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"Risk-based infrastructure design and assessment"

Prof Celeste Viljoen
Department of Civil Engineering
Faculty of Engineering

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Risk-based infrastructure design and assessment

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Biography of author

Celeste Viljoen is a professor of structural engineering at Stellenbosch University. She is a registered professional engineer with a PhD in structural health monitoring. Colleagues who have visited the Brackenfell area have likely crossed bridges designed by her. Her research focus is on risk-based decisions and structural reliability. She is widely involved in the standardisation of structural design and the advancement of probabilistic design principles that underpin this undertaking. She serves on the technical committees of the South African Bureau of Standards and the International Organization for Standardization and is a member of the international Joint Committee on Structural Safety. She contributed to updates in national and international standards for structural design. She regularly presents continuing professional development courses to practising engineers to disseminate advances in the field. To date she has graduated two DEng, 10 PhD and 10 MEng candidates with whom she authored and co-authored 31 journal articles, 44 peer-reviewed conference papers and three books. She has taught undergraduate and postgraduate courses on strength of materials, structural design, structural analysis, structural reliability and construction risk management. In September 2020 she was appointed as the first female vice-dean of the Faculty of Engineering. Celeste is married to Braam and they have two children, Joshua (8) and Lisa (6).

RISK-BASED INFRASTRUCTURE DESIGN AND ASSESSMENT

Prof Celeste Viljoen

ABSTRACT

Infrastructure forms the backbone of society and substantial resources are tied up in creating and maintaining these structures. Design, assessment and maintenance planning for infrastructure should ensure adequate structural performance, taking due account of uncertainties. Provisions in standards for structural design and assessment directly impact on the safety, economy and environmental sustainability of infrastructure solutions. This paper explains how probabilistic principles of structural reliability are utilised in standardised design to allow typical structures to achieve close-to-optimal reliabilities. I review our contributions towards improving the models and calibrations on which standardised design depends for South African extreme wind loading, shear design, buckling of cold-formed steel elements and design of liquid-retaining structures. Contributions towards advancing the risk-based design principles that should underpin infrastructure decisions include quantification of acceptable and optimal target reliability classes, recommendations to improve dam rehabilitation decisions in South Africa, risk-based estimation of sample sizes and exploring how provision for assessment of existing structures and climate change could be incorporated into design standards. Research forms an important component of the work, but so does teaching, presentation of continuing professional development courses to industry and community engagement through active participation in the technical committees of the South African Bureau of Standards, the International Organization for Standardization and the international Joint Committee on Structural Safety. Future advances in standardised design need to direct solutions to be significantly more sustainable. This will require a greater focus on using environmentally sustainable materials, extending the useful life of existing structures and exploiting advances in information technology, structural health monitoring, advanced analysis and risk-based methodologies.

Structural engineering is the art of modelling materials we do not wholly understand into shapes we cannot precisely analyse, to withstand forces we cannot properly assess in such a way that the public at large has no reason to suspect the extent of our ignorance.

Dr E.H. Brown (modified and popularised by Dr A.R. Dykes in 1977)

1. INTRODUCTION

Civil infrastructure is the backbone on which most societal and economic activity depends. The value of existing infrastructure is massive, as is the ongoing investment in infrastructure construction and maintenance, in many countries up to 10% of the GDP.

Standards and guidelines govern the design and construction of the built environment worldwide. Design, assessment and maintenance planning for infrastructure should ensure adequate structural performance, taking due account of uncertainties in the estimation of action effects and resistance. These include inherent randomness of loads and material properties, prediction model uncertainty, statistical uncertainty due to lack of data, systemic uncertainty due to change of use or climate, and even the risk of human error or acts of war. Probabilistic methods are ideally suited for this purpose. To the extent that their routine use may be embedded in the practice of structural design and assessment, the corresponding benefit of more optimal solutions may be realised. Therefore, improvements in quantification of input uncertainties and extension of the probabilistic basis of design in national and international standards and guidelines contribute meaningfully to a more sustainable built environment. The long design lifetimes typical of civil infrastructure imply that a greater emphasis on lifecycle costs is warranted, including maintenance and assessment of existing structures to extend their lifetimes, and also including the effects of climate change on structural loading and degradation.

There is room to improve by utilising advanced analysis and probabilistic methods to inform risk-based decision making towards a more sustainable built environment.

2. STANDARDISATION – UNDERPINNING THE BULK OF INFRASTRUCTURE DECISIONS

2.1 Probabilistic structural design

The principles of probabilistic design are captured in detail in [1,2]. In summary, the main principles of probabilistic design are the following:

a) Requirements of structural performance are captured by so-called limit states.
b) Parameters that affect performance are probabilistically described.
c) Target reliability seeks to meet the performance requirements in an economically optimal way, with societal requirements for structural safety as a lower-bound constraint.
d) The focus is typically on component design. Acceptable system behaviour should be separately confirmed (robustness requirement).

In terms of item (a), structures and structural elements must fulfil the following requirements with appropriate levels of reliability over their working life:

- They shall remain fit for use (serviceability limit state).
- They shall withstand extreme and/or frequently repeated combinations of actions occurring during their construction and anticipated use (ultimate limit state).
- They shall not be disproportionately damaged by accidental events (accidental limit state and robustness requirements).

In terms of item (c), the 'appropriate' target levels of reliability should account for the expected consequences of failure and the cost of reducing the risk of failure, where 'failure' may be understood to be failure to meet the limit state under consideration. Rackwitz [3] derived the generic target reliabilities that are widely utilised in standards.

Probabilistic design may be implemented in three levels of approximation: Semi-probabilistic design accounts for the uncertainties of design variables via characteristic values and partial factors, calibrated to approximately achieve target reliabilities. Full probabilistic design directly utilises case-specific probabilistic descriptions of uncertainties and more advanced methods to achieve the target reliability more exactly. Risk-based design adds consideration of the project-specific costs and consequences to determine both the optimal reliability and its corresponding design.

2.2 Standardised semi-probabilistic design for new builds

Semi-probabilistic design is widely deployed in structural standards for routine design. It allows simple procedures to systematically meet requirements in an approximately optimal way. Compared to a fully probabilistic approach, two main simplifications are made:

- Full probabilistic distributions for input variables are replaced by characteristic values (X_k) and partial factors (Y_x). These are calibrated to achieve generic target reliabilities (per limit state) over the class of structures covered by the scope of the standard. This removes the need for designers to have specialist knowledge of statistics and probabilistic methods.
- Load and resistance are separated based on generally conservative sensitivity factors. This allows loading standards to be calibrated separately from resistance standards.

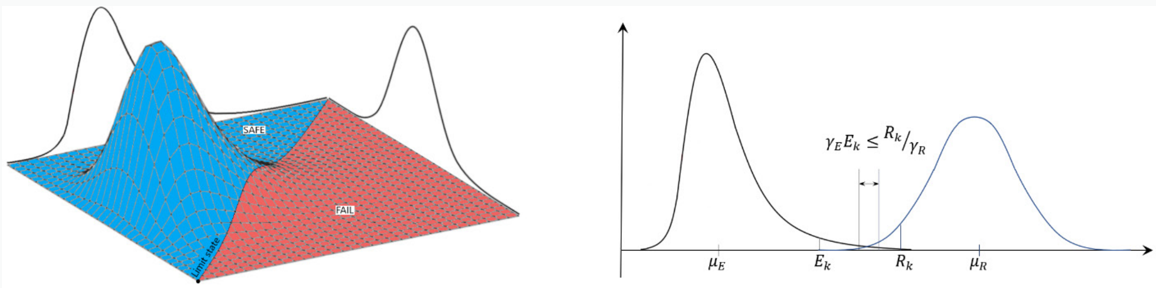

Figure 1(a) **Figure 1(b)**

Figure 1: Depiction of the simplification from full probabilistic to semi-probabilistic design.

Figure 1a depicts the likelihood of possible load-resistance combinations, some of which may imply failure. Figure 1b depicts the simplification of full probabilistic assessment to separate load and resistance provisions.

