TECHNICAL PAPER

JOURNAL OF THE SOUTH AFRICAN INSTITUTION OF CIVIL ENGINEERING

Vol 47 No 1, 2005, Pages 15–20, Paper 583

Reliability assessment of alternative Eurocode and South African load combination schemes for structural design

M Holický (Visitor) and J V Retief (Fellow)

The basic European standard for design of buildings and other engineering works, EN 1990 'Basis of structural design', provides alternative design procedures and parameters for which national choice is allowed. One of the most important decisions relates to the fundamental combinations of actions for persistent and transient design situations. It appears that the reliability of structural members, designed according to the alternative combination rules provided in EN 1990, may vary considerably.

In the presented study probabilistic methods of structural reliability are used to identify characteristic features of alternative combination rules and to propose a new efficient rule. The reliability performance of the present South African load and combination factors are compared to the alternative schemes.

It is confirmed that although the minimum level of reliability is achieved for the alternative load combination schemes, levels of reliability and consistency vary considerably over a limited but representative range of design situations. It is also shown that reliability of reinforced concrete members depends on the structural configuration.

INTRODUCTION AND BACKGROUND

Aligning national structural design codes of practice to international standards is motivated largely by professional and academic considerations. Commercial concerns recently provided a strong material motivation for international harmonisation of structural design standards in the context of continued integration of the global economy. The strongest driver for such harmonisation, however, is the sharing of technological advances by participating national industries.

Conversion of South African structural design codes of practice to limit states design during the last few decades (Kemp *et al* 1987; Milford 1988; SABS 1989; SABS 1992) is an illustration of national benefits derived from international technological development. The limited resources available for the future development of structural design codes in South Africa make it essential to carefully consider international development in this field and adapt advances efficiently into local practice.

The most notable recent development of international structural design standards is the establishment of Eurocode, such as outlined for instance by Gulvanessian (2003). The extensive degree of harmonisation that is being achieved across a wide range of national structural design practices, provision for an extended range of structural types and the various structural materials is captured in Eurocode EN 1990 'Basis of structural design' (CEN 2002). Such harmonisation could only have been achieved by improving the rational basis of design and utilising an extended reliability framework to provide for the wider range of design situations to be considered (Gulvanessian *et al* 2002).

Consideration of Eurocode as reference to future South African code development is not only justified as a measure of improved international harmonisation and providing a coherent technological base for the process of advancing the scope and standard of South African codes: The fact that the British codes, which traditionally served as reference to South African codes, are being replaced by Eurocode makes this a logical course of action. The transition from British codes to Eurocode can also serve as a model to South African efforts.

Structural design functions to relate actions and resistance

The reliability basis for designing the range of structures, structural materials, safety and functional requirements and the actions to which these facilities are exposed form a vital framework for a coherent suite of structural design standards. The design functions which relate actions and combinations of actions with structural resistance or functional performance in turn translate the conceptual reliability framework into practical design procedures.

Alternative design functions and related partial safety factors for load combinations are provided in EN 1990 in order to accommodate the range of design practices of Eurocode member states and to allow for nationally determined levels of reliability. However, the alternative design functions and parameters also result in significantly different design results and levels of reliability. The selection of appropriate design functions and calibration of nationally determined parameters is therefore an important task for any national regulatory authority.

The extended range of design functions allowed for in EN 1990 also accommodate the present practice for load combination schemes and partial safety factors used in the South African Loading Code, and thereby removes

MILAN HOLICKÝ was born in 1943 and received his civil engineering degree from the Czech Technical University (CTU) in 1965 and his PhD from the University of Waterloo, Canada, in 1971. Since 1965 he has been employed at the Klokner Institute of CTU in Prague and lectures at the Faculties of Civil Engineering and Architecture. He is an author or co-author of more than 280 scientific publications, including textbooks and five monographs. He is actively participating in international research within CIB, JCSS and ISO and is a member of CEN TC 250 Structural Eurocodes. He is coordinator of the European Project Development of Skills Facilitating Implementation of Structural Eurocodes.

JOHAN RETIEF retired from the Department of Civil Engineering, University of Stellenbosch, in 2001 and as director of the Institute for Structural Engineering in 2003, but is still actively involved in research. He was employed by the Atomic Energy Corporation before joining the University of Stellenbosch in 1990. His field of interest in research is risk and reliability in civil engineering, with special reference to structural reliability and its application to codified design, and project risk management. He graduated from the University of Pretoria in 1962, studied at Imperial College, Stanford University, and received a DSc Eng 1978, again from the University of Pretoria.

a critical source of incompatibility between South African structural design codes and Eurocode, as evaluated by Kemp *et al* (1998). The reliability performance of the alternative design schemes of Eurocode and its comparison of South African practice is therefore a key consideration in evaluating the future use of Eurocode as reference to South African code development activities.

Scope of paper

This paper considers and evaluates the alternative design functions and load combination schemes provided for in Eurocode EN 1990 (CEN 2002). The evaluation is done in terms of attaining a level of reliability better than the target value, to be achieved consistently across a limited range of design conditions which are representative of practical design situations. For this purpose the resistance of two structural concrete elements are used. Furthermore, an innovative load combination scheme is proposed to improve some of the limitations of the EN 1990 design functions. The South African design functions, which agree in format with one of the EN 1990 alternatives but use different partial factors, are then compared with the other design functions.

Probabilistic models of basic variables are specified, taking into account principles provided in ISO standards (ISO 1998; ISO 2001) and recommendations of the Joint Committee on Structural Safety (JCSS 1996; JCSS 2001). Time variant random properties of actions are approximated using the simplified Turkstra (1970) rule. Results of the reliability analyses are compared using available software products (Petschacher 1997; RCP Munich 1999), a specialised program intended for calibration purposes (JCSS 2003) and own products developed using MATLAB and MATHCAD software.

