Safety philosophy for existing structures and partial factors for traffic loads on bridges

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Evaluating and upgrading existing structures becomes more and more important. For a large part of the existing infrastructure and buildings the design life has been reached or will be reached in the near future. These structures need to be reassessed in order to find out whether the safety requirements are met. Not only for new structures but also for the existing stock the Eurocodes are starting point for the assessment of the safety. However, it would be uneconomical to require all existing buildings and civil engineering works like bridges to comply fully with these new codes and corresponding safety levels. The assessment of existing structures therefore differs from the design situation. This paper describes the main differences with respect to the relevant reliability requirements and develops a set of partial factors to be used in reassessment of existing bridges under traffic load.

Keywords: Reliability, existing structures, bridges, partial factors

1 Introduction

For a large part of the existing buildings and infrastructure the design life has been reached or will be reached in the near future. This is because a huge part of the existing stock has been built in the sixties of the previous century. These structures need to be reassessed in order to investigate whether the safety requirements are met. The assessment of existing structures is becoming more and more important for social and economical reasons, while most codes deal explicitly only with design situations of new structures. The assessment of an existing structure may, however, differ much from the design of a new one. Due to deterioration and damage it is general practice to inspect existing structures and if necessary to repair and/or strengthen them. In general, the safety assessment of an existing structure differs from that of a new one in a number of aspects [1,2]. The main differences are:

1. Increasing safety levels usually involves more costs for an existing structure than for structures that are still in the design phase. The safety provisions embodied in safety standards have also to be set off against the cost of providing them, and on this basis improvements are more difficult to justify for existing structures. For this reason and under certain circumstances, a lower safety level is acceptable.
2. The remaining lifetime of an existing building is often less than the standard reference period of 50 or 100 years that applies to new structures. The reduction of the reference period may lead to reductions in the values of representative loads as for instance indicated in the Eurocode for Actions [11].
3. For an existing building or bridge structure actual measurements with respect to geometry, material properties and behaviour under normal or design circumstances (e.g. settlements, cracks, corrosion, survival of certain loads, etc.) may be made in order to reduce uncertainty.

In the following sections, the safety philosophy for existing structures is discussed. First, briefly the reliability levels in terms of the β values for new structures are given; then for existing structures the required β values are presented with motivation. Based on this, for existing bridges under traffic load, the partial safety factors are derived using a full probabilistic approach.

2 Reliability levels for new structures

Modern safety standards express the safety target for new structures in probabilistic terms [3-5]. Here, Eurocode EN 1990 [12] is followed. The safety level of a structure or part of a structure is in principle expressed as the probability of failure for a relevant period of time. Instead of the probability of failure, however, use is made of the reliability index β. The reliability index β has a direct relation to the probability of failure P (see Table 1).

In practice the method to establish the desired safety level is by a proper choice of the following parameters:

- the consequence class for the structures
- the characteristic loads
- the load factors γ_f and the combination factors Ψ
- the design rules and material properties
- the material factor γ_m

The load and material factors are chosen in such way that a safety level (expressed by β) belonging to the vigouring consequence class is obtained. Eurocode EN 1990 gives three consequence classes CC1, CC2 and CC3 [12]. In Table 2, for new structures, the β values are provided for these consequence classes. For new structures, the subscript *n* is used for the β values.

In general, the codified partial factors for loads as well as resistance may be considered as being in line with these starting points. As Table 2 shows, an exception is made for the wind loading. Given the standard variable load factor $\gamma_Q = 1.5$, a wind load dominated building has a lower reliability level then the target value. According to [6, 7], the values shown in the last column of Table 2 seem to be appropriate. For bridges, important

Table 1: reliability index β and probability of failure P

reliability index β	probability of failure *P*
1.0	0.16
2.0	0.023
3.0	0.0013
4.0	0.000032

Table 2: Reliability index for new structures [12] and [6, 7]

Consequence class	Consequences of failure		Wind load not dominating	Wind load dominating
	Loss of human life	Economic damage		
1	Small	Small	$\beta_n = 3.3$	$\beta_n = 2.3$
2	Considerable	Considerable	$\beta_n = 3.8$	$\beta_n = 2.8$
3	Very large	Very large	$\beta_n = 4.3$	$\beta_n = 3.3$

buildings and large civil engineering structures, the target value is β = 4.3. The reliability index is intended to be used in correspondence with a reference period equal to the design working life of the structures, usually 50 years for buildings and 100 years for bridges.

