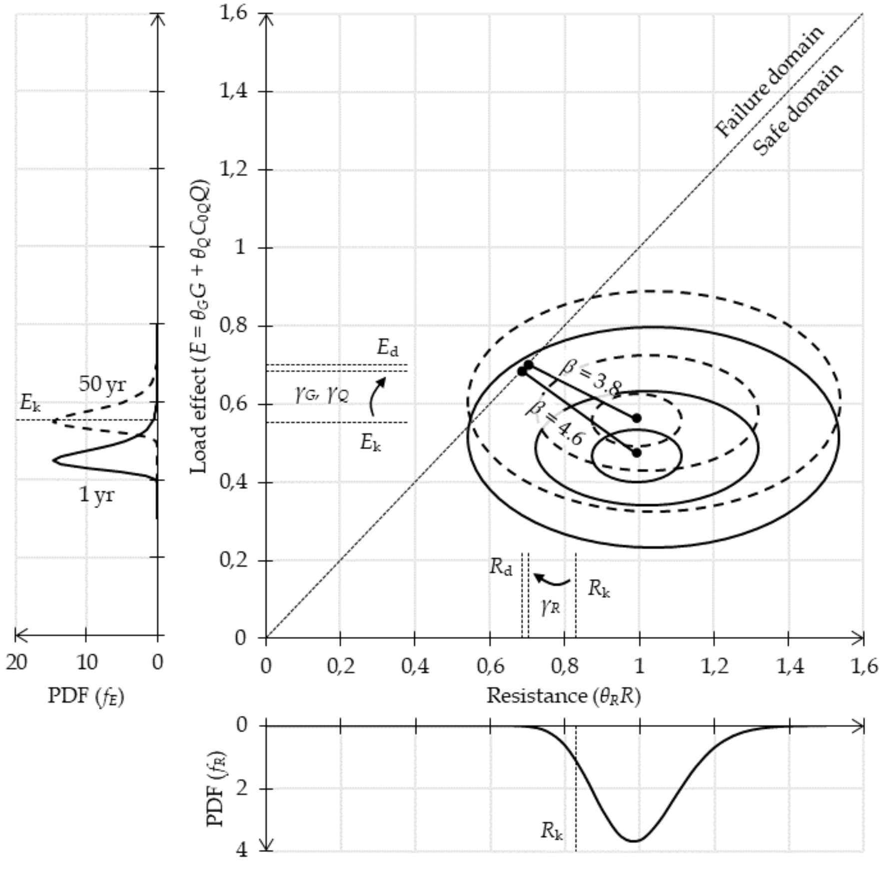

Figure 9. Joint probability distribution of the resistance and load effect

Table 11. Collected minimum annual reliability indices Eurocode

Reliability class	ULS, D/L = 70/30 (concrete bridge)	ULS, D/L = 30/70 (steel bridge)	FLS (steel bridge)
RC1	3.8	4.1	3.9
RC2	4.3	4.6	4.3
RC3	4.8	5.0	4.7

Table 12. Proposed minimum annual reliability indices Eurocode

Reliability class	Reliability index
RC1	3.8
RC2	4.3
RC3	4.7

6.2 NEN 8700

The same approach as for the Eurocode may be followed for NEN 8700. The minimum values from Table 7 for the disapproval and renovation level have been collected in Table 13.

Table 13. Collected minimum annual reliability indices NEN 8700

Category	Reliability class	ULS, D/L = 70/30 (concrete bridge)*	FLS (steel bridge)
Disapproval	CC1b	2.1	2.8
	CC2	2.8	3.2
	CC3	3.7	3.8
Renovation	CC1b	3.1	-
	CC2	3.7	-
	CC3	4.1	-

* for D/L = 70/30 the values are slightly higher

Again here a simpler approach would involve taking the minimum of the ULS and FLS values (Table 14). However, the minima following from human safety as given in the background document to NEN 8700 (TNO, 2011) are slightly higher (Table 2). Therefore, these values are adopted for disapproval.

Table 14. Proposed minimum annual reliability indices NEN 8700

Category	Reliability class	Reliability index
Disapproval	CC1b	2.3
	CC2	3.4
	CC3	4.0
Renovation	CC1b	3.1
	CC2	3.7
	CC3	4.1

6.3 ROK/RBK

The minimum values have also been determined for the ROK/RBK requirements; they are collected in Table 15. None of the values are below the minima in relation to human safety as described in NEN 8700.

