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Target reliability for new road bridges in South Africa

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Currently, there are no recommendations of target reliability for new-build road bridges tailored to developing countries. Target reliabilities in existing international literature reflect the economic and social circumstances associated with developed countries and may not be economically or societally acceptable for South Africa. This research determines target reliability for new-build road bridges specifically for a South African context. Economic cost optimisation and a societal risk approach are considered. Cost optimisation considers the cost of safety and consequences of failure. The life quality index methodology is used to consider limits on target reliability from a societal risk perspective. Additionally, the effect of structural redundancy on target reliability in bridges is investigated. Target reliabilities from cost optimisation are shown to be slightly higher than those from SANS 10160-1 and correspond with those from ISO 2394:2015. Recommendations of annual target reliability for new-build road bridges in South Africa are proposed – a typical value of $\beta = 4.2$ is recommended. Societal limits on target reliability correspond with target reliability from cost optimisation for bridges with minor consequences of failure and do not govern new-build bridges in South Africa. Target reliability reduces with increasing measures of bridge redundancy; a means by which to consider the effect of structural redundancy on target reliability is also proposed.

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INTRODUCTION

Target reliability values for generic structures in South Africa (SA) are well defined in SANS 10160-1 (SANS 2018), with appropriate information regarding consequence classes and associated consequences of failure. Target reliability for the design of bridges in SA, however, is not as clearly defined. Current code provisions for the design of new-build bridges in the form of TMH7 (CSRA 1998), based on the 1978 CEB-FIP model code (CEB-FIP 1978), give little information about target reliability levels, consequence classes or the level of reliability that is currently implied by design thereto. This is due to the fact that at the time of compilation of TMH7, circa 1980, the use of reliability as a basis for structural design was still in its infancy. Recent research (Lenner *et al* 2021; Van der Spuy 2020) indicates that the currently achieved levels of reliability from TMH7 may not be in line with those from SANS 10160-1. Irrespective, TMH7 is now outdated and needs considerable revision or replacement. One of the foundational bases behind a potential revision or replacement of TMH7 is that of target reliability for new-build road bridges in SA.

Target reliability is broadly dependent on the costs associated with increasing safety and the economic, societal and environmental consequences of structural failure. As such, target reliability is determined through cost optimisation. In cases where human safety is at risk in the event of structural failure, additional constraints are imposed in the form of societal absolute-minimum reliability levels. These lower limits on reliability safeguard against realisations of structures that subject members of society to unacceptably high probabilities of death or injury, even though they may be cost-optimal. These are imposed based on an assessment of the efficiency of risk reduction measures from a societal perspective.

Extensive guidance on target reliability can be found in existing literature, including proposed target reliabilities based on cost optimisation and societal lower limits. These are proposed based on the economic and societal circumstances in developed countries, however, and the direct application of these to a developing nation without consideration of the economic circumstances and societal preferences unique to that society is unlikely to yield appropriate target reliability (Rackwitz 2000; Faber & Virguez Rodriguez 2011; *fib* 2020). Target reliability for generic structures in SA for a design life of 50 years (1 year) with

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moderate consequences of failure is given as β_t = 3.0 (4.0), as back-calibrated from existing practice by Milford (1988) and Kemp *et al* (1987), and later retained by Retief and Dunaiski (2009). Comparing this to the EN 1990 (EN 2002) equivalent of β_t = 3.8 (4.7), it is unlikely that simply adopting existing recommendations for developed countries would be appropriate for SA without considering the prevailing economic and societal preferences.

Furthermore, the effect of structural redundancy has only been included in the determination of target reliabilities in SA by distinguishing between brittle and ductile failure modes, and a higher target reliability being necessitated by the former. Highly redundant structures have lower consequences in the case of ULS (ultimate limit state) failure on an element level than non-redundant structures and should thus have lower target reliabilities as a consequence. However, there is currently very limited opportunity for a designer to incorporate structural redundancy into the choice of target reliability in SA.

This research therefore aims to determine target reliability values for new-build road bridges on structural level, based on cost optimisation and societal lower limits specifically for a South African context, considering the effects of structural redundancy. The results are then compared to those from SANS 101060-1 and existing literature from developed countries.

REVIEW OF EXISTING LITERATURE

Target reliability in new-build road bridges is the higher of values determined from cost optimisation and societal requirements. A brief summary of the determination of target reliability through cost optimisation is presented [see Rackwitz (2000) for comprehensive details and discussion], followed by a discussion on the effects of structural redundancy. The lower limits on target reliability from a societal perspective are then summarised.

Target reliability through cost optimisation

Target reliability obtained from cost optimisation in structures was formalised into a codified form by Rackwitz (2000). This methodology makes use of a generic function that describes the total cost of the structure, Z(d), in conjunction with a reliability limit state for ULS failure ($g_{ULS}(d)$). Both the cost function and the limit state are dependent on a decision parameter, d, that most cost-efficiently increases the marginal reliability (longitudinal or shear reinforcement area, concrete strength, effective depth, etc). Costs considered in the optimisation include initial construction costs $C(d)$, obsolescence costs $A(d)$ and costs related to ULS failure $D(d)$. The costs related to fatigue and ageing are not considered in this case, as the majority of the SA bridge stock is of reinforced or prestressed concrete construction, the design of which is not typically governed by fatigue or ageing of materials. When determining ULS target reliability, the costs of SLS (serviceability limit state) failures have typically been neglected in previous research and standards (ISO 1998; ISO 2015; Van Coile *et al* 2017; *fib* Bulletin 80 2016). This is likely due to the subjectivity of the costs associated therewith and their relatively small effect on target reliability. Inspection and maintenance are assumed to be included in the construction costs. The total cost function is thus given as:

$$Z(d) = C(d) + A(d) + D(d) \tag{1}$$

The construction costs are defined by $C(d) = C_0 + C_1 \cdot d$, where C_0 are construction costs unrelated to the decision parameter and C_1 are those costs dependent on changes to d. Obsolescence costs, based on renewal theory (Pandey *et al* 2015), account for losses incurred when the structure is demolished and reconstructed at the end of its intended design life and are given as $A(d) = (C_0 + C_1 \cdot d + A) \cdot \omega/\gamma$. This includes the "wasted" construction costs $C_0 + C_1 \cdot d$, as well as the costs of demolition and clearing, A. These are multiplied by an obsolescence rate, ω, that describes the likelihood of the structure becoming obsolete before the end of its intended design life, be that due to economic, social or functional reasons, amongst others (Diamantidis *et al* 2018). Obsolescence costs occur in the future and must be discounted back to present value using the discount rate for exponential discounting, $\approx 1/\gamma$, assuming a stationary Poisson process for obsolescence event occurrences. The other potential future costs are similarly discounted to present value.

