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Effects of increasing design traffic load on performance and life-cycle cost of bridges

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ABSTRACT: Due to the onerous and expensive nature of preventative and essential maintenance of existing bridge infrastructure, it is prudent to look into methods of improving the life-cycle safety and cost of newly constructed bridges at the design phase. In an effort to acheive economy in material quantities and initial cost, the structural capacity of these bridges is often at the required minimum target level. This paper investigates the effects that increased design traffic loading have on the initial construction cost and whether that could be balanced by a reduced requirement for financial intervention in the mid to later stages of the bridge's design-life. This is achieved by conducting a life-cycle performance and cost assessment on a reinforced concrete slab bridge that is designed to increasing standard traffic loads.

1 INTRODUCTION

Preserving a functional and serviceable civil infrastructure network requires complex methods to devise optimum strategies to schedule expensive preventative and essential maintenance of existing bridge stock (Estes and Frangopol 2001). 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 live loading (Nowak et al. 1993, Nowak 1993). 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). Often, there has not been sufficient funds for owners of bridge stock to replace, intervene, or even prioritise investment (Ellingwood 2005, Frangopol and Liu 2007, Frangopol 2011, Frangopol and Soliman 2016, Pakrashi et al. 2011, Frangopol and Bocchini 2012).

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), it is prudent to look into methods of improving the life-cycle safety and cost of newly constructed bridges at the design phase. At this stage, design traffic loading is typically specified in normative codes as a function of bridge geometry, with section capacities being designed accordingly (Dawe 2003). In practice, the structural capacity is minimised at the ultimate limit state in an effort to reduce material quantities and initial construction costs. Achieving such economy is often accomplished at the expense of structural robustness from a life-cycle perspective. This paper investigates to what extent design traffic loading has on life-cycle safety and cost assessments for bridge structures. The effects that increased design traffic loading would have on the initial construction cost of a more robust bridge could be balanced by a delayed deterioration to the minimum performance threshold and, consequently, a reduced requirement for financial intervention in the mid to later stages of the bridge's design-life.

In this paper, a brief summary of the major bridge design and assessment standards used in the UK & Ireland will be presented, and the effect of the various definitions of normative traffic loading, and thus minimum flexural capacity, 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 a simply supported reinforced concrete slab bridge. An 80 year life-cycle reliability assessment is presented for the five iterations of the bridge design, as well as an associated life-cycle cost assessment that is required to keep the bridge above a minimum acceptable performance threshold, defined as the target reliability index β_T. It will be shown how a small relative increase in the flexural capacity at the design stage, and thus initial construction cost, results in a significant offset in the expect cost of failure, and thus the total expected life-cycle cost.

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 only subsequent 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 in an 80 year period (Dawe 2003) has, in some instances, resulted in an almost doubling the flexural load effects between *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). The effects on changes in the design standards on the induced bending moment of a simply supported bridge can be seen in Figure 1.

BS 153–Standard specification for girder bridges (BSI 1937) recommendeded 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

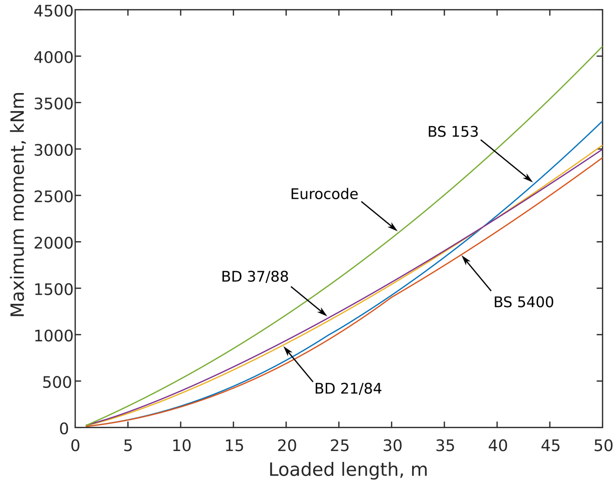

Figure 1: Maximum bending moment with increasing spans for changing live load definitions

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.

