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Publication date	2019-01-01
Publication information	Hanley, Ciarán, Dan M. Frangopol, Denis Kelliher, and Vikram Pakrashi. “Variations of Safety Factors for Bridges over Their Lifetime Considering Changing Live Load Definitions,” 2019.
Conference details	40th IABSE Symposium, Nantes, France, 19-21 September 2018
Item record/more information	http://hdl.handle.net/10197/10792

Variations of safety factors for bridges over their lifetime considering changing live load definitions

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Abstract

Design of long lasting structures would require insights from the past. In this regard, design of future long-life bridges and the assessment of current ageing bridges can particularly benefit from understanding the evolution of safety over lifetime. This is an important issue since there has been an evolution of design live loads over the lifetime of such bridges in the past and such changes in definition can influence the computed estimates of safety; while the true state of the structure remains the function of the site-specific live loads. Considering the limitations around instrumenting individual bridges, it is important to investigate the variations of such computed estimates over their lifetime. Previous studies around this problem have focused on the changes in terms of reliability indices over time and life-cycle costs due to degradation. However, the key change in safety estimates for owners of bridges based on changes in live load definitions will be reflected in terms of safety factors, which remain a key index for taking commercial and safety intervention decisions for a bridge stock. This paper presents the variation in safety factors of representative slab, beam and prestressed bridges for changing live load definitions and compares it with corresponding changes in reliability index parameter importance factors for the life-cycle of such bridges. The work provides an insight to how future changes in anticipated changes in live loading definitions can influence safety factors and provides guidance around better built bridges for the future.

Keywords: Bridge, Live Load, Reliability Index, Life-Cycle, Safety Factor, Slab, Prestressed, Beam, ageing.

1 Introduction

Structural assessment techniques are central to the selection of intervention options for deteriorating bridges to maintain an adequate level of safety throughout the network. Structural reliability methods [1] are popular for quantifying the safety of structures over the lifetime of both individual and networks of bridges [2–8]. The effective allocation of capital resources seeks to minimise the inherent risks associated with investments through the use of advanced methods; and reliability methods are an effective tool for the monitoring of an asset base and, thus, allowing the prioritisation of intervention and investment requirements in a more careful and rational manner. Intervention can be focused to address the most important parameters that govern the safety of the bridges, as highlighted by the parametric sensitivity and parameter importance factors, which are beneficial by-products of reliability assessments [9].

Performance indicators are used as a significant decision tool when evaluating intervention options when structural safety is of primary concern. Even after considering a full probabilistic regime, it is important to assess how the markers of safety, expressed as a reliability index or other performance indices, have changed over time with changing benchmarks of live loading [10]. Site-specific live loading, related to extreme value distributions fitted to assumed or observed data has shown to have significant potential for assessing the effects of live loading [11–15]. Not using such methods can thus misinform bridge managers and stakeholders by significantly underestimating the true performance measure of the bridges within their networks. In most European countries, the basis of assessment calculations is the same as for the design of new bridges [16].

2 Assessment Methodology

2.1 Normative Traffic Load Variation

The evolution of normative traffic load specifications in the UK and Ireland, from the suggestion of nominal wheel loads to a standard loading curve (SLC), is detailed at length by Dawe [17]. While many minor changes to these normative documents have been made in the past century, the five major changes and their effect on structural safety have been discussed by Hanley et al. [10,18]. The significant changes in normative code were shown to be BS 153 [19], BS 5400 [20], BD 21/84 [21], BD 37/88 [22], and the introduction of the Eurocode [23].

As can be seen in Figure 1 where the Eurocode loading can produce a bending moment of as much as three times that of the initial bending moment M_0 due to BS 153 in a simply support span. This ratio can be expressed as M_x/M_0, where M_x is a bending moment produced by a subsequent normative standard x, and reduces to 2 at 13 m under Eurocode and 1.25 at 50 m. For the 16 m spans considered in this paper, the ratios of M_x/M_0 for BS 5400, BD 21/84, BD 37/88, and Eurocode are 0.96, 1.36, 1.43, and 1.86, respectively.

Throughout this change in live load definitions, there has been no substantive change in the capacity models or in the partial factors for materials γ_m; for which the concrete factor $\gamma_{m,c}$ for the ultimate limit-state is 1.5 and the reinforcing and prestressing steel factor $\gamma_{m,s}$ is 1.15, for both the British Standards [24,25] and Eurocode [26].

