Calibration of partial factors in the safety assessment of existing concrete slab bridges for shear failure

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The assessment of the structural safety of existing bridges and viaducts becomes increasingly important in many countries due to the age of the structures and an increase in traffic loads. Many structures need to be reassessed in order to find out whether the safety requirements are met. Most existing standards, however, are developed for the design of new structures. This paper summarises the recent developments with respect to the specification of the target reliability levels for existing structures. It appears from total life cost optimisation that application of the same target reliability levels for existing structures as for new structures is uneconomical. Further, in some cases the cost optimisation seems to yield rather low reliability levels and human safety criteria become decisive for specification of the target reliabilities of existing structures. In this paper old concrete slab bridges without shear reinforcement are studied. Probabilistic calculations are performed in order to calibrate partial factors satisfying the target reliabilities under traffic load. These partial factors can be used by engineers in level I probabilistic calculations. In this way the often over-costly application of safety standards intended for new structures can be avoided in the reassessment of existing structures.

Key words: Slab bridge, shear, safety, partial factors, probabilistic

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 reliability assessment of existing structures differs from new structures in a number of aspects including:

- Increased safety levels usually involves more costs for existing structures than for new structures.
- The remaining working life of existing structures is often different from the standard design working life of 50-100 years assumed for new structures.
- Information on actual structural conditions may be available for the assessment of an existing structure (inspections, tests, measurements).

At present, existing structures are mostly verified using simplified procedures based on the partial factor method commonly applied in design of new structures. Such assessments are often conservative and may lead to expensive upgrades. More realistic verification of the actual performance of existing structures can be achieved by probabilistic methods describing the uncertainties of the load and resistance variables by appropriate probabilistic models.

Specification of the target reliability levels is required for the probabilistic assessment of existing structures. It was recognised by Steenbergen and Vrouwenvelder (2010), Sykora et al. (2011) and Zwicky (2010) that it would be uneconomical to specify for all existing buildings and bridges the same reliability levels as for new structures.

In the following sections, first the shear force assessment of existing concrete slab bridges is discussed. Then, the safety philosophy for existing structures is briefly discussed; the reliability levels in terms of β-values for new structures are given; for existing structures the required β-values are summarised as been derived in Steenbergen and Vrouwenvelder (2010) and Vrouwenvelder and Scholten (2010).

Based on this, for existing concrete slab bridges under traffic load, the partial safety factors for the shear force assessment are derived using a full probabilistic approach.

2 Shear force assessment of concrete slab bridges

In the past, a large number of concrete slab bridges have been built in the Netherlands. Most of the old slab structures have no shear reinforcement. Furthermore the design rules in the design period (until 1974) were mainly based on the service limit state. This is one of the main reasons for reassessing all concrete slab bridges in the Netherlands built in that period. Most of the concrete slab bridges were designed for having two lanes. Sometimes in the design a side lane was added, which could be used in the future as an additional traffic lane. The concrete slab structures which were built in and over the Dutch highways have a relatively short span. For this type of slab structure, four traffic lanes was the maximum span to cross another highway road. In this way the maximum length of the slab structure in this study can be set to 20 m, which can be compared to the minimum length class of the bridge structures according to the Eurocode.

The old slab structures were designed having concrete in the current class C15/20. However, the amount of cement and grain size used in the concrete mix was high and coarse, respectively. Tests on concrete cores obtained from these old bridges now have shown that the actual strength of the concrete is on average C50/60, see Vervuurt et al. (2012). The strength of the reinforcement of the old slab structures was lower than the strength of the reinforcement used today. The yield stress was low (about 220 MPa) and the reinforcement was not ribbed but plain. The reinforcement ratio of old slab bridges is in the range of 0.4-1.2%.

Around 1963, the design traffic load was 600 kN. The corresponding vehicle consisted out of three axles, each with an axle load of 200 kN and axles distances of 1 m and 4 m. The actual vehicle configuration from the Eurocode EN 1991-2 is more compact (two axles of 300 kN and an axle distance of 1.2 m). This is another main reason for reassessing the concrete slab bridges for the shear force capacity. The ratio of dead weight and permanent load versus traffic load is an important factor while determining the reliability in a reassessment of an existing bridge structure.