Costs related to ULS failures are given by $D(d) = (C_0 + C_1 \cdot d + H) \cdot p_{f,ULS}(d)/\gamma$ and are affected by the probability of ULS failure, $p_{f,ULS}(d)$. The H term relates to indirect costs due to damage in the case of ULS failure (compensation for life and limb lost, demolition and clearing of the site, loss of service, economic costs, etc). The $C_0 + C_1 \cdot d$ terms (direct structural costs) reflect the assumption that, in the case of ULS failure, the structure or part thereof is systematically rebuilt after failure. Additionally, $D(d)$ implies that a future single passing into the ULS failure domain results in damage to the structure (member failure, partial or total collapse).

Due to the vastly different environments, locations, construction methods, age and condition of individual bridges, estimating the indirect economic failure costs (excluding compensation for life and limb) present in H is complex and contains notable uncertainty (Imam & Chryssanthopoulos 2012). In-depth methodologies are presented in Lee *et al* (2006), and De Brito and Branco (2011). However, much of the information required to use these models is unlikely to be available at design stage, in addition to the complexity and time required to execute the methodologies. As such, indirect economic costs are often simply approximated using value-of-time estimates of monetary units per person-hour, in conjunction with the expected delays caused due to the reconstruction/repair of the specific bridge (Imam & Chryssanthopoulos 2012). Guidelines for the estimation of failure costs related to compensation for lost life and limb that contribute to H are given later in the section titled *Societal limits on target reliability for South Africa* (page 16).

The total cost is usually normalised by C_0, which negates the need for an explicit specification of initial cost. The final, normalised form of the cost function is denoted as $z(d)$ and is given in Equation 2. The optimal reliability is then the point where d minimises the total cost of the structure, i.e. where $\partial z(d)/\partial d$ is zero, $d = d_{opt}$.

$$z(d) = \left(1 + \frac{C_1}{C_0} \cdot d\right) + \left(1 + \frac{C_1}{C_0} \cdot d + \frac{A}{C_0}\right) \cdot \frac{\omega}{\gamma} + \left(1 + \frac{C_1}{C_0} \cdot d + \frac{H}{C_0}\right) \cdot \frac{p_{fULS}(d)}{\gamma} \tag{2}$$

The cost function is evaluated in conjunction with a generalised ULS considering the difference between resistance and load effects, $g_{ULS}(d) = R(d) - S$. In the generic formulation, the decision parameter is assumed to affect the mean value of the structural resistance, μ_R, only with negligible effect on the load. The mean value of resistance is therefore given by $\mu_R = d$ and the mean value of load, μ_S, is unaffected by changes in d. The difference between μ_R and μ_S is varied through increases in d, i.e.

a design engineer would adjust μ_R as part of the design process. The ULS probability of failure is thus decreased by increases in d. The load and resistance are also affected by their respective coefficients of variation (CoV), V_R and V_S. Target reliabilities based on this cost optimisation procedure, from ISO 2394:2015 (ISO 2015), are given later in the section titled *Current recommendations of target reliability* (page 13).

Effect of redundancy on target reliability in bridges

Most basis-of-design standards include a clause to the effect that structural damage suffered should not be disproportionate to the cause of the adverse effect. To satisfy this requirement, structural redundancy/robustness in design is encouraged. However, the effect of redundancy in generic structures is difficult to quantify because of the wide range of structures, structural arrangements and materials that exist. Due to the more uniform function of bridges, the quantification and effect of redundancy are generally simpler. Redundancy factors have been applied to either load or resistance in bridges through various modifying factors in the American AASHTO LRFD (AASHTO 2012) and Canadian CAN/CSA-S6-06 (CSA 2006) design methods, though seldom with a direct link to the target reliability (Ghosn *et al* 2014).

In this research, a simplified approach to structural redundancy in bridges is proposed that is included in the cost optimisation, which links it directly to target reliability. In typical applications of the generic cost optimisation, a single breach into ULS failure of an element results in the collapse of the entire structure, as is evident from the 3rd term in Equation 2, where the reconstruction cost of the structure ($C_0 = C_1 \cdot d$) and the remaining failure costs (H) are multiplied by the probability of ULS failure. In the case of a single-span bridge where there is no redundancy, this is a reasonable assumption. In a bridge with multiple spans or where each span is effectively a "standalone" structure, however, this assumption is less reasonable. The effects of redundancy can be incorporated into this formulation by consideration of the effect of redundancy on the ULS probability of failure; however, this is often complex and involves the use of subjective conditional probabilities. Alternatively, the costs of failure can be adjusted to account for the portions of the structure that remain intact after a ULS failure event. The latter approach is adopted here through the use of a redundancy factor as described below.

Consider a multiple-span bridge that uses precast concrete girders, where each span is effectively separate from adjacent spans. In the case of failure of a bridge girder, typically only one span will partially or fully collapse, leaving the rest of the bridge largely intact. Alternatively, a pier may fail, typically causing two of the spans to collapse, but the remainder of the bridge may remain intact. This will not apply in a case where failure casts doubt on the fidelity of the remaining parts of the structure. In the former case, however, the cost of reconstruction will be proportional to the collapsed parts and not the entire bridge, due to the redundancy in design. In *fib* Bulletin 80 (2016), the number of fatalities in bridge collapses was found to be strongly correlated to the collapsed span length. Indirect economic losses due to delays, detours, etc, are also likely to be correlated to a similar extent, as only the collapsed section will need to be reconstructed, implying that the bridge will return to service sooner than if the entire bridge had collapsed. As such, the consequences of failure in terms of life, limb and indirect economic costs are assumed to be strongly correlated with the collapsed length.