3 Safety in combination with a shorter design life time

It can be a matter of debate what changes are allowed to the design values of the loads on structures in the case of a shorter design life time. Usually the shortening of the reference period for the variable loads (wind, snow, etc) results in a decrease of the representative values. The Eurocodes give formulas to calculate these reductions. On the other hand, establishing the safety factors for a shorter design life time, both economic arguments and limits for human safety play a role. The latter play a role because of the maximum allowable annual probability of failure, calculated for small probabilities through $P_{failure} = \Phi(-\beta)/T$ with T the design life time in years. This annual probability of failure may not exceed the limits for human safety. This means that too short life times may lead to unacceptable large probabilities of failure. In the following sections it turns out that a minimum design life time of 15 year is therefore to be required in structural design. The economic arguments and limits for human safety are discussed successively in Section 3.1 and 3.2 and brought together in Section 3.3.

3.1 Economics

If only economic optimization is considered and the failure probability increases approximately linear in time, it makes sense to use the same target failure probability or reliability index regardless the design life time. As a result the partial factors do not change when the design life is changed.

For example the reliability index β = 3.8 corresponds to a probability of failure of about 10^{-4}. This probability is for the whole reference period, regardless of the length of it. A structure designed with β = 3.8 for a period of 1 year has a probability of failure $P_f = 10^{-4}$. A structure designed with β = 3.8 for 50 years has in each arbitrary year a probability of failure of $(1-10^{-4}) = (1-P_f)^{50} \Rightarrow P_f = 2 \cdot 10^{-6}$ ($\approx 10^{-4}/50$, for small probabilities), which is much smaller. The partial factors are the same for both periods, however, the representative loads are larger for the long period. This indeed makes sense as it is more economical to invest in safety measures if one can profit from it for a longer period of time. Therefore, a shorter design live does not provide an argument for a reduction of β.

3.2 *Human safety*

If human safety is the governing factor in the design, one generally wants to have a constant annual failure probability. The probability to die as a result of an accident (traffic, falling from the stairs) is about 10^{-4} per year in the Netherlands. For other countries this is in the same order of magnitude. It is certainly not accepted in society that the probability to become the victim of structural failure is larger than the normal probability to die as a result of an accident. For that reason it is agreed to establish the maximum probability to become the victim of structural failing as order of magnitude 10^{-5} per year.

In EN 1990 [12], the reliability level is described in qualitative terms with respect to the danger of life and the economical damage. Here, for each consequence class, this qualitative description is translated in to the following conditional probabilities P_1 for loss of human life.

CC1:	small consequence for loss of human life	$P_1 = 10^{-3}$
CC2:	considerable consequence for loss of human life	$P_1 = 3 \cdot 10^{-2}$
CC3:	high consequence for loss of human life	$P_1 = 3 \cdot 10^{-1}$

These probabilities are conditional, probabilities given the fact that a structural component fails. The probabilities relate to individual persons that are in the building on a regular basis. The corresponding probability of failure P_g of a structural part can now be calculated for one year:

$$P_g \cdot P_1 < 10^{-5} \tag{1}$$

CC1: $P_g \leq 10^{-2} \Rightarrow \beta \geq 2.3$

CC2: $P_g \leq 3 \cdot 10^{-4} \Rightarrow \beta \geq 3.4$

CC3: $P_g \leq 3 \cdot 10^{-5} \Rightarrow \beta \geq 4.0$

The effect of a longer reference period (t) can be incorporated in approximation by reducing the β values with $C \log t$:

$$\text{CCl:} \quad P_g \leq \; t \; 10^{-2} \quad \Rightarrow \quad \beta \geq \Phi^{-1}\{ \; t \; 10^{-2}\} \approx 2.3 - 1.10 \log t$$
$$\text{CC2:} \quad P_g \leq t \, 3 \cdot 10^{-4} \quad \Rightarrow \quad \beta \geq \Phi^{-1}\{t \, 3 \cdot 10^{-4}\} \approx 3.4 - 0.75 \log t$$
$$\text{CC3:} \quad P_g \leq t \, 3 \cdot 10^{-5} \quad \Rightarrow \quad \beta \geq \Phi^{-1}\{t \, 3 \cdot 10^{-5}\} \approx 4.0 - 0.60 \log t$$

Note that the value of $3 \cdot 10^{-4}$ per year for CC2 corresponds well with the recommendation in [7, Chapter 7.2] for moderate failure consequences and relative large costs of safety measures.

*3.3 **Economics and human safety in structural design***

In this section the above mentioned arguments of economics and human safety are translated into a practical method that can be used by structural engineers.

In Figure 1, the annual failure probability as a function of the design working life has been plotted for new structures in consequence class 2. As stated above, considering economic optimization only, the β values remain the same for different design life times; this means that the probability of failure per year is obtained as $P_{failure} = \Phi(-\beta)/T$, with T the design

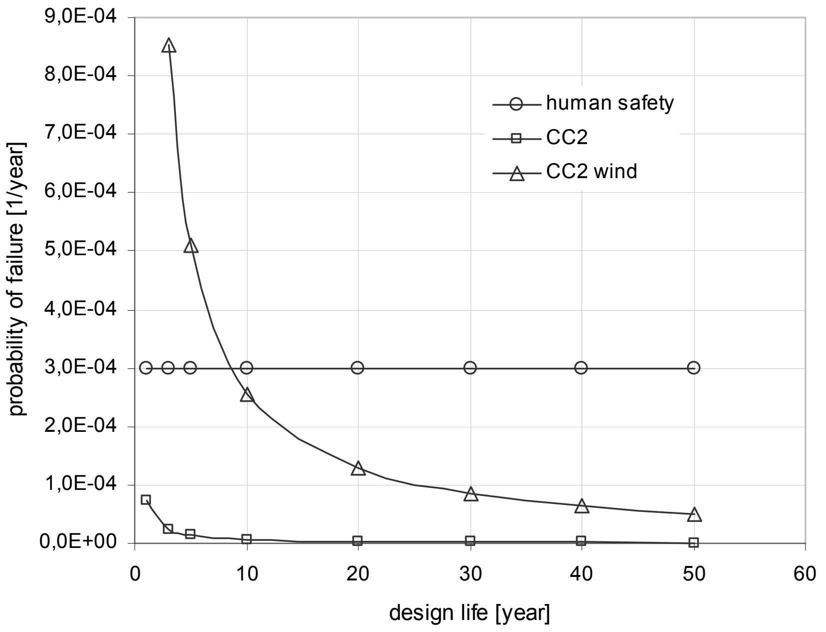

Figure 1: Annual failure probability as a function of de design working life, new structures CC2.