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 partial factors based on engineering judgement at the time, due to the absence of probabilistic information. 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.

BD 21–The assessment of highway bridges and structures (Highways Agency 1984) was introduced in 1984 revise 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 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.

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. 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).

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's, 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:

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

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:

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

where R is the resistance/capacity of the element under consideration, and S represents the applied load. In this assessment, the flexural performance function 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}.

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

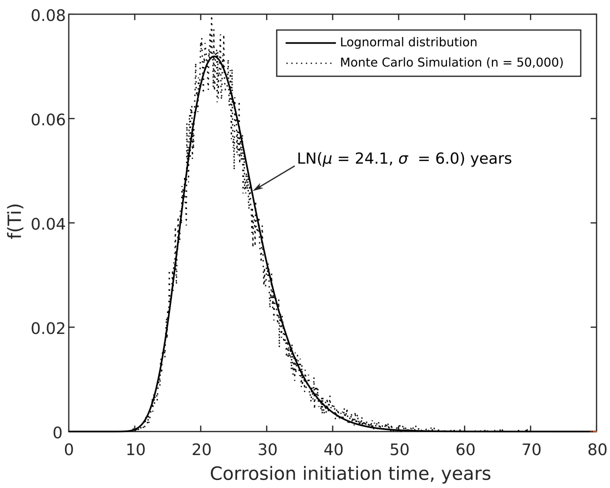

Figure 2: Probability density function of corrosion initiation time using lognormal distribution and Monte Carlo Simulation

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 is defined as:

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

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 1, and the deterministic constant coefficients C_{ij} are functions of the deterministic parameters defined in Table 2.

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 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.

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, Akgül and Frangopol 2005b, Kenshel and O'Connor 2009):

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

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

Table 1: Random variables for assessment (All RV's have lognormal distributions, with the exception of λ_{Prob}, which has a Gumbel distribution)

Tag	Variable	Description	μ	σ
X_{01}	A_s	Area of flexural steel reinforcement (mm^2)		
		– BS153	5523.79	276.189
		– BS5400	5404.67	270.233
		– BD21/84	6506.89	325.345
		– BD37/88	6691.12	334.556
		– Eurocode	7802.63	390.131
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}	Live load uncertainty factor	1	0.2
X_{09}	λ_{Prob}	Probabilistic load adjustment factor	0.4101	0.02466

Table 2: Deterministic parameters for assessment

Tag	Par.	Description	Value
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

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:

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

$$\Delta D_j(t) = r_{corr}\,(t - T_i) \tag{6}$$

where $D_{0,j}$ is the initial diameter of the steel bars and strands; $\Delta D_j(t)$ 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 LIFE-CYCLE COST MODEL (FRANGOPOL ET AL. 1997)

The life-cycle cost model used in this assessment was developed by Frangopol et al. (1997) to optimise the inspection and repair of deteriorating structures, the procedure of which is briefly summarised here. The expected total life-cycle cost C_{ET} is the sum of the various cost components of the structure; initial construction C_T, routine preventative maintenance C_{PM}, inspections C_{INS}, repair C_{REP}, and failure C_F.

$$C_{ET} = C_T + C_{PM} + C_{INS} + C_{REP} + C_F \tag{7}$$

The initial construction cost C_T is taken as a function of the volume of concrete and steel in the section, and can be expressed as:

$$C_T = C_C A_c L + C_S A_s L \tag{8}$$

where C_C and C_S is the unit cost of concrete and steel per m^3, respectively; A_c and A_s is the area of concrete and steel in the section, respectively; and L is the length of the section being considered. In order to account for the reduction of A_s away from the position of maximum bending moment, it is suggested to factor A_s by 0.75 (Lin and Frangopol 1996). However, as this model does not consider the effect and cost of shear reinforcement, the total value of A_s will be accounted for to simulate the cost of shear reinforcement.