2.3 Assessment of existing structures

There is a vast global portfolio of existing structures and an increased emphasis on managing societal resources in a sustainable way. Optimal maintenance of existing structures and suitable guidance for their assessment, with a view to extending their useful life, are therefore becoming more important.

Design codes for new build structures are not ideally suited for the assessment of existing structures, because existing structures differ from new builds in some essential ways: The cost of increasing reliability is an order of magnitude higher for the retrofitting of existing structures, which implies a lower optimal (target) reliability based on economic optimisation. The required or remaining service life may differ from what was assumed for new builds. For existing structures, measurement data can be used to update knowledge of material properties and geometry and even site-specific loading. Interventions are constrained by the existing structural system.

Two very recent developments [4,5] significantly advanced the state of standardised provisions for the assessment of existing structures by providing methodologies for quantitative reliability verifications consistent with the structural reliability principles of [1].

Advanced methods, including assessing the value of information [6], designing systems for structural health monitoring and incorporating monitoring data in rehabilitation decisions can unlock future gains.

3 CONTRIBUTIONS TO RISK-BASED INFRASTRUCTURE DECISIONS

3.1 Overview

The focus of my research is on structural reliability and risk-based decision making. Impactful dissemination is achieved through active participation in various national and international standardisation efforts and the organising of training and collaboration events. In this regard I participate(d) in the (in some cases ongoing) revisions of SANS 51990-1-1 [7], SANS 10100-3 [8], SANS 10160-1 [9], SANS 10160-3 [10], ISO 2394 [1] and ISO 13824 [11] as a member of technical committees of the South African Bureau of Standards (SABS) and the International Organization for Standardization (ISO). I organised numerous national continuing professional development courses to disseminate relevant updates to structural design standards, hosted a training event for the international Joint Committee on Structural Safety (JCSS) in 2018, served as co-organiser of the 2021 JCSS workshop on existing structures and serve as a member of the organising committee for the International Probabilistic Workshop to be held in Stellenbosch in 2022.

My contributions, amply mixed with the efforts of postgraduate students, colleagues and collaborators more talented than I, can be categorised along these main themes:

- Sections 3.2 to 3.4: Load and resistance models, including model uncertainty
- Section 3.5: Risk-based decision making
- Section 3.6: Existing structures.

3.2 Loading

Detailed below are substantial contributions towards the development and calibration of models for the prediction of South African extreme wind loads for structural design. This work resulted in improvements to the specification of wind loads for structural design in SANS 10160-1 [9] and SANS 10160-3 [10].

In a separate effort, assessment of traffic loading on bridges [12,13] revealed the need for updating the traffic load models of TMH7 [14]. Colleague Prof. Roman Lenner is taking the lead and our ongoing collaboration is expected to result in meaningful updates to the TMH7 load model. In this regard, the SANRAL-funded research project 7a.1 is in advanced stages of approval and intends to update the TMH7 traffic load model based on probabilistic assessment of weigh-in-motion data of heavy vehicles.

Wind loading extremes and the revision of SANS 10160 (parts 1 and 3)

Wind loading for structural design is specified in SANS 10160-3. Due to the absence of significant snow loading in South Africa, wind loading is the dominant environmental design action for structures in South Africa, with particular significance for long-span roof structures. Wind load depends essentially on three inputs, namely the free-field wind at the location of the structure, the influence of the local terrain on the free-field wind and the interaction between the wind and the structure. The Davenport wind load chain [15] informs provisions in SANS 10160-3 and most design codes internationally. The design wind pressure is hereby determined as the product of various wind load components, treated as statistically independent random variables. Design wind loads for ultimate limit states require appropriate extreme value estimates of low-probability events.

A 2010 review of the SANS 10160 South African loading standard (all eight parts, including Part 3: Wind actions), under the guidance of Prof. Johan Retief and the late Prof. Peter Dunaiski, was ongoing when I started my employ at Stellenbosch University and joined the effort. This review identified the need for better free-field wind data and raised questions regarding the probabilistic modelling of wind pressure and the calibration of its associated partial factors due to the significant differences observed between the SANS 10160-3 model and those used in other international codes (see Figure 2 [16]).

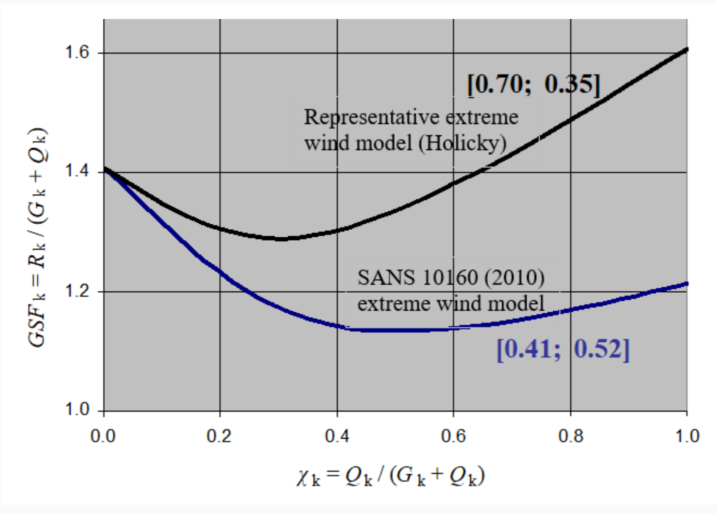

Figure 2: Implied reliability requirement for wind actions [16]

The description of free-field wind in South Africa suffered from a lack of data both in spatial density and in series length. The first probabilistic model of South African extreme wind loads [17] used data from only 14 weather stations to produce a map of characteristic wind speeds (see Figure 3a and Figure 7a), which was used in a slightly adapted form in SANS 10160-3 until its 2018 revision. Important to the SANS 10160-3:2018 update was the inclusion of an updated map of characteristic (2% annual exceedance probability) wind speeds, based on the PhD research of Kruger [18-20], which dramatically improved the spatial resolution of the map by including carefully quality audited data from 76 weather stations. The problem remained that many of these stations only had 10 years' worth of wind data, and none had more than 20 years of data, which implied that significant statistical uncertainty remained in the extreme value extrapolations that underpinned the updated map.

Figure 3(a)

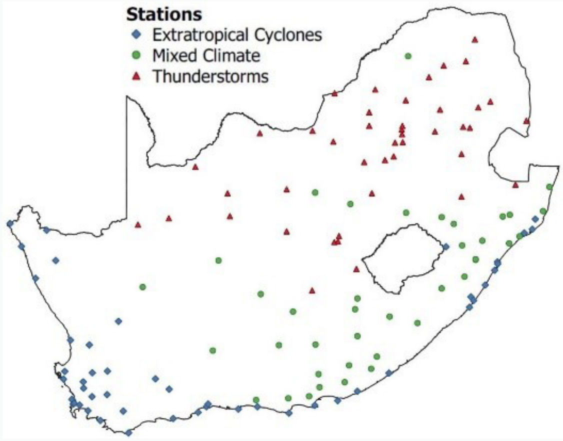

Figure 3(b)

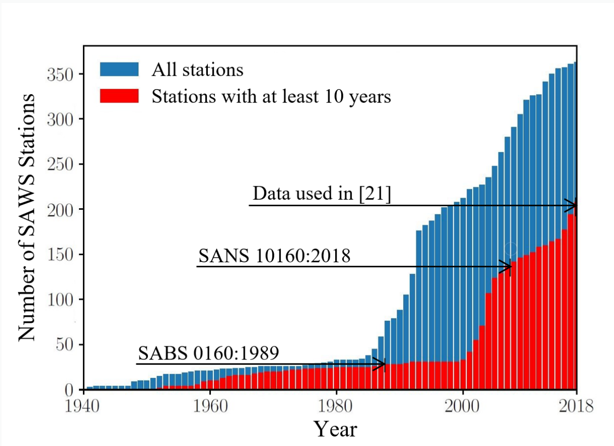

Figure 3(c)

Figure 3: Spatial distribution of weather stations used to develop the (a) 1989 wind map; (b) proposed future wind map [21]; (c) Available wind data in the SAWS database increased dramatically since 1990.