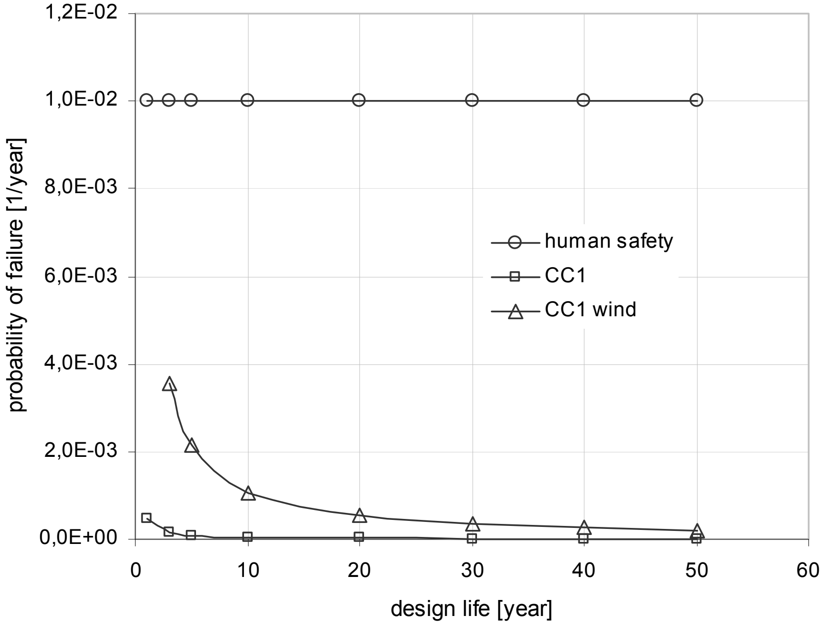

Figure 2: Annual failure probability as a function of de design working life, new structures CC1.

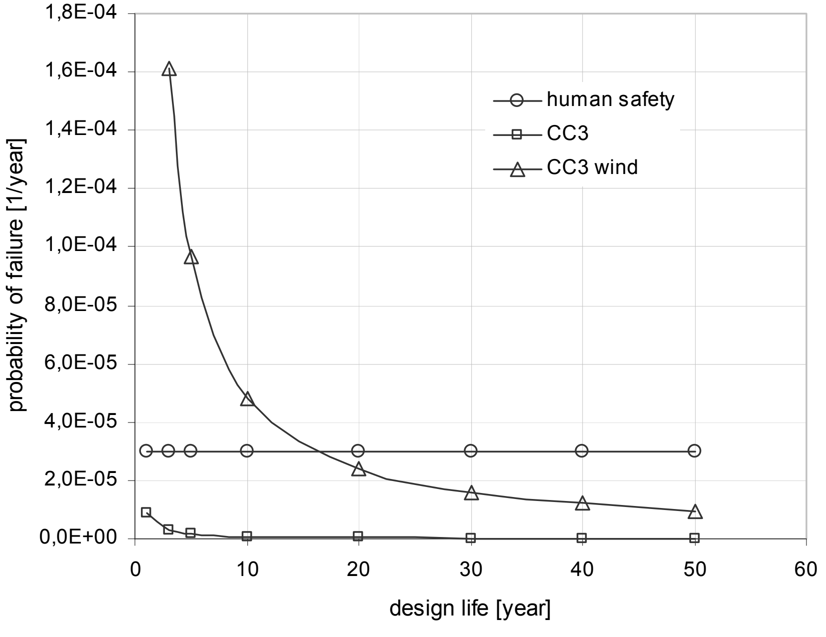

Figure 3: Annual failure probability as a function of de design working life, new structures CC3.

life. This probability is plotted for $\beta = 3.8$ and $\beta = 2.8$ (wind dominant). Also the lower boundary for human safety $P_g \leq 3 \cdot 10^{-4}$ for CC2 is displayed. For consequence class 1 and 3 the same is shown in Figures 2 and 3 respectively. From these figures it can be concluded that for CC2 and CC3 in the case of wind dominated structures and design lives smaller than 15 years the probability of failure exceeds the level for human safety. In that case, instead of raising partial factors for short periods, the Dutch Code simply demands a minimum design life of 15 years for CC2 and CC3. In Table 3 the lower β limits for human safety are collected for both a 1 year and a 15 year design life. In Section 4 these values will be employed.

Table 3: Lower β limits for human safety

Consequence class	1 year design life	15 year design life
1	2.3	1.1
2	3.4	2.5
3	4.0	3.3

4 Safety targets for existing structures

In this section the required β values are derived for two types of decision.
First we have the level below which the structure is unfit for use. If this safety level is not reached, the authorities have to send immediately a notification that the structure has to be closed and to be adapted. Secondly, we have the safety level for repair of existing structures.

