Contributions to the Implementation of the Principles of Reliability to the Standardized Basis of Structural Design

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Declaration

By submitting this thesis electronically, I declare that the entirety of the work contained therein is my own original work, that I am the authorship owner thereof (unless to the extent explicitly otherwise stated) and that I have not previously in its entirety or in part submitted it for obtaining any qualification.

Signature: JV Retief

Date:

Abstract

Implementation of the principles of structural reliability widely impacted on recent improvements in structural performance. Improvements in the rational basis for the design of structures in turn have a bearing on the ability of structural engineers to contribute to the safety, functionality and economy of structures to accommodate the activities and infrastructure serving society. This dissertation presents a number of investigations that can broadly be classified to explore and advance the implementation of reliability concepts and procedures in standardized structural design. The context of the investigations is provided by various activities on the development of revised or new South African standards for structural design, utilising international standards as reference base. The principles of reliability provide the common basis for the harmonization of national and international standards, unification between various standards which are common to specific structures and reliability assessment for classes of design variables and performance functions.

Specific investigations considered the general basis of structural design required for a suite of standards; the reliability modelling of actions and their combinations, including wind loading, imposed roof loads and crane induced loads; structural resistance, including structural concrete shear and cracking performance, the reliability performance of pile foundations. Generalisation of the detailed investigations consists of the identification of the attributes of structural design standards that could serve as the basis for meta-standard drivers for standards development and their management. A common theme in many investigations is the consideration of model uncertainty and the need for its proper quantification for use in reliability assessment. Accordingly generalisation consists of the compilation of a classification scheme for classes of model uncertainty and systematic procedures for the investigation and implementation of model uncertainty. The scheme can also be used to provide a basis for planning of research activities on which model development can be based.

The investigations confirm the potential for value to be added at the interface between reliability theory and design practice, despite the maturity of the field. Examples presented in the dissertation include detailed investigations on design variables such as South African strong wind characteristics and wind load reliability models, extensive investigations on concrete shear resistance models and their uncertainties, and pile foundation reliability calibration. General investigations on the reliability basis of design contributed to demonstrate the achievement of harmonisation between the new South African Loading Code SANS 10160:2010 and Eurocode to the extent that the South African standard serves as an example of the application of Eurocode beyond Europe. A common basis also serves to unify national standards for the design of various structural materials ranging from steel to geotechnical materials, having widely diverse origins ranging from adoption to local development.

Finally the investigations reveal both remaining topics begging further investigation and a methodology for prioritisation and integrating the outcomes into the general reliability framework.

Opsomming

Die implementering van die beginsels van struktuurbetroubaarheid het oor ŉ wye front tot onlangse verbeterings in die verrigting van strukture bygedra. Die verhoogde rasionele basis vir die ontwerp van strukture stel struktuur-ingenieurs in staat om beduidende bydraes te maak tot die veiligheid, funksionaliteit en kostes van strukture wat bydra om gemeenskaps-aktiwiteite en infrastruktuur te huisves. ŉ Aantal ondersoeke word in hierdie verhandeling aangebied, wat breedweg geklassifiseer kan word as ŉ verkenning en bevordering van die implementering van konsepte en prosedures van betroubaarheid in gestandaardiseerde struktuur-ontwerp. Die konteks waarin die ondersoeke uitgevoer is, is die ontwikkeling van nuwe of hersiene Suid-Afrikaanse standaarde vir struktuur-ontwerp, waartydens internasionale standaarde as verwysingbasis aangewend word. Die beginsels van betroubaarheid dien as gemeenskaplike basis vir die harmonisering van nasionale en internasionale standaarde, die unifikasie tussen die onderskeie standaarde wat gebruik word by die ontwerp van spesifieke strukture, sowel as die assessering van die betroubaarheid van klasse van ontwerp-veranderlikes en verrigtingsfunksies.

Benewens ondersoeke na die algemene betroubaarheidsbasis vir struktuur-ontwerp soos van toepassing op ŉ stel van standaarde; is ondersoek uitgevoer op aksies wat inwerk op strukture en hul kombinasies, insluitende windbelasting, opgelegde dakbelastings en belastings wat deur oorhoofse hyskraan-installasies geïnduseer word; struktuur-weerstand, insluitend dié van beton teen skuifkragte en kraak-vorming; die betroubaarheidsverrigting van heipaal fondamente. Veralgemening van die ondersoeke behels die identifikasie van die attribute van standaarde vir struktuur-ontwerp wat dien as meta-standaard aandrywing vir die ontwikkeling van standaarde en bestuur van die proses. Die noodsaak daarvan om voldoende voorsiening te maak vir die bydrae van model-onsekerheid in die assessering van betroubaarheid is ŉ gemeenskaplike tema tot vele van die ondersoeke. ŉ Veralgemeende skema vir die hantering van model-onsekerheid is op grond van hierdie ondersoeke opgestel, waardeur hierdie klas van ondersoek sistematies beplan en uitgevoer behoort te word, self ook vir model-ontwikkeling.

Die ondersoeke bevestig die potensiaal vir verdere toevoeging van waarde deur ondersoeke oor die tussenvlak tussen betroubaarheidsteorie en ontwerp-praktyk, ten spyte van vordering wat reeds gemaak is. Voorbeelde van spesifieke detail ondersoeke sluit die gemelde karakterisering van wind-belasting, omvattende ondersoeke na modelle vir beton skuifweerstand en heipaal betroubaarheids-kalibrasie in. Algemene ondersoeke sluit in die demonstrasie van die harmonisering van die nuwe Suid-Afrikaanse Las-kode SANS 10160:2010 en Eurocode, tot so ŉ mate dat die nasionale standaard beskou kan word as die implementering van Eurocode buite Europa. ŉ Bydrae word ook gelewer tot die unifikasie van die diverse nasionale struktuur-standaarde.

In die finale instansie lê die verhandeling ŉ aantal onderwerpe bloot wat met vrug verder ondersoek kan word, asook ŉ metodologie vir die prioritisering en integrasie van sodanige ondersoeke in ŉ oorhoofse raamwerk vir struktuurbetroubaarheid.

Acknowledgements

The encouragement of Professor Gideon van Zijl to me to be able to put together the output of continued academic activities into this dissertation was critical to bringing this task to fruition. Furthermore, his continued support in giving me the opportunity to maintain involvement in the Department of Civil Engineering long past my dated shelve life has a direct bearing on the contents of the dissertation. Similar support by the late Professor Peter Dunaiski laid the foundation for a special opportunity well beyond my expectations.

In addition to prompting the compilation of the dissertation, Professor van Zijl also served together with Dr Celeste Viljoen as supervisors to this ‘new student’. In addition to the much appreciated guidance received from them, it was also a privilege to work with Dr Viljoen in the field of our common interest and to experience her enthusiasm and abilities.

Amongst the number of colleagues with whom I had the opportunity to engage, Professor Milan Holický played a special role as mentor, sharing extensive activities, exchanges, discussions and friendship which had a significant impact on my insight and appreciation of the field of structural reliability.

The extensive list of co-authors included in the material presented in this dissertation is indicative of the many colleagues and former students who have become colleagues subsequently, who share with me the joy of the investigations and the understanding that has emerged.

Dedications

Dedication of this dissertation to Marie implies not only an appreciation for her understanding and support when I was compiling this document, or even for the material that it represents, but for sharing a lifetime of joy and companionship that is actually beyond comprehension.

Our culture encodes a strong bias either to neglect or ignore variation. We tend to focus instead on measures of central tendency, and as a result we make some terrible mistakes, often with considerable practical import.

Stephen Jay Gould

FULL HOUSE – The Spread of Excellence from Plato to Darwin

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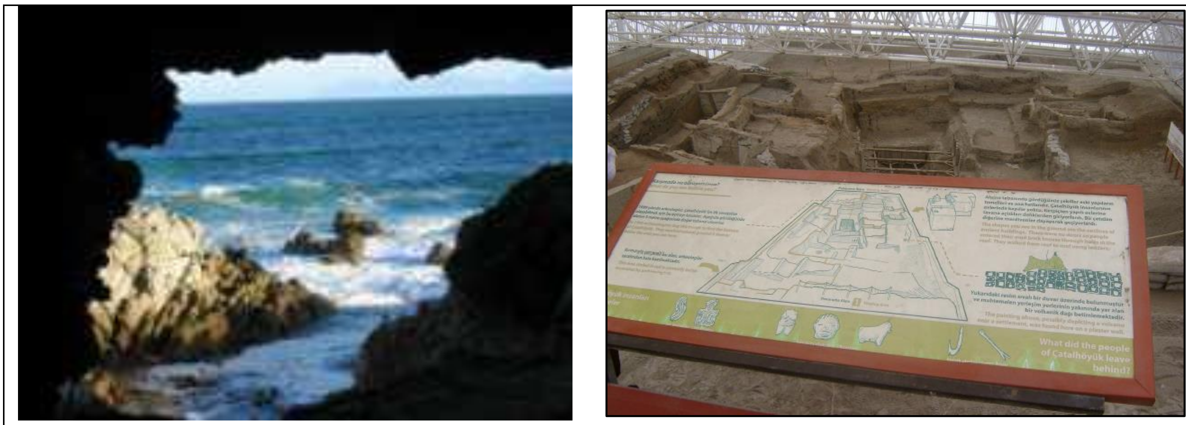
Abbreviations

ACC	African Concrete Code
CEN	European Committee for Standardization
EC2	Eurocode 2 or EN 1992 Structural Concrete
EN / ENV	European Norm/Standard / Voluntary edition
FORM	First Order Reliability Method
GPM	General Probability Model
GSF	Global Safety Factor $R_m/(G_m+Q_m)$
LoA	Level of Approximation
LS-D / pfLS-D	Limit States Design / partial factor Limit States Design
MCFT	Modified Compression Field Theory (for concrete shear resistance)
NDP	Nationally Determined Parameter (for Eurocode National Annex)
R2k	Response 2000 (for MCFT shear resistance prediction)
S-LS / U-LS	Serviceability / Ultimate Limit State
VSIM / VSIM-A	Variable Strut Inclination Method (for concrete shear resistance) / as modified

Chapter 1: Structures, their Design and a Reliability Basis for Performance

1.1 Background to the Development of Concepts of Structural Reliability

The close association between the cultural development of humankind and the use of physical artefacts also includes the sheltering and protection against the elements of nature, not only at a personal level but also to provide a protected environment for activities and goods. As sheltering progressed from the use of natural protection of caves to self-constructed dwellings and buildings, the need to provide for the load bearing capacities of the constructions emerged. Examples of the line of development is illustrated graphically but in a sketchy manner in Figure 1.1, from the Late Stone Age caves around the South Cape, to early constructions circa nine thousand years before present at Çatal Hüyük in Turkey or more elaborate constructions such as Hagar Qim in Malta circa five thousand years before present through to modern structures forming the Tokyo city-scape in an environment exposed to extreme atmospheric and seismic exposure.



Klipgat Cave, De Kelders, Western Cape

Çatal Hüyük proto-city, Anatolia, Turkey

Hagar Qim Temple, Malta

Tokyo’s seismic resistant buildings

Figure 1.1 *Human settlements advancing from (a) sheltering against the elements in Klipgat Cave, (b) Neolithic proto-city of clay brick houses with rooftop plazas and entry at Çatal Hüyük (c) Neolithic Temple at Hagar Qim (d) to highrise Tokyo buildings with extensive resistance against the elements*

Structural development obviously followed a progressive line of development from the emergence of empirical rules based on observations of failure and survival to the application of rational structural mechanics models as basis for design rules, complemented and characterised by testing and measurement. Risks of failure were accounted for by applying the most advanced technology to dare the elements head on.

The formal treatment of structural reliability introduced a new dimension, complementing the judgement based provision for structural performance derived from a structural mechanics approach. Even the use of structural reliability principles to obtain appropriate design measures evolved from nominal provisions derived from the experience base of acceptable existing practice, to elaborate reliability modelling and calibration based on risk optimisation as basis for determining acceptable structural performance levels.

Concepts of structural reliability evolved into the formal formulation of the basis of structural design that serves to imbed provisions for structural performance from the top, providing general requirements and procedures, down to specific design verification procedures; even extending to the way in which structural mechanics models are presented and qualified, beyond the scope of the design process to the level of specification of materials, construction procedures and the associated quality management.

1.2 Development of Concepts of Reliability in Structural Engineering

The notion that risk is not merely something society is exposed to is clearly identified and stated by Bernstein (1996): "The ability to define what may happen in the future and to choose among alternatives lies at the heart of contemporary societies. Risk management guides us over the vast range of decision-making, from allocating wealth to safeguarding public health". Engineers are commended particularly in this general treatise: "Without a command of probability theory and other instruments of risk management, engineers could never have designed the great bridges that span our widest rivers, homes would still be heated by fireplaces or parlour stoves, electric power utilities would not exist" Bernstein states furthermore: "The word 'risk' derives from the early Italian word *risicare*, which means 'to dare'. In this sense, risk is a choice rather than a fate."

This line of thought is expressed explicitly in the civil engineering context in two leading texts, firstly by Benjamin and Cornell (1970): "(The) new concern is with making *decisions* involving economic gains and losses when uncertainty exists in the decision maker's mind regarding the state of nature. This new emphasis, with its new interpretations and its new methods, is far more appropriate and natural for civil engineers, whose profession is more closely involved than any other in the economical design of one-of-a-kind systems subject to the uncertain demands of natural en man-made environmental factors." A similar approach is taken by Ang and Tang (1984) in the presentation of concepts of decision, risk and reliability under uncertainty.

A number of books are published dedicated specifically to structural reliability, notably one by Holický (2009) which is of specific interest to this dissertation due to the close association of the material presented to the background to the Eurocode *Basis of Structural Design* EN 1990:2002. In addition to the presentation of reliability theory and modelling procedures, information is given on representative probability models and generic calibration procedures that can be applied as a point of departure for constructing and assessing the reliability basis of design procedures.

Principles of structural reliability are integrally deployed in a semi-probabilistic design approach such as the partial factor limit states procedures used by Eurocode. Levels of application range from the specification of basic variables such as material properties to the target level of reliability to which design procedures are calibrated; from the representation of extreme environmental conditions to the unified treatment of structural materials such as steel, concrete and their composites, even geotechnical design. By using any of these levels of reliability as reference, the corresponding characteristics of the standard can be identified for assessment in terms of the functionality of the standard; with specific reference to higher levels of functionality related to the technical management of its development.

Progressively higher levels of the functionality of a standard that could be identified are:

(i) The technical performance of a specific standard can be related to the reliability basis for design verification within the scope of the standard;

(ii) Management of the development of the standard in relationship to complementary standards that may apply to the class of structure (e.g. loading and the respective materials-based standards, including foundation design, for buildings) can be related to reliability measures through which unification between the set is achieved;

(iii) The utilisation of a common body of knowledge and experience can be closely associated with the harmonisation of standards between various countries, regions or internationally as derived from a common reliability basis of design.

These concepts for the characterisation of standards for structural design and their development can be illustrated by reliability calibrations of Eurocode design models and procedures for their application in accordance to South African needs and conditions. Generalisation of these concepts may be useful even to Eurocode member states during the application of Eurocode in their country and to appreciate the differences of such application by other member countries. At the highest level, structural reliability concepts could serve as useful vehicle and basis for the development of international standards, contributing to global harmonisation and full utilisation of the common body of knowledge on structural performance.

1.3 Presentation of Contributions

A set of investigations are presented that can be characterised as the application of the theory of structural reliability that ultimately contributes towards the advancement of the rational basis for structural design as implemented in design standards. The context is the development of a new generation of South African standards for structural design, following the introduction of limit states design standards to the country about two decades before initiating a thorough review. The nature of the investigation is to explore opportunities to improve limitations and to advance the reliability basis of structural design for incorporation in new standards.

The Eurocode suite of design standards and background to it served as a rich reference base for such investigations. There was nevertheless a clear need specifically to provide for South African conditions and engineering practice. There were also opportunities to clarify some issues imbedded in the approach taken by Eurocode, even improving on these. The radical differences in the institutional and technical environment between Europe and South Africa were identified as a fundamental issue to be addressed. Investigations along these lines resulted in a degree of generalisation of the body of investigation which should be of interest to other countries endeavouring to transfer Eurocode specifically, even to the development of standards for structural design in general. Participation in the activities of the Technical Committee of the International Organisation for Standardisation concerned with the *Basis for Structural Design and Actions*, ISO TC98 provides a suitable platform for incorporation of some concepts internationally.

1.3.1 Characteristics of investigations

In addition to the series of investigations evolving over a period of time, practical considerations dictated that the investigations have the appearance of being dispersed across a number of topics. Due to a common basis of the identification of specific topics where further investigation was justified and the ultimate vision for the results to serve as reliability basis for structural performance against the background of standardised design, it is possible to present the body of investigations in some coherent form. Characterisation of the conditions and contributions may serve as basis for the classification of the set of publications.

***Research Platform*:** Appreciation of the utility and merit of the field of risk and reliability in the Department of Civil Engineering, Stellenbosch University provided the opportunity to build up a research activity and competence in this field. Various research programs in Structural Engineering provided the platform for fundamental investigations, graduate research projects, contributions to standardisation of structural and geotechnical design, national and international networking contributing to an extension of the knowledge base that could contribute to the research, development and application.

Research Program**:** Such favourable conditions challenged the Candidate to provide the entry level knowledge and critical topics for graduate research, relate and transfer the efforts to design practise, keep up with and contribute to international developments and expertise, and transfer these to the local scene. Nevertheless this needed to be managed with a relatively sparse resource base initially, progressively growing over the years both within the research team and in terms of colleagues in practice.

Cooperative Network**:** A prominent feature of the research activities presented in this dissertation is its cooperative nature, as implied by the extended research platform outlined above. In many instances investigations were shared by the Candidate with specialists in the field of structural and geotechnical engineering or with international experts in reliability. Consequently the bulk of the references presented in this dissertation is shared with an extensive body of Co-authors. However, the multiplying effect also contributed significantly to the ability of the Candidate to compile the body of knowledge forming the basis for this dissertation.

Research Emphasis**:** The most effective way of advancing risk and reliability in the local environment was judged by the Candidate to place the emphasis on applications, providing for local needs; further investigations where deficiencies could be identified; linking in to the advances made elsewhere, as adjusted to local conditions. The emphasis on local conditions and applications can be contrasted to focussing on front end development. With the emphasis on application, development of reliability procedures is directed more towards the improvement of existing practice, as opposed to the advancement of reliability models. However, it is often found that local needs are generic and therefore also of general interest. Examples are the gap between reliability models on which codification is based and the development of advanced reliability models; similarly the concept of model uncertainty of design functions is well established, but treated rather superficially.

Levels of Contribution**:** The comprehensive body of contributions can be classified into three concentric levels:

(i) *Collective Contribution*: At a technical management level, the set of publications presented by the Candidate should be considered as being representative of the collection of the cooperative network; the research platform; the institutional research network; the progressive program of development followed by the campaign presented in this dissertation as outlined above.

(ii) *Research Framework*: The individual contributions presented in this dissertation are arranged as the building blocks or components of a scheme of structural reliability, representing a research framework to represent a reliability system in accordance with its implementation in design.

(iii) *Basis of Structural Design*: The central role of the standardised basis of structural design and its direct relationship represent the specific focus of the dissertation. A selection of publications on this topic, as authored or co-authored by the Candidate, is discussed in more detail.

1.3.2 Presentation mode

The case for this dissertation is based on a set of publications cited throughout the document. Accepting that it is not practical to even summarise these papers here, the level of presentation is intended to be limited to statements motivating the respective investigations, giving indicative results, with an emphasis placed on the conclusions. This approach is based on the premise that full account of the investigations are given in the referenced paper.

One consequence of this mode of presentation is that only a sparse set of references from the rich body of knowledge on the field under scrutiny is presented here. The only defence against such practice is that a more extensive representation would require a substantial extension of the scope of the dissertation to do justice to the respective authors and material, which is already done in the references on which the dissertation is based.

The joint contributions of co-authors to the cited papers are duly acknowledged. Although the implication is that not only original contributions by the author of this dissertation is reflected here, the material is presented on the basis of engagement with colleagues and co-workers, sharing the delights of problem identification, assessment, conclusion and often application. Co-authorship is therefore presented on the positive side of the balance sheet; attesting to the insights gained by the Candidate through the engagement; based on an assumption that the willingness of co-authors to cooperate reflects a positive gain to them as well.

Apologies are made for inconsistencies in the style and format of material extracted from cited references, typically caused by the presentation of material in pictorial format, where adjusting of the style is judged not really to add value to the dissertation.

1.4 Outline of Dissertation

The development and application of the principles of reliability have the character of a system, with many classes, levels and interrelationships, making it impossible to establish a single logical sequence of all the components. In addition, involvement in developing models, methodologies and applications took place over many years; taking the relevant perspective of the role of structural reliability as the case may be at that instance.

The topics submitted here are consequently arranged around:

- The main focus of the presentation, which is the application of reliability in standards development;
- Specifically as the basis for using Eurocode as reference for the next generation of South African design standards;
- Starting with initial investigations and followed by more detailed investigations;
- Often of topics identified during standards development;
- In conclusion a few themes are presented where the specific investigations are generalised or interpreted at a higher level of abstraction.

This development is arranged in the following themes and chapters:

***Background and Initial Investigations*:** A brief outline of the general theory of structural reliability and the basis for its conversion into semi-probabilistic design procedures is presented in Chapter 2. A few preliminary reliability studies are reported in Chapter 3. It turned out that these investigations provided a good preparation; at least an anticipation of subsequent contributions by the Candidate to standardisation at various levels and instances, or demonstrating the merit of the systematic treatment of reliability in the process.

***Standards Development*:** Contributions to standards development for South Africa by the Candidate are presented in Chapters 4 – 6. The formulation of the general approach taken with the revision of the previous South African Loading Code SABS 0160:1989 into the present SANS 10160:2010 is presented in Chapter 4. The introduction of a separate head standard SANS 10160-1 is motivated and elaborated on in this chapter. The role of reliability in the provisions for actions is covered in Chapter 5, with specific reference to imposed loads, wind loading, crane loading and pile foundations as part of the geotechnical basis of design. These two chapters are largely based on material from *Background to SANS 10160 – Basis of Structural Design and Actions for Buildings and Industrial Structures* (Retief & Dunaiski (Eds) 2009). Investigations related to standardisation of procedures for the structural use of concrete with which the Candidate were involved are included in Chapter 6; with specific reference to concrete resistance in general, shear resistance in particular, cracking serviceability of water retaining structures, a case of reliability based inspection of structural concrete and model uncertainty.

***Specific Reliability Investigations*:** Although there is not a strict division between reliability modelling specifically in terms of standards development or specific topics that are treated in greater depth, a few investigations, initiated by or contributed to by the Candidate, are reported as individual studies in Chapter 7. Topics included in this chapter are strong wind investigations related to severe wind damage and modelling, the South African strong wind climatology, the reliability modelling of wind loading; further investigations on the reliability assessment of pile foundation resistance; a brief report on the generalisation of the reliability basis of standards development; an overall scheme for the treatment of model uncertainty as encountered in various investigations reported throughout the dissertation.

***Basis for Standards Development*:** The concept of taking a higher level of abstraction of the role of standards for structural design is introduced in Chapter 7; a more elaborate treatment of the process is then given in Chapter 8. An exposition is given of how the development of this topic progressed from:

- Capturing some principles for standards development on which decision making is done implicitly by standards committees, to
- Providing a rudimentary scheme of principles based on these observations
- With sufficient substance to be able to demonstrate, perhaps in hind sight, how this framework
    - Can be used proactively for strategic and operational use in the development of a given standard;
    - Ultimately initiated a program of standards development in terms of clearly formulated objectives and resources.

***Generalisations*:** A feature that emerges from the set of investigations is that there is an interplay between specific investigations in some depth and the broader scheme of development; with specific investigations providing more refined models that contribute to the general advancement of the reliability basis of design; inversely identifying critical opportunities for such advancement by the selection of specific refined investigations. Once stated, this is an obvious trait of applied engineering research; on the other hand, clear recognition and management of such a process should contribute to ensure the maximum synergy between a series of investigations. Some observations of this process that can be identified from the suite of investigations reported in this dissertation are presented in Chapter 9.

***Conclusions and Recommendations*:** The broad scope of investigations presented in this dissertation makes it almost impossible to draw conclusions on the specific results and outcomes obtained. At the other extreme overall observations may tend to be so general as to become self-evident. The conclusions presented in the final chapter will therefore address and substantiate the motivation for the endeavour:

- To demonstrate the merit and utility of extending the structural mechanics basis for structural design to include provisions based on risk and reliability as a rational basis for ensuring sufficient and economic structural performance;
- To indicate that such considerations are relevant across all components of structural design from actions:
    - Through structural resistance from steel to geotechnical design,
    - At levels ranging from specific failure modes to meta-standard considerations of the role and function of design standards;
- Observations are made on the route of development from specific investigation to a standardised and calibrated design procedure and the inherent nature of reliability:
    - To apply the development ‘partially’ to model components;
    - That can subsequently be merged and generalised.

Recommendations are based largely on the many needs and opportunities for further investigation and improvement of design practice. It is trusted that these pointers may inspire future researchers towards such enterprises.

Structural mechanics specialists are particularly challenged by pointing out that a proper grasp of any specific model can only be claimed if the limits of assumptions and approximations and experimental verification can be expressed quantitatively in terms of inherent variability and uncertainties.

1.5 Context of Presentation of Campaign

Contributions by the Candidate to the development of standards for structural design do not qualify for inclusion in this dissertation. However, the context within which the campaign of investigations took place, plays an important role in appreciating the utility of this body of information and the role of the Candidate beyond its academic contents. An outline of the various capacities within which the investigations were made is thus provided here, with a degree of subjective interpretation of the way in which the Candidate used these positions to initiate, lead, execute or assess the various activities.

1.5.1 Academic and research platform

The positions and duties of the Candidate within the Stellenbosch University Department of Civil Engineering and the Institute of Structural Engineering served as platform for the research campaign submitted in this dissertation. Relationships with colleagues and co-workers who feature as co-authors of the papers on which the dissertation is based, are indicated. These duties should be qualified by the fact that it was performed as Emeritus Professor on a part-time appointment for most of the time.

Director of the Institute of Structural Engineering (ISE) (1991 – 2002): The most important initiative of the Candidate was the co-founding together with the late Prof Peter Dunaiski of the Centre for the Development of Steel Structures (CDSS) to convert graduate activities, consisting mainly of individual bursaries from the SA Institute of Steel Construction, into a managed research program based on extended industry support, leveraged by the THRIP initiative sponsored by the DTI.

CDSS Project Team member (1998+) & Project Leader (2011-2014): Members of the CDSS played a leading role in the development of a revised South African Loading Code SANS 10160 and related research, amongst other activities (see Section 1.5.2). The Candidate shared responsibilities with Prof Dunaiski and took over the main duties when he passed away.

APERCS Team Member: Following the launch of a research program on advanced concrete materials by Prof Gideon van Zijl, the Candidate took responsibility for investigations related to the reliability performance of structural concrete. These efforts fed into the activities of the Working Group on the revision of the South African Concrete Code led by Prof Jan Wium.

Water Research Commission (WRC) Project Team Member: The Candidate took some early initiatives on exploring research funding for the development of a South African standard for the design of water retaining structures. Two WRC projects were led by Prof Jan Wium and Dr Celeste Viljoen respectively, with the Candidate being a project team member; including a Working Group member for the development of SANS 10100-3 led by Dr Viljoen. The Candidate also participated in a WRC project on design procedures for dam freeboard design, providing for input on dam risk assessment.

International Collaborators: Extensive efforts were made by the Candidate to engage leading international researchers in the field of structural and geotechnical reliability, resulting in many exchange visits to maintain such a network. A prominent example of such collaboration is the appointment of Prof Milan Holický as Extraordinary Professor in the Department of Civil Engineering. Another example is the hosting of the ISO TC98 2011 Annual Meeting in Stellenbosch.

Risk and Reliability Research Group: The candidate took the responsibility to build up a research group on Risk and Reliability in Civil Engineering. Strong support from Management over the years is gratefully acknowledged. Since most of the work is related to some applications, the activities are generally imbedded in the various programs of the ISE. Activities are closely integrated with extensive experience and activities in the development of standards for structural design. Establishment of the R & R Research Group has been so successful that the Candidate can now retreat to the comfortable position of mentorship. The group which is now led by Dr Viljoen includes Mr van der Klashorst, Prof Holický, Dr Dithinde as research associate, now joined by Dr Lenner.

The research group contributed to the emergence of a strong group playing a leading role in standards development, including Prof Peter Dunaiski[†], Dr Hennie de Clercq, Prof Peter Day, Prof Jan Wium, Prof Gideon van Zijl, with Dr Adam Goliger closely associated with this group. The nett effect is that contributions from the group go beyond that of the individual; it includes taking new initiatives, sometimes at strategic level, taking on overall and project responsibilities and in the process organically taking care of gaps and deficiencies; facilitating implementation of standards, recording background information and bases for decision making; doing complementary research. The Candidate is privileged to make contributions within such a team.

Academic Output and Research Supervision: The primary objective of all the initiatives outlined above is to provide opportunities for graduate study and supervision for Masters (9), PhD (9) and DEng (2) candidates; with completed numbers shown in brackets and one PhD and DEng each in progress. Some claim for credit in contributing to the healthy growth in the research effort of the Department, and the Division of Structural Engineering and Informatics in particular, can be substantiated by the relative contribution of doctorate candidates (10 out of 39) and 9% of the Departmental publication units over the last 15 years.

Colleagues and Co-Workers: In order to clarify the relationship of the Candidate to colleagues and co-workers who co-authored the papers submitted in this dissertation, the following list is provided:

> ***International Colleagues*:** Professors D Diamantidis; MH Faber; M Gizejowski; M Holický; AR Kemp†; M Maes; H-J Niemann; KK Phoon; M Sykora.
>
> ***Colleagues*:** Dr C Viljoen; Dr M De Wet; Prof PE Dunaiski†; Mr G Maritz; Mr E van der Klashorst; Prof GPAG van Zijl; Prof JA Wium.
>
> ***Supervised Co-Workers*:** Dr A Bester; Dr GC Cloete; Prof PW Day; Dr M Dithinde; Dr AM Goliger; Prof F Hugo; Dr GM Ker-Fox; Dr AC Kruger; Dr KK Mensah; Dr C van Dyk; Dr JS Warren-Dymond; Mr J Botha; Mr WW Brand; Mr JH De Lange; Mr PJ De Villiers; Mr UA Huber; Mr M Jacobsohn; Mr J Marengwa; Mrs CH McLeod; Mr A Muhimua-Joao; Mr TR Ter Haar.

Not all co-workers contributed to the publications submitted in this dissertation. The co-authors not listed above are generally related to international colleagues.

Merit and Awards: A spate of awards fully rests on the merit of the receivers; the reward of being associated with these awardees as supervisor is claimed by the Candidate on the basis of the harder he tries to engage in supervisory capacity with engineers of quality, the luckier he gets:

- Emeritus Professor Fred Hugo (DEng) was awarded the *Degree Doctor of Engineering, honoris causa* in 2014 from Stellenbosch University.
- Adjunct Professor Peter Day (DEng) received the *SAICE Engineer Award* for 2014.
- Dr Cobus van Dyk (PhD) received the *Young Engineer Award* from SAICE for 2014.
- Dr Mahongo Dithinde (PhD) and the Candidate shared the *JE Jennings Award* for 2014 for the best geotechnical paper, awarded by the SAICE Geotechnical Division.
- Dr Greg Ker-Fox (PhD, MScEng) was the *Risk Manager of the Year* for 2007.

1.5.2 Standards development – General

Involvement in the development of National Standards for structural design provided a natural avenue for implementation of the development of risk and reliability. Although contributions to standards do not qualify as academic output, the process of scrutiny by project teams can arguably be considered to represent strict peer review. Nevertheless, involvement in Standards Technical Committees (TC), Project Teams (PT) and Working Groups (WG) are provided here as part of the context for the contributions submitted by the Candidate.

The following interactions between academic research and standards development should be identified and managed to obtain the best benefit to the engineering profession and practice:

- The primary technology input and advancement of design standards derive from research. A major premise of this dissertation is that there remains extensive scope to enhance structural performance through reliability based design.
- Strict censorship is applied by practice to identify needs for standardised procedures that are sufficiently operational and effective. Full acceptance of quantitative reliability procedures is still a challenge; both as the result of the limited success with demonstrating the merit of the approach by reliability experts and the inability of other TC members in interpreting structural mechanics models in reliability terms.
- Available research results need to be interpreted to serve as background information for optimal joint decision making by practitioner and researcher TC members. A major effort by the Candidate and associates to provide such background is represented by this dissertation.

- Deficiencies in available standards serve as source and motivation for research programs and priorities. Several cases are presented where research topics were identified from the background to standards development.

The three cases of the development of the South African standards for loading, structural concrete and water retaining structures provided an opportunity to consider (i) the reliability performance of both actions and resistance (ii) three modes of standards development, consisting of adoption, adaptation of a reference standard and the de novo development of a standard. This provided the opportunity to take a general view on structural design standards and their development.

1.5.3 Development of the South African Loading Code SANS 10160

Stages of Development: The activities of the *SAICE Working Group on Revision of the SA Loading Code* (WG-LC) went through several clear stages of development. The pre-WG activities are described by Day (2013), describing an interest by the Geotechnical Division in Eurocode EN 1997 *Geotechnical Design* that led to the South African National Conference on Loading in 1998 as precursor to the WG-LC in 1999. An important factor inhibiting the implementation of EN 1997 in South Africa was the incompatibility of SABS 0160:1989 with Eurocode action combination schemes (Day 2013). The involvement of the Candidate with the WG-LC during its various stages can be summarised as follows:

- **Concepts for SABS 0160 procedures:** Up to 2003 various schemes for the advancement of the procedures of the previous Loading Code were considered. The Stellenbosch group (Dunaiski, Retief and Ter Haar) considered action combination schemes, imposed loads and crane induced loads. Dr Adam Goliger explored options for wind loading procedures; with the Candidate getting involved having supervised his PhD project. When Prof Jan Wium joined Stellenbosch University, he took on an assessment of provisions for seismic design. Other WG-LC members served to comment on possible ways to go forward. It was however still a divergent process, without systematic progress.
- **Exploring Eurocode as Reference standard:** The Candidate was invited in 2002 by Prof Haig Gulvanessian, Chair Eurocode CEN TC250/SC1 *Actions on Structures* and Project Leader *Basis of Structural Design* to attend SC1 meetings as an observer. Attendance to SC1 meetings by the Candidate, Prof Dunaiski and Mr Tim ter Haar provided access to background documents and drafts of converting Eurocode from the voluntary (ENV) to the normative (EN) version on which adoption by member states would be based. (Additional comments on Eurocode interaction is provided below.) The upshot was however that a trial implementation of Eurocode procedures were performed during 2003, with the following main outcomes:
    - ***Action combination scheme*:** The SABS 0160 procedures turned out to be compatible with one of the options provided for in EN 1990, removing a critical inconsistency between established South African practice and Eurocode.
    - ***Self-weight and imposed loads*:** Clear advantages of the more up-to-date and comprehensive Eurocode were evident, as determined by a Stellenbosch assessment (Retief, Dunaiski, De Villiers).
    - ***Wind actions*:** Joint investigations (Goliger, Retief, Dunaiski) indicated substantial advances achieved by Eurocode, but critical adjustments were needed to adapt procedures to local conditions and practice.
    - ***Crane induced loads*:** Assessment by a Stellenbosch project team (Dunaiski, Barnard, Warren, Retief) established the advantages of basing local procedures on Eurocode as derived from the corresponding DIN standard which also serves as basis for ISO and other international standards.
- **SANS 10160 Development Phase:** The Stellenbosch group, led by Prof Dunaiski and the Candidate then jointly developed a proposal for a comprehensive revision of SANS 10160 based on adaptation of various Eurocode Standards (EN 1990, EN 1991, EN 1997, EN 1998) and selected Parts, but scaled substantially to South African conditions and practice. This proposal was accepted by the WG-LC in 2004 and a full draft of eight parts was essentially completed in 2008.

    - The Stellenbosch group took full responsibility for the overall development; managed jointly by Prof Dunaiski and the Candidate. Such an arrangement was essential, considering that the Loading Code developed from a single standard of 123 pages to a set of eight related standards totalling 375 pages.
    - Five *Champions* (Day, Dunaiski, Goliger, Retief, Wium) took the lead in preparing the draft standards for approval by the WG-LC as follows:
        - The Candidate – Part 1 *Basis of structural design* including accidental design.
        - Prof Dunaiski – Part 2 *Self-weight and imposed loads*, jointly with the Candidate.
        - Dr Goliger – Part 3 *Wind actions* assisted by the Candidate and Prof Dunaiski.
        - Prof Wium – Part 4 *Seismic actions and general requirements for buildings*.
        - Dr Day – Part 5 *Basis for geotechnical actions and actions*. The Candidate was involved in ensuring a unified approach with Part 1; input on pile design was provided together with Dr Dithinde.
        - Prof Dunaiski – Part 6 *Actions induced by cranes and machinery* with input by the Candidate and Dr Warren-Dymond on load calibration.
        - Prof Dunaiski – Part 7 *Thermal actions* and Part 8 *Actions during execution*, jointly with the Candidate.
- **Publication by SABS as National Standard:** Considerable efforts were made to get the Working Group Draft (WGD) approved and published by SABS as South African National Standard, including the withdrawal of SABS 0160:1989. The process was jointly managed and administered by Prof Dunaiski and the Candidate, consisting of the following main steps:
    - Final editing of the WGD, re-editing and correcting various drafts by SABS, including version control and archiving throughout the development process, liaison with the various Champions, managing a process of independent review by a panel of *Readers*.
    - Reactivating an appropriate SABS committee structure for balloting the various draft stages. This process that took place somewhat independently from the WG-LC program is set out in more detail below.
    - Correction of errata in SANS 10160:2010 required the publication of an updated 2011 version. Note that for historical reasons, reference is made throughout the dissertation to the 2010 version.
- **Implementation of SANS 10160:** Responsibility for SABS 10160 was formally transferred from the SAICE WG-LC to the relevant SABS Technical Committee, ultimately residing with SABS TC98 *Structural and Geotechnical Design Standards* SC 98/01 *Basis of Design and Actions*. Nevertheless, the Stellenbosch group took initiatives to consolidate the experience gained and induce the utilisation of the new South African Loading Code. These activities consisted mainly of the following:
    - Publication of the Background Report – *Background to SANS 10160.* This report was initiated by the Candidate, served jointly as editor with Prof Dunaiski and contributed extensively to its contents (Retief & Dunaiski (Editors) 2009).
    - Presentation of a series of seminars on the standard, including an extensive set of lecture notes and examples. Three series of seminars consisted of
        - An introductory series in 2008 allowing for informed public comments at the Draft South African Standard (DSS) stage,
        - An induction series after publication of the standard, presented in 2011, repeated in 2012;
        - Advanced seminars on selected topics followed in 2013 and 2014.

1.5.4 Development of standards for structural concrete

The treatment of actions and their combinations gets the most attention when applying reliability to design procedures. This is justified by the wide range of design conditions, with self-weight being an important but relatively predicable class of loading at the one end, through severe conditions from use and the environment that could occur during the life of the structure, to extreme conditions that need to be considered even if there is only a low probability of occurrence during the service life of the structure. It is therefore somewhat surprising that the reliability implications of structural resistance turn out to be of similar importance than that related to actions. Moreover, the importance of gross error as a dominant source of insufficient resistance is indicated by the emphasis given to quality management in the reliability basis of structural design given by recent standards (ISO 2394:2015; EN 1990:2002; SANS 10160:2010).

Opportunities arose to consider the reliability assessment of structural resistance as applied to the design of concrete structures. Involvement with the Working Group on revision of SANS 10100-1 *Structural Concrete – Design* represents a case where the adoption of Eurocode EN 1992-1-1 has been decided on. The development of a standard on the design of concrete water retaining structures SANS 10100-3 represents a case where a new standard is being developed.

Structural Concrete SANS 10100-1: When Prof Jan Wium took on the duty to lead the Working Group on the revision of SANS 10100-1 the Candidate took responsibility for matters related to the reliability basis of design for concrete structures. The emphasis was however more on performing background investigations and modelling. The strategy of adoption of EN 1992-1-1 left limited opportunity for standards formulation and development as such. On the other hand it provided an opportunity to assess the reliability performance of a completed standard.

The following topics were identified by the Candidate where further assessment was justified:

- Integral assessment of the standard from a reliability perspective, ranging from the basis of design procedures, for example partial material factors; to the implications for the balance of the standard, such as models for various failure modes, quality management.
- The implications of differences between conditions within the Eurocode environments and South Africa; for example provisions for specialist applications such as high strength concrete.
- Identification of specific failure modes in need for further analysis; notably shear resistance.
- Generalisation of specific findings; for example proper provision for model uncertainty.

Water Retaining Structures SANS 10100-3: The program for development of a South African National Standard for the design of water retaining structures contains all the elements of an ideal situation for standards development: Research on which extensive background information could be compiled, including the experience represented by various reference standards, launching the formal standardisation so effectively that a properly resourced and managed program was complemented by a strong contingency of experienced practitioners, resulting in a fast track schedule for advancing the draft where it could be processed by the standards body.

The Candidate claims some credit for most of the stages of the program, with the emphasis on the initiation and initial phases, including a strategic structuring in terms of technical needs and objectives, alignment of the various components of the program and obtaining the resources for execution. The most important resources however were the competence and abilities of Prof Jan Wium leading the first WRC project and Dr Celeste Viljoen leading the second WRC project and the Work Group activities.

1.5.5 Standardisation committees

A clear distinction should be made between pre-normative background investigations from which Working Groups proceed on the one hand, and the formal review process for which standardisation committees take responsibility. Since these formalities are vital to successful standardisation, the Candidate took an interest in ensuring that approval of a highly technical standard is done expertly and expediently.

At the initial stage SABS 0160:1989 resorted under the main relevant SABS committee SABS TC59 *Construction Standards* with wide representation across the construction industry. Together with Prof Dunaiski and Mr Dirk Loubser from SABS steps were taken to reactivate the dormant TC59 SC59I *Basis for Design of Structures* in time for approval of the drafted SANS 10160. Prof Dunaiski took over the chair from Prof Alan Kemp when he retired; the Candidate took over the chair in 2011.

Amidst a process of restructuring SABS TC98 the Candidate promoted the establishment of one Technical Committee dedicated to standards for structural design. The obvious motivation is to ensure that TC members are sufficiently competent to take care and judge on the specialist nature of these standards. However, the need to ensure unification between the various South African standards with different reference bases is an important consideration. This is a topic that is referred to recurrently throughout the dissertation, with the emphasis on reliability concepts serving as unifying platform for the diverse range of structural materials, from steel to geotechnical materials, all to be served by a common loading code.

The proposal was made to align the SABS TC with ISO TC98 *Bases for Design of Structures*, but to structure the TC in accordance with Eurocode CEN TC250 to include materials-based design standards. SABS TC98 *Structural and Geotechnical Design Standards* was ultimately constituted on 31 July 2012. Prof Peter Day was elected as chair, as nominated by the Candidate. SC01 *Basis of Design and Actions (including Seismic)* was chaired by the Candidate for the first year. Prof Jan Wium accepted the chair of SC02 *Design of Concrete Structures*, as nominated by the Candidate. By taking a long-term view on standards development, Prof Chris Roth was nominated by the Candidate to take over the chair of SANS TC98/SC01. The candidate maintains membership of SABS TC98, SC01 and SC02 as part of the Stellenbosch University delegation. Since 2005 the Candidate also serves as SABS representative to ISO TC98; bridging many phases of SABS's membership of ISO TC98; now properly managed as part of the SABS TC98 international liaison.

1.5.6 International standards development

International interaction is vital for a healthy process of standards development in South Africa. However, special efforts need to be made to develop and maintain involvement in standards development, simply as a result of geographic constraints, more importantly due to limited organisational embedment into standards generating organisations. The Candidate therefore appreciates the opportunities referred to above, consisting of contributing to ISO TC98 activities and liaison with Eurocode CEN TC250, specifically SC1 on actions on structures. This also provided opportunities for interaction with the Joint Committee on Structural Safety (JCSS), serving as body doing pre-normative background investigations on risk and reliability feeding into standardisation proper at different levels.

ISO TC98 *Bases for Design of Structures*: The main duties as SABS delegate consist of participating in the activities of the main TC, the two active subcommittees SC02 *Reliability of Structures* and SC03 *Loads, Forces and other Actions* and related Working Groups. Two specific initiatives with which the Candidate was involved because they are viewed to play a significant role in international harmonisation, are summarised here:

ISO 2394 Revision: When ISO 2394:1998 *General Principles on Reliability for Structures* was routinely reapproved in 2008, the Candidate initiated a process for reviewing the standard. A proposal was tabled jointly with Prof Milan Holický in 2009 at the ISO TC98 Annual Meeting.

- An ad hoc group (Holický, Retief, Maes) was appointed to motivate revision and to prepare a proposal.
- Various planning sessions were held (i) Stellenbosch (February 2010) to prepare an outline of a New Work Item Proposal (NWIP); (ii) Munich (April 2010) submitting the proposal for comment to the JCSS; (iii) Darmstadt (4-5 November 2010) to finalise the NWIP.
- The proposal was approved at the ISO TC98/SC2 Annual Meeting in Delft (2 December 2010), with Prof Michael Faber (JCSS President) appointed as Convenor and the Candidate as member of the management team.
- A related activity was to set up a South African Mirror Group to comment on the various drafts of the standard.

***ISO 2394 Annex D*:** Based on the experience with the successful inclusion of Part 5 *Geotechnical Basis of Design and Actions* into SANS 10160:2010, the Candidate promoted explicit provision for geotechnical design in the revised Standard. Accordingly, Annex D *Geotechnical Reliability Based Design* was included in ISO 2394:2015.

- The Candidate served as liaison with a separate team led by Prof KK Phoon to develop Annex D. This function included assisting in launching the development of the annex and in editing the final version to be consistent with the scope and format of the Standard. The final editing was done jointly with Dr Mahongo Dithinde.

***ISO 22111 Revision*:** Although ISO 2394 is essential to establish a standardised approach to structural reliability, it is ISO 22111 that has the potential to convert general principles to the operational level for standardised semi-probabilistic design. In spite of voting to maintain ISO 22111:2007 *Bases for Design of Structures – General Requirements*, the Candidate volunteered at the 2012 ISO TC98 meeting to prepare an assessment of the sufficiency of the present standard and its possible revision.

- A report recommending revision to reflect the implications of the revised ISO 2394 and enhance the level of international harmonisation was accepted at the 2013 Annual Meeting
- Following additional assessment jointly with Prof Milan Holický the Candidate was appointed as Convenor for such a review at the 2014 Annual Meeting.
- The first ISO TC098 SC2/WG8 meeting is scheduled for 17 July 2015 in Vancouver BC, Canada in conjunction with ICASP12; chaired by the Candidate.
- The basis for the revision of ISO 22111 was accepted at the WG8 Vancouver meeting, comprising of (i) a standardised expression of *requirements for semi-probabilistic design* in accordance with the newly formulated *principles* of ISO 2394:2394; (ii) based on harmonized procedures extracted from leading international standards.

***Eurocode Liaison*:** The invitation by Prof Haig Gulvanessian to attend meetings of Eurocode CEN TC250/SC1 *Actions on Structures* referred to above, resulted in various activities in which the Candidate was involved:

- The primary benefit was to obtain early information of the drafted standards. It included the background information; going beyond motivation for codification decisions, also revealing the wider technology base and expertise, even to the point of competing for inclusion.
- At a higher level the striking differences between the Eurocode and South African institutional and technical conditions could be observed, allowing for adjustments when transferring the core of Eurocode standards and procedures to South Africa.
- It was nevertheless surprising to observe that Eurocode development in terms of national implementation was not ahead of the development of SANS 10160 – the development of the normative EN standards, as converted from the voluntary ENV version, which was initiated in 1998, still needed the development of national annexes for each standard and part before national implementation could be launched.
- Progress with the development of SANS 10160 was presented regularly at the WG1 meeting by the author. The South African standard was presented as an example of extending Eurocode beyond the European borders, adapted to local conditions.
- Various meetings were held with CEN TC250 chairmen (Prof Bossenmaier, Prof Calgaro) and various CEN and BS Secretariat representatives for Eurocode in order to explore extended liaison and cooperation.

Chapter 2: Concepts of Structural Reliability

This chapter serves to give a palette of all the components of structural reliability analysis as reference and context for all the individual investigations, serving to provide some coherence out of what may appear to be ad hoc and fragmented; conversely allowing for some deeper insight to (hopefully) emerge from the fragmented components.

The theory of structural reliability, from which concepts are derived for the formulation of semi-probabilistic design procedures, is extensively presented in the literature; to the extent that the material is covered regularly in academic courses. An outline of the underlying theory and the way it is implemented into models that serve to reflect the performance of structures in reliability terms would nevertheless be beneficial as background to evaluate and appreciate the contributions presented in this dissertation on various investigations on the development and application of such reliability models. The outline is limited to concepts that can assist the non-specialist in appreciating the utility of a reliability based approach towards structural design, as opposed to a fundamental treatment and review to improve the insight of the reliability specialist. Furthermore, full treatment of all aspects of structural reliability is not attempted due to the extensive scope of such a venture, even though the outline is limited to standardised application; because conceptually each element of the process can be improved, enhanced and advanced on the basis of reliability concepts.

2.1 Reliability Representation of Structural Behaviour

2.1.1 Reliability performance function

Any deterministic function g in terms of variables $\{x_1; x_2; \dots x_n\}$ for instance modelling an action on a structure or the sectional resistance of a structural element can be converted into a probabilistic or reliability function by converting the variables into random or basic variables $\{X_1; X_2; \dots X_n\}$ in terms of probability models and associated distribution parameters to reflect variability or uncertainty of the basic variables. Due to the complexity of obtaining general solutions for $g(X_1; X_2; \dots X_n)$ various classes of simplification are applied for reliability modelling of structural performance and to derive practical design procedures. The key to these simplifications follow from the fact that an explicit analytical solution can be obtained for a linear function as given by Equation (2.1) when the basic variables are represented by Gaussian or Normal distributions with distribution parameters given by the mean (μ_X) and standard deviation (σ_X).

$$g(X_1; X_2; \dots X_n) = a_O + a_1X_1 + a_2X_2 + \dots + a_nX_n \tag{2.1}$$

For this case $g(X_1; X_2; \dots X_n)$ is also Normally distributed with distribution parameters given by Equation (2.2):

$$\mu_g = a_O + a_1\mu_1 + a_2\mu_2 + .. + a_n\mu_n \tag{2.2a}$$

$$\sigma_g^2 = (a_1\sigma_1)^2 + (a_2\sigma_2)^2 + .. + (a_n\sigma_n)^2 \tag{2.2b}$$

On the premise that the Normal distribution represents the most basic provision for variability or uncertainty of the input variables, requiring information on the best estimate used for the mean (μ_X) and relating some indication of dispersion to the standard deviation (σ_X), a totally new dimension is brought into deterministic modelling; even if allowing only for the transparent and rational treatment of judgement based measures that form an integral part of good engineering practice.

