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Reliability and parameter index considering changes in bridge traffic loading definitions

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Abstract

With the continued evolution of traffic loading specifications, safety classifications of bridge structures are subject to change, independent of the actual condition of the structures at that point in time. As investment decisions are often based on these safety classifications, a reclassification of safety level due to changing of traffic load definitions can lead to misinterpretation of the actual state of the structure, and thus lead to a misallocation of resources. Should a reclassification of safety occur after a change in traffic load specification, the question as to whether modern design codes are producing more or less robust bridges than previous design codes is raised. To investigate this, three bridge structures were assessed for evolving definitions of traffic load. Using deterministic and probabilistic methods, critical limit-states were assessed and the associated reliability indices and parametric sensitivity factors were determined and compared across various code specifications. This comparison allowed for the evaluation as to how the evolution of traffic load over time influences the computed safety of bridge structures.

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1 Introduction

Quantification of structural safety and redundancy for bridges is an important process in network maintenance management (Akgül and Frangopol, 2003; Frangopol and Nakib, 1991; Weninger-Vycudil et al., 2015) and is strongly dependent on the effects of traffic loading (Nowak et al., 1993; Nowak, 1993). Markers of quantification have evolved from basic definitions of allowable stress indices, to limit-state design, and, eventually, to fully probabilistic reliability analysis (Ellingwood, 1996; O'Connor and Enevoldsen, 2007; Dawe, 2003). While new bridge structures conform to and benefit from the acknowledgement of epistemic and aleatory uncertainties (Ang and Tang, 2007) through normative documents (Cornell, 1969; Benjamin and Lind, 1969; Shah, 1969; Lind, 1972; Rosenblueth and Esteva, 1972), much of the global bridge stock originate from a time when the design of structures was based on basic models and engineering judgement.

A review of the national bridge stock in six European countries showed that the majority of bridges were built in the post-war period of 1945–1965 (Žnidarič et al., 2011), while in the United States, the average age of the national bridge stock is 42 years, 11% of which is said to be structurally deficient and 25% said to be "functionally obsolete" (ASCE, 2013). On the other hand, there has not been sufficient funds for owners of bridge stock to replace, intervene, or even prioritise investment (Ellingwood, 2005; Frangopol, 1999, 2011; Frangopol and Liu, 2007; Pakrashi et al., 2011; Frangopol and Bocchini, 2012).

Performance indicators are used as a significant decision tool when evaluating intervention options when structural safety and redundancy are of primary concern (Frangopol and Nakib, 1991; Frangopol and Estes, 1997; Saydam and Frangopol, 2011; Frangopol and Saydam, 2014; Saydam and Frangopol, 2015; Dong and Frangopol, 2016; Frangopol and Soliman, 2016; Sabatino et al., 2016; Zhu and Frangopol, 2016; Frangopol et al., 2017). Even after considering a full probabilistic regime, it is important to assess how the markers of safety, expressed as a reliability index β or other performance indices, have changed over time with changing benchmarks of traffic loading. The evolution of such indices over time, combined with degradation patterns and maintenance intervention is yet to be investigated. Site-specific traffic loading, related to extreme value distributions fitted to assumed or observed data, through weigh-in-motion (WIM) technology, has shown to have significant potential for assessing the effects of traffic loading (O'Connor et al., 2001; O'Connor and O'Brien, 2005; Caprani and O'Brien, 2010;

O'Brien et al., 2015a,b). However, too often is the performance of bridges within a network, and thus economic decisions made regarding intervention options, determined using generalised normative descriptions of traffic loading that are subject to change over time. The use of such methods can thus misinform bridge managers and stakeholders by significantly underestimating the true performance measure of the bridges within their networks.

In this paper, a brief history of the major bridge design and assessment standards will be presented, and the effect of the various definitions of normative traffic loading will be shown on the performance indicators, in this case the reliability index β (Ditlevsen and Madsen, 1996; Melchers, 1999; Pakrashi and Hanley, 2015), of three simply supported concrete bridges of the same span. These changes will be benchmarked against β from site-specific traffic loading, and the effect changing normative traffic loading has on the probabilistic model will be shown through parametric sensitivities and importance factors (Madsen et al., 1986). The type of bridges used in this assessment were chosen based on their proliferation within mainland Europe and the UK (Žnidarič et al., 2011). An 80 year reliability assessment is also presented, showing how β can transition below a minimum acceptable threshold at a single point-in-time due to normative changes couple with typical degradation effects.

2 Evolution of Normative Traffic Loading

Prior to the latter 19th century, traffic loading on bridges was not of primary concern to the bridge builder, as this load was considered light relative to the self-weight of the structure itself (Henderson, 1954). It was due to the emergence of the traction engine that the effect of traffic loading on bridges became an important design criteria. 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 (2003) and is summarised in Table 1. While many minor changes to these normative documents have been made in the past century, the five major changes will be discussed in this paper; *BS 153* (BSI, 1937), *BS 5400* (BSI, 1978), *BD 21/84* (Highways Agency, 1984), *BD 37/88* (Highways Agency, 1988), and the introduction of the *Eurocode* (CEN, 1994).

2.1 BS 153

BS 153–Standard specification for girder bridges (BSI, 1937) was developed by the British Standard Institution (BSI) in 1937 for the design and construction of girder bridges, *part 3* of which dealt with the application of traffic loading. The standard recommended the use of a standard loading train (SLT) with a unit load of 1 ton/axle, and 15 units to be applied per 10 ft of lane width, and a 10 ft headway between vehicles. Additionally, it was specified to apply a uniformly distributed load (UDL) of 4.02 kN/m^2 (84 lb/ft^2) to account for pedestrians and light traffic. Further revisions of this standard introduced what is now known as 'abnormal' loading, with the previous loading being referred to as 'normal' loading, as well as the increase in applied units from 15 to 22 to account for general traffic increases. Furthermore, computational ease was improved with the introduction of a standard loading curve (SLC) to replace the standard loading train. The SLC specified a UDL as a function of span, with a higher UDL for shorter spans to account for the increased likelihood of a single span being fully loaded by trucks. Additionally, a knife-edge load was to be applied across the lane width of 39.4 kN/m (2700 lb/ft) at a location within the span to produce the worst shear force effect.