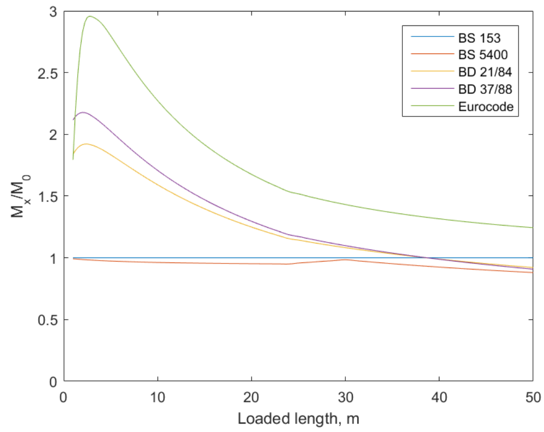

Figure 1. Maximum bending moment ratio M_x/M_0 with increasing spans for changing live load definitions

2.2 Structural Modelling

In order to assess the effects of uncertainty on the reliability analysis of a number bridges and bridge networks, three single span, simply supported bridges were evaluated as a case study. The bridges used for the analysis have reinforced concrete slab, reinforced concrete beam-slab, and prestressed concrete beam construction; of which is largely seen in national bridge stock in Ireland [27]. The general arrangement and cross-sections of these bridges can be seen in Figure 2, and they all comprise a span length of 16 m (Table 1).

Table 1. General bridge dimensions

Attribute	Value
Number of spans (No.)	1
Overall length (m)	16
Width out-to-out (m)	10.4
Width of footway (m)	1.5
Width of carriageway (m)	6.4
Road surface (mm)	100
Number of lanes (No.)	2

This span was chosen with regard to the available probabilistic load model. This was developed by Chryssanthopoulos et al. [28] and Cooper [29], and was derived as a static load model with a uniformly distributed load (UDL) of 27 kN/m and 2 axle loads of 300 kN each, factored by a statistically defined variable λ_{Prob} with a Gumbel distribution; extrapolated from WIM data on motorway bridges in the UK. The variable λ_{Prob} corresponds to a major road that experiences a traffic volume flow per direction per day of 10,000 [30]; with the statistical parameters defined in Hanley [10] being one year parameters for a Gumbel distribution.

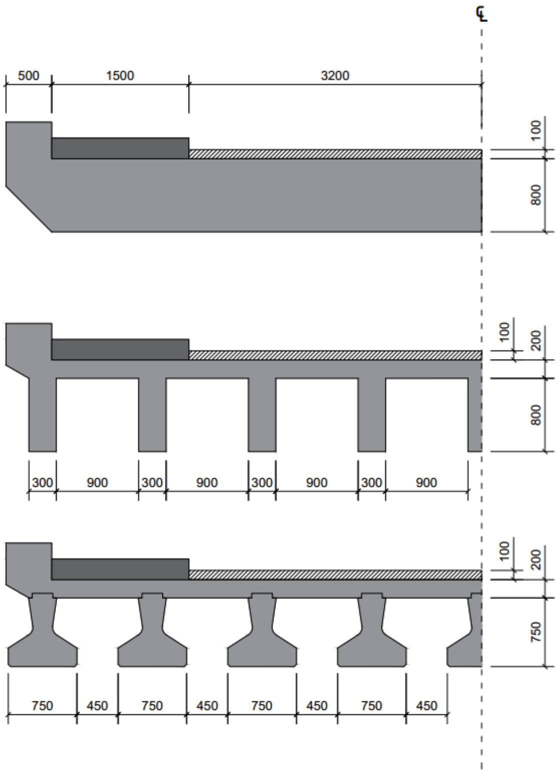

Figure 2. General arrangement of bridges

In this assessment, the flexural limit-state g was analysed; having been identified as the critical limit-state in recent assessments carried out in Ireland [29]. The flexural capacity Mu was tested against the bending moment effects of the self-weight of the bridge MDL, the superimposed dead load of the road surface MSDL, and the various bending moments produced by changing traffic load specifications MLL.