The structural redundancy can therefore be measured in a simplified form for bridges using a redundancy factor, η_{red} as shown in Equation 3, where L_c is the length of bridge that is likely to collapse in the case of ULS failure, and L_b is the total length of the bridge. A redundancy value of $\eta_{red} = 0$ indicates no redundancy, e.g. a single span bridge with no alternate load path or a bridge where the structural integrity of the entire bridge depends on the reliability of a single member. Conversely, a value of η_{red} near one indicates a highly redundant bridge of multiple spans that are independent of one another, or where multiple girders make up the superstructure.

$$\eta_{red} = 1 - \frac{L_c}{L_b} \tag{3}$$

The redundancy factor is incorporated into the cost optimisation, as shown in Equation 4. The damage cost as a result of a ULS failure is scaled by 1-η_{red}, reflecting the effect that increased structural redundancy has on reducing the costs and expected consequences of failure.

$$z(d) = \left(1 + \frac{C_1}{C_0} d\right) + \left(1 + \frac{C_1}{C_0} d + \frac{A}{C_0}\right) \frac{\omega}{\gamma} + \left(1 - \eta_{red}\right) \cdot \left(1 + \frac{C_1}{C_0} d + \frac{H}{C_0}\right) \cdot \frac{p_{fULS}(d)}{\gamma} \tag{4}$$

Societal limits on target reliability

Target reliabilities from cost optimisation are optimal from an economic perspective, but these are subject to minimum reliabilities that are introduced from a societal risk perspective. In new-build structures, ULS target reliability from cost optimisation is typically higher than societal risk criteria (Steenbergen *et al* 2015). Popular approaches aimed at quantifying these limits are the consideration of individual and group risk requirements and the life quality index (LQI) method, amongst others. Individual risk criteria are not considered for bridges, however, due to the negligibly small duration of time that any one individual spends on/under a bridge (*fib* Bulletin 80 2016). Group risk criteria is the method preferred by *fib* Bulletin 80 (2016), but this method is subjective, and the parameters used therein are biased towards a developed-country environment. The LQI criteria are therefore used here to provide lower limits on target reliability for South Africa, as a developing country. The LQI has the advantage of considering society-specific willingness and ability to pay to avoid injury or death, which enables the limits to be specifically applied to a South African society.

LQI approach

The LQI methodology, initially developed by Pandey and Nathwani (2004), is used as a means by which to evaluate whether semi-voluntary risks taken by members of society are acceptable or not. It considers the ability and willingness of a society to pay for an increased probability of survival and is thus unique to different societies. The LQI has been adapted over time to be more readily used in cost optimisation applications for structural reliability, notably by Rackwitz (2005) and Fischer *et al* (2012; 2019). When combined with cost-optimisation principles, the LQI is used to impose society-specific lower limits on target reliability. Target reliability levels below these limits are considered unacceptable from a societal risk perspective. The structural reliability-specific LQI criterion in terms of cost-optimisation principles is given as:

$$-C_x \cdot \frac{g}{q} \cdot \frac{N_{PE}\, k}{\gamma_s} \cdot \frac{\partial p_{f,ULS}(d)}{\partial d} \leq \frac{\partial (C(d) + A(d))}{\partial d} \tag{5}$$

Equation 5 is based on the principle that, where human life is at risk in the case of failure, an investment into increasing societal safety (marginal cost on the right) must be made that is at least the equivalent of what society is willing to pay to save the number of statistical lives at risk in the case of failure (left). All parameters are to reflect a purely societal risk perspective. Notably, the $D(d)$ term is neglected on the right-hand side in comparison to Equation 2, as the reduction of this term constitutes a monetary benefit rather than a cost; see Fischer *et al* (2013) for further discussion. The g, q and C_x terms are the gross domestic product (GDP) per capita available for risk reduction, a constant depending on the ratio of work-to-leisure time and a demographic constant, respectively, from the classic LQI criterion. The $C_x \cdot g/q$ term is interpreted as the societal willingness to pay (SWTP) to save a statistical life (G_x). The product of the number of people exposed to harm in the case of failure (N_{PE}) and the probability of death given failure (k) gives the number of expected fatalities in the case of failure, N_F. Additionally, the γ_s term is a societal discount rate, as opposed to one from a private investment perspective. The societal discount rate takes cognisance of intergenerational discounting, as is appropriate from a societal perspective. Simplifying, the form found in ISO 2394:2015 (ISO 2015) is obtained in Equation 6. The minimum reliability level is then obtained as a function of societal costs, as shown in Equation 7.

$$-\frac{\partial p_{f,ULS}(d_{hs})}{\partial d} \leq \frac{C_1(\gamma_s + \omega)}{G_x \cdot N_F} = K_1 \tag{6}$$

$$\beta = -\Phi^{-1}[p_{f,ULS}(d_{hs})] \tag{7}$$

The final target reliability is then the highest of that from cost optimisation and human safety. Societal limits imposed by the implementation of the LQI methodology are shown in the following section.

Current recommendations of target reliability

A brief summary of existing recommendations on target reliability from international sources is now considered. As alluded to in the previous sections, target reliability depends on the consequences of failure and the cost to increase safety. Quantitative ranges of these parameter values are given in Table 1. The consequences of failure are described in the JCSS Probabilistic Model Code (PMC) (JCSS 2001) as the total failure costs as a function of the initial construction costs, i.e. $\rho = (C_0 + C_1 \cdot d + H)/(C_0 + C_1 \cdot d)$. From this, it is convenient to have an approximation of the costs of failure excluding reconstruction costs (H), as a ratio of the initial construction costs: $H/C_0 \approx \rho - 1$. The relative cost ranges from the JCSS PMC are shown in Table 1 and correspond to the qualitative descriptions of the consequence classes of minor (CC1), moderate (CC2) and large (CC3) consequences of failure found in, for example, SANS 10160-1 (SANS 2018) and EN 1990 (EN 2002). Target reliability for structures with $9 < H/C_0$ is recommended to be evaluated using a full cost-benefit analysis and is not considered in this study. From these ranges, Fischer *et al* (2019) back-calculated order of magnitude relative costs of increasing safety (C_1/C_0), as shown in Table 1. The midpoints of these ranges are typically used as being representative of the range and are used throughout this study.

