ABCB

National Construction Code

Handbook

Structural Reliability Verification Method

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Preface

The Inter-Government Agreement (IGA) that governs the Australian Building Codes Board (ABCB) places a strong emphasis on reducing reliance on regulation, including consideration of non-regulatory alternatives such as non-mandatory handbooks and protocols.

This Handbook is one of a series produced by the ABCB developed in response to comments and concerns expressed by government, industry and the community that relate to the built environment. The topics of Handbooks expand on areas of existing regulation or relate to topics which have, for a variety of reasons, been deemed inappropriate for regulation. They provide non-mandatory advice and guidance.

The Structural Reliability Handbook assists in understanding the Verification Methods **BV1** and **V2.1.1** contained within the National Construction Code (NCC) Volumes One and Two respectively. This Handbook addresses the methodology in developing the Verification Methods in generic terms, and is not a document that sets out a specific process of using the Verification Methods or an alternative structural reliability process. It is expected that this Handbook will be used to guide solutions relevant to specific situations in accordance with the generic principles and criteria contained herein.

The original version of this document was first released to support the introduction of BV1 and V2.1.1 in NCC 2015. Subsequently in 2016, minor editorial changes were made to ensure consistency with NCC 2016.

BV1 and V2.1.1 were revised for NCC 2019, with coinciding changes made within this Handbook.

Contents

REMINDER

This Handbook is not mandatory or regulatory in nature and compliance with it will not necessarily discharge a user's legal obligations. The Handbook should only be read and used subject to, and in conjunction with, the general disclaimer at page i.

The Handbook also needs to be read in conjunction with the relevant legislation of the appropriate State or Territory. It is written in generic terms and it is not intended that the content of the Handbook counteract or conflict with the legislative requirements, any references in legal documents, any handbooks issued by the Administration or any directives by the Appropriate Authority.

1 Background

The NCC is a performance-based code containing all *Performance Requirements* for the construction of buildings. To comply with the NCC, a solution must achieve compliance with the Governing Requirements and the Performance Requirements. The Governing Requirements contain requirements about how the Performance Requirements must be met. A building, plumbing or drainage solution will comply with the NCC if it satisfies the *Performance Requirements*, which are the mandatory requirements of the NCC.

This document was developed to provide guidance to practitioners seeking to demonstrate compliance with the Verification Methods **BV1** and **V2.1.1**. These Verification Methods may be used to demonstrate compliance with Performance Requirements **BP1.1** and **BP1.2** in NCC Volume One and **P2.1.1 (a)**, **(b)** and **(c)** in NCC Volume Two.

1.1 Scope

The Handbook is structured to first provide the reader with an introduction into structural reliability and then provide further discussion on BV1 and V2.1.1.

Further reading on this topic can be found with the references located in the body of this document.

1.2 Design and approval of Performance Solutions

The design and approval processes for solutions using BV1 and V2.1.1 is expected to be similar to that adopted for demonstrating compliance of other NCC Performance Solutions. Since the design approval process for Performance Solutions varies between the responsible State and Territory governments it is likely to also be the case with Performance Solutions adopting BV1 and V2.1.1 and requirements should be checked for the relevant jurisdiction.

Notwithstanding the quantified input and acceptance criteria, other qualitative aspects which are discussed in this document require assessment and analysis throughout the design and approval process. The advice of an appropriately qualified person should be sought to undertake this assessment and analysis where required, and may be aided by the early and significant involvement from regulatory authorities, peer reviewer(s) and / or a technical panel as appropriate to the State or Territory jurisdiction.

1.3 Using this document

General information about complying with the NCC and responsibilities for building and plumbing regulation are provided in Appendix A of this document.

Acronyms and symbols used in this document are provided in Appendix B.

Appendices C, D and E provide details of the derivations of the action models and the reliability indices

References are also provided in Section 6.

Different styles are used in this document. Examples of these styles are provided below:

NCC extracts

Examples

Alerts

Reminders

2 Structural Performance Requirements

Performance Requirements **BP1.1** and **BP1.2** of NCC Volume One contain a comprehensive list of documents that support the Deemed-to-Satisfy (DTS) Provisions, while **P2.1.1(a)**, **(b)** and **(c)** have supporting DTS Provisions through Acceptable Construction Manuals and Acceptable Construction Practices. These manuals and documents cover most aspects of the Limit State Design Method for most construction materials. However, if designers wish to or have to operate outside of the DTS pathway they must develop a Performance Solution. **BV1** and **V2.1.1** are designed to support solutions following these Verification Methods under a Performance Solution path.