Applied probabilistic methods of structural reliability are used to identify characteristic features of each combination rule and to formulate general recommendations. Newly obtained results concerning reinforced concrete structures extend findings of earlier studies concerning members made of different materials (SAKO 1999; Holický & Marková 2000; 2003). It appears that the reliability of structures, designed according to the alternative combination rules, may vary considerably. The optimum combination rule should comply with a required reliability level (say the reliability index 3,8 for a 50-year time period) and should provide uniform distribution of reliability with different ratios of variable and the total load.

The combination rules presently applied in South Africa (Ter Haar *et al* 2001) and their application to concrete resistance calibration (Ter Haar & Retief 2002; Retief *et al* 2002) are considered. General recommendations are based on probabilistic analysis of basic concrete members. It appears that further calibration concerning more complicated types of concrete members is needed and possible future studies are proposed.

FUNDAMENTAL LOAD COMBINATIONS

Alternative design schemes

In the following analysis the combination of three actions is considered: permanent action G, imposed load Q (leading) and wind W (accompanying). The EN 1990 design procedure (CEN 2002) for the fundamental combination of these loads in permanent and transient design situation introduces three alternative procedures denoted here as A, B and C. Assuming linear behaviour of structural members, actions G, Q and W and their characteristic values G_k, Q_k and W_k denote generally appropriate load effects (not the original actions).

Design Scheme A

Considering the formula (6.10) in EN 1990, the design value of action effect E_d is given in terms of the partial factors γ_G, γ_Q and γ_W for actions G, Q and W respectively (for EN 1990 partial factors for both variable actions are equal, $\gamma_Q = \gamma_W$) and the combination factor ψ as

$$E_d = \gamma_G\, G_k + \gamma_Q\, Q_k + \gamma_W\, \psi_W\, W_k \tag{1}$$

Design Scheme B

An alternative procedure is provided in EN 1990 by twin expressions (6.10a) and (6.10b) where the combination factor ψ is also applied to Q in the first expression and a reduction factor ξ is applied to the permanent action in the second expression

$$E_d = \gamma_G\, G_k + \gamma_Q\, \psi_Q\, Q_k + \gamma_W\, \psi_W\, W_k \tag{2}$$

$$E_d = \xi\, \gamma_G\, G_k + \gamma_Q\, Q_k + \gamma_W\, \psi_W\, W_k \tag{3}$$

The less favourable action effect from (2) and (3) should be considered.

Design Scheme C

In addition, EN 1990 allows further modification of the alternative B simplifying equation (2) by considering permanent loads only. Thus the load effect is then the less favourable action effect resulting from (3) and the simple relationship $E_d = \gamma_G\, G_k$.

Design Scheme D

In addition to the Eurocode combinations A, B and C, combination D is considered here as a modification of the combination A, considering the original partial factor of permanent load $\gamma_G = 1{,}35$ and a reduced partial factor of variable actions γ_Q given as

$$\gamma_Q = 1 + 0{,}5\, \chi \tag{4}$$

Here the parameter χ denotes the ratio of variable actions Q_k+W_k to the total load G_k+Q_k+W_k given as

$$\chi = (Q_k+W_k)/(G_k+Q_k+W_k) \tag{5}$$

Note that the parameter χ is often applied to investigate load effects under various intensities of variable and permanent actions. A realistic range of χ from 0,1 to 0,6 is found for structures in practice. However, in some special cases the load ratio χ may be very low if not zero (eg for underground garages or where geotechnical loads play a prominent role).

If the leading action in equations (1) and (2) is wind W, then instead of reducing the wind action W by the factor ψ_W, the imposed load Q should be reduced by an appropriate factor ψ_Q.

Range of applicability

For a given design value of the load effect E_d the characteristic values of individual actions G_k, Q_k, W_k can be expressed using variables χ and the ratio $k = W_k/Q_k$ of the accompanying action W_k to the main action Q_k, as follows:

$$G_k = \frac{E_d}{[\xi]\gamma_G + \dfrac{([\psi_Q]\gamma_Q + k[\psi_W]\gamma_W)\chi}{(1+k)(1-\chi)}}\;; \tag{6a}$$

$$Q_k = \frac{\chi\, G_k}{(1+k)(1-\chi)}\;; \tag{6b}$$

$$W_k = k\, Q_k \tag{6c}$$

The factors ξ, ψ_Q and ψ_W indicated in (6a) in square brackets are applied in the same way (either yes or no) as in equations (1) to (4) for alternative combination rules A, B and C.

For Design Scheme A, equation (1) is valid in the whole range $0 \le \chi \le 1$, whereas in Design Scheme B, equation (2) is valid in the range $0 \le \chi \le \chi_{lim,B}$ and equation (3) is valid in the range $\chi_{lim,B} \le \chi \le 1$. Correspondingly, for Design Scheme C equation (4) is valid in the range $0 \le \chi \le \chi_{lim,C}$ and equation (3) in the range $\chi_{lim,C} \le \chi \le 1$. The limiting values $\chi_{lim,B}$ and $\chi_{lim,C}$ derived from equations (2) to (5) are as follows:

$$\chi_{lim,B} = \frac{\gamma_G(1-\xi)(1+k)}{\gamma_G(1-\xi)(1+k) + \gamma_Q(a-\psi_Q) + \gamma_W k(b-\psi_W)} \tag{7}$$

$$\chi_{lim,C} = \frac{\gamma_G(1-\xi)(1+k)}{\gamma_G(1-\xi)(1+k) + \gamma_Q a + \gamma_W k b} \tag{8}$$

where the auxiliary variable $a = 1$ and $b = \psi_W$ for $k \le (1-\psi_Q)/(1-\psi_W)$ (imposed load Q is the leading action) and $a = \psi_Q$ and $b = 1$ when $k > (1-\psi_Q)/(1-\psi_W)$ (wind load W is the leading action).