Table 1 Consequences of failure and costs of increasing safety

Consequence Class	H/C_0	Relative costs of increasing safety	C_1/C_0
Minor (CC1)	$H/C_0 \leq 1$	Low	$10^{-4} \leq C_1/C_0 < 10^{-3}$
Moderate (CC2)	$1 < H/C_0 \leq 4$	Medium	$10^{-3} \leq C_1/C_0 < 10^{-2}$
Large (CC3)	$4 < H/C_0 \leq 9$	High	$10^{-2} \leq C_1/C_0 < 10^{-1}$

Table 2 gives annual target reliabilities, according to SANS 10160-1, EN 1990 and ISO 2394:2015 (SANS 2018; EN 2002; ISO 2015), as a function of the consequences of failure and the cost of safety measures, where applicable. Note that lifetime target reliability values given in SANS 10160-1 and EN 1990 relate to the probability of failure for a 50-year reference period and have been converted to a 1-year reference period in Table 2, assuming independent failure events [see Holický *et al* (2015) and Holický *et al* (2018) for relevance of reference periods]. From Table 2, higher consequences of failure necessitate higher levels of reliability. Conversely, higher levels of reliability are warranted by lower relative costs of increasing safety, as increasing safety is relatively inexpensive. The ISO 2394:2015 (ISO 2015) recommendations indicate that the limits imposed by the LQI criterion coincide with the target reliability for minor consequences of failure and therefore do not govern for new-build structures in developed countries.

Table 2 Annual target reliabilities from SANS 10160-1, EN 1990 and ISO 2394:2015

Failure consequences	SANS 10160-1	EN 1990	ISO 2394:2015 (Cost opt)**	ISO 2394:2015 (LQI)
Minor (CC1)	3.7	4.2	3.1 / 3.7 / 4.2*	3.1 / 3.7 / 4.2*
Moderate (CC2)	4	4.7	3.3 / 4.2 / 4.4*	
Large (CC3)	4.4	5.2	3.7 / 4.4 / 4.7*	

* For high / medium / low relative cost of safety measures
** Taken from the JCSS PMC (JCSS 2001)

TARGET RELIABILITY FOR NEW-BUILD BRIDGES IN SOUTH AFRICA

The target reliability values in Table 2 are intentionally given for a generic structure so as to be broadly applicable to a wide range of structures. However, there is benefit to be gained by setting target reliability specifically for bridges, as bridges have structural behaviour, construction costs and consequences of failure that are distinctly different from generic building structures. Furthermore, the societal lower limits imposed in developed countries may not be suitable to a local context. As such, this section tailors target reliability specifically to new bridges designed in a South African context on structure level. Target reliability from cost optimisation, considering structural redundancy, is investigated, after which the lower limits on reliability imposed by the societal LQI criteria are investigated.

Target reliability in South Africa through cost optimisation

To determine target reliability from a cost optimisation perspective, parameters for the cost optimisation must be chosen to be used in conjunction with Equation 2.

Table 3 Parameters for the cost optimisation

Parameter	Value
γ	0.08
ω	0.01
Resistance	$LN(d, 0.175)$
Load	$LN(1, 0.15)$
A/C_0	0.2
H/C_0	Varied (0 to 9)
C_1/C_0	Varied ($5 \cdot 10^{-2}$ to $5 \cdot 10^{-4}$)

Table 4 Annual target reliabilities from cost optimisation for bridges in South Africa

Failure consequences	Cost of increasing safety		
	High $10^{-2} \leq C_1/C_0 < 10^{-1}$	Medium $10^{-3} \leq C_1/C_0 < 10^{-2}$	Low $10^{-4} \leq C_1/C_0 < 10^{-3}$
Minor ($H/C_0 \leq 1$) CC1	3.3	**3.9**	4.4
Moderate ($1 < H/C_0 \leq 4$) CC2	3.6	**4.2**	4.6
Large ($4 < H/C_0 \leq 9$) CC3	3.8	**4.3**	4.8

As most road bridges in SA are made of either reinforced or prestressed concrete, the parameters chosen are biased more towards concrete construction. Annual target reliabilities for steel bridges will be marginally higher ($\Delta\beta \approx 0.1$), due to the reduced uncertainty in the structural resistance. The parameters are shown in Table 3 and discussed below.

The structural resistance is assumed time-invariant and is represented by a lognormal distribution (Holický 2009) in the ULS reliability limit state with a mean value that is linearly increased by the decision parameter. Sýkora *et al* (2015) report CoV for concrete resistance in flexure of 0.1, and 0.2–0.25 for concrete in shear. Holický (2009) suggests a value between 0.1 and 0.18. A CoV of $V_R = 0.175$ is used here for the structural resistance, which includes provision for model uncertainty. Lenner *et al* (2021) and Van der Spuy (2020) concluded that the load effect of traffic loading in SA is best modelled by a censored generalised extreme value distribution. In previous research by Steenbergen *et al* (2018) and Fischer *et al* (2019), however, it was found that the combined load effect of permanent (normal distribution) and variable loading (extreme value distribution) was well represented by a lognormal distribution for V_R, $V_S < 0.3$. The combined load effect is therefore modelled by a lognormal distribution. In Holický (2009), a V_S value between 0.03 and 0.1 for permanent load is recommended. Annual maxima of traffic loads on bridges have a CoV of $V_S \approx 0.03 - 0.15$ (ARCHES 2009). In short bridges where the variable traffic load dominates, a higher V_S is appropriate, whereas for medium to long spans, the magnitude of permanent and variable load is more equal, resulting in a lower V_S value. An allowance is made, however, for the effect of wind loading on lighter bridges, seismic actions and other variable loads which typically have $0.3 \leq V_S$ (Ghosn & Moses 1991). A value of $V_S = 0.15$ is therefore used for the combined load effect here, which includes model uncertainty.

A relative demolition and clearing cost of $A/C_0 \approx 0.2$, as used in Rackwitz (2000), seems to be a reasonable assumption for bridges in SA from the authors' previous experience, although this will vary depending on the specific design. The choice of A/C_0 value has a relatively small effect on the target reliability (Rackwitz 2000) and thus an approximation is sufficient. A lower obsolescence (renewal) rate of $\omega = 0.01$ is appropriate for bridges where, at the end of the design life, it is likely to be replaced by a structure with the same function (Diamantidis *et al* 2018; Rackwitz 2000). The discount rate is one of the most notable differences between a developed and a developing country. Those for the former are typically in the range of 0–3% (Rackwitz 2000), whereas those for the latter generally range from 8–15% (Mullins *et al* 2014). Rackwitz (2000) uses a value of 3%, whereas the discount rate in SA is on the lower limit of the developing country range, at 8% (Mullins *et al* 2014; Harrison 2010; Campos *et al* 2015). Using the parameters from Table 3, the cost optimisation is performed using Equation 2. All analyses are performed using the first order reliability method (FORM) within the PyRe/Pystra Python reliability module. The results are shown in Figure 1 and Table 4 and discussed below.