In essence, the Verification Methods may be used to derive the load capacity reduction factor ϕ for situations where no appropriate ϕ is given in NCC referenced documents.

Alert:

As BV1 and V2.1.1 are identical, for the remainder of this document, they will be referred to as the Verification Method for simplicity.

The NCC provides four clauses regarding structural Performance Requirements. Only the first two clauses, **BP1.1** and **BP1.2** of NCC Volume One and sub-clauses **P2.1.1(a)**, **(b)** and **(c)**, of NCC Volume Two address general structural performance. The concept of a reliability index can be used to quantify the structural performance of these clauses.

In order to meet these Performance Requirements through the Verification Method, any new and/or innovative structural component or connection is required to demonstrate that it achieves or exceeds the target reliability indices using the method outlined. This Handbook is mainly concerned with illustrating how this can be achieved.

BP1.1 and **P2.1.1 (a)** and **(b)** consist of two listings:

(i) required performance attributes and
(ii) factors to be considered, namely the actions to which a building 'may reasonably be subjected'.

The list of performance attributes covers the serviceability performance, strength performance and robustness. The Verification Method is applicable mainly to strength performance.

BP1.2 and **P2.1.1(c)** cover general principles in formulating structural resistance that will be described in Section 3 of this Handbook.

The Verification Method is one way, but not the only way, to demonstrate compliance with the NCC Performance Requirements. There are other structural Performance Requirements in the NCC that are not covered by this Verification Method.

3 Methodology for the structural reliability index Verification Method

3.1 General

The Verification Method uses the reliability index as a benchmark for strength verification. Structural reliability index is an overall concept covering structural actions, response and resistance, workmanship and quality control, all of which are mutually dependent.

It uses the reliability index as a means for benchmarking the strength of structures subjected to known or foreseeable types of actions such as permanent, imposed, wind, snow and earthquake. As such, it involves mainly structural actions and resistance. The levels of workmanship and quality control are assumed to be maintained in accordance with current standards and practice and appropriately accounted for in the resistance model.

The calculation of the reliability indices used in the Verification Method follows the general reliability principles of ISO 2394 'General principles on reliability for structures' but with a simplifying assumption of using lognormal distributions to represent both actions and resistance. While the computational procedures are similar, it is emphasised that the Verification Method procedure only provides a benchmark that may or may not have any relation to the real probability of failure due to the number of assumptions that have to be made. In this context, it is to be noted that the values of the reliability indices thus obtained are NOT independent of the calculation process.

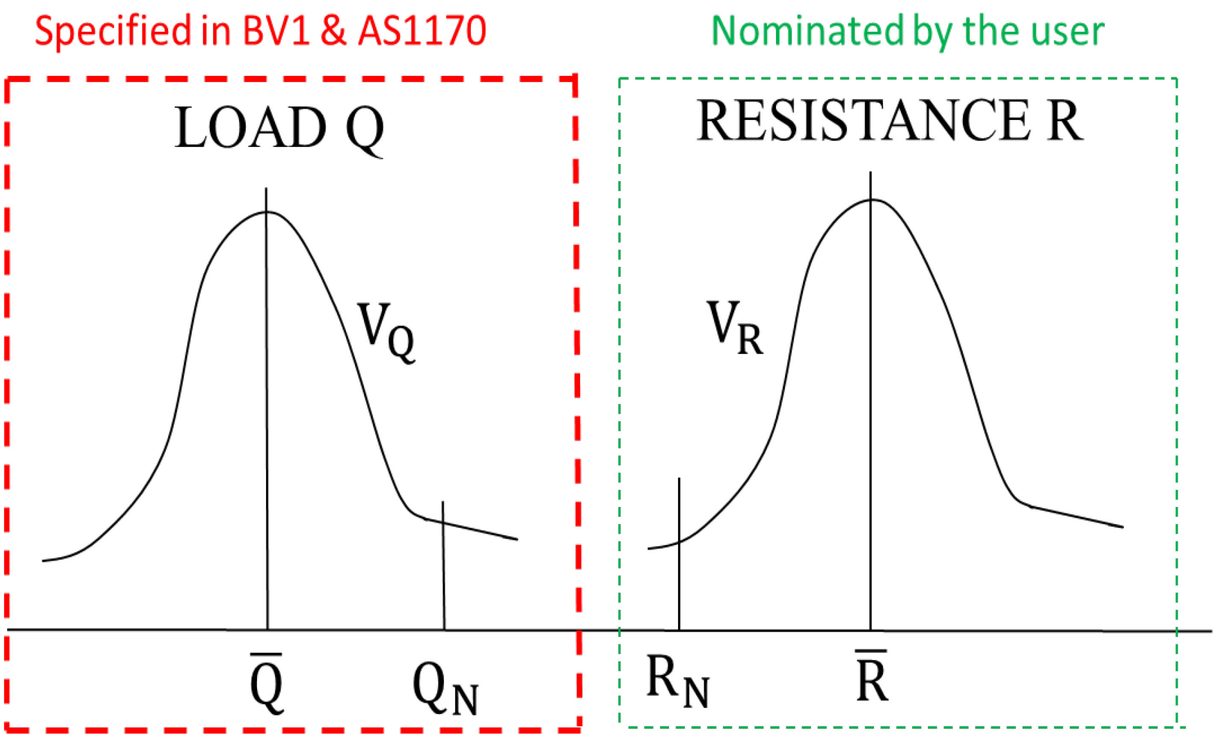
Structural components and connections, for which there is no corresponding NCC DTS Provisions or referenced documents, can choose to meet the target reliability indices in order to demonstrate compliance (i.e. to satisfy the relevant Performance Requirements for strength). Options are provided either to meet pre-set target values or to establish one's own target values based on the principle of comparison to DTS Provisions.