REPRESENTATIVE RESISTANCE OF REINFORCED CONCRETE MEMBERS

Two structural elements have been selected to be representative of reinforced concrete structures in this assessment, namely the flexural resistance of a tensile reinforced concrete beam and the resistance of a reinforced concrete column subjected to a concentric axial load.

Reinforced concrete beam

The design value of flexural resistance R_d of the reinforced concrete beam is given as

$$R_d = A_s\, f_{yk}/\gamma_m\, [h - d_1 - 0{,}5\, A_s\, (f_{yk}/\gamma_m)/(\, b\, \alpha\, f_{ck}/\gamma_c)] \tag{9}$$

where A_s denotes the area of reinforcement, f_{yk} and f_{ck} denote the characteristic strengths of reinforcement and concrete, h is the height and b is the width (considered here as a deterministic quantity equal to 0,3 m) of the beam cross-section, d_1 axial distance of the bars to the beam bottom, α the factor taking into account the long-term load effects on concrete compressive strength (considered here as a deterministic quantity equal to 0,85), γ_m and γ_c the partial material factors for steel and concrete strength considered here to have the values 1,15 and 1,5 respectively.

It is further assumed that the beam is designed (with height h and reinforcement area A_s specified) on the basis of an 'economic design', that is, when $R_d = E_d$ and

Table 1 Probabilistic models of basic variables

Type of variable	Symbol X	Basic variable	Distr	Units	Char value	μ_X	σ_X
Action	G	Permanent action	N	MN/m²	G_k	G_k	0,1 G_k
	Q	Imposed (50 years)	Gum	MN/m²	Q_k	0,6 Q_k	0,21 Q_k
	W	Wind (1 year)	Gum	MN/m²	W_k	0,3 W_k	0,15 W_k
	W	Wind (50 years)	Gum	MN/m²	W_k	0,7 W_k	0,245 W_k
Material properties	f_c	Concrete strength	LN	MPa	20	30	5
	f_y	Reinforcement strength	LN	MPa	500	560	30
Geometric data	A_s	Reinforcement area	Det	m²	nom	nom	0
	h	Beam height	N	m	0,6	0,6	0,008
	h, b	Column dimensions	N	m	0,3	0,3	0,01
	d_1	Reinforcement distance	Gam	m	0,03	0,03	0,006
Model uncertainties	K_E	Load uncertainty	LN	–	1,0	1,00	0,05
	K_R	Resistance uncertainty	LN	–	1,0	1,10	0,15

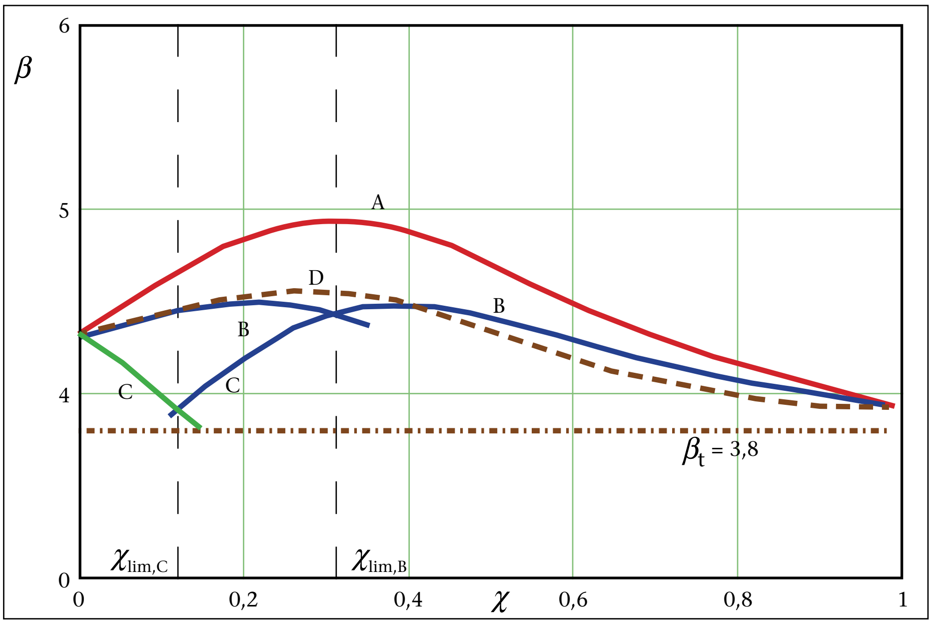

Figure 1 Reinforced concrete beam reliability index β vs load ratio χ for ρ = 1 %

no additional safety margin due to realistic dimensioning (that leads to the inequality $R_d > E_d$) is taken into account.

In the reliability analysis a structure is usually considered to be safe if the resistance R is greater than the load effect E, both considered as random variables. Thus, when the limit state function (reliability margin) $g(X) = R - E$ is greater than 0, the structure is safe; X being the vector of basic variables. In the case of a beam the limit state function can be written as

$$g(X) = K_R\, A_s\, f_y\, [h - d_1 - 0{,}5\, A_s\, f_y\, /(b\, \alpha\, f_c)] - K_E\,(G + Q + W) \qquad (10)$$

where K_R and K_E are coefficients of model uncertainties for resistance R and action effects E. Note that all variables in equation (10) are considered as random variables having a certain type of probability distribution.