The practical use of the reliability function is not so much to obtain a general solution to it, but to set it to estimate the probability of the function exceeding a certain limit, conventionally expressed as a reliability performance function given by Equation (2.3):

$$g(X_1; X_2; \dots X_n) = a_O + a_1X_1 + a_2X_2 + .. + a_nX_n = 0 \tag{2.3}$$

The probability that $g < 0$, typically representing structural failure in a limit state function, is given by the cumulative Normal distribution function Φ as:

$$P(g(X_1; X_2; \ldots X_n) < 0) \quad = \quad P_f(g) \quad = \quad \Phi(-\mu_g/\sigma_g) \quad = \quad \Phi(-\beta) \tag{2.4}$$

Where β represents the distance of the mean of g to the origin of the function in units of its standard deviation; β is conventionally defined as the reliability index for a reliability performance function. A graphic representation of the reliability performance function, the failure probability and the reliability index is given in Figure 2.1.

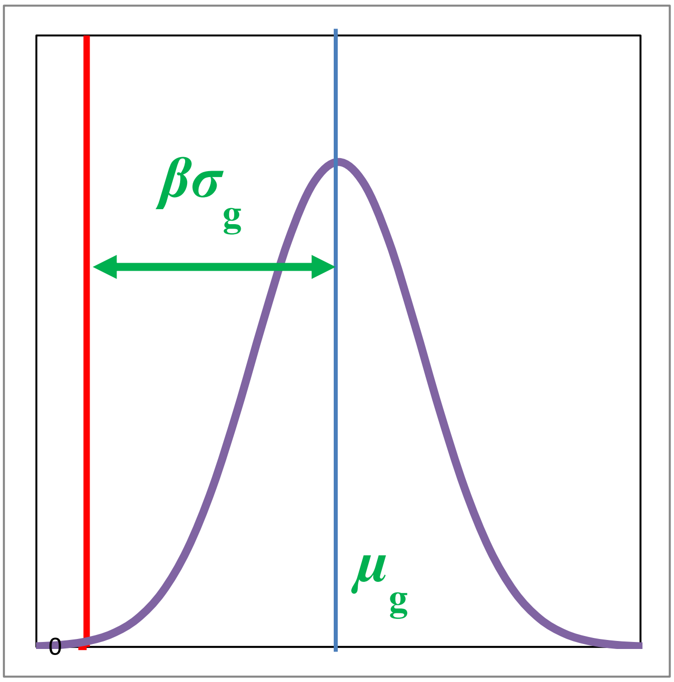


Figure 2.1 Probability density representation of the performance function g = 0

The convenience of this simplified reliability representation of structural performance is that design values can be derived for each basic variable for a given failure probability $P_f(g)$ expressed in terms of a target reliability (β_T), with Equation (2.5) giving the relationship between $P_f(g)$ and β_T where $\Phi^{-1}(\,.\,)$ is the inverse cumulative Normal function; Equation (2.6) being the expression for the design value ($x_{d,i}$) for basic variable X_i or the partial factor (γ_i) that can applied to the mean value of the basic variable to obtain the design value; the factor α_i is defined as the direction cosine or sensitivity factor, indicating the contribution of basic variable X_i to the standard deviation of the performance function (σ_g). Note that the sign of α_i is important, determining whether the partial factor $\gamma_i < 1$ or $\gamma_i > 1$, or whether the mean value needs μ_i to be reduced or increased to obtain the design value $x_{d,i}$. A notable feature of the sensitivity factor is given by Equation (2.6d).

$$\beta_T \; = \; \Phi^{-1}(P_f(g)) \tag{2.5}$$

$$x_{d,} \; = \; \mu_i - \alpha_i\beta_T\sigma_i/\mu_i \quad = \; \mu_i(1 - \alpha_i\beta_T V_i) \quad = \; \gamma_i\mu_i \tag{2.6a}$$

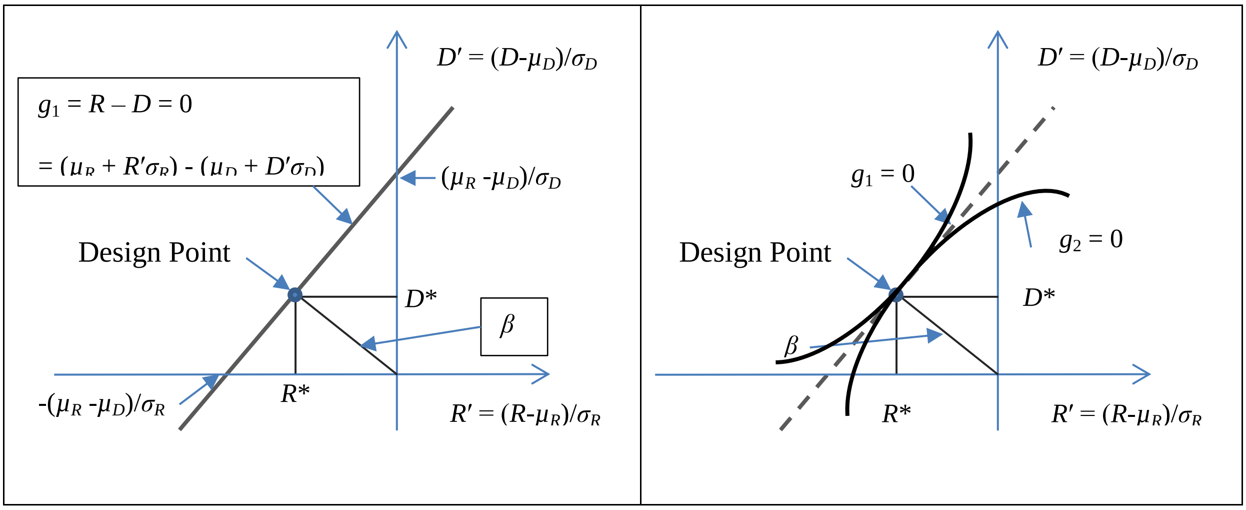
$$\gamma_i \quad = \; 1 - \alpha_i\beta_T V_i \tag{2.6b}$$

$$\alpha_i \; = \; a_i\sigma_i/\sigma_g \tag{2.6c}$$

$$(\alpha_1)^2 + (\alpha_2)^2 + \ldots + (\alpha_n)^2 \; = \; 1 \tag{2.6d}$$

The n-dimensional vector of design values $\{x_{d,i}\}$ provides the coordinates of the design point in an n-dimensional hyper-space on which the distribution of the n basic variables are represented. The design point represents the most likely failure point, expressed as the distance in units of σ_g of a failure or limit surface from the origin of the hyper-space taken at $\{\mu_i\}$. A graphical 2-dimensional representation of a performance function in terms of structural resistance (R) and actions or demand (D) is given in Figure 2.2. From the geometry the value for β can be derived to be:

$$\beta = \frac{\mu_R - \mu_D}{\sqrt{(\sigma_R)^2 + (\sigma_D)^2}} \tag{2.7}$$



(a) Linear Performance Function

$g = R - D$

(b) Non-Linear Performance Functions

$g_1 = g_1(R; D)$; $g_2 = g_2(R; D)$

Figure 2.2 Geometrical representation of 2-dimensional performance function

By considering the solution of the performance function $g(.)$ only at the design point, it is possible to obtain an approximate solution for the case of a non-linear function. It can be shown that such a solution is obtained when the terms $a_i\sigma_i$ in Equation (2.2) are replaced by the partial derivatives of $g(.)$ with respect to each basic variable X_i in turn at the design point, as given by Equation (2.8); using the notation ()* to indicate design point values. Since the design point coordinates are not known, the solution can only be obtained iteratively, solving for both β and $\{x_{d,i}\}$ or $\{X_i^*\}$.

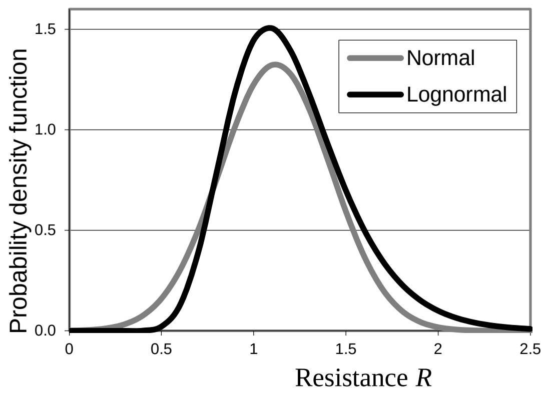
$$\sigma_g{}^2 = \left[\left(\frac{\delta g}{\delta X_1}\right)^* \sigma_1\right]^2 + \left[\left(\frac{\delta g}{\delta X_2}\right)^* \sigma_2\right]^2 + \ldots . + \left[\left(\frac{\delta g}{\delta X_n}\right)^* \sigma_n\right]^2 \tag{2.8}$$

The iterative numerical solution of $g(.)$ allows for solving non-analytical functions by using numerical differentiation of the algorithm or program representing $g(.)$. Another innovation is to obtain approximate solutions in cases where non-Normal distributions are used to model the basic variables. This can be done by replacing the general distributions with probability density and cumulative functions $f_x(X)$ and $F_x(X)$ by equivalent Normal distributions at the design point. The parameters of the equivalent Normal distribution $\{\mu_i^N; \sigma_i^N\}$ can be derived by equating the respective probability density and cumulative probability values at the design point.

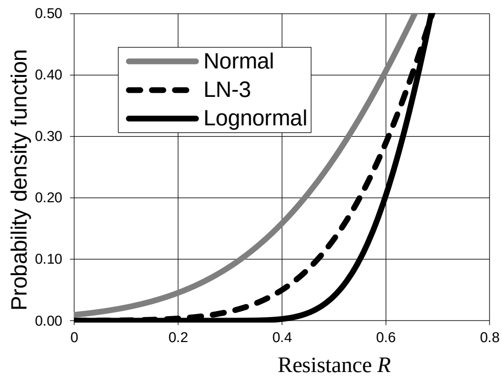
2.1.2 Basic variables

Input into reliability modelling is provided by the probabilistic representation of random variables that derives from either inherent variability, such as extreme environmental conditions or uncertainties associated with the given variable, such as limited statistical data on which probability models or distribution parameters are based. Model uncertainties represent another class of uncertainty that needs to be included in the reliability performance function. The level of approximation can range from using a nominal Normal distribution with parameter values based on judgement, to elaborate extreme value models based on extensive statistical treatment. Parametric assessment of model uncertainty for crack width reliability assessment is an example of nominal treatment (McLeod, Retief, Wium 2013); strong-wind models for South Africa are examples where elaborate analysis is done (Kruger, Retief, Goliger 2013a). The systematic treatment of model uncertainty and examples are provided by Holický Sykora and Retief (2014) and Holický, Retief and Sykora (2015).

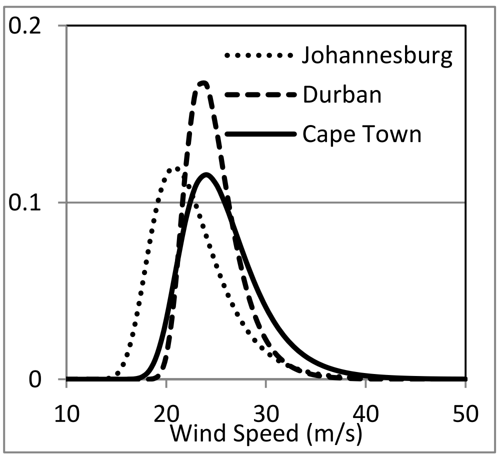
Typical probability distributions for basic variables are for material properties (*X*) or resistance (*R*) to be modelled as Lognormal distributions, variable actions (*Q*) as Extreme Value such as Gumbel distributions and permanent actions (*G*) as Normal distributions. When sufficient data is available to make a realistic estimate of the skewness of a distribution the General Lognormal (also called 3-Parameter Lognormal) provides a suitable model. Examples of typical distributions are shown in Figure 2.3, with the distribution for *R* based on parameters for pile resistance (Dithinde & Retief 2013) and for *Q* based on typical strong-wind parameters (Kruger, Retief, Goliger 2013a). The sensitivity of the distributions for the selected probability function is quite clear; nevertheless applications should be moderated by the level of approximation of standardised reliability based design procedures.



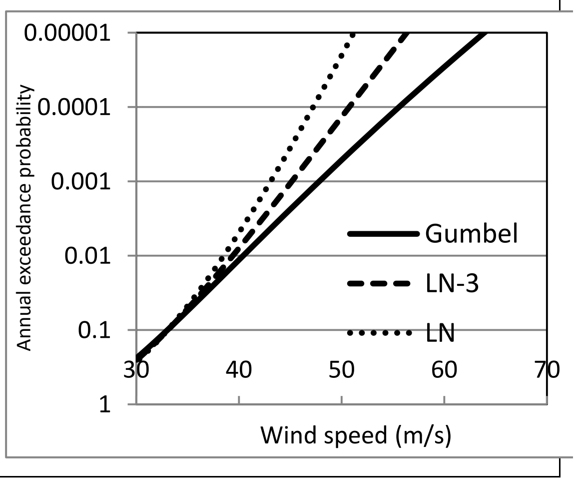
(a) Typical resistance probability density compared to Normal distribution



(b) Lower tail of resistance probability density with LN-3 accounting for skewness



(c) Gumbel Extreme Value distribution for wind speed for metropolitan areas



(d) Comparison of Gumbel to Lognormal distribution for Cape Town

Figure 2.3 Typical probabilistic density functions representing resistance (R) and variable actions (Q)

2.1.3 Target reliability

The setting of target levels of reliability for structural performance straddles the interface between societal perceptions and interests and rational technical solutions to its needs and that of the economy and the role of the construction industry. In typical pragmatic engineering fashion, target levels of reliability were initially derived from design practice that was deemed to be acceptable. In spite of an apparent case of self-referencing, this approach provided the opportunity to identify inconsistencies and to provide a (more) rational basis for improving safety and functionality. The next step was to derive target reliability levels from a process of optimisation, either on an economic basis or on a system of societal values in the case of life safety. An example of a parametric economic optimisation analysis is presented by Holický and Retief (2011).

From the extensive body of literature on setting target levels of reliability, only a summary of typical operational values are considered here. A compilation of values from ISO 2394:1998 and EN 1990:2002 is given in Table 2.1 (Retief & Dunaiski 2009a); the way in which a reference target level of reliability can be differentiated in terms of performance classes is shown in Table 2.2. The reference target level is indicative of the normal conditions provided for in standardised design, posing no special conditions in terms of consequences of failure or of unconventional costs in the construction of the structure.

The indicative nature of the values presented in Tables 2.1 and 2.2 should be noted, as well as the coarse character of the resolution of the respective classes and associated target levels, representing steps of about 10^{-1} in failure probability. Even at this level it is nevertheless superior to an implicit provision for uncertainty. There is clearly scope for more detailed analysis based on improved models and uncertainty data.

Table 2.1 Target reliability levels (β) according to ISO 2394 and EN 1990

Relative cost of safety measures	**ISO 2394 Minimum values for β**			
	Consequences of failure			
	Small	Some	Moderate	Great
High	0	1,5 (A)	2,3	3,1 (B)
Moderate	1,3	2,3	3,1 (C)	3,8 (D)
Low	2,3	3,1	3,8 (D)	4,3 (E)
A	for serviceability limit states β = 0 for reversible and β = 1,5 for irreversible states			
B	for fatigue limit states β = 2,3 to 3,1 depending on the possibility of inspection			
For ultimate limit states the safety classes:		C β = 3,1	D β = 3,8	E β = 4,3

Reliability Class	**EN 1990 Minimum values for β**					
	Ultimate LS		Fatigue		Serviceability LS	
Reference period	1 year	50 years	1 year	50 years	1 year	50 years
RC1	4,2	3,3				
RC2	4,7	3,8 (F)		1,5 to 3,8	2,9	1,5
RC3	5,2	4,3 (G)				
F	With ISO 2394 clause 4.2(b) *moderate safety costs & RC2 consequences* , but EN 1990 is more conservative; EN1990 value agrees with ISO 2394 for either *low safety cost or great consequences*					
G	The EN1990 value for RC3 agrees with ISO 2394 for *low safety cost and great consequences*					
ISO: 2,3 – 3,1	EN: 1,5 – 3,8	Fatigue:	ISO 2394 – restricted range; EN1990 – range from *serviceability LS equivalent* to *ultimate LS*			

Importantly, reliability assessment and the consequent target levels of reliability do not include provision for the effect of gross error, due to the difficulty of modelling such events. Consequently target reliability levels are often expressed as notional reliability. However, by recognising the direct link between reliability based design and quality management as the basis for controlling gross error, this qualification on the nature and relevance of reliability modelling is fading out.

Table 2.2 Differentiated target reliability levels (β) derived from various sources

PERFORMANCE CLASS	β	SOURCE
Ductile, gradual modes of failure (Reference)	**3,0**	Milford (1988; 1998) SABS 0160-1989
Brittle, sudden modes of failure	4,0	
Connection details between components	4,5	
Safety class (SC) → Reference Class	3,1	ISO 2349 EN 1990
SC – *Consequences Great* or *Cost Moderate*	3,8	
SC – *Consequences Great* and *Cost Moderate*	4,3	
Fatigue – *Inspection possible*	2,3/1,5	ISO 2349 (EN 1990 – 1,5)
Fatigue – *Inspection not possible*	3,1	ISO 2349 (EN 1990 – 3,8)
Serviceability - *Irreversible*	1,5	ISO 2349, EN 1990
Serviceability - *Reversible*	0	ISO 2349

2.1.4 Examples illustrating the use of reliability modelling

A basic semi-probabilistic design function relating structural resistance to actions on a structure is used as basis for an example to demonstrate how a performance function can be expressed as reliability model for the design function as shown in Table 2.3. A sequence of analysis is followed, starting off with the discrete design function (Step 1) which includes partial design functions which should be derived from reliability theory to achieve a target level of reliability; the probabilistic reliability function expressed in terms of basic variables is expressed as a performance function g with $g = 0$ indicating the failure limit (Step 2). A specific set of partial factors are based on typical values for target reliabilities, sensitivity factors (Step 3), theoretical expressions for partial factors as a function of the probability distribution of the basic variable (Step 4), to derive an illustrative design function (Step 5). By selecting a set of action values, the design value for resistance can be derived (Step 6) from which the probability models for the basic variables can be derived (Step 7) as input to the reliability model which can be solved using a typical iterative numerical procedure. The results (Step 8) can be obtained for example from a procedure provided by Holický (2009); deriving not only the reliability obtained for the specific case ($\beta = 3.24$, as compared to $\beta_T = 3.0$); but also comparisons between the partial factors and sensitivity factors for the specific solution and generic values.

Similar parametric analyses serve to assess various calibration considerations such as the target reliability, the contribution of the various sources of uncertainty, sensitivities for generalisations such as generic α_X-values, ratios of permanent and variable action values. From such analyses emerge the motivation for refinement of models, modelling and data; at the same time the power of semi-probabilistic design to provide a rational basis for such advancement is demonstrated.

Table 2.3 Example illustrating relationship between a design function and parameters, the reliability model in terms of basic variables

DESIGN PROCEDURES		**← CONVERSION →**		**RELIABILITY MODELLING**
1. Symbolic Design Function		3. Reliability Derivatives		2. Symbolic Performance Function
$\left(\frac{\theta_k}{\gamma_\theta}\right)\left(\frac{R_k}{\gamma_R}\right) > \gamma_G G_k + \gamma_Q Q_k$ Where: θ_k Model Factor and its partial factor γ_θ R_k; G_k; Q_k; Resistance, Permanent & Variable actions with respective partial factors γ_X		Target reliability β_T = 3.0 Sensitivity factors (generic): Resistance; actions; secondary values $\{\alpha_R; \alpha_E; \alpha_S\}$ = {0.8; -0.7; ±0.4} Basic variables X: Characteristic values (X_k) = mean value (μ_X)		$g = \theta R - (G + Q) = 0$ Basic variables: Probability distributions; Coefficient of variation V_X θ – Lognormal; V_θ = 0.1 R – Lognormal; V_R = 0.15 G – Normal; V_G = 0.1 Q – Gumbel (Extreme Value); V_Q = 0.3
5. Semi-Probabilistic Design Function	←	4. Semi-Probabilistic Design Parameters	←	
Apply generic partial factors to symbolic design function: $\left(\frac{\theta_k}{1.1}\right)\left(\frac{R_k}{1.4}\right) > 1.1G_k + 1.8Q_k$		Generic partial factors: γ_θ = EXP($\alpha_S\beta_T V_\theta$) = EXP(0.4x3.0x0.1) = **1.13** γ_R = EXP($\alpha_R\beta_T V_R$) = EXP(0.8x3.0x0.15) = **1.43** γ_G = 1 – $\alpha_S\beta_T V_G$ = 1 + 0.4x3.0x0.1 = **1.13** γ_Q = 1 – (0.45 + 0.78LN(-LN(p)))V_Q = **1.80** (where p = $\Phi(-\alpha_E\beta_T)$ = 0.9821)		A semi-probabilistic design function can be derived without solving the performance function by deriving partial factors for all basic variables from: - Target reliability & generic sensitivity factors - Theoretical expressions for partial factor for distribution functions & V_X for basic variables
6. Design Example	→	7. Probability Models	→	8. Reliability Analysis
Design Example: Let G_k = Q_k = θ_k = 1.0 From Design Function → R_k = **4.5**		Basic variables: θ – LN(μ_X; σ_X) = LN(1.0; 0.1) R – LN(4.5; 0.675) θ_R – LN(4.5; 0.81) [$V_{\theta R} = (0.15^2 + 0.1^2)^{½}$] G – N(1.0; 0.1) Q – Gu(1.0; 0.3)		see table below

$\boldsymbol{\beta_T}$ = 3.24		$\mathbf{P_F}$ = 6.0 x 10^{-4}	
	$\boldsymbol{X^*}$	$\boldsymbol{\gamma_Q = X^*/\mu_X}$	$\boldsymbol{\alpha_X}$
$\boldsymbol{\theta_R}$	3.06	1.47	0.64
G	1.04	1.04	-0.12
Q	2.02	2.02	-0.76

2.1.5 Features and utility of reliability representation

The primary features of the reliability representation of structural performance is the ability to combine all sources of variability and uncertainty into one function for analysis to obtain the combined effect; furthermore the process can be extended to derive a semi-probabilistic version of the function for which design parameters can be derived to obtain a pre-specified or target level of reliability. However, this is only the starting point for more refined analysis, both (i) to improve the background reliability data, models, even critically assess the structural mechanics models; (ii) enhance the decision making process associated with standards development and the effectiveness of the ultimate application of the standards.

A key characteristic of reliability analysis solutions is the relative stability of the sensitivity factors (α_X) across a wide scope of conditions that may influence the relative contributions of various sources of uncertainty, resulting in stable solutions for the design parameters such as partial factors (γ), as given by Equation (2.6). Furthermore, these solutions remain stable even for values not exactly on the design point, but still satisfying the performance function $g(\,.\,) = 0$. The result is that it is possible to derive a comparatively simple set of design parameters to satisfy the relatively coarse set of target levels of reliability across a wide scope of application.

An important feature of the nature of a reliability approach that is almost taken for granted is the ability to generically ‘partialise’ provisions for actions and resistance not only into separate procedures, but even into separate standards altogether, yet achieving reasonably stable solution for a wide scope of design conditions, structural classes, sets of actions, structural materials, even failure modes. Such separate treatment on the other hand necessitates the proper formulation of a common basis of structural design, calibrated on the principles of reliability for any set of standards applying to a given set of structures, such as a country or region.

A downside of the relative leniency allowed by reliability procedures is the diversity of reliability based approaches and lack of standardisation used by various groups, whilst similar performance levels are effectively achieved. Unless background information is properly recorded and published, it is not possible to share, exchange or compare the data, knowledge and experience which is in short supply for the enhancement of design procedures.

2.2 Design Application of Reliability

Converting the theoretical reliability concepts into operational design procedures requires the representation of a collection of structural mechanics models for all situations that can be identified through experience based expertise to be sufficiently representative and reasonable to provide for the load bearing performance of the structure during its service life. One way in which to represent the process is to consider the three main elements of a standardised design procedure:

(i) The structural mechanics model(s) (see Equation (2.1)) on which semi-probabilistic design procedures can be based (see example of Section 2.1.4);
(ii) The reliability models, including the associated models for the basic variables from which calibrated partial factors and characteristic variables can be specified;
(iii) A scheme of conditions such as limit states and design situations requiring design verification, based on expertise, judgement and experience.

This scheme is obviously biased towards a reliability based perspective, but is presented here simply to counterbalance the emphasis often placed on the other two elements in both standards development and use; thereby missing the opportunities for engineering improved structural performance by the enhanced application of the principles of structural reliability. Two classes of design measures can consequently be identified:

(i) Quantitative design verification measures, consisting of a reliability based verification of the structural mechanics models to describe load generating and resistance effects;
(ii) A management scheme to define requirements for structural performance and systematic procedures for defining the limit states and design situations for compliance verification.

The characterisation of these classes of reliability based design measures are presented in this section, with the addition of a third class to represent considerations falling outside the strictly technical scope of a given standard, serving as input to its development and context for its use:

(1) **Reliability kernel:** The way in which the reliability performance for a given limit state, design situation, failure mode, consequence class and associated target level of reliability is verified, serves as a *reliability kernel* on which the overall design process is built.
(2) **Reliability Framework:** A multi-dimensional scheme is devised, to provide for all the conditions within the scope of the standard to which a structure may be exposed during its service life, for which a class of consequences can be associated in case of failure.
(3) **Meta-Standard Scheme:** All the considerations that establish the function, purpose, methodology, reference source of technology and experience, relations to related standards can be classified as meta-standard requirements, input and background to the standard.

2.2.1 Reliability kernel

From the theory of structural reliability the following components or steps provide an outline of the reliability kernel on which any semi-probabilistic design procedure should be based, including a nominal discussion of some important features of each component:

(i) **The structural mechanics model(s)** required for relating actions on the structure to its load bearing performance or resistance. Conventionally the performance function relates combined action effects to resistance in terms of element or sectional forces for a given failure mode. Although this includes the structural analysis for the integral structure, this topic is treated conventionally in the general requirements of standards, considered to be part of the structural engineering input. With the mounting prominence given to accidental actions and structural robustness, structural analysis could serve as a more formal link between element based design and system failure.

(ii) **The reliability function** describes the performance function as a probabilistic function in terms of the basic variables consisting of random models. The basic variables are classified as actions, materials and geometry; various levels of model uncertainty are treated as random model factors. The bias of specified basic variables is reflected by the ratio of mean to characteristic values. The reliability function is used for the calibration of all the design parameters such as partial action, material and resistance factors.

(iii) **The design function** provides a semi-probabilistic version of the structural mechanics function with the addition of design parameters, calibrated to achieve or exceed a pre-specified or target level of reliability.

2.2.2 Reliability framework

The specific set of cases and situations for which a design function, as derived from the reliability kernel outlined above, needs to be specified is formulated in the reliability framework. The principle on which these cases are based is that the *selected design situations shall be sufficiently severe and varied so as to encompass all conditions that can reasonably be foreseen to occur during the executions and use of the structure* (SANS 10160-1 clause 5.1.2). The recent formulation of the principles of structural reliability presented in ISO 2394:2015 establishes risk to serve as basis for reliability based design, implying that the consequences of failure also need to be taken into account. Although consequences of failure at the various limit states are implicitly reflected in standardised design, they should now be recognised more formally in the designation of selected design situations to which reliability kernel based procedures should be applied.

The reliability framework can be considered to be built up in the following stages:

(i) **A reference case** is formulated as standard, from which adjustments are / can be made depending on deviations, including the following elements:
- A reference target reliability for ultimate limit state ductile failure;

- A reference class of structure, typically one for which general experience in design and construction is available and for which no exceptional risks are posed or performance levels are required.

(ii) **Reliability classes** are defined, allowing for adjustments as needed:
- Reliability classes are defined in very general terms, considering life safety and environmental impact.
- More appropriate and detailed guidance is obtained for consequence classes on which robustness procedures are based.

(iii) **Limit states** to reflect the safety (ultimate) and functionality (serviceability) of the structure, with further subdivisions of design situations:
- Ultimate limit state in terms of time related conditions: Transient, Persistent, Accidental;
- Serviceability in terms of Irreversible, Reversible, Long-term, Appearance

(iv) **Associated failure modes, expressed as design situations** serve as operational rule for the principle that *sufficiently severe and varied conditions* need to be considered; classified into the degree of warning of imminent failure:
- Failure type, such as ductile, brittle, fatigue, equilibrium.

(v) **General requirements and prerequisites** include specifically competence and experience to the levels of specialisation for given classes of structure.

2.2.3 Meta-Standard scheme

Standards development has the appearance of a well organised and properly specified character; considering all the components, levels of refinement and associated variability and uncertainty. However, additional information relevant to the function and use of a standard but not included in the normative clauses or informative annexes can be classified as meta-standard information. Formal documentation may include official acknowledgement to define its regulatory or compulsory status or professional status; background documentation to record the technology basis for the standard; commentaries to provide guidance on its intentions, interpretation and use; the relationship between the standard and related standards may be specified internally, sometimes complemented by external guidance; in a similar manner the scope of application may straddle the interface with the meta-standard scheme, but clear definition of limits of application and the basis for extending its use to conditions where additional expertise in design and construction is not often clearly specified; ownership of the standard is important to define responsibility for the maintenance and advancement of a standard or a specific field of standardisation, with the more general topics such as actions on structures or the basis of design most problematical in this regard.

Meta-standard information is particularly critical to define the objectives for the development of a new standard, even for the revision of an existing standard. Even where formal committees exist under the auspices of a national standard body, the function of these committees arguably tend to focus on matters being proposed and presented for authorisation, rather than taking ownership responsibilities. The most important element for a successful process is the need for ownership; serving as basis for leadership and generating resources, particularly when new initiatives are launched or substantial upgrades of existing standards are envisaged; see for instance Retief and Wium (2012) and Wium, Retief and Viljoen (2014).

Two specific initiatives for the development of South African standards with which the Candidate was involved, illustrate the role of meta-standard considerations that were vital to the standards development process: The revision of the SA Loading Code SABS 0160:1989 into SANS 10160:2010 was initiated by the South African National Conference on Loading in 1998, with the formulation of the objectives for the revised standard based on the conclusions of the conference, as narrated by Day (2013) and referred to by Retief and Dunaiski (2013). The basis for the development of a new standard for the design of water retaining structures, as sponsored by the Water Resources Commission, is reported by Barnardo-Viljoen, Mensah et al (2014); in addition to the development of the standard through a representative working group in the conventional manner, provision is made for supporting research, gathering of background information for decision making and technical management of working group activities.

A few specific topics that illustrate the concept of meta-standard considerations are the general basis on which semi-probabilistic standards should be based; the way in which reliability concepts should serve as common basis for related standards; the relationship between reliability classes and quality management:

Qualifier for semi-probabilistic standards: The general principles for the use of semi-probabilistic design can be derived from ISO 2394:2015 to apply to *structures for which the consequences of failure and damage are well understood and the failure modes can be categorized and modelled in a standardized manner* (clause 4.4.3). Furthermore the basic principles for *semi-probabilistic safety formats shall comprise* (of) *consequence class categorisations; design situations* (Clause 9.2). This strict qualifier for semi-probabilistic design procedures and standards puts a constraint on the scope of application of such standards, requiring sufficient experience to categorically expect failure consequences to be limited to the levels on which reliability assessment is based. Similar strict requirements also apply to the competence and experience base of designers and constructors.

Common reliability basis of design: Provisions for the basis of design as derived from the principles of reliability extends beyond the scope of any specific standard providing for only one component of the design of a given structure or class of structures; therefore the contributions from related standards can be considered to be meta-standard information. The way in which the common reliability based procedures for structural performance as defined in SANS 10160-1 applies to the various standards stipulating actions and materials-based design for structural resistance is shown schematically in Figure 2.4.

BASIS of STRUCTURAL DESIGN	
GENERAL	- Requirements – Competence & Experience - Reliability framework – Limit states & Design situations - Verification procedures – Quantitative design expressions

ACTIONS Reliability Provisions	**RESISTANCE** Material-Independent Provisions
GENERAL - Classification of actions - Basic variables - Limit states & Design situations - Combination schemes - Partial factors	- Multiple failure modes, provision for - Ductile/brittle failure differentiation - Material properties as basic variables - Alternative design formats - Reliability class adjustment

ACTION STANDARDS Generic Procedures per Standard	**MATERIALS-BASED STANDARDS** Generic Procedures (Based on Concrete)
- Prediction models → characteristic values - Action classes · Permanent – self-weight · Variable actions – environmental: Wind, Temperature · Variable actions – use: Imposed, Industrial · Accidental actions – seismic	VERIFICATION - Specified material properties (characteristic values) - Partial factors RELIABILITY MANAGEMENT - Structural analysis & failure modes - Detailing, e.g. compliance with ductility assumptions Quality control for compliance

Figure 2.4 Schematic arrangement of the cascading of reliability measures throughout the set of standards applying to building and industrial structures related to SANS 10160

From an assessment of structural performance in terms of recorded failures reported by Schneider (1997) interfaces between the different elements of the design and construction process of structures are particularly vulnerable to gross error beyond the scope of the reliability models on which the scheme presented in Figure 2.4 is based. An overarching reliability basis of design is therefore vital to ensure the unified treatment of the overall design process.

The addition of a head standard SANS 10160-1 to the loading code is therefore an important instrument for the designer to ensure consistency between for example the actions on the structure, provision for structural resistance where combinations of the various construction materials may be used, geotechnical foundation design, even provisions for resistance against earthquakes; all done within the context of the emergence of ensuring the robustness of structures. Other examples illustrating the close link between reliability compliance and the detailed procedures of each standard used in a design are quality control of concrete strength or detailing of reinforcement and proper placing and assumptions about ductile behaviour of failure modes.

Quality management and related classification systems: Although provisions for quality management is now nominally included in the basis of structural design in SANS 10160-1, strictly speaking it forms part of meta-standard considerations, since it forms part of the specification of the design and execution process, rather than an inherent part of reliability compliance verification. However, due to not only the close coupling to reliability performance derived from the requirement to verify the assumptions such as competence, experience and compliance of the specifications for execution, materials, products, etc., but more directly that structural failure more often results from non-compliance of quality measurements than of insufficiency of the design procedures (see for example Schneider 1997). A two-way interrelationship between reliability based performance and quality management can be identified:

(i) The QM measures that need special attention for acceptable performance should be identified from the design process; this includes for example material properties, dimensional control, detailing of all load bearing parts.

(ii) Inversely QM should ensure that all the assumptions and premises on which design procedures rest are validated; requiring special attention during the design process to take account of situations where the structure is not sufficiently robust to deficiencies in the QM measures.

An example of the link between reliability classification and levels of quality management in SANS 10160:2010 is provided in Section 4.3.7.

Chapter 3: Reliability Modelling for Structural Design

The conversion of models for structural performance expressed in terms of reliability modelling into operational design procedures imbedded in design standards plays a central role in the material presented in this dissertation. A logical arrangement of the material would be to focus on the development of the reliability basis of structural design, the background investigations, calibration, formulation and implementation. Since this theme emerged over time, rather than being set as an objective for a properly managed program, a more general arrangement is required to capture the investigations and its progression. General investigations were initially of an exploratory nature that turned out to be useful preparation for exploiting the opportunities that came about with the need to revise the South African Loading Code SABS 0160:1989. At later stages the general investigations took on the nature of generalisation to characterise the process of the development of operational design procedures and to link that up to international practice.

A certain progression can nevertheless be distinguished, consisting of the following interrelated steps:

- **Individual Investigations:** Following a period of little activity in standards development in South Africa since the introduction of the first generation of limit states development, even less so with reliability assessment, efforts were made by the Candidate and co-workers to set up reliability models for the existing local standards for reassessment of the related structural performance.
- **International Alignment:** Such reliability modelling was subsequently extended, mainly in cooperation with Prof Milan Holický, to be done in harmonisation with international practice; with special reference to Eurocode procedures, models and conventions, when Eurocode was identified as potential reference to the revision of the South African Loading Code SABS 0160:1989.
- **Basis for Adoption of Standards:** The possibility that the local experience could be generalised as a showcase for adapting international standards to local conditions, with special reference to Eurocode, prompted the next level of investigations.
- **Harmonised Standards:** Ultimately the investigations led to the formulation of general principles of the development of reliability based standards for structural design that could feed into international harmonisation of standards development.

The related investigations are presented subsequently in the order of these steps.

3.1 General Review of the Reliability Basis of Structural Performance

The general investigations on the reliability basis of structural performance explored such topics as accounting for the developmental nature of the economy, schemes for assessing the combination of actions; the reliability performance of structural steel and concrete in terms of both the ultimate and serviceability limit states. A limited degree of capturing the experience gained concludes the preliminary and general investigations.

3.1.1 Reliability based assessment of structural performance

Early investigations on reliability assessment of structural performance represent limited studies on interesting issues that nevertheless, provided some insight and development of methodologies that were used later. Firstly a risk based assessment is done for optimal structural design under socio-economic conditions applying to a developing economy. Subsequently the reliability assessment for serviceability for structural concrete and steel respectively was explored.

The redefinition of the socio-economic fabric of South Africa prompted the question on the implications for the standards for structural design in terms of optimal levels of reliability. A model for the optimal reliability of a structure under probabilistic load presented by Kanda and Ellingwood (1991) was used to explore the implications for structural design for development projects (Retief 1996a).

Utility theory was used to model the adjusted socio-economic value system of a development project in terms of the development benefits versus the constraints on resources. On this basis an estimate of the downwards adjustment of the target reliability could be obtained. It is furthermore postulated that the acceptance of development conditions would lead to an increase in uncertainties caused by factors such as reduced skills levels and enforcement of the quality management regime. This would lead to increasing costs to achieve a given optimal reliability level.

Based on a parametric sensitivity application of optimal reliability levels and the associated load factors, it is demonstrated that the detrimental effect of increased uncertainty will exceed the savings of accepting an adjusted value system. It is concluded that within the constraints of structural design as such, well proven standard practice is optimal, even for development projects where resources are limited. These conclusions are based on a limited desktop investigation, which would require hard data to be validated. It nevertheless provides a useful benchmark when considering the range of conditions under which structural projects may be undertaken nationally, or in comparison to international practice.

Deflection control represents an important serviceability limit state requirement for the design of structural concrete. Nevertheless, reliability performance is only represented through the bias of characteristic or nominal material parameters, with little attention given to reliability modelling and calibration. This situation provided the motivation for investigating the reliability performance of representative codified deflection design procedures (Retief 1996b). The investigation included provision for model uncertainties for the immediate and long term deflection procedures, derived from comparison of measured to predicted values.

Model uncertainty was shown to be the dominating source of uncertainty, as compared to the variability of the various material parameters that were modelled as basic variables. Furthermore, the performance of alternative procedures largely results from model uncertainty probability characteristics of the respective procedures. The importance of model uncertainty turned out to be an important component of the reliability assessments and investigations reported in this dissertation. The investigation confirmed that standardised deflection control procedures result in reasonable but unspecified levels of reliability, although reliability could be controlled through proper calibration.

An innovative approach was taken in considering the serviceability assessment of steel structures by basing reliability modelling on quantitative expert measurement of basic variables (Ter Haar, Retief and Dunaiski 1998). The application of the classical hypothesis testing method to calibrate experts as described by Cooke (1991) was enhanced to provide for the measurement of uncertainty distribution parameters, including model uncertainty, from expert surveys. The investigation demonstrated how sufficient information could be obtained from an expert survey to perform a reliability assessment of the serviceability performance of a representative portal frame structure. The results obtained are shown in Table 3.1.

Table 3.1 Serviceability assessment of portal frame steel structure

Serviceability Limit State		**FORM Analysis Results**	
User Requirement	Load Condition	Reliability Index β	Failure Probability
Damage to cladding	Live Load	0.76	0.22
Damage to cladding	Wind Load	1.26	0.10
Visually objectionable	Dead Load	1.56	0.06

3.1.2 Reliability calibration methodology – Local practice

A number of investigations on the calibration of standards for structural design were reported in a series of papers that were compiled in anticipation of the revision of the South African Loading Code SABS 0160:1989. Starting off with an exercise to set up a methodology for structural code calibration (Ter Haar & Retief 2001), this methodology was applied specifically to the reliability assessment of loading procedures (Ter Haar, Retief, Kemp 2001). Two related papers addressed the reliability assessment of structural concrete resistance; firstly to derive generic resistance factors which are consistent with loading provisions (Ter Haar & Retief 2002); secondly to review the reliability performance of various concrete failure modes (Retief, Maritz, et al 2002).

The methodology presented by Ter Haar and Retief (2001) was devised to set up a logical, rational, systematic and efficient process of calibration. Provision is made for the treatment of the various classes of loads and their combinations; the generic treatment of structural resistance; alternative load combination schemes and their associated partial factors. A concise way in which design procedures can be assessed in terms of matching with reliability requirements expressed as sufficient and consistent exceedance of minimum reliability. Sample results are given in Figure 3.1 in terms of the ratio of nominal resistance (R_n) to total nominal dead and live load (D_n+ L_n) as a function of the ratio of live to total load (L_n)/(D_n+ L_n). The load ratio can also be expressed as n = L_n/D_n. The ideal load factor indicates the requirement to achieve the target reliability, against which alternative load combination schemes, shown as two linear functions, can be compared; the first function applying when n < 1 (or L_n < D_n) and the second function applying when n >1.

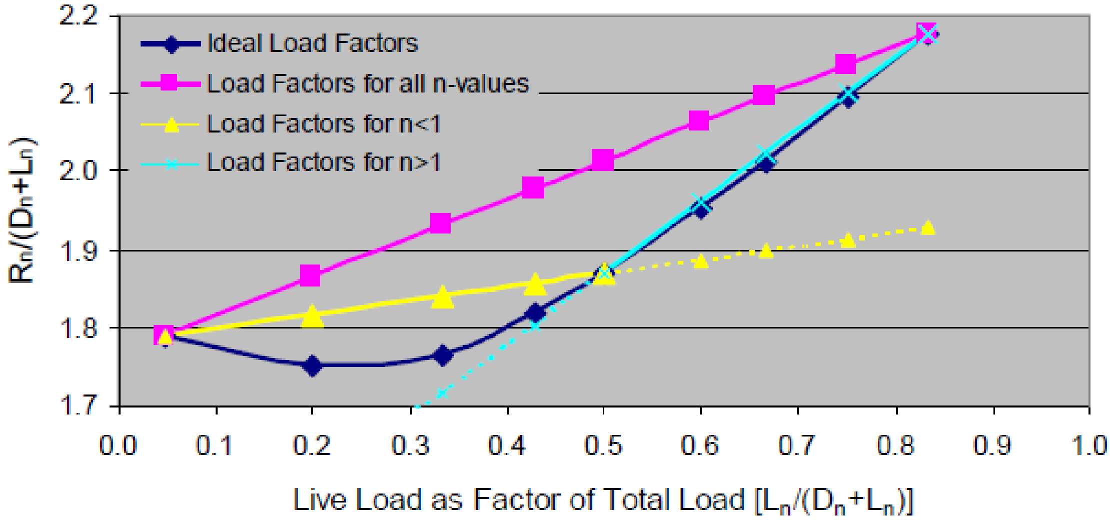


Figure 3.1 Resistance as Factor of Total Load for different Sets of Load Factors

The reliability calibration methodology was further refined and applied to assess the load factors of the South African Loading Code SABS 0160:1989, as reported by Ter Haar, Retief and Kemp (2001). By selecting a partial load factor for dead load (γ_D), the various partial factors for resistance ($\phi = 1/\gamma_R$), live load (γ_L) and wind load (γ_W) could be derived as a function of a parametrically selected resistance coefficient of variation (R_{cov}). In addition to the load models used originally for the calibration of SABS 0160:1989, the corresponding models reported for Eurocode and ASCE-7 were used to derive calibrated partial factors.

The respective sets of partial factors are listed in Table 3.2 for a target reliability index value $\beta = 3.0$ and selecting $\gamma_D = 1.2$. In spite of some significant differences in the distribution parameters for the SABS, Eurocode and ASCE models, reasonable agreement is obtained for the partial resistance factor ϕ. Reasonable agreement is obtained between SABS and ASCE for γ_L whilst differences for the values based on the Eurocode load models can be explained in terms of low values for the mean live load used in this case. Differences in γ_W can be related primarily to significant differences for the respective mean values for the wind load model. The SABS wind load reliability model appeared to be out of line with other models. It was concluded that this discrepancy required further investigation.

Table 3.2 Target partial safety factors for $\beta = 3.0$ and $\gamma_D = 1.2$

R_{cov}	Target Resistance Factor, ϕ			Target Live Load Factor, γ_L			Target Wind Load Factor, γ_W		
	SABS	EURO	ASCE	SABS	EURO	ASCE	SABS	EURO	ASCE
10%	0.75	0.75	0.75	1.65	1.39	1.74	1.10	1.36	1.70
15%	0.66	0.67	0.67	1.56	1.30	1.63	1.03	1.28	1.58
25%	0.50	0.50	0.50	1.42	1.18	1.48	0.90	1.14	1.40

The generic resistance factors derived from partial load factor calibration was further explored by Ter Haar and Retief (2002). The paper presented the manner in which the results of the loading code calibration and in particular the target resistance factors should be used as input for the calibration of the concrete code. An outline of the two stage calibration scheme is the following: Stage 1 consists of starting off with single load partial factors as the basic case; these factors are then combined in multiple sets of load combinations to derive generic resistance factors. Stage 2 uses the resistance factors as input to assigning target reliabilities to respective failure modes; then derives partial material and resistance factors as based on the various sources of uncertainty. It was anticipated that the calibration of concrete partial factors might lead to a feedback to load calibration.

In an accompanying paper Retief, Maritz et al (2002) presented a review of structural concrete reliability with reference to the South African Concrete Code SABS 0100:1992. Such a review was required considering that BS 8110, on which SABS 0100 was based, was to be replaced by Eurocode. Various stages and levels were identified for reliability performance adjustment within the limit states design approach for structural concrete design, such as setting the target level of reliability; calibration and selection of partial factors; specified characteristic and nominal basic variables; levels of quality control and its relationship to uncertainties to be taken into account.

A limited parametric reliability analysis was performed as summarised in Table 3.3. The analysis was based on a compilation of probability distribution parameters from the literature, including model uncertainties from own investigations. From the limited assessment it is clear that reliability levels are sufficiently varied to justify a comprehensive and systematic survey. Specific observations are that various basic variables play dominant roles for the respective failure modes, requiring careful optimisation for the selection of partial factors for design. Notably, previous practice does not systematically reflect increased conservatism for cases with larger variability and uncertainty. This does not reflect well on engineering judgment on which previous standards are based. This confirms the need for systematic and rational reliability assessment and calibration to ensure proper structural performance.

Table 3.3 Results of parametric reliability analysis of reinforced concrete (RC) and post-tensioned (PT) concrete elements

	Column	**RC Flexure**	**PT Flexure**
Reliability Model			
Distribution	Lognormal	Lognormal	Normal
Bias	1.10 - 1.12	1.15	1.15
CoV	12% - 14%	19%	11%
Resistance Factors Derived from Loading Code Calibration for Reliability Model			
$\phi_{Unbiased}$ $(R_{Calibrated} / R_{Expected})$	0.68 - 0.71	0.59	0.79
$\boldsymbol{\phi_{Nominal}}$ $(R_{Calibrated} / R_{Nominal})$	**0.76 - 0.79**	**0.68**	**0.90**
Nominal Partial Safety Factors			
f_y	1.04	1.03	1.03
A_s	1.00	1.01	1.01
D		**0.79**	1.01
B		1.00	1.00
f_{cu}	**0.78 (= 1/1.28)**	1.08 - 1.14	1.16
A_c	0.98		
MF	**0.92**	1.04	**0.86**
Comparison with Resistance Factors from Concrete Code			
$R_{SABS} / R_{Nominal}$	0.61 - 0.62	0.81 - 0.85	0.62
$R_{SABS} / R_{Calibrated}$	0.80	1.18 - 1.25	0.78

3.1.3 South African structural design standards and their development

The initial investigations assessing the reliability performance of structural design were based on the set of South African standards in use at the time. These standards represented the first generation of standards based on limit states procedures. Design parameters were based on principles of reliability, albeit in some cases only at the conceptual level or with limited calibration. At the time it became clear that the time was ripe for launching the next round of reliability based standards development. The investigations reported above can therefore be considered to have done some groundwork for such an effort.

Some stock taking of South African structural design standards and their development at that stage is presented by Retief, Dunaiski and Wium (2005). It is shown that the South African Loading Code SABS 0160:1989 represented the most advanced level of explicit use of reliability concepts to derive design parameters. Incidentally this standard was also fully developed locally. Conversely the materials based design codes derived from the adoption of standards of various countries, with the reliability concepts simply transferred to local practice. Unification between the reliability basis and structural performance levels for determining design loads and the respective materials based resistance verification was identified as in important requirement for the next generation of design standards.

An outline of South African experience with setting up a general basis of design in the loading code also to stipulate material independent requirements for structural resistance presented by Retief, Dunaiski and Wium (2005) is followed up by an outline of provisions that would be required for the development of an African Concrete Code (ACC) (Retief 2006). An important consideration for such a standard is that as a stand-alone standard, it would need to be self-contained in terms of defining the basis of design. Provision needs to be made to establish an appropriate level of reliability consistently across the scope of the standard. Reliability needs to be treated explicitly and transparently, however, in order to allow for adjustment to local conditions and practice across the continent.

3.2 Harmonisation of local practice

The presentation of the paper by Ter Haar & Retief (2001) at the Malta Conference *Safety, Risk and Reliability – Trends in Engineering* led to a chain of events that provided the opportunity to the Candidate to transfer local reliability assessment investigations to a level where it could be compared to international activities. The related events included participation in a JCSS Bazaar on code calibration in Malta; attendance of various JCSS activities, notably a *Workshop on Reliability Based Code Calibration* held in 2002; attendance of Eurocode WG1 as observers. Through these channels and on a cooperative basis reliability assessment of local practice could be done in a manner that would be consistent with Eurocode development practice.

3.2.1 Eurocode based assessment of local practice

A comparison between the reliability performance obtained from Eurocode and SABS 0160:1989 was motivated by the premise that limited resources for local calibration provides the motivation to consider international development in order to adapt advances into local practice (Holický & Retief 2005). Various action combination schemes allowed by Eurocode are compared to the SABS 0160 scheme for representative structural concrete failure modes. The results of one analysis are shown in Figure 3.2, comparing the reliability performance of SABS 0160 (E) with some of the Eurocode options (B & C) and an innovative option (D); expressed as the reliability index value obtained (β) as a function of the ratio of live to total load (χ).