The cost of lifetime preventative maintenance C_{PM} is described as the linear combination of the cost of preventative maintenance at year one C_{main}, and the age of the structure at the time of the preventative maintenance t. To account for future costs, C_{PM} is the sum of the net present value costs of each occurrence of routine preventative maintenance:

$$C_{PM} = \sum_{i=1}^{t} C_{main,i} \frac{1}{(1+r)^{t_i}} \tag{9}$$

where r is the net discount rate.

The total expected inspection cost C_{INS} is defined as:

$$C_{INS} = \sum_{i=1}^{m} C_{ins} \frac{1}{(1+r)^{t_i}} \tag{10}$$

where m is the number of inspections; and C_{ins} is the cost of the inspection method used (Mori and Ellingwood 1994), which is a function of the detectable damage intensity η and cost of an ideal inspection α_{ins}, which is to be taken as a fraction of the initial cost C_T.

In the development of this model, Frangopol et al. (1997) have shown that the number of inspections m have a significant influence on C_{ET}, as this variable has a direct influence on the number of repair activities carried out, which reduces the overall probability of failure P_f and thus the expected cost of failure C_F. To establish C_{REP} and C_F, an event tree can be constructed whereby for each inspection, a decision can be taken on whether to initiate a repair activity or not. A constraint imposed on the model is that if damage η is detected during any inspection, a repair activity must be carried out. This decision is thus based on the probability of detecting damage at the time of an inspection, or the probability of damage not being detected. For simplicity, in this assessment it is assumed that each repair activity returns the bridge to its initial reliability index β_i. Thus, for each node of the event tree, the decision on whether to conduct a repair activity or not will directly influence the failure probability at the following node on the event tree for the next inspection.

The lifetime failure probability $P_{f,life}$ of the bridge for any number of m inspections can be defined as:

$$P_{f,life} = \sum_{i=1}^{2^m} P_{f,life,i} P(B_i) \tag{11}$$

where $P(B_i)$ is the probability that any path on the event tree occurs, and $P_{f,life,i}$ is the maximum failure probability for that path. The expected total cost of repair is then defined as:

$$C_{REP} = \sum_{i=1}^{2^m} C_{rep,i} P(B_i) \tag{12}$$

where $C_{rep,i}$ is the net present value repair cost at each node, as a function of the effect of the repair activity e_{rep}.

Finally, the expected failure cost C_F is defined as:

$$C_F = C_f P_{f,life} \tag{13}$$

where the failure cost C_f is a function of the initial cost C_T, in this assessment assumed to be 50,000C_T.

5 RESULTS

5.1 *Life-Cycle Reliability Assessment*

A reinforced concrete (RC) slab bridge was designed for the five design traffic loads discussed previously, with the model being constrained to change only the minimum required flexural steel area A_s to account for the changing flexural capacity demands (Table 1).

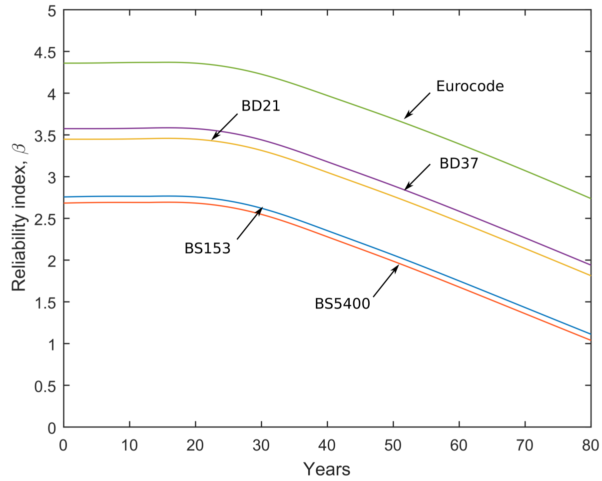

Figure 3: Life-cycle reliability index for RC slab bridge for various normative codes

The life-cycle performance was evaluated through a time-dependent reliability analysis, considering the time-variant degradation of A_s due to the uniform corrosion model. 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 (Enright and Frangopol 1998). The mean value of T_i for this RC slab bridge was 24.1 years. The loss of cross-sectional area of flexural steel specified under each design load was determined using equation 6 and used to determine the time-dependent reliability index (Figure 3).