Reinforced concrete column

The design value of the resistance R_d for the short reinforced concrete column assuming cross-sectional dimensions $h \times b$ and negligible eccentricity is given as

$$R_d = A_s\, f_{yk}/\gamma_m + 0{,}8\, h\, b\, f_{ck}/\gamma_c \qquad (11)$$

The column design parameters are again determined considering an 'economic design', that is, when $R_d = E_d$. In case of the column the limit state function can be written as:

$$g(X) = K_R\,(A_s\, f_y + 0{,}8\, h\, b\, f_c) - K_E\,(G + Q + W) \quad (12)$$

where K_R and K_E are again coefficients of model uncertainties for variables R and E.

PRINCIPLES OF RELIABILITY ANALYSIS

The probability of failure of the structure P_f is the basic reliability measure used in this study. It can be expressed on the basis of a limit state performance function $g(X)$ defined in such a way that a structure is considered to survive if $g(X)> 0$ and to fail if $g(X) \leq 0$. An example of the function g(X) is given by equations (10) and (12). In a general case the failure probability P_f can be determined using the integral

$$P_f = \text{Prob}(g(X) \leq 0) = \int_{g(X)\leq 0} \varphi_g(X)\,\mathrm{d}X \qquad (13)$$

where $\varphi_g(X)$ denotes joint probability density distribution of the vector of basic variables X, which may not be, however, available.

Assume that both the resistance $R(X)$ and the load effect $E(X)$ represent a single variable Z used to analyse structural performance (eg axial force, bending moment or stress that is represented by $R(Z)$ and $E(Z)$). Then the integration indicated in expression (13) may be simplified and the probability P_f can then be expressed as:

$$P_f = \text{Prob}(g(Z) \leq 0) = \int_{-\infty}^{\infty} \varphi_E(Z)\Phi_R(Z)\,\mathrm{d}Z \qquad (14)$$

where $\varphi_E(Z)$ denotes the probability density function of $E(Z)$; $\Phi_R(Z)$ the distribution of $R(Z)$. To use equation (14) both the probability density function $\varphi_E(Z)$ and the distribution function $\Phi_R(Z)$ must be known (at least in an approximate form). Procedures based on expression (14) are used in this study.

An alternative measure of the reliability level (used in this study) is provided by the reliability index β (see Annex C of EN 1990 (CEN 2002)) related to the probability of failure P_f as

$$P_f = \Phi(-\beta) \qquad (15)$$

where Φ is the cumulative distribution function of the standardised normal distribution. The reliability index β is frequently used, as its numerical values are more convenient to handle than values of failure probability P_f. EN 1990 recommends a target value of reliability index β_t = 3,8 (for a one year period β_t = 4,7) that corresponds to the probability of failure $P_f = 7{,}24 \times 10^{-5}$ for the Ultimate limit states of buildings designed for a fifty-year period.

PROBABILISTIC MODELS OF BASIC VARIABLES

As mentioned above in the reliability analysis, all basic variables are considered as random variables, usually described by a certain type of probability distribution and parameters. The probabilistic models of basic variables X used in this study are summarized in table 1. The models of basic variables indicated in table 1 are chosen taking into account data provided by the JCSS (2001). Where appropriate, characteristic values are also selected to be in agreement with the EN 1990 (CEN 2002) specifications, notably for G, Q, W, f_c and f_y. Note that the reinforcement area A_s, together with the beam width b = 0,3 m and coefficient of long-term concrete strength α = 0,85 for the beam are considered as deterministic values.

RELIABILITY ANALYSIS

The language of technical computing MATHCAD has been used for developing special purpose software products applied in the presented analysis. Commercially available software products (eg VaP (Petschacher 1997); COMREL (RCP Munich 1999)) can be used in more complicated cases than considered here (when expression (14) cannot be used directly). These software products were used in this study to verify results obtained by developed software products based on numerical integration indicated by expression (14).

Results of analysis

The results of the reliability analysis, indicated by the reliability index β versus the ratio of imposed to total load χ, are shown in figures 1 and 2 for the reinforced concrete beam, and in figures 3 and 4 for the column. In both cases, the load ratio of variable actions is taken as k = 0 (ie without wind load) which proves to be more critical than for k > 0

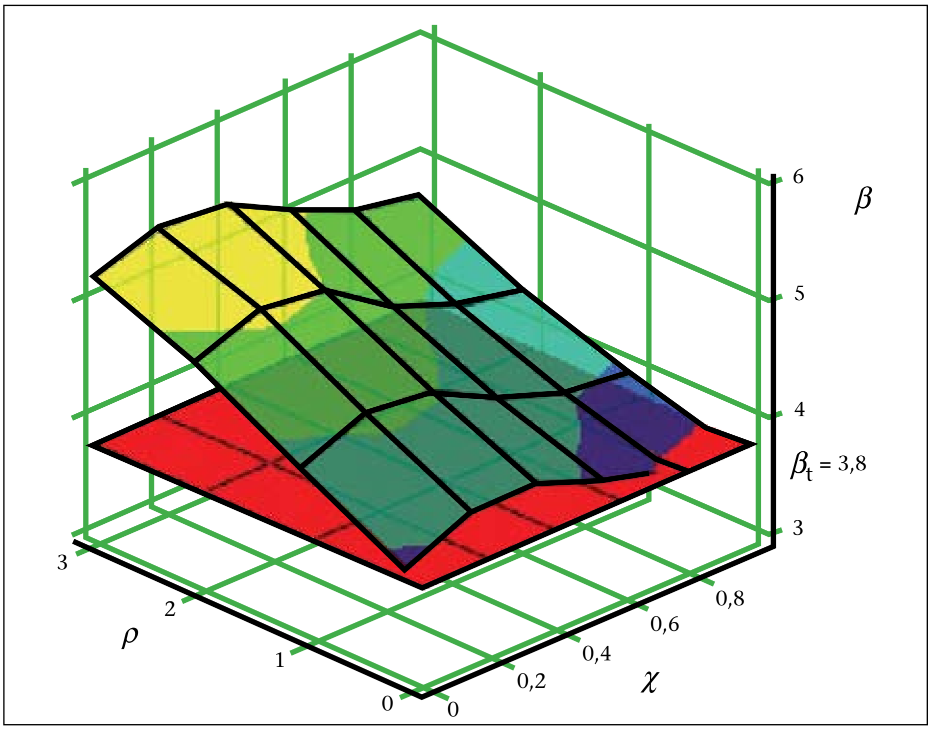

Figure 2 Reinforced concrete beam reliability index β vs reinforcement ratio ρ and load ratio χ for Design Scheme D