From this analysis, a number of key observations could be made that have had a direct bearing on subsequent investigations and decisions:

- The SABS 0160:1989 action combination scheme format fully complies with the alternative formats allowed by Eurocode EN 1990:2002. Values of partial load factors differ, but the selection of appropriate values is allowed by EN 1990 as a national prerogative on safety.
- Consistency of reliability of the SABS 0160 scheme is similar to various Eurocode options; differences in absolute values can be ascribed to the partial factors as applied.

- Reliability performance levels are sensitive not only to the respective formats for action combinations, but also to the resistance failure mode and design parameters, as indicated by basic cases for structural concrete.

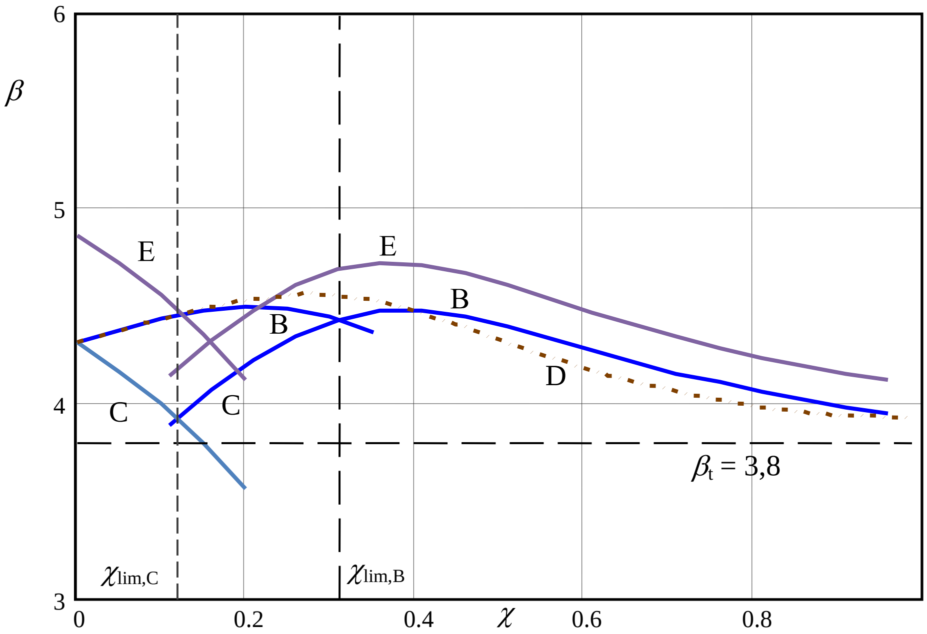


Figure 3.2 *Comparison of design scheme E (SABS 0160:1989) with other Eurocode alternatives (B, C, D) for a reinforced concrete beam having the reinforcement ratio ρ = 1%.*

The need for further assessment is confirmed by the investigation, with the objective to investigate the reliability performance of the SABS 0160 action combination scheme. The preliminary investigations indicated that attention should be given to improve both the achievement of minimum but optimal levels of reliability consistently and its interrelationship with structural resistance.

3.2.2 Structural resistance

The reliability basis of design for structural resistance was first considered in general terms by Holický, Retief and Dunaiski (2007) and then specifically for selected reinforced concrete members to provide background to a future revision of the South African Concrete Code SANS 10100-1. The sensitivities of the uncertainty representation for structural concrete resistance for various design considerations are demonstrated from the Eurocode experience: alternative failure modes and the corresponding influence of the amount of reinforcement; alternative reliability models for resistance; the influence of design procedures based on mean or characteristic material parameters. These analyses are complemented by an assessment of the influence of model uncertainty in terms of alternative failure modes; amount of reinforcement; alternative design parameters such as geometry.

Based on the results and assessments presented by Holický, Retief and Dunaiski (2007), the following themes of the resistance performance of structural concrete should be considered when formulating the reliability framework in the standardised basis of design and design procedures:

- Consideration of the various failure modes in terms of the consequences of their occurrence, and an appropriate classification system for them;
- Specification of design values for structural resistance in terms of characteristic values of material properties and geometry;
- Treatment of model uncertainty for the respective design resistance prediction procedures;
- Appropriate partial factors required to exceed reliability levels consistently and efficiently.

In conclusion Holický, Retief and Dunaiski (2007) demonstrated the utility of combined investigations based on the experience gained from the combined reliability assessment for the development of Eurocode and specific investigations done within the South African context and situation. General observations were made that the reliability calibration of structural resistance according to the principles of the basis of structural design is of equal importance to that for the specification of actions and their combinations. A systematic and rigorous process is required for this purpose. However good judgment based on capturing experience with satisfactory structural performance is still required.

The topic of the reliability performance of structural concrete resistance is pursued further by Holický Retief and Dunaiski (2007) by considering the respective roles and contributions of steel and concrete and its influence in the derived partial material design factors. The paper utilises reliability models relevant to the context of Eurocode developments which are applied within the context of South African applications. Due to the focus on local applications, further elaboration on this paper is presented in Chapter 6 on contributions to the reliability assessment of structural concrete.

Chapter 4: Development of the South African Loading Code SANS 10160

Contributions to the development of a revised South African Loading Code was approached by the Candidate essentially from the perspective of structural reliability and its application in standardised procedures for structural design. This approach provided the opportunity to be involved not only with the treatment of actions on structures, but also the overall measures taken to achieve appropriate levels of structural performance at the highest level, to ensuring the unified treatment of each component of the design process. A critical view was consequently taken of the need and function of the formal presentation of limit states design procedures in terms of reliability based requirements, the classification and specification of actions and their combinations and the material independent requirements and procedures for structural resistance reliability.

The integral view taken of the revision of SABS 0160:1989 into SANS 10160:2010 emphasised the need to capture the background information on which decisions about the revised standard were based. Perhaps more importantly, recording of the background ensures a certain level of discipline to the standardisation process. The compilation and publication of *Background to SANS 10160* by Retief and Dunaiski (Editors, 2009) was the result of this level of involvement in the development of the revised loading code. In addition to co-editing the Background Report, contributions were made to nine out of the twelve chapters, four as leading author. Further reporting on the investigations and assessments captured on the various topics is presented in subsequent sections.

In the typical pragmatic fashion of the standardisation of codes for structural design, the conversion of the comprehensive and integrally structured Eurocode set of standards into a South African Loading Code was done by serious but simple decision making processes followed by the Working Group. Nevertheless, the need to establish a proper basis for such decision making became clear to the Candidate. More than that, the need for the careful formulation of the attributes such as the stakeholders, strategic and regulatory role, technology base was identified as basis for the development of any standard for structural design, as indicated in Section 2.2.3. This realisation led to some deliberations on the *theory* of standards for structural design by the Candidate.

This chapter summarises involvement and contributions to the development of SANS 10160:2010, organised in sections on the overall approach taken, the basis of design, various actions included in the standard; finally presenting some thoughts on guidelines on the function of design codes.

4.1 Overview of the Development of SANS 10160:2010

4.1.1 Background to the Development of SANS 10160:2010

The basis for the formulation of the new South African Loading Code SANS 10160:2010 is presented by Retief and Dunaiski (2009a) in the introductory chapter of the Background Report (Retief & Dunaiski (Ed) 2009). The informative introductory sections of SANS 10160-1:2010 *Background*, *Relationship with Eurocode*, *Outline of Parts* serve as executive summary of the first chapter of the Background Report. Motivation is provided for maintaining the scope and performance levels of SABS 0160:1989; substantially overhauled the provisions for actions, including extensions relevant to its scope; extracted and adapted from the related Eurocode standards. Brief statements were made on attributes of the new standard such as its basis of design; scope of application, selection of the scope of actions; the regulatory function of the standard primarily as an instrument to the profession, rather than serving as a regulatory basis for the authorities; its relationship to Eurocode. The fact that Eurocode is formulated on the basis of member countries fully maintaining responsibility for safety (reliability) levels, represented a significant degree of freedom in the adaptation process. Another beneficial consideration was that the development of SANS 10160 ran in parallel with the finalisation of the normative (EN) version of Eurocode from the voluntary (ENV) standard, together with national implementation through the subsequent development of national annexes for all the relevant parts.

A thorough analysis of Eurocode is presented by Retief and Dunaiski (2009b) to serve as background to the adaptation of the relevant parts to South African conditions and practice. The analysis includes the presentation of the technological basis for the structural mechanics and reliability procedures and advances made in these fields; levels of harmonisation achieved across member countries and unification between the various parts of Eurocode. These attributes of Eurocode are all assessed in terms of its relevance to South Africa.

The activities to revise the Loading Code was initiated in 1999, but only picked up momentum first with a trial use of Eurocode in 2004; leading to full implementation starting in 2005. An overview of the course and scope of development is given by Dunaiski, Retief and Goliger (2006). The review records the brief formulated for the SAICE Working Group on the Revision of the SA Loading Code, provides motivation for the selection of Eurocode as primary reference standard and the approach taken for the adaptation; assesses the implications of the changes to be brought about. In conclusion it is pointed out that "although the scope of structures provided for and the general level of reliability of the current SABS 0160:1989 is maintained in SANS 10160, the provisions for the basis of design, actions, load models and procedures are substantially revised, updated and extended". Of particular relevance to this dissertation is the conclusion that "the extended reliability framework and range of design situations are resulting in an improved consistency of reliability and are allowing for reliability differentiation as well as an additional limit state for accidental design situations".

Another review of the final Loading Code, published as SANS 10160:2010 Parts 1 to 8, is presented by Retief and Dunaiski (2011). Similar background information is given on the new Loading Code as the information given previously, however commenting in this case on the final product rather than work in progress. The way in which SABS 0160:1989 and Eurocode served as reference base for SANS 10160:2010 is commented on. This review serves effectively as an executive summary of the Background Report (Retief & Dunaiski (Eds) 2009).

4.1.2 National and International Perspective

The experience gained with the development and implementation of SANS 10160:2010 provided the opportunity to consider the state of South African standards for structural design viewed from an international perspective, as reported by Dunaiski, Retief and Barnardo (2010). This review points out the importance of the South African Loading Code in providing a unified basis for the diverse set of materials based design standards as adopted from various countries, mixed with locally developed standards. Conversely the need for maintaining consistency with existing materials based standards whilst introducing a new Loading Code is indicated. The introduction of a formal part on the basis of structural design, set to maintain existing levels of reliability, serves the dual function of unified design with the various materials based standards and a common platform for structural performance.

It is concluded that SANS 10160 paves the way for the incremental adoption and adaptation of Eurocode parts as local standards. Experience gained subsequently with design standards for structural concrete could also be utilised when the future development of standards for structural steel is considered.

4.1.3 Observations on the Background to SANS 10160

In the development of standards for structural design there is an intense focus on the technical contents of the standard. Typically academic members promote advancement of the technical level of procedures; practitioners consider the implications of implementation. Decision making consists of a champion submitting an approach and procedures, to be debated and vetted by the full committee. Voting and commenting on draft versions are done in accordance with standard procedures. At the strategic level of code making the process is often done intuitively, at best based on experience. Furthermore, code development is mostly an incremental process: even if a new standard represents a substantial advancement, there is an existing standard or a defined need that serves as the default terms of reference on which decision making is based.

From the stark contrast between SABS 0160:1989 and Eurocode as the potential reference for the new SANS 10160:2010, a clear need emerged to provide a conceptual basis for how to approach the code making process. An effort was made to formulate basic principles on which decisions could be based. In many cases decisions were made in pragmatic code making fashion, from which the principle could then be derived. More important than maintaining the logical sequence of application following from formulating the principles for code making, is that such principles should ultimately be formulated and recorded. The absence of a clearly defined logical sequence for design standard development and formulation often leads to endless debate.

The integral nature of considering structural design standards from the vantage point of structural performance in terms of reliability provides an opportunity for devising the principles of code making in terms of the attributes of standards in general, as applied to any specific standard under consideration. From the background to SANS 10160 presented above emerged some rudimentary theory of structural standards. This topic will be developed further following a more technical review of contributions to the contents of SANS 10160.

4.2 Outline of SANS 10160:2010

4.2.1 Relationship between SANS 10160 Parts and Materials-based Standards

A direct relationship is intended between SANS 10160:2010 from which actions on structures should be determined and the materials-based design standards on which the resistance or load bearing properties of the structure should be based. Part 1 serves as Head Standard to provide the general limit state procedures on which design verification should be based. At the same time the basis for the specification of actions on the structure as provided by Parts 2 – 8 is provided. Provision is also made for material independent requirements for structural resistance which need to be complied with by separate material based design standards for the range of structural materials. Part 4 represents a special case requiring the integral treatment of seismic action and material-based structural response. Part 5 on geotechnical design similarly requires special treatment to relate actions and material characteristics; in addition serving to enable the use of Eurocode procedures consistently with SANS 10160. Figure 4.1 presents a schematic arrangement of the collection of South African standards for the design of building structures.

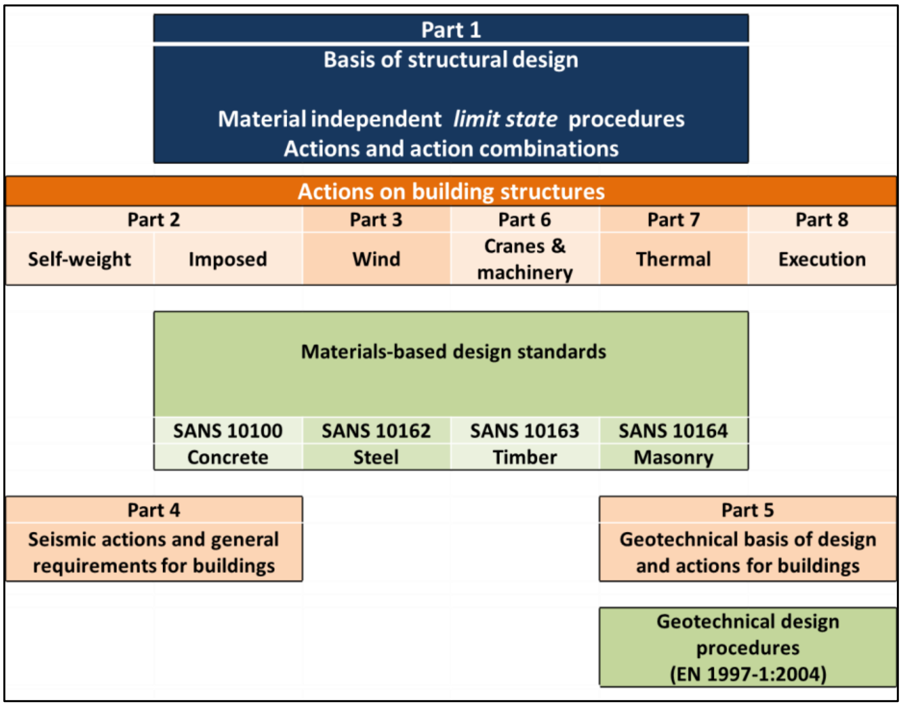


Figure 4.1 Collection of South African standards for building structures; Part 1 serving as Head Standard for both actions (Parts 2-8) and resistance given by separate standards for structural materials

4.2.2 Attributes of standards for structural design

Structural design standards usually develop through an iterative process of updating and improving a current standard for which substantial experience has been gathered. When a new generation of standards are introduced however, it is necessary to consider the function of the standard, and consequently the objective with its development. Such requirements applied to the formulation of SANS 10160:2010. The following attributes of a structural design standard have a decisive influence on its formulation:

- Regulatory function in setting safety requirements by authorities
- Statement of acceptable design practice as expressed by the profession
- Role and function of the specific standard in relation to other design standards
- Scope of application of structures provided for
- Scope of contents of design procedures included (comprehensive versus selective; standard practice versus advanced procedures)

These attributes are determined by the primary sponsors of the structural design standard, who take responsibility for its development, use and maintenance. Such ownership is traditionally taken by regulatory authorities, industry groups or the engineering profession.

Responsibility for the South African Loading Code is taken by the engineering profession, with some support given by industry groups for the various materials-based design standards.

4.2.3 Reference to SABS 0160:1989

The reference base of SABS 0160:1989 is essentially maintained in terms of its role and function in structural design practice as follows:

- **Scope of structures:** Buildings and similar industrial structures; including buildings with crane support structures as an important class of industrial buildings.
- **Design verification method:** The use of reliability-based *partial factor Limit States Design* (pfLSD) procedures is maintained.
- **Range of loads:** Provision for self-weight; imposed loads for floors, roofs and partitions; wind loads; seismic loads and design were maintained from SABS 0160, with updates to incorporate recent developments.
- **Scope of procedures:** The procedures are primarily directed towards general design practice for standard structures.
- **Materials-based standards:** Consistency with current materials-based standards had to be maintained. The onus is placed on standards still using allowable stress design to make the necessary adaptations for using the revised reliability based limit states procedures.
- **Reference level of reliability:** The present level of reliability is judged to be appropriate due to the absence of any evidence that it is insufficient (Milford, 1988, 1998), is found to be similar to North American practice and provides the basis for maintaining consistency with materials-based standards.

A number of deficiencies in SABS 0160 were identified at the 1998 South African National Conference on Loading, requiring particular attention during the revision process:

- **Wind loads:** The SABS 0160 procedures for wind loads are based on outdated models that required substantial revision.
- **Seismic actions and design:** The seismic design procedures had no credibility amongst designers in the seismic regions of the country, requiring critical re-evaluation.
- **Geotechnical design:** There is substantial inconsistency between structural and geotechnical design practice in the design of foundations and other earth-retaining structural components of buildings and industrial structures.
- **Technology base:** Although there is an extensive experience base for structural design, constraints on resources limit the systematic capturing of such experience. Similarly, research capacity is limited to the investigation of specific topics, rather than the comprehensive development and calibration required for code development.

4.2.4 Technology base and principles for reference to Eurocode

Reference to Eurocode (CE 2002) was identified not only as remedy to the deficiencies identified in SABS 0160, but also as potential technology base for a revised South African Loading Code and beyond. The advantages of using Eurocode as technology base for SANS 10160:2010 include the following:

- **Advanced standard:** Eurocode represents the compilation of a set of standards that incorporates the most advanced procedures from its member states, supported by extensive research over several decades. The advances include, for example, the introduction of a head standard to define a common reliability-based basis of design, advances in structural fire design and provision for advanced materials such as high performance concrete.
- **International harmonisation:** A high degree of harmonisation has been achieved, whilst remaining deficiencies can be clearly identified and assessed.
- **Comprehensive standards:** The scope of application is comprehensive in terms of structures, materials, conditions and relevant procedures. Internal consistency in design is achieved across the range of structures, from buildings through bridges, reservoirs, towers etc.; structural steel to geotechnical design; from self-weight to earthquake loads.
- **Range of conditions:** Environmental conditions range from the cold Nordic countries to the Mediterranean; institutional conditions range from member states where design standards are part of the law to situations similar to that in South Africa.
- **Selection of Eurocode Parts:** All Eurocode Parts relevant to the combination of the scope of buildings and SABS 0160:1989 were considered. This implied the extension of the SABS 0160 scope and consideration of nine Parts from EN 1990, EN 1991, EN 1997 & EN 1998. Only the sections and procedures relevant to the scope of SANS 10160:2010 were however utilised.

The principles followed in referencing SANS 10160:2010 to Eurocode consisted of the following:

- **Consistency with Eurocode:** Full consistency with Eurocode is maintained,
  - Providing for incremental extension of SANS 10160:2010 or the introduction of other standards from Eurocode.
- **Format, layout and style:** SANS 10160:2010 is compiled into the format of South African standards,
  - Including a compact layout; as opposed to the elaborate Eurocode formulation to allow for NDP options with a separate National Annex.
- **Reliability levels:** Due to the wide tolerances of reliability allowed by the NDP options,
  - The current reliability levels could be maintained for SANS 10160:2010, whilst achieving consistency with Eurocode within the restricted scope of application.
- **Standard level of practice:** Advanced procedures from Eurocode were considered to be beyond the scope of SANS 10160:2010; in a few cases procedures taken over from Eurocode were simplified;
  - Sufficient consistency was, however, maintained to allow for the use of advanced Eurocode procedures locally by specialists (e.g. dynamic effects of wind loads).
- **Provision for local conditions:** The general Eurocode procedures use local environmental conditions to determine appropriate representative values for wind, temperatures and seismic ground movement.

4.3 SANS 10160-1 Basis of Structural Design

The fundamental and integral role of structural reliability in the formulation of modern structural design standards forms a prominent theme in the contributions presented in this dissertation. This role is explicitly captured in the basis of structural design which is formulated in accordance with the principles of structural reliability. The formal introduction of SANS 10160:2010 Part 1 *Basis of structural design* as head standard of the South African Loading Code therefore provides a focus point of the dissertation. In addition to taking Eurocode EN 1990 as the reference standard for Part 1, the background to EN 1990 was critically reviewed to consider the scope and level of local implementation. On the one hand EN 1990 demonstrated how reliability concepts could be used to harmonise diverse national design practices from member states and to unify design procedures across diverse action classes, structural materials and structure types. On the other hand the complexities required for accommodating national jurisdiction on safety and the comprehensive nature of Eurocode, providing for all structures for buildings and civil engineering works demanded substantial scaling down in adaptation of the basis of design to local conditions and needs.

4.3.1 Review of Eurocode basis of structural design

The review of EN 1990 is reported as part of the overall review of Eurocode by Retief, Dunaiski and Holický (2009). A schematic representation of EN 1990 in the ten Eurocode standards, consisting of 58 individual parts, is shown in Figure 4.2. In addition to formulating the partial factors limit state design principles and procedures for design verification applications, with special reference to actions and their combinations, EN 1990 also stipulates material independent requirements for structural resistance, serving as requirements for formulating the basis of design procedures in the respective materials based standards. Formally EN 1990 is based of ISO 2394:1998 *General Principles on Reliability for Structures*. Extensive contributions are also made by the Joint Committee on Structural Safety (JCSS), such as the Probabilistic Model Code (JCSS-PMC 2001) and *BACKGROUND DOCUMENTATION Eurocode 1 (ENV 1991)* (ECCS 1996), Gulvanessian, Calgaro and Holický (2002). This body of information represents an important, if not decisive component of the technology basis of Eurocode.

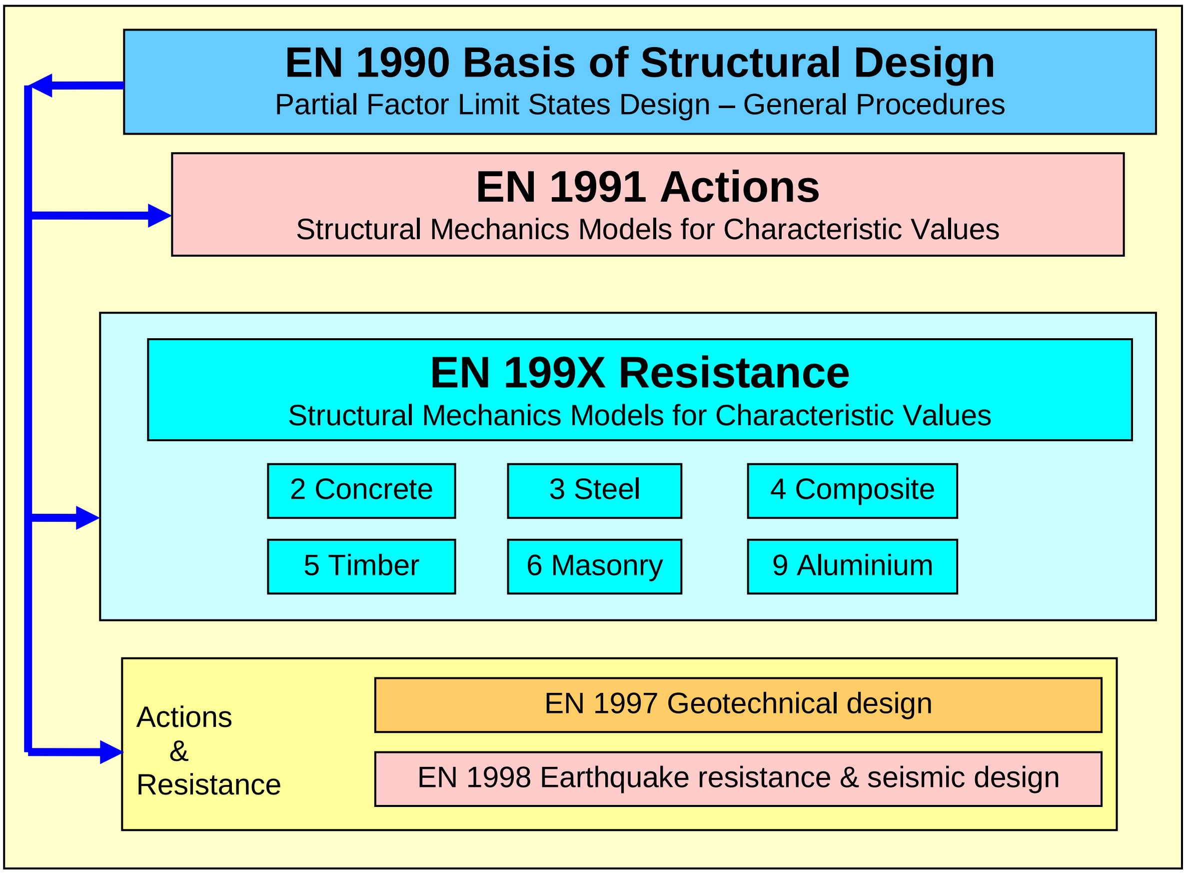


Figure 4.2 Arrangement of Eurocode Standards indicating compliance of EN 1991 – 1999 to EN 1990

The assessment of the reliability basis of Eurocode benefitted from the review of this topic by Holický, Retief and Dunaiski (2009). This review demonstrates how the principles of reliability as captured by ISO 2394:1998 taken as point of departure, are converted into operational procedures for design verification. It is concluded that the options for procedures, design parameters and specification of basic variables identified as Nationally Determined Parameters for Eurocode member states provides sufficient freedom to formulate a South African basis of structural design in terms of local conditions, practice and preferences that is nevertheless consistent with Eurocode. Due to the fundamental role of the basis of design in achieving sufficient reliability of structural performance, consistency with other Eurocode standards can be achieved also for the specification of actions on structures; in principle also for materials based standards.

4.3.2 Reliability assessment of SANS 10160 Part 1

The rationale for the basis of structural design given in Part 1 of the South African Loading Code SANS 10160:2010 in terms of the principles of structural reliability is presented by Retief and Dunaiski (2009). The concept of standardising *a common basis for defining design rules relevant to the construction and use of the wide majority of buildings and civil engineering works, whatever the nature or combination of the materials used* is formalised with the presentation of the International Standard ISO 2394 *General principles on the reliability for structures* (SANS 2004). In comparison to SABS 0160:1989, in SANS 10160-1 provision is made for a more elaborate reliability framework in terms of design situations, differentiated limit states for both the generic ultimate and serviceability limit states; the wider diversity of the extended range of actions which are stipulated, and extension of materials provided for, with specific reference to geotechnical design. The basis of structural design also provides the underlying principles on which harmonisation with international practice is achieved. The stipulations of Part 1 are therefore also assessed from the perspective of its contribution to international harmonisation of structural design practice.

The general objectives with the revision of the SABS 0160-1989 procedures in terms of partial factor based Limit States Design (pfLS-D) that have been considered include the following:

- **Updating reliability based procedures:** Considering the more extensive development of reliability based limit states design as standardised in ISO 2394-1998 and applied in EN 1990-2002
- **Improved performance:** Utilise the potential of pfLS-D in improving the performance of structures in terms of safety (achieving sufficient reliability) and economy (removing unnecessary/inefficient conservatism)
- **Reliability framework:** Taking account of the extended reliability framework presented in the basis of design, as derived from EN 1990
- **Consistency of reliability:** Therefore achieving the general objective of improving the consistency of reliability across the range of design situations within the scope of the revised standard
- **Array of actions:** Providing for the extended array of actions that can be specified due to the clear formulation of design situations within the reliability framework; considering the particulars of the respective actions for which stipulations are provided
- **Structural resistance:** Strengthening the provisions and requirements for structural resistance in terms of the application of action combination schemes for the array of structural materials and their failure modes
- **Geotechnical design:** Considering the specific and unique requirements of geotechnical design and the treatment of geotechnical actions, to be consistent with the pfLS-D procedures used in the standard

Salient features of Part 1 that are assessed and motivated by Retief and Dunaiski (2009) include the following:

- Motivation for maintaining the reference level of reliability indicated by SABS 0160:1989
- Modification of the SABS 0160:1989 action combination scheme to improve consistency of reliability; justification of consistency with Eurocode as an interpretation of one of the NDP options; comparisons with alternative Eurocode schemes
- Adjustment of Eurocode reliability classes; alignment of various classification systems such as for quality management; robustness consequence classes, seismic importance classes; geotechnical categories
- Calibration of partial load factors; adjustment for reliability classes
- Provisions for the inclusion of the geotechnical basis of design and actions for buildings

4.3.3 Compatibility of action combination scheme with Eurocode

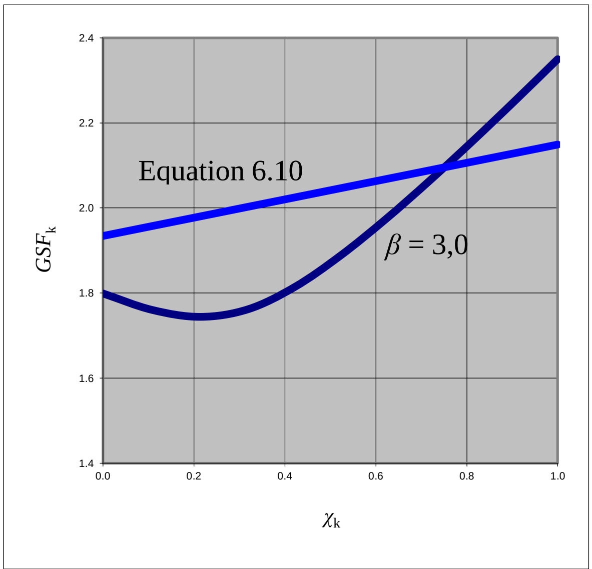
Compatibility with Eurocode action combination schemes was identified as a critical issue and constraint regarding the implementation of Eurocode in South Africa (Day 2013). Following an assessment of the alternative schemes allowed by EN 1990, the SABS 0160 scheme is then compared in terms of its acceptability and performance. Finally the revised scheme used in SANS 10160-1 is assessed for Eurocode compatibility and reliability performance.

The reliability performance of EN 1990 Expression 6.10 (see Table 4.1) is given in Figure 4.3 for a representative case based on a reliability model for imposed office floor load, typically used in reliability calibration. The resistance factor is set in accordance with the EN 1990 procedure of setting $\beta_{T,R} = \alpha_R\beta_T$ = 0,8 x 3,0 = 2,4 or $p_{f,R}$ = 0,8*10^{-2} which requires γ_R = 1,43.

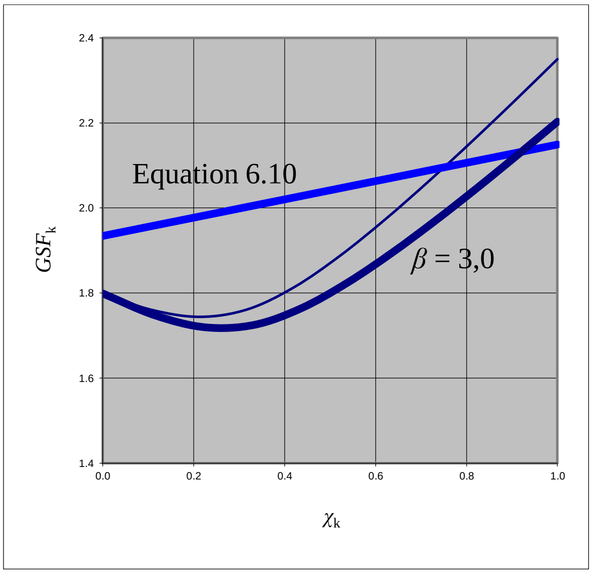
As far as satisfying the reliability requirements, Expression 6.10 generally achieves this objective across the practical range of design conditions. As shown in Figure 4.3(a), a large degree of conservatism is however achieved at the lower practical ranges of χ_k and decreasing as χ_k increases towards the crossover value at χ_k > 0,75 (Q_k > 3G_k) where the reliability is insufficient.

Table 4.1 Expressions for combination of permanent (G_k) and variable (Q_k) actions)

Standard	Expression #	Expression Simplified for Single Variable Action
SABS 0160	4 (e)	$1{,}5D_n$
	4 (f)	$1{,}2D_n + 1{,}6Q_k$ (Imposed) $1{,}2D_n + 1{,}3Q_k$ (Wind)
EN 1990	6.10	$1{,}35G_k + 1{,}5Q_k$ $(\gamma_G G_k + \gamma_Q Q_k)$
	6.10 (a) 6.10 (b)	$1{,}35G_k + 1{,}5\psi_0 Q_k = 1{,}35G_k + 1{,}05Q_k$ $(\psi_0 = 0{,}7\ \text{typically})$ $1{,}35\xi G_k + 1{,}5Q_k = 1{,}15G_k + 1{,}5Q_k$ $(\xi = 0{,}85)$
	6.10 (b UK)	$1{,}35\xi G_k + 1{,}5Q_k = 1{,}25G_k + 1{,}5Q_k$ $(\xi = 0{,}925)$
	6.10 (a-mod)	$\gamma_G G_k$
ASCE-7	Clause 2.3.2-1	$1{,}4D_n$
	Clause 2.3.2-2	$1{,}2D_n + 1{,}6Q_k$



(a) Nominal characteristic bias for Q_k (k_Q = 0,96)



(b) Increased characteristic bias for Q_k (k_Q = 0,90)

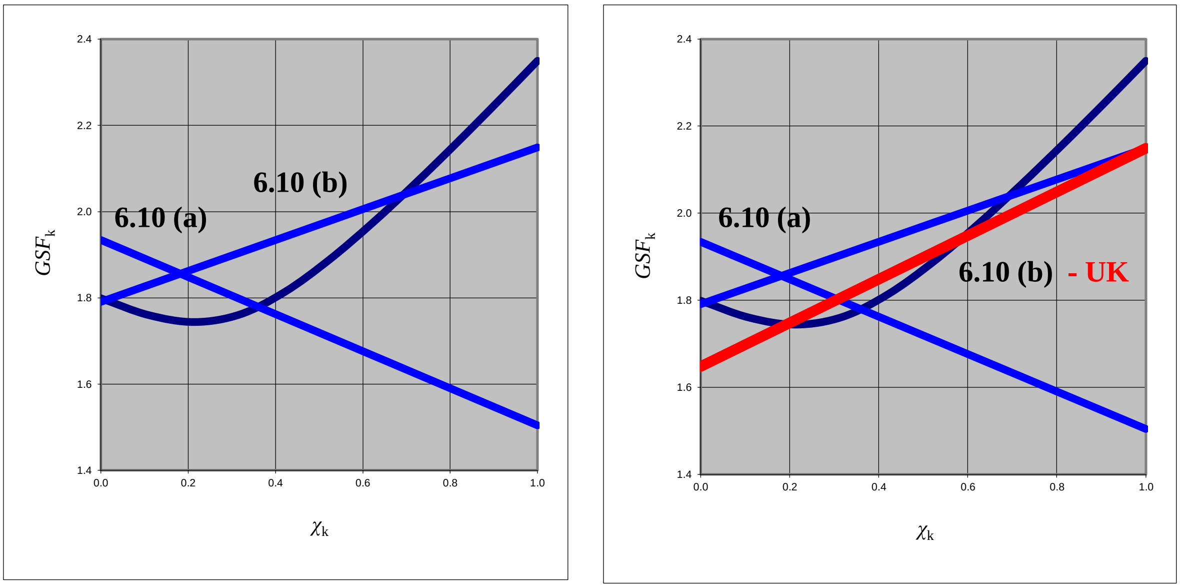
Figure 4.3 Reliability compliance of EN 1990 Expression 6.10

The effect of a more conservative characteristic specification of the characteristic variable action Q_k, is shown in Figure 4.3(b). Although the formal specification of Q_k in EN 1990 is that it is nominally the expected lifetime maximum value, the general conservatism applied in Eurocode will likely be closer to that presented in Figure 4.3(b) than the case used in this assessment as shown in Figure 4.3(a).

The comparison of EN 1990 Expressions 6.10 (a) & (b) are shown in Figure 4.4(a), with the UK modification shown in Figure 4.4(b). The large and variable over-conservatism of Expression 6.10 in the mid ranges of χ_k is improved by the dual Eurocode Expressions 6.10 (a) & (b). The consistency of reliability is also improved. The lack of reliability for $\chi_k > 0{,}6$ ($Q_k > 1{,}5G_k$) is more acute, although the same argument about larger conservatism for Q_k as discussed for Expression 6.10 applies here. The UK modification of Expression 6.10 (b) improves the consistency of reliability up to $\chi_k = 0{,}5$ but with some over-conservatism; the lack of reliability for large χ_k is moderated somewhat.

The reliability performance of the SABS 0160 Expressions 4 (e) & (f) is shown in Figure 4.5. In Figure 4.5 (a) the results are shown when a resistance factor $\gamma_R = 1/\varphi_R = 1{,}42$ is used to achieve a probability of resistance failure $p_{f,\,R} = 10^{-2}$ as specified in SABS 0160. The reliability requirements are generally satisfied by the primary Expression 4 (f) over a wide range of practical χ_k values. At G_k dominating conditions Expression 4 (e) takes over albeit in an inefficient manner, with a sharp transition and large conservatism for G_k only. The transition occurs at $\chi_k = 0{,}158$ or $Q_k = 0{,}1875G_k$.

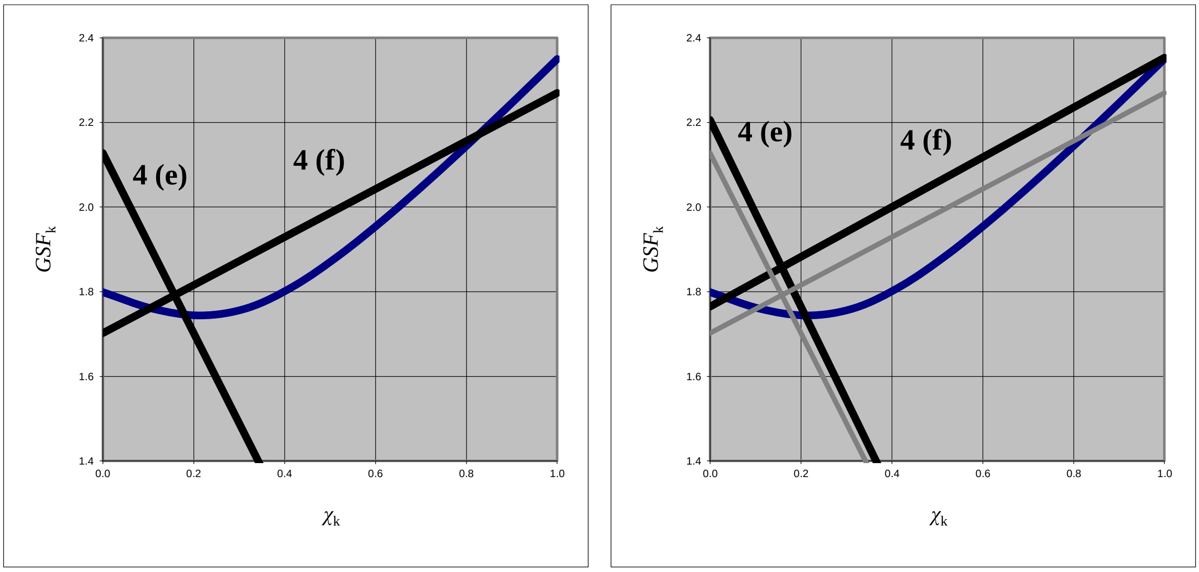
Insufficient reliability is achieved at the other extreme, where $\chi_k > 0{,}8$ or $Q_k > 4G_k$. A possible solution would be to increase the resistance factor γ_R, with results as illustrated in Figure 4.5(b). This is however a hypothetical solution since it would not be proper to solve insufficiencies in action functions by adjusting resistance. Nevertheless it provides an illustration of the vertical shift of the design function due to an increase in the resistance factor.



(a) Expression 6.10 (a) & (b)

(b) Expression 6.10 (a) & (b UK)

Figure 4.4 Reliability compliance of EN 1990 Expression 6.10 (a) & (b)



(a) Resistance at $\gamma_R = 1/\varphi_R = 1{,}42$ ($p_{f,R} = 10^{-2}$)

(b) Resistance at $\gamma_R = 1/\varphi_R = 1{,}47$ ($p_{f,R} = 0{,}5*10^{-2}$)

Figure 4.5 Reliability compliance of SABS 0160 design functions

4.3.4 Revised action combination scheme for U-LS

Whilst maintaining the basic tenet of the SABS 0160 scheme for the combination of permanent and variable actions G_k & Q_k, for various reasons its modification was investigated. The technical motivation for the reassessment was to improve the reliability performance of the design functions. A more practical consideration was to improve the consistency of reliability when permanent actions dominate – this was considered to be particularly important when provision is made for geotechnical design. From a harmonisation point of view the possibility of improving consistency with one of the main EN 1990 options was also a consideration. The reliability performance of reference functions as assessed above was used to devise a scheme that fits into the relationship with Eurocode, provides for the revised scope of application and could be applied effectively and economically.

The simple adjustment is made of replacing the partial factors $\{\gamma_G; \gamma_Q\}$ for permanent (dead) and variable (live) actions from SABS 0160 Expression 4(e) of {1,5; 0} with the load case representing dominant permanent (P) action (STR-P). The partial factors are adjusted accordingly to {1,35; 1,0}, with γ_Q applied to the leading variable action only. Note that both Expression 4(e) and STR-P are consistent with allowable special applications of EN 1990 Expression 6.10(a).

The degree to which the discontinuity and inconsistency in reliability for the SABS 0160 procedure is smoothed out by the introduction of the STR-P load combination applied in SANS 10160-1 is shown in Figure 4.6: this is indicated by the improved approximation of the combined {STR; STR-P} load combinations of the curved graph indicating the requirement for the global safety factor (GSF_k) to achieve the target beta of β_{target} = 3,0 as a function of the ratio of variable to total action χ_k; all variables expressed in terms of their characteristic values X_k. The practical implications of this action combination scheme are demonstrated in Figure 4.7, indicating that STR-P is only relevant in extreme cases of dominating permanent actions G_k.

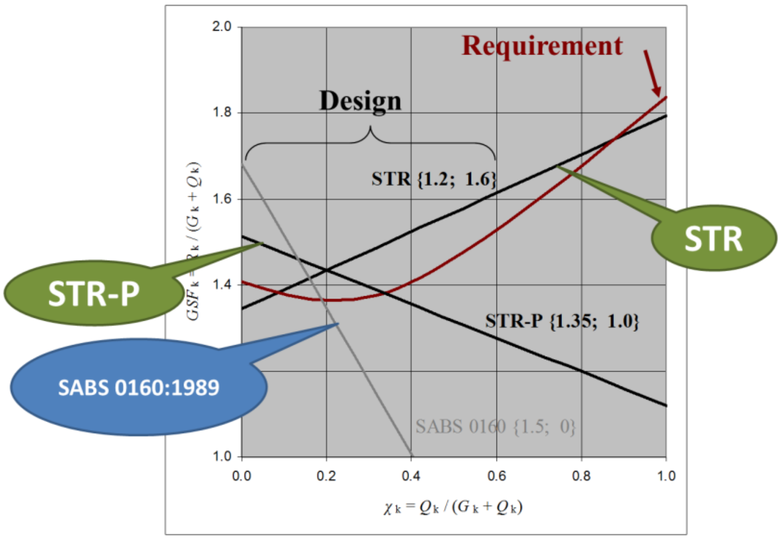


Figure 4.6 *Comparison of previous and present action combination schemes*

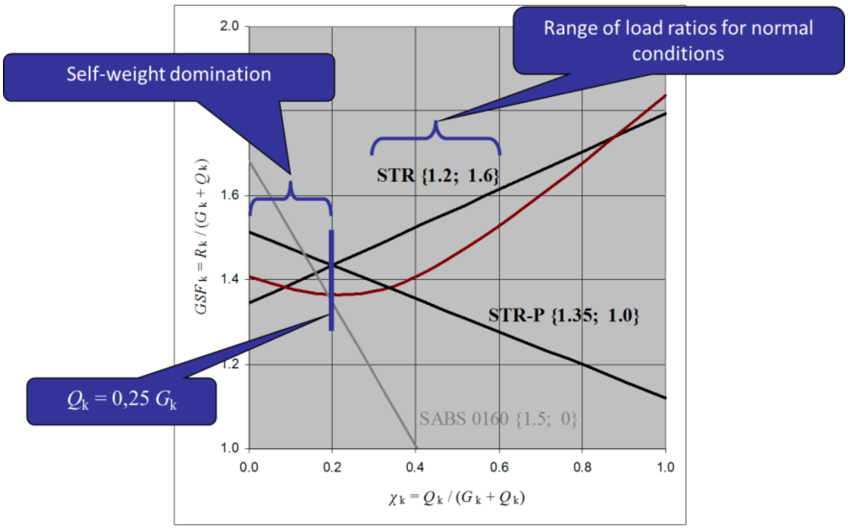


Figure 4.7 *Controlling design expression as a function of the variable action fraction (χ)*

4.3.5 Wind action calibration

The partial factor for wind actions used in SABS 0160 of γ_W = 1,3 derives from the (anomalously) low value for the ratio k_W of mean to characteristic (specified) value of the probability model for wind used for South African conditions, as compared to models generally used, e.g. given by Holický (2009). The effects of the two alternative wind load probability models are shown in Figure 4.8. Due to a general increase in wind loading resulting from the introduction of Eurocode models in SANS 10160-3 (see Section 5.2.1), it was decided to maintain the value of γ_W = 1,3 from SABS 0160 until a sound reliability basis for adjustment could be established (see Section 7.2 for the presentation of subsequent investigations).

The reliability implications of applying the dual action combinations scheme STR and STR-P stipulated in SANS 10160-1, as presented in the previous section, to the two probability models with k_W = 0,42 (SABS 0160 model) and k_W = 0,7 (Holický model) is demonstrated in Figure 4.8. It is clear that the design functions are over-conservative if a mean of 0,42 applies, but more reasonably provide for the more realistic value of 0,7. An indication of the practical implications of the {STR; STR-P} action combination scheme for wind is shown in Figure 4.9, indicating that the influence of permanent actions become more prevalent for wind loading.

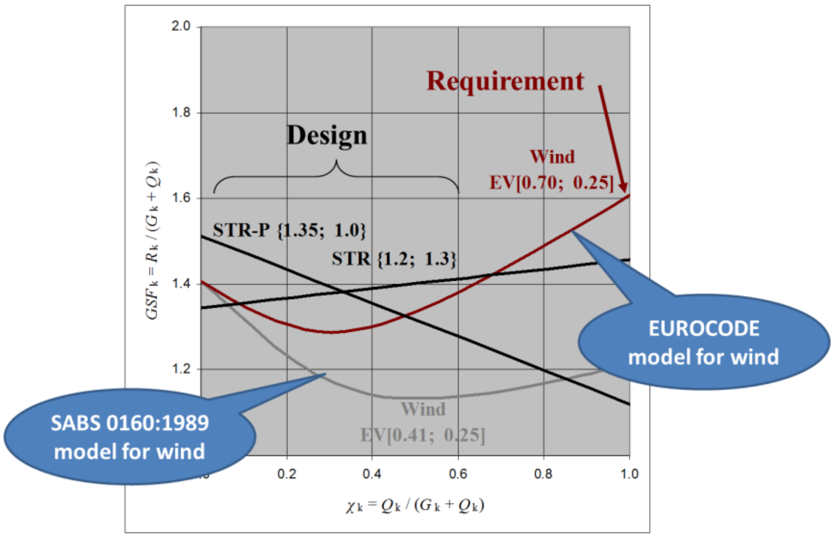


Figure 4.8 Comparison of action combination scheme with reliability requirement for wind

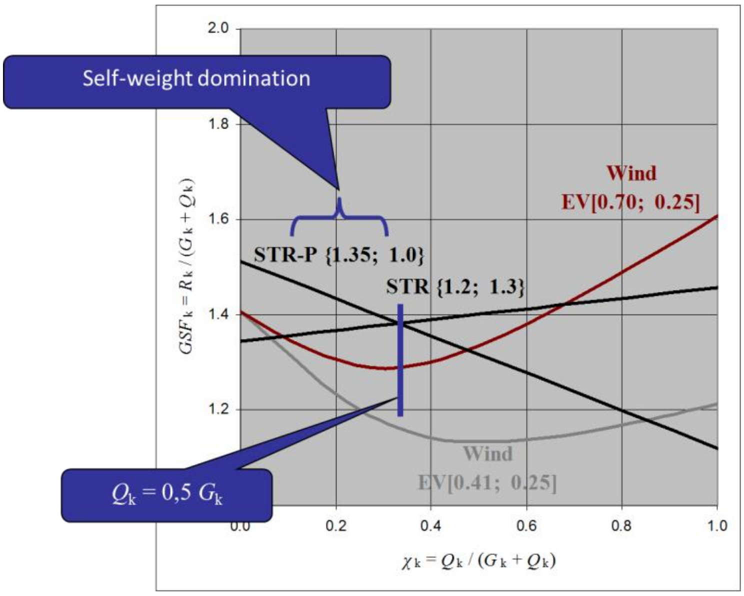


Figure 4.9 Controlling design expression as a function of the wind action fraction (χ)

From the investigations on wind loading reliability performance, the need was identified to investigate the reliability models for South African conditions, consisting of the role of strong wind probability models and the more general contributions of time invariant wind engineering mechanisms. Subsequent investigations of the reliability performance of wind loading are reported under separate headings, with Section 5.2 discussing contributions to the development of SANS 10160-3 on wind loading and Section 7.1 on strong wind investigations in general.