Significant research effort was spent since to update the design wind load provisions: In [22-27] we assessed the wind load model using Bayesian hierarchical updating and recalibrated the partial factor for wind loading to be used in limit states design. Botha [22] identified that the anomaly between the South African model and its international counterparts may be a result of human error in the transfer of statistical parameters of free-field wind from an initial report [17] to a subsequent journal publication [28].

We made a thorough assessment of both bias and uncertainty in all components of the SANS wind load chain through comparison of its predictions to various leading standards and to published experimentally measured values. Bayesian hierarchical updating (Figure 4) was used to propose an updated probabilistic wind pressure model for South Africa, also including the improved free-field wind description of Kruger [19]. Reliability assessment using the updated model identified the free-field wind as the primary driver of reliability, while uncertainty in pressure coefficients and terrain roughness are also important contributors.

We calibrated the partial factors for wind loading in ultimate limit states, considering generalised cases of resistance variability to account for different structural materials, which may range from steel (low variability), concrete and timber to masonry (high variability) (see Figure 5). This work resulted in revised partial factors for wind loading in SANS 10160-1:2018.

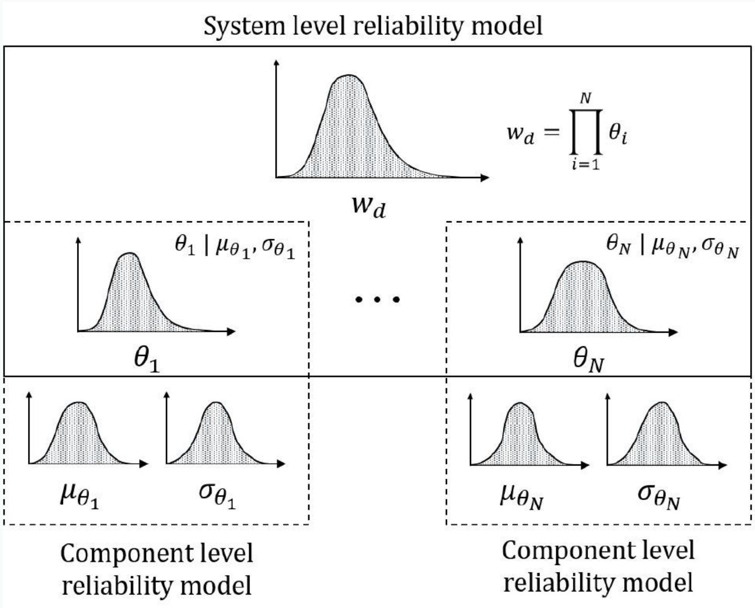

Figure 4: Hierarchical Bayesian updating [25]

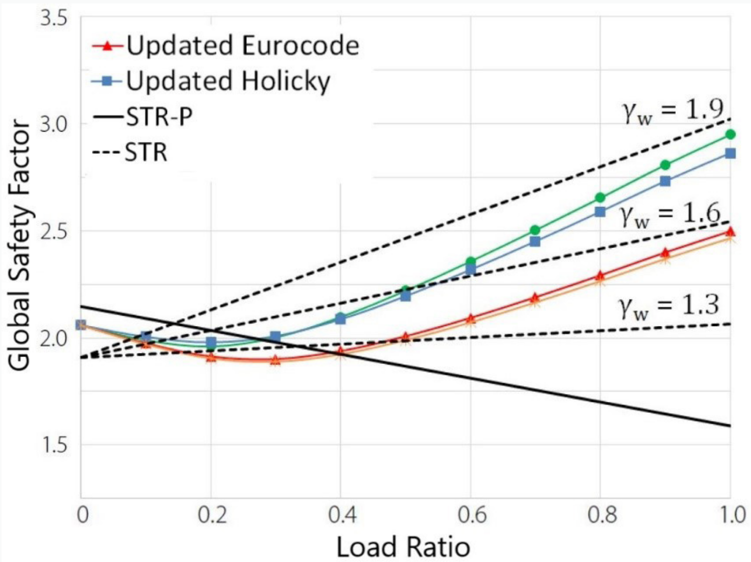

Figure 5: Calibration of partial factor for ULS wind loads [24]

Future updates to the wind map in SANS 10160-3 will result from [21,29,30]. The PhD work of Bakker [21] substantially added to the existing data of Kruger by extracting annual extremes from 2007 to 2018 for 132 stations (see Figure 3b), quality controlling and classifying them by climatic mechanism, correcting for surface roughness and using these to derive probabilistic descriptions of free-field wind per location. The additional 10 years of annual extremes per station were a significant improvement, at many stations doubling the available sample size. Nevertheless, the sample size remains small for extreme-value extrapolation and could introduce substantial model variance.

To address this, we made improved estimates of the shape of the probability distribution (see Figure 6) based on preconditioning the data by an exponent [29] and incorporating data from surrounding sites:

Preconditioning by an exponent of 1 implies that the Gumbel extreme value distribution is fitted to wind speed, while preconditioning by 2 implies that it is fitted to wind pressure. Something in-between minimises prediction error.

Extreme value extrapolation based on site data only is expected to produce unbiased estimates, but with high variance, particularly for small samples, as is the case here. Incorporating data from surrounding sites introduces a bias if the wind climate is not homogeneous, but reduces the variance in the prediction. The optimal balance (that minimise expected extreme value prediction error) between the two approaches could be found for each site to depend on the sample size and the regional sample coefficient of variation. The weighting leans to 'regional' when the sample size is small and the region is relatively homogeneous, but towards 'site' design when the sample size is larger and the region less homogeneous. Interestingly, it could be shown that the current formulation, which specifies design wind pressure as the product of the characteristic (site) wind pressure and a partial factor (regionally calibrated), already constitutes a trade-off between the two approaches.

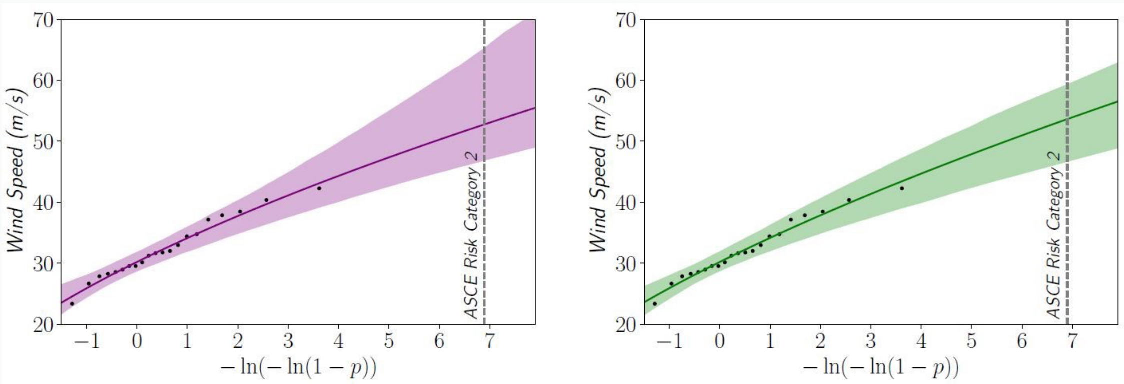

Figure 6(a) **Figure 6(b)**

Figure 6: (a) Single-series preconditioned Gumbel; (b) regionally preconditioned Gumbel [21]

We used these insights, together with the expanded dataset, to estimate design wind speeds [21] that should satisfy the target reliability specified in the South African standard. The corresponding characteristic wind speeds shown in the updated map in Figure 7(c) are generally lower compared to the current map in Figure 7(b). In this way, the reduction in statistical uncertainty achieved through these advances will result in real savings on wind-dominated designs, without compromising reliability.