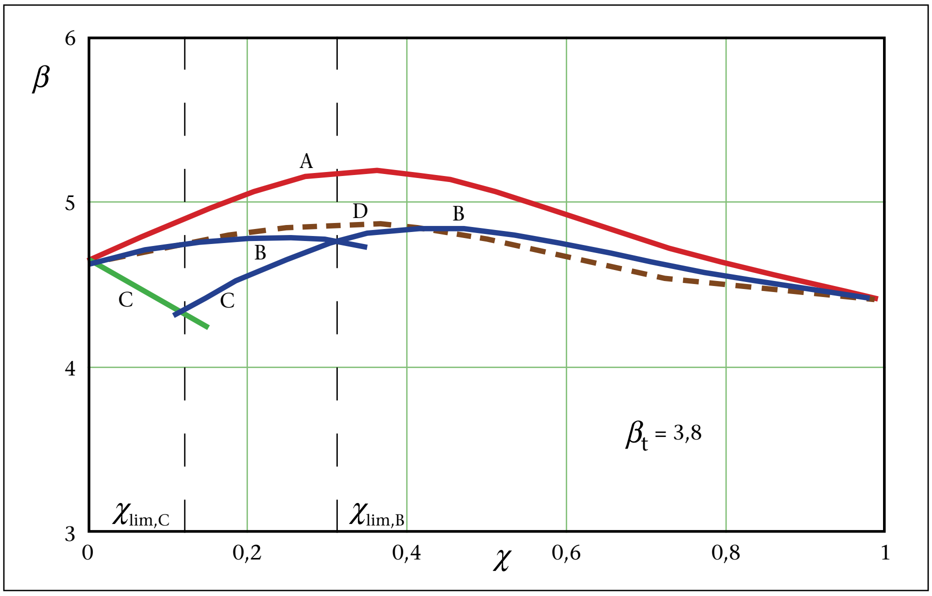

Figure 3 Reinforced concrete column reliability index β vs load ratios χ for ρ = 1 %

(Holický & Marková 2000; 2003; SAKO 1999). Figures 1 and 3 show the results for a reinforcement steel ratio ρ of 1 %, for the various load design combination Design Schemes A, B, C and D. Figures 2 and 4 show the reliability for Design Scheme D as a function of ρ and χ.

Characteristics of reliability

The alternative design schemes are assessed in terms of their ability to achieve the necessary levels of reliability and whether this is done in an economic manner, that is, without unnecessarily high levels of reliability, across the range of representative design situations.

Minimum level of reliability

The results show that the minimum level of reliability can be attained comfortably over a wide range of conditions by all the alternative design schemes. Design situations where the reliability level comes close to the limiting value are somewhat exceptional. This confirms the fact that design schemes are largely sound refinements of experienced-based practice. The essence of the exercise therefore does not seem to be to maintain a minimum level of reliability but rather to utilise an opportunity to improve the efficiency of structural design.

Consistency of reliability

The differences between the alternative design schemes are clearly based on their ability to maintain consistent levels of reliability, or expressed in practical terms, to achieve economically efficient design limits. Design Scheme A shows a particularly large range of reliabilities with β varying by almost a value of 1 unit for beams and 0,7 units for short columns across the range of load ratio χ. The over-conservatism is particularly acute for χ within the range of 0,2 – 0,5 units, which is of particular practical significance.

Alternative Design Scheme B agrees with A at the extremes of load ratios with χ close to 0 and 1 respectively, and also shows the tendency of high values of β within the practical range of load ratios, but with a significant reduction of about 0,4 units of its maximum value. This improvement comes at the difficulty of having to consider two equations for each design situation.

Alternative Design Scheme C manages to shift the transition somewhat outside the range of most frequent design situations. The penalty for this benefit is that the reliability β varies significantly and discontinuously for χ < 0,3 and is particularly low at $\chi_{\text{lim, C}}$, the transition point between the two equations.

A relatively consistent reliability without any discontinuity is achieved with Design Scheme D. By using a single design function, the same level of reliability is achieved as with Design Scheme B.

Reliability across the range of design situations

In addition to considering the load ratio χ, different design situations are also represented in this analysis by the two types of structural elements and their associated failure mechanisms, as well as the parametric assessment of the effect of the steel reinforcement ratio ρ.

Similar results are obtained for beam and short column reliability for the various design schemes in terms of relative values of reliability across the range of load ratios, but with a consistent difference in reliability for the two structural elements. Figures 1 and 3 show that the reliability for beams seems to be lower than that for columns, with differences in β of about 0,3 – 0,4 units.

The sensitivity of reinforced concrete elements to the amount of reinforcement is shown in figures 2 and 4. An important feature is that the trends for the two structural elements are different. With an increase in steel, the reliability of beams *increases* whilst it *decreases* for columns. The increase in β for beams is more than 1 unit with ρ increasing over the parametric range to 3 % and for columns the decrease is about 0,5 units with an increase in ρ to 8 %.