The target reliabilities obtained from the cost optimisation for bridges in SA overall are slightly higher than those in

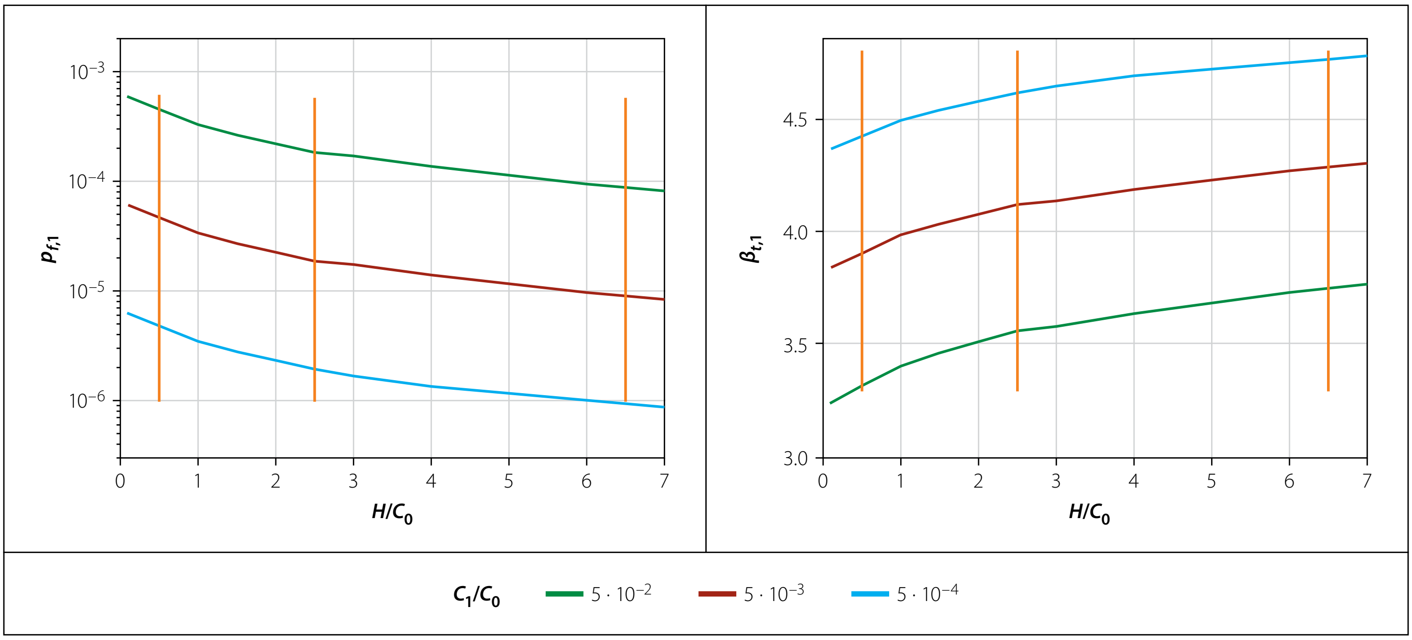

Figure 1 Annual probability of failure and target reliability for South Africa as a function of the relative costs of safety (C_1/C_0) and consequences of failure (H/C_0)

SANS 10160-1 (SANS 2018) and the generic cost optimisation from ISO 2394:2015 (ISO 2015) in Table 2. Target reliability from EN 1990 (EN 2002) remains the highest in all consequence classes, though direct comparison is made difficult due to EN 1990 $\beta_{t,1}$ values being influenced by back-calibration to existing practice. Annual target reliability for a generic structure with moderate failure consequences and medium costs of safety, $\beta_{t,1} = 4$, is the default value used in SANS 10160-1, with the equivalent for bridges determined as $\beta_{t,1} = 4.2$. Though this is slightly higher than the SANS 10160-1 equivalent for generic structures in SA, it is slightly lower than that determined by Nowak (1999) ($\beta_{t,1} = 4.4$) for bridges in the USA. The increase in $\beta_{t,1}$, compared to the ISO 2394:2015 generic optimisation, is due to higher values of V_R and V_S being used in ISO2395:2015 to account for a wider range of resistance and loads that are more variable than those present in the case of bridges only. Increasing both V_S and V_R to 0.3 results in a reduction of $\beta_{t,1}$ values in the order of ≈ 0.3, indicating the relatively low sensitivity to choices of V_S and V_R. The increases in $\beta_{t,1}$ from ISO 2394:2015 are, however, slightly offset by the higher discount rate used for South Africa, which reduces the target reliability. It should be noted here that choosing a value of $\beta_{t,1}$ slightly higher than that from the optimisation is not essentially wrong. It will not be optimal, however, as the total normalised structural cost typically only increases marginally for decision parameter values slightly above the optimal point, i.e. when $d_{opt} < d$ (unless C_1/C_0 is high), as can be seen in Appendix B of Rackwitz (2000).

The analysis has thus far not considered structural redundancy. The following section explores the concept of structural redundancy in bridges and how it affects target reliability.

Consideration of structural redundancy in bridges

Cost optimisation is now repeated considering the effect of structural redundancy by varying $0 \leq \eta_{red} < 1$. The results are shown in Figure 2 as a function of η_{red} for minor, moderate and large consequences of failure and costs of increasing safety as before. Target reliability is shown to reduce with increasing redundancy. For moderate consequences of failure and medium costs of increasing safety, the target reliability reduces from $\beta_{t,1} = 4.2$ at $\eta_{red} = 0$ to 4 and 3.7 for $\eta_{red} = 0.4$ and 0.8, respectively,