$$g = R - S = M_u - M_{DL} - M_{SDL} - M_{LL} \qquad (1)$$

For computational efficiency, the limit state equations are expressed in parametric form [2], whereby the random variables X_{ij} and the deterministic parameters Y_{ij} are decoupled, and groups of Y_i are combined into deterministic constant coefficients C_{ij} in the limit state equations. The limit state functions for the reinforced concrete slab, reinforced concrete beam, and prestressed concrete beam bridges under consideration are defined, respectively, as:

$$g_{slab,m} = \left(C_{01} A_s f_y \gamma_m \lambda_d - C_{02} \frac{A_s^2 f_y^2 \gamma_m}{f_c}\right) - C_{03}\lambda_c - C_{04}\lambda_s - C_{05}\lambda_{prob} \qquad (2)$$

$$g_{beam,m} = \left(C_{11} A_s f_y \gamma_m \lambda_d - C_{12} \frac{A_s^2 f_y^2 \gamma_m}{f_c}\right) - C_{13}\lambda_c - C_{14}\lambda_s - C_{15}\lambda_{prob} \qquad (3)$$

$$g_{pres,m} = \left(C_{21} A_{ps} C_{23} f_{pu} \gamma_m \lambda_d - C_{22} \frac{A_{ps}^2 C_{23} f_{pu}^2 \gamma_m}{f_c}\right) - C_{24}\lambda_c - C_{25}\lambda_s - C_{26}\lambda_{prob} \qquad (4)$$

Where the random variables A_s, A_{ps}, f_c, f_{pu}, f_y; the uncertainty factors λ_x and γ_m; and the deterministic constant coefficients C_{ij}, being functions of the deterministic parameters, are defined elsewhere [31–34].

2.3 Life-cycle deterioration

The corrosion model used in the lifetime assessment of the bridges was based on a uniform reduction in flexural steel area, assumed here to be caused by chloride only [34]. The time to initiation of corrosion T_i is commonly obtained using Fick's 2nd law of diffusion [32,33,35]:

$$T_i = \frac{C^2}{4D_c}\left[\mathrm{erf}^{-1}\left(\frac{C_s - C_{cr}}{C_s}\right)\right]^2 \tag{5}$$

where C is the concrete cover to flexural reinforcement (mm); C_{cr} is the critical chloride concentration (%); C_s is the surface chloride concentration (%); D_c is the chloride diffusion coefficient (mm^2/year); and erf is the error function. In this analysis, C_{cr}, C_s, and D_c are treated as random variables with a lognormal distribution; with values (μ,σ) of (0.037,0.0056), (0.15,0.015), and (110,12.1), respectively [36]. While this represents only one method for modelling corrosion, its use is adequate here for establishing a comparative discussion across the various traffic load models. Once the time to corrosion initiation is determined, time-variant flexural steel $A_s(t)$ area can be found as:

$$A_s(t) = \frac{\pi}{4}\sum_{j=1}^{n}\left[D_{0,j} - \Delta D_j(t)\right]^2, \qquad \Delta D_j(t) = r_{corr}(t - T_i) \tag{6}$$

where $D_{0,j}$ is the initial diameter of the steel bars and strands; $\Delta Dj(t)$ is the amount of section lost after time t; n is the number of bars; and r_{corr} is the rate of corrosion of the flexural steel. While r_{corr} is a function of the constant rate in time i_{corr} and the corrosion coefficient value C_{corr}, here r_{corr} (mm/year) is modelled as random variable with a lognormal distribution, with a mean μ and standard deviation σ of 0.0762 and 0.0223 for the RC bridges [32], and 0.0571 and 0.017 for the PC bridge [33].

Using Equation 5, the time to corrosion initiation T_i was evaluated using a Monte Carlo simulation of 50,000 samples, and fitting a lognormal distribution as a good estimate [36]. The mean value of T_i for both RC bridges was 24.1 years, and for the PC bridge is 15.4 years for the first layer of steel and 51.8 years for the second layer of steel. The loss of cross-sectional area of flexural steel is the considered damage effect for the probabilistic and deterministic life-cycle assessment in the next section.

3 Results

3.1 Variation of Safety Indices

The effect of corrosion on the reliability index for the three bridges can be seen in Figure 3 where the time-varying reliability index is presented for the probabilistic load assessment. Included in the figure is a normative safety index based on code-defined loading; including 'jumps' in this safety index that account for the changing code specifications over time. For the flexural limit-state, this safety index is defined as the ratio between the maximum moment under traffic loading M_d and the moment capacity of the section M_u; where values in excess of 1 represent failure of the limit-state under a normative LRFD assessment. The 'jumps' in safety index represent periods where code-defined traffic load models changed to a newer model, and the rapid change in safety index are thus expected when using only these normative models.