4.3.6 Accidental actions and robustness

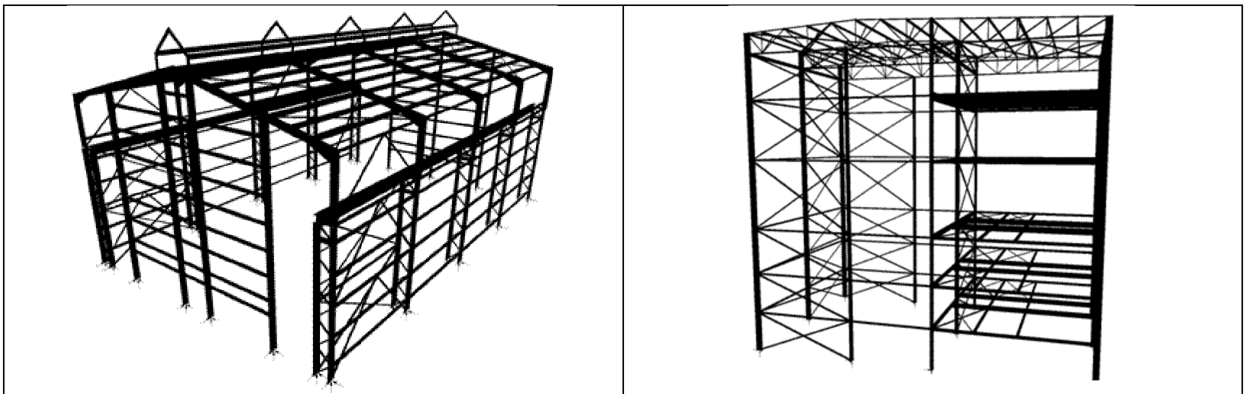
The more formal and systematic treatment of accidental actions and provisions for structural robustness is an illustration of the extension of the scope of SABS 0160 as based on Eurocode. An adaptation was made however, to include the basis of design requirements and procedures from EN 1991-1-7 to the more logical position in the head standard SANS 10160-1. This required the rearrangement and some reformulation of the requirements for the accidental design situation.

Noteworthy is the presentation of the general requirements of the provisions for abnormal events (accidental actions, robustness) in the non-normative *Introduction* of Part 1 in order to ensure that an unreasonable burden of responsibility is not placed on the structural engineer; beyond the specific stipulations given in the normative standard, in addition to informative guidelines given in an annex.

The balance of the arrangement for the accidental design situation is done in terms of a separate sub-clause under requirements; provisions for identified accidental actions and unidentified actions (robustness) as a separate design situation of the provisions for the ultimate limit state; further elaboration for the treatment of robustness is provided in an informative annex for *design for consequences of localised failure from unspecified causes*.

The importance of seismic design to contribute to the general robustness of structures can be illustrated by a review of detailing requirements for structural concrete elements to provide seismic resistance presented by Wium and Retief (2004). Furthermore, it should be noticed that a basic assumption of reliability modelling and the specification of the reference case for the target reliability is based on ductile failure modes (see Section 2.1.3). Therefore detailing requirements for seismic resistance form an integral part not only of general robustness, but indeed also of the basic assumptions of structural reliability.

An investigation on the implications of the requirements for the treatment of robustness for the design of industrial steel structures is reported by Jacobsohn, Retief and Dunaiski (2010). Two industrial steel structures representing different structural concepts were selected for robustness assessment, as shown in Figure 4.10. The industrial portal frame structure represent a building housing general manufacturing, including a heavy overhead travelling crane. The multi-storey building represents the combination of a conventional multi-story industrial building and a fixed structure supporting a process installation. An elaborate application of the procedures for design for the consequences of localised failure to the two diverse structural systems were used to compile a systematic general scheme for the robustness assessment of industrial steel structures, as shown in Figure 4.11.



(a) Industrial portal frame structure	(b) Multi-storey process building structure

Figure 4.10 Industrial steel structures selected for robustness assessment

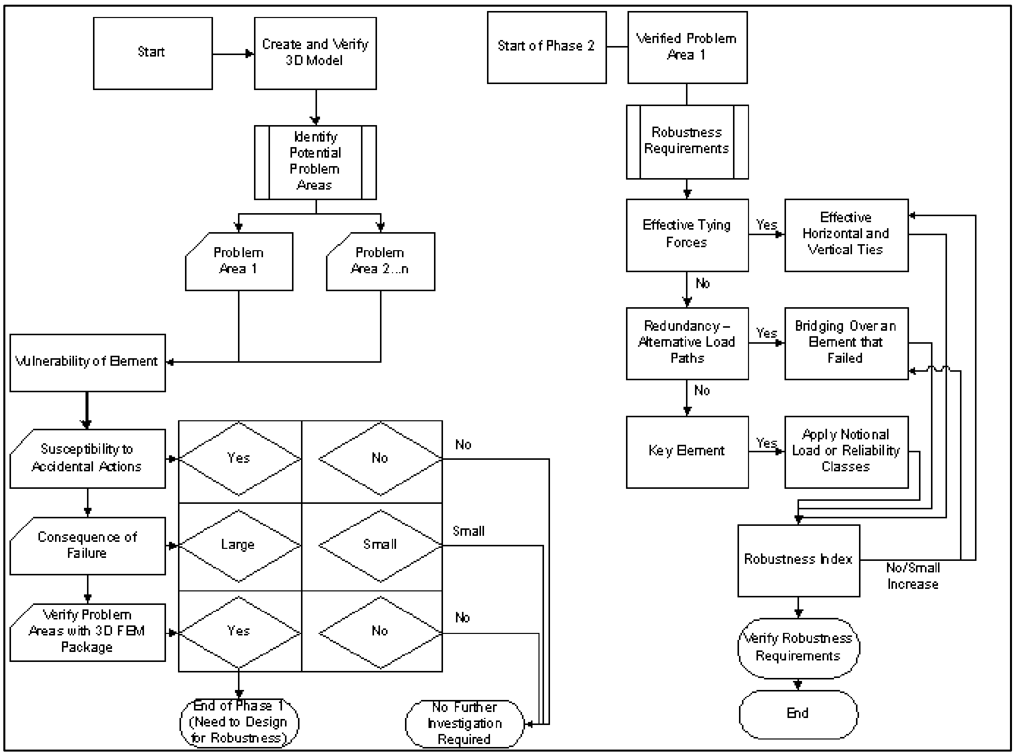


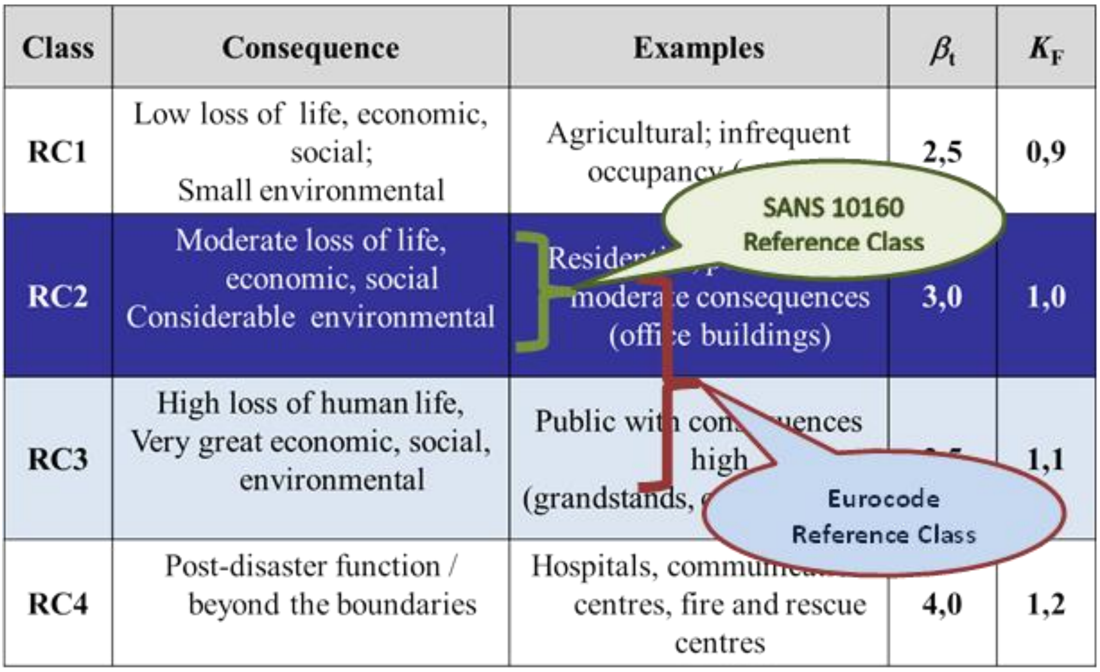
Figure 4.11 Proposed flow-diagram for applying robustness requirements to steel structures

An important consequence of the formal treatment of accidental design situations is that the treatment of seismic design is reclassified as an accidental case, which was introduced from Eurocode. Another observation is that the system of building types for reclassified consequence classes provides a clearer indication of the reliability classification of structures, as compared to the generic classification of reliability classes proper (see Section 4.3.8).

4.3.7 Reliability classification

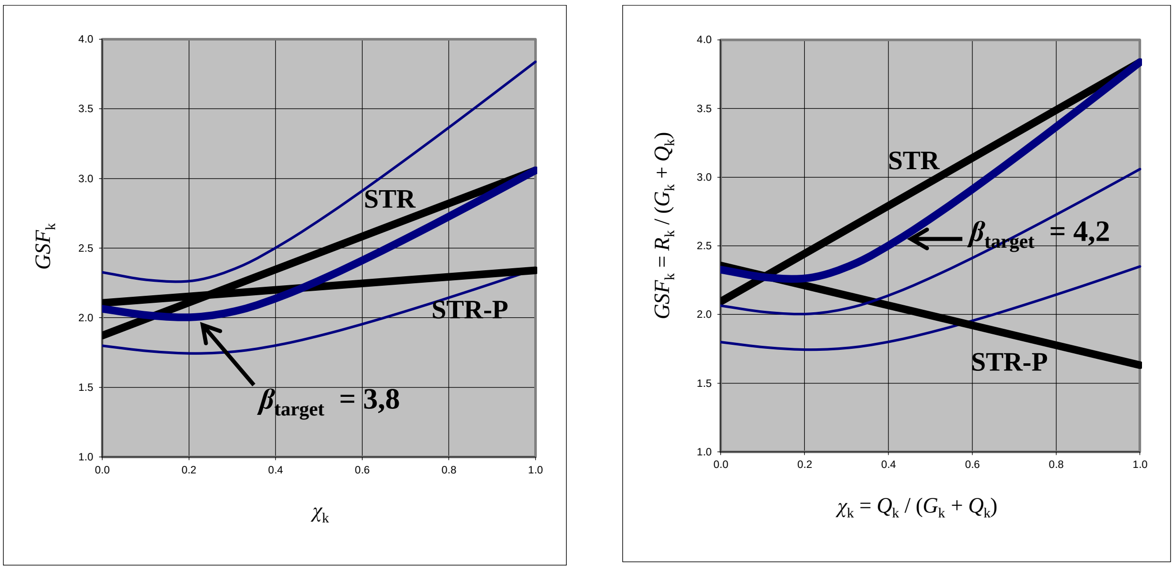
The design procedures of SANS 10160 are intended to achieve a target level of reliability $\beta_T = 3{,}0$; as applied to Reliability Class 2 (RC2) in the reliability classification system. This corresponds to the lower part of Eurocode RC2, as shown in Figure 4.12. The consistent four level classification system employed in SANS 10160 is summarised in Figure 2.4 (see Section 2.2.3)

Adjustment for reliability classes may be afforded by applying a simple multiplication factor K_F {0,9; 1,0; 1,1; 1,2} to the unfavourable actions for {RC1; RC2; RC3; RC4} respectively. Adjustments for resistance are recommended to be achieved through quality management measurements. The way in which the adjusted SANS 10160-1 action combinations schemes performs in terms of adjusted target reliabilities is shown in Figure 4.13 for target reliabilities corresponding to Eurocode RC2 and RC3. This confirms the effectiveness of the adjustment of the actions through K_F.



Class	Consequence	Examples	β_t	K_F
RC1	Low loss of life, economic, social; Small environmental	Agricultural; infrequent occupancy (	2,5	0,9
RC2	Moderate loss of life, economic, social Considerable environmental	Residen… moderate consequences (office buildings)	3,0	1,0
RC3	High loss of human life, Very great economic, social, environmental	Public with consequences high (grandstands,	[illegible]	1,1
RC4	Post-disaster function / beyond the boundaries	Hospitals, communication centres, fire and rescue centres	4,0	1,2

Figure 4.12 Reliability class scheme given by SANS 10160-1



(a) Target reliability β_{target} = 3,8

$K_{F,G} = K_{F,Q}$ = 1,16; $K_{F,R}$ = 1,0

(b) Target reliability β_{target} = 4,2

$K_{F,G} = K_{F,Q}$ = 1,22; $K_{F,R}$ = 1,0

Figure 4.13 Performance of SANS 10160 design functions adjusted for target reliability

4.3.8 Alignment of classification systems

The classification system used in Eurocode was reviewed and adjusted for SANS 10160-1. The alignment of levels of Quality Assurance (QA); Reliability Classes (RC1 – RC4) (Figure 4.12) and the associated reference target reliability levels (β_t) is shown in Table 4.2; also shown are the classification system for consequence classes providing for structural robustness, building seismic classes and geotechnical categories on which the level of practice should be based. Such alignment and interrelationship between different types of design approaches and procedures can be enhanced through meta-standard management of standards development activities.

Table 4.2 Example SANS 10160 combination schemes for the serviceability limit state

RC	β_t	QA	Accidental Consequence Class	Seismic Class (public safety)	Geotechnical Category
RC1	2,5	Basic	Single occupancy ≤ 3 storeys	Minor (Agriculture)	Small structure; no stability/movement
RC2	3,0	Normal	Residential, office ≤ 4 storeys	Ordinary	Conventional structure/foundation
RC3	3,5	Extended	Residential, office 5-15 storeys	Important (schools; assembly)	Ground/structure require geotechnical input
RC4	4,0	Regulated	Public in large numbers, stadia > 5 000	Vital (hospital, fire, power)	Large; unusual; complex; abnormal risk

4.3.9 Parametric assessment of resistance variability

The reliability performance of the modified SANS 10160-1 design functions is presented in Figure 4.14. The effect of the variability of the resistance, as given by V_R {0,10; 0,15; 0,20; 0,25} is shown. The resistance factor γ_R is determined to comply with the limit of $p_{f,R} = 10^{-2}$ for the probability of the resistance below the design (factored) value.

4.3.10 Serviceability criteria

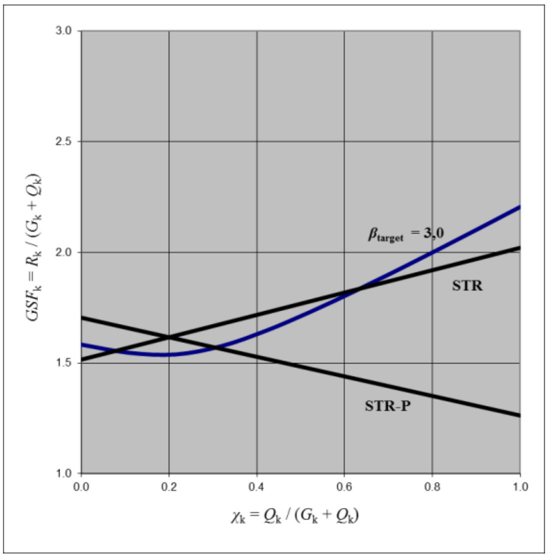
Provisions for the serviceability limit state (S-LS) were substantially reviewed for SANS 10160-1, in accordance with the requirements of EN 1990. The most important development is the introduction of differentiated serviceability performance levels, which then required a review of the associated serviceability criteria.

The action combination scheme presented in EN 1990 is followed in SANS 10160-1, but with the simplification which results in the use of a single combination factor ψ, as opposed to the three level scheme of EN 1990 which is used in the formulation of differentiated S-LS action combinations. The resulting action combination schemes for serviceability are illustrated in Table 4.3.

Table 4.3 Example SANS 10160 combination schemes for the serviceability limit state

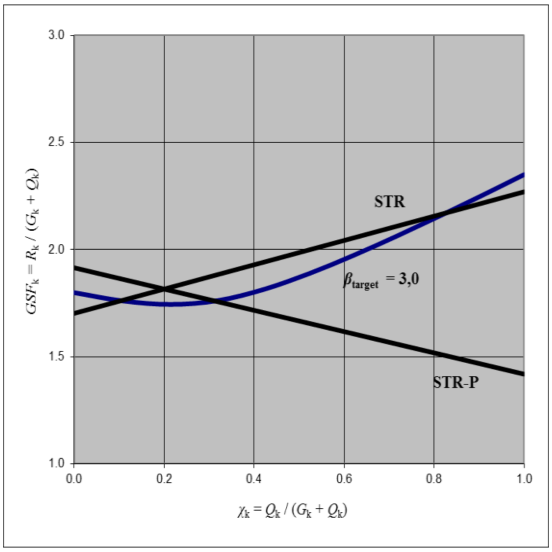
SERVICEABILITY LIMIT STATE		
Irreversible	$1{,}1G_k + 1{,}0Q_k + \sum_{i>1}\psi_i Q_{k,i}$	Imposed leading (at characteristic value)
	$1{,}1G_k + 0{,}6Q_k + \sum_{i>1}\psi_i Q_{k,i}$	Wind leading (at *reduced* characteristic value)
Reversible; Long-term; Appearance	$1{,}1G_k + \sum \psi_i Q_{k,i}$	All variable actions at *arbitrary-point-in-time* value

The serviceability criteria as set in SABS 0160:1989 are based on the International Standard ISO 4356:1977. These criteria were reclassified to provide separately for the irreversible and the reversible/long-term/appearance S-LS respectively. The limiting values were systematically reviewed and assessed. A summary of the reassessed criteria as presented in SANS 10160-1 Annex C (Informative) *Recommended criteria for deformation of buildings* are shown in Table 4.4 and Table 4.5 for the irreversible and reversible S-LS respectively.



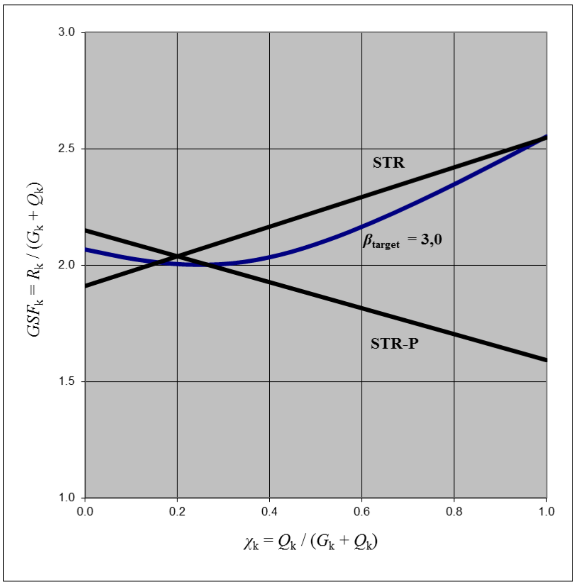
(a) V_R = 0,10

γ_R = 1,26 ($p_{f,R}$, = 10^{-2})



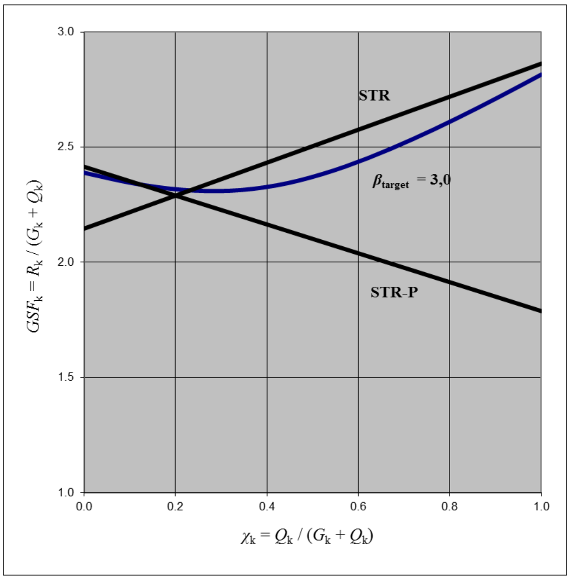
(b) V_R = 0,15

γ_R = 1,42 ($p_{f,R}$, = 10^{-2})



(c) V_R = 0,20

γ_R = 1,59 ($p_{f,R}$, = 10^{-2})



(d) V_R = 0,25

γ_R = 1,79 ($p_{f,R}$, = 10^{-2})

Figure 4.14 Reliability performance of SANS 10160 design functions across parametric range of resistances

Table 4.4 Summary of serviceability criteria for the irreversible limit state

Criterion	Deformation	Effect	Actions; Comments
Storey/500	Terminal: Vertical members	Partitions	Wind: Elastic effect
Span/500	Terminal: Non-cantilever	Partitions	Differential settlement: Elastic & creep
Span/500 to /300	Medial & Terminal: Cantilever & non-cantilever	Partitions	Imposed: Elastic & creep; Self-weight & pre-stressing: Creep
Span/250 to /125	Terminal: Cantilever roof	Roof covering	Imposed: Elastic & creep; Self-weight & pre-stressing: Creep
Span/100	Terminal: Cantilever	Support	Differential settlement: Elastic & creep
Storey/100	Terminal: Vertical members	Support	Wind: Elastic effect

Table 4.5 Summary of serviceability criteria for the reversible, long term and appearance limit state

Criterion	Deformation	Effect	Actions; Comments
Building height/500	Horizontal terminal deflection: High-rise		
Span/300	Medial deviation: Floor	Use (curvature)	Self-weight & imposed: Elastic & creep
Visible length/250 or 15mm	Terminal deviation: Roofs & cantilever floors	Appearance	Differential settlement, self-weight & imposed: Elastic & creep
Or 30mm	Medial deviation: Roofs & floors		Self-weight & imposed: Elastic & creep
Span/100	Terminal: Non-cantilever	Use (slope)	Differential settlement: Elastic & creep

4.4 Conclusions from background investigations on SANS 10160-1:2010

The most important elements of SABS 0160 which are maintained in SANS 10160-1 include:

- The scope of application,
- The function of the standard to provide for standard (non-specialist) practice;
- The reference reliability level, including the application of simplified procedures such as the application of the Turkstra rule for combinations of actions.

An important outcome of the consistency that has been maintained in SANS 10160-1 with SABS 0160 is that existing materials-based limit states design standards can be directly applied together with SANS 10160!

The scope of provisions for the basis of structural design is however expanded substantially, as derived from EN 1990, including:

- The formal treatment of the basis of design in a separate Part of the standard;
- The extended reliability framework in terms of differentiated limit states for both the ultimate and the serviceability limit states,
- The related action combination schemes;

- The more formal reliability basis for the reliability framework, reliability differentiation, specification of basic variables and partial factors, the basis for design being assisted by testing.

SANS 10160-1 also accommodates the extended range of actions included from Eurocode into Parts 2 to 8. Provision for geotechnical design and actions as treated in SANS 10160-5, as derived from EN 1997-1 *Geotechnical design – General rules* has a notable influence on the provisions for action combination schemes.

The requirements for the basis for structural resistance are only implied in SANS 10160-1, but explicit specification would really relate to standards committees for materials-based design standards and would therefore lie outside the scope of design procedures. Sufficient consistency with EN 1990 has been maintained so that it could serve as basis for future development of the next generation of materials-based standards.

SANS 10160-1 *Basis of structural design* thus represent an important link between South African structural design practice and Eurocode as a reference and source of information from which structural design procedures are adapted to South African conditions and scaled to local practice.

4.5 International harmonisation and standardisation

Involvement of the Candidate with international standards development for structural design as the representative of SABS TC98 *Structural and Geotechnical Design Standards* to ISO TC98 *Bases for Design of Structures* provided an opportunity for investigations related to the background to various ISO standards on the reliability basis of structural design. Another theme was the consideration of the merit and nature of the revision of two standards, ISO 2394:1998 *General Principles on Reliability for Structures* and ISO 22111:2007 *Bases for Design of Structures – General Requirements*.

4.5.1 Background to the revision of ISO 2394

Based on the theory of structural reliability Holický took the lead in exploring the development of generic design parameters in a standardised format in Holický and Retief (2010) and the derivation of target levels of reliability through a process of optimisation as reported by Holický and Retief (2011). Generic partial factors are derived for structural resistance, permanent load and variable actions based on representative distribution parameters. The influence of the time-independent contributions for wind loading is explored, identifying this topic as something that needs further investigation. The related investigation (Holický, Retief, Sykora 2015) explores the topic further, by presenting the reliability models and their associated application in a suitable manner for use in standardised procedures, such as a future version of ISO 22111.

In the follow-up investigation (Holický & Retief 2011) an optimisation scheme is set up to select target levels of reliability on the basis of comparing marginal safety cost to expressions of failure cost as function of various model parameters for the consequences of failure. It is demonstrated that the target level of reliability is sensitive to the ratio of the cost of failure C_f and the C_1 cost per unit of the decision parameter, which can vary over a wide range of values. Illustrative parametric results are shown in Figure 4.15 in comparison to values for target β-values listed by ISO 2394:1998 and the JCSS Probabilistic Model Code.

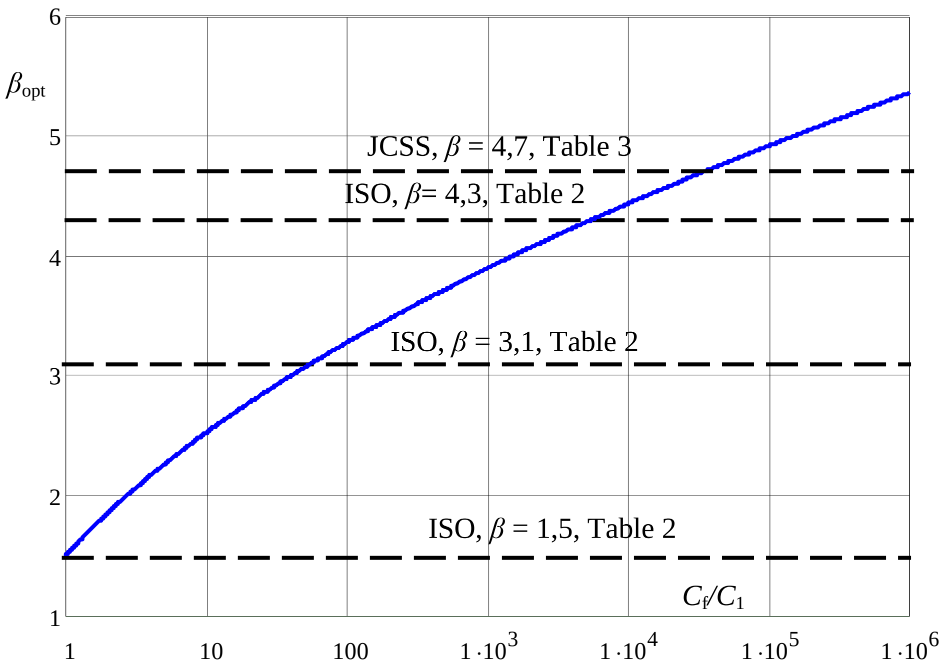


Figure 4.15 Target level of reliability as a function of the ratio of failure cost C_f and cost proportional to increased reliability C_1

A proposed revision of ISO 2394:1998 (Holický, Retief et al 2010) serves as basis for reviewing the merit of the standard and the motivation for its revision (Retief and Holický 2010). It is noted that ISO 2394 plays a central role in standards development, both for related ISO Standards and other standards that are based on the principles of reliability such as Eurocode EN 1990 and *fib* Model Code. Related ISO standards on application of ISO 2394 principles include ISO 22111 on general requirements, ISO 13822 on existing structures and ISO 13823 on provisions for durability. ISO standards on actions include provisions for snow, wind, ice, waves and currents and seismic actions. The Probabilistic Model Code of the JCSS provides basic concepts and models that are relevant to the scope of ISO 2394. Advances in risk assessment include ISO 13824 and the JCSS *Risk Assessment in Engineering*. These developments place an imperative on updating ISO 2394 to set it at a refined level of reliability principles. It is noted in particular that risk management should play a central role in the review process. Although it cannot be claimed that the revised standard ISO 394:2015 followed the directives given in this paper, it can be confirmed that these sentiments were confirmed by the team of experts who have formulated the revision.

4.5.2 Proposal for revision of ISO 22111:2007

An initiative to revise the standard ISO 22111:2007 is reflected in a similar manner in the paper by Holický, Retief et al (2015). The objective with such a revision is based on updating the standard in the light of the new version of ISO 2394 that will come into effect shortly, particularly since some informative material on operational design approaches have been omitted from the new version of ISO 2394, although the basis for the semi-probabilistic approach is now stated more directly. The most important motivation for the revision of ISO 22111:2007 is that it does not yet fully reach its potential to serve as basis for harmonisation between the different ways in which the principles of reliability have been applied to operational semi-probabilistic design by different countries and regions. Holický, Retief et al (2015) provides an outline of the reliability basis that could serve to enhance harmonisation. Such a development will be particularly useful to South Africa, with its legacy of standards being adopted from various reference sources, to achieve maximum benefit from the common knowledge and experience base of structural design and performance worldwide.

4.5.3 Background to ISO 2394 Annex D

The incorporation of Annex D *Geotechnical Reliability Based Design* was primarily motivated by the Candidate on the basis of promoting consistency between structural and geotechnical design on the basis of the principles of structural reliability. However, the mutual benefit of broadening the base of application of ISO 2394 as such and providing a high level international platform for geotechnical reliability based design practice is just as important. Particularly considering the perspective of geotechnical reliability based design, efforts are now made to promote the initiative of ISO 2394 Annex D: The publication of ISO 2394 will be announced in GeoRisk Letters (Phoon & Retief 2015); and an extensive review and background are submitted by Phoon, Retief, Ching et al (2015). The extensive material compiled to develop Annex D will be published as a monograph *Reliability of Geotechnical Structures in ISO 2394* (Phoon & Retief 2015(Editors)).

Chapter 5: Reliability Assessment of SANS 10160 Actions

Contributions to the assessment and development of specifications of actions for the new South African Loading Code SANS 10160:2010 Parts 2 – 8 are primarily captured in the respective chapters in the Background Report (Retief & Dunaiski (Eds) 2009), complemented by a few papers dealing with specific investigations having fed into the standardisation process. The perspective taken throughout was to ensure the best possible reflection of the principles of reliability and the related compliance with Part 1. This included the overall theme of converting the comprehensive Eurocode standards to the specific scope of SANS 10160.

Notably the Eurocode parts considered are not limited to EN 1991 *Actions on Structures*, but included also EN 1996 *Geotechnical Design* and EN 1997 *Design Provisions for Earthquake Resistance of Structures*. The same principles for the adaptation of the Eurocode Parts 2 – 8 were followed as was the case for Part 1: limiting the scope of provisions to what is relevant to building structures and excluding advanced design conditions (for example dynamic structural response to wind loading) are examples of a concerted effort to simplify the local standard.

5.1 Provisions for General Actions – Parts 2, 7 & 8

SANS 10160-2:2010 represents a thorough review of the provisions for loading due to self-weight and imposed floor, roof and balustrade loads. The introduction of new standards for thermal loads and actions on structures during execution (or construction) represents a significant extension of the scope of the South African Loading Code, demanding careful consideration. The accidental actions of impact and internal explosions classified as general actions by Eurocode were judged to be too specialised to be included in SANS 10160. The introduction of the comprehensive treatment of structural fire design was judged to be beyond the capacity of the reviewing Working Group. Background to the formulation of SANS 10160 Parts 2, 7 and 8 is provided by Retief & Dunaiski (2009b).

5.1.1 Actions due to self-weight

The assessment of the Eurocode provisions for self-weight loads presented by Retief & Dunaiski (2009b) is limited to a critical review of the topic, without the need for any modelling or decision making. The review mainly discusses the conceptual basis for self-weight loads such as the general model, load classification, sources of uncertainty; the way in which the Eurocode specifications are implemented in Part 2. The substantial extension of appended tables of densities of structural and non-structural materials is a notable improvement of the standard.

5.1.2 Imposed loads

The revision of the provisions for imposed loads on building structures afforded the opportunity to assess it in terms of the scope of loads which are provided, together with the classification of occupancies; and finally the specified minimum values to be applied. The assessment of minimum imposed loads is based on a comparison with other standards, with the emphasis on generally accepted international standards. An initial assessment of the provisions for imposed loads of SABS 0160:1989 is reported by Retief, Dunaiski and De Villiers (2001), consisting of a comparison of the specified loads of a representative set of national standards. This assessment indicates significantly lower values specified by SABS 0160 for certain occupancy classes. The fundamental load models for imposed loads are considered by Retief and Dunaiski (2009b), followed by a comparison of alternative occupancy classification schemes and ultimately an assessment of the new specified loads, changes from the previous values and various implementations of the Eurocode values.

It is concluded that a more refined occupancy classification system may appear to be more complicated, but actually facilitates the selection of appropriate values and accordingly limits the need to be unnecessarily conservative. This is illustrated by the way in which the Eurocode EN 1991-1-1 general nationally determined parameters are modified by the BS National Annex for the UK. It is clear that there are still opportunities to improve the specification of such well-established design parameters as imposed loads on buildings.

Furthermore, in spite of the availability of reliability models for imposed loads based on extensive load surveys, transparent linking to specified values is still lacking. One specific exception is the specified value for imposed roof loads which is modified on the basis of a survey reported by Retief & De Villiers (2005). Expert measurement was used to derive reliability models for the distribution and magnitude of imposed roof loads during the construction and maintenance stages.

5.1.3 Imposed roof loads

A comparative survey of provisions for imposed roof loads in structural design standards reveals a wide range of minimum load values, with provisions of South African (SABS 0160:1989) and Australian (AS 1770.1 1989) standards on the low side (Retief, Dunaiski, De Villiers 2001). A lack of information on roof loads makes it difficult to assess differences in code specifications. Reliability models for imposed roof loads on light industrial steel structures during construction and maintenance based on expert measured variables are developed to establish a scientific rationale through which the codified design values may be assessed effectively (Retief & De Villiers 2005).

The expert measurement procedure is carefully designed to satisfy criteria for scientific measurement such as objectivity, repeatability and empirical control. Experts' estimates of variables quantifying roof load mechanisms are weighed or rejected according to a classic hypothesis testing procedure (Cooke 1991) based on related seed variables on which independent information is assembled. The roof load mechanisms are then converted into equivalent uniformly distributed load on a large area represented by a building frame (EUDL), and a small area represented by a purlin.

The expert survey was designed to measure practical experience in construction and maintenance from structural engineers and steel and roofing contractors and foremen. A set of 31 experts were selected with 7 – 49 years and an average of 21 years of experience. Roof loading mechanisms were identified during a preliminary survey amongst a limited group of 9 experts, averaging 26 years of experience.

Pertinent roof loads identified from the initial survey and selected for the extensive survey consisted of workers on the roof during installation of roof sheeting and maintenance and repair respectively, and stacked roof sheeting during installation. Large loaded areas were represented by the load on one structural frame, and small areas by the load on a two-span purlin. Both average values and maximum values defined as those occurring in one out of twenty building sites were considered.

Average values were observed independently at construction sites to serve as seed variables in the calibration of experts. The surveyed variables are summarised in Table 5.1. The number of workers and stacking of cladding during constructions were observed at fourteen construction sites to serve as seed variables.

Table 5.1 Values for expert measured roof load variables

	Variable	**Confidence level values**		
		5 %	**50 %**	**95 %**
	Construction			
1	Maximum number of workers on a frame	3.5	5.3	8.0
2	Maximum number of workers on a purlin	1.6	2.2	3.2
3	Maximum number of bay's cladding	1.3	2.2	3.4
	Maintenance			
4	Maximum number of workers on a frame	1.6	2.5	3.3
5	Maximum number of workers on a purlin	1.2	1.5	1.7

The optimum decision maker was calibrated to consist of six experts of which five were structural engineers and one a roofing contractor. Normalised weights varied between 0.14 and 0.20 for the contribution of the respective experts. The measured values for the respective maximum imposed roof load variables are summarised in Table 5.1. The quantitative imposed roof load mechanisms were converted into equivalent uniformly distributed load (EUDL) by calculating load effects on a structure and selecting the cases resulting in the largest value for the EUDL.

The results are summarised in Figure 5.1, comparing the AS and SABS provisions for imposed loads to the values derived from 1:20 maximum conditions representing a 5 % fractile characteristic value, interpreted at various confidence levels. It is concluded that the construction roof loads compares reasonably well with the AS and SABS code specified values for large areas, with SABS agreeing with a more conservative interpretation of the results, and AS with a medium degree of conservatism. For SABS the load for small areas is significantly underestimated, even for the least conservative interpretation of the results.

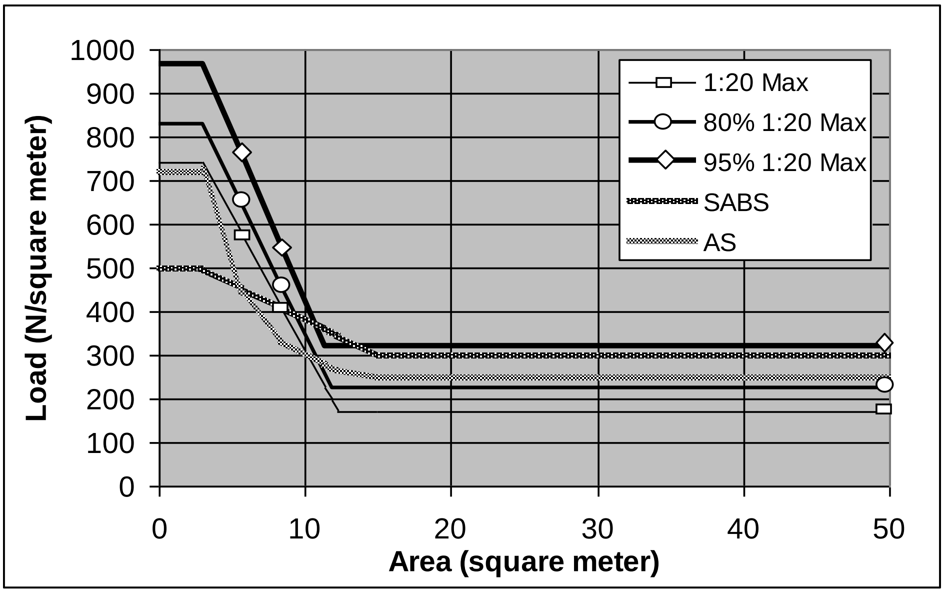


Figure 5.1 Alternative specification for representative construction load for AS and BS Standards, indicating the characteristic load (1:20 Max) at various confidence levels

Imposed roof load during the operational use of the structure is treated as a separate load case. The degree of conservatism has little influence on the interpreted results for maintenance loads, with characteristic loads shown in Figure 5.2. The derived loads are substantially lower than the AS specifications, but again exceed the SABS specification for small areas.

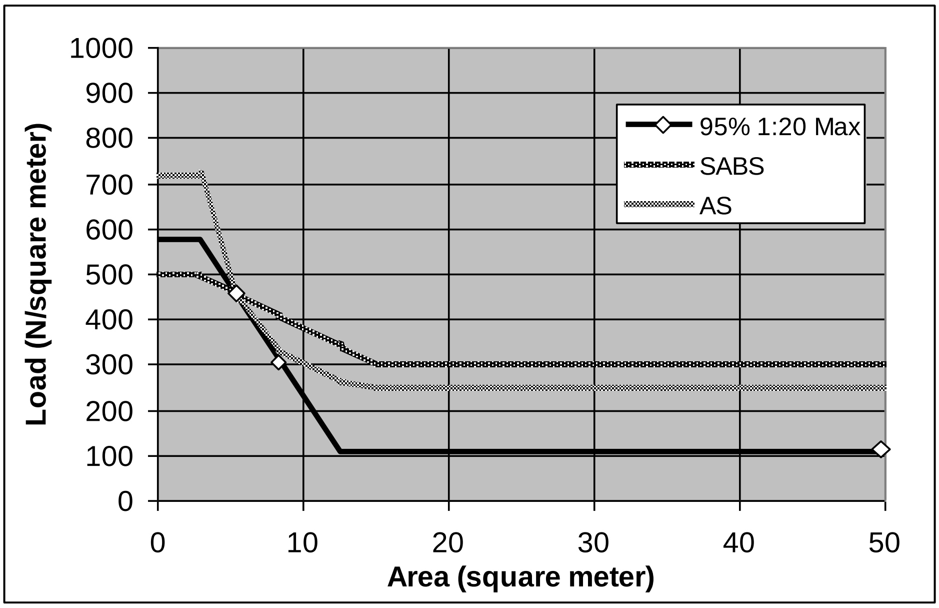


Figure 5.2 Maximum maintenance loads for AS and SABS Standards compared to the characteristic imposed load (1:20 Max) at 95% confidence level

5.1.4 Conclusions on general actions

Although the standard for thermal actions on building structures represents the introduction of new provisions, implementation of this extension to the scope of the new Loading Code did not require any serious assessment or development work. Revision of the maps on characteristic extreme maximum and minimum temperatures extracted from TMH-7 *Code of practice for the design of highway bridges and culverts in South Africa* may be considered in the future.

The new and innovative Eurocode Standard on the actions on the structure during its execution or construction is of such general nature that it could essentially be taken over as is from its Eurocode version, with modifications only for consistency with the other Parts of SANS 10160, particularly Part 1, and omitting the annex on special considerations for bridges. Nevertheless, it addresses an aspect of structural performance and reliability that forms a notable part in the record of structural failures (Schneider 1997). Its formal implementation and integration into standard design practice is of great importance.

5.2 Wind Actions

Wind loading constitutes the dominant environmental action on buildings within the South African climate, where snow loading does not occur. Therefore, the requirements to provide for wind actions on buildings form an important and substantial part of the revision of the Loading Code. A review of developments in standardised procedures for the determination of wind actions in the design of structures in order to establish the general basis for the formulation of new provisions for wind actions is presented by Goliger, Retief and Dunaiski (2009). Following a general review of alternative standards that could serve as reference, the implications of using Eurocode for this purpose is presented. The main issues were the representation of the South African thunderstorm climate of strong winds and a general increase in wind actions which would result from the application of Eurocode procedures.

5.2.1 Adaptation of Eurocode to South African procedures for wind actions

Based on the general review outlined above, guidelines for the adaptation of the Eurocode EN 1991-1-4 to Part 3 of SANS 10160:2010 were formulated and implemented as reported by Goliger, Retief et al (2009). Assessment and motivation for the provisions in terms of the scope of structures, representation of the local strong wind climate, terrain categories and boundary layer profiles are presented.

The main features of the revised procedures for determining wind actions for structural design, as presented in Part 3 of SANS 10160 consist of the following (Goliger, Retief et al 2009):

- **Scope of structures:** The SANS 10160 scope of structures is restricted to reduce the complexity whilst providing for most of the design situations encountered in general practice.
- **Compatibility with EN 1991-1-4:** Sufficient compatibility with the Eurocode procedures is maintained to consider them to be within the tolerances of Eurocode Nationally Determined Parameters. Furthermore, for design applications beyond the limited scope of SANS 10160-3 South African conditions are stipulated sufficiently to allow for the use in South Africa of advanced Eurocode procedures.
- **South African strong wind climate:** The representation of the South African strong wind climate is based on an adaptation of the SABS 0160-1989 specifications, with limited adjustments and improvements to represent the EN 1991-1-4 format. The limited record of strong wind observations and differences in coastal and inland gust characteristics indicate a need for updating the South African design wind map.
- **Application of wind loads:** The incorporation of updated and modern procedures for the conversion of the free-stream wind speed into wind loads is considered to be a major improvement on the revised procedures. The higher wind loads that generally result from the stipulations are accepted as deriving from updated stipulations based on improved information. The net effects of the revised stipulations are assessed in the following chapter.

Due to the complex nature of wind actions on structures and the consequent difficulty in providing clear and unambiguous stipulations, their application requires knowledgeable and responsible treatment. A number of steps in the process require particularly careful consideration due to their influence on the outcome of the design:

- **Terrain category:** Due to the wide range of terrain roughness conditions that are represented simply by a set of four terrain categories, this requires the careful and conservative selection of the appropriate category. The large distances required for full development of wind velocity profiles for rougher terrains should particularly be noted!
- **Topography of terrain and surroundings:** Consideration should be given to the application of wind tunnel testing or modelling for important or wind sensitive structures for complex topography or transition of terrain conditions.
- **Structural configuration:** More complex or composite geometry of structures which deviate from the basic and symmetric shapes provided for, would require careful and conservative treatment or wind-tunnel testing.
- **Structural response:** The possibility of dynamic structural response for both integral behaviour and locally for components or subsystems should be kept in mind.

5.2.2 Strong-wind map for South Africa

In spite of the substantial revision of the wind load procedures presented in SANS 10160-3:2010, the strong-wind map for the country is essentially maintained from SABS 0160:1989. The need for updating the South African design strong wind map was identified by Dr Adam Goliger and the Candidate. Such an investigation was identified firstly to resolve the complexity of the South African strong wind climate (Goliger & Retief 2002). Furthermore the record of strong wind observations was expected to be substantially extended from that used for the SABS 0160 map. An investigation was then launched as the PhD project of Dr Andries Kruger. The detail of the extended investigation is reported in Section 7.1.3, whilst the relevance to SANS 10160-3 is outlined here.

The base map for gust winds shown in Figure 5.3 is reported by Kruger, Retief and Goliger (2013b), based on the data presented by Kruger, Retief and Goliger (2013a).

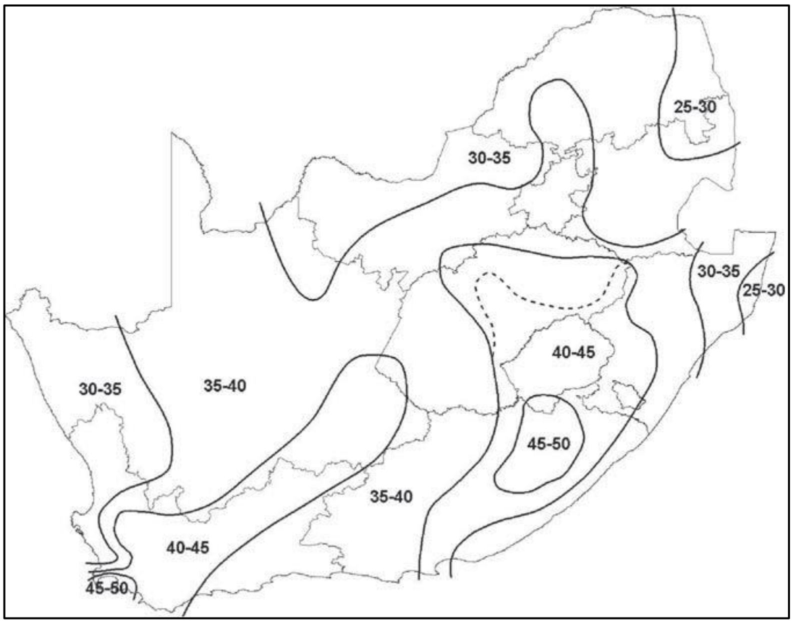


Figure 5.3 Proposed characteristic gust wind map for South Africa, values in m/s

Subsequent assessment of the information on the strong-wind characteristics of South Africa led to the drafting of a revised design map to be included in SANS 10160-3, as shown in Figure 5.4 (Kruger, Goliger and Retief 2015), indicating wind load values per municipal district, assigned in 4 m/s intervals.

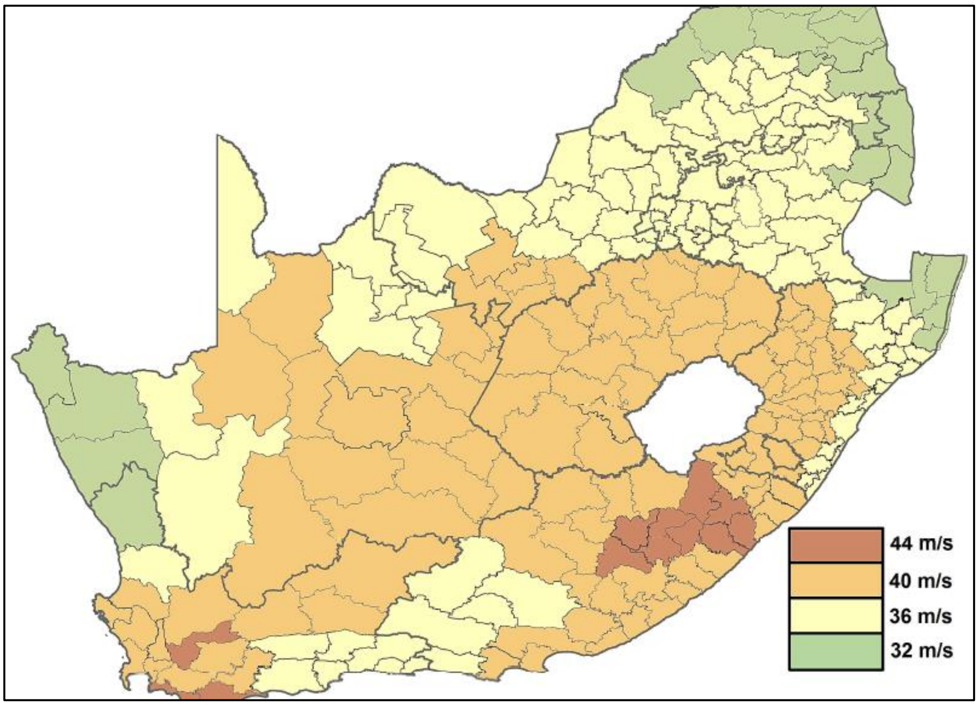


Figure 5.4 Proposed design wind map for South Africa based on local municipal boundaries

5.2.3 Reliability model for wind loading under South African conditions

The basis of design reliability calibration of the load factor for wind loading reported in Section 4.3.5 indicates the anomalous nature of the strong wind probability model for South Africa. An extended investigation was consequently launched to reassess the reliability based on the updated strong wind probability models reported by Kruger, Retief and Goliger (2013a) together with a new assessment of other sources of uncertainty for wind loading (Retief, Barnardo-Viljoen & Holický 2013).

The wind pressure on a structure may generally be written as the combination of the time variant effect of the wind velocity v or Q_{Ref} and the time invariant processes as given in Equation (5.1). The variables are defined in Table 5.2, together with listing of typical distribution values; ρ is the deterministic value of the air pressure. Difficulties arise however to access the basis of the distribution parameters, particularly for the time independent wind engineering processes, in spite of their presentation in the JCSS Probabilistic Model Code (JCSS 2001). The investigation differentiated accordingly between the aleatoric time dependent variability and the epistemic time independent process used to convert the free field wind into loading on the structure.

$$Q = 0{,}5\rho v^2 c_r c_a c_g c_d = Q_{\mathrm{Ref}} c_r c_a c_g c_d \tag{5.1}$$

As discussed in Section 4.3.5, these representative models lead to wide tolerances in the calibration of wind load design procedures. These deficiencies provide the motivation for extended investigations on wind loading, as presented in general terms in Sections 7.1 to 7.2.

Table 5.2 Representative distribution parameters and time dependence for wind load.