2.2 BS 5400

The introduction of *BS 5400–Steel, concrete, and composite bridges* (BSI, 1978) in 1978 transitioned standards to the limit-state philosophy, whereby partial factors could be applied to both load and resistance variables (Allen, 1975). *Part 2* of the standard dealt with the application of traffic loads, and recommended a 5% characteristic value for the ultimate traffic load; having a 5% chance of occurring within the design life of the structure, set as 100 years. The limit-state philosophy is designed to allow for the benefit of statistical knowledge to more accurately model expected scenarios. However, at the introduction of *BS 5400*, such data was not available, and so nominal loading and partial factors were specified, based on engineering judgement at the time. The SLC from *BS 153* was retained, except with a constant UDL of 30 kN/m/lane up to a span of 30 m. For simply supported spans, this resulted in a maximum midspan bending moment slightly less than that prescribed in *BS 153*, for which a divergence begins from the 30–50 m span range (Figure 1).

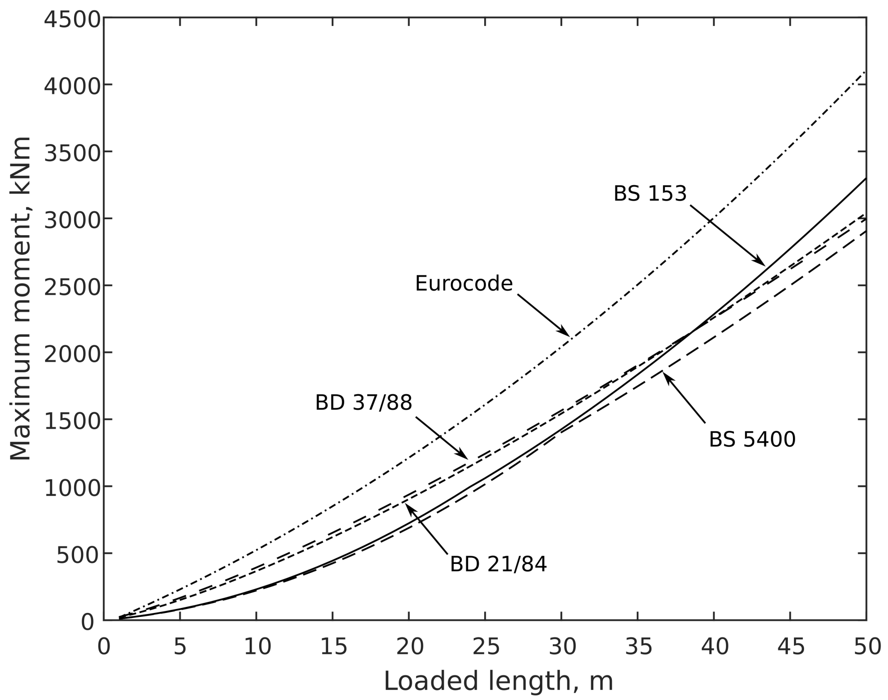

Figure 1: Maximum bending moment with increasing spans for changing traffic load definitions

2.3 BD 21/84

BD 21–The assessment of highway bridges and structures (Highways Agency, 1984) was introduced in 1984, revising some provisions of *BS 5400* for shorter spans. Specifically, the furthest departure was the elimination of a constant UDL for spans under 30 m, to be replaced by a curve that was fully variant with span length, and defined by a single formula as a function of length. The apparent lifetime of a bridge was extended to 120 years, so whereby a 5% characteristic ultimate load over the design life resulted in a total return period for the ultimate load of 2,400 years. The development of this code involved a more rigorous calibration of partial factors using statistical methods than the previous standard employed. The SLC was developed under the assumption that shorter spans are more likely to be fully laden with convoys of large vehicles than larger spans, and thus envelopes were made of the worst load effects for a variety of spans, and a new single SLC was derived from the results. The effect of the elimination of a constant UDL for spans under 30 m can be seen through the deviation between maximum bending moments for *BS 5400* and *BD 21/84* in Figure 1.

2.4 BD 37/88

Due to the general expected increase in total weight of European vehicles, the SLC of *BD 21/84* was updated in *BD 37–Loads for highway bridges* (Highways Agency, 1988) to account for a 40 tonne gross weight vehicle, as opposed to that of *BD 21/84* which accounted for 38 tonnes. This code also featured a 'composite' version of *BS 5400*, which included specifications for railway loading. The effect of this code is seen in greater prominence for spans above 50 m, but produces a minimal change in flexural load effects from *BD 21/84* (Figure 1).

2.5 Eurocode

The development of *EN 1991-2: Eurocode 1: Actions on structures. Traffic loads on bridges* (CEN, 1994) introduced four separate load models to account for the vertical load being applied to bridges, with Load Model 1 (LM1) corresponding to what has been referred to as normal loading, for spans between 5–200 m, and a carriageway width of up to 42 m. LM1 was derived from real European traffic data, and specified an ultimate load exceedence rate of 5% in 50 years, or a return period of 1000 years (Bruls et al., 1996). LM1 departed from previous representations of normal traffic loading by eliminating the SLC defined UDL and invariant KEL, and replacing them with a series of constant UDL, invariant with span length, in adjacent lanes and a tandem axle system of point loads. As can be seen from the comparison of bending moments in Figure 1, LM1 of *Eurocode* results in the most onerous of load effects of the presented normative standards.

3 Development of Bridge Models

In the assessment of civil engineering structures, a true representation of the structural safety can only be obtained through probabilistic methods which can account for load, material, and model uncertainties. The reliability index β is a measure of structural safety, which is a function of the probability of failure P_f and can be expressed as:

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

Table 1: Development of traffic loading rules, abridged from Dawe (2003)