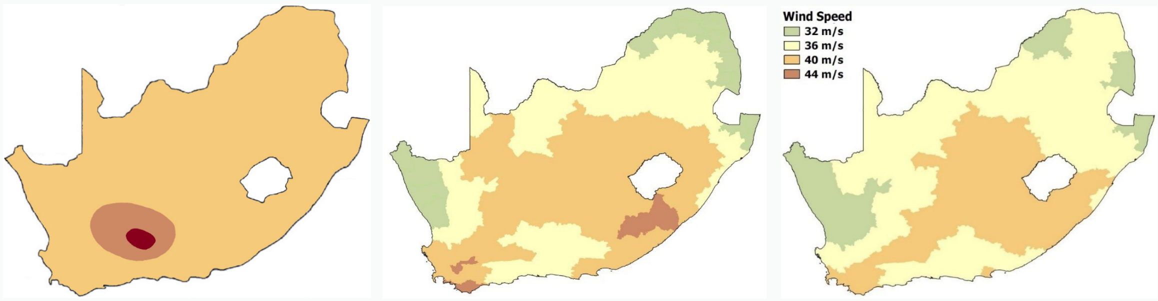

Figure 7(a) **Figure 7(b)** **Figure 7(c)**

Figure 7: Characteristic gust wind speed maps for South Africa: (a) 1989 wind map; (b) current 2018 wind map; (c) proposed future wind map

Some wind load components in the Davenport wind chain are time-dependent, such as the free-field wind at the location of the structure, while others that relate to fixed physical conditions such as terrain and structural geometry are time-independent. When design loads are determined as a function of the service life of the structure, time-independent components should not be scaled together with time-dependent components. Where this was done for simplicity's sake, hidden safety is achieved, which may not be economically optimal. We attempted to quantify this hidden safety in [31].

3.3 Resistance

My involvement in various SABS technical committees, including as convenor of the working group for the development of a national standard for the design of reinforced concrete liquid-retaining structures, has provided several opportunities to serve national interest. Assessment of reliability of newly adopted design procedures confirmed adequate performance in the case of shear design, but exposed room for improvement in the case of cold-formed steel element design. At the same time, research initiatives contributed to the international pool of knowledge that informs reliability-based design. Quantification of model uncertainty for crack width prediction models and for leakage in the presence of autogenous self-sealing provides a basis for calibration of semi-probabilistic serviceability limit state (SLS) design of liquid-retaining structures. Exploratory work advanced the application of reliability principles towards improved design of composite floors in severe fire and for vibrations.

Reliability of the VSIM shear design provisions of EN 1992-1-1 adopted in SANS 51992-1-1

SABS TC98/02 recommended that the European standard EN 1992-1-1 (EC2) be adopted as the local standard for structural concrete design to replace SANS 10100-1, The structural use of concrete [32]. Arguably the most significant implied change to current practice of this adoption is the adoption of Variable Strut Inclination Method (VSIM) provisions for shear design. Cladera and Mari [33] were concerned that the VSIM prediction model is systematically sensitive to the amount of stirrups provided in design and overestimates shear capacity at high stirrup quantities (see Figure 11a). Consequently, we set out to do an assessment of reliability of the EC2 shear provisions across the range of application.

Initial investigations [34] revealed the importance of quantifying model uncertainty well and of using this as a basis for choosing an appropriate general probabilistic model (GPM) that facilitates unbiased prediction of shear capacity across the range of application. It is also the parameter that most influences the assessed reliability.

The EC2 shear design formulation was assessed [35-38] to have high reliability for low levels of shear reinforcement, high concrete strength and large beam depth and lower reliability (but still adequate) with increased levels of shear reinforcement, reduced concrete strength and beam depth for both rectangular (see Figure 8a) and I-beam cross sections. The very high EC2 reliability obtained at low levels of shear reinforcement can be ascribed to the neglect of concrete contribution to VSIM shear strength predictions, which is more significant in lightly shear reinforced beams and to the limit imposed by EC2 VSIM on the concrete strut angle. Reliabilities of other standardised formulations where also assessed (see Figure 8b).

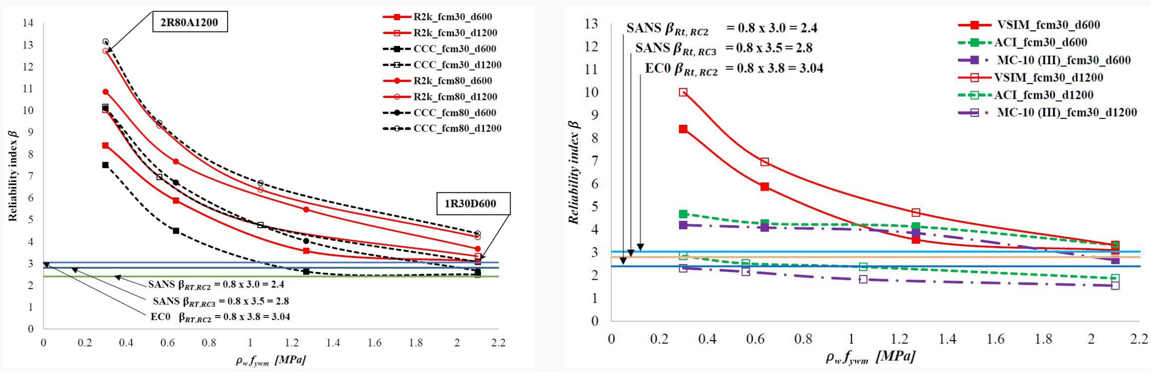

Figure 8(a) **Figure 8(b)**

Figure 8: Reliability performance of (a) various VSIM designs, compared to (b) provisions from other standards

Design considerations for concrete liquid-retaining structures: Prediction of load-induced crack widths and autogenous self-sealing and its influence on the reliability of serviceability limit state design

Load-induced cracking of reinforced concrete is an important consideration in the design of liquid-retaining structures. Serviceability limit state design requirements that seek to limit leakage by means of limiting the allowed crack widths routinely govern the design of these structures. The ongoing development [39-42] of SANS 10100-3: Design of concrete liquid-retaining structures [8], currently at SABS TC draft stage, provides a unique opportunity to improve on the reliability basis of these designs.

In [43-47] we quantified model uncertainty for the crack width prediction models of EN 1992 [48], MC 2010 [5] and BS 8007 [49] for both flexural and tension cracking under short- and long-term loading. Experimental data on load-induced cracking from literature were assembled in a database. Subsequently, quantified model uncertainties for the selected crack models confirmed their dominant influence on reliability and established the MC 2010 model as the most suitable GPM based on low bias and consistent performance. All models displayed high variances. While short-term cracking behaviour could be well quantified, the lack of data on long-term performance hampered its assessment.

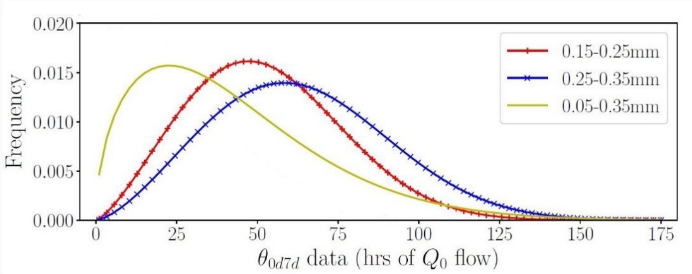

Figure 9(a)

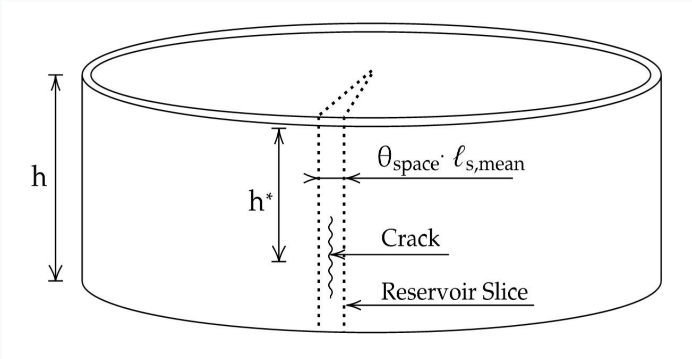

Figure 9(b)

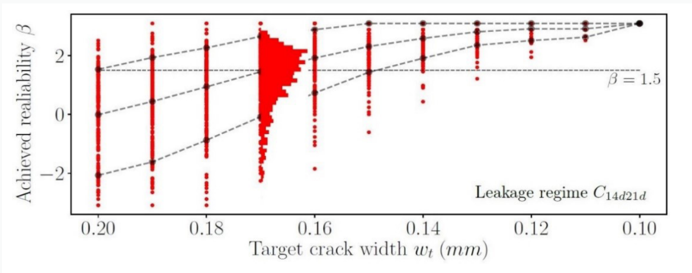

Figure 9(c)