It is evident that the design code that produced the most adverse bending moment (*Eurocode*) and thus demanded the largest flexural capacity yields the greatest life-cycle performance, with the opposite being true for the least onerous code, *BS 5400*. Additionally, it can be seen that the five codes considered produce life-cycle β curves that are bunched into three different 'bands', which generally follow the time period with which they were developed, and thus the philosophy regarding structural design at these times; from allowable stress-based design of *BS 153* to the more sophisticated *Eurocode*, derived from probabilistic code calibration.

5.2 *Life-Cycle Cost Assessment*

The objective of the life-cycle cost model used here is to minimise C_{ET} while ensuring a minimum level of structural performance at all times, and then to determine the optimum inspection and repair strategy that achieves this goal. The structural performance is indicated through β and P_f, with a minimum performance indicator being the target reliability index β_T. In this assessment, β_T is set at a value of 2.5, which corresponds to a P_f of 0.0062. As the expected cost of failure C_F is a function of the lifetime probability of failure $P_{f,life}$, and $P_{f,life}$ is heavily related to the number of inspections m conducted, it has been shown by Frangopol et al. (1997) that C_F can be minimised

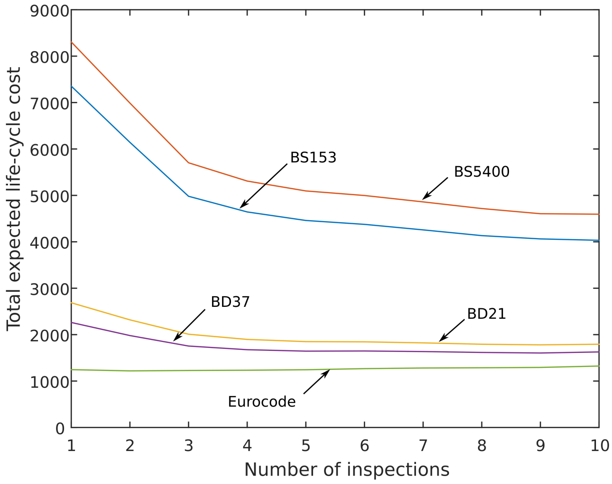

Figure 4: Variation in total expected life-cycle cost against increasing number of inspections for each design load

by an optimum number of inspections m_{opt}. This is because only after an inspection can a repair activity be carried out; an activity which will improve β and lower P_f. Thus, as m increases, the likelihood of failure and thus $P_{f,life}$ and C_F are reduced. However, there exists a diminishing returns point when $P_{f,life}$ is low enough to keep C_F as a minor component of C_{ET}, and for increasing values of m, C_{ET} rises with the expected cost of inspection C_{INS}.

The effect of more onerous traffic load requirements on C_{ET}, as well as the effect of increased inspections m is shown in Figure 4. In the figure, there exists a gulf between C_{ET} for the earlier codes, *BS 153* and *BS 5400*, and that for the more modern codes; *BD 21*, *BD 37*, and *Eurocode*. Despite the required increase in C_T for the modern codes, the values of C_{ET} are significantly lower, and less dependent on the number of inspections carried out. This is explained by the effective reduction in C_F due to the improved β provided by the increased A_s demanded by the modern codes. The effect that this increased A_s has on C_{ET} can be seen in Figure 5. It is clear that while there is very little relative change in C_T, there is a significant reduction in C_{ET} to the point where the number of inspections m loses significant importance.

The effect of increasing values of m and A_s on C_{ET} and C_F can be visualised in the 3D surface plots in Figures 6 & 7, respectively. It can be seen in Figure 7 that C_F, and consequently $P_{f,life}$, lower to a near zero point for 10 inspections under *Eurocode* designed A_s. It is clear from these figures that the increased demand of A_s, and thus the small relative increase in C_T, has a greater influence on C_{ET} than the inspection regime.