Symbol	Variable	Time Dep.	Mean	CoV w_Q
Q_{Ref}	Annual pressure extreme	Yes	0.8-1.0	0.25
c_{r}	Terrain roughness	No	0.8-1.0	0.15
c_{a}	Aerodynamic shape	No	1.0	0.20
c_{g}	Gust effects	No	1.0	0.15
c_{d}	Dynamic effects	No	1.0	0.15

5.3 Provision for Geotechnical Design in SANS 10160

The inclusion of SANS 10160:2010 Part 5 *Basis for Geotechnical Design and Actions* into the South African Loading Code represents a major innovation in the development of the standard series. The interrelationship between initiatives on geotechnical design standards in South Africa, reference to Eurocode EN 1997 *Geotechnical Design* and limitations placed by SABS 0160:1989 are extensively described by Day (2013).

The most pertinent concern was the inconsistency of SABS 0160 with the Eurocode basis of design of EN 1990. Resolving this inconsistency as described in Section 4.3.3 then allowed the introduction of a geotechnical basis of design into South African practice. Of general concern in this presentation is the unified treatment of the basis of structural design as stipulated by Part 1 and geotechnical design stipulated by Part 5 of SANS 10160. Specific contributions were also made with respect to the reliability performance of pile foundations and the associated calibration of design procedures. Only the calibration contributions are presented here, whilst the background investigations on pile foundation reliability modelling and assessment is reported separately in Section 7.4.

5.3.1 The basis of geotechnical design

The provisions made in SANS 10160 for geotechnical design and the opportunities that this creates for the use of an international geotechnical design code, specifically EN 1997-1: 2004, in South Africa are presented by Day & Retief (2009). The changes made to Part 1 *Basis of Design* to accommodate geotechnical limit states and the introduction of new provisions for geotechnical design as Part 5 *Basis of Geotechnical Design and Actions* are described. The background to decisions taken during the revision process is provided. Most important is the fact that Part 5 is extracted from EN 1997-1, thereby ensuring both consistency and the relevance of the background material from Eurocode.

From the three alternative design approaches allowed by EN 1990 & EN 1997-1, Design Approach 1 was selected (only) for Part 5. This decision is based on the principled argument of applying partial factors at the source of uncertainty, that is, at the respective actions and material properties; the practical consideration is that Design Approach 1 most closely represents existing South African practice.

The following topics are treated by Day & Retief (2009):

- Selection of the partial factors for the dual design calculations {STR; STR-P} & GEO are discussed and the selected values compared to recommended Eurocode values.
- Verification of the serviceability limit state is done consistently with Part 1;
- Selection of geotechnical actions is consistent with the scope of geotechnical design relevant to buildings;
- Geotechnical categories and their implications are consistent with the general four-level classification system used in SANS 10160;
- Informative guidance is given to structural engineers on the design of spread foundations, axially loaded piles and earth pressure;
- Guidance is provided on the application of Part 5, including its use in conjunction with Eurocode.

Extensive elaboration on these topics is provided by Day (2013).

5.3.2 Calibration of pile foundation design

The calibration of partial factors for uncertainty in pile resistance models ($\gamma_{R,d}$) for South African conditions reported by Dithinde & Retief (2014) is directly applicable to SANS 10160-5 for local geotechnical design. The calibration utilises a database of Southern African pile tests (Dithinde, Phoon et al 2011) for which the resistance statistics are reported by Dithinde and Retief (2013a). Additional background to the reliability assessment of pile foundations is presented in Section 7.2.

The summary model factor statistics listed in Table 5.3 for various pile classes in terms of soil type and construction practice can be used to derive a corresponding set of values for $\gamma_{R,d}$ as shown in Table 5.4.

Table 5.3 Model factor (M) statistics as a function of Pile Class

Pile class	n	*Mean* m_M	*Confidence - 75%* $m_{M;\,-0,75}$	*Std.Dev.* $s_{M;}$	*Upper CI SD 75%* $s_{M;\,+0,75}$	*COV*	*Skewness*	*Kurtosis*
D-NC	28	1.11	1.03	0.36	0.40	0.33	0.35	-1.15
B-NC	30	0.98	0.93	0.23	0.26	0.24	0.14	-0.19
D-C	59	1.17	1.12	0.3	0.32	0.26	-0.01	-0.74
B-C	53	1.15	1.10	0.28	0.30	0.25	0.36	0.49
D	87	1.15	1.11	0.32	0.34	0.28	0.1	-0.95
B	83	1.09	1.05	0.28	0.30	0.25	0.41	0.47
NC	58	1.04	1.00	0.30	0.32	0.29	0.55	-0.37
C	112	1.16	1.13	0.29	0.30	0.25	0.15	-0.29
ALL	170	1.1	1.07	0.31	0.32	0.28	0.24	-0.75

Table 5.4 Partial factor $\gamma_{R,d}$ values based on mean and conservative 75% confidence level probability moments

Pile class	*n*	*n-1*	*Confidence - 75%*	*Upper CI SD 75%*	*COV*	$t_{n-1}^{0.95}$	X_d	$\gamma_{R,d}$
D-NC	28	27	1.03	0.4	0.39	1.703	0.544	1.8
B-NC	30	29	0.93	0.26	0.28	1.699	0.583	1.7
D-C	59	58	1.12	0.32	0.29	1.672	0.701	1.4
B-C	53	52	1.1	0.3	0.27	1.675	0.702	1.4
D	87	86	1.11	0.34	0.31	1.663	0.674	1.5
B	83	82	1.05	0.3	0.29	1.664	0.658	1.5
NC	58	57	1	0.32	0.32	1.672	0.593	1.7
C	112	111	1.13	0.3	0.27	1.659	0.732	1.4
ALL	170	169	1.07	0.32	0.30	1.654	0.659	1.5

The implications of the reliability calibration for pile resistance under South African conditions and considering the influence of soil type and pile construction method can be summarised as follows:

- The present value of $\gamma_{R,d}$ = 1,5 for the partial factor for uncertainty in the pile resistance model stipulated in SANS 10160-5:2010 will achieve the target level of reliability under all conditions considered here and for pile design in accordance with static pile resistance models for cohesive and non-cohesive soils; albeit with a degree of inconsistency in the reliability achieved.
- A reduced value of $\gamma_{R,d}$ = 1,4 can be justified as based on an undifferentiated dataset; however with concern about consistency and confidence in reliability for the special case of driven piles in non-cohesive soils.
- A differentiated scheme of values of 1,3 and 1,5 for cohesive and non-cohesive soils respectively complies with exceeding the target level of reliability with improved consistency.
- The anomalous behaviour of the case of driven piles in non-cohesive materials, more generally resistance predictions in such soils, warrant further investigation.
- The results from this relatively large dataset of pile tests and predictions methods are relatively coherent when compared to a representative range of pile and soil types and calculation methods from an extensive pile test dataset and $\gamma_{R,d}$ values calculated from this database (Paikowsky, Birgisson et al 2004).

5.4 Crane-induced Actions

The inclusion of crane-induced actions on buildings in Part 6 of SANS 10160:2010 derives from the inclusion of this action class in SABS 0160:1989. This inclusion is also one of the reasons for the inclusion of industrial buildings in the title and scope of SANS 10160. Involvement in Part 6 is based on participation in early parametric investigations on comparisons of crane loads based on SABS 0160 and Eurocode respectively (Warren, Dunaiski, Retief et al 2004) and reporting on future development (Dymond, Dunaiski et al 2006) as a preamble to the reliability assessment of crane induced actions (Warren, Retief, Dunaiski 2005). The emphasis is however on the final reliability assessment of crane induced actions according to the design procedures stipulated in Part 6 (Dymond, Retief, Dunaiski 2009).

The calibration of the partial load factors for crane-induced loads on structures represents a demonstration of a typical classic calibration procedure, done comprehensively to consider representativeness of the cases investigated, alternative design function formats in terms of selected scheme of partial factors, contributions of the various basic variables, sensitivity analysis, all done in terms of a comprehensive design function consisting of both action and resistance basic variables. The investigation presented by Dymond, Retief, Dunaiski (2009) therefore serves not only the purpose of crane load factor calibration but also as a reference to calibration procedures in general.

(i) **Definition of the scope of the calibration:** The scope of the code in terms of design procedures defines the scope of calibration. Crane parameters related to loading were based on a set of more than 500 cranes from a leading crane supplier. A set of three cranes were selected to represent the ranges of governing parameters, classified as small, medium and large installations.

(ii) **Definition of the code objective:** The primary objective of the code provisions is that a minimum level of reliability should be exceeded in all cases; complemented by achieving such reliability economically and consistently. The target reliability is set in accordance with Part 1 at $\beta_T = 3{,}0$.

(iii) **Definition of the code format:** The code formats investigated consisted of four sets of partial factors providing for alternative arrangements of the combinations of three load classes – crane self-weight, hoist load and horizontal load.

(iv) **Development of limit states equations:** The reliability limit states functions were obtained by converting the design crane load models from Part 6 and resistance models for steel and concrete structures into functions of the relevant basic variables, adding model factors for model uncertainty for the calculation of vertical and horizontal load effects. The functions were set up for crane girders and columns directly subjected to loading. Theoretical section properties that exactly satisfy the code requirements were set up in order to avoid any bias deriving from practical design decisions, such as selecting section properties.

(v) **Development of stochastic models:** The single duty cycle representative of the operation of four crane classes {light; medium; heavy; very heavy} was devised. The mean and standard deviation of extreme loads as a function of the number of duty cycles were then obtained through simulation. A distinction was made of two classes of control of overloading of the crane, as an important factor having an influence on extreme crane loads.

(vi) **Determination of optimal partial load factors:** A systematic process was devised to select the controlling conditions amongst the large number of crane classes and configurations, with parametric treatment of reasonable ranges of load ratios, for each of the alternative code formats.

(vii) **Verification of partial load factors:** Sensitivity assessment of reliability modelling and partial load factors: A sensitivity analysis of the calibration process provides an indication of the sensitivity for the selection of parameters made to obtain representative results. Sensitivity factors for basic variables provide an indication of the relative contribution of the various sources of uncertainty to obtaining sufficient reliability.

The calibration results obtained are summarised in Table 5.6 for the respective set of partial factors for crane load (γ_C), differentiated into self-weight (γ_{Csw}) and hoist load (γ_{Ch}), and separate provision for horizontal loads (γ_H). The final step in the process is the decision making for selecting an appropriate code format and the value of the associated partial factors, taking the step from background exercise to implementation of standardised procedure.

Table 5.5 Calibrated ULS partial load factors

Code Format	Good Control				Poor Control			
	γ_C	γ_{Csw}	γ_{Ch}	γ_H	γ_C	γ_{Csw}	γ_{Ch}	γ_H
1	1,69				1,90			
2		1,62	1,73			1,62	1,96	
3	1,43			1,26	1,59			1,26
4		1,35	1,44	1,27		1,35	1,62	1,27

From the reliability assessment it is concluded that a single partial factor is sufficient to achieve consistent reliability for the design process. Although there is some concern that the present load factor of γ_C = 1,6 does not fully comply with the reliability requirements, this is largely as a result of model uncertainty for predicting horizontal load effects, indicating the need for further investigation of this issue. It is clear however that the Eurocode value of γ_C = 1,35 would not achieve sufficient reliability.

The assessment indicates that calibrated partial factors are not sensitive to crane classes. However, differentiated reliability classes could be considered, to be reflected by differentiated partial factors; consistent with Part 1, but not implemented in Part 6, neither in Eurocode.

The background investigations provide insight into the reliability implications related not only in terms of the design procedures provided in Part 6 for the various crane load types and their representation, but also to the range of design conditions covered by the scope of the standard. The calibration clearly demonstrates that overloading beyond specification significantly reduces the reliability achieved by standardised design procedures.

Chapter 6: Structural Concrete Resistance Performance

The power of reliability based structural design derives from the effective way in which all the sources of variability and uncertainty can be combined in a single expression from which design factors could be derived to verify that a specified level of reliability is exceeded. This expression then serves as kernel which is applied to the comprehensive range of conditions within the scope of the reliability based design procedure, standard or suite of standards. The treatment of variable actions served as the trigger for developing reliability procedures to be able to specify safe but economical design loads. Therefore the initial attention has been, and still is on models and calibration of actions.

It turns out that structural resistance is also of significance and importance. On the one hand this is simply due to the variability of material properties and uncertainties in modelling and predicting resistance. On the other hand the provision of sufficient resistance is the explicit process through which reliability can be achieved. Somewhat more subtle is the dominant role of gross error in actual structural performance, with quality management being the related counter measure; all related to structural resistance (see Section 1.5.4). Although the attention given to the reliability basis of structural resistance is more subdued, progress is nevertheless substantial and significant.

The reliability assessment of structural concrete therefore formed part of the investigations presented in this dissertation. The initial investigations reported by Retief, Maritz et al (2002) and Ter Haar and Retief (2002) (see Section 3.1.2) were followed up by a combination of generic reliability assessments and specific investigations on aspects of structural concrete performance such as shear resistance and crack width prediction reliability.

6.1 Standardised Design of Concrete Structures

6.1.1 Basis of structural concrete design

An initiative to consider the development of an African Concrete Code (ACC) provided the opportunity to consider the basis of design for structural concrete in a comprehensive manner as it would apply to a stand-alone design standard (Retief, Dunaiski et al 2006). The assessment was done by considering the nature and function of the basis of design in general terms; then specifically in terms of an ACC; lastly by reviewing reliability issues. The assessment was done by considering specific provisions for structural concrete reliability for an ACC, and serving as common basis for related design standards such as for loading, foundation design and other structural materials that could be used together with concrete in a single facility. The assessment was referenced to Eurocode and based on South African experience in the adaptation of related Eurocode standards.

The technical scope of the basis of structural design can be summarised as:

- Requirements for the performance of the structure;
- Procedures to ensure that all the requirements are considered in the design process;
- Reliability levels with which the requirements comply during the design life of the structure;
- General design approach to be taken, consisting of
  - The specification of limit states for safety and functionality;
  - Identification of design situations with sufficient severity to represent all possible conditions;
- Specification of the basic variables for actions, materials and geometric properties of the structure and the application of the actions;
  - Partial factors to be applied to the basic variables to obtain design values for the various limit states and design situations;
- The application of structural analysis to establish the distribution of internal forces in the structure and its displacements; the use of testing in the design process;
- Design verification procedures for the design situations and structural resistance.

The following scheme nominally follows a table of contents of an ACC formulated from a basis of design perspective:

- Objectives, context and technical basis of the standard
- Scope of structures and aspects of design provided for
- Basis of structural design, including the general performance of the structure, in addition to structural concrete design requirements
- Materials specifications for concrete, reinforcement and prestressing steel
- Provisions for durability
- Structural design, including structural analysis, limit states, the respective failure modes for the various structural elements
- Detailing, providing general specifications and provisions for specific structural elements
- Special applications such as precast construction, light weight aggregate concrete structures
- Specification of construction procedures

The main elements of the design verification scheme, conventionally formulated in the Loading Code or a separate Head Standard, consist of the following:

- Limit states: Ultimate; Serviceability; Durability
  - Ultimate Limit state: Equilibrium; Structural; Fatigue; Geotechnical
  - Serviceability Limit state: Irreversible; Reversible; Quasi-permanent
- Design situations
  - Persistent & Transient
  - Accidental: Robustness; Fire (treated through prescriptive rules); Seismic (provide for seismic actions and earthquake resistance)
- Characteristic values: Actions; Material properties; Geometry
- Partial factors
  - Actions: {permanent; variable & combination}
  - Resistance: {materials & modelling; design situations; failure modes}
- Verification rules
  - Relations between actions and resistance
  - Structural resistance: Failure modes

As an example provided to demonstrate the effects of the alternative design procedures Figure 6.1 compares the two Eurocode schemes Equation 6.10 and Equations {6.10 a & b} and the dual SANS 10160 {a & b} expressions for a representative case for structural resistance with a coefficient of variation V_R = 0,15 and representative models for self-weight and imposed loads.

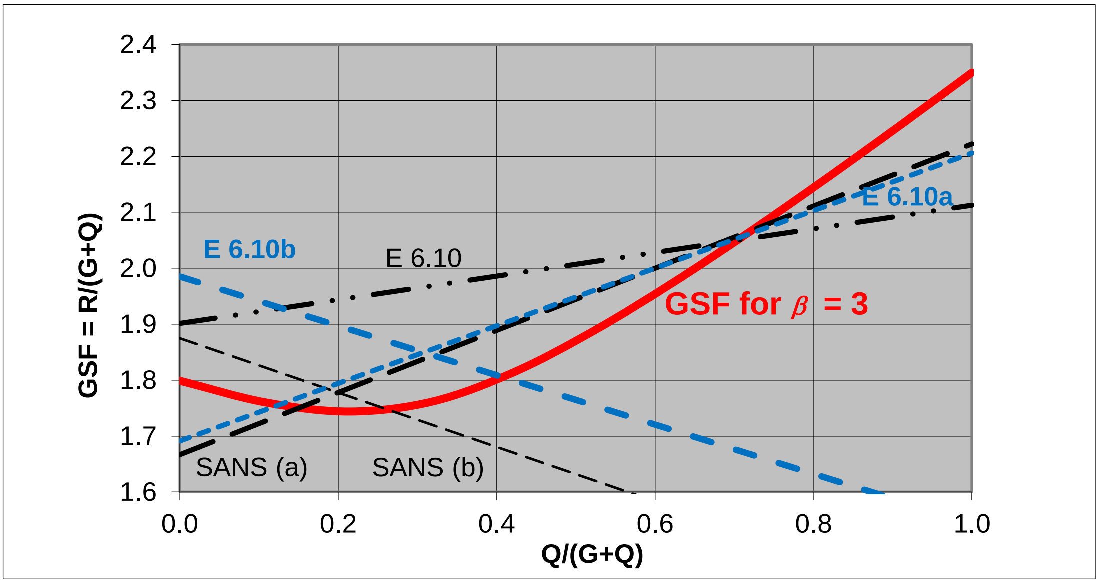


Figure 6.1 Alternative action combination schemes, compared to the target of β = 3,0

A survey of the reliability basis of design for structural concrete applied in Eurocode as seen from the South African context is presented by Mensah, Retief, Barnardo (2010). An important consideration was to verify that SANS 10160-1 is sufficiently equivalent to Eurocode EN 1990 to be able to adjust EN 1992-1-1 for structural concrete to South African conditions. Of particular interest is the parametric analysis to assess the way in which quality control measures can be used to adjust the design outcome for structural reliability classes (RC1 – RC4) other than the reference class RC2 (see Section 4.3.7 and Figure 4.12). In Figure 6.2 it is demonstrated how adjustment of quality control levels can be used to comply with higher reliability classes, instead of a conventional adjustment of partial factors. At the same time the sensitivity of partial factors to the level of quality control can be observed. Such sensitivity is of particular significance when Eurocode design verification procedures are transferred to South African conditions.

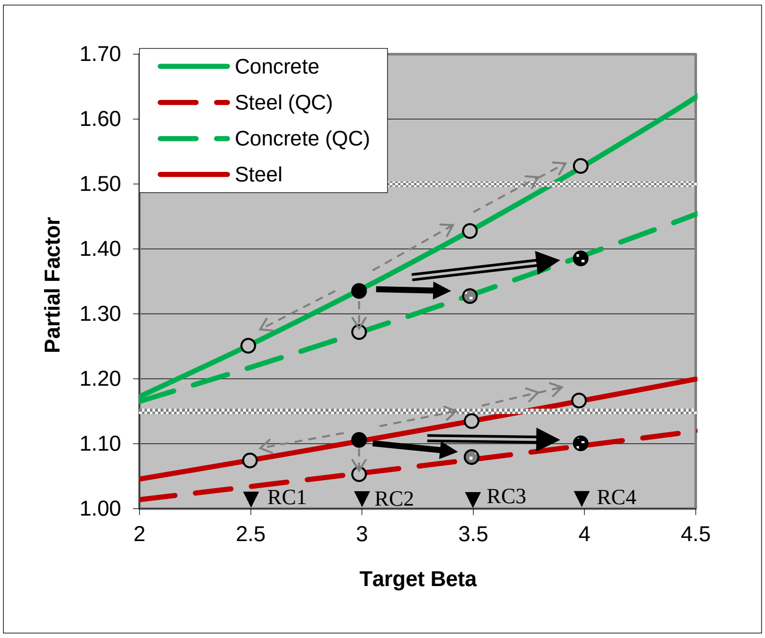


Figure 6.2 Adjustment for Reliability Classes {RC1; RC3; RC4} from RC2 by improved Quality Control (QC) instead of adjusted partial factors for steel and concrete

6.1.2 Reliability analysis for structural concrete

Parametric studies to derive partial material factors for representative reinforced concrete members as a function of various design parameters are presented by Holický, Retief and Dunaiski (2007). The significance of model uncertainty is demonstrated by including information on model factor statistics based on datasets derived from published test results. The sensitivity of model factors to different design models, failure modes and certain design parameters is also demonstrated, as derived from datasets of experimental tests. The reported assessment served as an important indicator to future investigations on the reliability performance of structural concrete.

A more detailed assessment of the derivation of partial factors for the design of structures is reported by Holický, Retief and Wium (2010). The results of a reliability based approach to define values for steel and concrete resistance variables (material factors) as it can be used in the revised concrete design code is presented. The assessment was motivated by the pending adoption of Eurocode EN 1992-1-1 to replace SANS 10100, which had been based on the discontinued BS 8110. Reliability functions were set up for reinforced concrete slabs, beams and short columns. Basic variables for materials, geometry and nominal model uncertainty as compiled for Eurocode were extracted by Holický from working material for the JCSS Probabilistic Model Code (JCSS 2001). The assessment was however done in terms of the South African basis of structural design as given by SANS 10160-1:2010.

The assessment was done considering the influence of the reinforcement ratio and sensitivity of the results to model uncertainty. Standard First Order Reliability Method (FORM) analysis and calibration techniques were used, together with extended assessment of sensitivity factors to determine the relative contributions of the basic variables. Sample results of partial factors for basic variables as a function of section geometry is shown for slabs and columns respectively in Figure 6.3, with the corresponding sensitivity factors shown in Figure 6.4 serving as diagnostic tools.

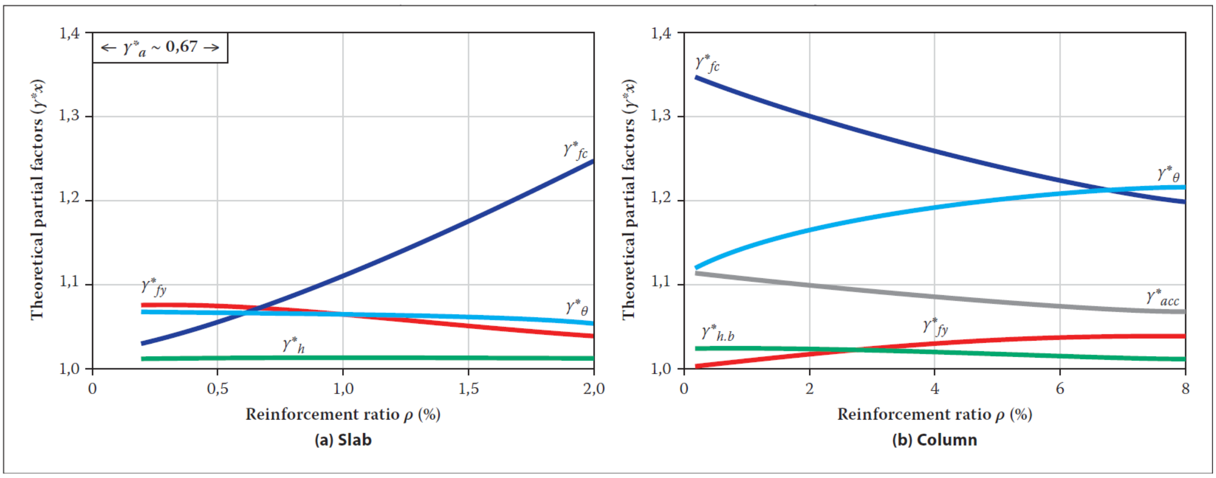


Figure 6.3 Theoretical partial factors (γ^*_x) for slabs and columns.

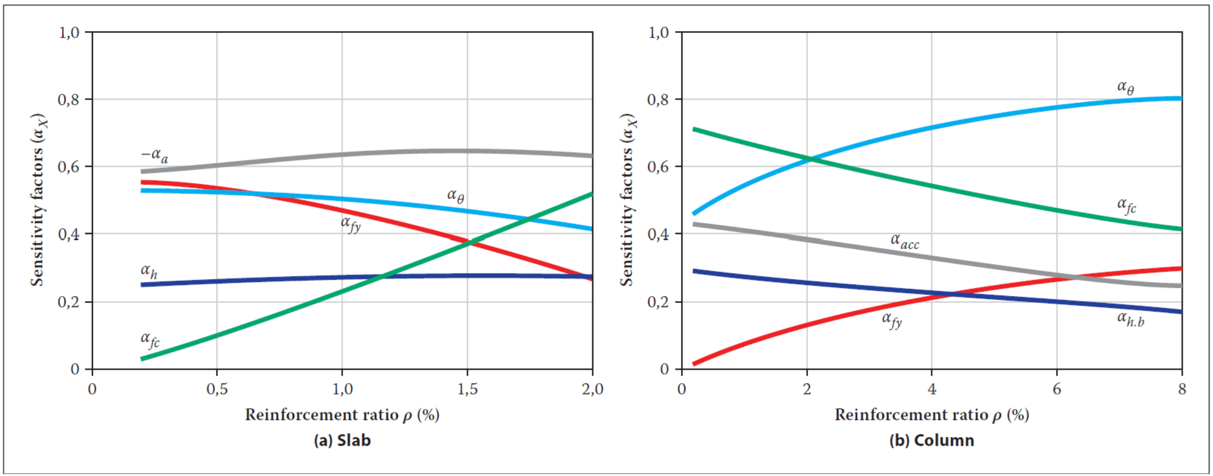


Figure 6.4 Sensitivity factors (α_X) for slabs and columns.

The more detailed assessment confirmed the observations from the initial analysis (Holický, Retief, Dunaiski, 2007) on the different reliability behaviour of different reinforced concrete elements in terms of derived partial factors as a function of reinforcement ratios. Specified characteristic material properties play a significant role in reliability performance, confirming the importance of quality control measures. It is nevertheless concluded that in terms of present South African practice of the target reliability for resistance of $\beta_{R,T}$ = 2,4 and partial factor scheme of material factors, values of γ_s = 1,10 and γ_c = 1,4 are sufficient, and also provides for the effects of modelling uncertainty and geometry across the operational range of steel reinforcement ρ for the two classes of structural elements. It is also however clear that the partial factors do not only reflect the effects of material strengths, but also provide for other sources of uncertainty which are applied at unfactored nominal values in design expressions. Further research on more representative sets of failure modes, model uncertainty and the role of quality control is recommended.

6.2 Structural Concrete Shear Resistance

An assessment of the reliability performance of the shear resistance of regular reinforced concrete sections with shear reinforcement in the form of stirrups represents a case where the scope of the investigation exceeds even the most severe category of model factor investigation presented by Holický, Retief and Sykora (2015): Instead of providing for a case where model uncertainty dominates the reliability performance for the failure mode, the design function under consideration as such demands careful assessment. Alternative ways are needed in which the representation of a General Probabilistic Model (GPM) for shear resistance is expressed. This extended analysis places additional demands on the test dataset for the determination of model factor statistics: The complexity of the structural mechanics of shear resistance, even when restricted to the failure mode of stirrup yield, requires an extensive test base for assessment and places demands on approximate design methods that need to be discerned; particularly the nonlinear behaviour of shear resistance versus the amount of shear reinforcement. The test data should also be sufficiently extensive to be able to characterise correlations between shear predictions for alternative models and levels of approximation across the range of design parameters within the scope of application under consideration.

6.2.1 Scope of Investigation

The essence of the investigations is published by Huber (2005), Mensah (2012, 2015), with specific aspects of the investigation reported by Huber, Retief and Wium (2004), Mensah, Barnardo-Viljoen and Retief (2013), Mensah, Retief and Barnardo-Viljoen (2013a; 2013b; 2013c). However, due to the preliminary nature of these results, the emphasis of the contributions reported here is on the methodology of the investigation. Results are presented to illustrate how improved insight into shear resistance and its reliability performance directs the next steps in the investigation.

The investigation can be characterised as the interplay between the three main components of a typical reliability assessment; however defined in this case in more general terms than just the assessment of a single performance function, due to the complexity of the problem:

(i) **Design function under investigation:** Initially the SANS 10100 method (similar to BS8110) was assessed; compared to the Eurocode ENV function (ECV2); extended to a Modified Compression Field Theory (MCFT) design method. Finally the EN 1992-1-1 (EC2) Variable Strut Inclination Method (VSIM) was assessed, complemented by a modification (VSIM*), the Response 2000 (R2k) best estimate MCFT procedure and various *fib* Model Code 2010 Levels of Approximation (LoA 1 & 3).

(ii) **Test dataset:** An initial dataset of 99 cases (DS-1) was used for the SANS investigation, extended in the follow up campaign to a set of 222 tests (DS-2) complying with the requirements for the application of VSIM; with a subset of 116 tests (DS-2A) for which sufficient information is reported to be able to use R2k.

(iii) **Reliability modelling and complementary assessments:** Two separate sets of GPM functions were used, in both cases requiring numerical solution of the performance functions due to the non-analytic nature of the functions. The GPM included model factors derived from the corresponding datasets listed in (i).
   (a) GPM for initial investigation: Model factor statistics were obtained from DS-1 for the SANS, ENV2 & MCFT models.
   (b) GPM for the final investigation: VSIM; VSIM* & R2k/MCFT representations of reliability performance were made.
   (c) Complementary assessments included various approximations for the representation of basic variables in the GPM;
   (d) Comparisons were also made between the design values of the alternative procedures for representative cases;
   (e) Best estimate predictions played an important diagnostic role in the inter-comparison between different models and experimental results as well as trends as a function of selected design parameters, namely the amount of shear reinforcement and concrete strength.

6.2.2 Comparison of shear design values

The initial comparison of SANS 10100 shear design procedures to the two reference approaches used by Eurocode EN2 and the MCFT AASHTO limit States Bridge Code reported by Huber, Retief and Wium (2004) became more acute with the decision to adopt EN 1992-1-1 as the South African Concrete Code. A comparison of design values for shear resistance for the three methods is shown in Figure 6.5 as a function of the amount of shear reinforcement. The shear reinforcement is conveniently expressed as the ratio $A_s f_y / bs$ where A_s, f_y and s are the section area, yield strength and spacing of the reinforcement and b is the beam width. Figure 6.5 indicates the relative conservatism of the SANS method in comparison to the reference cases. The wide range of values and the relatively high values of the EN2 approach indicates the need for further assessment. The sensitivity of the AASHTO method for the moment/shear force ratio (M/V) in [m] units provides an indication of one of the sensitivities of shear resistance.

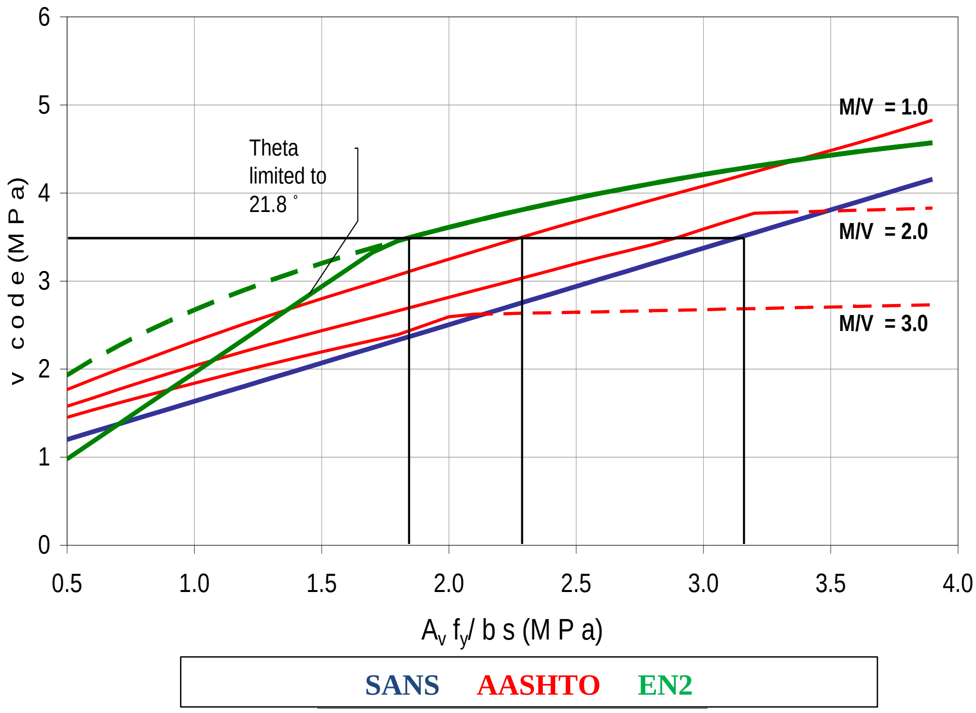


Figure 6.5 Parametric assessment of shear resistance as a function of stirrup reinforcement for alternative design methods

6.2.3 Indicative model factor statistics

Model factor statistics are indicative of the aggregate prediction function performance across the parametric range of the test dataset. For this purpose model uncertainty can be expressed in terms of a model factor (θ) as defined by the multiplicative relationship [6.1]:

$$R(\boldsymbol{X},\boldsymbol{Y}) = \theta(\boldsymbol{X},\boldsymbol{Y})\, \mathrm{R}_{\mathrm{model}}(\boldsymbol{X}) \qquad (6.1)$$

where R is the true response of a structure as estimated from test results and structural conditions; $\mathrm{R}_{\mathrm{model}}$ is the model resistance as an estimate of the resistance based on a structural mechanics model; $\boldsymbol{X}$ is the vector of basic (random) variables X_i included in the model; and $\boldsymbol{Y}$ is the vector of variables neglected in the model, but possibly affecting the resistance.

A summary of model factor statistics is presented in Table 6.1 for alternative shear prediction models (Holický, Retief, Sykora 2015) in terms of the mean (μ_θ) and standard deviation (σ_θ), reflecting the systematic bias from test values and dispersion around the mean respectively. A basic interpretation of the statistics is to determine the bias in terms of the number of standard deviations (#σ_θ) from the unbiased value of 1.0 indicating unconservative realisations of θ; or the probability of exceeding this limit P(θ < 1.0), assuming a normal distribution as a second moment approximation.

The degree of dispersion is particularly important in reliability terms, specifically so for the VSIM model under investigation. As a reference point the dispersion of shear resistance resulting from shear link failure can be expected to be similar to a coefficient of variation for steel strength of around 0,06 – 0,08. By comparing σ_θ given in Table 6.1 with the dispersion of steel strength, it can be projected that the model factor θ completely dominates the reliability performance of shear resistance. Significant differences in μ_θ provide an indication of systematic effects or bias of the prediction method, with values < 1.0 signifying systematic unconservatism. The bias can be characterised by expressing it in terms of the number of standard deviations from the unbiased value of 1.0 ($\#\sigma_\theta$) or the probability of unconservative model factor realisations P(θ < 1.0); as tabulated for the various cases in Table 6.1.

Table 6.1 Summary statistics for the Model Factor (θ) for alternative shear prediction models.

Assessment	Initial Assessment			Extended Assessment		
Prediction Model	BS	VSIM (99 Tests)	MCFT	VSIM	VSIM* (226 Tests)	MCFT (116 Tests)
Mean μ_θ	1.23	1.36	1.15	1.65	0.84	1.14
StdDev σ_θ	0.20	0.38	0.22	0.50	0.18	0.20
$\#\sigma_\theta$	1.17	0.95	0.69	1.31	-0.91	0.68
P(θ < 1.0)	0.12	0.17	0.25	0.09	0.82	0.25

6.2.4 Model factor trends

The next level of interpretation of model factors is to consider trends with related design parameters to reflect dependencies not properly being taken into account in the relevant prediction method. Whereas bias can relatively easily be taken care of through the introduction of a suitable partial model factor (Dithinde & Retief 2014), parameter dependencies can only be resolved through a more fundamental adjustment. Alternatively it can be tolerated, by accepting (known) inconsistency in reliability, with some concern about regions of limited reliability.

The significant trend of the model factor for VSIM (θ_{VSIM}) against the product ($\rho_v f_{yv}$) of the shear reinforcement ratio and shear steel yield strength is well established, as shown in Figure 6.6 (Mensah, Retief, Barnardo-Viljoen 2013b). In fact, this issue, also reflected by the model factor statistics reported above, is the main driver for this investigation. Accordingly, the main concern is the significant decrease in θ_{VSIM} with increasing amount of shear reinforcement $\rho_v f_{yv}$.

When the design constraint on the strut angle (θ_s) of cotθ_s < 2.5 (or θ_s > 21.8°) is removed, the correlation between the corresponding model factor ($\theta_{\mathrm{VSIM*}}$) is removed. This effect led to the introduction of VSIM* as a possible prediction model to serve as basis for the GPM for shear resistance. The absence of such correlation of the model factor for the more advanced R2k model for the MCFT θ_{R2k} with shear reinforcement provides the basis for considering this prediction method as the basis for an alternative GPM unrelated to the VSIM model under investigation.

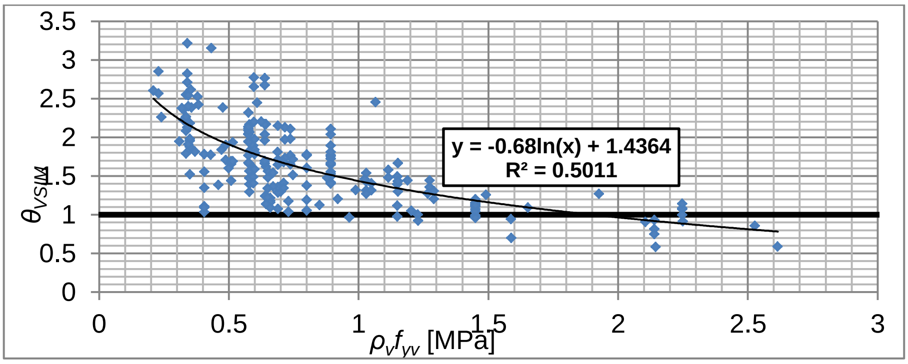


Figure 6.6 Model Factor (θ_{VSIM}) for VSIM in relation to the amount of shear reinforcement

Correlations between the model factor and other design variables were also assessed for the alternative prediction methods R2k and VSIM-A where the limiting angle of 21,8° is not applied. Mildly significant correlations were obtained, as tabulated in Table 6.2, indicating the Pearson correlation coefficients for the shear reinforcement ($\rho_v f_{yv}$), shear span to depth ratio (a/d), longitudinal reinforcement (ρ_l), shear section width (b_w) and depth (d) and mean concrete strength (f_{cm}). The significance of the correlations can be considered in interpreting calibration results.

Table 6.2 Pearson correlation coefficients between model factors for alternative prediction models and design parameters.

Design Parameter	VSIM	VSIM-A	R2k
$\rho_v f_{yv}$	-0.66	0.01	-0.24
a/d	0.12	0.29	-0.12
ρ_l	0.06	0.36	-0.25
b_w	0.09	-0.36	0.25
d	0.06	-0.45	-0.07
f_{cm}	0.13	-0.38	-0.02

6.2.5 Best estimate assessment

The importance of mean values of the model factor and trends with design parameters motivated an investigation on the characteristics of prediction values for shear resistance provided by the various models. This could furthermore be compared to the characteristics of the test data. The basis for the comparison is taken as $\rho_v f_{yv}$, not only as a significant trend factor for VSIM, but also as the primary shear design variable. An updated version of the results presented by Mensah, Retief and Barnardo-Viljoen (2013c) is given in Figure 6.7, showing the shear strength normalised as a mean shear stress $V/b_w d$ versus shear reinforcement $\rho_v f_{yv}$.

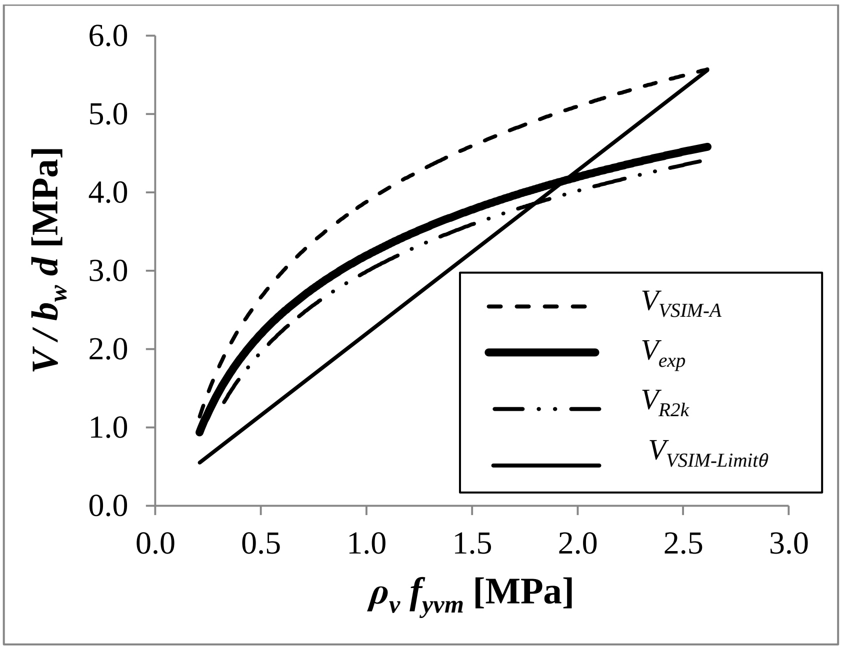


Figure 6.7. Trend of $V/b_w d$ vs. $\rho_v f_{yvm}$ for experimental results compared to trends of VSIM, VSIM-A and R2k

The most striking feature of the trend of test results is the nonlinearity of shear resistance, expressed as a stress, with $\rho_v f_{yv}$; showing a significant decrease in gaining strength with an increase in stirrup reinforcement. The curvature is particularly strong with $\rho_v f_{yv}$ at about 0.5 MPa, approaching a linear relationship beyond 1.5 MPa. This trend is clearly indicative of the underlying structural mechanics behaviour of stirrup failure. Note that the database was carefully screened to include only this mode of shear failure.

The linear behaviour of VSIM clearly does not match this trend, firstly substantially underestimating shear strength, then unconservatively overestimating it at higher values of $\rho_v f_{yv}$. This mismatch provides an explanation of the trend of $\{\theta;\ \rho_v f_{yv}\}$ shown for VSIM in Figure 6.6.

In a similar manner the trends of VSIM-A (where the limit to cotθ < 2,5 is released) and R2k predictions explain the much improved model factor statistic for these models: Similar and improved values for the dispersion (σ_θ) directly follow from the similarities in the shape of the corresponding graphs. Although the direction of the respective biases (μ_θ) shown in Table 6.1, being less than and larger than 1.0 respectively, is consistent with the results shown in Figure 6.7, the magnitude of bias is not consistent. This inconsistency can be explained by the fact that the VSIM-A and R2k graphs do not fully reflect the wide range of design parameters aggregated into the graph of test results, as can be inferred from the range of Pearson correlation coefficients listed in Table 6.2.

6.2.6 Reliability assessment of VSIM

The results of the reliability performance of the EC2 VSIM shear design method as obtained by Mensah (2015) are shown in Figure 6.8. The intention of the presentation of the results is not so much to interpret it in terms of its significance to the reliability performance of the VSIM design method, as important as this certainly is. Of relevance at this stage is the ability to discern the influence of significant design factors, identified as the amount of shear reinforcement and concrete strength, through reliability analysis. Furthermore the degree of agreement and differences between the reliability models VSIM-A and R2k that were applied is of significance.

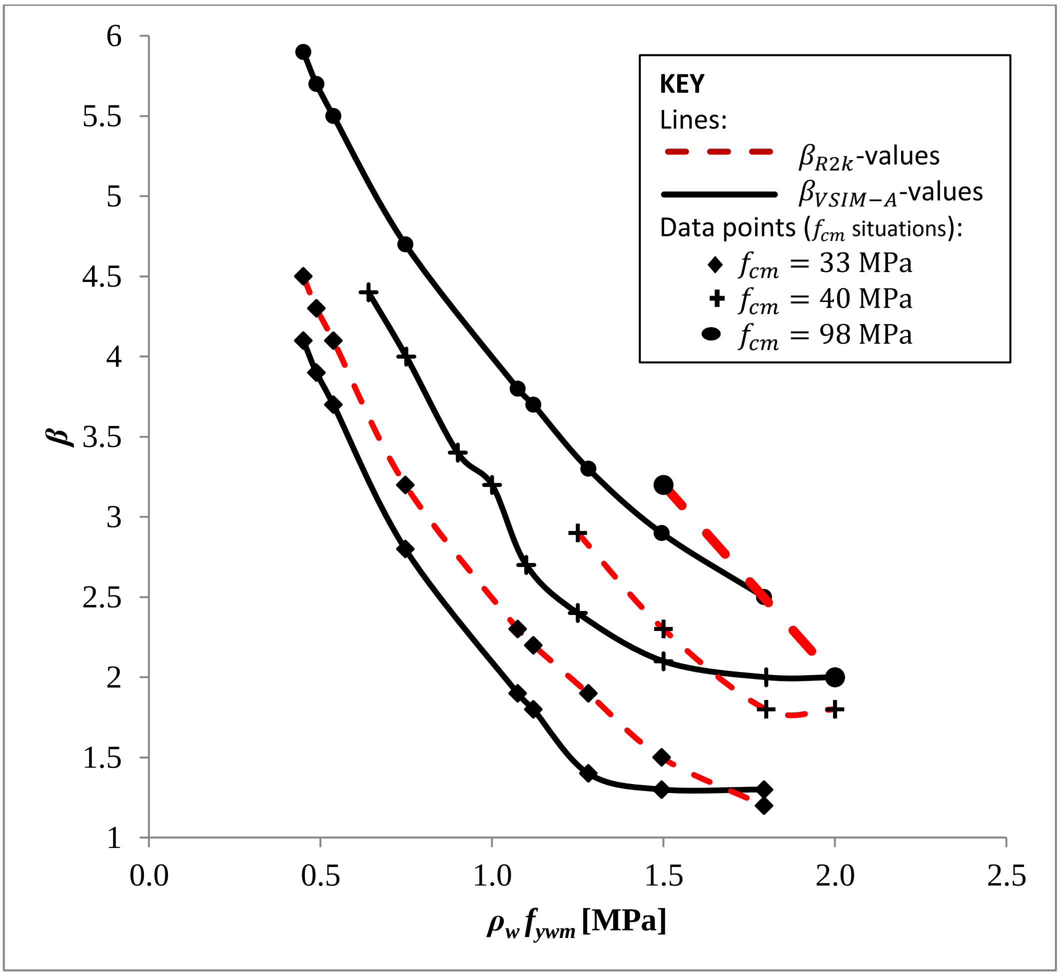


Figure 6.8 β_{VSIM-A} *and* β_{R2k}*-values compared parametrically against* $\rho_w f_{ywm}$ *and* f_{cm}

On the one hand it can be appreciated that the results significantly enhance an understanding of the reliability performance of this particular shear design method, not only characterising a significant trend, but providing insight in the mechanism at the root of such a performance. On the other hand remaining differences indicate that assessment to a next level of detail would be justified, unless the more rational approach of modifying, even replacing the method is considered. It is nevertheless clear that all avenues of investigation have not yet been exploited.

6.2.7 Reliability-based Research Methodology

The following observations can be made on multi-dimensional and –level nature of the investigation of the reliability performance of concrete shear design procedures, even including an extended version to consider modelling uncertainty as an important component of such an investigation:

- The experimental database serves not only as basis for the characterisation of a model factor, but also to assess the performance of the model under investigation as such through the identification of systematic dependencies with design parameters.
- Further extension is achieved by considering alternative shear prediction models as basis for General Probability Models for shear resistance.
- Extension provides the opportunity to consider the respective and combined effects of systematic biases and trends of best-estimate prediction models and various sources of uncertainty.
- The extended approach can be classified as prediction model assessment and development, as an extension of the conventional reliability assessment of a specific standardised design procedure.

An extended investigation utilises three main sets of information as represented by

(i) The experimental dataset
(ii) The structural mechanics predictions model(s) used as basis for design
(iii) The reliability representation of the performance function.

In addition to the combined use of all three sets of information, various stages of the investigation consist of pairwise consideration of two sets of information, with:

(1) **Model validation:** {(i) & (ii)} representing conventional validation of structural mechanics models by comparison to test results, usually consisting of a limited number of tests.
(2) **Reliability assessment:** Conventional reliability assessment and calibration consist of considering {(ii) & (iii)}.
(3) **Uncertainty and trend assessment:** However, the reliability interrogation of the test dataset {(i) & (iii)} can also reveal biases, trends and uncertainties.
(4) **Model building:** Bias, trends and uncertainties can assist with model building {(i); (ii) & (iii)}.

6.3 Concrete Water Retaining Structures

An investigation of the reliability basis of design of concrete water retaining structures represents an example of the use of reliability modelling of a performance function for cracking control to assess the serviceability limit state as the controlling design requirement: Progressive steps in the reliability assessment of design crack width procedures are reported by Holický, Retief and Wium (2009), McLeod, Wium and Retief (2012) and McLeod, Retief and Wium (2013), ranging from the formulation of a basic reliability crack width model to the systematic investigation of the influence of design parameters across a representative range of conditions. This specific serviceability limit state forms part of the basis of design that is extended to topics such as the scope of the standard, its relationship to the related standards on structural concrete and loading, a range of reference standards, transition from existing design and construction practice; as reported by Barnardo-Viljoen, Mensah et al (2014).

6.3.1 Reliability of crack width prediction and design

The general design procedures of crack width control in the design of concrete structures are assessed by Holický, Retief and Wium (2009) for the special case of the design of water retaining structures. The scope of an appropriate investigation is identified to consist of the following steps:

(i) Establish the scope of application from a survey of representative structures;
(ii) Formulate the controlling limit state function for cracking in terms of structural behaviour and the associated mechanics models and crack width limits;
(iii) Determine the level of reliability that is suitable for the limit state (irreversible serviceability) and reliability class of water retaining structures as a specialist structure;
(iv) Determine the representation of the relevant actions (hydrostatic load, thermal, shrinkage and creep strains) and their combinations;
(v) Compilation the set of basic variables with their probability models;
(vi) Consider the influence of construction procedures and the associated quality management procedures related to curing and thermal effects.

The paper serves as a pilot investigation considering only an indicative range of conditions for which a reliability assessment of cracking of a section under tension is considered for a representative structure and two levels of limiting crack width. It is demonstrated that crack width serviceability requirements control the amount of reinforcement by a wide margin: As much as 2 to 5 times the amount of reinforcement required for the ultimate limit state is needed for limiting crack widths of between 0,2 mm to 0,05 mm. It is demonstrated that the large amount of reinforcement required for crack width control can be reduced by between 15% and 30% when reliability based design is used instead of codified design.