Date	Event/publication	Comment
End of 19th century		Principal live loading on bridges deemed to be due to crowd loading. UDL used for design of bridge decks, for example 4.8 kN/m^2 for Hungerford Suspension Bridge
1904	Restriction on vehicle weights	8 ton limit for single axle, 12 ton limit for gross vehicle weight
1923	BS 153 *Part 3: Loads and stresses*	Traffic live loading to be specified by the Engineer. Impact factor inversely proportional to span.
1931	MoT *Standard loading for highway bridges*	Standard Loading Curve. Deterministic approach using equivalent UDL and KEL, with allowance for impact. Heavy wheel load introduced for short span structures.
1937	BS 153 Part 3 (1st revision)	Introduced Types A and B loading. Impact allowance varied with span
1954	BS 153: Part 3A (2nd revision)	Appendix A introduces Types HA and HB loading. HA comprises deterministic formula loading based on 22-ton vehicles, and an alternative wheel loading. HB loading with axle number and spacing based on typical abnormal trailers of the day; axle loads are heaviest allowed by law. (metricated in 1972)
1973	DoE technical memorandum (bridges) BE 5/73, *Standard highway loadings*	Loads applicable to all highway structures except steel box girders. Required a minimum of 30 units of HB loading for public roads. HA UDL capped at 31.5 kN for loaded lengths up to 6.5 m. HA wheel load and HB loading assumed to cover design of short spans.
1978	BS 5400 Part 2, *Specification for loads*	Introduction of limit state design. HA loading based on 24-tonne vehicles. HA UDL capped at 30 kN/m for loading lengths up to 30 m. Minimum UDL intensity now required to be 9 kN/m. Minimum of 25 units of HB required for public roads. HB loading (and HA wheel load) assumed to cover design of short spans.
1982	DTp BD 14, *Loads for highway bridges*	Implemented BS 5400: Part 2 for loaded lengths up to 40 m.
1984	DTp BD 21, *The assessment of highway bridges and structures*	HA loading re-derived for Construction and Use vehicles, taking into account effects of overloading, lateral bunching and impact factor of 1.8. Loading derived for a full range of spans (i.e. no longer capped for short spans).
1988	DTp BD 37, Loads for highway bridges (composite version of BS 5400: Part 2). Incorporated in DMRB in 2001	Revision of BS 5400: Part 2: 1972 containing revised HA loading; short span based on BD 21/84, enhanced long span derived statistically from live traffic data. Covers spans up to 1600 m.
1994	CEN, ENV 1991-3. *Eurocode 1: Basis of design and actions on structures. Part 3: Traffic loads on bridges*	European pre-standard for traffic loads on bridges. Covers spans up to 200 m. Constant UDL for all spans and tandem axle systems. 3 m notional lanes. (Issued in 2000 together with UK NAD. Constant UDL for all lanes across carriageway.)

where Φ is the standard normal cumulative distribution function. The probability of failure P_f is the probability of violation of a specified limit-state $g = 0$, and for structural safety assessments can be expressed as:

$$P_f = P(R - S \leq 0) = P[g(R, S) \leq 0] = P[g(X) \leq 0] \tag{2}$$

where R is the resistance/capacity of the element under consideration, and S represents the applied load. In this assessment, the flexural performance g was analysed, and so the flexural capacity M_u was tested against the bending moment effects of the self-weight of the bridge M_{DL}, the superimposed dead load of the road surface M_{SDL}, and the various bending moments produced by changing traffic load specifications M_{LL}.

$$g = R - S = M_u - M_{DL} - M_{SDL} - M_{LL} \tag{3}$$

For computational efficiency, the limit state equations are expressed in parametric form (Akgül and Frangopol, 2004a), 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. For the three bridges under consideration (RC slab, RC beam, PS beam), the limit state equation for flexural failure are defined as:

$$g_{slab,m} = \left(C_{01} A_s f_y \gamma_m \lambda_d - C_{02} \frac{A_s^2 f_y^2 \gamma_m}{f_c} \right) - C_{03}\lambda_c - C_{04}\lambda_s - C_{05}\lambda_{LL} \tag{4}$$

$$g_{beam,m} = \left(C_{11} A_s f_y \gamma_m \lambda_d - C_{12} \frac{A_s^2 f_y^2 \gamma_m}{f_c} \right) - C_{13}\lambda_c - C_{14}\lambda_s - C_{15}\lambda_{LL} \tag{5}$$

$$g_{prestressed,m} = \left(C_{21} A_{ps} f_{pu} \gamma_m \lambda_d - C_{22} \frac{A_{ps}^2 f_{pu}^2 \gamma_m}{f_c} \right) - C_{23}\lambda_c - C_{24}\lambda_s - C_{25}\lambda_{LL} \tag{6}$$

where the random variables A_{ps}, A_s, f_c, f_{pu}, f_y, and the uncertainty factors λ_x and γ_m are defined in Table 2, and the deterministic constant coefficients C_{ij} are functions of the deterministic parameters defined in Table 3. The distributions chosen for the variables in Table 2 are based on guidelines presented by Akgül and Frangopol (2004b, 2005a,b).

The probabilistic load model used in this paper was developed by Chryssanthopoulos et al. (1997) and Cooper (1997), and was derived as a static load model with a uniformly distributed load (UDL) and two axle loads, factored by a statistically defined variable λ_{Prob} with a Gumbel distribution; extrapolated from WIM data on motorway bridges in the UK. This model is

Table 2: Random variables for all bridges (All RV's have lognormal distributions, with the exception of λ_{Prob}, which has a Gumbel distribution)

Bridge	Tag	Variable	Description	μ	σ
Slab	X_{01}	A_s	Area of flexural steel reinforcement (mm^2)	6835.35	341.7675
	X_{02}	f_{cu}	Compressive strength of concrete (N/mm^2)	50	7.5
	X_{03}	f_y	Yield strength of reinforcing steel (N/mm^2)	500	50
	X_{04}	γ_m	Model uncertainty for flexure	1	0.1
	X_{05}	λ_c	Concrete weight uncertainty factor	1	0.1
	X_{06}	λ_s	Surfacing weight uncertainty factor	1	0.25
	X_{07}	λ_d	Effective depth uncertainty factor	1	0.02
	X_{08}	λ_{LL}	Traffic live load uncertainty factor	1	0.2
	X_{09}	λ_{Prob}	Probabilistic load adjustment factor	0.4101	0.02466
Beam	X_{11}	A_s	Area of flexural steel reinforcement (mm^2)	5192.69	259.6345
	X_{12}	f_{cu}	Compressive strength of concrete (N/mm^2)	50	7.5
	X_{13}	f_y	Yield strength of reinforcing steel (N/mm^2)	500	50
	X_{14}	γ_m	Model uncertainty for flexure	1	0.1
	X_{15}	λ_c	Concrete weight uncertainty factor	1	0.1
	X_{16}	λ_s	Surfacing weight uncertainty factor	1	0.25
	X_{17}	λ_d	Effective depth uncertainty factor	1	0.02
	X_{18}	λ_{LL}	Traffic live load uncertainty factor	1	0.2
	X_{19}	λ_{Prob}	Probabilistic load adjustment factor	0.4101	0.02466
Prestressed	X_{21}	A_p	Area of prestressing steel (mm^2)	3892	194.6
	X_{22}	f_{cu}	Compressive strength of concrete (N/mm^2)	50	7.5
	X_{23}	f_{pu}	Prestressing steel strength (N/mm^2)	1670	83.5
	X_{24}	γ_m	Model uncertainty for flexure	1	0.1
	X_{25}	λ_c	Concrete weight uncertainty factor	1	0.1
	X_{26}	λ_s	Surfacing weight uncertainty factor	1	0.25
	X_{27}	λ_d	Effective depth uncertainty factor	1	0.02
	X_{28}	λ_{LL}	Traffic live load uncertainty factor	1	0.2
	X_{29}	λ_{Prob}	Probabilistic load adjustment factor	0.4101	0.02466