This contention can be borne out by evaluating the repair strategies for each bridge, based on the number on inspections that return a minimum value of C_{ET}. From Figures 4–6, this can be seen to result in m_{opt} of 9, 9, 5, 5, & 2 for *BS 153* & *Eurocode*, respectively. In assessing the repair options available, two strategies were adopted: to repair the structure to its original state after each inspection, and to repair the struc-

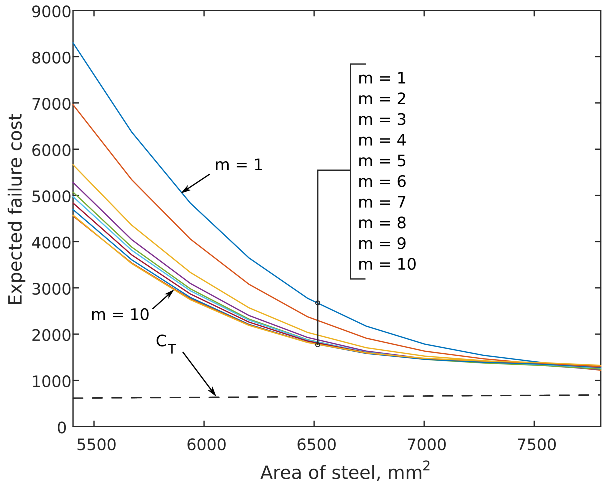

Figure 5: Diminishing total expected failure cost for increasing area of steel and increasing number of inspections

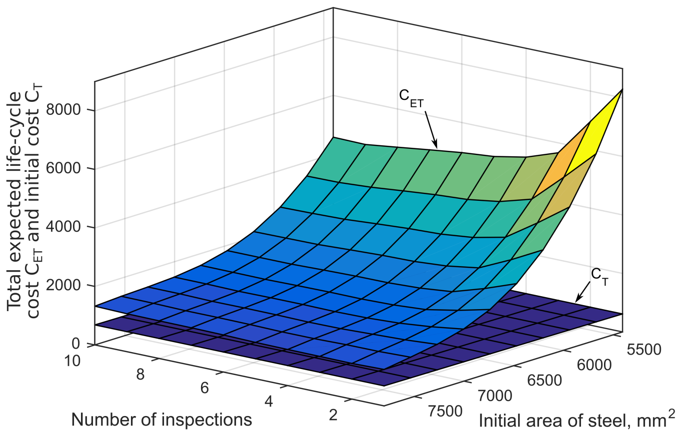

Figure 6: Effects of increasing inspections and area of steel on total expected life-cycle cost

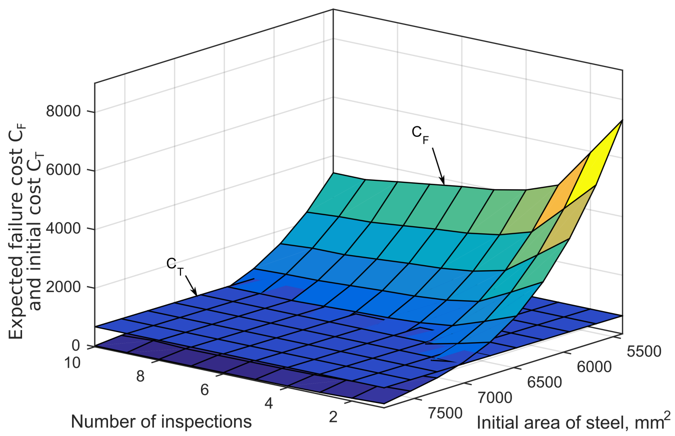

Figure 7: Effects of increasing inspections and area of steel on total expected failure cost