Table 15. Collected minimum yearly reliability indices ROK/RBK

Level	ULS, $D/L = 70/30$ (concrete bridge)	FLS (steel bridge)
Minimum (disapproval structures secondary roads)	2.8	3.2
Usage	3.7	3.9
New	4.9	4.9

Taking the minimum of both the ULS and FLS cases leads a value of 2.8 for the minimum values used for the disapproval of structures in secondary roads. This value is lower than the minimum requirement for human safety on an annual basis (see Section 2.2). Therefore, as done in the section above, a value of 3.4 is proposed. For new structures (CC3) the usage of a lifespan of 100 years leads to a value of 4.9 for the annual reliability in the governing year. This is slightly higher than the RC3 value following from Eurocode requirements (Section 6.1). For the annual reliability one single value should be selected, therefore the value 4.7 is proposed. The annual reliability values are presented in Table 16.

6.4 ***Corresponding influence coefficients***

If annual reliability requirements are adopted, this has an impact on the framework of the Eurocode. As shown in Section 5, the influence coefficients change with a reference period of 1 year. The magnitude of the change depends on the variability of the load and on the dead-to-live load ratio. For an annual reliability assessment, the influence coefficient of the resistance becomes lower, while it becomes larger for the load effect. It is suggested to replace the current influence coefficients:

Table 16. Proposed minimum yearly reliability indices ROK/RBK

Level	Reliability index
Disapproval structures secondary roads	3.4
Usage	3.7
New	4.7

$$\alpha_R = 0.8, \quad \alpha_E = -0.7$$

for the design life case to:

$$\alpha_R = 0.7, \quad \alpha_E = -0.8$$

for the annual reliability case. Note that the influence coefficients are slightly conservative because $\sqrt{0.8^2 + (-0.7)^2} = 1.063 > 1$. They also represent an average, since the actual influence coefficients differ from case to case. E.g. a smaller value of α_R may be expected for a steel structure than for a concrete structure as the inherent material variability of concrete is larger. Similarly, a smaller value of α_E may be expected if the loading is dominated by self-weight, in contrast to a highly variable action such as wind. For these reasons, not too much importance should be assigned to the exact values of α_R and α_E .

7 Discussion

The study documented in this article is exploratory in nature. Not all cases have been considered. This holds with respect to the probabilistic model that has been adopted, the self-weight to variable load ratios and the traffic load parameters. In this study only the 70/30 and 30/70 ratios have been considered for a bridge with a span of 50 m. More combinations could be thought of, but the cases studied in this article have been chosen to represent reasonable practical boundary cases.

If the full range of possibilities is explored, it is likely that cases closer to the theoretical extremes are found. If all loading on the structure follows from self-weight (100/0 case), the reliability in the first year will be equal to that of the entire lifetime. One can also imagine a case where all loading follows from variable actions (0/100 case). Then, the yearly reliability will be close to the theoretical maximum, but not equal because the resistance of the structure is fully correlated in time. If the variability of the resistance is very small with respect to the variability of the loading ($\alpha_R \approx 0$), then the theoretical maximum is reached (Poisson process). In Table 17 an overview is given of the theoretical extremes and cases considered for the Eurocode requirements.

Table 17. Minimum annual reliability of the Eurocode theoretical extremes and cases considered

Reliability class	Theoretical minimum	ULS, D/L = 70/30	ULS, D/L = 30/70	Fatigue	Theoretical maximum
RC1	3.3	3.8	4.1	3.9	4.3
RC2	3.8	4.3	4.6	4.3	4.7
RC3	4.3	4.8	5.0	4.7	5.1

A trend break may occur for uncommon extreme combinations. This cannot be easily counteracted by choosing the theoretical minima as target annual reliability values, as this would result in much lower reliability requirements for the common (not extreme) cases. Therefore, a minor trend break is unavoidable for a small number of extreme situations.

8 Conclusions

On the basis of calculations for the ultimate limit state (ULS) and fatigue limit state (FLS) it becomes apparent that the annual reliability varies with time. In particular, the following was found:

1. In case of a relatively large contribution of self-weight to the total load and time-dependent aspects (deterioration, increasing load, etc.) do not play a significant role, the lowest annual reliability is found at the beginning of the lifetime. This typically holds for (elements in) bridges where the load is self-weight dominated.
2. In case of a relatively small contribution of self-weight to the total load and time-dependent aspects (deterioration, increasing load, etc.) do not play a major role, the lowest annual reliability is also found at the beginning of the lifetime. The annual reliability remains more constant in time than in the first case. This typically holds for (elements in) bridges which are not self-load dominated.
3. If fatigue deterioration dominates with regard to the reliability of the structure, the degradation of strength causes that the lowest annual reliability (conditional failure probability) is found at the end of the lifetime. The same is expected for other degradation mechanisms, such as corrosion and alkali-silica reaction.

Based on these findings, a different simplified assessment strategy applies to each limit state:

1. Ultimate limit state (ULS): The first year of the lifetime should be assessed in the (semi-)probabilistic reliability analysis. The analysis does typically not include the elapsed service life.
2. Fatigue limit state (FLS): The last year of the lifetime should be assessed in the (semi-) probabilistic reliability analysis. The analysis should include the elapsed service life explicitly to account for sustained damage.