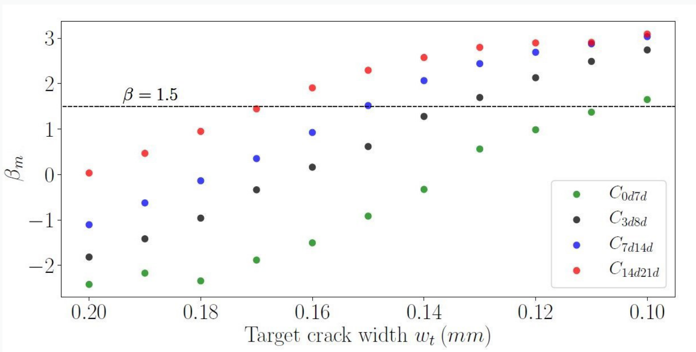

Figure 9(d)

Figure 9: (a) Expected leakage probabilistically quantified for different crack width ranges; (b) reservoir leakage estimation model; (c) reliability of different designs as a function of crack width for long stabilisation regime; (d) mean reliabilities of practical designs as a function of crack widths and test regimes (adapted from [50])

In [50,51] we probabilistically quantified the leakage that may be expected through tension-induced through-cracks considering the beneficial effect of autogenous self-healing of concrete. Initial flow is significantly uncertain and depends on the crack width. Thereafter, autogenous self-healing will reduce the crack width and associated flow, again with uncertainties and dependent on the crack width. A database of leakage measurements from literature was assembled and utilised to account for the uncertainty in predicted initial flow and in autogenous self-healing. This allowed us to probabilistically quantify the expected leakage (expressed in terms of initial flow, see Figure 9a) as a function of crack width. Increases in crack widths have a compound effect on leakage by notably increasing initial leakage, decreasing the probability of sealing and increasing the time taken to seal. As the bulk of autogenous self-healing occurs within the first few days, the beneficial effect of a stabilisation period prior to water tightness testing is quantified.

The SLS reliability of a tension-governed reinforced concrete (RC) reservoir was assessed in [50], considering the above likelihood of self-sealing and uncertainty in predicted crack widths and spacings (see Figure 9b). We showed that the achieved SLS reliability depends substantially on the target crack width and watertightness test times. Extending this assessment to a range of reservoirs that covers the scope of practice confirmed that smaller target crack widths and longer water tightness test times increase achieved reliability (see Figure 9d). However, even for a given target crack width and leakage regime, a substantial range of achieved reliabilities is observed depending on reservoir configuration (see Figure 9c). Therefore, current design provisions do not effectively achieve target reliability across the range of practical application. Reservoir-specific crack width targets are proposed to achieve target reliability more consistently.

Structural reliability of thin-walled cold-formed steel elements

The use of thin-walled cold-formed steel as a structural material has gained traction in South Africa with the adaption of AS/NZS 4600 as SANS 10162-2 [32] in 2011.

However, we found in [52-54] that careful consideration should be paid to the calibration of partial reduction (safety) factors for the complex modes of buckling, depicted in Figure 10, that characterise the typical failure modes of these elements. Due to the lightweight nature of these structures, they are sensitive to wind loading, and ongoing updates to the wind load models should be incorporated in future calibration [53,55]. The load-resistance sensitivity factors for these elements can be significantly different from the norm [31] and had to be properly characterised as part of reliability assessments that consider both load and resistance. Indications are that the current provisions do not deliver adequate reliability for the global buckling failure mode, nor for local-global buckling, although adequate reliability is achieved for distortional buckling and for local buckling modes [54]. This is due to high prediction model uncertainty for global buckling and is of particular concern for longer unbraced compression elements.

The self-tapping screwed connections often used in these structures were found to typically fail in tilt-and-bearing, for which the reliability seems sufficient [56,57].

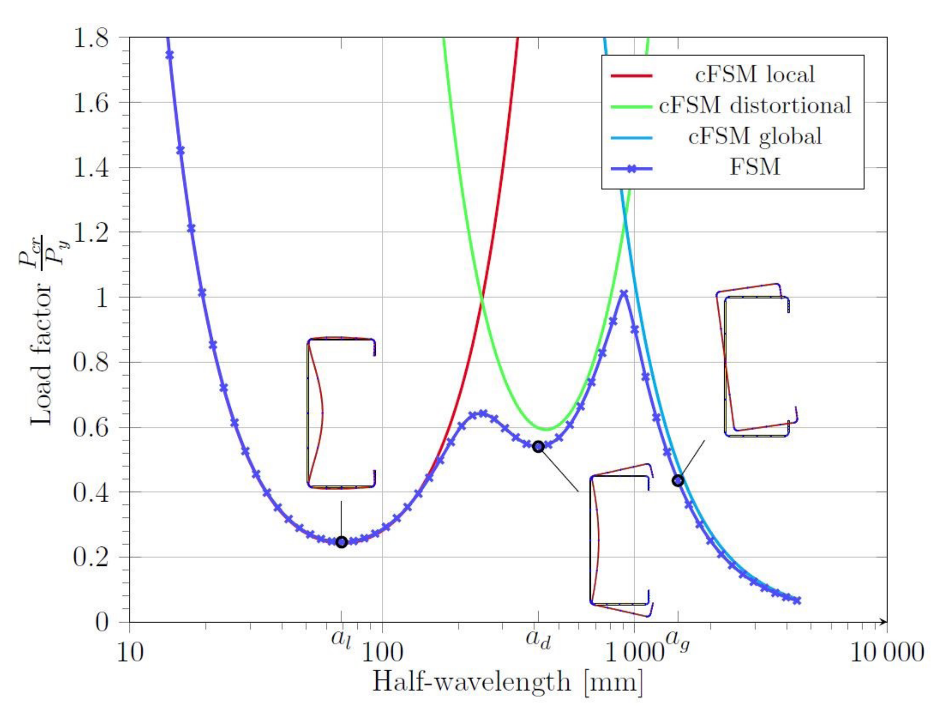

Figure 10: Typical signature curve showing three buckling modes

Composite suspended slabs: Accidental fire action and vibration serviceability

In [58] we assessed the reliability of composite floor slabs in severe fires based on probabilistic quantification of fire loads and corresponding capacity according to the slab-panel method. We showed that reliability depends strongly on fire load energy density, even within a given fire hazard category. There remains a need for characterisation of the model uncertainty of the slab-panel method, which would require experimental testing.

Acceptable vibration performance is an important limit state to consider for light composite floors. In [59] we showed that reliability-based design that accounts for the uncertainty in what is deemed acceptable vibration, uncertainty in vibration prediction models and the probabilistic nature of step frequencies can produce more rational designs than deterministic approaches.

3.4 Model uncertainty

Prediction model uncertainty is often an important contributor to reliability performance, as demonstrated in sections 3.2 and 3.3. Where this is the case [60], model uncertainties should ideally be characterised against a database of tests representative of the phenomenon under investigation. Such characterisation should extend parametrically over the relevant range of design situations to pick up underlying sensitivities that could be remedied during calibration. By way of example, Figure 11a shows the ratio of experimental shear capacity to VSIM shear predictions over a practical range of designs. VSIM's tendency to overestimate shear capacity at high levels of stirrup reinforcement makes it a poor GPM for shear capacity prediction. Figure 11b compares this model to alternative GPM candidates, of which R2k provides the best performance due to its consistency over the practical design range (not shown) and its relatively low variability.