For the RC slab bridge, the initial reliability index under probabilistic loading is 3.68, where it remains at this level until the onset of corrosion, and degrades to a final β of 2.05 over the 80 year period. For the same duration, an LRFD assessment would yield an initial safety index of 0.78 under BS 153, and have a final safety index of 1.36 under Eurocode; representing a load model producing a bending moment that is 36% over capacity of the section. If BS 153 was used at year 80 for the LRFD assessment, the safety index would be 1.03; just 3% over estimated capacity and a 33 percentage point (pp) difference under the newer Eurocode model. Similar results can be seen for the RC beam and prestressed concreted bridges; where the beam bridge has a β of 4.79 at year 0 and 2.29 at year 80, and the prestressed concrete bridge has a β of 4.92 at year 0 and 3.51 at year 80.

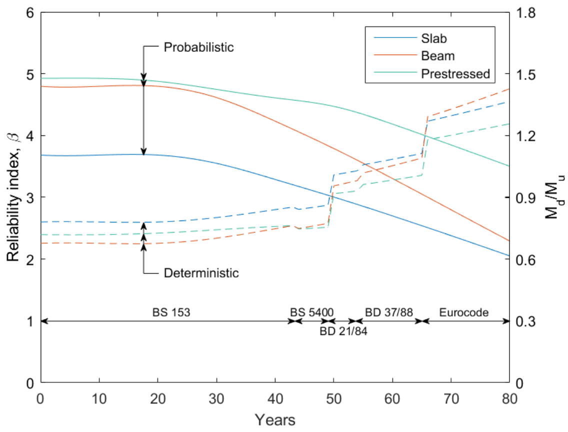

Figure 3. Life-cycle reliability index and normative safety index for flexure

For the normative LRFD assessment, the RC beam bridge has an initial safety index of 0.68 under BS 153 and 1.43 at year 80 under Eurocode. Again, the variation in the safety index at year 80 is significant, with a 45 pp difference to BS 153 for a safety index of 0.98. For the prestressed bridge, the year 0 and 80 safety indices were 0.72 and 1.26, respectively; where the 1.26 was a 38 pp difference over a year 80 safety index of 0.88 under BS 153. While these end variations in safety index are somewhat large, it is perhaps an inappropriate comparison to compare the safety indices determined under BS 153 and Eurocode, as these codes never operated sequentially. Thus, it is important to consider these 'jumps' in safety index as these occur in the transition from one code to the next.

Two large jumps are seen over the 80 year period; when BD 21/84 replaced BS 5400 and when Eurocode replaced BD 37/88. When BD 21/84 was introduced, the revised safety indices for the slab (1.01), beam (0.96), and prestressed (0.92) concrete bridges increased by 15 pp, 19 pp, and 17 pp over their BS 5400 values, respectively. This change caused violation of the flexural limit-state for the slab bridge, and near violation for the beam and prestressed bridges. When the next jump occurs, at the introduction of Eurocode, these limit-states would have been violated due to the ongoing deterioration due to corrosion, if rehabilitation hadn't taken place. Conversely, at the time of these 'jumps' (years 49 and 65), reliability assessment under the probabilistic load model gives values of β for the slab, beam, and prestressed bridge of 3.03, 3.83, and 4.49, respectively, at year 49, and 2.54, 3.05, and 4.02, respectively, at year 65. As maintenance and intervention decisions are often based on performance indicators such as β or the safety index, more commonly, the decision to intervene structurally on a bridge can be taken too hastily when code-defined loading is used instead of probabilistic loading, and lead to the misallocation of budgetary resources. Thus the use of β as the performance indicator over the lifetime of the bridge appears to provide a more stable assessment of safety than the normative LRFD assessment.

3.2 Effects on Importance Factors

While the effects of deterioration on β and safety indices have been shown over time (Figure 3) it is also worth examining the effect deterioration has on the importance factors α_i^2. Specifically, it is interesting to note the effect that this deterioration has on the ranking of the random variables in regard to α_i^2 (Figure).

Importance factors α_i^2 were determined to allow the relative ranking of random variables to aid the assessment process. These factors highlight those random variables which have the greatest influence on β, and thus which variables it would be beneficial to reduce the level of uncertainty. Random variables with low importance factors can afford to be modelled as deterministic parameters, without significant change in the computed β. Those with high importance factors should be prioritised when more detailed material assessments are deemed necessary.