Table 3: Deterministic parameters for all bridges

Bridge	Tag	Parameter	Description	Value
Slab	Y_{01}	b	Width of section considered (mm)	1000
	Y_{02}	b_L	Notional lane width (m)	3.2
	Y_{03}	d	Effective depth of section (mm)	724
	Y_{04}	L	Span length (m)	16
	Y_{05}	h_c	Height of concrete slab (mm)	800
	Y_{06}	t_s	Thickness of road surface (mm)	100
	Y_{07}	ρ_c	Self-weight on concrete (kN/m^3)	25
	Y_{08}	ρ_s	Self-weight of surface (kN/m^3)	24
Beam	Y_{11}	b_{eff}	Effective flange width (mm)	1200
	Y_{12}	b_L	Notional lane width (m)	3.2
	Y_{13}	b_w	Width of beam (mm)	300
	Y_{14}	d	Effective depth of section (mm)	924
	Y_{15}	L	Span length (m)	16
	Y_{16}	h_c	Overall height of concrete beam (mm)	1000
	Y_{17}	h_f	Thickness of concrete flange/slab (mm)	200
	Y_{18}	t_s	Thickness of road surface (mm)	100
	Y_{19}	ρ_c	Self-weight on concrete (kN/m^3)	25
	Y_{110}	ρ_s	Self-weight of surface (kN/m^3)	24
Prestressed	Y_{21}	A_b	Area of precast section (mm^2)	339882
	Y_{22}	b_{eff}	Effective flange width (mm)	1200
	Y_{23}	b_L	Notional lane width (m)	3.2
	Y_{24}	d	Effective depth of section (mm)	818.571
	Y_{25}	L	Span length (m)	16
	Y_{26}	h_c	Overall height of section (mm)	950
	Y_{27}	h_f	Thickness of concrete flange/slab (mm)	200
	Y_{28}	t_o	Thickness of overlap (mm)	50
	Y_{29}	t_s	Thickness of road surface (mm)	100
	Y_{210}	ρ_c	Self-weight on concrete (kN/m^3)	25
	Y_{211}	ρ_s	Self-weight of surface (kN/m^3)	24

variable based on traffic volume flow per direction per day for single year periods. In this paper, this volume was chosen to for a main road, for which the traffic flow volume specified by Cooper (1997) was 10,000.

Sensitivity studies can be carried out within the framework of reliability analysis and it is helpful in identifying and quantifying errors in design, modelling and construction (Frangopol, 1985a,b; Nowak and Carr, 1985). The importance of a variable to β is defined as the alpha-value α_i, which measures the sensitivity of β to a small variation in the mean-value μ_i of a basic random variable (Hohenbichler and Rackwitz, 1986):

$$\alpha_i = \frac{\partial \beta}{\partial \mu_i} \tag{7}$$

This parametric sensitivity factor α_i for the reliability index β with respect to a parameter θ is defined (Madsen et al., 1986) and developed (Bjerager and Krenk, 1989) as the derivative $\partial\beta/\partial\theta$. Furthermore, as part of a sensitivity analysis, parameter importance factors α_i^2 can be determined, identifying which of the modelled parameters have the greatest impact on the reliability index, and thus, the safety of the structure.

$$\sum_{i=1}^{n} \alpha_i^2 = 1 \tag{8}$$

These factors indicate through their ranking, expressed as a percentage, what parameters are important for monitoring within a system and to what extent they contribute to the probability of safety or failure. Also, for varying limit states or uncertainties, the ranking of these parameters within a system can change; emphasizing the fact that the contribution of a certain factor to a failure defined by a limit state is a function of the information available about the system and the associated confidence or accuracy of that information (Hanley and Pakrashi, 2016).

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 (Akgül and Frangopol, 2005a). The time to initiation of corrosion T_i is commonly obtained using Fick's 2nd law of diffusion (Akgül and Frangopol, 2004b, 2005b; Kenshel and O'Connor, 2009):

$$T_i = \frac{C^2}{4D_c}\left[\mathrm{erf}^{-1}\left(\frac{C_s - C_{cr}}{C_s}\right)\right]^{-2} \tag{9}$$

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 (Enright and Frangopol, 1998). Once the time to corrosion initiation is determined, time-variant flexural steel $A_s(t)$ area can be found as:

$$A_s(t) = \frac{\pi}{4} \sum_{j=1}^{n} \left[D_{0,j} - \Delta D_j(t) \right]^2, \quad \Delta D_j(t) = r_{corr} \left(t - T_i \right) \tag{10}$$

where $D_{0,j}$ is the initial diameter of the steel bars and strands; $\Delta D_j(t)$ 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 (Akgül and Frangopol, 2005b), and 0.0571 and 0.017 for the PC bridge (Akgül and Frangopol, 2004b).

4 Results

4.1 Reliability Assessment of Undamaged Bridges

An initial reliability assessment was conducted on the three bridges under consideration to determine the relative change in β for each variation in normative traffic loading, not considering degradation (Figure 2). As can be seen, despite an increase in β from *BS 153* to *BS 5400*, there is a consistent decrease in β with more recent normative traffic loading. Additionally, with more recent normative loads, the disparity between β for specified loading and the probabilistic load model is increased. As the return periods for the normative loading are quite high, this disparity between specified loading and site-specific probabilistic loading is expected; and so with greater disparity, more conservative structures are being designed, and thus the probability of the limit state being violated under regular use is lowered. This, however, can not be said to be the case for *BS 153* to *BS 5400*, which have much closer β's to the probabilistic load model. This would suggest that the load effects produced by the ultimate traffic load in these early codes are actually more representative of that produced by the typical traffic load from the probabilistic