The scope of the investigation is extended as reported by McLeod, Wium and Retief (2012) to include flexural and tension cracking for a comprehensive range of range of structural geometries, loading conditions and crack width limits as specified for water retaining structures. In addition to the probabilistic representation of basic variables, model uncertainty is represented explicitly in sensitivity analyses around best estimate information on reasonable models. The influence of the range of design parameters is investigated parametrically with the results shown in reliability analysis fashion of level of reliability achieved. A follow-up investigation presents the outcome of a similar calibration assessment in terms of partial design factors required to achieve a set target level of reliability that can be associated with stringent serviceability performance requirements (McLeod, Retief and Wium 2013).

The outcome of the investigations is to provide an assessment of the reliability performance of present design procedures, an understanding of the most important design conditions and sources of uncertainty influencing the reliability of crack width design and the identification of remaining research requirements needed to set design on a rational and transparent basis. It is demonstrated that further investigations on crack width modelling, including model uncertainty, appropriate crack width limits, the setting of optimal levels of reliability are justified by the controlling role of crack width provisions in the design of water retaining structures and the associated economic and service performance implications.

6.3.2 Basis for standardisation of the design of water retaining structures

Two stages of defining the background to the development of a standard for structural design can be identified:

(i) At the start of the standardisation process both the intended function of the proposed or revised standard should be formulated and the related technical basis should be presented as point of departure.
(ii) Such background should be updated to record the basis for decision making, particularly the addition of considerations to provide for experience from practice or additional refined investigations.

Such background is reported by Barnardo-Viljoen, Mensah et al (2014) for the development of a new South African standard dedicated to the design of water retaining structures as derived from pre-normative development and serving as input to an industry representative working group. This background incorporates a reliability perspective, with specific reference to the following considerations:

(i) Crack width design as discussed above represents a critical component of the performance requirements for a design standard, requiring that all the issues need to be resolved in a practical yet rational manner.

(ii) Although a dedicated design standard is intended, it needs to be used together with the general design standard for structural concrete, with its basis of design and actions specified in the South African Loading Code SANS 10160.

(iii) An array of standards serves as reference, with selected material to be represented consistently and equitably in the new standard. Of particular importance is the transition from previous practice, with the obsolete BS 8007 serving as the de facto local standard.

The following considerations related to the serviceability performance of water retaining structures are identified by Barnardo-Viljoen, Mensah et al (2014):

(i) **Performance level for cracking of concrete:** In EN 1992-1-1 concrete cracking is treated as a reversible SLS for which the least severe action combination scheme applies. The implication is also that a low level of reliability performance would apply when reliability analysis and calibration is done.
   a. The importance of WRS as specialist structure may justify a higher reliability classification, with associated performance levels. Generally WRS should be classified at the equivalent SANS 10160-1 Reliability Class 3; requiring upwards adjustment of partial factors and/or levels of quality management.
   b. Similarly the performance requirement for cracking should be set at a higher reliability level due to the importance of this specific limit state. When stating performance levels in terms of target reliability index values (β_T), typical values of β_T = 0,5 applies to cracking in buildings and β_T = 1,5 – 2,0 apply to the irreversible SLS.

(ii) **Reliability Assessment:** A reliability assessment of crack prediction based on the EN 1992-1-1 procedures for representative WRS confirms the importance of a number of factors (McLeod, Retief & Wium 2013):
   a. Reliability based design outcomes are more economical than the stipulated procedures, even when a high level of performance is set. Refined calibration of the design procedures can therefore result in more economical structures.
   b. The design outcome for the cracking SLS is significantly sensitive to the target level of reliability set as performance requirement: The amount of tensile steel for β_T = 1,5 and 2,0 is respectively 10% and 15% more than for β_T = 0,5 as default value.
   c. Nevertheless, the more stringent crack width limits stipulated in EN 1992-3 for WRS often result in a substantial increase in tensile steel required to satisfy performance requirements: When the crack width limit stipulated by BS 8007 of 0,2mm is reduced to 0,1mm or 0,05mm, the amount of tensile steel for a representative case is respectively increased by a factor of 1,4 and 2.

(iii) **Crack limit:** The rational basis for the more onerous crack limits stipulated in EN 1992-3 as compared to BS 8007 needs to be established. Probability based economic optimisation could provide such a rational basis, but would require input on the likelihood of self-healing for a range of crack widths, in addition to quantification of the consequences of SLS failure.

(iv) **Reliability of structural resistance and accompanying quality management:** Although the current version of the SANS 10100-3 (Draft) omits the onerous set of rules pertaining to cracking proposed by EN 1992-3 pending the results of a local support study, it appears likely that Tightness Classes will be prescribed in future and accompanied by locally suitable limits. Such action would call for a revised scheme of quality measures relating to Tightness Classes and allowable crack width limitations to ensure adequate performance of WRS in South Africa.

The development of a standard for the design of water retaining structures represents an unusual set of conditions for standards development, allowing for pre-normative investigations, management of the development, knowledgeable representation from practice to ensure sound engineering judgement and the continuation of investigations of remaining issues. This experience is captured by Wium, Retief and Viljoen (2014).

6.4 Risk Based Maintenance – Flood Protection Structures

The feasibility of extending the application of structural reliability to the development of a rational procedure for planning the maintenance of concrete flood protection structures is presented by Marengwa and Retief (2009a; 2009b). The first paper presents an overall survey of the function of flood protection structures as related to its structural performance, the role of inspection and monitoring in terms of reliability based limit states design, a general assessment of related deterioration and updating of material properties. The process is presented schematically in Figure 6.9.

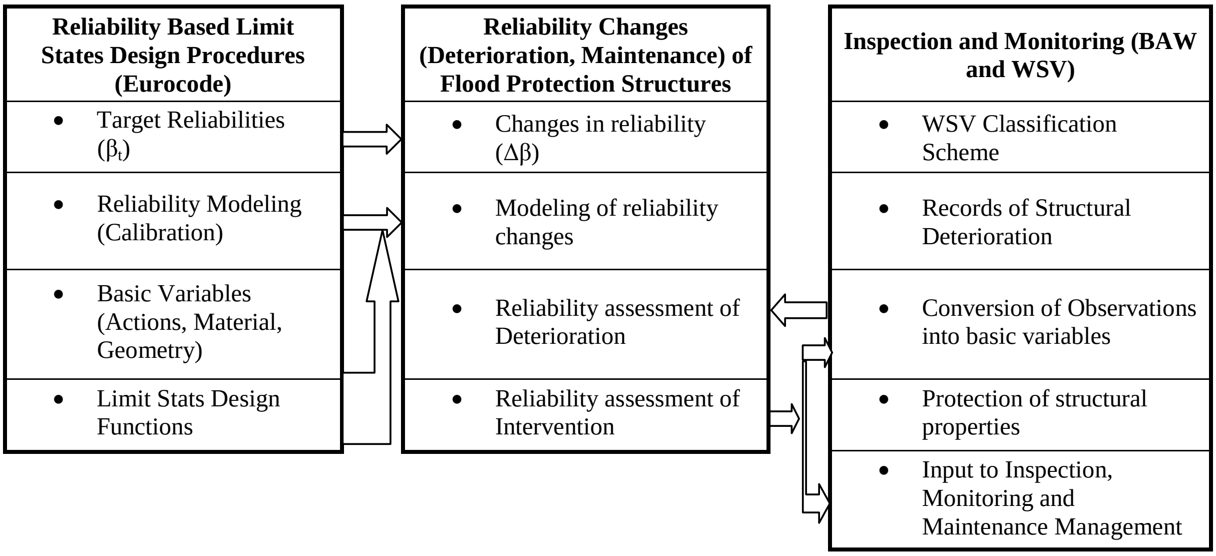


Figure 6.9 The use of inspection to update design information

In the follow-up investigation an outline is formulated of a Bayesian updating approach to devise an inspection and maintenance program to determine critical concrete characteristics and deteriorating effects which can be applied to reassess the reliability performance of the protection structures. Although the planning was ultimately not implemented, it nevertheless demonstrated the utility of applying a reliability based approach to assess the long term performance of critical protection structures exposed to extreme loads and deterioration processes.

6.5 Model uncertainty assessment for structural concrete

The proper treatment of model uncertainty in the reliability assessment of structural performance has been identified at an early stage of the campaign of investigations submitted in this dissertation, firstly specifically for structural concrete but extended in due course also considering load models and geotechnical reliability modelling considering pile foundation design. An outline of model uncertainty investigations related to the background of reliability assessment of design models for structural concrete is presented in this section. A more comprehensive treatment of model uncertainty investigations is presented in Section 7.4.

In the following investigations on reliability modelling of structural concrete, modelling uncertainty was included as a prominent source of uncertainty:

Serviceability deflection: Differences in deflection values obtained from various standardised procedures indicated deficiencies in the prediction models. Nominal comparisons between best estimate predictions for short and long term predictions and a limited set of laboratory deflection measurements provided indicative model factor statistics for the respective design procedures. The outcome of this investigation is presented in Section 3.1.1 as reported by Retief (1996b).

Serviceability crack control design: The critical role of crack width control for the design of water retaining structures served as motivation for the reliability assessment of crack width prediction presented in Section 7.3.1. Due to difficulties in deriving information on model uncertainty for crack width prediction, the initial assessment consisted of sensitivity analysis of best estimates for model uncertainty, as the first step of the investigation (McLeod, Wium, Retief et al 2012).

Flexural resistance: In spite of the relatively well behaved nature of the flexural resistance of under-reinforced concrete members as reflected by good agreement between various design procedures, the characterisation of model uncertainty was done for this case, based on comparisons of best estimate predictions for the various design procedures to an extensive database of test results extracted from the literature, as reported by Holický, Retief and Dunaiski (2007). Inclusion of information on all relevant design parameters and specific measurement of basic variable values forms an important requirement for the dataset. Assessment of possible correlation between model factor and design parameter values ensures the identification of all possible systematic effects or bias and trends that are not properly accounted for by the model.

Shear resistance of sections without stirrups: The derivation of model uncertainty for design models for the shear resistance of concrete sections without stirrups as shear reinforcement represents an investigation on the reliability performance of semi-empirical models (Holický, Retief, Sykora 2015). Model factor statistics were derived from comparison to an extensive dataset that includes again all relevant design parameter and basic value information. Important considerations identified with special reference to semi-empirical procedures include an assessment of the effective selection of parameters and coefficients; the range of application, as related to the dataset of input information to which the model is fitted; the unknown degree of adjustment or bias to account for the dispersion of measured to predicted to shear resistance values; the effectiveness (and reasonableness) of partial design factors in achieving an acceptable level of reliability. The model factor statistics derived from the assessment are presented in Table 6.3 for the semi-empirical models employed by BS 8110/SANS 10100-1 and Eurocode ENV 1992-1-1, in comparison to the more rational Modified Compression Field Theory (MCFT); also considering the influence of shear span to beam depth ratio (a/d) identified from the dataset as an important design parameter. The results indicate that the semi-empirical methods generally provide a more reasonable estimate of model factor bias (mean value close to 1,0) than that obtained for MCFT, but with similar dispersion values given by comparable standard deviation values. The obvious constraint that semi-empirical procedures should not be extrapolated beyond the limits of their underlying database should be noted.

Table 6.3 Statistical characteristics of model uncertainty for shear resistance of concrete sections without shear reinforcement.

Distr. Parameters	TOTAL DATABASE (231 tests)			SUBSET 1: a/d < 2.9 (47 tests)			SUBSET 2: a/d > 2.9 (184 tests)		
	BS	EN2	MCFT	BS	EN2	MCFT	BS	EN2	MCFT
Mean μ_θ	1.08	0.98	1.30	1.27	1.16	1.43	1.03	0.94	1.27
St Dev σ_θ	0.185	0.18	0.22	0.24	0.25	0.32	0.13	0.12	0.18

Shear resistance for sections with stirrup shear reinforcement: Provision for web shear reinforcement in the form of stirrups make a significant contribution to the shear resistance of reinforced concrete sections. However, the extended resistance does not lead to better control and predictability. The range of prediction models and levels of approximation is indicative of the difficulty of capturing the various mechanisms contributing to shear resistance in design procedures. This case is representative of the diagnostic use of model uncertainty investigations to explore the effectiveness of deriving simplified design procedures and the effectiveness of the theoretical model as such, as presented in Section 6.2 above. Such a campaign can be added as a fourth class of the assessment as outlined in categorisation of model uncertainty as presented in Section 7.6.

From the model uncertainty analysis done for concrete as summarised here and other elements of structural reliability reported elsewhere in this dissertation, the view emerges that this topic is not only relatively important in many cases of reliability based design procedures, but it is indeed a useful investigative tool for the assessment of structural mechanics models on which design procedures are based. A more systematic treatment of this observation is presented in Section 8.4.

Chapter 7: Specific Reliability Investigations

In this chapter reliability investigations are presented that are generally related to the standardisation of reliability based design procedures, but goes beyond implementation or background for the present set of standards. In most cases the need for further investigation were however identified during background investigations and calibration for standardisation; in a few cases as pre-assessment of a topic.

7.1 Strong Wind Investigations

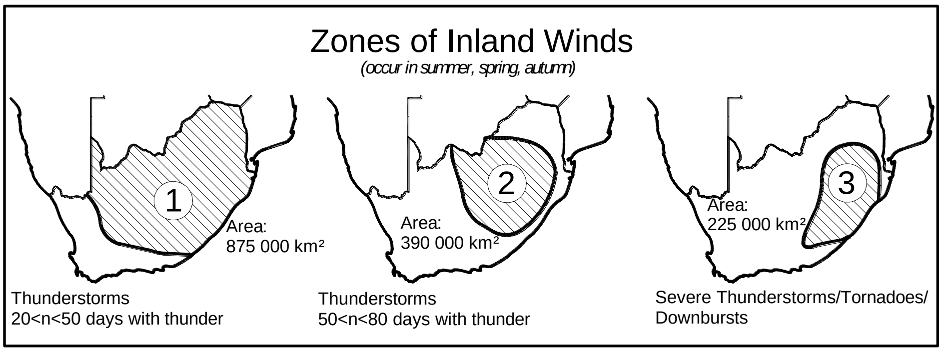
In Section 5.2 investigations focusing on design procedures for wind loading on structures are presented. In this section, complementary investigations on a range of strong-wind phenomena are reported. Initial investigations on strong-wind characteristics across Southern Africa and the phenomenology of damage to engineering and non-engineered facilities led to an extensive investigation on the strong-wind climate of South Africa, in turn providing input into design for wind loading. The ongoing investigation on the reliability characteristics of the time independent wind engineering processes of wind loading reported in Section 5.2.5 represents a case where insufficiencies in wind load reliability was identified as a topic to researched further. During later stages the investigations were extended to more general topics such as the projected effects of climate change on wind loading on infrastructure.

7.1.1 Severe wind phenomena in South Africa

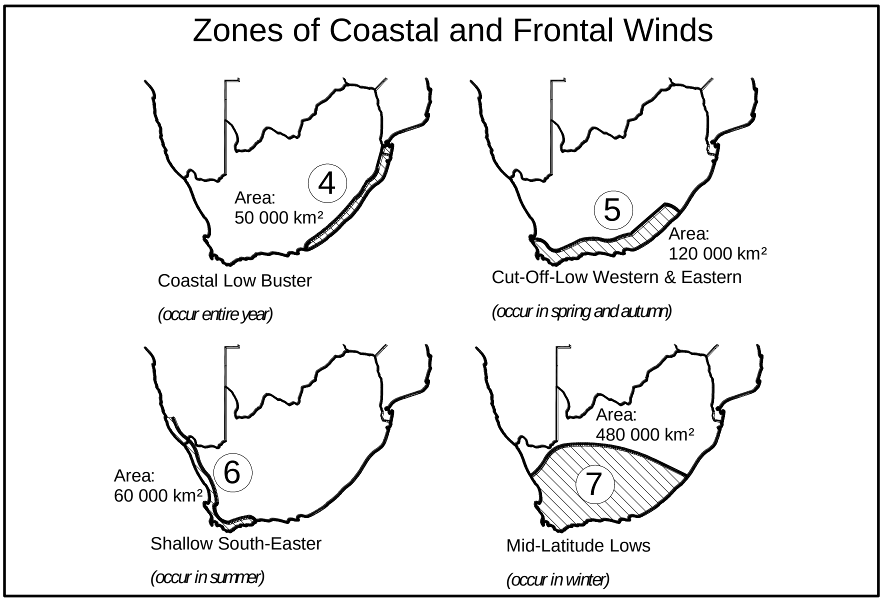
The compilation of a severe wind damage and disaster management support system, as presented above, requires input on severe wind phenomena for South Africa. The identification of various types of strong wind events for the country is presented by Goliger and Retief (2002a). This investigation also serves as the starting point for the characterisation of the strong-wind climate, serving as basis for structural design standards as discussed in Section 5.2.

Two distinct types of extreme winds were identified as dominant, namely convective inland and synoptic coastal winds, with some overlap between the respective regions. The two regions are shown in Figure 7.1, showing further zoning into thunderstorm intensities for the inland regions and different coastal/frontal winds for the synoptic type of wind. This classification serves as basis for characterisation in terms of occurrence rate and magnitudes of wind speed.

The approach taken to characterise South African thunderstorms in terms of generic footprints and rates of occurrence is reported by Goliger, Adam and Retief (2002). Footprint areas are obtained as a matrix in terms of {storm class; wind speed} with a three-level classification for the storm class and wind speed respectively. Idealised schemes of the vulnerability of engineered and non-engineered structures as a function of wind speed is proposed by Goliger and Retief (2007) based on an extensive survey of wind damage records (see Figure 7.2). The proportion of damage expected to happen to various classes was obtained in a similar manner.



(a)



(b)

Figure 7.1 Wind zones for (a) convective inland and (b) synoptic / coastal strong winds

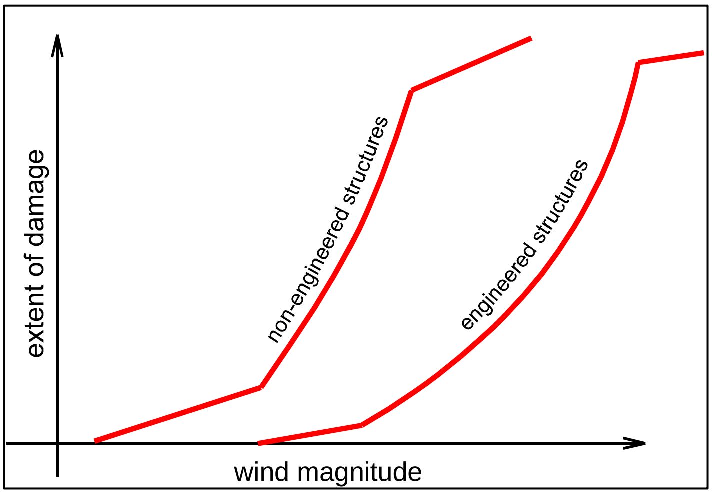


Figure 7.2 Idealised schematic comparison of wind vulnerability of engineered and non-engineered structures

7.1.2 *Wind disaster management model for South Africa*

The background to setting up the development of a wind damage and disaster management support model for South Africa is reported by Goliger and Retief (2001). This initiative was motivated by the record of the yearly distribution of wind damage events for South Africa shown in Figure 7.3. The main components of the model were identified to be related to the wind event characteristics and the vulnerability of the structures and facilities, ranging from engineered structures to informal settlements.

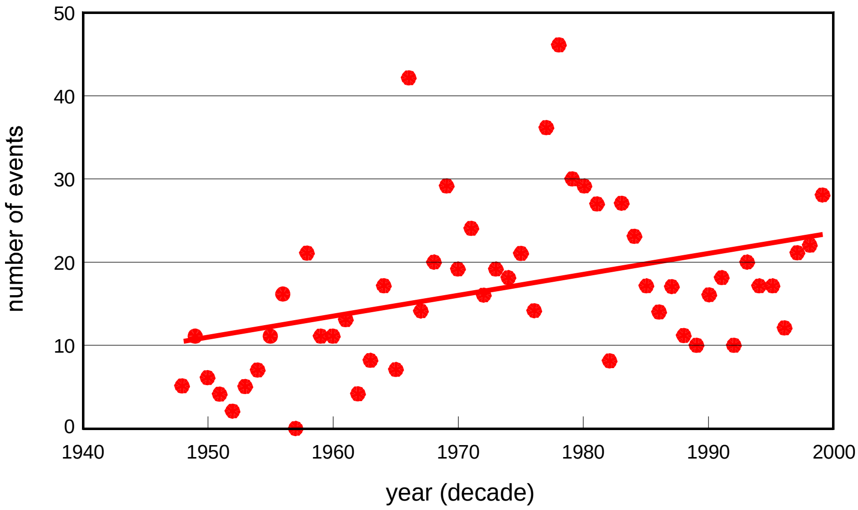


Figure 7.3 Annual distribution of wind damage events in South Africa

The wind damage and disaster management model is described by Goliger, Retief and Niemann (2003a) in terms of a genetic algorithm as shown schematically in Figure 7.4. Loss prediction is based by matching land development factors for an area under investigation with wind action factors. Development factors include the distribution of various classes of assets categorised in terms of wind damage vulnerability characteristics. Wind actions are expressed in terms of strong wind types and associated zones or regions; foot prints of wind events and associated rates of occurrence.

A demonstration of the generic algorithm is given by Goliger, Retief and Niemann (2003b), including representative information on the development characteristics of a selected densely populated and industrialised region in South Africa (Vanderbijlpark magisterial district) and occurrence rates of strong wind events as a function of their severity. From this information accumulated damage rates can be estimated.

An integrative overview of the development process as well as the major components of a wind damage and disaster risk model for South Africa is presented by Goliger and Retief (2004). The basic philosophy of the model, as well as various wind-related and land developmental factors, in the context of the South African conditions are discussed. It is concluded that the model provides the basis for application studies, albeit at a scoping level; as well as a platform for further research and development. It incorporates a number of innovative concepts and insights such as the differentiation between providing for wind loads in structural design and assessing wind damage for the spectrum of facilities. The derivation of wind damage exposure from climatic and spatial characteristics of the associated wind event types is a key element which forms the basis for the integrated wind damage calculation model.

7.1.3 *South African strong-wind climate*

The sparse information on which the design wind map of SANS 10160-3:2010 is based, as transferred from SABS 0160:1989, is treated in section 5.2.2. In this section a more detailed account is given of the investigations made to characterise the South African strong wind climate. The initial investigations reported above are substantially revised as based on extreme value statistical treatment of a new generation of wind records, fully taking account of climatic input to establish strong-wind generating mechanisms.

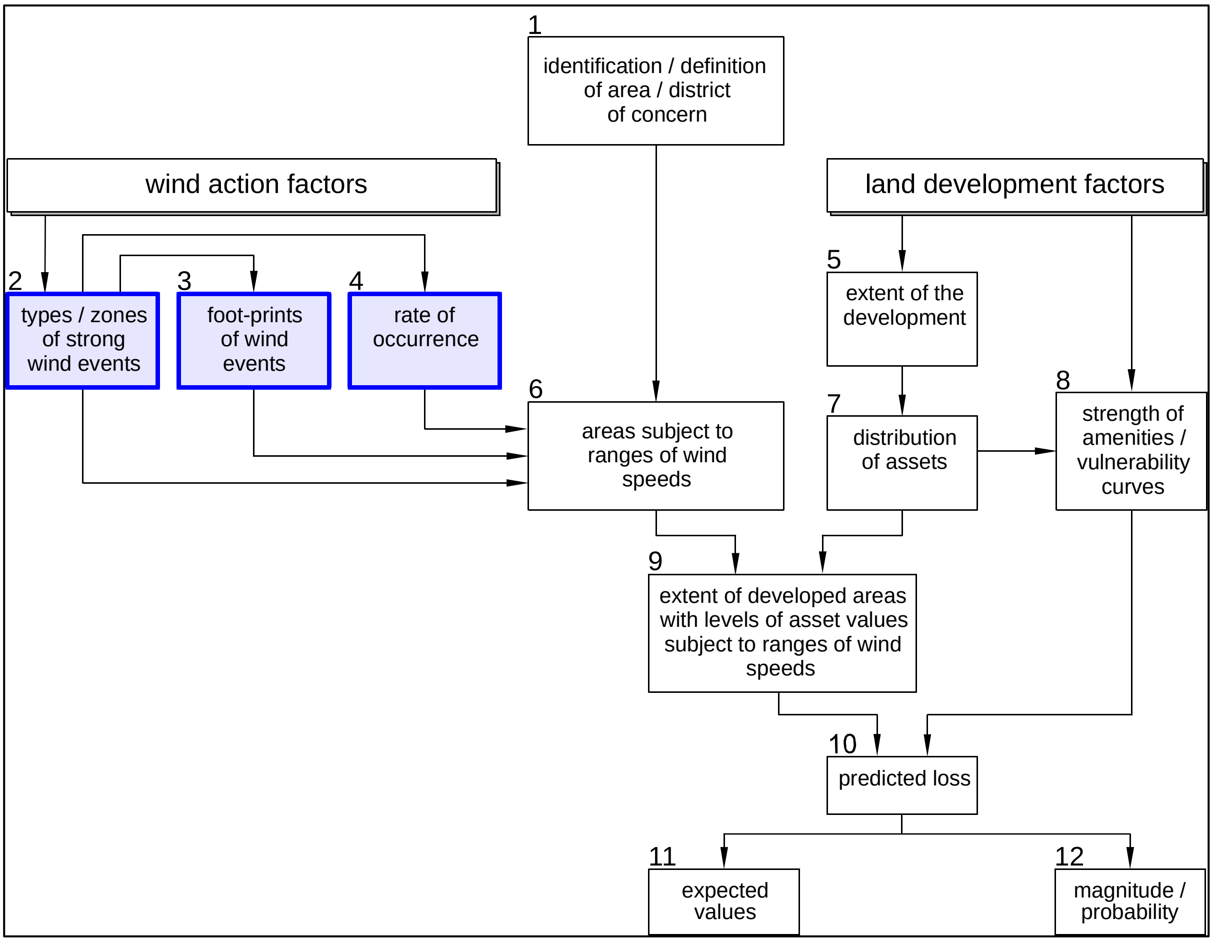


Figure 7.4 *Flowchart of generic algorithm for wind damage and disaster management model*

The difficulties of representing gust wind loads under climatic conditions consisting of a mix of synoptic and thunderstorm winds by the reference Eurocode procedure are discussed by Goliger, Retief and Dunaiski (2009) and Goliger, Retief et al (2009). Accordingly the first investigation centred about the identification of strong-wind generating mechanisms and to classify each observation to its associated conditions (Kruger, Goliger et al 2010). It was found that in addition to thunderstorms, for the other primary condition of synoptic winds four secondary conditions could also be identified, with geographic distribution as shown in Figure 7.5. The extensive overlap of the regions indicates the significant occurrence of regions with mixed strong-wind climates.

The implications of mixed strong-wind wind climates for a location or region are assessed by Kruger, Goliger et al (2012). The implications of not taking the mixed climate into account is demonstrated in Figure 7.6, indicating the substantial underestimation of the extreme wind distribution without the appropriate combination of differentiated strong wind datasets. Clusters of regions with similar dominant and combined climates are derived from the distribution characteristics of collections of AWS stations. Figure 7.7 shows an example of a set of three clusters for thunderstorm winds. Figure 7.8 shows how combinations of clusters are used to compile a map of the strong-wind climate of South Africa, consisting of areas of dominant and mixed climates.

The directional analysis of strong winds under mixed climate conditions is considered by Kruger, Retief and Goliger (2013c). It is demonstrated that the effect of directional separation is dependent on the directionality of different strong wind mechanisms at a given locality and the selection of sector angles. The results nevertheless demonstrate that directional separation is feasible for South Africa despite the complications of the mixed climate and short record lengths.

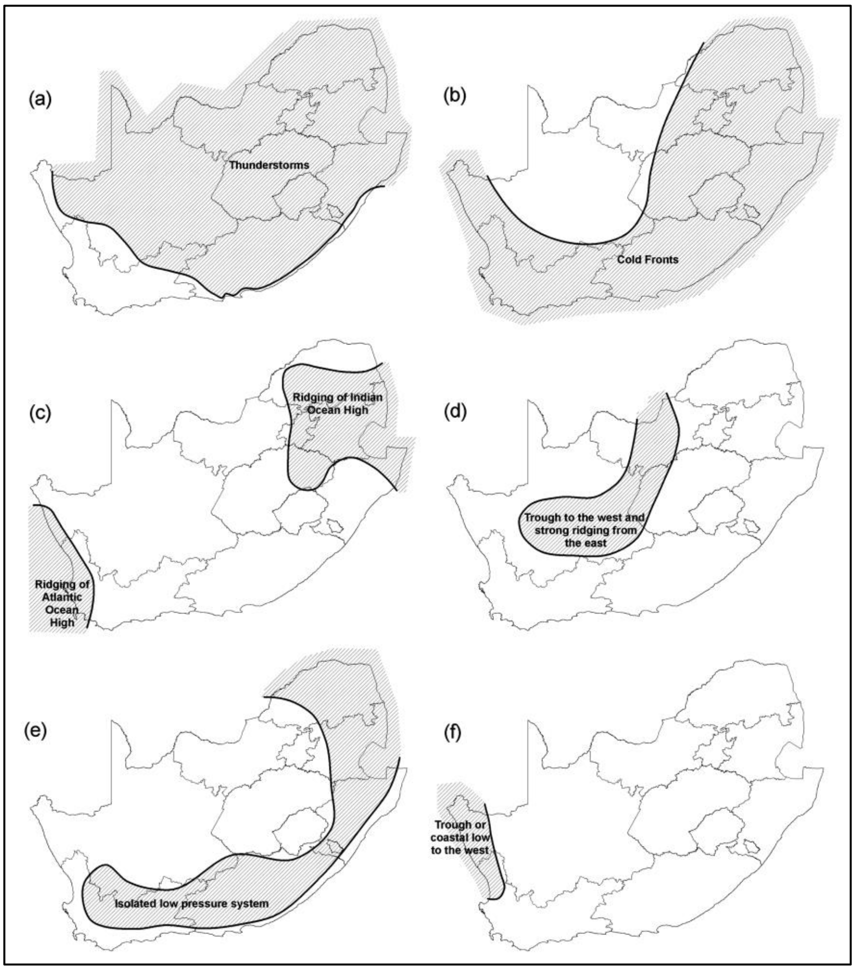


Figure 7.5 Geographical distributions of strong-wind generating mechanisms

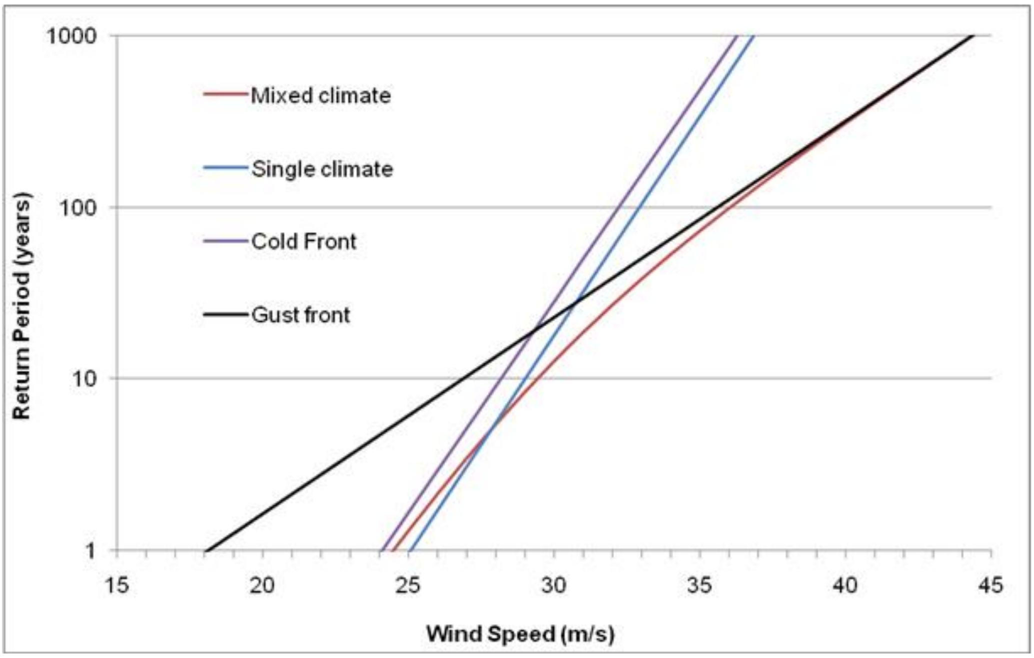


Figure 7.6 Single and mixed climate combinations of cold front and gust front mechanisms for Uitenhage

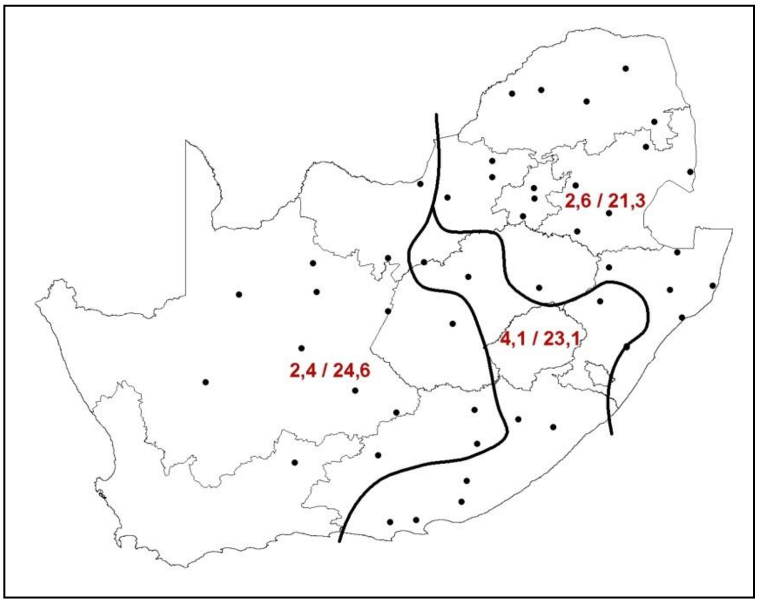


Figure 7.7 Clusters of thunderstorm regions, characterised by Gumbel distribution parameters for each cluster

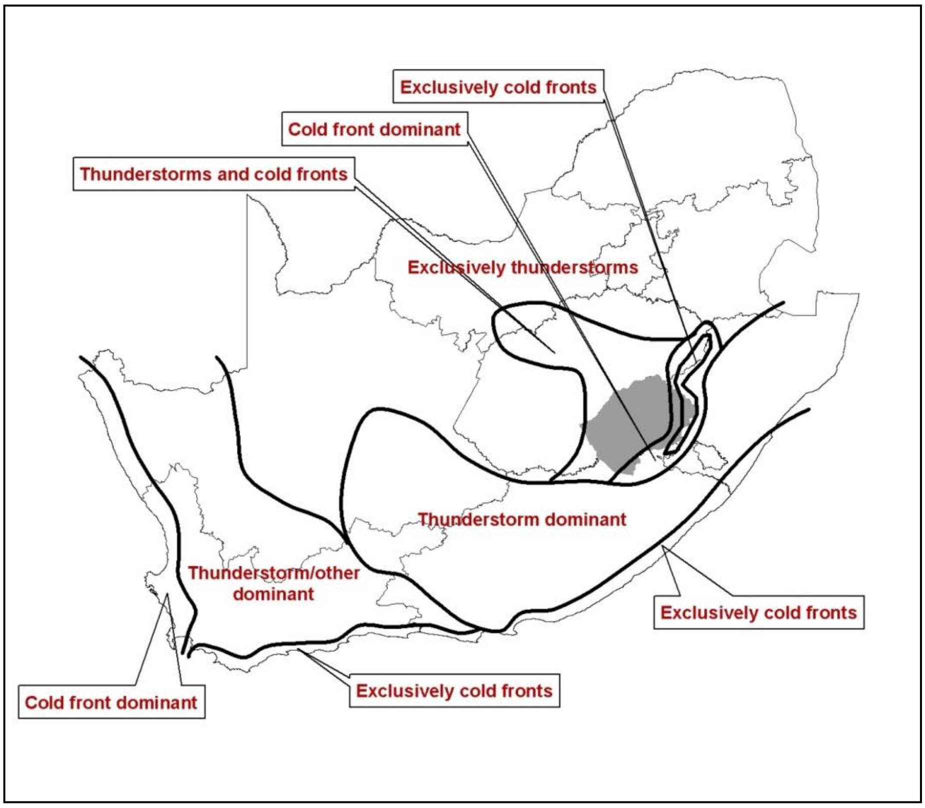


Figure 7.8 Mixed strong wind climate of South Africa

A paper by Goliger, Kruger and Retief (2013) summarises an investigation into the importance of the exposure of wind speed anemometers, based on the current network of South African Weather Service anemometers. The exposure of a large number of stations leads to distortions of the recorded wind speed data. Recommendations are made on taking account of potential deficiencies in the deployment and siting of anemometers to include provision for strong wind observations.

7.1.4 *Strong-wind models – Input to wind map*

The background of the proposed revised wind design map for South Africa presented in Section 5.2 is extensively recorded by Kruger, Retief and Goliger (2013a) presenting the statistical analysis of strong-wind records to derive probability models and (2013b) the mapping of the resulting characteristic wind speeds. In addition to the conventional Gumbel extreme value method, the General Extreme Value (GEV) and Peak-Over-Threshold (POT) with General Pareto Distribution (GPD) and Exponential (EXP) distributions were considered, as summarised in Figure 7.9. Such an array of alternative methods were considered to provide for short record lengths, sensitivities to outliers, inconsistencies in data skewness across regions and the occurrence of the various classes of mixed climate, gust and hourly mean values. Corrections were made for the terrain exposure for each AWS where this deviates from the open terrain requirement for specified strong-wind observations. The SANS 10160-3 procedures are based on characteristic wind speed with an annual exceedance probability of 0.02 (50 y return period), which were derived for each station from appropriate probability models.

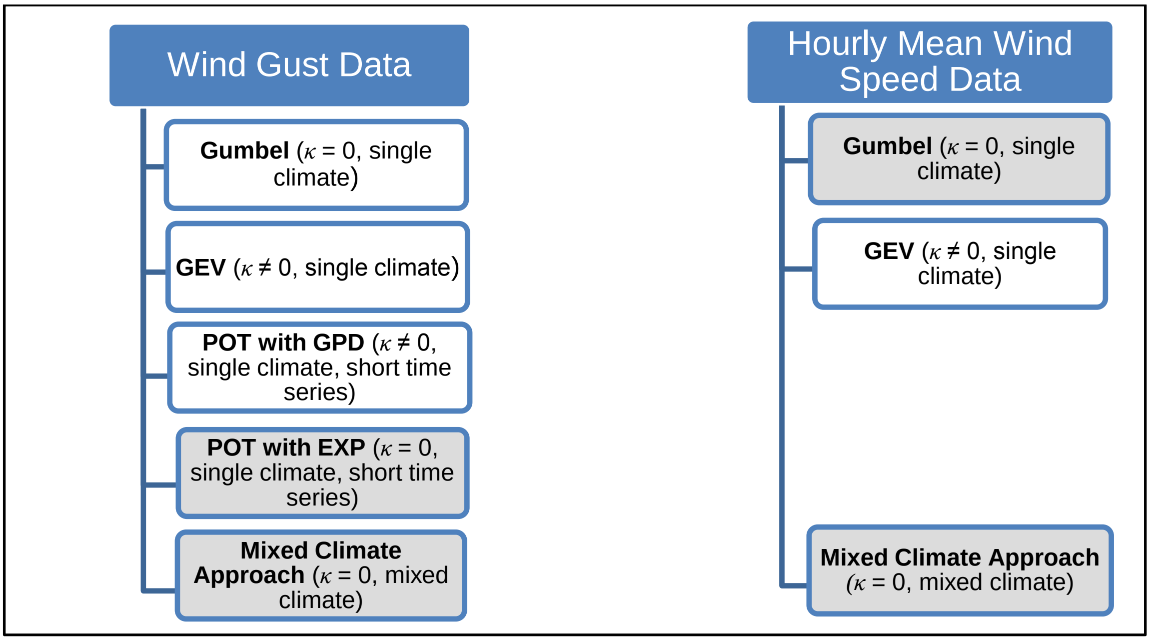
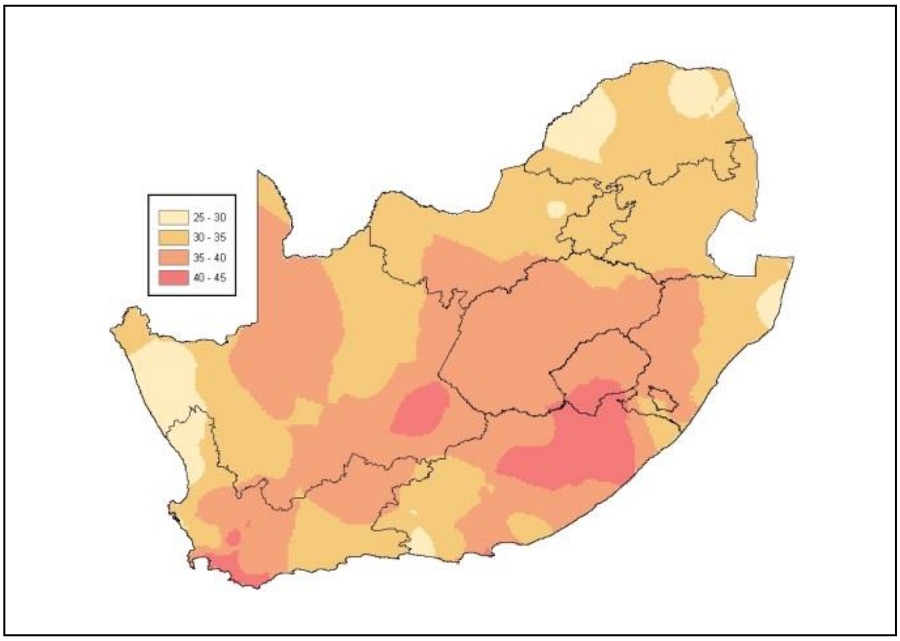


Figure 7.9 Alternative extreme value distribution methods applied to the strong-wind dataset

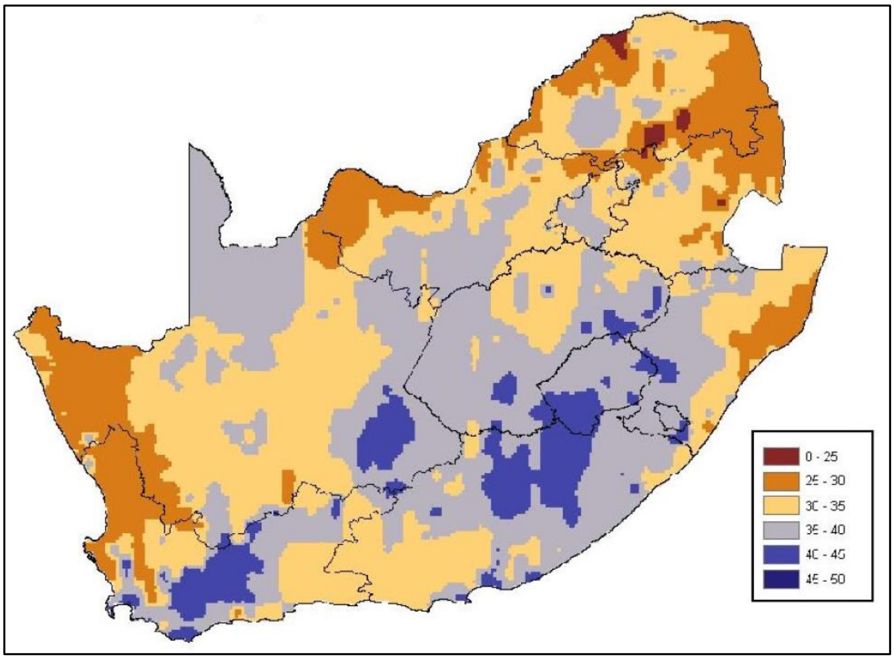
Two alternative methods of mapping the characteristic gust wind values are shown in Figure 7.10: A direct interpolation of the results for each of the 74 AWS positions are shown in (a) and the mapping on the conversion of the hourly mean value map with elevation-based interpolation and gust factor conversion shown in (b).

An integrated and smoothed map taking account also of uncertainties resulting from short record lengths in the detemination of the resolution of the results is shown in Figure 5.3 (Section 5.2.2). Noteworthy features of this map include:

- The general decrease in value from south to north;
- An area of relatively high values in the north of the Eastern Cape province, due to very strong thunderstorms that occur there from time to time;
- The close spacing of the contours in the Cape Peninsula, due to the complex topography;
- The extension of the 40-45 m/s region incorporating the eastern Free State up to North-West province, to include the regions of relatively strong thunderstorm gusts identified with cluster analysis (Kruger, Goliger et al 2012). Without this extension the 40-45 m/s region will follow more or less the dashed lines depicted on the map;



(a)



(b)

Figure 7.10 Alternative mapping of characteristic gust wind (a) direct interpolation (b) gust factor conversion of hourly mean map based on geographic features

7.2 Representative Reliability Models for Wind Loads

The investigations providing specific input to the provisions for wind loading on structures as presented in Section 5.2 have been extended to address aspects for which additional information can be expected to lead to improvements in design procedures due to better characterisation of strong winds, more refined wind engineering models and the subsequent advancement of wind load reliability calibration. Additional investigations related to wind load models are presented in this section.

7.2.1 Reliability model for free field strong wind

The diversity of the strong wind climate of South Africa is demonstrated by considering the extreme value probability distributions for the three major development centres in South Africa shown in Figure 7.11, based on the associated distribution parameters listed in Table 7.1 (Retief, Barnardo-Viljoen, Holický 2013a). By normalising the distribution to the characteristic wind speed defined as having a 0,02 annual exceedance probability ($V_{0.02}$) to represent the geographical distribution of strong winds; the dispersion (V_P), describing the temporal nature of strong winds can be determined. Figure 7.11(b)-(d) provides a representation of the dispersion of the three centres, including the range of dispersion characteristics for the regions surrounding the centres. In this manner the time variant component is further subdivided in a specified value $V_{0.02}$ reflecting geographical distribution and V_P reflecting the temporal probability distribution.

Table 7.1 Strong wind distribution parameters for major centres

DATA/PARAMETER	JHB	DBN	CPT
Record period (y)	14	16	16
Mean $\bar{x}$ (m/s)	24.4	26.1	27.7
Standard Deviation s	3.94	2.76	4.08
Dispersion α	3.07	2.16	3.18
Mode β	22.6	24.8	25.8
Skewness	0.77	1.01	0.75
Thunderstorm (TS) or Synoptic scale (SS)	TS	SS	SS

The investigation of a suitable representation of strong wind dispersion was subsequently extended across the country (Botha, Retief et al 2014). The normalised dispersion models were differentiated into the main strong wind mechanisms of thunderstorms and synoptic winds; upper and lower bound values were obtained for each case.

The envelopes obtained are shown graphically in Figure 7.12, with the extreme value distribution parameters summarised in Table 7.2. From these results it is concluded that the differences between the various strong wind mechanisms are small, in comparison to the range of conditions across the country. The effective average coefficient of variation for the normalised wind speed w_V of 11% can be related to a time dependent variability of wind loading of about 22%. To this should be added the uncertainty of the geographical manner in which V_P is approximated in wind load reliability assessment.

Table 7.2 Distribution parameters for the normalised strong wind speeds across South Africa

MECHANISM	MODEL	*a*	*B*	w_V
Thunderstorm	Upper Bound	0,11	0,56	0,22
	Lower Bound	0,04	0,83	0,06
Synoptic	Upper Bound	0,09	0,61	0,18
	Lower Bound	0,03	0,86	0,05
Combined	Average	0,07	0,73	0,11

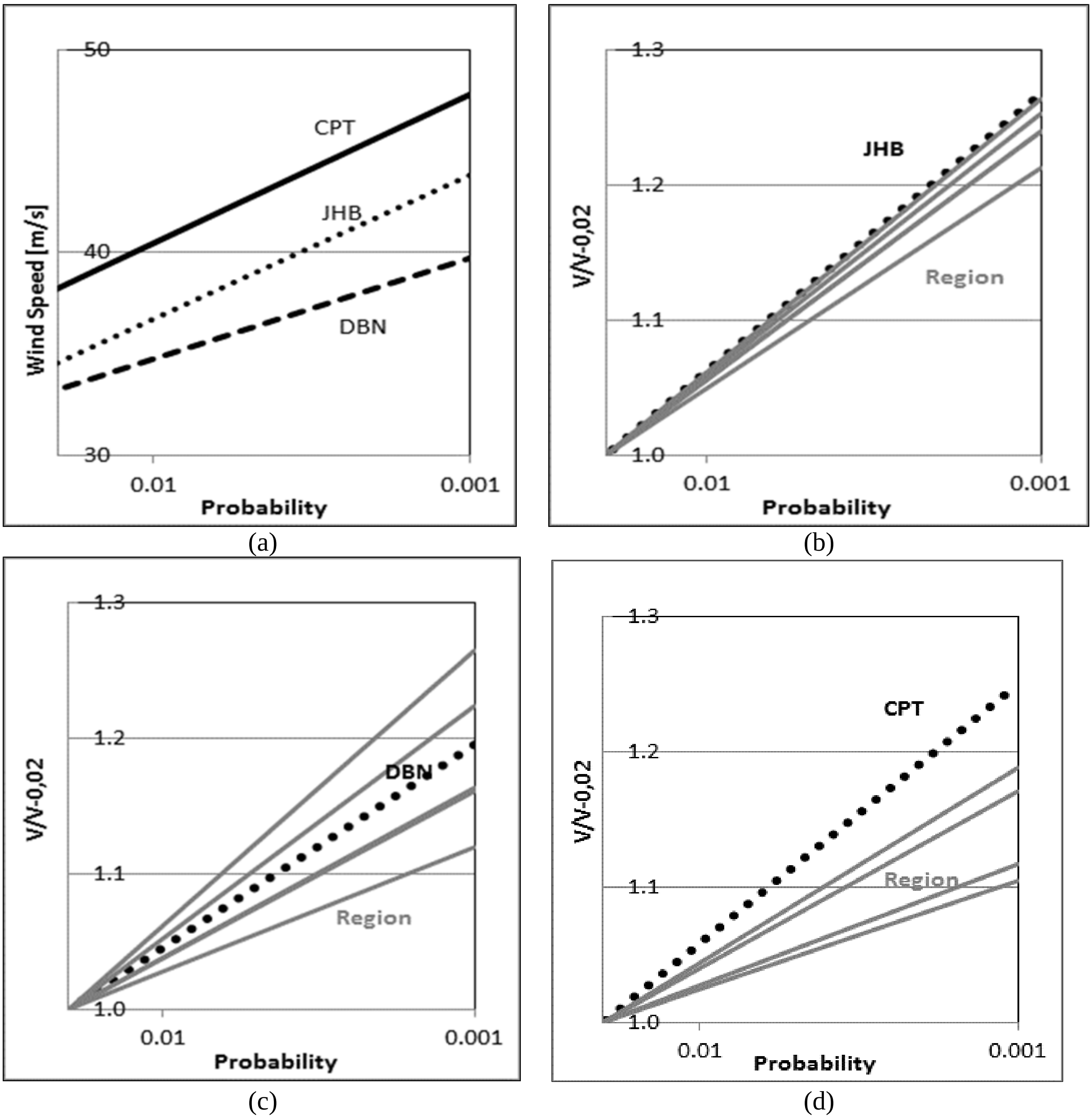


Figure 7.11 Representative strong wind probability models for Cape Town (CPT), Johannesburg (JHB) and Durban (DBN) and respective surrounding regions.