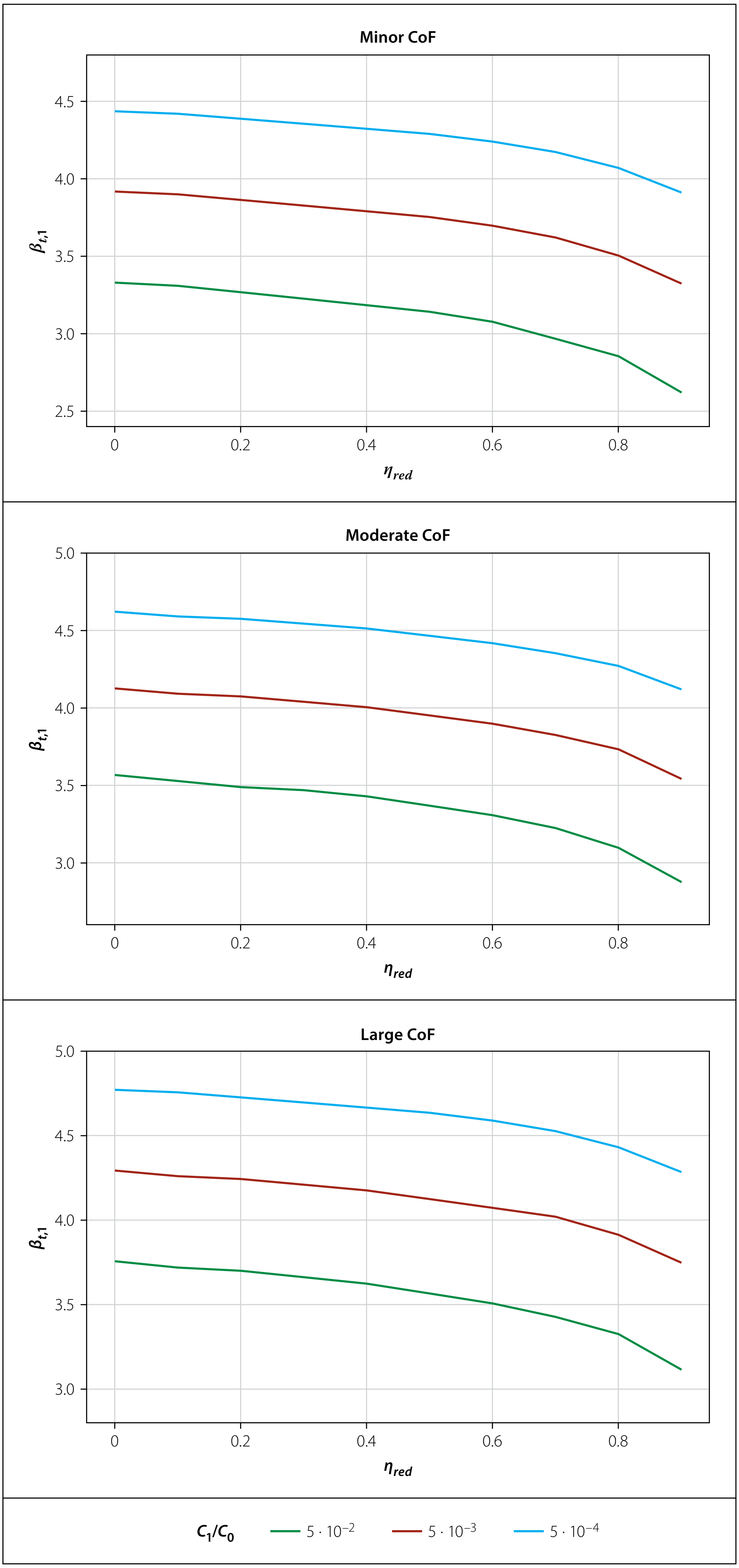

Figure 2 Annual target reliability as a function of the relative costs of safety (C_1/C_0) and redundancy factor η_{red} for minor, moderate and large consequences of failure (CoF)

showing the effect that redundancy plays in the choice of target reliability and the possible cost savings that may result.

Of course, estimating the redundancy of a bridge is not always straightforward. In the case of bridges where adjacent spans are effectively independent of one another, assuming a ratio of two span lengths to the total length of the bridge is a sensible estimate (assuming a pier fails). Post-tensioned bridges are more complicated, as failure in one span is likely to cause a measure of failure in adjacent spans, unless each span is tensioned separately. While the proposed definition of η_{red} will not be able to fully capture the redundancy of all realisations of bridge designs and geometries, the usefulness of η_{red} is its simplicity and direct link to target reliability. Obviously, a value of $\eta_{red} = 0$ is always a conservative assumption. Ultimately, the design engineer needs to use his/her discretion as to the η_{red} factor to use, based on the fraction of the bridge length that is likely to remain intact in the case of a ULS failure.

Societal limits on target reliability for South Africa

Societal limits do not typically govern target reliability for new-build structures. However, given the possible reductions in target reliability in the case of highly redundant bridges, it is important to check that the societal risk criteria are satisfied considering local societal preference. The parameters in Equations 5 and 6 are considered from a South African perspective.

In the development of the demographical constant (C_x) for use in the SWTP (G_x) in bridges, the use of the so-called Δ mortality reduction scheme (C_Δ) is appropriate, as it makes use of age-averaged discounting [see Rackwitz (2008) for discussion]. Both C_Δ and the constant related to the work-to-leisure time fraction, q, were determined for SA in Smit (2014), and Smit and Barnardo-Viljoen (2013) as 17.55 and 0.21, respectively. It is assumed that neither of these society-specific parameters have changed significantly since 2014. When considering the GPD per capita available for investment into risk-reduction measures, g, some sources suggest using the entire GDP per capita (ISO 2015). This seems unrealistic for a developing country, however, given that the parts of a country's GDP used for reinvestment into the economy and to pay off national debt cannot be used towards safety investment measures. Rackwitz (2008) therefore suggests using 60% of the GDP as