Establishing safety targets for existing structures, both economic arguments and limits for human safety play a role. For existing structures normally a shorter design life is employed, however, as shown in Section 3.1, this does not provide arguments for a reduction of β. On the other hand, increasing safety levels usually involves more costs for an existing structure than for structures that are still in the design phase; this is the reason that from an economical point of view the required safety level for existing structures is lower than for new structures. However the limits for human safety may not be exceeded. Both the economic arguments and the human are discussed in the Sections 4.1 and 4.2.

4.1 Economics

As stated earlier, based on economic arguments, the reliability index for existing structures may be reduced. This is the result of an economic optimization of the total building costs and the product of damage costs and the probability of failure.

The actual reliability index for new wind dominated structures is, as already mentioned, about one unit lower then the standard value of 3.8. Higher design wind loads are not accepted because of economic reasons. From that argument it seems reasonable to propose a level β_u below which existing structures are unfit for use:

$$\beta_u = \beta_n - \Delta\beta \tag{2}$$

with $\Delta\beta > 1.0$. The value $\Delta\beta = 1.5$ is chosen based on a crude study of economic optimization for existing structures [6] and corresponds to commonly accepted target safety levels for existing structures [14,15]. For Eurocode reliability class 2 a reduction by 1.5 means a shift from $\beta = 3.8$ to $\beta = 2.3$ and for the wind dominated cases from $\beta = 2.8$ to $\beta = 1.3$ (life time basis).

For repair a safety level β_r is defined:

$$\beta_n < \beta_r < \beta_u \tag{3}$$

leading to:

$$\beta_r = \beta_n - 0.5 \tag{4}$$

Here in the case of repair, it has to be avoided that too many structures designed according to old building codes and at that time were considered to be safe enough, have to be replaced or radically changed. Therefore the safety level β_r for repair is chosen so that structures that have been designed with old building codes can meet these requirements without any problem. $\Delta\beta = 0.5$ is on the average the difference between the safety levels for the new consequence classes of the Eurocode and the safety levels for the consequence classes of the old national Code.

This leads to the values in Table 4 indicating the economically optimal values for β in the cases of repair and unfitness for use. In this table, according to the Dutch NEN-8700 [13], consequence class 1 from EN 1990 has been subdivided into 1A and 1B, these classes are the same except for the fact that in 1A no danger for human life is present. However, these economic arguments should not lead to concessions in human safety. This is discussed in the following section.

Table 4: β values for repair and unfitness for use; based on only economic arguments

Consequence class	β_n new		β_r repair		β_u unfit for use	
	wn	wd	wn	wd	wn	wd
1A	3.3	2.3	2.8	1.8	1.8	0.8
1B	3.3	2.3	2.8	1.8	1.8	0.8
2	3.8	2.8	3.3	2.3	2.3	1.3
3	4.3	3.3	3.8	3.3	2.8	2.3

wn = wind not dominant

wd = wind dominant

4.2 ***Economics and human safety***

Here, the same limits as presented in Section 3.2 have to be observed. It was explained in Section 3.3 that, for human safety reasons, the Dutch code requires a minimum design life of 15 years; this holds for both new and existing structures. Therefore, the representative loads used in design will be equal to or larger than the 15 years values. Table 3 shows for human safety the required β values based on the minimum design life of 15 years and leading to the maximum annual probability of failure as defined in Section 3.1; these values are also adopted in this section. If the design life for existing structures is larger than 15 years, the required β values for human safety become some smaller. However, for existing structures the design life time is mostly about 15 years and for e.g. 20 years the differences are negligible small. So, here, the 15 year β values are adopted as the human safety requirement, which is a somewhat conservative approximation for existing structures with design lives larger than 15 years.

The same graphs as made in Figures 1-3 can be made in order to observe which β values have to be used. An example is given in Figure 4. For non wind dominated structures CC3, economic optimization would lead to $\beta = 4.3 - 1.5 = 2.8$.