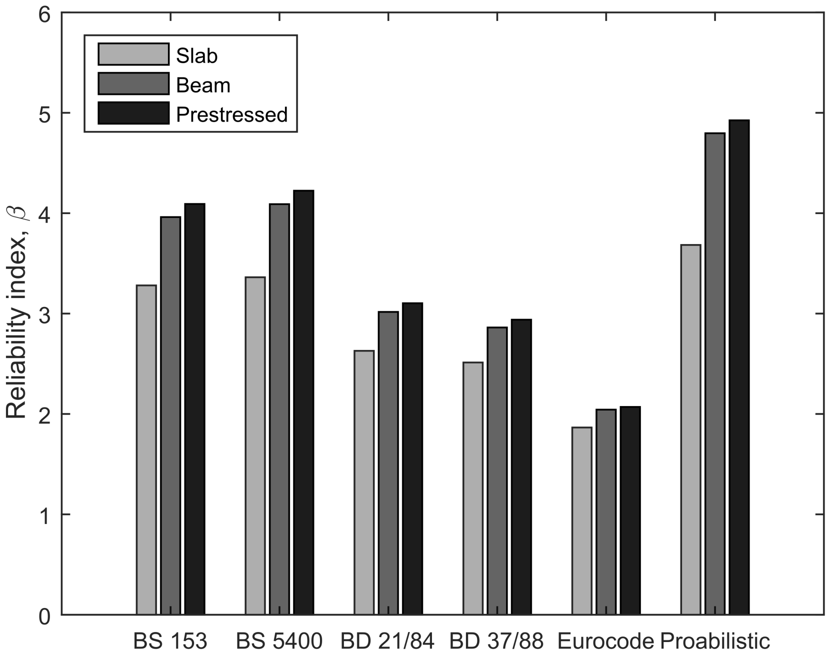

Figure 2: Change in reliability index with changes in code definitions, not considering structural degradation

model. This is problematic, as these ultimate loads are not expected to occur within the reasonable life-cycle of the bridge structure. The low relative value of β under *Eurocode* is expected given that it produces the most adverse bending moment of the presented standards (Figure 1). However, the discrepancy between this β and that for the site-specific loading suggests that it is perhaps too onerous for the purposes of assessment for existing structures, but designing new bridges to this requirement will produce more robust structures.

4.2 Parametric Sensitivity & Importance Factors

The importance factors α_i^2 were determined to highlight the random variables that have the greatest influence on β, for each iteration of normative traffic loading (Figure 3). The importance factors which demonstrate the biggest variation for every code iteration are for the random variables X_5 and X_8, which correspond to the uncertainty factors for concrete λ_c and live load λ_{LL}. This would suggest a diminishing role of the self-weight of the bridges as the traffic loading becomes more onerous. For RC and PC beam bridges, λ_{LL} has the highest importance factor across all the codes, with a lower bound value of 30.3% and 31.7% for *BS 5400*, and an

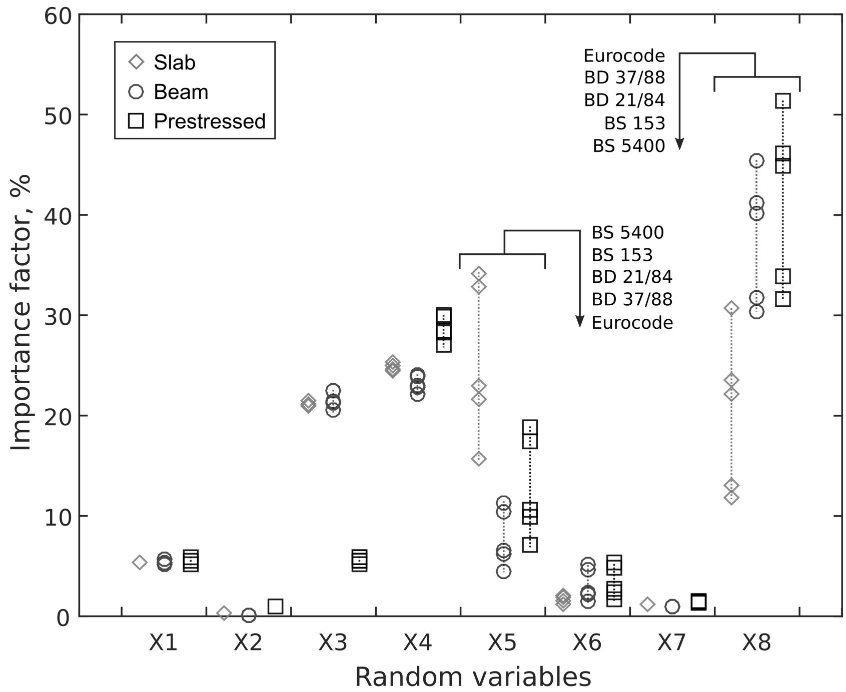

Figure 3: Importance factors of the random variables for each code specification

upper bound value of 45.3% and 51.4% for *Eurocode*, respectively. However, for the RC slab bridge, it can be seen that the importance factors for these random variables occupy the same range throughout the changing codes, except for an almost inverse relationship between the self-weight and the live load. For *BS 5400*, the importance factors for λ_c and λ_{LL} are 34.2% and 11.8%, respectively; whereas, for *Eurocode*, they are 15.6% and 30.7%, respectively. The greater influence of the self-weight is expected for the slab bridge, due to its inherent form of mass concrete, as opposed to the RC and PC beam bridges, which are lighter in nature. It can be seen that the importance factors for each of these variables are somewhat equal for *BD 21/84* and *BD 37/88*, before the more onerous traffic loading of *Eurocode* becomes the most dominant importance factor.