3.2 Probabilistic models

The reliability index takes actions and resistances, and represents these as random variables in probabilistic models. The models used for the Verification Method are shown in Figure 3.1. The action model, depicted on the left of the figure represents the probabilistic action, where the peak of the curve highlights the most frequent value, the spread, the variation and a nominal design action (such as permanent, imposed or environmental actions). Similarly, the resistance model represented by the curve on the right of the figure, shows the peak resistance, spread of variation and nominal resistance for a design material.

The distance between these two curves is the performance of the component under question. The Verification Method provides the action models in line with the appropriate parts of the Australian Standard and New Zealand Standard AS/NZS 1170 series and described in Appendix C of this Handbook.

Figure 3.1 Action and resistance models



Design requirement $\gamma\ Q_N \leq \phi\ R_N$

The basic information required for the action and resistance models are:

Table 3.1 Symbols from Figure 3.1

Action model:	Resistance model:
Q_m = Mean action	R_m = Mean resistance
V_Q = Coefficient of variation with respect to action	V_R = Coefficient of variation with respect to resistance
Q_n = Nominal design value of the action	R_n = Nominal design value of the resistance

3.3 Verification Method clauses

Under the design procedures for the Verification Method, a single capacity reduction factor, ϕ is chosen to be used with all design load combinations and load factors as specified in AS/NZS 1170.0. The simplest way to achieve this is to use the average value of ϕ's for permanent, imposed and wind actions computed for separate target reliability indices. The clauses for the Verification Method quoted below are self-explanatory:

BV1 Structural reliability

(a) This *Verification Method* is applicable to components with a resistance coefficient of variation of at least 10% and not more than 40%. For components with calculated value less than 10%, then a minimum value on 10% must be used.

(b) Compliance with BP1.1 and BP1.2 is verified for the design of a structural component for strength when–

(i) the capacity reduction factor ϕ satisfies–

$\phi \leq$ Average $(\phi_G, \phi_Q, \phi_W, \ldots)$,

where–

$\phi_G, \phi_Q, \phi_W, \ldots$ are capacity reduction factors for all relevant actions and must contain at least permanent (G), imposed (Q) and wind (W) actions; and

(ii) the capacity reduction factors ϕ_G, ϕ_Q, ϕ_W,… are calculated for target reliability indices for permanent action β_{TG} , for imposed action β_{TQ} , for wind action β_{TW}, …in accordance with Equation (1)–

Equation 1

$$\beta = \frac{\ln\left[\left(\frac{\bar{R}}{\bar{S}}\right)\sqrt{\frac{C_S}{C_R}}\right]}{\sqrt{\ln(C_R.C_S)}}$$

where–

$$\left(\frac{\bar{R}}{\bar{S}}\right) = \frac{\left(\frac{\gamma}{\phi}\right)}{\left(\frac{\bar{S}}{S_N}\right)}\left(\frac{\bar{R}}{R_N}\right);$$

$$C_R = 1 + V_R^2;$$

$$C_S = 1 + V_S^2$$

where–

$\frac{\bar{R}}{R_N}$ = ratio of mean resistance to nominal; and

$\frac{\bar{S}}{S_N}$ = ratio of mean to nominal; and

C_s = correction factor for action; and

C_R = correction factor for resistance

V_R = coefficient of variation of the resistance; and

V_S = coefficient of variation of the appropriate action as given in Table BV1.1; and