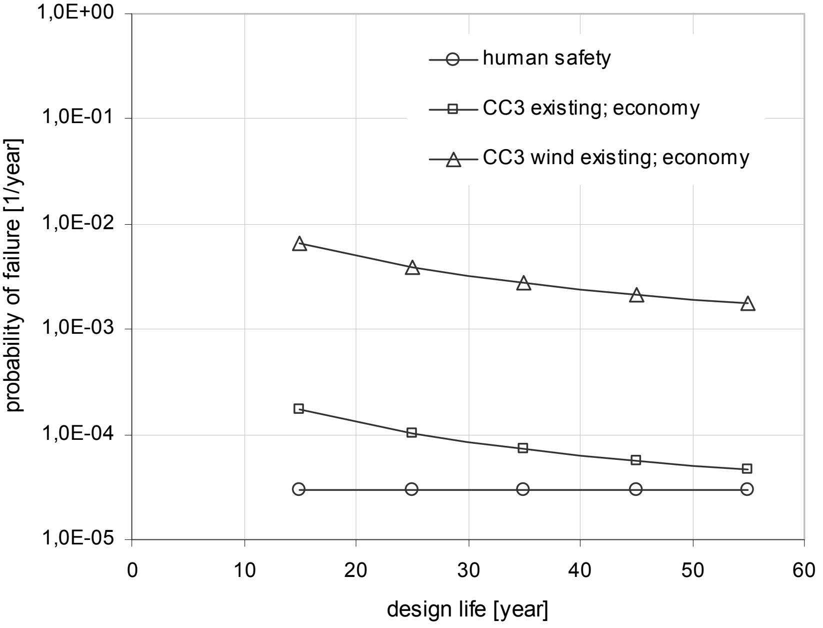

Figure 4: Annual failure probability as a function of the design working life, existing structures CC3

However we see here that for CC3 the human safety criterion is decisive. The β value becomes β = 3.3 (see Table 3, third column), this corresponds to the annual probability of failure of $3 \cdot 10^{-5}$ in Figure 4. For CC2 a similar analysis can be performed, also here the human safety criterion is always decisive. The β value becomes β = 2.5 (see Table 3, third column). For CC1B also the human safety criterion is decisive with β = 2.3. However in order to obtain uniformity, here we also take a minimum reference period of 15 years; in that case the value for the representative variable load increases and the reliability index decreases. It is found $\beta = \Phi^{-1}\{15 \cdot 10^{-2}\} = 1.04$ which is rounded off to 1.1 (see Table 3, third column) for human safety. For CC1A, no minimum reference period is chosen, because human safety is not decisive. The final values for β can be determined by taking each time the maximum of Tables 3 and 4. This has been done in Table 5.

Table 5: Required β – values for the minimum reference period

Consequence class	Minimum reference period	β_n new		β_r repair		β_u unfit for use	
		wn	wd	wn	wd	wn	wd
1A	1 year	3.3	2.3	2.8	1.8	1.8	0.8
1B	15 year	3.3	2.3	2.8	1.8	1.8	1.1*
2	15 year	3.8	2.8	3.3	2.5*	2.5*	2.5*
3	15 year	4.3	3.3	3.8	3.3*	3.3*	3.3*

wn = wind not dominant

wd = wind dominant

(*) = in this case the minimum limit for human safety is decisive

5 Probabilistic calculations for deriving partial factors for existing bridges

In this section probabilistic calculations are used in order to establish the partial factors that are needed to obtain the required reliability for existing bridges with a relatively small span under traffic load. In The Netherlands, the bridges in highways have to satisfy the requirements belonging to consequence class 3, so they must have a reliability index of β = 4.3 for new bridges, β = 3.8 for repair and β = 3.3 for unfit for use. Bridges in less important roads are in consequence class 2 and have to satisfy β = 3.8 for new bridges, β = 3.3 for repair and β = 2.5 for unfit for use.