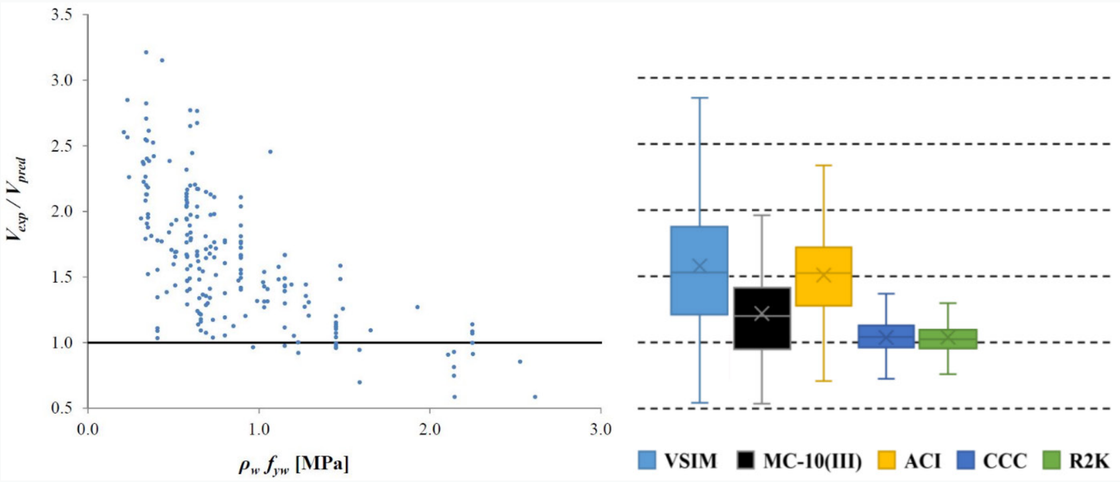

Figure 11(a) **Figure 11(b)**

Figure 11: (a) VSIM prediction model factors, compared to (b) other shear prediction models

3.5 Risk-based decision making

Contributions to the framework for risk-based decisions are summarised below, including considerations for provisions for climate change. The work detailed in [61,62] was the basis for the acceptable reliabilities specified in Annex G of ISO 2394, while contributions in [63,64] may find uptake in future revisions of SANS 10160-5 [65] and international equivalents.

Optimal and acceptable reliabilities for structural design

Target reliabilities for reliability-based design (see Table 1) have been derived based on generic economic optimisation [3]. While the consequence classes allow for risk to human life to be captured qualitatively, these targets do not explicitly account for the societal acceptability of structural designs with respect to life safety risks. Economically optimal designs may be implemented provided that their implied life safety is also acceptable (see Figure 12a). Acceptable reliabilities may be derived based on societal preferences for investment into life safety. Societal willingness to pay (SWTP) to save a marginal life may be quantified based on economic principles [66] that seek to direct limited available societal life safety investments into efficient risk reduction measures.

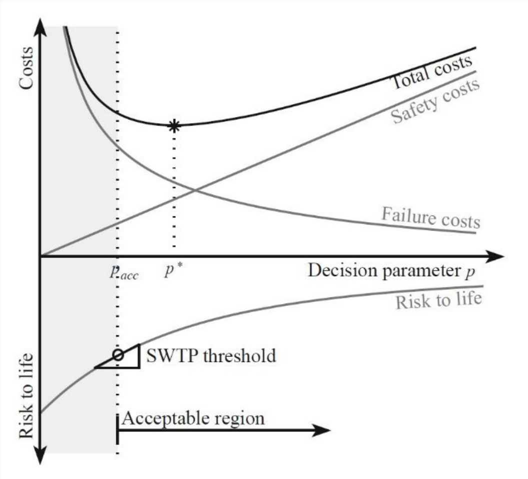

Figure 12(a)

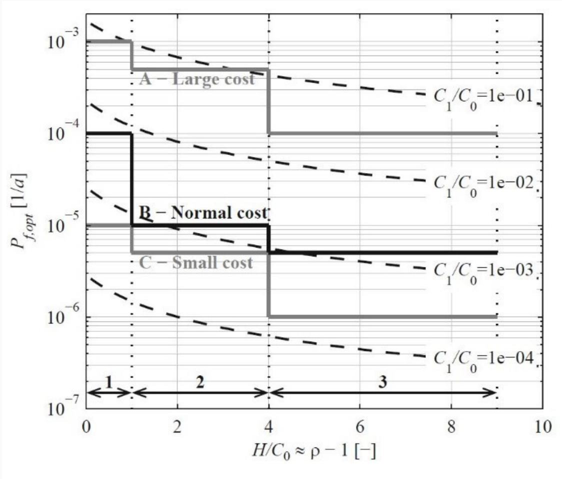

Figure 12(b)

Figure 12: (a) Monetary optimisation with societal acceptance criterion for investment into life safety; (b) comparison of optimal failure probabilities (dashed lines) to JCSS targets (solid lines) [62]

In [62] we extended the renewal framework used to derive optimal reliabilities [3] to also include life safety considerations. We quantified the qualitative descriptions for relative cost of safety and consequences of failure used in the JCSS target classification scheme (see Figure 12b) in terms of the ratios C_1/C_0 and H/C_0, where C_1 is the marginal costs associated with a change in the central safety factor, C_0 is the part of the construction costs that is independent of structural design and H is the failure costs excluding the reconstruction costs.

We derived acceptable reliabilities (see Table 2) that were the basis for the acceptable reliabilities specified in Annex G of ISO 2394 [1]. These depend on the relative marginal lifesaving costs K_1, a function of SWTP, the number of lives at risk and C_1. We showed that optimal reliability will also be acceptable if SWTP is used, in economic optimisation, as the compensation cost for lives lost.

Table 1: Target reliabilities and corresponding failure probabilities for a one year reference period and ultimate limit states [1]

Relative cost of safety measure	Consequences of failure		
	Minor	Moderate	Large
Large (A)	$\beta = 3.1$ ($P_f \approx 10^{-3}$)	$\beta = 3.3$ ($P_f \approx 5 \cdot 10^{-4}$)	$\beta = 3.7$ ($P_f \approx 10^{-4}$)
Normal (B)	$\beta = 3.7$ ($P_f \approx 10^{-4}$)	$\beta = 4.2$ ($P_f \approx 10^{-5}$)	$\beta = 4.4$ ($P_f \approx 5 \cdot 10^{-6}$)
Small (C)	$\beta = 4.2$ ($P_f \approx 10^{-5}$)	$\beta = 4.4$ ($P_f \approx 5 \cdot 10^{-6}$)	$\beta = 4.7$ ($P_f \approx 10^{-6}$)

Table 2: Minimum acceptable reliabilities and corresponding failure probabilities for a one year reference period and ultimate limit states [1,62]

Relative marginal lifesaving costs	Minimum acceptable reliability
Large (I)	$\beta = 3.1$ ($P_f \approx 10^{-3}$)
Normal (II)	$\beta = 3.7$ ($P_f \approx 10^{-4}$)
Small (III)	$\beta = 4.2$ ($P_f \approx 10^{-5}$)

Efficiency of the decision parameters utilised to adjust reliability

In the generic optimisation approach used by Rackwitz [3] to derive target reliability values, the central safety factor was used as generic decision parameter. In [67-69] we derived the economically optimal target reliability for a liquid-retaining structure in the serviceability limit state. We found that the case-specific decision parameter (in this case, area of reinforcement) is more efficient in adjusting the achieved reliability in the considered limit state (in this case, achieving acceptable crack widths) than its generic counterpart. Accordingly, the optimal reliability is significantly higher than what would have been assumed based on the generic guidance (Table 1). This has wider implications for the extent to which current guidance achieves economically optimal performance for other structures and limit states.

In [70,71] we explored different alternative decision parameters for the main failure modes of reinforced concrete elements. Some parameters were found to be meaningfully more efficient than others in influencing the achieved reliability for the same failure mode.

Risk-based sample sizes

Routine reliance on small sample sizes in the estimation of site-specific material strength for geotechnical design can lead to significant over- or underdesign. In [64] we developed a relation between the optimal sample size, the target reliability index and the liability ratio (expected damages in case of failure over the unit cost of testing) (see Figure 13) by accounting for the fact that the predictive reliability also depends on the uncertainty in parameter estimation. The optimal number of tests can therefore be determined in the trade-off between the cost of testing and the reduction in the risk of failure. The theory can be used to encourage better sampling practices, either through determining an optimal sample size or by using sample size-dependent partial factors. Smaller partial factors allowed by larger sample sizes would encourage the adoption of appropriate sample sizes for more economical designs than those currently realised.