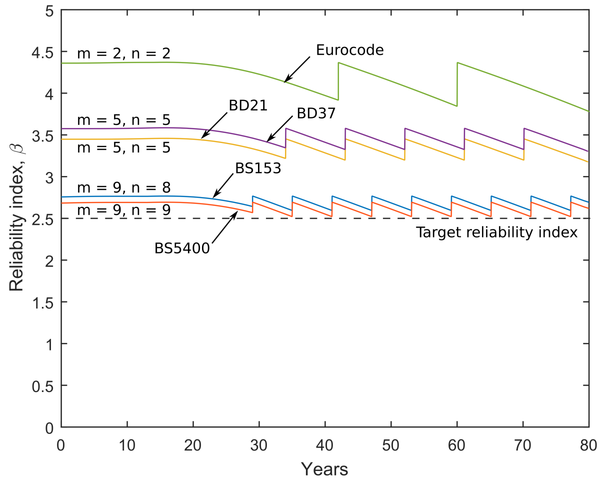

Figure 8: Effect of non-optimum repair strategy on β for uniform interval inspection for each design load

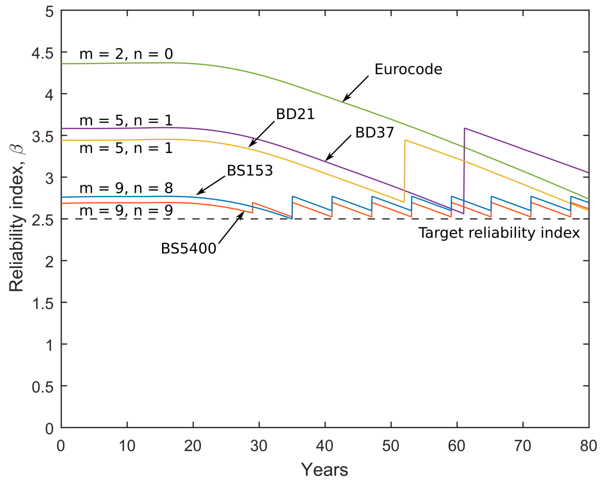

Figure 9: Effect of optimum repair strategy on β for uniform interval inspection for each design load

ture to its original condition at a time where it would fall below β_T before the next scheduled inspection. For both these strategies, a uniform inspection interval was asssumed. In this model, it was assumed that once chloride induced corrosion began in the original structure, it would continue to act on A_s immediately after repair. The result of the first strategy can be seen in Figure 8. Here the bridge is repaired prematurely for those designed under modern standards, resulting in a misallocation of resources, but is seemingly appropriate for the bridges designed by the less onerous standards. It is evident from Figure 9 that the second repair strategy results in a more sensible schedule of repair activities, to the point where the *Eurocode* designed bridge will maintain a level above β_T for the 80 year assessment period, under the presented deterioration model. Furthermore, bridges designed under *BD 21* and *BD 37* require only one repair activity, whereas those designed under *BS 183* and *BS 5400* require 8 and 9, respectively. This is further evidence of apparent life-cycle cost savings available with a small increase in the initial investment in the structure.

6 CONCLUSIONS

A structural reliability analysis was conducted on a RC slab bridge to assess the effect of changing definitions of normative traffic loading on safety classifications of the structure. It was observed that earlier codes produced less onerous flexural load effects and, as such, resulted a reduced demand for flexural capacity. It was shown that bridges produced under loading prescribed by modern standards produced bridges with a higher β assessed under a probabilistic load model, and resulted in a significantly reduced expected life-cycle cost, despite the increased initial construction costs due to a higher minimum requirement for flexural reinforcement. This increased initial cost was seen to be significantly offset with a lower expected cost of failure, which is a function of the probability of failure and thus the reliability index β.

This gives rise to the question as to whether there is an optimum point at which the initial cost can be increased to minimise the total expected life-cycle cost, and is there further variables that can be optimised at the design stage for bridges. Furthermore, the practical ways such a philosophy can be adopted in normative standards are unanswered; be it through refinements of partial factors for resistance variables such as the area of steel A_s or the compressive strength of concrete f_c, or through the a more holistic increase in safety factors regarding the applied traffic loading. Despite these remaining issues, it is clear from the presented results that there is scope for significant savings through a more conservative approach at the design stage.

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