3 Safety levels

3.1 New structures

Eurocode EN 1990 gives three consequence classes CC1, CC2 and CC3. In Table 1, for new structures, the β-values are provided for these consequence classes. For new structures, the subscript *n* is used for the β-values.

For bridges, important 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. For wind load, smaller target reliability indices are prescribed, see Steenbergen and Vrouwenvelder (2010).

Table 1: Reliability index for new structures (EN 1990, Vrouwenvelder et al. 2011, JCSS, 1996).

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	$\beta_n = 4.3$	$\beta_n = 3.3$

The shortening of the reference period can be observed from two prospectives: the representative value and the partial factors.

A shorter reference period for the variable loads (wind, snow, etc.) results in a decrease of the representative values; the Eurocodes provide expressions to calculate these reductions.

From the point of view of the partial factors, the question is if a shorter design life time justifies an adaptation of the partial factors. Establishing the safety factors for a shorter design life time, both economic arguments and limits for human safety play a role.
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 (Steenbergen and Vrouwenvelder, 2010). As a result the partial factors do not change in case the design life is changed. It is more economical to invest in safety measures if one can profit from it for a longer period of time.

Limits for human safety play an important role here because of the maximum allowable annual probability of failure, calculated for small probabilities through $P_{\text{failure}} = \Phi(-\beta) / T$ with T the design life time in years. This annual probability of failure may not exceed the limits for human safety (ISO 2394, Annex E.4). In Steenbergen and Vrouwenvelder (2010) it has been shown that for short reference periods the limits for human safety become determining and, instead of raising partial factors for short periods, it is derived that for CC2 and CC3 a minimum design life time of 15 year is to be required in structural design.

3.2 Existing structures

In Steenbergen and Vrouwenvelder (2010), two safety levels for existing structures are introduced. β_u is the level below which the structure is unfit for use. β_r is the level for repair of existing structures. Based on both economical arguments and limits for human safety the β-values for existing structures were established.

The values are collected in Table 2 indicating the values for β in the cases of repair and unfitness for use. In this Table, according to the Dutch Code NEN-8700, 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. In Table 2 also the limits for human safety are incorporated via target β-values based on the minimum design life of 15 years and leading to the maximum annual probability of failure accepted for human safety (ISO 2394)

Table 2: β-values for existing structures

CC	minimum reference-period	new β_n		repair β_r		unfit for use β_u	
		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

4 Partial factors for shear force resistance of concrete slabs without shear reinforcement

In this section probabilistic calculations are performed in order to establish the partial factors that are needed to obtain the required reliability for the shear force assessment of existing concrete slab 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 satisfy a reliability index of β = 3.8 for repair and β = 3.3 for unfit for use. According to EN 1990, art. B3.3(2) the partial factor for the resistance γ_m remains the same in all design situations and the partial factors for the load are adapted to satisfy the different safety requirements.

4.1 Set up of the probabilistic calculation

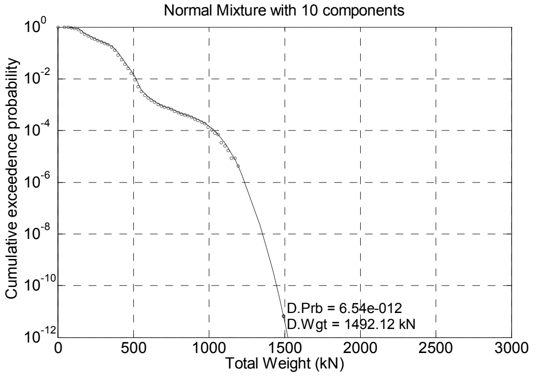
Considered is a remaining life time period of 15 years with loading by dead weight and traffic load only. Other loads play a minor role in the design and assessment of traffic bridges. The limit state function is in general terms:

$$Z = R - m_G\, G - m_T\, T \tag{1}$$

In section 4.3 this limit state will be rewritten in the case of a shear capacity assessment. In expression (1), R is the resistance of a structural element, $m_G\, G$ is the effect of the dead load 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 dead load 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. It is remarked that the large uncertainties associated with the traffic load are taken into consideration in its statistical distribution itself. The load G has a mean value 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 to a large extend the design load on the bridge (Vrouwenvelder et al. (1998)).
The statistical distribution of the weight T of a single truck has been derived from weigh in motion (WIM) measurements in April 2008 on the Dutch highway RW16 near the Moerdijk bridge. The distribution appears to be a mixture of normals (Steenbergen and Morales Napoles, 2012). The density can be written as:

$$f(T) = \sum_{i=1}^{n} \pi_i N_i (T; \mu_i, \sigma_i) \tag{2}$$

Each N_i represent a normal density with mean μ_i and standard deviation σ_i and each π_i is a non negative quantity and all sum to one. In Fig. 1 the distribution is shown, with the empirical (1 month of measurements, in dots) and fitted distribution function. The design weight and the estimated parameters of the mixture of normals (mean μ_i, standard deviation σ_i and mixing proportion π_i) are also shown.
In Steenbergen et al. (2012) 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.



π_i	μ_i	σ_i
7.56E-2	82.11	24.90
3.30E-1	157.18	22.38
3.49E-1	223.64	50.96
2.19E-1	381.82	44.24
2.22E-2	498.87	28.92
2.38E-3	573.94	38.68
6.14E-4	664.86	31.79
3.12E-4	770.74	33.93
1.28E-4	864.33	58.90
3.43E-4	983.16	92.54

Figure 1: Fitted analytical distribution of the vehicle weight, units kN

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

$$F_{15\,\text{year}}(T) = F(T)^{n} \tag{3}$$

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 is 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 shear strength the Eurocode EN-1992 model is used. The statistical distribution belonging to it is obtained from experiments. This is further discussed in section 4.3.

4.2 Design formulas

The design value of the strength is found with help of the basic design formula:

$$R_d = G_d + T_d \tag{4}$$

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

$$G_d = \gamma_G\, G_{rep} \tag{5}$$

$$T_d = \Psi_o\, \gamma_T\, T_{rep}, \text{ with } \Psi_o = 0.8 \tag{6}$$

Format 6.10b (traffic load dominant, partial factor for self-weight is reduced with a factor ξ):

$$G_{\text{d}} = \xi\, \gamma_{\text{G}}\, G_{\text{rep}} \tag{7}$$

$$T_{\text{d}} = \gamma_{\text{T}}\, T_{\text{rep}} \tag{8}$$

The representative value of the traffic load T_{rep} is defined as occurring once in the reference period and therefore follows from $1\text{-}F_{\mathrm{V}}(T_{\mathrm{rep}}) = 1/n$. This value is comparable to the representative value of the traffic load from EN 1991-2 for a bridge with a span of about 20 m (the type of spans under discussion here); see Steenbergen et al. (2012).
The representative value for G_{rep} is taken equal to T_{rep}/χ, where χ is varied from 0.25 to 2.0.

4.3 Shear force model

Point of reference for the shear force model (V=shear force) is the section in the structure where in the ultimate limit state (ULS) the design value of the shear force $V_{S;d}$ is acting. It holds:

$$V_{S;d} = V_{G;S;d} + V_{T;S;d} \tag{9}$$

with $V_{S;G;d}$ the design value of the shear force caused by the dead load and $V_{S;T;d}$ is the design value of the shear force caused by the traffic load.
The design value of the shear force strength $V_{R;d}$ is:

$$V_{R;d} = \tau_{1;d}\ b\ d\ = \tau_{1;rep}/\gamma_m\ b\ d \tag{10}$$

Here $\tau_{1;d}$ is the design value of the shear stress in the section where shear failure occurs. $\tau_{1;rep}$ is the corresponding representative value of the ultimate shear stress and γ_m is the corresponding material factor. b is the width of the considered section and d is the effective depth. With a unity-check 1.0 the criterion for the shear strength is just met, so:

$$V_{S;d} = V_{R;d} \tag{11}$$

Then it holds:

$$\gamma_G\ V_{G;S;rep} + \gamma_T\ V_{T;S;rep} = \tau_{1;rep}/\gamma_m\ b\ d \tag{12}$$

Therefore:

$$b\ d = (\gamma_G\ V_{G;S;rep} + \gamma_T\ V_{T;S;rep})\ /\ (\tau_{1;rep}/\gamma_m) \tag{13}$$

The overall safety is judged on the basis of the reliability index β that is determined from a probabilistic analysis on the basis of the limit state function:

$$Z = \tau_1 \; b \, d - (m_g \, V_G + m_T \; V_T) \tag{14}$$

Substituting (13) in (14) provides:

$$Z = \tau_1 \, (\gamma_G \; V_{G;S;rep} + \gamma_T \; V_{T;S;rep}) / (\tau_{1;rep} / \gamma_m) \; - \; (m_g \, V_G + m_T \; V_T) \tag{15}$$

The reliability index resulting from the evaluation of the limit state function is compared with the required values from Table 2. The partial factors for the load γ_G and γ_T are tuned so that the required β-values are obtained.