SOUTH AFRICAN COMBINATION RULE

SABS 0160:1989 Design Scheme

The present South African combination rule (SABS 1989), denoted here as Design Scheme E, corresponds to C above, but with the use of different partial factors. For a permanent load G and one variable load Q only, the combination E may be described by twin expressions:

$$E_d = 1,5\ G_k \tag{16}$$

$$E_d = 1,2\ G_k + 1,6\ Q_k \tag{17}$$

Note the difference in values of the partial factor of permanent load in equations (16) (γ_G = 1,5) and (17) (γ_G = 1,2), as compared to the partial factor recommended in EN 1990 (γ= 1,35) which applies to both equations for C. The partial factor for the variable load (γ_Q = 1,6) considered in equation (17) is also

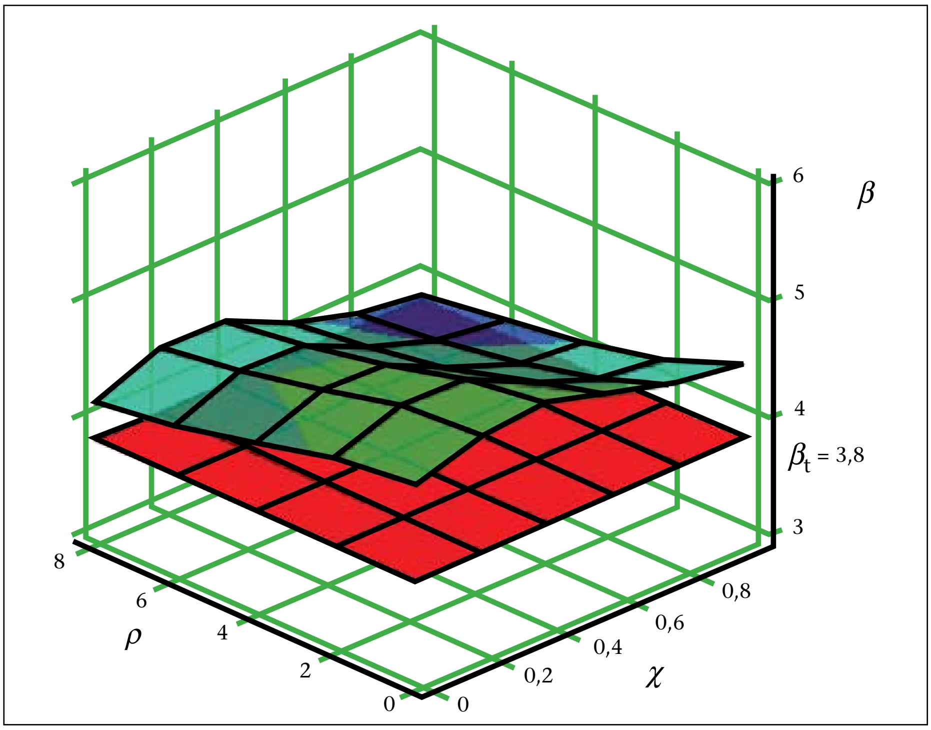

Figure 4 Reinforced concrete column reliability index β vs reinforcement ratio ρ and the load ratio χ for Design Scheme D

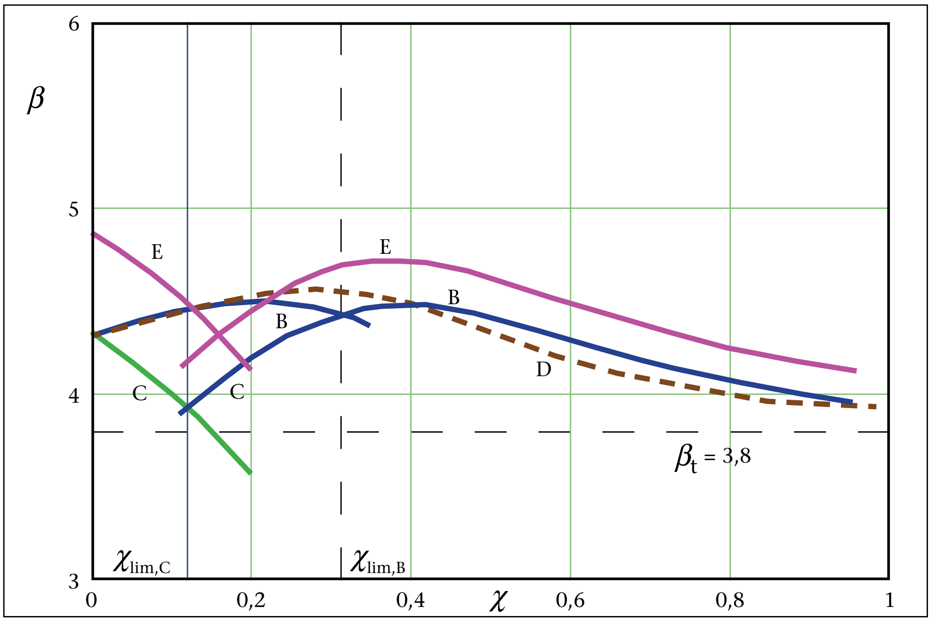

Figure 5 Comparison of Design Scheme E (SABS 0160:1989) with other alternatives for a reinforced concrete beam having the reinforcement ratio ρ = 1

greater than the value of γ_Q = 1,5 recommended in EN 1990.

SABS 0160:1989 Reliability

Considering the reinforced concrete beam having the reinforcement ratio ρ = 1 % and k = 0 (no wind load) and the theoretical models of basic variables indicated in table 1, figure 5 shows the reliability of Design Scheme E, together with that for Design Schemes B, C and D. This can also be compared to figure 1.