The parametric sensitivity α_i was demonstrated by assessing the effect on β of a 10% perturbation in the mean value of the random variables (Figure 4). It is evident that the most favourable random variables across the three bridges are X_1, X_3, X_4, and X_7, corresponding with $A_{s,p}$, $f_{y,pu}$, γ_m, and λ_d. The only random variable which exhibits any significant variation with changing normative codes is the model uncertainty for flexure γ_m, with the remaining favourable random variables maintaining their relative sensitivities. However, the variation re-

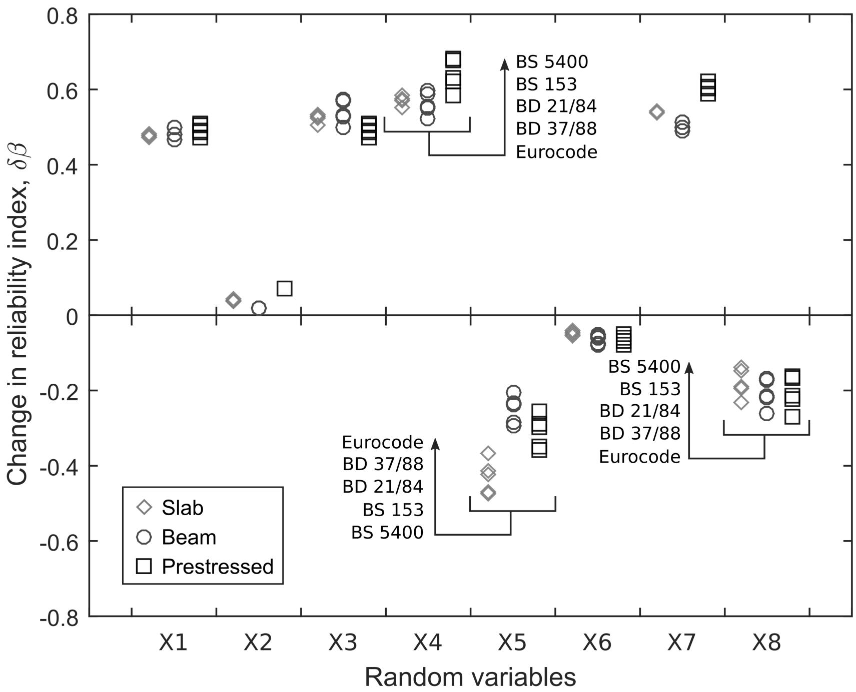

Figure 4: Parametric sensitivity of β for a 10% perturbation in the random variables

mains only slight, but is indicative of how the normative traffic loading becomes more onerous and, thus, more dominant in the probabilistic model. It is noteworthy how, for the PC beam bridge, the grade of prestressing steel f_{pu} has low stochastic importance (Figure 3), yet is in line with the grade of reinforcing steel f_y for the parametric sensitivity, even when f_y is stochastically more important. This can be attributed to the coefficients of variation (CoV) for the two random variables; with f_{pu} having a lower CoV (5%) than f_y (10%), due to the more controlled nature of manufacturing process of precast PC beams, as opposed to in-situ cast RC slabs and beams.

For the unfavourable random variables, X_5 (λ_c), $X_6(\lambda_s)$, and X_8 (λ_{LL}), it can be seen that the uncertainty factor related to concrete self-weight λ_c displays the greatest negative relative change in β for a 10% perturbation. Additionally, λ_c for the RC slab bridge has the greatest parametric sensitivity, which is consistent with the established importance factors (Figure 3). While the sensitivity of λ_c across the normative code variations remains the highest for the RC slab bridge, it can be seen that the relative ranking of sensitivities is switched between that for λ_c and λ_{LL} for the RC and PC beam bridges. This is more prevalent for the RC beam bridge, where the relative change in β for λ_c and λ_{LL} under *BS 5400* is -0.29 and -0.17, and under

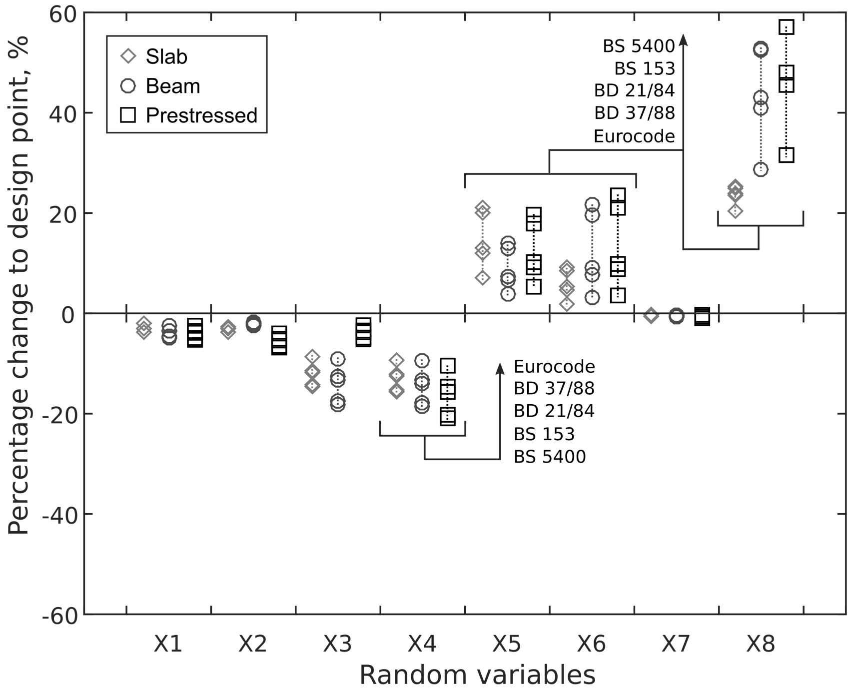

Figure 5: Relative change in the random variables at the design point for each code specification

Eurocode is -0.20 and -0.26, respectively. This shows the same somewhat inverted relationship between these two codes as has already been seen earlier. For the PC beam bridge, these two variables have a relative change in β of -0.36 and -0.16 under *BS 5400*, and then converge to -0.26 and -0.27 under *Eurocode*, respectively.

The percentage change in each of the random variables at the design point **u***, being the most likely point of failure, can be seen in Figure 5. The coordinates of the design point **u*** are:

$$\mathbf{u}* = -\alpha\beta \tag{11}$$

where α are directional cosines which represent the parametric sensitivity of the reliability index β. It is apparent that, under *Eurocode*, the variables require the least amount of deviation from the mean value to reach **u***, whereas for *BS 5400*, the variables require the largest deviation. This variation between the two codes is most pronounced for λ_{LL}, and is consistent with the relationship seen for the importance factors (Figure 3). Again, this further emphasises the more onerous nature of the more recent normative codes, over the earlier models.