The representative value of the shear strength $\tau_{1;rep}$ and the corresponding material factor γ_m are obtained from EN 1992-1-1. The shear strength τ_1 is a stochastic parameter that is described by empirically found relations.

4.3.1 Representative value of shear strength

The considered slab bridges are without shear reinforcement so here art. 6.2.2(1) of NEN-EN 1992-1-1 is used.

The design value of the shear strength, expressed as nominal shear stress, is:

$$\tau_{1,d} = \tau_{1,rep} \; / \; \gamma_m \tag{16}$$

with:

$$\gamma_m = 1.5 \text{ (partial factor resistance)} \tag{17}$$

$$\tau_{1,rep} = 0.18 \;\; k \;\; (100 \;\; \rho_l \;\; 0.85 \;\; f_{ckc})^{1/2} \tag{18}$$

where $k = 1 + (\,200\,/\,d\,)^{1/2} < 2.0$ with d [mm] the effective depth of the section, here we assume $d = 0.9 \cdot h$ with h the depth of the section. For the longitudinal reinforcement it holds $100\,\rho_l = \omega_0 < 2.0$ In (18), f_{ckc} is the characteristic cylinder compressive strength in MPa, determined from tests on cylinders taken from the existing structure. Conform EN 1992-1-1, the cylinder compressive strength f_{ckc} is equal to 0.82 times the cube compressive strength f_{ck}.

In expression (18), a long term factor of 0.85 is applied directly to the measured compressive strength. This is done according to NEN-EN 1992-1-1 (article 3.1.2(4)) when the strength is determined on specimens at an older concrete age than 28 days.

4.3.2 Distribution function of the shear stress

In CEB-bulletin 224, "Model uncertainties" the following expression is given for the mean value of the shear strength of slender structures:

$$\tau_{1;m} = 0.163 \; k \; (\omega_0 \; 0.85 \; f'_{cc;m})^{1/3} \tag{19}$$

where:

$$k = 1 + (200 / d)^{1/2} < 2.0; \quad d = 0.9\, h \tag{20}$$

ω_0 is the longitudinal reinforcement [%] with a maximum of 2.0; $f'_{cc;m}$ is the mean cylinder compressive strength in MPa and $f_{cc;m} = 0.82 \; f'_{c;m}$. So, expression (19) can be rewritten as:

$$\tau_{1;m} = 0.163 \; \{1 + (0.22 / d)^{1/2}\}(\omega_0 \; 0.82 \times 0.85 \; f'_{c;m})^{1/3} \tag{21}$$

Expression (19) is the result of laboratory experiments on 176 beams and the coefficient of variation was 0.13 as found by König and Fisher (1995). In the present study, a part of the variation is caused by the deviation of the real compressive strength of the concrete in the beam from the presumed value based on compressive tests on cylinders and cubes. Because in these test concrete was used that was made in the laboratory, it is assumed here that the coefficient of variation of the compressive strength in these test was about 0.05. This means that the coefficient of variation in the formula itself can be derived to be 0.12: combining V=0.12 and the mentioned coefficient of variation in f'_c (in expression (19) to the power 1/3) provides by good approximation a total coefficient of variation of 0.13 around the mean result of expression (19).
The expression of the shear force τ_1 that is used in the limit state function (15) becomes:

$$\tau_1 = f_M \; 0.163 \; \{1 + (0.22 / d)^{1/2}\} \; (\omega_0 \; 0.82 \; 0.85 \; f'_c)^{1/3} \tag{22}$$

For the factor f_M a lognormal distribution is assumed with a mean value of 1.0 and a coefficient of variation of 0.12 (see above).