In case the reliability for FLS (or any other deterioration) and for ULS are similar, the two effects may negate each other and the annual reliability of the complete system may be more or less constant in time.

To verify designs or assess existing structures annual reliability indices have been provided for each use case. The proposed values have been based on the smallest reliability values of the relevant cases studied. A small trend break with current regulations is unavoidable in some cases, but ultimately the annual reliability framework leads to more consistent reliability performance between structures.

In studying a more common case than the traffic load case, in which the variability of the time-variant load is larger, the change in partial factors appears small. It follows that, although the origin of the resistance and load effect is markedly different, the design point does not shift significantly. As a result, the change in partial factors is so small that they do not need to be adjusted if an annual reliability framework is adopted.

Acknowledgement
The authors wish to express their gratitude and sincere appreciation to the Dutch Ministry of Infrastructure and Water Management (Rijkswaterstaat) for financing the research work and the fruitful discussions in the course of this study.

References

Baptista, C, Reis, A. and Nussbaumer. A. (2017). Probabilistic S-N curves for constant and variable amplitude. *International Journal of Fatigue*. Volume 101, pp. 312-327.

Cajot, L.G., Haller, M., Conan, Y., Sedlacek, G., Kraus, O., Rondla, J., Cerfontaine, F., Lagerqvist, O. and Johansson, B. (2005). Probabilistic quantification of safety of a steel structure highlighting the potential of steel versus other materials. Report, Technical Steel Research, Contract No. 7210-PR/249.

DNV GL. (2016). DNVGL-RP-C203: Fatigue design of offshore steel structures. Recommended Practice.

European Committee for Standardization (CEN). (2011). Eurocode 0: Basis of structural design. Standard, EN 1990.

European Committee for Standardization (CEN). (2012). Eurocode 3: Design of steel structures – Part 1-9: Fatigue. Standard, EN 1993-1-9. Brussels, Belgium.

Fédération internationale du béton (fib). (2016). Partial factor methods for existing concrete structures. Bulletin 80, Recommendation, Task Group 3.1.

Hashemi, S.B., Maljaars, J. and Snijder, H.H. Partial factors for fatigue loads in the Eurocode system for road bridge design. In: Haugen et al. (Eds). *Proc. Esrel 2018*, London, Taylor and Francis Ltd. pp. 2339-2345.

International Organization for Standardization (ISO). (2015). General principles on reliability for structures. Standard, ISO 2394.

Joint Committee on Structural Safety (JCSS). (2001). JCSS Probabilistic Model Code. https://www.jcss-lc.org/jcss-probabilistic-model-code/

Leonetti, D., Maljaars, J. and Snijder, HH. (2017). Fitting fatigue test data with a novel S-N curve using frequentist and Bayesian inference. *International Journal of Fatigue*, Volume 105, pp. 128-143.

Leonetti, D., Maljaars, J. and Snijder, HH. (2020). Probabilistic fatigue resistance model for steel welded details under variable amplitude loading – Inference and uncertainty estimation. *International Journal of Fatigue*, Volume 135, pp. 1-18.

Maljaars, J. and Vrouwenvelder, A.C.W.M. (2014). Probabilistic fatigue life updating accounting for inspections of multiple critical locations. *International Journal of Fatigue*, Volume 68, pp. 24-37.

Nederlands Normalisatie-instituut (NEN). (2011). NEN 8700: Beoordeling van de constructieve veiligheid van een bestaand bouwwerk bij verbouw en afkeuren – Grondslagen. Standard.

Rijkswaterstaat (RWS). (2013). Richtlijnen Beoordeling Kunstwerken – Beoordeling van de constructieve veiligheid van een bestaand kunstwerk bij verbouw, gebruik en afkeur. Report, Version 1.1, 27 May.

TNO. (2011). Veiligheidsbeoordeling bestaande bouw – Achtergrondrapport bij NEN 8700. Report, TNO-060-DTM-2011-03086. https://publications.tno.nl/publication/34614949/zAI60C/TNO-060-DTM-2011-03086.pdf

TNO. (2012). Algemene veiligheidsbeschouwing en modellering van wegverkeerbelasting voor brugconstructies. Report, TNO-060-DTM-2011-03695-1814. https://publications.tno.nl/publication/34618452/balfBk/TNO-060-DTM-2011-03695.pdf

TNO. (2019). Reliability analysis on an annual basis applied to bridges in the Netherlands. Report, TNO-2018-R11640.

Vrouwenvelder, A.C.W.M. and Scholten, N.P.M. (2010). Assessment criteria for existing structures. *Structural Engineering International*, Volume 20, Issue 1, pp. 62-65.