For the reinforced concrete slab bridge, it can be seen that the variable with the highest important factor is $X_{i5}(\lambda_c)$ in the undamaged state. This is due to the fact that the slab bridge contains a larger portion of concrete per unit width than a beam bridge, and thus the unfavourable effect of the self-weight of the bridge is more pronounced; as can be seen with the corresponding important factors for X_{i5} in the reinforced concrete and prestressed beam bridges. Conversely, it can be seen that the other component of the bridge self-weight, in the form of the superimposed permanent load of the road surface X_{i6} is seen to be more of a significant factor for these two bridges, and it is deemed largely unimportant for the slab bridge.

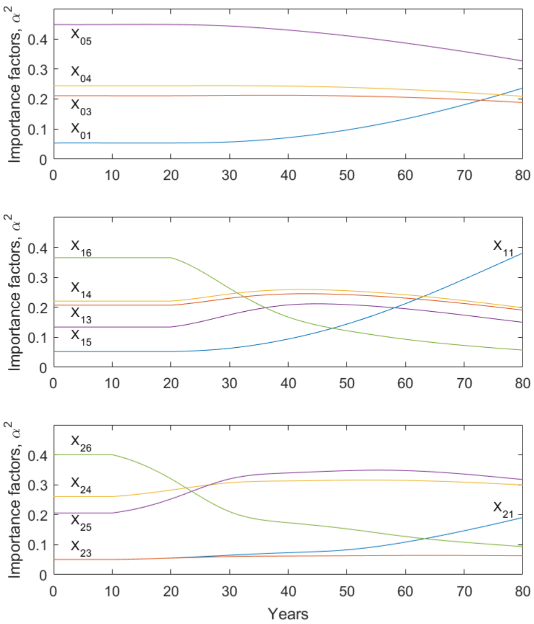

Figure 4. Variations in life-cycle importance factors for the a.) slab, b.) beam, and c.) prestressed concrete bridges

Further remaining large importance factors are seen for the variables $X_{i3}(f_{y,pu})$ and $Xi3(\gamma_m)$, however less importance is seen on the variable $X_{33}(f_{pu})$ for the prestressed concrete bridge. For all bridges, $Xi2(f_{cu})$ has a very low importance factor and can modelled deterministically without much adverse effect on the reliability model. When allocating resources for determining information to be included in the assessment model, these results would suggest that any chemical inspections or non-destructive testing of these bridge types should focus entirely on evaluating an accurate model for the PDF of $A_{s,ps}$ and f_{pu}, and it would be considered unnecessary to establish anything more than initial estimates of the properties of f_{cu}.

It can be seen that the variable X_{i1}, corresponding to the area of reinforcing or prestressing steel $A_{s,ps}$, experiences an increase in importance over time; this increase occurring after the onset of corrosion in the bridge. As $\Sigma\alpha_i^2 = 1$, any increase in α_i^2 for a specific variable is done so at the expense of other variables included in the model.

This is most prominent in the RC beam bridge, where X_{11} experiences its greatest increase of the three bridges, and is largely due to the decrease in α_i^2 of X_{16}; being the surfacing weight uncertainty factor λ_s. This importance factor is quite high, due to the large CoV of 0.25 in the initial model. As the ongoing deterioration increases the uncertainty in X_{11}, it can be seen that X_{16} degrades to an importance measure that is more in line with its actual effect on the bridge, qualitatively. Interestingly, this same decrease in s is seen in the prestressed bridge for the variable X_{16}, except in this case, X_{21} does not increase at a commensurate rate.

This can be attributed to the prefabricated nature of prestressed beams, where there is less uncertainty in A_{ps} and while it is still subject to degradation, its uncertainty is not affected to the extent of it reinforced concrete counterparts. Instead, the concrete weight uncertainty factor $\lambda_c(X_{25})$ gains a measure of importance over time, which represents uncertainty in the permanent load of the bridge. As the bridge deteriorates and loses its flexural capacity, the effect of the self-weight of the bridge can become more of a consideration, thus explaining its increase in importance. It is noteworthy that the grade of steel used in the reinforcing and prestressing X_{i3} does not gain any significant importance over time.

4 Conclusions

A comparative analysis conducted using limit-states demonstrated the effect of changing definitions of code-defined traffic loading on safety classifications of bridges. The results were compared with those for site-specific probabilistic loading to determine how representative the safety classification for a bridge assessed under specified loading was against a more realistic loading scenario. While the inclusion of deterioration modelling in the deterministic assessment is beneficial for assessing future life and durability prediction, it cannot account for future changes in normative documents.

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