To obtain the mean value of the stochastic parameter f'_c, the experimentally determined characteristic cube compressive strength is augmented with $1.64 \cdot 10$ MPa as the characteristic value is defined as the 0.05 fractile of a normal distribution. A large research program on existing Dutch existing concrete bridges has learned that a standard deviation of 10 MPa has to be used for the cube compressive strength, see Steenbergen and Vervuurt (2012).

4.3.3 Parameter study

In order to derive load factors that are valid for a broad application, different possible parameters combinations are used; they are listed in Table 3. For these combinations; limit state function (15) is evaluated using EN 1990 formats 6.10a and 6.10b. The results are discussed in the next paragraph.

Table 3: Parameter combinations

χ = ratio traffic/dead load	0.25-2.0 with steps of 0.05
Concrete strength	C28/35, C35/45, C40/50, C45/55 and C53/65
Reinforcement ratio	0.4%, 0.6%, 0.8%, 1.0% and 1.2%
d	0.5 m, 0.75 m, 1.0 m and 1.25 m

5 Results

For CC3 for different parameters combinations the β-values are calculated for the EN 1990 formats 6.10a and 6.10b. For this FORM calculations are performed using the software program Prob2B, see Courage and Steenbergen (2007). The results are plotted as a function of $\chi = T_{rep} / G_{rep}$. The intersection point of the two lines corresponds to the minimum value of the reliability index because in design the maximum of 6.10a and 6.10b is used. An example is given in Figure 2.

A series of similar calculations have been performed in order to derive that load factors γ_G and γ_T that guarantee the required β-values for all the parameter combinations in Table 3. The governing parameter combination of Table 3 was used for the final values for γ_G and γ_T; so in all cases of Table 3 the target reliability index are satisfied. In tuning γ_G versus γ_T it

was made sure that both values were lowered to the same extend with respect to the values for new structures. The final values of γ_G and γ_T are collected in Table 4.
It is observed that the obtained β-values are a little larger than required in Table 3. This is in accordance with the fact that shear failure is considered to be more brittle than failure due to bending and therefore one tends to take into consideration a somewhat higher safety level, because of the larger failure consequences in the case of brittle failure.

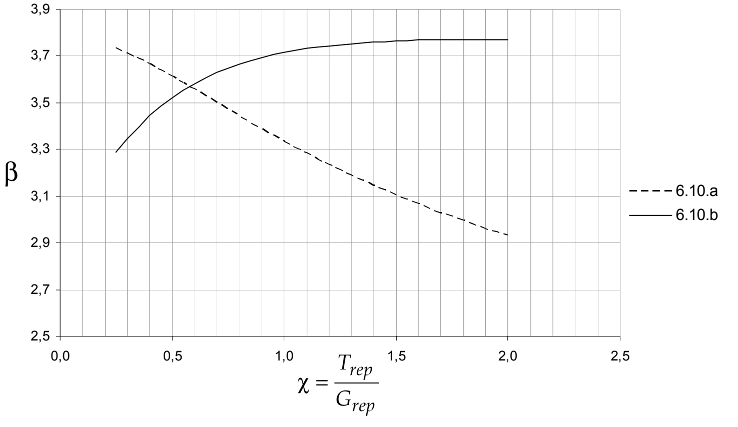


Figure 2: Calculation of the reliability index for the formats 6.10a and 6.10b

The load factors in Table 4 are now being used for the reassessment of concrete slab bridges with a relatively small span. In Steenbergen and Vrouwenvelder (2010) it is proven that these partial factors are also valid for other failure mechanisms such as bending.

Table 4: Load factors for existing structures

	Reference period [year]	Obtained β	Partial factor		
			Weight γ_G 6.10a	Weight $\xi\gamma_G$ 6.10b	Traffic γ_T
Repair	15	3.8	1.30	1.15	1.30
Unfit for use	15	3.4	1.25	1.10	1.25

6 Conclusions

In this article a theoretical background and corresponding results for the safety assessment of existing structures have been presented. Subsequently, for existing concrete slab bridges under traffic load, adapted partial factors for weight and traffic load have been established using full probabilistic calculations. These partial factors are lower than the ones used for new concrete structures. They are now being used for the reassessment of existing bridges under traffic load. Using this method; it turns out that quite a large number of existing old concrete slab bridges under traffic load still satisfy the safety requirements and do not need to be replaced.

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