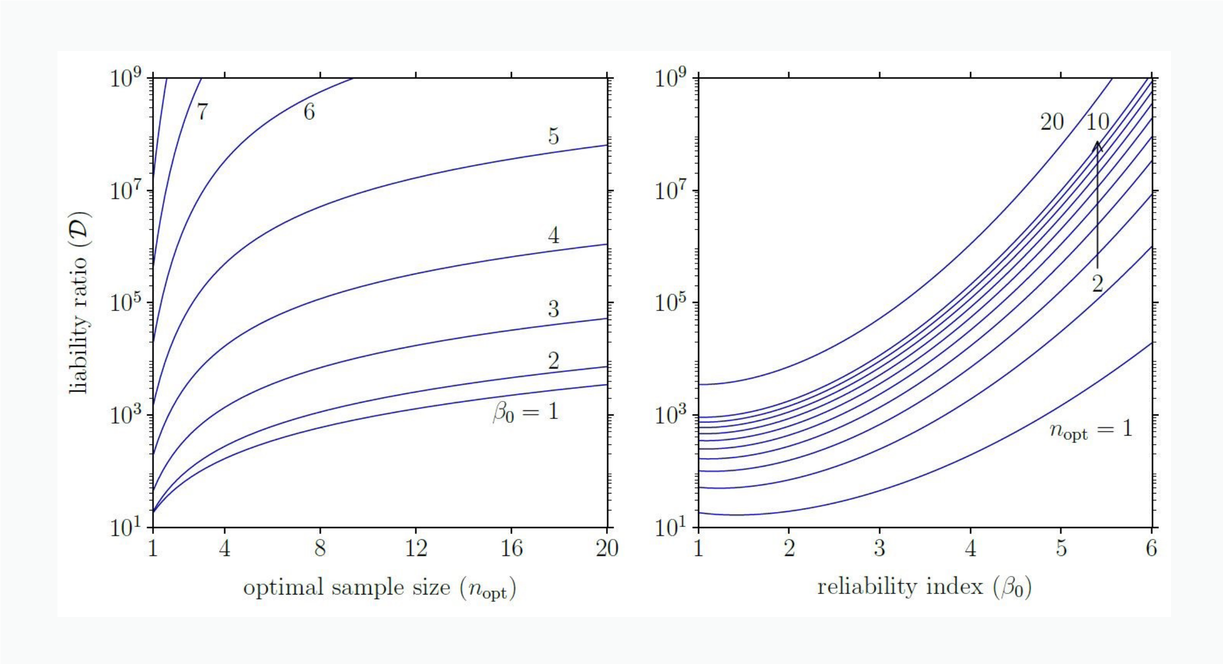

Figure 13: Charts for determining optimal sample size [64]

3.6 Existing structures

Contributions that concern the assessment of existing structures within a risk-based framework are summarised below. Future work will direct attention to provisions for the assessment of existing bridges, as envisioned in SANRAL-funded research project 7a.9.

Risk-based decisions in dam rehabilitation

The Department of Water and Sanitation (DWS) reports that many high-risk category dams need rehabilitation [72]. Limited resources should be allocated in the most efficient way possible.

In [73-75] we evaluated the risk-based methodology that the DWS used to make rehabilitation decisions. For a specific dam, the DWS evaluates risks against multiple acceptability criteria to assess the risk to human life and the economic, social, socio-economic and environmental impacts of dam failure. We found that the DWS overpredicts the loss of life for low warning times and underpredicts for long warning times, significantly so when shallow and slow flood conditions are expected, and that the DWS criteria incorrectly use the population at risk as a proxy for life loss. Replacing the DWS life loss prediction model with that of [76] and the DWS life risk diagram with the ANCOLD FN criteria [77] would go some way to improve decisions.

In [73,75,78,79] we evaluated the need for 11 actual dam rehabilitations done by the DWS based on their decision input values, which were kindly supplied. We found that only five of the 11 dams would be upgraded on pure economical optimisation considerations, and only one of the 11 upgrades was required based on SWTP. Figure 14 summarises the risk profiles of the projects.

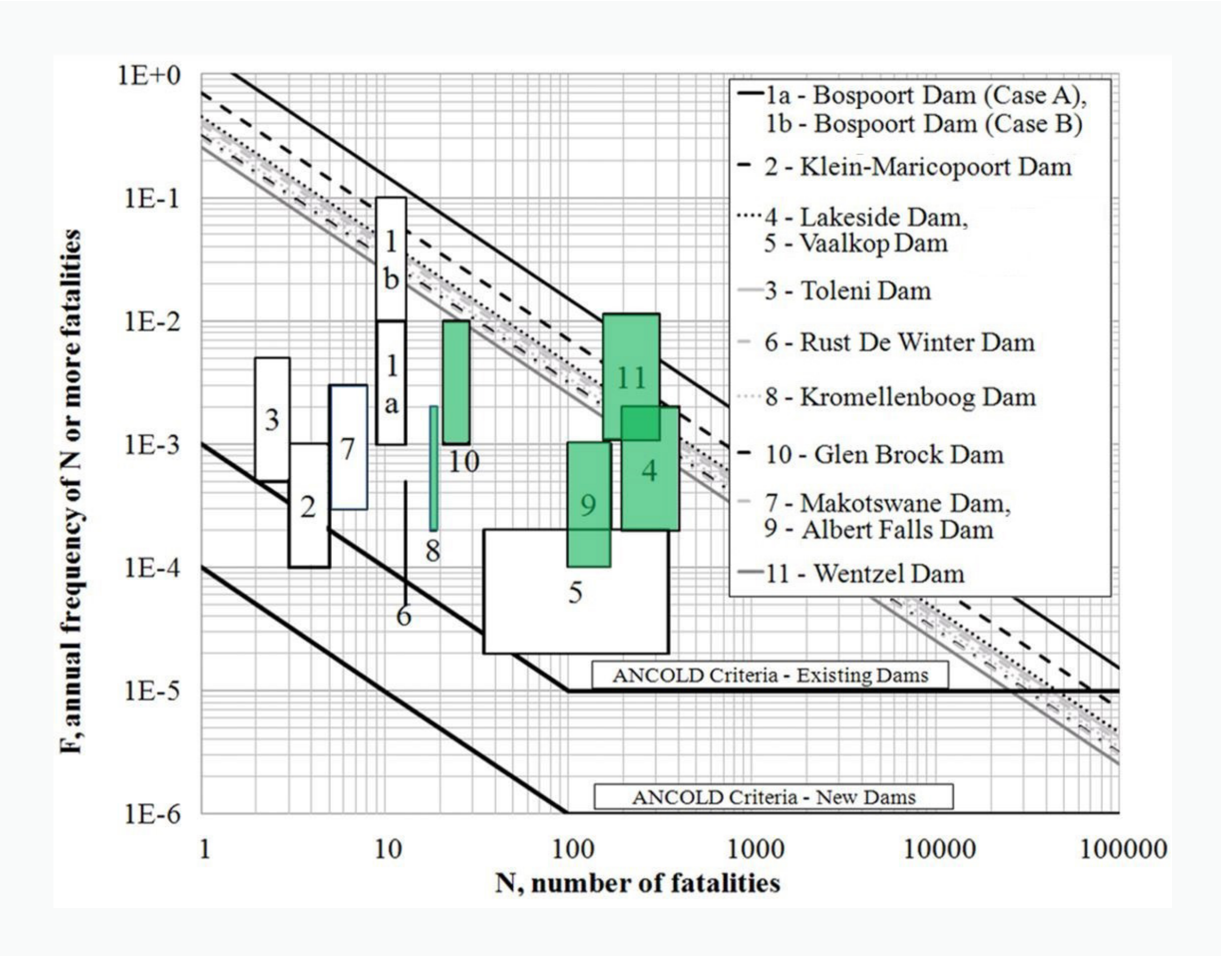

Figure 14: F-N risk profiles of dam rehabilitation projects (blocks) compared to project-specific SWTP requirements (dashed lines) and ANCOLD rehabilitation criteria; highlighted projects are economically optimal to rehabilitate (adapted from [75])

In [80,81] we propose a method to allocate available budget over a portfolio of dam rehabilitation projects in a way that would maximise the expected lives saved. Technology curves that display the efficiency of rehabilitation investments in terms of marginal lives saved allow for the identification both of dams where the greatest life savings may be achieved and of the extent of rehabilitation per dam that would be optimal (see Figure 15).