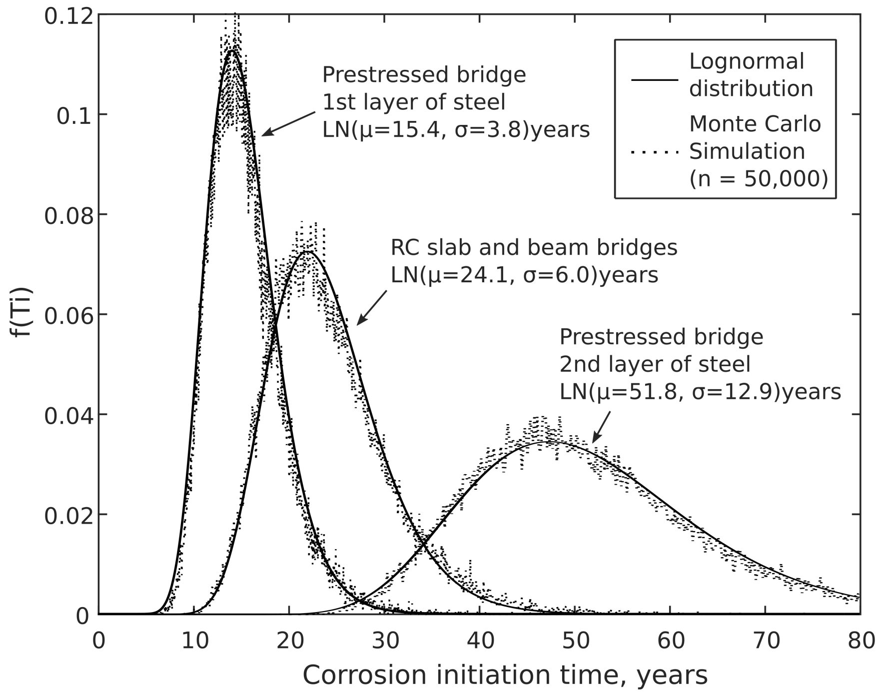

Figure 6: Probability density function of corrosion initiation time for each bridge with lognormal distribution and Monte Carlo Simulation

4.3 Life-Cycle Reliability Assessment

The life-cycle assessment was conducted through a time-variant reliability analysis, considering the time-variant degradation of flexural steel area due to the uniform corrosion model. Using equation 9, 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 (Enright and Frangopol, 1998). 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 was determined using equation 10 and plotted for each bridge over an 80 year period (Figure 7).

The effect of corrosion on β for the three bridges can be seen in Figures 8–10. Additionally, the lifetime reliability is presented for both a probabilistic load assessment, and an assessment based on normative loading; including 'jumps' in β that account for the changing normative specifications over time. For the RC slab bridge (Figure 8), the initial reliability index under normative loading (*BS 153*) β_n and under probabilistic loading β_p is 3.28 and 3.68, respectively. There is a slight jump in β_n with the introduction of *BS 5400*, but a significant drop in β_n to

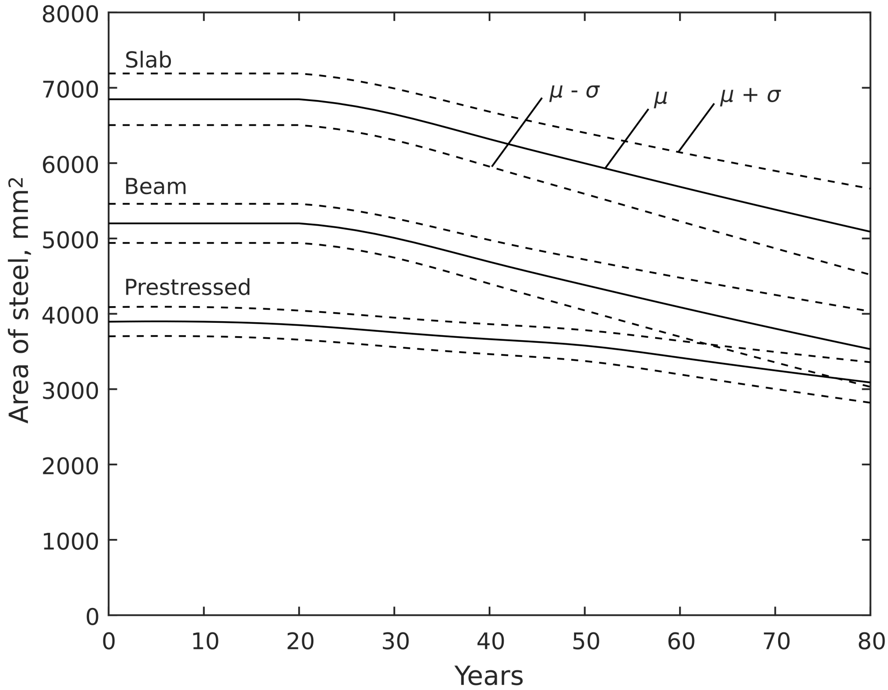

Figure 7: Deterioration of steel area on RC and prestressed bridges

below 2 with the introduction of *BD 21/84*. The next significant drop in β_n occurs with the *Eurocode*, to finish the 80 year period with a β_n of 0.43, compared to β_p of 2.05. These β profiles are similar for the RC and PC beam bridges (Figures 9 & 10). For the RC beam bridge, the initial values of β_n and β_p are 3.96 and 4.79, respectively, whereas the final values are 0.20 and 2.29; a significant difference. Similarly, for the PC beam bridge, the initial values of β_n and β_p are 4.09 and 4.92, respectively, whereas the final values again show a big difference at 0.93 and 3.51.

These end variations are expected based on the initial β values determined earlier (Figure 2). However, it is interesting that during a 20 year period in the second half of the total assessment period, there are two significant 'overnight' drops in β_n, each departing further away from β_p. Additionally, after the full 80 year period, β_p for each bridge never drops below β_n assessed under *BD 21/84* loading; first computed approximately 30 years prior. As maintenance and intervention decisions are often based on performance indicators such as β, the decision to intervene structurally on a bridge can be taken too hastily when normative loading is used instead of probabilistic loading, and lead to the misallocation of budgetary resources.