The dependence of the reliability β on the load ratio χ for Design Scheme E is similar to that of C but with β higher by about a value of 0,3 units and a larger difference for the lower range of χ. The low minimum reliability at the transition point for C is somewhat moderated by the γ_G = 1,5 of expression (16), but this results in the reliability for permanent load only (χ = 0) being then even higher than for the mid-range of load ratios. Reliability levels are indeed well above the EN 1990 recommended value (β_t = 3,8) and even more so for the present South African Loading Code (SABS 1989) and the draft proposal for the new South African standard (SAICE 2004) (β_t = 3,0).

ASSESSMENT OF DESIGN SCHEMES

Basis of assessment

The first general objective of a design scheme is to maintain levels of reliability above minimum target values. This assessment demonstrates that the alternative design schemes allowed in EN 1990 and in SABS 0160 comply fully with EN 1990 recommendations of a reliability index β of 3,8 for a 50-year time period. The general level of reliability is achieved not only through the partial safety factors introduced by the design schemes, but also due to the conservative bias introduced formally in Eurocode through the characteristic values and the inherent bias of modelling factors (see table 1).

The relative importance of these factors can be estimated by taking the reliability index to be the sum of $\alpha_i\beta_i$ where α_i is a sensitivity factor and β_i an expression of the bias in units of standard deviation of the particular basic variable i (or group of basic variables). According to this approximation the bias of the characteristic values contributes a value of 1,1 units (with α_{char} = 0,65; β_{char} = 1,64) to the total reliability index β value. The contribution from the bias of resistance modelling amounts to a value of 0,4 units. According to JCSS (2001) the resistance modelling bias can be as high as 1,2 (taken as 1,1 in table 1), in which case the contribution of this factor to the total β value could be as high as 0,8 units.

The critical problem however is to achieve at least a minimum level of reliability in a *consistent and economic* manner over a large range of design situations with a minimum of design equations and load and combination factors. Even for the basic case of considering the ratio of permanent to imposed loads, achievement of these objectives is problematical: Partial factors ensuring sufficient reliability when either of these loads dominate, cause uneconomically high reliability in the mid-range of load ratios. The economic importance of the high reliability levels can be assessed by applying a sensitivity analysis to the reliability models reported here. This shows that a reduction of β by 1 unit is achieved by reducing the section area (which can be related to the cost of the structure) by 13 % for the beam and 23 % for the column sections for a given load. This consideration provides the motivation for a critical evaluation of the alternative design schemes.

Alternative design schemes for actions

Alternatives are sought for EN 1990 expression (6.10) (Design Scheme A) because of its inconsistency in maintaining a level of reliability across the range of load ratios. With partial factors selected in order to maintain minimum levels of reliability in the ranges where either permanent or variable load respectively dominates, uneconomic designs are specified in the range of load ratios which occur most generally. This is largely the result of the fact that large deviations are not likely to occur for a number of variables at the same time.

Alternative EN 1990 twin expressions (6.10a) & (6.10b) (Design Scheme B) addresses this problem by applying a variation of the Turkstra (1970) rule (where large deviations for the various actions are in turn combined with expected values for the other actions) to achieve the best consistency in reliability. However, this is attained at the difficulty of requiring two cases to consider in the design process. Furthermore, the transition between the two design expressions occurs within the operational range of practical design.

The special application of expressions (6.10a) & (6.10b) (Design Scheme C) manages to shift the transition somewhat outside the operational range, but at the cost of reducing the consistency and increasing the discontinuity across the transition between the two alternative design equations. The level of reliability is also systematically reduced in the range of situations where permanent loads dominate, such as at the lower parts of heavier or larger structures.

The innovative Design Scheme D provides a solution to these inconsistencies and discontinuity in reliability by formulating the imposed load factor γ_Q in terms of the load ratio χ. The same level of reliability is achieved as with B, but with a single design expression. This scheme requires information on the ratio for the specific load effects related to the specific situation under consideration. Some adjustment will be required in design procedures, such as calculating design load effects at a later stage of the design process. Such requirements can be satisfied through appropriate design software.

Resistance reliability

The integral role of resistance to the total reliability of structures is demonstrated by the effects of the two representative reinforced concrete structural elements of beams and short columns, and the influence of the reinforcement ratio ρ in the respective cases.

Although the general trends in reliability for the two structural elements are the same for the alternative design schemes and the influence of the load ratios, the reliability levels obtained are different. This is the case even though the same resistance modelling factor is applied for the two elements in the simplified analysis. Furthermore, sensitivity analysis shows that the resistance modelling factor plays a major role in the reliability performance in this analysis.

The influence of structural resistance on the reliability is even better illustrated by the different trends shown for changes in steel reinforcement for beams and columns respectively. These differences in trends result from the complex interrelations of the contributions of the respective basic variables in terms of their bias and variability to the overall reliability. Sensitivity analysis applied to the beam model shows that the increase in reliability with increase in steel ratio is the result of an increasing conservative bias due to the rising influence of the large factored design concrete stress on the lever arm. This is a somewhat counter-intuitive result. For the short column the conservative bias of steel is found to be smaller than that of concrete, resulting in a reduced reliability with an increase in the steel ratio.

South African design scheme

The high levels of reliability achieved by Design Scheme E are particularly notable when it is considered that lower β_{target} values are set as the South African basis of design. However, this is an artefact of the conservative bias of characteristic values (as per Eurocode) for load and material factors used in the analysis, which are not (yet) applied in South African practice. Since the use of characteristic values could contribute as much as a value of 1,1 units to the reliability index, as indicated above, Design Scheme E would show significantly lower levels of reliability if 'true' models for the nominal specified load and material variables were used in the analysis.