*5.1 **Set up of the probabilistic calculation for existing bridges***

Considered is a period of 15 years with loading by dead weight and traffic load only. Other loads play a minor role in the design and assessment of traffic bridges. The limit state function is as follows:

$$Z = R - m_G G - m_T T \tag{5}$$

where R is the resistance of a structural element, $m_G G$ is the effect of the dead weight and $m_T T$ is the effect of the traffic load, where m_G and m_T represent model uncertainties. These model uncertainties have been estimated as normally distributed with a mean value of 1.0 and a coefficient of variation of 0.07 for self weight and 0.10 for traffic load. The model uncertainty both covers the schematisation of the load and the determination of the load effect by means of a structural calculation. The weight G has a mean value that is equal to the characteristic value and a coefficient of variation of 7 %. For the traffic load T the maximum weight is used of a truck combination that passes the bridge in a period of 15 years. We can make this assumption because in the case of a relatively small span, a single truck will determine the decisive load on the bridge [8]. The weight of a single truck has the following statistical distribution (units kN):

$$F_V(T) = 0.996 \times \Phi(200, 117) + 0.004 \times \Phi(600, 224) \tag{6}$$

This distribution has been derived from weigh in motion measurements in 2004 on the Dutch highway RW16 near the Moerdijk bridge. In [8, 9, 10] it was shown that this distribution provides values for the traffic load that are comparable to the ones from EN 1991-2, Traffic loads on bridges. In the expression above, $\Phi(\mu, \sigma)$ is the distribution function of the normal distribution with mean μ and standard deviation σ.

The distribution for the maximum truck weight in a period of 15 years follows from:

$$F_{15\text{ year}}(T) = F_V(T)^n \tag{7}$$

For n is used: $n = 15 \times 2 \times 10^6$, the number of trucks passing in 15 year.

The ratio between the self weight of the bridge and the traffic load will be varied in order to study the influence of that ratio on the result. The ratio $T_{\text{rep}} / G_{\text{rep}}$ is called χ. For the strength R a lognormal distribution is taken with a coefficient of variation $V_R = 0.10$. This is a common value primarily for the bending resistance of steel of concrete structures. The mean value of R results from the design value via:

$$R_m = R_d \exp(\alpha\beta V_R) \tag{8}$$

where α is the sensitivity coefficient and β the required reliability index. The design value of the strength is found with help of the basic design formula:

$$R_d = G_d + T_d \tag{9}$$

Here, the maximum of formulas 6.10a and 6.10b from EN 1990 [12] has to be taken.
Format 6.10a (weight dominant):

$$G_d = \gamma_G \, G_{\text{rep}}$$
$$T_d = \Psi_o \, \gamma_T \, T_{\text{rep}}, \quad \text{with } \Psi_o = 0.8 \tag{10}$$

Format 6.10 b (traffic dominant):

$$G_d = \xi \, \gamma_G \, G_{\text{rep}}$$
$$T_d = \gamma_T \, T_{\text{rep}} \tag{11}$$

The representative value of the traffic load T follows from:

$$1 - F_V(T_{\text{rep}}) = 1/n \tag{12}$$

With the above mentioned value for n and the distribution for F_V a value T_{rep} = 1560 kN results. This value is comparable to the representative value for a bridge with a span of about 20 m (the type of spans under discussion here); see [8]. The representative value for G_{rep} is taken equal to T_{rep}/χ, where χ is varied from 0.25 tot 2.0.

5.2 Results for the partial factors

For each set of partial factors γ_G and γ_T (see Tables 6 and 7) the reliability index belonging to the probability of failure P(Z<0) is calculated and plotted as a function of χ = $T_{\text{rep}}/G_{\text{rep}}$. Both the reliability indices for format 6.10a and format 6.10b are plotted.
An example is shown in Figure 5 where for the partial factors belonging to CC3, repair (see Table 6) the reliability level is plotted. In Figure 5, the maximum of 6.10a and 6.10b has to be taken as stated above. It can be observed that the target value in this case of β = 3.8 is not entirely obtained. However in the old Dutch Building Code NEN 6702 a value of β = 3.6 was prescribed for new structures, so this value is accepted for the time being.
The same exercise is performed for CC3, the level at which the structure is unfit for use and CC2, the levels repair and unfit for use. The corresponding partial factors γ_G and γ_T are determined in such a way that a sufficient safety level results from the probabilistic calculations. Finally the partial safety factors that are given in Table 6 and 7 are obtained for use in structural calculations. These values will be incorporated into the Dutch National Annex of EN-1990-Annex A2, Bridges.