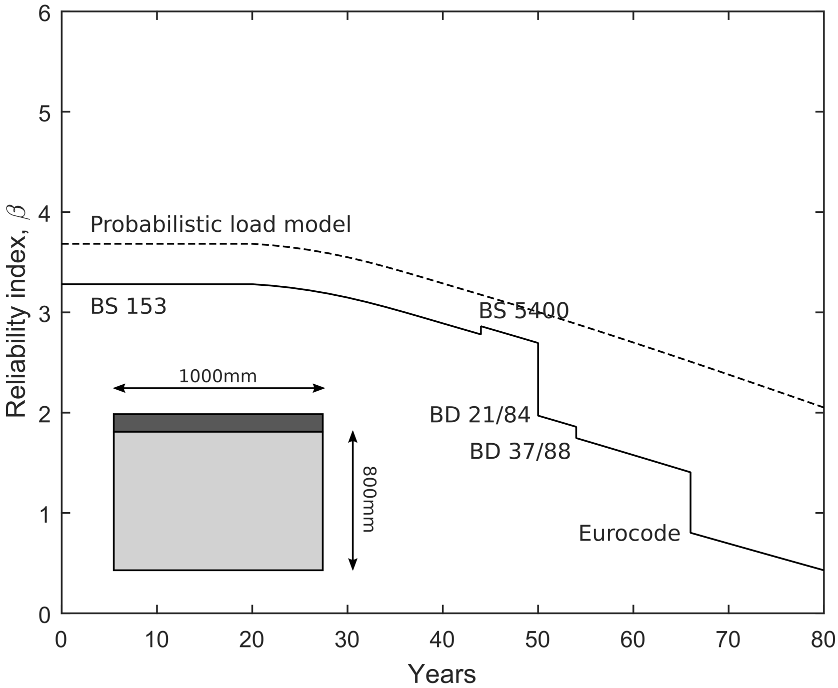

Figure 8: Life-cycle reliability index for RC slab bridge with adjustments for changing normative codes

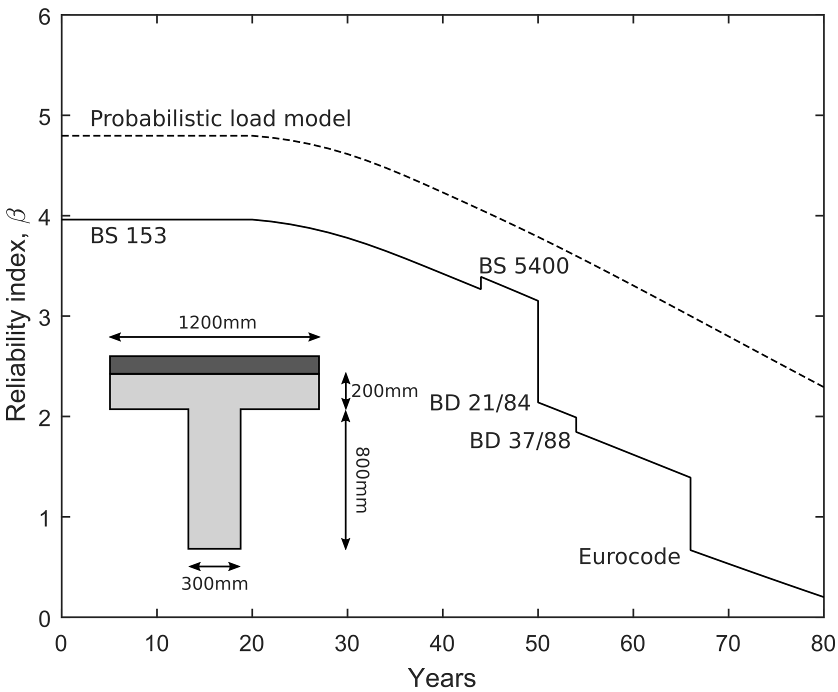

Figure 9: Life-cycle reliability index for RC beam bridge with adjustments for changing normative codes

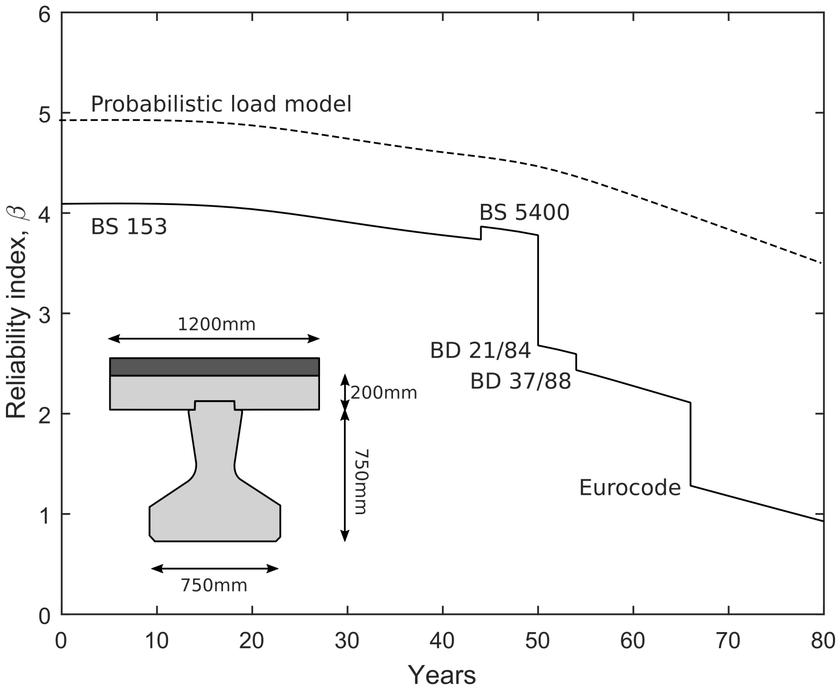

Figure 10: Life-cycle reliability index for prestressed concrete bridge with adjustments for changing normative codes

5 Conclusions

A structural reliability analysis was conducted on three bridges to assess the effect of changing definitions of normative traffic loading on safety classifications of the structures. These 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. It was observed that earlier codes produced less onerous flexural load effects and, as such, resulted in reliability indices closer to that determined under the probabilistic load model. This, however, results in a situation where bridges designed and assessed under these early codes are regularly being subjected to close to their ultimate loads. As these normative loads were said to have a large return period, such proximity between the 'typical' and 'ultimate' loading is not an expected or desirable scenario.

Given the disparity between β for the probabilistic load model and the more recent normative codes, it is evident that bridge structures designed and constructed according to these standards should have a higher resistance capacity than seen in bridges designed to the extent of the earlier standards. It can thus be suggested that bridges designed to the extent of the modern standards will perform better in β when assessed against a probabilistic load, and it has been shown that bridges designed to the more onerous load conditions can result in a reduction in the life-cycle cost (Hanley et al., 2016). However, the apparent disconnect between modern and probabilistic loading suggests that the use of normative loading in the assessment of existing bridge structures is not best practice for an economical life-cycle asset management, and under such circumstances the use of site-specific information and probabilistic load modelling can lead to a higher accuracy and reflective of the true safety of the bridge.

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