More significant is the inconsistency of the South African design scheme in the range of load ratios where permanent loads dominate. At the transition between the two expressions (16) & (17) where $\chi = 0,16$ there is a sharp discontinuity in reliability and it is at a *minimum*; when only permanent actions occur at $\chi = 0$ conditions are not all that different, but the reliability is at an overall *maximum*. Within the scope of building and related structures presently provided for in SABS 0160:1989, the importance of design situations where these inconsistencies occur are presently considered to be secondary to the advantage of having a single dominant design expression.

CONCLUDING REMARKS

The newly available European standard EN 1990 provides alternative design procedures and parameters that should be specified in the national annexes of member states. The Eurocode standards recognise the responsibility of regulatory authorities in each member state and guarantee their right to determine values related to regulatory safety matters at national level. One of the duties is the selection of reliability-based limit states design combination schemes for actions and its relations to resistance for the different construction materials, with the appropriate partial safety factors.

The alternative design procedures specified in Eurocode in some cases lead to significantly different reliability levels. More important are the differences in achieving consistency in reliability by the alternative schemes across the range of design situations. The selection of an effective combination scheme is therefore a critical element of setting reliability levels in structural design.

The design scheme presently applied in South African structural design practice can be interpreted as one of the alternatives provided for in EN 1990. This presents an opportunity for increased referencing to Eurocode. Such referencing to Eurocode would compensate for the future loss of British structural design standards as reference, and also contribute to international harmonisation and access to the underlying structural engineering technology.

Although the reliability performance of the present South African design scheme is shown here to be within that of the alternative design schemes provided for in EN 1990, the results provide motivation for reconsideration of this scheme. If the scope of structures provided for in the South African standard is to be extended in such a manner as to increase the importance of design cases where permanent loads dominate, such as to provide for geotechnical effects, the poor performance of the present South African design scheme under these conditions need to be reassessed.

Further calibration studies concerning more complicated structural elements made of various materials are needed. Short-term objectives of these activities should include desired improvement of theoretical models of basic variables, including model uncertainties. Long-term objectives should consist of further harmonisation of rules for the load combinations and subsequent adjustment of partial factors for materials and actions.

REFERENCES

CEN (Comité Européan de Normalisation) 2002. European Standard EN 1990 Eurocode – Basis of structural design.

Gulvanessian, H 2003. Eurocodes – the new design environment. *Eurocodesnews*, 1, July 2003, www.eurocodes.co.uk.

Gulvanessian, H, Calgaro, J-A & Holický, M 2002. *Designers' guide to EN 1990 Eurocode: Basis of structural design*. Thomas Telford.

Holický, M & Marková, J 2000. Verification of load factors for concrete components by reliability and optimization analysis: background documents for implementing Eurocodes. *Progress in Structural Engineering and Materials*, 2(4):502–507.

Holický, M & Marková, J 2003. Reliability of concrete elements designed for alternative load combinations provided in Eurocodes. *Acta Polytechnica*, 2003(1):29–33.

ISO (International Standards Organisation) 1998. General principles on reliability for structures. International Standard ISO 2394:1998.

ISO 2001. Basis for design of structures – assessment of existing structures. ISO 13822:2001.

JCSS (Joint Committee on Structural Safety) 1996. Background documentation, Part 1 of EC 1 Basis of Design.

JCSS 2001. Probabilistic model code. JCSS working materials, http://www.jcss.ethz.ch/.

JCSS 2003. CodeCal Excel sheet developed by JCSS, http://www.jcss.ethz.ch/.

Kemp, A R, Milford, R V & Laurie, J A P 1987. Proposals for a comprehensive limit states formulation for South African structural codes. *The Civil Engineer in South Africa,* 29(9):351–360.

Kemp, A R, Mahachi, J & Milford, R V 1998. Comparisons of international loading codes and options for South Africa. South African National Conference on Loading, Midrand, September 1998.

Milford, R V 1988. Target safety and SABS 0160 load factors. *The Civil Engineer in South Africa*, 30(10):475–481.

Petschacher, M 1997. *VaP Variable Processor, version 1.6*. ETH Zurich.

RCP (Reliability Consulting Programs) Munich 1999. COMREL Version 7.10, www.strurel.de.

Retief, J V, Maritz, G, Ter Haar, T R, Brand, W W & Muhimua-Joao A 2002. Structural concrete reliability models for design code assessment. Concrete for the 21st Century, Joint iDEA, C&CI & CSSA Conference, Midrand, 2002.

SABS (SA Bureau of Standards, Pretoria) 1989. Code of practice: The general procedures and loadings to be adopted in the design of buildings**.** SABS 0160:1989 (amended 1990**).**

SABS 1992. Code of practice: The structural use of concrete, Part 1: Design. SABS 0100:1992.

SAICE 2004. Working Group on SA Loading Code. Working draft of the revision.

SAKO 1999. Joint Committee of NKB and INSTA-B. *Basis of design of structures. proposal for modification of partial safety factors in Eurocodes*. NKB Committee and Work Reports 1999:01 E.

Ter Haar, T R, Retief, J V & Kemp, A R 2001. Calibration of load factors for the South African loading code. In A Zingoni (ed), *Proceedings* of the International Conference on Structural Engineering, Mechanics and Computation, Cape Town. Elsevier.

Ter Haar, T R & Retief, J V 2002. *Resistance factors for structural design codes as derived from loading code calibration*. Concrete for the 21st Century, Joint iDEA, C&CI & CSSA Conference. Midrand, 2002.

Turkstra, C J 1970. Application of Bayesian Decision Theory. Study No 3: structural reliability and codified design. Solid Mechanics Division, University of Waterloo, Ontario, Canada.