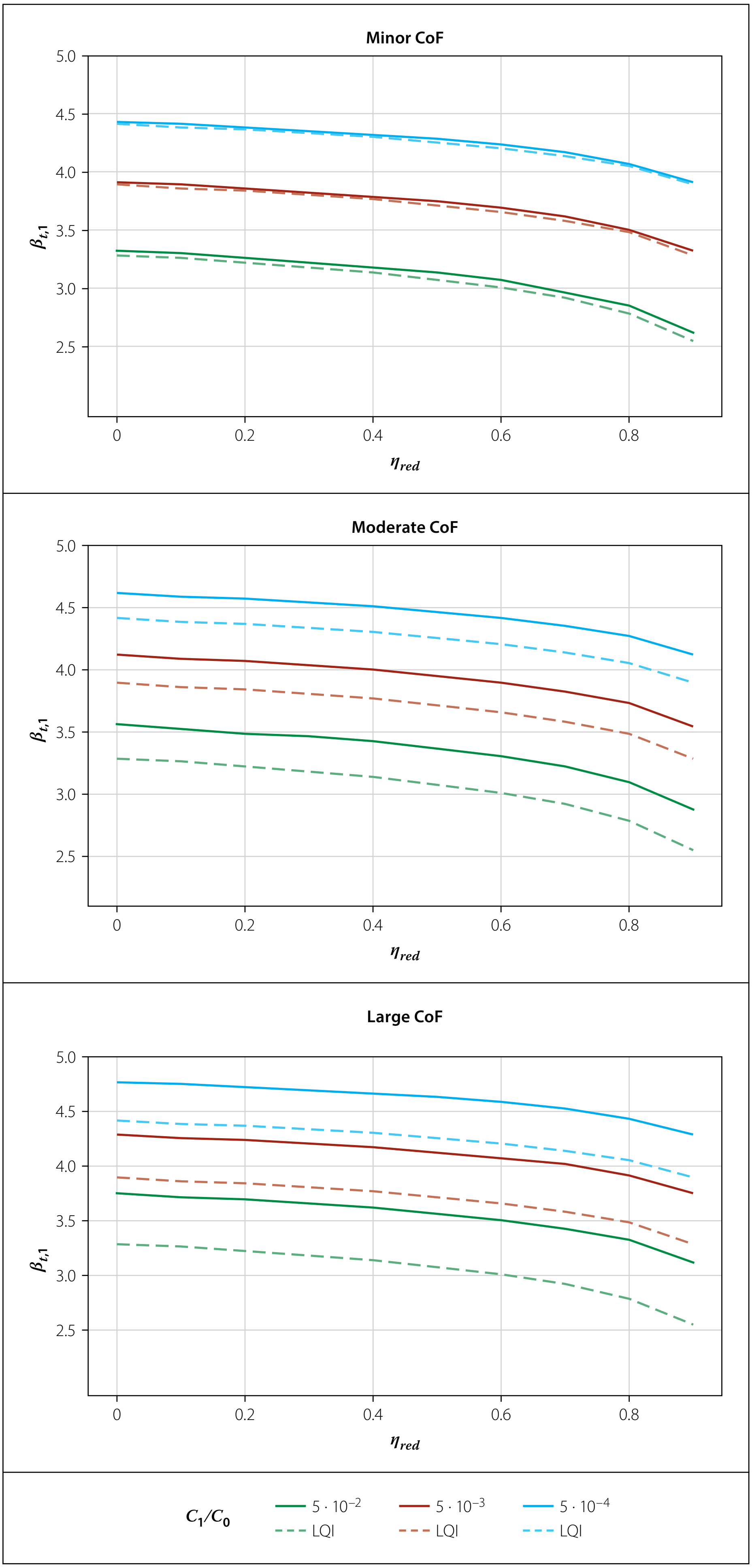

Figure 3 Annual target reliability with LQI societal limits for South Africa as a function of the relative costs of safety (C_1/C_0) and redundancy ratio for minor, moderate and large consequences of failure (CoF)

Table 5 Parameters for SA lower societal target reliability limits on the basis of LQI

Parameter	Value
γ_s	0.02
ω	0.01
G_Δ	$R\ 4.3 \times 10^6$
C_1	Varied as a function of C_0
N_F	$(1 - \eta_{red}) \cdot 0.09 \cdot L_b$

Table 6 Example bridge geometry/parameters

Parameter	Value	Parameter	Value	Parameter	Value
Spans	5	C_0	R20 million	Piers (wall type)	4
Span length/ width	12 / 11 m	L_b	60 m	Pier width/ thickness/height	10 / 0.4 / 6 m

a value for g. The SA GDP for 2019 is used to avoid the (assumed) relatively short-term effects of the Covid-19 pandemic on the economy. The GDP for 2019 was R5.1 trillion (National Treasury SA 2019), and with an estimated population of 58.8 million (Statistics South Africa 2019), the available GDP per capita for risk reduction is R52 041. From the above, the SWTP is calculated as $G_\Delta = 17.55 \cdot 52\ 041/0.21$ = R4.3 million.

The annual sustainable discounting rate, γ_s, for inter-generational societal decision-making is generally in the vicinity of 2–4% (ISO 2015). Smit (2014) calculates a value of $\gamma_s \approx 2\%$ for SA on the basis of an overlapping age group model for a design life of 100 years. The obsolescence rate used is $\omega = 0.01$, as before. As the cost of increasing safety is generally given as a ratio of the initial cost, a fixed value needs to be assumed for C_0 in order to make use of the LQI limits. As such, a 100 m long, 12 m wide, 4-span, post-tensioned twin-spine highway bridge is considered as a typical application, with $C_0 \approx$ R30m. The number of fatalities expected in the case of failure, N_F, is estimated as a function of the collapsed bridge length as $0.09 \cdot L_c$ from an investigation of bridge collapses in *fib* Bulletin 80 (2016). As before, the collapsed bridge length varies as a function of the redundancy ratio. The number of expected fatalities is therefore given as $N_F = (1 - \eta_{red}) \cdot 0.09 \cdot L_b$. It should be noted that the bridge collapses considered in the *fib* Bulletin 80 (2016) are limited in number and were predominantly of Class CC3 or higher. The estimate of N_F may therefore be conservative for CC2 and CC1 and may contain a fair degree of uncertainty. The cost of increasing safety is varied as a function of C_0. The LQI criteria are calculated as per Equation 6 and are shown in comparison to the target reliabilities from cost optimisation in Figure 3.

At $\eta_{red} = 0$, the annual LQI reliability limits for high, medium and low relative costs of increasing safety are given as $\beta_{t,1,LQI} = 3.3$, 3.9 and 4.4, respectively for moderate CoF.

From Figure 3 and Table 4 it can be seen that the LQI limits correspond exactly with the target reliability for bridges with minor consequences of failure, similarly to the LQI results from ISO 2394:2015 for generic structures in the context of a developed country (Table 2). The lower societal limits on reliability therefore do not govern target reliability for new-build road bridges in SA. Given that they are on the verge of governing for minor consequence of failure, however, they may well govern target reliability for some existing bridges, but this requires further research to confirm. The LQI requirements here are lower than those for generic structures in developed countries as a result of the lower SWTP in SA, as well as due to the difference in the estimation of the number of fatalities in bridges versus generic structures in the case of ULS failure (collapsed bridge length versus collapsed floor area). Recommendations of annual target reliability for new-build road bridges in SA are therefore given by Table 4. The effects of structural redundancy on target reliability can be incorporated using Figure 2.