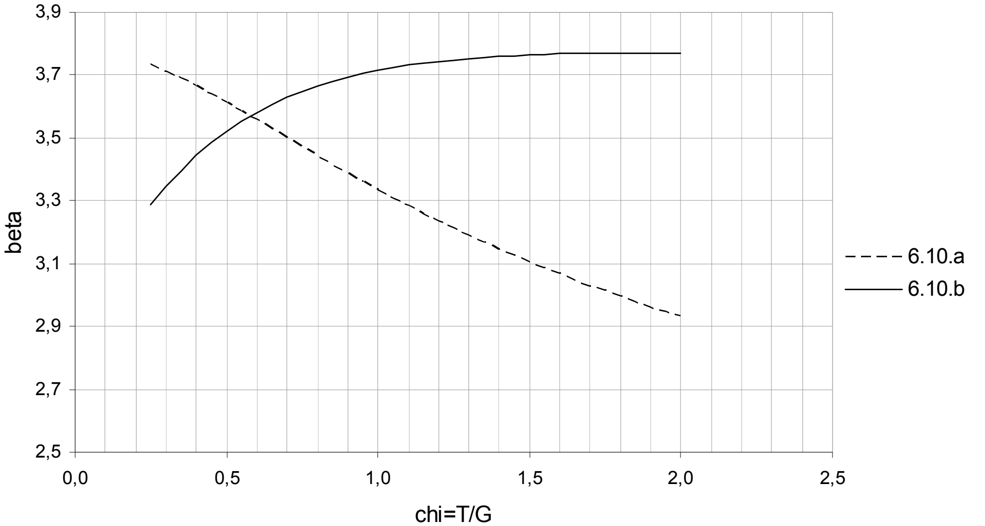

Figure 5: Reliability index for CC3 repair using the proposed rules of verification

Tabel 6: Partial factors for traffic bridges in highways, CC3

	Reference period [year]	Obtained β	Partial factor		
			Weight γ_G 6.10a	Weight $\xi\gamma_G$ 6.10b	Traffic γ_T
new	100	4.3	1.40	1.25	1.50
repair	15	3.6-3.8	1.30	1.15	1.30
disapproval	15	3.3	1.25	1.10	1.25

Tabel 7: Partial factors for traffic bridges CC2

	Reference period [year]	Obtained β	Partial factor		
			Weight γ_G 6.10a	Weight $\xi\gamma_G$ 6.10b	Traffic γ_T
new	100	3.8	1.30	1.20	1.35
repair	15	3.1 – 3.2	1.25	1.10	1.20
disapproval	15	2.7 – 2.8	1.10	1.10	1.10

6 Conclusions

In this article a theoretical background and corresponding results for the safety assessment of existing structures have been presented. The first choice is to compare the structure with the code requirements for newly built structures, either the present ones or the ones at the time of erection. However, there may be many occasions where this level is not reasonable or economically justifiable. In that case authorities may be willing to relax the requirements. Such reduced requirements should be based upon the latest codes, but with reductions in partial factors, design working life and representative load values. The reductions however should not affect human safety. In this article the reliability levels for repair and disapproval of existing structures have been established. For economic reasons a reduction $\Delta\beta = 0.5$ in the reliability index has been proposed for the repair level and a reduction $\Delta\beta = 1.5$ for closing down a structure. In the latter case, however, the limit value introduced for human safety becomes decisive in most of the cases; here a minimum design life of 15 years has to be observed. Subsequently, for existing bridges under traffic load adapted partial factors for weight and traffic load have been established using full probabilistic calculations. These can be used for reassessment of existing bridges under traffic load.

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