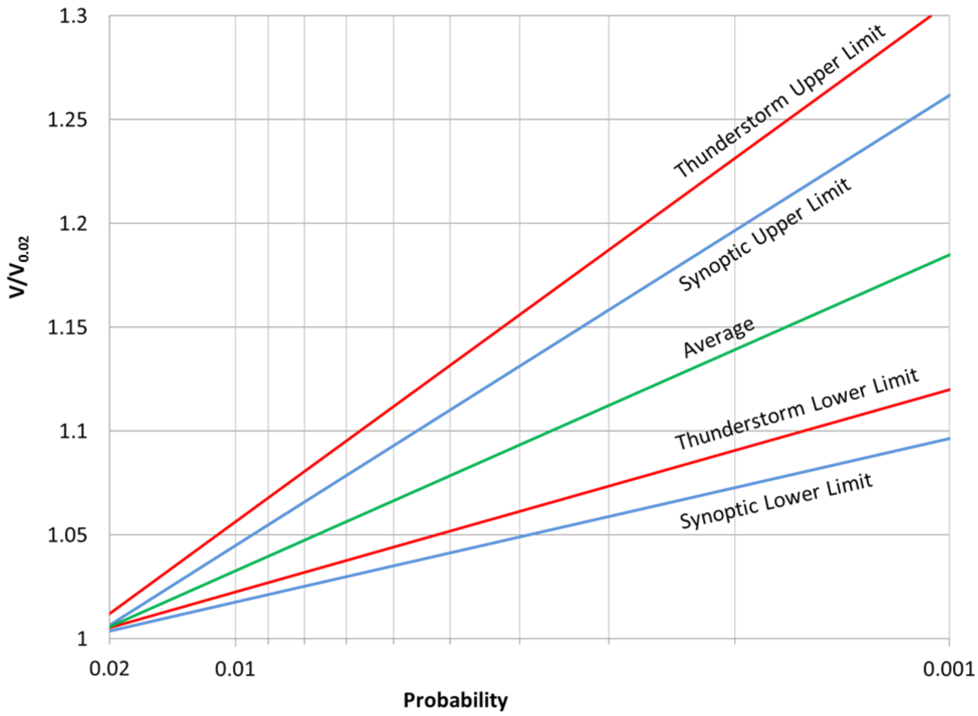


Figure 7.12 Range of normalised probability distributions for South Africa differentiated in terms of strong-wind generating mechanism

7.2.2 Time independent reliability model for wind loading

Investigations on the primary time independent components of wind loading are reported by Botha, Retief and Viljoen (2015), exploring and assessing differences in wind load standards as an indicator of the uncertainties resulting from the approximations implemented in standardised design procedures. Comparisons are made for an initial selection of representative structures. Terrain roughness and pressure coefficients are treated both separately and in combination. The various structural design standards considered are SANS 10160-3 (2011), EN 1991-1-4 (2005), BS NA EN 1991-1-4 (2010), AS-NZS 1170-2 (2011), ISO 4353 (2009), ASCE 7 (2010) and NBC (2010).

Average values for the coefficient of variation w_X of 0,33 for pressure coefficients and 0,11 for terrain roughness were obtained, giving a value of 0,35 when they are subsequently combined. For the investigation using combined code procedures, a range of 0,24 to 0,28 is obtained for the total time independent wind load uncertainty. An illustration of the results obtained for the combined wind loading procedures is given in Figure 7.13.

7.3 Generalisation of Wind Load Investigations

An integral view of the various investigations on the strong wind load climate of South Africa and the derivation of reliability models for wind loading is provided by several investigations ranging from comparative assessment of strong wind climates to the presentation of the chain of investigation from engineering climatology to wind load design procedures, closing the circle again by considering the possible impact of climate change on infrastructure design.

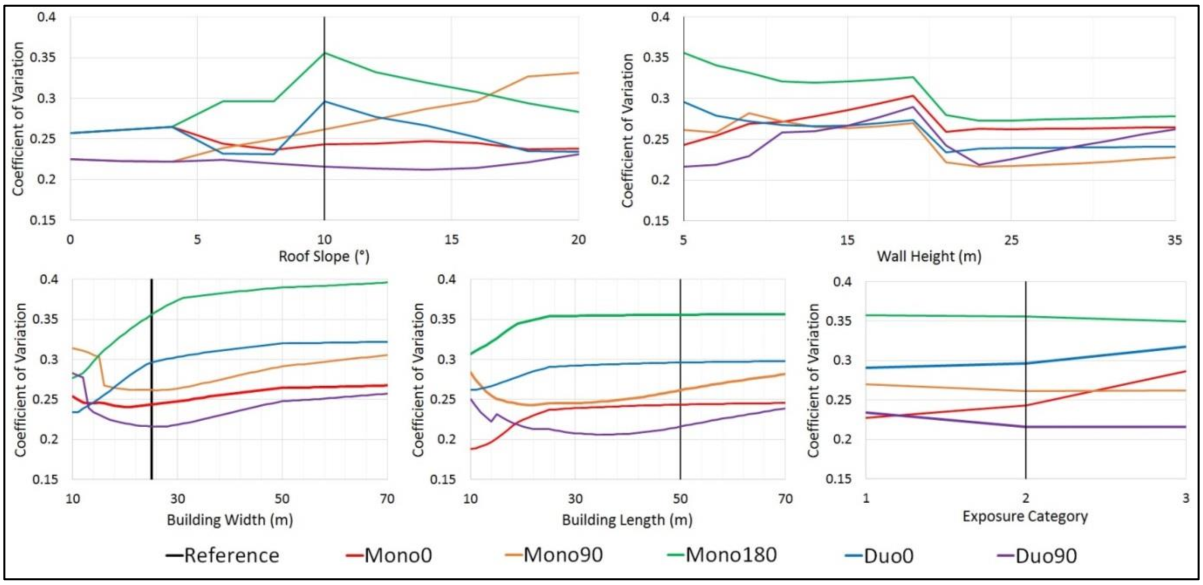


Figure 7.13 Coefficients of variation plotted against varied parameters for a reference structure for various load cases consisting of mono and duo-pitch roofs with 0°, 90° and 180° wind attack

7.3.1 Comparative assessment between South Africa and Poland

A comparative study of wind loading investigations between Poland and South Africa provided an opportunity to summarise local conditions and design provisions based on conditions and practice in comparison to related considerations for Poland (Goliger, Zuranski et al 2013). Joint experience on structural wind damage serves as an introduction to the survey; as based on the strong wind features of the two countries. The implications of the voluntary adoption of Eurocode procedures by South Africa versus the regulatory imperative for Poland are considered; together with the transition from previous procedures for the respective countries.

A general comparison of the strong wind climatology of the two countries confirms the dominance of synoptic winter storms across Poland; whilst the mixed climate consisting of synoptic storms and meso-scale thunderstorms has a complex geographical distribution for South Africa. The relative rates of occurrences of synoptic storms, hail storms, lightning and tornados are compared for the two countries. Examples of wind damage are presented by reporting surveys made for the respective countries. Rather than exploring a phenomenological approach to counter wind damage, conventional structural design against wind loading is then considered.

In the implementation of Eurocode, Poland is restricted to the stipulation of allowed design parameters in a National Annex. South Africa has the freedom to adapt procedures into a South African National Standard SANS 10160-3. In addition to generic similarities of converting from an existing standard to a completely new one, the technical issues requiring assessment were quite similar for the two countries: wind averaging time and the associated gust factor procedures; the selection of terrain categories; assessment of pressure coefficients; provision of a design wind map; the selection or calibration of an acceptable wind load partial factor.

From a South African perspective the differences in wind climate and design approach limit the technical benefit of the study. However, the similarities of issues to be resolved when implementing Eurocode in the two countries are quite revealing. The study confirms the benefits of sharing the Eurocode body of knowledge, whilst having the freedom to adapt the procedures to South African conditions.

7.3.2 Scheme to relate structural design to climatology

Another opportunity to generalise the process of the development of procedures to represent strong-wind loading on structures is represented by an outline of the integral treatment of the application of climatic investigations to derive models for strong-winds on which design procedures could be based (Kruger, Goliger et al 2014). The description of the process is summarised in Figure 7.14, indicating the combination of statistical and climatic models, applied within a reliability framework to derive strong-wind reliability models and maps to derive strong-wind partial safety factors as calibrated to pre-set performance or reliability levels.

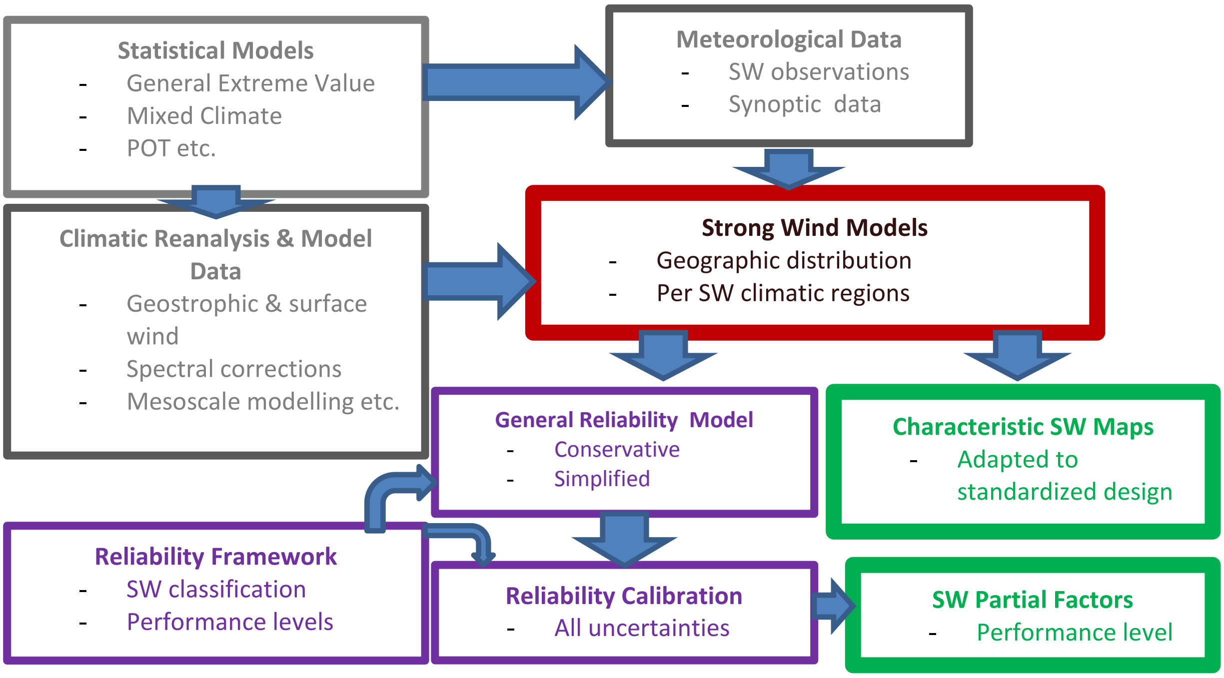


Figure 7.14 Integral development of design wind speed and loading as derived from climate data

7.3.3 Strong-winds, climate change and infrastructure design

The interrelationship between climatic investigations and structural design is further explored in assessment of risk based design adjustment for climatic change (Retief, Diamantidis et al 2014). This study demonstrates how design procedures could be adjusted for changes in the rate and intensity of strong-wind occurrences due to climate change. It provides further elaboration of a preceding general assessment of the future influence of climate change on infrastructure design for South Africa and Germany, with special reference to strong-winds for South Africa (Retief, Diamantidis et al 2013). The following conclusions can be made from this survey of the present state of the interrelationship between climate change and extreme loads on infrastructure:

(1) Climate change will have an impact on future extreme environmental actions on structures and should be considered; however more data, models and other information are necessary in order to better extrapolate future data.
(2) Periodic review of statistical data and probability models related to environmental actions such as wind, flood etc. is necessary.
(3) Reliability methods and reliability based design values are sufficiently flexible to be able to accommodate and implement additional information from this periodic review.
(4) Practical safety measures (protective or mitigation) can be applied in many cases.
(5) These measures should be based on risk acceptance criteria combined with cost optimization.
(6) A performance based approach is recommended for the verification of global behaviour of infrastructure under extreme environmental conditions and climate change.

7.4 Reliability Assessment of Pile Foundations

7.4.1 Model factor statistics

The calibration of partial factors for pile design in accordance with SANS 10160-5 reported in Section 5.3.2 forms part of a general investigation on the reliability of pile foundations constructed under South African conditions. The investigation is grounded on a dataset of pile tests for which associated design values are determined in order to derive a dataset of observations of the pile model factor (*M*) as the ratio of interpreted test results to its prediction based on the static pile method. Full account is given by Dithinde, Phoon et al (2011) of the pile tests and interpretation of test results; pile capacity predictions based on recorded material and design parameters; model factor statistics and its assessment. Sufficient information is given to serve as a dataset of pile tests for Southern African geotechnical conditions and design practice. A refined assessment of the dataset and its relevance to local design reliability performance is provided by Dithinde and Retief (2013a).

Various subsets and their combinations are given in terms of soil classes Cohesive (C) and Non-Cohesive (NC) and construction type Driven (D) and Bored (B). The total number of 174 tests is subdivided accordingly into D-NC (29), B-NC (33), D-C (59) and B-C (53). Statistical assessment includes testing for outliers and removing tests where warranted; testing for correlation with design parameters for the respective soil/construction classes; determining the appropriate probability model through goodness-of-fit analysis. The summary model factor statistics are listed for the various subsets in Table 5.4 of Section 5.3.2. Noticeable differences in model factor statistics in terms of distribution parameters can be observed for the various pile classes, in particular for non-cohesive soil.

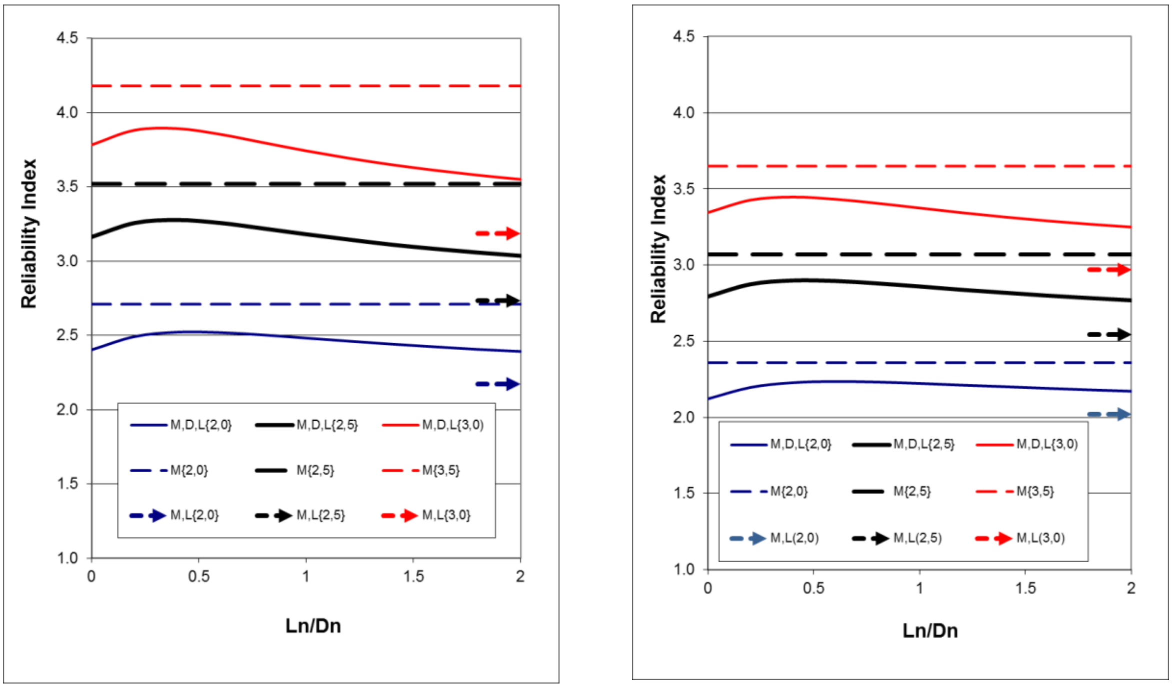
7.4.2 Implicit reliability of existing practice

An assessment of the implicit levels of reliability of working stress design practice (WSD) generally applied in South Africa is presented by Retief and Dithinde (2013). Two approaches are presented, considering pile resistance only (shown in Table 7.3) and considering the total reliability by including probability models for live (L_n) and dead (D_n) loads shown in Figure 7.15.

Table 7.3 Range of implicit reliability values βI and associated pile classes (FS = 2,5)

Range	Pile Class	Lognormal ($\beta_{I,Rep}$)
Special	Driven piles in non-cohesive soil (D-NC)	3,1
Low	Non-cohesive soil (NC)	3,2
Mid	**Combined group (ALL)**	**3,5**
Mid +	Driven piles (D)	3,7
Mid +	Bored piles (B)	3,8
Mid +	Bored piles in non-cohesive soil (B-NC)	3,75
High	Driven piles in cohesive soil (D-C)	4,1
High	Cohesive soil (C)	4,2
High	Bored piles in cohesive soil (B-C)	4,3

Figure 7.15 indicates the implied reliability of existing design practice utilising three optional factors of safety FS{2,0; 2,5; 3,0} based on probability models for model uncertainty (M), dead (D) and live (L) loads, with the resulting graphs labelled as M,D,L(2,0); M,D,L(2,5); M,D,L(3,0) respectively. The results are shown as a function of the ratio of live to dead load (L_n/D_n), with the extreme case of live load only indicated as →M,L(FS). The implied reliability where only the effect of model uncertainty is included is shown by the dash line graphs labelled as M(FS). The implied reliability for the special case of driven piles in non-cohesive soil (D-NC) is also shown, indicating significantly lower reliability under these conditions.



(a) Combined set (ALL) (b) Special case (D-NC)

Figure 7.15 Implicit reliability index values (β) for WSD pile design as function of the live to dead load ratio (L_n/D_n)

The results of the analysis indicate that the reliability index values for ultimate limit state failure of single piles implicit for present design practice vary with the pile class. However, the influence of the probability model applied is more significant. Based on conventional and standardised procedures for reliability analysis, a representative implicit reliability index value $\beta_{I,Rep}$ = 3,5 is obtained, corresponding to a probability of failure $P_f = 2.10^{-4}$. The values for various sets of pile conditions range from β_I = 3,1 ($P_f = 1.10^{-3}$) to β_I = 4,3 ($P_f = 1.10^{-5}$). This compares well with target levels of reliability for structural and geotechnical performance of β_T = 3,0 as set in SANS 10160 Part 1 *Basis of structural design*.

7.4.3 Reliability of SANS 10160-5 procedures

An assessment of the reliability performance of the new Limit States Design (LS-D) procedures for pile design stipulated by SANS 10160-5:2010 is presented by Dithinde and Retief (2013b), using the pile resistance statistics generated from local pile load tests, load statistics from the previous South African loading code (SABS 0160), and partial factors prescribed in SANS10160-5. Representative results showing the reliability achieved as a function of the ratio of live to dead load (L_n/D_n) for the respective pile classes are shown in Figure 7.16. The key conclusions drawn from the analyses on the implied reliability of the new procedures are as follows:

- SANS 10160-5 does not achieve a consistent level of reliability as β values vary with pile classes as well as with individual cases represented by the L_n/D_n ratios within the same pile class. This is attributed to lack of rigorous calibration of partial factors capturing the distinct soil types for the geologic region of Southern Africa as well as the local pile design and construction experience base.
- The β values obtained are influenced by the soil type, but not so much by the pile installation methods. This implies that resistance partial factors and model factors should be differentiated on the basis of soil properties.
- The β values for all pile classes are above the target β of 3.0 for the reference class of structures as specified in SANS 10160-1. If redundancy due to group and system effects is accounted for, the β values will become significantly higher than the target β of 3.0 indicating that the resistance factors in SANS10160-5 adopted from BS EN1997-1 are conservative and tend to be

uneconomical for Southern Africa. Therefore it appears that in implementation of limit state design, local calibration studies are inevitable.

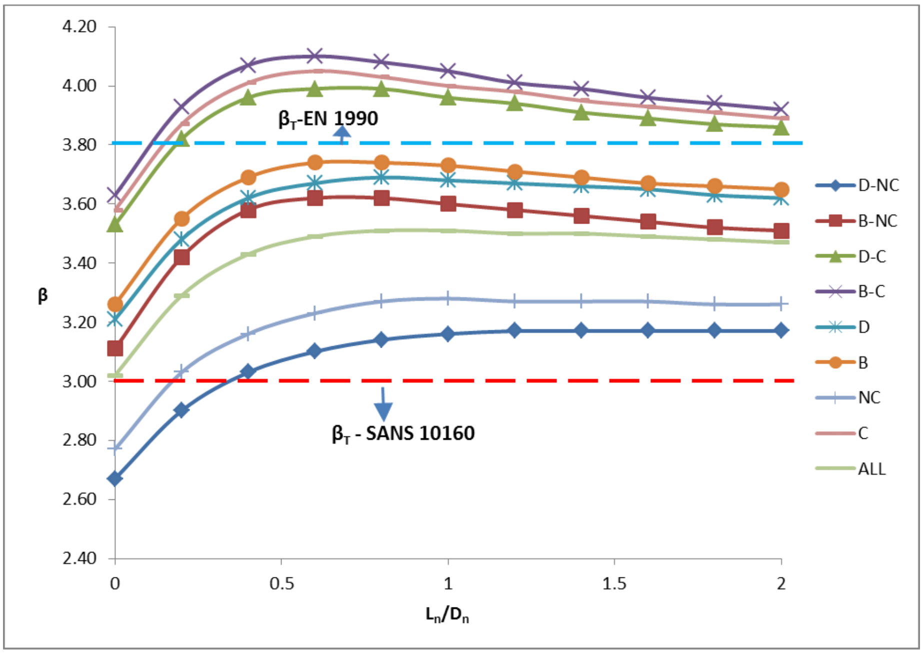


Figure 7.16 Pile resistance reliability levels (β) as a function of load ratios for different soil and pile classes based on SANS 10160-5.

This investigation serves as a preamble to the partial model factor calibration presented by Dithinde and Retief (2014), as reported in Section 5.3.2.

7.5 The Reliability Basis for Standards Development

The merit of accepting Eurocode as reference for the next generation of South African standards for structural design is generally accepted locally. Due to wide differences in construction, environmental and institutional conditions, such implementation of Eurocode requirements and procedures into South African standards is not a straightforward process. The only rational manner in which standards from elsewhere could be adopted, adapted and calibrated for a specific set of conditions is to base the process on the principles of structural reliability. Retief, Barnardo-Viljoen and Dithinde (2011) describes the way in which calibration procedures are used to assess action combinations schemes and associated partial factors, and for structural resistance procedures with special reference to geotechnical design, concrete and water retaining structures. This investigation could be considered as a review and generalisation of the background to the basis of design provided by SANS 10160-1 as presented in Section 4.3.

The reliability assessment for the conversion of Eurocode input to South African conditions consists of setting of the local target level of reliability, establishing a base case reliability performance function conventionally expressed in terms of office floor loading, considering various parametric classes of reliability and resistance characteristics; for alternative South African and Eurocode action combination schemes. This assessment is based on the background provided by Retief and Dunaiski (2009b). Reliability calibration for structural concrete is illustrated as basis for the possible adoption of EN 1992-1-1 (Holický, Retief, Wium 2010) and consideration of the effects of quality levels of construction as derived from Mensah, Retief and Barnardo (2010). Provision for geotechnical design is based on an investigation by Dithinde, Phoon et al (2011).

The representative reliability modelling provided by Retief, Barnardo-Viljoen and Dithinde (2011) served to confirm the utility of having a common reliability basis of design between standards as different as Eurocode and the South African standards, whilst allowing for calibration to local conditions. It can be noticed that the more explicit and rational the design procedures are, the easier it is to provide for widely different conditions whilst maintaining harmonisation between related standards. This feature of reliability based design is an important concept leading to the generalisation of standards development into a coherent framework, as presented in Chapter 8.

7.6 Model Uncertainty Assessment Procedures

The significance of model uncertainty associated with the structural mechanics models from which design procedures are derived is noted repeatedly throughout this dissertation, often as motivation for certain reliability investigations that are then presented. From the overall picture clear demonstration is given that model uncertainty for both action and resistance models can rank in importance with that of the uncertainty (epistemic) and variability (aleatoric) probability dispersion of basic variables such as actions ranging from permanent actions through to environmental variable actions; for structural material properties ranging from steel to geotechnical materials.

The general practice followed presently of using nominal representation of model uncertainty in calibration of design procedures clearly is in need of being strengthened and improved by explicit treatment and representation. Furthermore, model factor assessment provides an opportunity to fully utilise test results not only in reliability calibration, but also in the assessment and improvement of alternative structural mechanics design models. This section provides contributions towards the systematic presentation of properly assessed model factor probability models and the way in which such results can be used systematically in reliability assessment and calibration.

Progression of the development of the systematic treatment of model uncertainty took a step forward with the report by Holický, Sykora et al (2013) based on the combination of resources with Holický and Sykora for the systematic compilation of model uncertainties for concrete structures. This cooperation was extended by providing an outline of a methodology for the systematic assessment of model uncertainty as reported by Holický, Sykora and Retief (2014). The model uncertainty methodology is elaborated and illustrated by selected cases of application as included by Holický, Retief and Sykora (2015). The methodology is largely based on experience with investigations for structural concrete (see Section 7.4) with reference to pile foundation design serving as basis for the statistical treatment and analysis of the underlying database (see Sections 6.3.2 and 8.3). However, the methodology as presented in this section also applies to other elements of reliability representation.

The main properties of uncertainties in resistance models can be treated stepwise, starting off from the resistance model and model uncertainty observations dataset and its quality assessment, ending up at a reliability function on which the design function is based (Holický, Sykora et al 2013). Two classes of models should be distinguished:

- Engineering models with strictly defined assumptions (beam and sectional approaches)
- Complex models based on general principles of structural mechanics with much wider options and potentially higher uncertainties. Shear resistance of sections with shear stirrup reinforcement is identified to fall in this class.

Further assessment of model uncertainty for structural concrete reported by Holický, Sykora and Retief (2014) is motivated by the observation that currently used probabilistic models and derived factors for the model uncertainties are mostly based on intuitive judgements and limited data. This often leads to unrealistic description of the model uncertainties. The reported study attempts to improve definitions of model uncertainties and propose a general methodology for their quantification by comparing experimental and model results. The investigations demonstrate and confirm that model uncertainty should always be related to a specific model and scope of its application.

7.6.1 General methodology

The methodology for model factor assessment briefly introduced by Holický, Sykora et al (2013) can be extended further by characterisation of model uncertainty classes; elaborating on the database from which model factors are derived and the statistical assessment of model factor observations; the development of a probabilistic description (model) for model uncertainty; concluding with a general discussion of the application of model uncertainty representation.

The relative importance of model uncertainty with respect to uncertainties contributed by the basic variables should be taken into account in reliability modelling and assessment. The following classification of effects and treatment of model uncertainty provides a convenient framework:

1 ***Nominal effect*:** Partial factors for basic variables are adjusted to provide a nominal contribution of model uncertainty; for guidance, this approach could be followed when the FORM sensitivity factor (see *ISO 2394:2015* and *EN 1990:2002*) for model uncertainty is less than 0.4 × 0.8 = 0.32. This approach is typically taken across a general class of conditions, for example differentiating only between models for permanent and variable actions; or for the resistance of structural materials.

2 ***Significant effect*:** The contribution of model uncertainty is sufficiently significant to justify a separate partial factor, typically with a sensitivity factor between 0.32 and 0.8; considering importance of model uncertainty to be equivalent to the basic variables. Explicit provision for model uncertainty allows for differentiation between various failure modes, execution process, levels of approximation for the resistance of a specific material class or proneness to local instabilities. Examples include, respectively, bending or shear, rolled or welded steel sections, ordinary- or high-strength concrete and different classes of steel cross-sections.

3 ***Dominating effect*:** Limitations of functions to comprehensively predict structural resistance in operational terms result in model uncertainty to be the dominating factor for some cases even for routine situations included in standardized design procedures. Special attention to the assessment of model uncertainty is then required to ensure the economic attainment of sufficient reliability. Examples include the shear resistance of reinforced concrete sections and pile foundation resistance.

Another step was taken in the development of a systematic review of model uncertainty with specific reference to resistance models for structural concrete, as reported by Holický, Retief and Sykora (2015). The extended paper attempts to improve definitions of model uncertainties, propose a general methodology for their quantification by comparing experimental and model results, and suggest their treatment in practical applications.

Although model uncertainty expressed as the model factor θ is treated as similar to a basic variable, it is inherently more complex in relating a theoretical model as a function of basic variables to test values consisting of an experimental setup and the physical equivalent of the variables. That is why the assessment of model uncertainty requires a systematic and thorough methodology to account for these two entities when arriving at a probabilistic model for θ based on the statistical analysis of the dataset of observations. The degree of detail to which the assessment needs to be done depends on the relative importance of model uncertainty to the relevant performance function; ranging from only nominally relevant to the dominant source of uncertainty.

The methodology for model factor assessment and application presented by Holický, Retief and Sykora (2015) is primarily formulated in terms of it having a dominating effect (Category 3) as indicated in the categorisation above. It is based on a number of investigations where this is the case. The following steps should be considered when the model uncertainty for a specific theoretical model is investigated, as based on a dataset of test results for the model:

- Characterization of the assessment to determine the scope of investigation;
- Dataset of test results, either as a dedicated investigation or from a compilation of diverse tests;
- Compilation of a database of model uncertainty observations;
- Statistical assessment of the dataset to derive a suitable probability model for θ;
- Application of the model considering its operational use and
- Further investigations including acquisition of new test data or use of a more advanced model, where warranted.

The set of experimental observations of resistance $\{R_i\}$ for which all basic variables $\boldsymbol{X}$ and related parameters $\boldsymbol{Y}$ are available serves as the final measure for determining the contribution of the corresponding theoretical model to reliability. The main attributes of the dataset of test results include:

- The number of tests;
- The sample space (Λ_O) of variables $\boldsymbol{X}$ and $\boldsymbol{Y}$ and records of values;
- The quality of test results, values of variables, testing equipment and its calibration, control of boundary conditions;

- The availability of any information that might indicate the tolerances of the test results.

The following principles should be followed when compiling the dataset:

- Proper *identification of the measured resistance* is of particular importance, requiring explicit verification in the record.
- *Correspondence between the test result and* the failure mode predicted by *the theoretical model* should be verified, e.g. differentiating between ductile or brittle failure modes.
- Implications of the ability of the model to *differentiate between various sub-divisions of a failure mode* in testing are important, e.g. reinforcement or concrete failure; tests for shear resistance may be contaminated by the combined effects of shear and flexure.

The database of model uncertainty observations $\{\theta_i\}$ is derived from the comparison of the model predictions $\{R_{\mathrm{model},i}\}$ to the test dataset $\{R_i\}$. The model predictions are made with the theoretical model on which the design function is based. The following steps should be taken when compiling the database:

- Any design bias should be excluded from the calculation of R_{model} e.g. representing material properties by true or mean values rather than characteristic values.
- Although certain design parameters are not explicitly applied in the model, they may affect model uncertainty and should be included in the assessment process.
- In principle efforts should be made to obtain values of all design parameters by measurements. However, it may be practically sufficient to:
  - Use previous experience e.g. for steel modulus of elasticity
  - Use accepted relationships with measured basic variables e.g. estimating concrete tensile strength from measured compressive strength

In addition to the quality assessment of the physical observations indicated above (meta-data in statistical terms), the set of model uncertainty observations $\{\theta_i\}$ shall be analysed using basic methods of mathematical statistics. This includes consideration of the following:

1. *Unbiased sampling*. Strictly speaking the set of test results should represent unbiased sampling. However, experimental campaigns to measure structural resistance are typically devised for the investigation of the effect of certain ranges of basic variables for a defined sample space (Λ_{O}), rather than for the unbiased sampling of the design population space (Λ_{d}).
2. *Representativeness of the sample space*. An indication of the representativeness of Λ_{O} to Λ_{d} can be obtained by considering the ranges and distributions of basic variables from the experimental dataset in the form of histograms.
3. *Correlation between model uncertainty and variables*. A scatterplot of model uncertainty observations against any basic variable provides an indication of the degree to which the basic variable is accounted for in the model, both in terms of the trend of observations and scatter around the trend.
4. *Outliers*. The presence of outliers may greatly influence calculated statistics leading to biased results. Outliers may increase the variability of a sample and decrease the sensitivity of subsequent statistical tests. Outliers should not be rejected outright since they may represent truly extreme observations. However, the relevant meta-data on the specific outliers are scrutinised typically for errors or indications of not qualifying for inclusion into the sample space.
5. *Sample statistical moments*. The final dataset can then be used for conventional statistical assessment by estimating distribution parameters. Estimates by the Method of Moments are independent of an underlying distribution whilst the estimators by the Maximum Likelihood method obtained for an assumed distribution are deemed statistically more efficient. Ideally θ should be unbiased; pragmatically the model should intentionally include a limited degree of conservative bias ($\mu_\theta > 1$); the occurrence of an un-conservative bias ($\mu_\theta < 1$) is evidence that model development is not necessarily reliably pragmatic.
6. *Statistical uncertainty*. The statistical uncertainty in estimates of distribution parameters needs to be considered when a small sample is available; i.e. when the number of tests is less than 10 to 30 depending on the dispersion of model uncertainty.

7. *Probability description of model uncertainty*. Goodness of fit tests provide a statistical tool to select an appropriate type of probability distribution. However, conventionally probability models for model uncertainty are preselected as based on accepted reliability modelling practice.

7.6.2 Advanced methodology

Model uncertainty investigations can be applied diagnostically to explore the effectiveness of deriving simplified design procedures and the effectiveness of the theoretical model as such. Such a campaign can be added as a fourth class of the assessment as outlined in categorisation of model uncertainty given above. Two examples are discussed to demonstrate the investigative and diagnostic use of model uncertainty assessment, namely for the shear resistance of reinforced concrete sections, without and with the provision of shear stirrup reinforcement respectively.

Model uncertainty for shear resistance of sections without shear reinforcement is assessed for two semi-empirical models used in BS 8110 and EN 1992-1-1 and an application of the more rational Modified Compressive Field Theory (MCFT). The results indicate that the semi-empirical methods generally provide a more reasonable estimate of model factor bias (mean value close to 1,0) than that obtained for MCFT, but with similar dispersion values given by comparable standard deviation values. The obvious constraint that semi-empirical procedures should not be extrapolated beyond the limits of their underlying database should however be noted.

Only a preliminary version of the extensive assessment of the reliability performance of the shear resistance regular reinforced concrete sections with stirrup shear reinforcement is reflected in the methodology presented by Holický, Retief and Sykora (2015). This investigation is presented here in Section 6.2, concluded in Section 6.2.7 with the presentation of a generalisation of the investigation into an advanced application of model uncertainty as an investigative tool: Characterisation of model uncertainty can be used beyond its objective for incorporation as an additional source of uncertainty when calibrating design procedures. It can be used to assess a given design model and its underlying structural mechanics basis, including the identification of limitations to its scope of application, bias of the underlying model, trends related to design parameters, in addition to the conventional measurement of dispersion of model factor observations. This can be applied to alternative design or advanced models, which could also serve as an independent reliability model for the failure mode under investigation. This provides an opportunity for an extension of the methodology presented by Holický, Retief and Sykora (2015) to include a fourth level of general reliability based investigation.

7.6.3 Reliability based development for advanced materials

The model development procedures presented in this section, as derived from model uncertainty investigations presented in Section 6.2 for structural concrete and Section 7.4 for pile foundations, are based on the premise of established practice with an extensive body of knowledge, testing and experience; albeit still lacking in rigorous reliability treatment. An interesting case is that of the development of new and innovative structural materials, such as strain hardening cementitious compounds (SHCC), in order to derive operational design procedures which are consistent with conventional design practice. An outline of a reliability based development scheme for design procedures for SHCC is presented by Dymond and Retief (2009) as an example of a way in which reliability concepts can be used to derive indicative design procedures from limited information on structural mechanics models; then apply inverse reliability analysis to select an effective testing program for further advancement.

The proposed scheme is fully compatible with the reliability basis of design procedures utilised in SANS 10160-1, as derived from Eurocode EN 1990 and ISO 2394. Although the proposed scheme is broadly consistent with the reliability based research methodology discussed in Section 6.2.7, there are noticeable differences from conventional reliability development of design procedures. Essentially these differences derive from the fact that the process is driven to a large extent on uncertainties which compromises the ability to demonstrate the advantages of the advanced materials; on the other hand the careful planning of an experimental campaign devised to address the critical sources of uncertainty provides an effective strategy of counter measures.

Chapter 8: The Basis for Development of Standards for Structural Design

The diverse set of investigations on structural reliability modelling, assessment, calibration and design application demonstrates the close interrelationship between reliability and the complementary fields of structural engineering contributing to standards for structural design. Generalisation of this role of structural reliability is represented by leading standards such as ISO 2394 *General principles on the reliability for structures* and the JCSS *Probabilistic Model Code*.

The transfer of standards selected from the comprehensive Eurocode suite to a specific set of standards suited to South African needs prompted many questions related to the characteristics of standards for structural design; leading to another level of generalisation: *What are the principles on which standards for structural design are based and how do they influence their development and implementation?* If such a question appears to be too open-ended, at least the more specific question could be posed: *What methodology could be followed to derive some principled arguments for the development of structural design standards for South Africa, within the context of making full use of reference standards from elsewhere?* Although many of these concepts are imbedded in the standards or their development through pragmatic treatment by standards development groups, the premise of these contributions is that the principles can only be scrutinised and applied in optimal management of standards development if these principles are presented explicitly.

8.1 Characterisation of Standards for Structural Design

A first attempt to characterise standards for structural design was made following the decision to use the relevant Eurocode Standards and Parts as reference for the revision of SABS 0160:1989 as reported in Section 4.1.1. Obvious differences between SABS 0160 and even a selection of Eurocode Parts raised many questions on how to proceed with the adaptation of the technical contents of the reference standards to local needs, as discussed throughout Chapters 4 & 5. Retief and Dunaiski (2006) derived the main features of standards for structural design from their historical development, based on South African experience, with the objective to derive principles that could serve as guidelines for future development. Because this presentation was not formally published in proceedings, the main points are summarised here due to the importance of this first step towards the development of a higher level view of structural standards and their development.

Although the development of structural standards are observed to be of a fragmentary nature, at least in South Africa, some general trends can be observed; such as improved rationality from experience based rules to reliability based limit states design. International harmonization plays an important role in various modes of sharing experience and methodologies; but requiring a rational basis in order to be effective or even possible. Progressively standards need to provide for increased magnitude and complexity of structures together with higher expectations or requirements from society on safety and economy.

A first identification of the main features of standards for structural design is made in terms of their function and purpose, differentiating between a regulatory function or serving the profession in the discharge of their responsibilities to clients and the public; the scope of a standard, related to the class of structures provided for and the associated standards; the technology basis of a standard, ranging from experience and judgement to structural mechanics and reliability modelling.

8.2 SANS 10160:2010 Implementation

The formulation of the background to SANS 10160:2010 as reported in Section 5 served as the next step in establishing general principles for standards development. This is stated as follows by Retief, Dunaiski and Day (2009): The various sections of the *Background Report* are generally compiled into three steps consisting of:

a) Review of advances in standard practice within the context of South African conditions;
b) Selection of appropriate procedures, models and values for requirements and stipulations;
c) Assessment of the implications of the stipulations in comparison to present practice.

Where justified, the various steps are dealt with in separate chapters of the Background Report (Retief & Dunaiski (Eds) 2009). In the background to the basis of design (Retief & Dunaiski 2009b), an overview is also given of the integral process of revision as it evolved with time and from which general principles have been distilled. The review of Eurocode from the perspective of the revision of SANS 10160 (Retief, Dunaiski and Holický 2009) might be considered to go beyond a strict provision of the background to the revised standard, but is vital to the concept of deriving principles for structural standards from the characterisation of Eurocode and SANS 10160 respectively.

8.3 Principles of Standards Development

The function of standards for structural design is generally accepted as a statement of requirements, serving to fulfil its regulatory function. It also provides acceptable standardised procedures to assist practitioners to demonstrate compliance to the requirements. The view of standards in terms of their technical formulation of verifiable requirements is however not sufficient to explain the process of standardisation of structural design. A more general perspective of the process is needed. An evident issue is the motivation for the development of standards, which subsequently provides the driving force of such development. Particularly the resources for such development and ownership of the standard become relevant. Related issues include the scope of a standard and its relationship with other standards; the level of advancement of the procedures; the degree to which provision is made for local conditions, design and construction practice. In addition, a basic question is whether only well established procedures can rightfully be standardised or whether new techniques and advanced procedures are confirmed to be acceptable by the standard. A higher level issue is the basis for harmonisation with international practice.

These questions served as motivation for the development of a framework for standards development that could serve as reference base for reviewing the adaptation of Eurocode standards as South African standards. Countries outside Europe may benefit from the more fundamental approach described here when considering the application of Eurocode. More generally, the meta-standardisation framework can be used to provide the directives for the program of standards committees or the activity of a specific task group.

The initial formulation of the framework was presented by Retief and Wium (2010) mainly consisting of the expression of the methodology, based on the identification of meta-standard drivers or attributes and their deliberate alignment to achieve an effective standards development programme. Examples of the attributes of standards and their development include the following:

- Interest groups - authorities and regulators, professionals, industry;
- Mechanisms – research, construction practices;
- Utilities – harmonisation, experience.

The value added by the use of a standard as expressed in terms of its attributes serve as driving potential or motivation for standards development.

An extended formulation of the framework for the development of standards for structural design and demonstration of its application is presented by Retief and Wium (2012); serving as basis for this section. A supplementary objective of the investigation is to justify that the development of structural standards is not only a professional activity, but also one worthy of research and systematic development. There is similarity between this proposed field of investigation and that of structural reliability, which also strives to provide an integral basis for structural standards and their formulation. Interrelated procedures may provide a rich field for research on the development of structural standards.

8.3.1 Technical characteristics of structural standards

The general technical contents of structural standards consist of an expression of requirements for structural performance and acceptable rules and procedures for proving compliance; expressed in terms of structural mechanics and reliability models; tempered with a substantial element of experience based judgement from the perspective of practice. The following elements are proposed to reflect the main characteristics of structural standards:

(i) Requirements, expressed in non-specific format;
(ii) Scheme of situations and conditions for which compliance have to be established – limit states and design situations;
(iii) Structural mechanics models on which verification procedures are based;
(iv) Reliability measures applied to the models;
(v) Complementary rule-based procedures are often needed to express requirements to reflect the associated assumptions and best practice procedures.

These elements can be classified in terms of their engineering science, logic or judgement-based nature. Since they play different roles in the function of the standard, they can be used to apply the appropriate emphasis. Examples are the structural mechanics models related to verification procedures; reliability measures directed towards the regulatory function of the standard; organisation and rules reflecting experience-based judgement.

8.3.2 Meta-technical characteristics

The direct interrelationship of the technical elements of a structural standard and the context in which it operates provide the basis for the meta-technical characterisation of a standard. Examples of such interrelationships which provide a vital link between the technical contents of the standard and its context are:

- Function of standard
  - Stipulating regulatory requirements;
  - Assisting the designer in discharging of professional duties;
  - Enabling the use of a structural class, product or material.
- Relation to other standards, typically to be used together in a specific design
  - Fully stand-alone, providing comprehensively for a class of design;
  - Part of a unified set of standards, sharing a common performance base;
  - Harmonised to related standards, sharing or exchanging the reference technology.
- Relationship to the construction process
  - Dependence on and directing the quality management regime of construction practice;
  - Provides the basis for construction procedures and specifications;
  - Standardise or lead the structural aspects of construction practice.

8.3.3 Attributes of structural standards

The attributes of structural standards and the environment in which they operate provide the driving potential for the dynamics of their continued development and improvement, use and maintenance. The attributes can be considered to be the value related characteristic as seen from its context of various *value systems*, such as assisting designers in executing their duties and responsibilities; higher levels of professional interest, including assurance of structural safety and serving regulatory functions; or serving various classes of trade and commerce. A clear identification of the value-based attributes of structural standards and the potential field in which they operate will assist the profession to manage their development and use. The following classes of attributes are proposed for use in setting up a framework for the development of structural standards:

Stakeholders: Attributes related to interested groups or stakeholders include the authorities representing public interests in structural safety; various professional interest groups ranging from designers through to researchers; commercial interest groups from the construction industry to organisations promoting the use of structural materials and products. The role of the European Union in sponsoring the development of Eurocode in order to foster a common market for construction is a prime example of stakeholder driven standards development.

Standardisation: The authority given to a standard through the standardisation process provides formal endorsement to the stipulated requirements and compliance verification procedures. These attributes relate the standard to its regulatory role or the authority on which the designer can demonstrate due care. The standard provides the necessary authority for the subsequent guidebooks, design tools and software which interpret the stipulations in operational terms.

Technology: The level of technology on which a standard is based represents an important set of attributes, ranging from experience based procedures to professional endorsement of advanced structural engineering practice. Technology related attributes range from specific procedures, for example the determination of characteristic wind load on structures, to the integral technology base of the standard to represent best practice.

Strategic objectives: Attributes that can be assigned to the development of a standard in strategic terms provide a powerful motivation for the development of the standard or a suite of standards. A negative example is the fragmented suite of South African standards for structural design that results from the lack of strategic objectives for their development and maintenance.

Operational objectives: Operational attributes can be interpreted from the role of the standards in the chain of functional requirements, conceptual design, verification, specification and execution. Progressive strategies only have meaning when they can be executed through properly managed and resourced operational activities for development and implementation of the standards.

8.3.4 Mapping of Eurocode to South African conditions

The sets of structural standards that resulted from the Eurocode program and the present set of South African standards are fairly representative of the extremes of the range of standards development. When considering the transfer of Eurocode standards to South African conditions, it is instructive to map the Eurocode process and set of standards in terms of the attributes of the two sets, with particular emphasis on mismatches.

Eurocode represents a strongly focussed development program for structural standards that evolved over several decades into an elaborate set of ten standards, 58 parts and multiple sets of national annexes for the ultimate deployment as operational design standards. This is achieved through sustained institutional support motivated to facilitate the exchange of construction services (construction works and related engineering services) and to improve the functioning of the internal market. In contrast the South African structural standards evolved from diverse efforts by various professional and industry driven interest groups. Incidentally the (previous) South African committee structure for design standards is modelled on ISO, with structural standards diluted amongst a plethora of construction standards technical committees. This is in contrast to Eurocode where a single committee manages the development program.

A brief assessment of Eurocode and South African structural standards respectively is made with a view of deriving guiding principles for transferring Eurocode standards to South Africa. In this process the merit of Eurocode as an up to date, advanced, comprehensive and extensively unified set of standards is accepted, and sits at the background of the assessment. The emphasis is placed on mismatches in the local conditions as compared to the Eurocode environment that need to be taken into account to achieve an effective and optimal process of transfer.

Table 8.1 provides a summary of a generic assessment of the main issues that arise when the transfer of Eurocode standards to South Africa is considered in terms of the attributes that apply to the two sets of standards. Guiding principles for executing such a transfer process are included in the assessment.

8.3.5 Application to South African standards development

Three South African standards are at various stages of being transferred from Eurocode. They provide an opportunity to demonstrate how the respective processes can be explained in terms of the effective influence of attribute driven considerations. The three standards are the Loading Code SANS 10160 which was published in 2010; a revision of the Concrete Code SANS 10100 for which the adoption of EN 1992-1-1 has reached an advanced stage; a new standard for water retaining structures. The three standards are assessed in Table 8.2 in terms of the set of attributes.