Example

The choice of target reliability of a new bridge is illustrated here using an example. Consider a reinforced concrete road-over-road bridge, constructed using a superstructure that allows spans to behave independently of adjacent spans, with geometry and characteristics as shown in Table 6. Consider the same bridge, placed along two separate routes, for indirect cost comparison purposes. Bridge A is on a principal arterial route with an average annual daily traffic (AADT) of 100 000 vehicles and Bridge B is on a major arterial route with an AADT of 10 000 vehicles.

To determine an appropriate value of target reliability from Figure 2, the CoF excluding construction costs (H/C_0), and CoS (C_1/C_0) must be estimated, first assuming that the bridge has no redundancy ($\eta_{red} = 0$). Assume that the collapse of one pier led to the collapse of the entire bridge (60 m), the direct cost part of H related to compensation for life and limb lost is estimated as (from Table 5): $G_\Delta \cdot N_F = 4.3 \times 106 \cdot (1 - \eta_{red}) \cdot 0.09 \cdot L_b$ = R23.2 mil. The indirect economic costs are approximated for Bridges A and B, based on the time-cost due to delays, as shown in Table 7. As previously mentioned, these estimates are highly bridge-specific and contain considerable uncertainty. Assume that delays caused by traffic congestion or re-routing of traffic amount to 5 and 10 minutes per trip over a reconstruction time of 60 days for Bridges A and B, respectively. The AADT is broken up into various vehicle types, their proportion of total traffic and mean vehicle occupancy. The time-cost is then estimated based on the number of time-hours per person lost. Costs related to light- and heavy-goods vehicles are calculated based on the lost number of vehicle hours. The total consequences of failure excluding reconstruction, H, are then given as R114 million for Bridge A and R41.5 million for Bridge B, showing the notable variation in consequences of failure as a function of bridge location. With a C_0 value of R20 million, Bridges A and B have H/C_0 values of 5.7 (large) and 2.1 (moderate), respectively.

The decision parameter in the case of pier failure is typically the area of reinforcing (per metre width) along the height of the pier. The cost of the area of reinforcing in the four 10 m wide, 6 m tall wall type piers is thus (assuming a reinforcing cost of R14 000/ton and Y20-150 on each face, per metre):

$$C_1 = 4 \times 2 \times 2\ 095 \times 10^{-6}\,\frac{\text{m}^2}{\text{m}} \times 10\text{m} \times 6\text{m} \times 7.8\,\frac{\text{ton}}{\text{m}^3} \times \frac{\text{R}14\ 000}{\text{ton}} = \text{R}109\ 811$$

giving $C_1/C_0 = 5.5 \times 10^{-3}$ corresponding to medium CoS.

Thus, using Figure 2 with $\eta_{red} = 0$, $C_1/C_0 \approx 5.5 \times 10^{-3}$ for large and moderate CoF, the annual target reliability for Bridges A and B is estimated as 4.3 and 4.2, respectively.

Now consider a more realistic case, where the failure of one pier causes only the

Table 7 Estimation of indirect economic costs for example bridge

Vehicle type	Percentage of total traffic[a]	Mean vehicle occupancy[b]	Cost (R/hr/pp)[d]	Bridge A AADT 100 000		Bridge B AADT 10 000	
				Vehicle/day	Cost (R/day)	Vehicle/day	Cost (R/day)
Motorcycle	4	1.1	93	4 000	34 100	400	3 410
Buses	1	25	93	1 000	193 750	100	19 375
Taxis	2	14	93	2 000	217 000	200	21 700
Passenger car	65	1.4	93	65 000	705 250	6 500	70 525
Light goods	23	N/A	170[c]	23 000	325 833	2 300	32 583
Heavy goods	3	N/A	190[c]	3 000	47 500	300	4 750
				Total cost (5 min delay per day, 60 days) R91 406 000		Total cost (10 min delay per day, 60 days) R18 281 200	

a From SA DoT (2015). "Other" vehicle type ignored.
b From Stone *et al* (2018).
c Cost given in R/hour/vehicle.
d Adapted from Imam and Chryssanthopoulos (2012). Converted to Rand, adjusted by a ratio of SA to European GDP per capita and adjusted for inflation to 2019 values.

two adjacent spans to collapse or partially collapse (24 m, $\eta_{red} = 1 - 24/60 = 0.6$), leaving the rest of the bridge intact. The direct reconstruction and compensation costs reduce by ≈ 24/60 from R23.2 million to R9.3 million. Due to the reduced reconstruction time for a shorter collapsed length, the indirect costs are assumed to reduce by the same proportion, from R91.4 million and R18.3 million to R36.6 million and R7.3 million for Bridges A and B, respectively. While the indirect costs are unlikely to reduce to quite the same extent due to time-invariant costs, the assumption should be sufficient. From Figure 2, using the original CoF and CoS classes (to avoid double-accounting for the effect of the redundancy), with $\eta_{red} = 0.6$, the target reliabilities reduce to ≈ 4.0 and 3.9 for Bridges A and B, respectively.

CONCLUSION

This research considered target reliability in new-build road bridges for South Africa, given that there are no target reliability recommendations for these in existing literature. Target reliabilities for road bridges are determined both through economic cost optimisation and from a societal risk perspective using the life quality index (LQI), specifically for a South African context. The economic cost optimisation considered the cost of increasing safety and the consequences of failure, while the LQI criterion considered the cost of increasing safety and a South African societal willingness to pay to save a statistical life. Additionally, the effect of redundancy on target reliability was investigated through the use of a structural redundancy factor. Target reliability for South African bridges was found to be slightly higher than that for generic structures and is governed by cost optimisation for all typically considered consequence classes and cost of safety ranges. Recommendations of target reliability for new-build road bridges in South Africa are proposed. The annual target reliability index of a typical bridge in South Africa is recommended as $\beta = 4.2$. Lower limits on target reliability from LQI criteria were found to be equivalent to those from cost optimisation for bridges with minor consequences of failure, similarly to ISO 2394:2015. Target reliability for some existing bridge structures may therefore be governed by the LQI criterion and should be considered in future research. An increase in structural redundancy was shown to reduce the target reliability, due to the lowered consequences of element/section failure in a bridge with redundancy. A means by which to consider the effect of structural redundancy on target reliability has also been proposed.

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