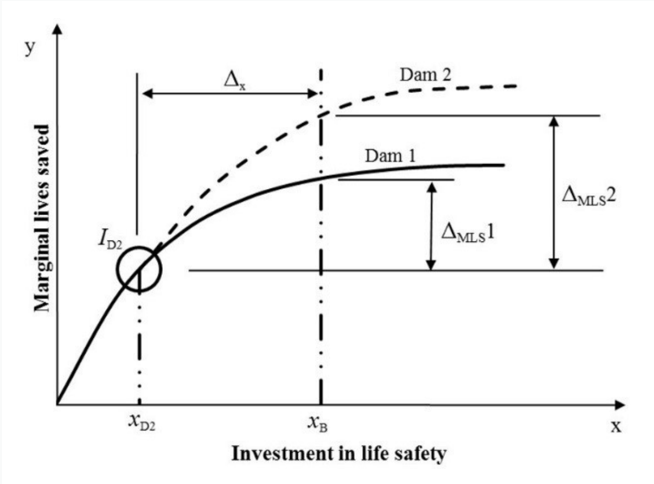

Figure 15: Technology curves as a tool to allocate budgets to maximise expected lives saved [81]

Adjusted partial factors for existing structures

In general, lower target levels of reliability may be accepted for the assessment of existing structures on the basis that the cost of increasing safety through rehabilitation is an order of magnitude higher than for new builds for similar consequences of failure. In [82] we identified suitable target reliability values for design, assessment of existing structures and societal requirements, respectively, that align with the principles of [2, 62,83]. Subsequently, on the assumption that underlying load and resistance models used in calibration of design provisions remain applicable for assessment, we derived partial factors for assessment (see Table 3 and Figure 16). These may be used to formulate equivalent standardised assessment rules for a substantial class of existing structures and conditions encountered in engineering practice, and to identify the need for more refined assessment where applicable.

Table 3: Partial factors for design (D), assessment (A) and societal requirement (Ref) [82]

		RC2			RC3	
X	V_x	D	A	Ref	D	A
G	0.095	1.20	1.13	1.11	1.23	1.17
Q	0.22	1.60	1.33	1.24	1.76	1.48
R	0.10	1.27	1.17	1.14	1.32	1.23
	0.15	1.43	1.27	1.21	1.52	1.37
	0.20	1.62	1.38	1.29	1.75	1.57

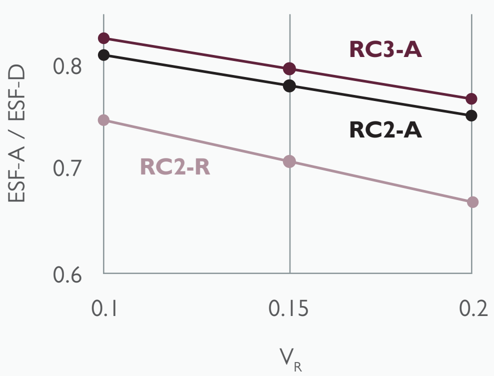

Figure 16: Ratio of effective safety factor for assessment (ESF-A) and societal requirement (ESF-R) to design (ESF-D) [82]

Recently, [4,84] significantly advanced the state of standardised provisions for the assessment of existing structures by providing methodologies for quantitative reliability verifications consistent with the structural reliability principles of ISO 2394 [1]. In [85] we reviewed these advancements and outline of a possible development path for implementation in South Africa.

Measurement-based updating of reliability for existing structures

Observed extreme load events on existing structures (from proof loading or during service) can provide information on their possible resistance capacity. Updated structural reliability based on such observations would allow better monitoring and maintenance planning.

Colleague and first author Prof. Nico de Koker exploited this in [86]. The genius of the developed formulation lies in its computational efficiency (which is substantially better than alternatives) and its general applicability. It adapts the popular FORM methodology to incorporate proof load type measurements to update the reliability of existing structures. Its use is illustrated in Figure 17, where measurements of the water table level are used to update reliability of a granular embankment forming a seawall in a mining operation.

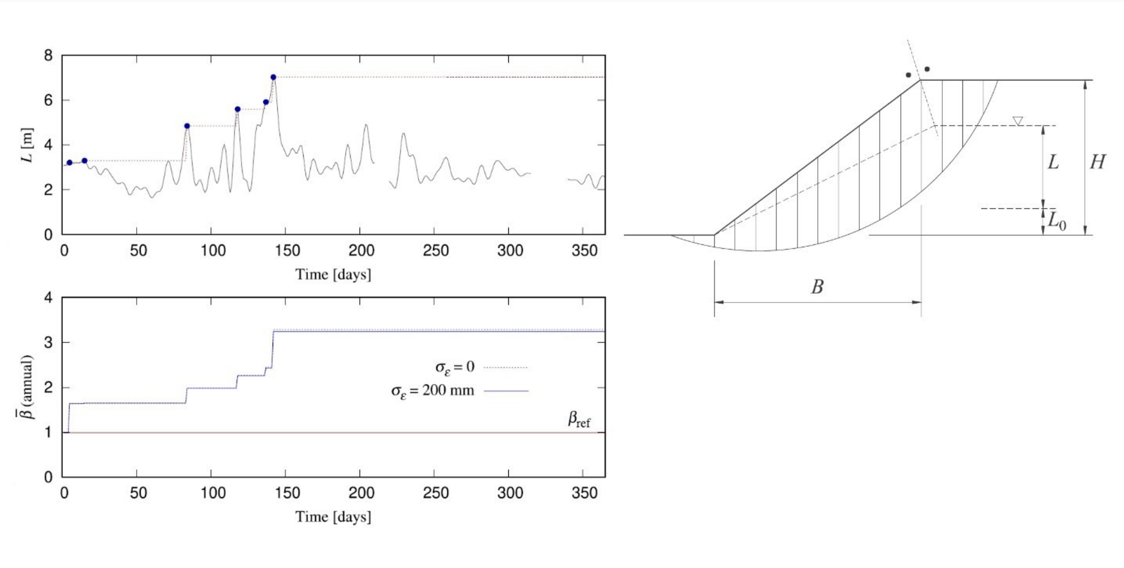

Figure 17: Updating reliability based on measurements of the water table level in a seawall (adapted from [86])

Climate change

Due to the long service lives of infrastructure, consideration should be given to provisions for climate change in design standards. Evidence of climate change effects, which can be observed from conditions in Germany and South Africa [87], may have a direct bearing on economic infrastructure performance and design. Risk-based approaches in structural design provide a proper basis for considering the effects of climate change [88].

In [89] we explored how climate change may be incorporated as an accidental design situation in the partial factor limit states format to allow differentiated action to be taken by the designer in operational decision making.

4. CONCLUSION

Infrastructure forms the backbone of society and substantial resources are tied up in creating and maintaining these structures. A range of considerations and uncertainties needs to be accounted for in the quest for optimal design and maintenance. Provisions in standards for structural design and assessment directly impact on the safety, economy and environmental sustainability of infrastructure solutions. Principles of structural reliability and optimisation are utilised in standardised design, but room for improvement remains.

My contributions centre around improving the models on which standardised design depends and advancing the risk-based design principles that (should) underpin infrastructure decisions. Research forms an important component of the work, but so does teaching, including the presentation of continuing professional development courses to industry and community engagement through active participation in the technical committees of the SABS, ISO and JCSS.

Future advances in standardised design need to direct solutions to be significantly more sustainable. This will require a greater focus on using environmentally sustainable materials, extending the useful life of existing structures and exploiting advances in information technology, structural heath monitoring, advanced analysis and risk-based methodologies.

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