Table 8.1 Attribute based assessment of Eurocode transfer to South Africa

EUROCODE	SOUTH AFRICA	GUIDELINES
Stakeholders		
Institutional as primary sponsors, -generating support from industry sectors	Professional, -With support from industry sectors	Establish an efficient organisational structure: -Policy and strategy; -Development program starting with existing standards; -Generate resources; -Oversee and manage
Standardisation		
Primary motivation, with strategic objectives of having common standards across an extensive market -Comprehensive range of standards -Exceptional level of unification.	Present approach pragmatic to expedient -Restricted range, directed to normal practice -Diverse reference standards -Absence of unification -Significant gaps	Primary local motivation for standardisation -Unification of local standards; -Proper reference base -Trade considerations of secondary importance to stakeholders
Technology		
Advanced technology achieved, imperative for -Comprehensive application scope -Achieving consensus	Consisting of -Reference standards -Restricted level of advancement; -No systematic advancement	Technology measures -Ensure that standard does not exceed local practice; -Transfer technology base *with* standards -Launch systematic development program
Strategic		
Sustained strategic initiative took process through successive stages and spawned concurrent activities -Implementation as operational national standards -Increase level of harmonisation -Initiate next round of development -Promote use beyond Europe	Absence of a coherent strategic view on structural standards -Professions accepts overall responsibility, on a voluntary basis -Key stakeholders delegate (abdicate) responsibility to profession -Progressive industry groups are the most active players -Coherent program & limited resources are critical deficiencies -Opening up of African markets for designers, contractors and suppliers	Use transfer program to marshal stakeholders -Reorganised structure (see above); -Establish formal relationship with Eurocode initiative -Launch prioritised incremental transfer program -Establish principles for complementary or alternative approaches -Use feedback from experience related to strategic objectives -Establish contacts with stakeholders in Africa
Operational		
Activities include -Implementation as national standards -Promotion, training, supporting literature & software -Monitoring differences through NDP database	Low key standards development on a case-by-case basis -Driven by ad hoc initiatives -Development is only effective when driven by attribute potential	Augment existing activities -Base activities on fundamental principles -Establish attribute-based potential -Initiate new activities to fulfil strategic objectives -Develop initiatives towards industry wide funding for standards development and maintenance

Table 8.2 Attribute based assessment of specific South Africa standards transferred from Eurocode

LOADING CODE	CONCRETE CODE	WATER RETAINING STRUCTURES
Stakeholders		
Diffuse but very wide range -Design practitioners are the primary stakeholders -Stakeholders of all structural standards -Regulators, indirectly	Potentially more clearly identifiable: -Design practitioners -Cement & concrete industry -Indirect interest derived from common nature of structural material	Specific set of stakeholders: -Specialist design and construction practitioners serving: -Water authorities
Standardisation		
Extensively optimised for local conditions -Leading standard for Actions on buildings; unification of standards; common design philosophy -Fully consistent with Eurocode	Leading standard for general design practice as most common structural material -Replace SANS 10100 (BS 8110) with EC 2-1-1 -Transfer present standards for materials & construction procedures to EC equivalent, (or vice versa in local version)	-Presently no local standard -Reference BS8007 replaced by EN1992-3 (scope not matching) -Dependent on general concrete standard
Technology		
Eurocode provides reference technology -Consistent with previous technology base -Provision for local conditions – environment; practice -Adjusted performance levels to local economic conditions -Spawned related research in almost all fields	-Level of the standard advances the standard design technology base -Does not attract much attention, considered to be mature field. -Standardisation should stimulate supporting research -Specific fields include performance, reliability and quality management	Sponsored project allowed for investigation of selected topics -Materials and construction practice -Limiting cracking requirements
Strategic		
Provides critical first step towards transfer -Advancing level of technology, harmonisation -Enabling use of EN-1997-1 -Extendable to other areas -Potential for regional use	-General procedures serve as platform for specialist applications -Promoted as basis for African Concrete Code	Specialist but high priority field
Operational		
-Profession took the lead -Development severely constrained by lack of resources	-Lack of resources is the primary motivation for simple adoption -Access to design aids seen as important advantage -Quality management critical to limit use of standard within experience base	Water Research Commission sponsored pre-normative development -Need to initiate standardisation process

8.3.6 Synthesis into attributes framework

The core function of structural design standards in the process of design is generally well understood by the structural engineering profession. Proper understanding of the standards however requires an appreciation of its contextual function, in terms of its role in regulation, professional duties, responsibilities and progression, and the advancement of trade and commercial interests. Although the primary focus in the development, assessment and use of a standard is on its technical nature, its contextual function is intricately related to the technical requirements and rules. Many attributes of the standard are determined by the contextual function such as its regulatory status and function, professional function, relationship to trade and industry, technology base and responsibility for development, maintenance and application of the standard.

A set of attributes of standards for structural design was identified and presented in a structured manner. The attributes are based on the interest of stakeholders, benefitting from the inherent advantages of standardisation; technology advances from research or practice creating the body of knowledge which is released through standardisation; the possibilities of using standards to achieve strategic goals; providing the basis for practical operational activities, of which guidance to code committees and the performance of structural design represents the ultimate activities.

These attributes clearly provide the driving potential for the continuous development of structural standards. The relative importance of the respective attributes is dependent on the conditions within a specific standard or set of standards operate. This was illustrated by an assessment of Eurocode and South African structural standards, with specific reference to differences between these two sets, both viewed from the perspective of transferring Eurocode to South Africa. Furthermore, specific clarification of the scheme of attributes is provided by considering three South African standards at various stages of development.

The main advantage of an attribute framework for the development of structural standards is to initiate, plan and manage such a program on this basis right from the outset. Since Eurocode can only be transferred to South Africa step by step, there is ample opportunity to do so in the future. The scheme, or at least the concepts, should also be of benefit to many countries considering the transfer of Eurocode. Where the next stage of the evolution of Eurocode is presently contemplated, the concepts presented here may also be of benefit.

8.4 Applications of Standards Development Framework

The framework for the development of standards for structural design serves as a useful reference base for future planning or to assess experience with standards development. A few examples of assessments that were done against the framework are presented in this section.

8.4.1 Design standards for structural steel

The set of South African standards for the design of steel structures provides a demonstration for the lack of unification of local structural standards, illustrated by the adoption of standards from Canada and Australia for local use. The framework for the development of standards for structural design serves as basis for an assessment of the next generation of South African structural standards, with specific reference to design standards for structural steel (Dunaiski, Retief, Barnardo 2010).

A concise assessment is provided of the attributes of Eurocode that were considered with the development of the South African Loading Code SANS 10160:2010: Institutionally Eurocode was developed as an official program of the European Union, but allowing for jurisdiction over safety to be kept by member states. The primary objective was to promote trade and strengthen the competitiveness of the European construction industry. The comprehensive scope of Eurocode followed from its high level sponsorship. Eurocode shares the extensive technology base from its member states. It achieved extensive harmonisation across Europe and unification between the ten standards consisting of fifty-eight parts.

The main features of SANS 10160 are summarised as maintaining consistency with existing South African practice, whilst implementing advances and updates of requirements and procedures. Whilst maintaining consistency with Eurocode, full provision is made for local conditions and design practice. It is concluded that SANS 10160:2010 can be regarded as significant extension of Eurocode beyond European borders. The framework for standards development as presented by Retief and Wium (2010) is used as basis for giving an outline of mapping selected Eurocode parts from EN 1990 *Basis of Structural Design*, EN 1991 *Actions on Structures*, EN 1997 *Geotechnical Design* and EN 1998 *Design of Structures for Earthquake Resistance* to the needs and conditions as extracted from SABS 0160:1989.

An assessment is made accordingly on how the twenty parts of Eurocode EN 1993 *Design of Steel Structures* could serve as reference for future South African structural steel design standards. However, it appears that what could be considered to be the strength of EN 1993 in the European context as a set of parts that are extensively harmonised across the region and unified with the other

Eurocode standards; full treatment of specialist structures such as bridges, tanks, masts, poles, piling and more; include advances such as structural fire design; takes it well beyond the South African needs. A systematic strategic review of the future of South African steel design standards could nevertheless assist in making decisions on how to proceed.

8.4.2 South African experience with standards development

A review of recent experience with standards development and some generalisation as lessons from this experience is presented by Wium, Retief and Viljoen (2014). Three South African structural standards have been revised, by making careful use of the advances made by Eurocode. For each of the revised standards, a unique process has been followed, being a result of the needs, expertise, and prevailing local conditions. The revisions vary from adapting of the Eurocode, for the Loading Code SANS 10160 and for Concrete Water Retaining Structures SANS 10100-3, to adopting of the Eurocode EN 1992-1-1 for concrete buildings, with a South African National Annex. The process for the revision of each standard demonstrates how an international reference standard can be used to establish local standards by allowing for local conditions and requirements.

No clear advantage can be discerned between the options of either adoption or adaptation of selected Eurocode Parts to provide for local application and conditions. A significant technical effort is required for both options. Technical advances are balanced by the need to accommodate the range of conditions amongst member countries. The possibility of providing for specific local conditions from the wide scope of Eurocode conditions is neutralized somewhat by the tight arrangement of Eurocode Parts needed to cover the scope without any duplication.

It should however be pointed out that the adoption of EN 1992-1-1 does represent the most ideal situation, as this standard replaces BS 8110 in the UK, the standard on which the present SANS 10100-1 is based. Furthermore the BS National Annex served as point of departure for the adopted standard. Nevertheless, this did not solve and resolve the difficulties of finding sufficient resources for the development of local design standards, although this consideration served as initial motivation for following the adoption route. Resolving both technical issues of local concern and complying with adoption rules required an extensive input into the process.

The adopted standard EN 1992-1-1 has a much wider scope than that of the present South African Concrete Code SANS 10100-1. For example, provision is made for a much higher range of concrete strengths, lightweight aggregated concrete, even aspects of prestressed concrete may be considered not to represent common local practice. In many cases these topics represent specialist fields of structural concrete, rather than an advancement of general or common practice. Since the competence of both designers and constructors form a vital part of the reliability basis of structural performance through the associated quality management programs, the distinction between general practice and specialist competence is not clear from the comprehensive standard. A clear warning is therefore given not to take the inclusion of specialist fields of the structural use of concrete in the SA Concrete Code as a license for use in general practice.

In contrast to the adoption of the Eurocode general part on structural concrete, the information on SANS 10100-3 presented by Wium, Retief and Viljoen (2014) provides a clear demonstration of:

- An effective operational program for achieving a strategic objective of developing a new standard for the design of concrete water retaining structures
- By generating the resources from proper alignment of the interests of various stakeholders
- To resolve an extensive range of technical issues through a balance between research input and experience based judgement.

Although the project was not launched *a priori* in terms of the standards development framework as formulated by Retief and Wium (2012), the framework can be identified to be imbedded in the successful development of SANS 10100-3.

The loss of BS 8007 as an extension of BS 8110 when both standards were replaced by Eurocode EN 1992-3 *Liquid Retaining and Containment Structures* and EN 1992-1-1 caused an important local deficiency, since BS 8007 served as the effective South African standard for water retaining structures. Furthermore, the scope EN 1992-3 and its associated standard EN 1991-4 *Actions on Structures – Silos and Tanks* exceed that of the immediate South African needs.

Through the identification of the need for the development of a new standard serving South Africa's needs, the support of the Water Resources Commission (WRC) could be obtained as a sponsoring stakeholder. This allowed for both background research and the compilation of background information (Barnardo-Viljoen, Mensah et al 2014) in order to launch and effective working group consisting of both academics and practitioners. Operational objectives consisted of a tight management program for the working group and related activities. Technical matters that could be resolved to a sufficient level to proceed with standardisation included specific basis of design requirements, loading, crack width prediction and limits, concrete mix design and the control of the heat of hydration and drying shrinkage. Meta-standards considerations include the selection of requirements and procedures from the two primary reference standards EN 1992-3 and BS 8007 and a range of other related standards.

Finally it is concluded by Wium, Retief and Viljoen (2014) that the South African experience can be interpreted as an exercise in simplification of Eurocode for a limited scope of application. It appears however that a substantial technical effort would be required to do so across the Eurocode member countries; at the same time resulting in substantial loss in the degree of allowance for local setting of performance levels or accommodating other national preferences. Conversely an exercise of developing a simplified version of Eurocode could contribute to reducing the diversity of the Nationally Determined Parameters posted in the National Annexes by the respective member countries.

Chapter 9 Observations on the Nature of Reliability Investigations

The reliability investigations presented in the dissertation can be classified in accordance with various schemes, which is a typical characteristic of a system with multiple nodes and links. A convenient way of arrangement is to assemble the topics in the sequence of their development: the only logical basis is the emergence of advancement as driven by insight and opportunity. Imbedded in such a process is the interplay between the general survey of a field of investigation and the identification of a specific topic requiring further attention. Such a pattern can surely be identified to emerge from the series of investigations. Close scrutiny of specific investigations reveal different classes of reliability investigations, as suited to the nature of the topic identified for further enquiry. The objective of this Chapter is to capture some observations and insights into the nature of structural reliability investigations that may be of use in planning new campaigns of investigation.

9.1 Pattern of Development

From the material presented in the dissertation, a pattern of interplay can be observed between an overview assessment of a field of application of reliability and risk modelling and the focussed investigation of a specific topic within such a field. The platform for such interplay is provided by the nature of the utility of reliability modelling to serve as common basis to treat all aspects related to the safety and functional performance of structures on a unified basis:

- *General and specific investigations*: Specific investigations on reliability assessment of actions and structural concrete resistance led to generalisation into investigations on the integral basis of design; subsequently the process proceeds in the inverse direction of identification of specific issues from the overview assessment;
    - An investigation on the serviceability requirements for steel structures (see Section 3.1.1) can be related to the revision of serviceability criteria for SANS 10160 based on a re-interpretation of the reference ISO standard (see Section 4.3.10).
- *Imposed loads*: A general survey of the provisions of various standards (Section 5.1.2) prompted an investigation on imposed roof loads based on expert surveys (Section 5.1.3).
- *Wind loading*: A general survey on provisions for wind loading in SANS 10160-1 (5.2.1) initiated specific investigations on the strong wind climate of South Africa (Section 7.1.3) required for updating the design wind map (Section 7.1.4) and the development of more refined reliability models for wind loading for South Africa as basis for recalibration of design procedures (Section 7.2).
- *Geotechnical design*: The basis of geotechnical design as stipulated in SANS 10160-5 (Section 5.3) provided the platform for reliability modelling and calibration of pile foundation design procedures (Section 7.4).
- *Crane loading*: Extensive surveys on procedures for determining design crane induced loads as background to SANS 10160-6 served as point of departure for the development of reliability models and calibration studies on crane induced loads (Section 5.4).
- *Structural concrete*: General reliability assessment serving as basis of design for structural concrete (Section 6.1.1) identified the need for assessing the reliability model for flexural resistance and extensive investigations of provisions for shear design (Section 6.2).
- *Water retaining structures*: A similar general assessment of the basis of design of concrete water retaining structures (Section 6.3.2) identified crack width design as a critical topic that could benefit from reliability modelling and assessment (Section 6.3.1).

The fundamental application of structural reliability modelling is either to perform an analysis to determine the reliability or probability that a structure will achieve a given limit state; or the inverse process to derive appropriate design measures in order to exceed a given target level of reliability. These applications can be extended to obtain generalised classes of design conditions; or to investigate a component of a case or class of conditions, such as actions or resistances. In each case the fundamental reliability modelling concept will be applied with an appropriate approach and methodology (see also Section 6.2.7). A few such specific types of application were encountered in the investigations reported here.

9.2. Generic Reliability Calibration

For broad based calibration of codified design parameters, performance limit states are typically expressed in terms of normalised probability models for the various components of actions and resistance, without any representation of the relevant structural mechanics models. In this manner generic reliability calibration is done to characterise reliability performance which represents a comprehensive scope of application without recourse to the structural mechanics models from which the basic variables are derived.

The calibration of the action combinations scheme of SANS 10160-1 as reported in Section 4.3, more specifically the assessment of partial factors presented in Sections 4.3.4 – 4.3.5, represents such a generic reliability calibration. On the one hand such an exercise provides useful insight into the general reliability trends for alternative action combination schemes across the range of load ratios, typical resistance variability, target levels of reliability; in addition to comparisons with alternative approaches such as from Eurocode and SABS 0160 in this case. On the other hand generic reliability calibration is critically dependent on the probability models representing the main sources of uncertainty.

In Section 4.3.5 the importance of the proper provision for systematic effects or bias provided for wind loading in this case is demonstrated. The same consideration applies to the influence of bias introduced through the use of characteristic values of the underlying basic variables, as demonstrated by Holický, Retief and Dunaiski (2007). Model uncertainty is another influencing factor that is often not properly treated, or at best only represented nominally in the normalised probability models for actions and resistance utilised in generic reliability calibration exercises.

In spite of its simplified nature, generic reliability calibration represents a very influential class of reliability assessment when considering its role in code making decision making. Its power resides in the clear way in which trends are presented, with an extensive range of technical detail collapsed into a sufficiently representative model. Generic calibration represents the most important interface between the reliability specialist and general structural engineering practice.

As indicated above, the development of proper normalised models for the representative models for action and resistance classes forms an important foundation for generic calibration. The extent of such an investigation is illustrated in Section 5.2.3 and Section 7.1.4 for wind loading.

9.3 Reliability Data from Expert Measurement

The lack of hard data is often cited as the motivation for not using a reliability approach in design and related procedures and decisions. Liberal use is then made of engineering judgement, although it is implicitly subjective and not transparent. The formal measurement of expert judgement is therefore a useful methodology to bridge the gap of uncertainty.

The two investigations on the use of expert measurement to assess the reliability performance of the serviceability of steel structures (Section 3.1.1) and roof loads (Section 5.1.3) respectively represent the development of a methodology to apply expert measurement systematically and scientifically in structural reliability. In addition to setting up the classical method for measuring engineering design parameters, the first investigation extended the range of application to reliability through the explicit measurement of uncertainty. In the second investigation the use of direct observation as the basis for determining seed data on which experts are calibrated is introduced. In both cases explicit reliability models could be derived, ready for use in analysis and to derive calibrated design procedures. The serviceability investigation can be characterised to be of the generic class with limited structural modelling, whilst the roof load investigation used representative structural mechanics procedures to derive roof loads in terms of load effects on the structure.

It can be concluded that expert measurement is a useful tool for scientific engineering investigations, with the potential to be used to complement laboratory measurement. It is however quite distinct and at a different level than the popular opinion surveys which does not allow for proper control and assessment of the results.

9.4 Comprehensive Reliability Based Assessment

The reliability assessment of structural concrete shear resistance presented in Section 6.2 is also discussed in Section 7.6.2 as an example of an advanced case of model factor assessment that requires reliability modelling beyond the direct conversion of the design function into a reliability function. Such generalisation can be taken one step further by considering the reported investigation as a reliability assessment of concrete shear resistance in general, by utilising both an experimental database of shear resistance measurement and alternative design methodologies. Reliability modelling then does not only assess the performance of given methodologies, but also determine sensitivities and trends to relevant design parameters in addition to the role of basic variables and the residual model uncertainty (see Section 6.2.7).

This specific case of reliability investigations can therefore be used as reference for the planning and execution of a comprehensive assessment of any structural mechanics model. Such an approach can be contrasted to the conventional structural mechanics based approach consisting of a single campaign of testing, albeit on various components of the process ranging from failure through the contribution of sub-elements to the measurement of material properties. Although such mechanics based approach may include some statistical treatment of data, this is typically not expressed in terms of probability models for the basic variables or the combined effect of various sources of uncertainty, including model uncertainty (see also Section 7.6.3).

Chapter 10 Conclusions and Recommendations

The identification of different emerging approaches and methodologies presented in the previous chapter highlights the diversity of all the material presented in the dissertation by formulating yet another way in which the investigations can be viewed. At the same time a single unified theme emerges, consisting of the utility of the reliability based approach for the derivation of effective design procedures. In addition it provides a methodology for advancing the understanding of structural performance. On this basis the merit of the reliability basis of structural design is substantiated, as derived from the collection of investigations presented in this dissertation.

10.1 Levels of Reliability Modelling

In the introductory chapter it is claimed that

- The reliability basis of design serves to harmonise standardised design practice internationally, allowing for the utilisation of the shared body of knowledge and experience on structural performance;
- It serves to unify the procedures providing for the various actions, structural materials and classes of structures;
- At the detail level, reliability modelling and calibration serve to ensure proper provision for all relevant limit states, design situations and failure modes in terms of probability models for basic variables and model uncertainties.
- Furthermore reliability modelling serves as the basis for investigating structural mechanics models on which design procedures are based as a general methodology that can be applied in background research for the development of design standards.

The following investigations by the Candidate serve to substantiate these claims:

Harmonisation: The transfer of a selection of the extensive body of standardised procedures and background information from Eurocode to the substantially different conditions applying to South Africa can be considered to be a demonstration of the power and value of a common reliability based approach resulting in effective harmonisation.

Unification: Based on the Eurocode example of a Head Standard, the unification between the South African Loading Code (SANS 10160:2010) with equitable treatment of various classes of actions on structures, provision for geotechnical design and confirming its use with present and future materials-based design standards reported here, demonstrates the value of a common approach to derive unified procedures for rather diverse components of the design process, as presented in different standards.

Specific design models: Various specific investigations demonstrate the use of reliability modelling, assessment and calibration to derive or provide background for the set of design procedures as compiled into a design standard. Pertinent examples include wind loading, crane induced loading, concrete shear resistance and pile foundation resistance.

Reliability based investigations: Several examples are provided of how the various levels of reliability based assessment can be synthesised into a generalised approach of investigation: A systematic treatment of model uncertainty can be extended into an approach for the investigation of a structural mechanics model, where reliability concepts are treated integrally with testing and modelling, typically of a prediction model. A framework for standards development is devised by deriving attributes of design standards from the implicit principles on which they rest, with the objective to assist with the motivation, resourcing, management and decision making required for successful standardisation projects.

10.2 Mapping of Investigations

The set of investigations is classified in Figure 10.1 to provide some order and to clarify relationships. The overall classification is based on standardisation and reliability investigations, with background to the standards serving as the link, grouped together with generalised investigation approaches. Under the heading of Standardisation the relevant standards are listed only, without inclusion of any contributions towards their development forming part of the dissertation due to the difference between standards development and academic investigations. However, involvement with standards development by the Candidate, as outlined in Section 1.5, were vital to serve as platform for investigations either to assist in decision making or to capture the background to such decisions on standardised procedures.

10.3 Concluding Observations

On the basis that the body of the dissertation consists mainly of brief outlines of the various investigations and a summary of each set of conclusions, the final conclusions presented here are directed primarily to the outcome of the integral campaign as summarised in Figure 10.1. Accordingly the following observations are made on the utility of reliability modelling and assessment as basis for standardised structural design, as explored by the Candidate:

- **Reliability Kernel:** The relatively basic kernel of a reliability performance function of basic variables, that can be solved to derive design parameters to achieve a target level of reliability, can be scaled up and extended to derive appropriate design verification within an elaborate reliability framework of limit states, design situations and reliability classes. This is demonstrated by the background provided to the SANS 10160 *Basis of structural design* given in Part 1.
- **Unified Reliability Basis of Design:** The unified treatment of all the components of design such as relating the combination of various actions and resistance for various structural materials is demonstrated by the background to SANS 10160 Parts 2, 3, 5 & 6 and the resistance of concrete in shear and pile foundations, amongst others; all of these elements treated in terms of complementary reliability models.
- **Performance of Structural Performance Models:** The selection of specific structural mechanics models in need of further investigation, modelling, assessment and ultimately calibrations is demonstrated for various cases:
  - The need to refine the reliability representation of strong-wind for wind loading was identified as part of the assessment of SANS 10160 Part 3 *Wind actions*. This resulted in a series of investigations that were initiated by the standardisation process and delved into the South African strong-wind climate; with a complementary investigation on the combined effects of time variant and time invariant components of wind load reliability modelling.
  - Although the investigation of concrete shear resistance was based on standardised procedures, this study had a more general origin deriving from the diverse approaches taken by various standards. Accordingly this study extended into multiple structural mechanics and reliability models based on an extensive database of published tests.
  - The resistance of foundation piles started off as an independently identified topic in need of proper reliability assessment of local practice. Ultimately this investigation was linked to standardised design by way of the assessment of the reliability of existing practice and providing the basis for calibrated design procedures.
- **Model Uncertainty:** The formulation of a systematic procedure to approach the representation of model uncertainty and to derive appropriate reliability models is based on various case studies throughout the campaign, specifically test databases for pile foundation resistance, various concrete failure modes and reliability models for wind loading.

STANDARDISATION ← BACKGROUND & GENERALISATION → RELIABILITY INVESTIGATIONS

STANDARDISATION

INTERNATIONAL:

ISO 2394 – Reliability Principles
ISO 22111 - Design Requirements

NATIONAL:

SA-LC SANS 10160

Part 1	Basis of structural design
Part 2	Self-weight & Imposed loads
Part 3	Wind loading
Part 5	Geotechnical design
Part 6	Crane induced load

STRUCTURAL CONCRETE

SA Concrete Code SANS 10100-1

SA Water Retaining Structures SANS 10100-3

BACKGROUND & GENERALISATION

S4.5	Theoretical basis for target reliability
S4.5	Assessment for revised principles
S4.5	Standardised reliability basis of design

S4.4 BACKGROUND to SANS 10160

S4.1/2	Development approach
S4.3	BoSD & Action combinations
S5.1	General actions
S5.2	Wind
S5.3	Geotechnical basis of design
S5.4	Crane load reliability model

STANDARDS DEVELOPMENT FRAMEWORK

S7.5	Reliability as basis for development
S8.3/4	Principles, attributes & framework

S7.6 MODEL UNCERTAINTY

S7.6.1	Assessment methodology
S7.6.2	Advanced reliability based investigation methodology

RELIABILITY INVESTIGATIONS

Chapter 3 INITIAL INVESTIGATIONS

S3.1	VARIOUS: Adjusting reliability; Serviceability; Calibration methodology
S3.2	REVIEW: Comparison SA practice & Eurocode - Action combinations, resistance

S7.1-S7.3 WIND

S7.1	Severe wind & disaster management
S7.1	Strong-wind climate & design wind map
S7.2	Reliability models for wind loading
S7.3	GENERAL: Comparisons (Poland); Climate change

S7.4 PILE FOUNDATIONS

S7.4.1	Model Factor statistics
S7.4.2	Existing practice - implicit reliability of practice
S7.4.3	Standard (new) - implied reliability of procedures

Chapter 6 STRUCTURAL CONCRETE

S6.1	Basis of structural concrete design
S6.2	Reliability: Shear resistance design
S6.3	Water Retaining Structures: Cracking reliability
S6.4	Model uncertainty for structural concrete

Figure 10.1 Classification scheme of reliability based investigations related to standardised structural design

- **Standards Development Methodology:** The formulation of a framework for the development of structural design standards at meta-standard level, to serve as guidance for the process, consists of the synthesis of various elements of the overall scheme:
  - The concept of deriving an explicit framework for standards development was initiated by the patent advantage of the rational and transparent nature of structural reliability, allowing for effective harmonisation and unification of diverse standards and practices; this served as point of departure for the development of the framework.
  - The formulation of the approach taken to adapt Eurocode procedures to South African conditions served as a test-bed for such a framework, particularly by raising questions and issues on differences between the respective sets of conditions; on how and why do these considerations impact on the development of SANS 10160.
  - Considering the present diverse set of South African materials-based standards and their future advancement and what future role reference to Eurocode or other international standards will and should play, confirmed the need to capture the experience with SANS 10160. This is subsequently confirmed with the adoption of EN 1992-1-1 *Design of concrete structures* and the development of the new SANS 10100 Part 3 *Design of concrete liquid retaining structures*.
  - Although the process of the development of International Standards on structural design is strictly regulated by ISO requirements, many meta-standard considerations still need to be taken into account, for example during the revision of ISO 22111 *Bases for the design of structures – General requirements*.

10.4 Recommendations

In spite of the significant advances made with the implementation of reliability concepts in standardisation of semi-probabilistic limit states design procedures over several decades and the limited contributions reflected in this dissertation. together with the clear advancement from judgement based provisions for structural safety and performance, there remain a need and opportunities to continue with further development and implementation. A few recommendations are made here, based on the outcome of the campaign presented in this dissertation:

- There is a clear need for improved appreciation of the value and utility of reliability concepts by the non-specialist:
  - Standards committees should particularly consider and provide for the reliability implications of the various design procedures and the associated implications for gross error and its relationship with specified quality management measures.
  - Designers can benefit from the utility of the reliability basis in understanding the implications beyond the mechanistic application of the verification procedures.
- Conversely, present reliability models and calibration campaigns are simplified, selective, with limited substantiation through structural mechanics modelling and testing. Although the introduction of more elaborate schemes of design situations and cases appears to be more complicated, in fact it assists designers in selecting the appropriate procedures for unique structures and conditions.
- The implications of the stipulation by ISO 2394:2015 that semi-probabilistic design applies to the condition that categories of conditions and consequences of failure can be properly identified and characterised, need to be explored further both to ensure compliance and to capitalise on opportunities it may provide.

10.5 Final Comment

The dissertation explores the interface between reliability modelling and structural design: For reliability assessment there is no real final answer; in contrast, structural design standards provide decision support to the designer, where very definite answers are required to be relevant at all. Structural design practice will benefit from researchers providing a rational basis for standardised procedures and designers having a more reflective approach towards the uncertainties that are imbedded in these procedures. Both parties can benefit from their colleague's perspective in pursuing their respective endeavours.

REFERENCES – GENERAL

1. Ang AH-S and Tang WH (1984). *Probability concepts in engineering planning and design: Volume 2 Decision, risk and reliability.* John Wiley & Sons, New York.
2. Benjamin, Jack R & Cornell, C Allin (1970). Probability, Statistics, and Decision for Civil Engineers. McGraw-Hill Book Company.
3. Bernstein, Peter L (1996). *Against the Gods – The Remarkable Story of Risk*. John Wiley & Sons, ISBN 0-471-12104-5.
4. BS 8007:1987, British Standard: Code of practice for design of concrete structures for retaining aqueous liquids, British Standards Institute, London, 1987.
5. BS 8110:1985. Structural use of concrete – Part 1: Code of practice for design and construction. British Standards Institution.
6. CE (Commission Européenne) (2002). Guidance Paper L *Application and use of Eurocodes.* Document CONSTRUCT 01/483 Rev.1. Brussels.
7. Cooke RM (1991). Experts in uncertainty: Opinion and subjective probability in science. Oxford University Press, New York.
8. Day PW (2013). A contribution to the advancement of Geotechnical Engineering in South Africa. DEng Dissertation submitted to Stellenbosch University. http://hdl.handle.net/10019.1/79868
9. ECCS (1996). Background Documentation Eurocode 1 (ENV 1991). European Convention for Constructional Steelwork, Brussels. Report No 98. JCSS Project Team Eurocode 1.1.
10. EN 1990:2002. Eurocode – Basis of structural design. European Standard, European Committee for Standardization, Brussels.
11. EN-1992-1-1:2004. Eurocode 2: Design of concrete structures – Part 1-1: General rules and rules for buildings. European Standard, European Committee for Standardization, Brussels.
12. Gulvanessian H, Calgaro J-A and Holický M (2002). Designer's guide to EN 1990 Eurocode Basis of Structural Design. Thomas Telford. 192p.
13. Holický, Milan (2009). Reliability analysis for structural design. SUN MeDIA, Stellenbosch. ISBN 978-1-920338-11-4.
14. ISO 2394:2015. General principles on the reliability for structures. International Standards Organization.
15. ISO 22111:2007. Basis for design of structures – General requirements. International Standards Organization.
16. JCSS (2001). Joint Committee on Structural Safety Probabilistic Model Code. ISBN 978-3-909386-76-6. http://www.jcss.byg.dtu.dk/Publications/Probabilistic_Model_Code.aspx
17. JCSS 2002. Probabilistic model code. JCSS working materials, ISBN 978-3-909386-79-6. Available at http://www.jcss.byg.dtu.dk/Publications/Probabilistic_Model_Code .
18. Kanda J, Ellingwood B (1991). Formulation of load factors based on optimum reliability. Structural Safety, Vol. 9, 197-210.
19. Mensah, K.K. 2014. *Reliability Assessment of Structural Concrete with Special Reference to Stirrup Design*. PhD dissertation submitted to Stellenbosch University, South Africa. http://hdl.handle.net/10019.1/96913
20. Milford RV (1988). Target safety and SABS 0160 load factors. The Civil Engineer in South Africa, 30 (10) pp 475-481, October 1988.
21. Milford RV (1998). SABS 0160: Philosophy and load factors. South African National Conference on Loading. SAICE & SAISC, 9-10 September 1998, Midrand, South Africa.
22. Paikowsky SG, Birgisson B, McVay M, Nguyen T, Kuo C, Baecher GB, Ayyub B, Stenersen K, O'Malley K, Chernauskas L and O'Neill M (2004). Load and resistance factors design for deep foundations. NCHRP Report 507, Transportation Research Board of the National Academies, Washington DC.
23. SABS 0100-1:2000. The structural use of concrete, Part 1: Design. South African Bureau of Standards.
24. SABS 0160:1989. Code of practice for the general procedures and loadings to be adopted in the design of buildings. South African Bureau of Standards
25. SANS 10160:2010. Basis of structural design and actions for buildings and industrial structures. Parts 1 to 8. SABS Standards Division, Pretoria. ISBN 978-0-626-24757-7.
26. Schneider J (1997). Introduction to Safety and Reliability of Structures. IABSE Structural Engineering Documents 5.

REFERENCES – CITED TO CONTRIBUTE TO THE DISSERTATION

1. Barnardo-Viljoen C, Mensah KK, Retief JV, Wium JA, Van Zijl GPAG (2014). *Background to the Draft SA National Standard for the Design of Water Retaining Structures*. Concrete/Beton
2. Botha J, Retief JV, Holický M, Barnardo-Viljoen C (2014). *Development of probabilistic wind load model for South Africa*. InVento 2014 XIII, 22-25 June 2014, Genova, Italy.
3. Botha J, Retief JV, Viljoen C (2015). *Variability of time independent wind load components*. Accepted for 12th International Conference on Applications of Statistics and Probability in Civil Engineering, ICASP12 Vancouver, Canada, July 12-15, 2015.
4. Day PW, Retief JV (2009). Provisions for geotechnical design in SANS 10160. In Retief & Dunaiski (Ed) *Background to SANS 10160* Sun Media, pp 189-203.
5. Dithinde M, Phoon KK, De Wet M, Retief JV (2011). Characterization of model uncertainty in the static pile design formula. *ASCE Journal of Geotechnical and Geo-environmental Engineering*, Vol 137, No 1, Jan 2011, pp 70-85.
6. Dithinde M, Retief JV (2013a). Pile design practice in southern Africa: I Resistance statistics. *Journal of the South African Institute of Civil Engineering*, Vol 55, No 1, April 2013, pp 60-71.
7. Dithinde M, Retief JV (2013b). *Reliability index of pile foundations designed to SABS 10160-5*. Zhang et al (Ed) Geotechnical Safety & Risk IV, pp 431-436. Taylor & Francis. ISBN 978-1-138-00163-3.
8. Dithinde M, Retief JV (2014). Resistance model uncertainty factors for ultimate limit state design of pile foundations. *Journal of the South African Institute of Civil Engineering.* Vol 56, No 2 August 2014 pp 37-42.
9. Dunaiski PE, Retief JV, Barnardo C (2010). *Harmonization of South African Standards for Structural Design to International Practice*. 9th Pacific Structural Steel Conference (PSSC 2010), Beijing, China, China Architecture and Building Press 2010: 14-21.
10. Dunaiski PE, Retief JV, Goliger AM (2006). *Proposed new South African loading code SANS 10160*. Steel 50 Conference, Johannesburg, South Africa, SA Institute of Steel Construction 2006: 1-11.
11. Dymond JS, Dunaiski PE, Retief, JV, De Lange JH (2006). *The future of crane support structure design in the South African context*. Steel 50 Conference, Johannesburg, South Africa, SA Institute of Steel Construction 2006: 1-12.
12. Dymond JS, Retief JV, Dunaiski PE (2009). Reliability assessment of crane induced actions. In Retief & Dunaiski (Ed) *Background to SANS 10160* Sun Media, pp 297-306.
13. Goliger AM, Kruger AC, Retief JV (2013). Representivity of wind measurements for design wind estimates. EACWE13, Cambridge UK.
14. Goliger AM, Retief JV (2001). *Background to Wind Damage model for Disaster Management in South Africa.* Zingoni A (Ed) Proceedings of the International Conference SEMC 2001, Cape Town, 2 - 4 April 2001 Elsevier.
15. Goliger AM, Retief JV (2002). *Identification of zones of strong wind events in South Africa*. J of Wind Engineering and Industrial Aerodynamics, 90 1227-1235.
16. Goliger AM, Retief JV (2007). Severe wind phenomena in southern Africa and the related damage. J Wind Engineering and Industrial Aerodynamics, 2007; 95 : 1065-1078.
17. Goliger AM, Adam BF, Retief JV (2002). *Proposed characterisation of South African thunderstorms; generic footprints and rate of occurrence*. Proceedings of the 2nd International Symposium on Advances in Wind & Structures, Busan, Korea, August 2002, pp. 667-674.
18. Goliger AM, Retief JV, Dunaiski PE (2009). Review of codification of wind-loading for structural design. In Retief & Dunaiski (Ed) *Background to SANS 10160* Sun Media, pp 129-148.
19. Goliger AM, Retief JV, Dunaiski PE, Kruger A (2009). Revised wind-loading design procedures for SANS 10160. In Retief & Dunaiski (Ed) *Background to SANS 10160* Sun Media, pp 149-166.
20. Goliger AM, Retief JV, Niemann H-J (2003a). *Wind damage / disaster model, Part 1: Generic algorithm*. 11th International Conference on Wind Engineering, Lubbock, Texas, 2 – 5 June 2003.
21. Goliger AM, Retief JV, Niemann H-J (2003b). *Wind damage / disaster model, Part 2: Application*. 11th International Conference on Wind Engineering, Lubbock, Texas, 2 – 5 June 2003.
22. Goliger AM, Retief JV, Niemann H-J (2004). *Development process and components of a wind disaster model for South Africa*. Proceedings 2nd SEMC, Cape Town, SA. 5 – 7 July 2004, pp 843-849. A Zingoni (Ed). Taylor and Francis, London.
23. Goliger A, Zuranski J, Gizejowski M, Gaczek M, Retief J, Kruger A, Dunaiski P, Fiszer S, Cwik M (2013). *Comparative study between Poland and South Africa: Wind climates, the related damage and*

implications of adopting the Eurocode for wind action on buildings. Archives of Civil Engineering, LIX, 1, 2013, pp 51 – 95.

24. Holický M, Retief JV (2005). *Reliability Assessment of Alternative Load Combination Schemes for Structural Design*. Journal SAICE, Vol. 25, No 1, pp 15 – 20, 2005.
25. Holicky M, Retief JV (2010). *Probabilistic basis of present codes of practice*. 4th International Conference on Structural Engineering, Mechanics and Computation (SEMC 2010), University of Cape Town, Cape Town, South Africa, CRC Press 2010: 397-402.
26. Holicky M, Retief JV (2011). *Theoretical basis of the target reliability*. 9th International Probabilistic Workshop, Braunschweig, 17-18 November 2011.
27. Holicky M, Retief JV, Diamantidis D, Viljoen C (2015). On standardisation of the reliability basis of structural design. *12th International Conference on Applications of Statistics and Probability in Civil Engineering, Vancouver, Canada, July 12-15, 2015.*
28. Holický M, Retief JV & Dunaiski PE (2007). *The reliability basis of design for structural resistance*. SEMC 2007, Cape Town South Africa, Millpress 2007: 1735-1740.
29. Holický M, Retief JV & Dunaiski PE (2009). The reliability basis of Eurocode. Chapter 1-4 in Retief & Dunaiski (Ed) *Background to SANS 10160* Sun Media, pp 85-103.
30. Holicky M, Retief JV, Maes M, Faber MH (2010). New Work Item Proposal for the revision of ISO 2394. Submitted to ISO TC98, 28th Annual Meeting, Delft, Netherlands.
31. Holicky M, Retief JV, Sykora M, (2014). *General approach to model uncertainty.* Proceedings 12th International Probabilistic Workshop, Bauhaus University Weimar, pp 99-109.
32. Holicky M, Retief JV, Sykora M, (2015). *Assessment of model uncertainty for structural resistance.* Accepted by Probabilistic Engineering Mechanics Journal, Elsevier.
33. Holicky M, Retief JV, Wium JA (2009). *Probabilistic design for cracking of concrete structures*. 7th International Probabilistic Workshop, TNO Nederland, Delft, Netherlands, TU Delft 2009: 87-98.
34. Holicky M, Retief JV, Wium JA (2010). Partial factors for selected reinforced concrete members: Background to a revision of SANS 10100-1. *Journal of the South African Institution of Civil Engineering* 2010; 52(1) : 36-44.
35. Holicky M, Sykora M, Barnardo-Viljoen C, Mensah KK, Retief JV (2013). *Model uncertainty in reliability analysis of reinforced concrete structures.* 5th International Conference on Structural Engineering Mechanics and Computation, SEMC 2013. 2-4 September 2013, Cape Town.
36. Holicky M, Sykora M, Retief JV (2014). *General approach to model uncertainty.* Proceedings 12th International Probabilistic Workshop, pp 99-109. Bauhaus-Universität Weimar.
37. Huber UA, Retief JV, Wium JA (2004). *Structural concrete shear resistance*. Conference: Developing concrete to serve practical needs. 13-14 October 2004, Midrand, South Africa. Pp 113-125.
38. Jacobsohn M, Retief JV, Dunaiski PE (2010). *Assessment of robustness in the design of steel structures,* Zingoni (Ed) Advances and Trends in Structural Engineering, Mechanics and Computation, 639-642 Taylor & Francis ISBN 978-0-415-58472-2.
39. Kruger AC, Goliger AM, Larssén XG, Retief JV (2014). *Optimal Application of Climate Data to the Development of Design Wind Speeds*. 94th AMS Annual Meeting, 2-6 February 2014, Atlanta Ga, USA.
40. Kruger AC, Goliger AM, Retief JV, Sekele S (2010). Strong wind climatic zones in South Africa. *Wind and Structures Journal* 2010; 13(1) : 37-55.
41. Kruger AC, Goliger AM, Retief JV, Sekele S (2012). Clustering of extreme winds in the mixed climate of South Africa. *Wind and Structures Journal* 2012; 15(2) : 87-109.
42. Kruger AC, Retief JV, Goliger AM (2013a). Strong winds in South Africa: I Application of estimation methods. *Journal of the South African Institute of Civil Engineering*. Vol 55, No 2, pp 29-45.
43. Kruger AC, Retief JV, Goliger AM (2013b). Strong winds in South Africa: II Mapping of updated statistics. *Journal of the South African Institute of Civil Engineering*. Vol 55, No 2, pp 46-58.
44. Kruger AC, Retief JV, Goliger AM (2013c). Directional analysis of extreme wind under mixed climate conditions. EACWE13, Cambridge UK.
45. Kruger AC, Goliger AM, Retief JV, (2015). Revised design wind map for SANS 10160-3 – Report to SABS TC098/SC02
46. Marengwa J, Retief JV (2009a). *Risk based maintenance planning for flood protection walls*. 31st International Conference on Coastal Engineering, Convention Centre Hamburg, Hamburg, Germany, World Scientific 2009: 4315-4327.
47. Marengwa J, Retief JV (2009b). *Risk management of concrete flood protection walls by application of Bayesian networks*. 7th International Probabilistic Workshop, TNO Nederland, Delft, Netherlands, TU Delft 2009: 359-383.
48. McLeod CH, Retief JV, Wium JA (2013). *Reliability of EN 1992 crack model applied to South African water retaining structures.* 5th International Conference on Structural Engineering Mechanics and Computation, SEMC 2013. 2-4 September 2013, Cape Town.

49. McLeod C, Wium JA, Retief JV (2012). Investigation of reliability model for cracking in South African reinforced concrete water retaining structures. 10th International Probabilistic Workshop, Stuttgart, Germany, IGS 2012: 141-155.
50. Mensah KK, Barnardo-Viljoen C, Retief JV (2013). *A comparison of the Variable Strut Inclination and alternative stirrup design methods.* 5th International Conference on Structural Engineering Mechanics and Computation, SEMC 2013. 2-4 September 2013, Cape Town.
51. Mensah KK, Retief JV, Barnardo C (2010). *Structural reliability and the basis of design for concrete structures*. 4th International Conference on Structural Engineering, Mechanics and Computation (SEMC 2010), Zingoni A (Editor), Cape Town, South Africa, CRC Press 2010: 1047-1050.
52. Mensah KK, Retief JV, Barnardo-Viljoen C (2013a). *Review of the reliability basis of structural design and its application to structural concrete in South Africa.* International Conference on Advances in Cement & Concrete Technology in Africa (ACCTA 2013). 28-30 January 2013, Johannesburg, South Africa.
53. Mensah KK, Retief JV, Barnardo-Viljoen C (2013b). *Eurocode 2's Variable Strut Inclination Method for shear, it's modelling uncertainty and reliability calibration*. Engineering a Concrete Future: Technology Modeling & Construction – fib symposium. 22-24 April 2013, Tel Aviv, Israel.
54. Mensah KK, Retief JV, Barnardo-Viljoen C (2013c). *Reliability based application of Eurocode 2's Variable Strut Inclination Method for shear*. 11th International Conference on Structural Safety & Reliability (ICOSSAR). 16-20 June 2013, New York NY, USA.
55. Phoon KK, Retief JV (2015). ISO2394:2015 Annex D (Reliability of Geotechnical Structures). Accepted by GeoRisk: Assessment and Management of Risk for Engineered Systems and Geohazards,
56. Phoon KK, Retief JV, Ching J, Scwekendiek, Wang Y, Zhang Y (2015). Some Observations on ISO 2394:2015 Annex D (Reliability of Geotechnical Structures). Submitted to Structural Safety, Elsevier.
57. Phoon KK, Retief JV (Editors) (2015). *Reliability of Geotechnical Structures in ISO 2394.* Contracted to Taylor and Francis Group.
58. Retief JV (1996a). Structural codes of practice and development - a risk perspective. Proceedings: Structural Steel Developing Africa. SAISC, Johannesburg (1996a).
59. Retief JV (1996b). The reliability of ACI and BS methods of deflection design of reinforced concrete members. Paper 582. 21st Conference on Our World in Concrete and Structures. Singapore (1996b).
60. Retief JV, Barnardo-Viljoen C, Dithinde M (2011). Reliability basis for adopting Eurocodes as South African standards. ICASP 11, Zurich, 1-4 August 2011.
61. Retief JV, Barnardo-Viljoen C, Holicky M (2013). *Probabilistic models for the design of structures against wind loads.* 5th International Conference on Structural Engineering Mechanics and Computation, SEMC 2013. 2-4 September 2013, Cape Town.
62. Retief JV, De Villiers PJ (2005). *Reliability Model for Imposed Loads on Inaccessible Roofs of Light Industrial Steel Buildings*. Australian Structural Engineering Conference 2005, Newcastle,11 – 14 September 2005.
63. Retief JV, Diamantidis D Barnardo-Viljoen C, Van der Klashorst E, (2013). *Extreme actions and climate change: Experience gained in South Africa and Germany*. Civil Engineering and Environmental Systems Journal. Taylor and Francis. http://dx.doi.org/10.1080/10286608.2014.912640
64. Retief JV, Diamantidis D, Barnardo-Viljoen C, Van der Klashorst E, (2014). *Application of Risk Based Infrastructure Design Concepts to Provisions for Cliamte Change*. 94th AMS Annual Meeting, 2-6 February 2014, Atlanta Ga, USA.
65. Retief JV, Dithinde M (2013). Pile design practice in southern Africa: II Implicit reliability of existing practice. *Journal of the South African Institute of Civil Engineering.* Vol 55, No 1, April 2013 pp 72-79.
66. Retief JV, Dunaiski PE (2006). *Structural design standards in the context of decisions in civil engineering*. 2nd IFED Forum, April 26-29, 2006 – Lake Louise, Canada.
67. Retief JV, Dunaiski PE (Editors) (2009). *Background to SANS 10160 – Basis of Structural Design and Actions for Buildings and Industrial Structures*. Sun Press. ISBN 978-1-920338-10-7.
68. Retief JV, Dunaiski PE (2009a). The Limit States Basis of Structural Design for SANS 10160-1. Chapter 1-2 in Retief & Dunaiski (Ed) *Background to SANS 10160* Sun Media, pp 25-55.
69. Retief JV, Dunaiski PE (2009b). Review of provisions for General Actions in SANS 10160. Chapter 2-1 in Retief & Dunaiski (Ed) *Background to SANS 10160* Sun Media, pp 105-128.
70. Retief JV, Dunaiski PE (2011). *The new South African Loading Code SANS 10160*. Concrete-Beton 2011; 127 : 8-14.
71. Retief JV, Dunaiski PE, Day PW (2009a). An overview of the revision of the South African Loading Code SANS 10160. In Retief & Dunaiski (Ed) *Background to SANS 10160* Sun Media, pp 1-24.
72. Retief JV, Dunaiski PE, De Villiers PJ (2001). *An Evaluation of Imposed loads for Application to Codified Structural design.* Zingoni A (Ed) Proceedings International Conference SEMC 2001, Cape Town, Elsevier.

73. Retief JV, Dunaiski PE, Dymond JS, Wium JA (2006). *Considerations to establish a basis for structural concrete design for application in the development of an African Concrete Code*. African Concrete Code Symposium 2006, Stellenbosch, 29-30 November 2006.
74. Retief JV, Dunaiski PE, Holicky M (2009). Review of Eurocode from the perspective of the revision of SANS 10160. In Retief & Dunaiski (Ed) *Background to SANS 10160* Sun Media, pp 57-83.
75. Retief JV, Dunaiski PE, Wium JA (2005). *South African structural design standards development.* The First African Concrete Code Symposium. Tripoli, Lybia. 28 – 29 November 2005.
76. Retief JV, Holicky M (2010). *Review of the ISO 2394 reliability basis of structural design*. 4th International Conference on Structural Engineering, Mechanics and Computation (SEMC 2010), University of Cape Town, Cape Town, South Africa, CRC Press 2010: 1043-1046.
77. Retief JV, Maritz G, Ter Haar TR, Brand WW, Muhimua-Joao A (2002). *Structural concrete reliability models for design code assessment.* Concrete for the 21st Century, 13 - 14 March 2002. Eskom Convention Centre, Midrand, Gauteng.
78. Retief JV, Wium JA (2010). *Principles for development of standards for structural design*. IABSE-*fib Conference Codes in Structural Engineering*. Dubrovnik, Croatia. Pp 181-188.
79. Retief JV, Wium JA (2012). *Principles and application of Structural Design Code development in South Africa*. Structural Engineering International 2012; 22(2) : 182-189.
80. Ter Haar TR, Retief JV (2001). *A Methodology for Structural Code Calibration.* International Conference: Safety, Risk and Reliability, Malta, March 21-23 2001.
81. Ter Haar TR, Retief JV (2002). *Resistance factors for structural concrete design codes as derived from loading code calibration.* Concrete for the 21st Century, 13 - 14 March 2002. Eskom Convention Centre, Midrand, Gauteng.
82. Ter Haar TR, Retief JV, Dunaiski PE (1998): Towards a more rational approach of the serviceability limit states design of industrial steel buildings. Journal of Constructional Steel Research Vol 46 No 1-3, pp 152-153, Paper 283. Second World Conference on Steel in Construction. 11-13 May 1998. San Sebastian, Spain.
83. Ter Haar TR, Retief JV, Kemp AR (2001). *Calibration of Load Factors for the South African Loading Code.* Zingoni A (Ed) Proceedings International Conference SEMC 2001, Cape Town, 2 - 4 April 2001 Elsevier.
84. Warren JS, Dunaiski PE, Retief JV (2004). *A comparison between crane induced load effects from SABS and Eurocode.* 2nd SEMC, Cape Town, SA. 5 – 7 July 2004, pp 887-883. A Zingoni (Ed). Progress in Structural Engineering, Mechanics and Computation. Taylor and Francis, London.
85. Warren JS, Retief JV, Dunaiski PE (2005). *Reliability models of overhead traveling crane loading for code calibration*. 9th International Conference on Structural Safety and Reliability (ICOSSAR 2005), Rome, 19 – 23 June 2005.
86. Wium JA Retief JV (2004). *The South African loading code and detailing of reinforced concrete elements for seismic loading*. Conference: Developing concrete to serve practical needs. 13-14 October 2004, Midrand, South Africa. Pp 127-138.
87. Wium JA, Retief JV, Viljoen C (2014). *Lessons from Development of Design Standards in South Africa.* 37th IABSE Symposium, Éngineering for Progress', Sept 3-5, 2014, Madrid, Spain.