γ = appropriate load factor for the action as given in AS/NZS1170.0; and

ϕ = capacity factor for the appropriate action; and

Table BV1.1 – Annual action models

Action	Mean/ Nominal	Coefficient of variation of the action
Permanent Action (γ_G = 1.35)	$(\bar{G}/G_N)$ = 1.00	V_G = 0.10
Imposed Action (γ_Q = 1.50)	$(\bar{Q}/Q_N)$ = 0.50	V_Q = 0.43

Action	Mean/ Nominal	Coefficient of variation of the action
Wind Action (γ_W = 1.00) (Non-cyclonic)	$(\bar{W}/W)$ =0.33	V_W =0.49
Wind Action (γ_W = 1.00)(Cyclonic)	$(\bar{W}/W_N)$ = 0.16	V_W = 0.71
Snow Action (γ_S = 1.00)	$(\bar{S}/S_N)$ = 0.29	V_w = 0.57
Earthquake Action (γ_E = 1.00)	$(\bar{E}/E_N)$ = 0.05	V_E = 1.98

(iii) the annual target reliability indices β_{TG}, β_{TQ}, β_{TW},… are established as follows:

(A) For situations where it is appropriate to compare with an equivalent Deemed-to-Satisfy product, a resistance model must be established for the equivalent Deemed-to-Satisfy product and β_{TG}, β_{TQ}, β_{TW} must be calculated for the equivalent Deemed-to-Satisfy product in accordance with Equation (1). The target reliability indices β_{TG}, β_{TQ}, β_{TW},…thus established , shall not be less than those given in Table BV1.2 minus 0.5.

(B) For situations where it is not appropriate to compare with an equivalent Deemed-to-Satisfy product, the target reliability index β shall be as given in Table BV1.2.

Table BV1.2 – Annual Target reliability indices

Action	Target reliability index β
Permanent action	4.3
Imposed action	4.0
Wind, snow and earthquake action	3.7

Application of Table BV1.2:

1. Table BV1.2 is applicable for components that exhibit brittle failure similar to concrete as specified in AS 3600.

2. For components with creep characteristics similar to timber as specified in AS1720.1, the target reliability index for permanent action must be increased to 5.0.

3. The above target reliability indices are based on materials or systems that exhibit creep or brittle failure characteristics similar to timber and concrete. Table BV1.2 may also be applicable to materials or systems that exhibit creep or brittle failure differently to steel, timber or concrete provided that the creep or brittle nature of the material or system are properly accounted for in the design model.

4. The above target reliability indices are also applicable for materials or systems that exhibit ductile failure characteristics.

(c) The resistance model for the component must be established by taking into account variability due to material properties, fabrication and construction process and structural modelling.

4 Construction of the resistance model

As seen in 3.3, the action models and the calculation procedures have been specified in the Verification Method, the user however needs to provide a model of resistance for the component under consideration before the determination of the appropriate ϕ factor can be carried out under 3.3.

The purpose of creating a resistance model is to account for all sources of uncertainties in the determining the resistance of a structural component or connection. The resistance R, a random variable, is related to the standard specified resistance R_N, a deterministic value, through the general equation below. In which K_m, K_f, K_s … are factors that contribute to the uncertainties in the assessment of the resistance and are random values assumed to be statistically independent.

$$R = K_m . K_f . K_s \ldots R_n$$

The sources of uncertainties must include, but are not limited to the following:

- variability in the mechanical properties of the materials;
- variation in dimensions as the result of fabrication or construction processes; and
- uncertainties in the structural modelling of the component.

Another source of uncertainty is the rounding due to a structural component's availability in discrete sizes. This source of variability is ignored as it is generally conservative to do so.

If all the above variables are assumed to be of lognormal distribution and statistically independent, then the mean value and the coefficient of variation of R can be established as follows (see Fig 4.1):

$$\text{Mean }(R) = \text{Mean }(K_m). \text{ Mean }(K_f). \text{ Mean }(K_s)\ldots R_n$$
$$(V_R)^2 = (V_{Km})^2 + (V_{Kf})^2 + (V_{Ks})^2 + \ldots$$

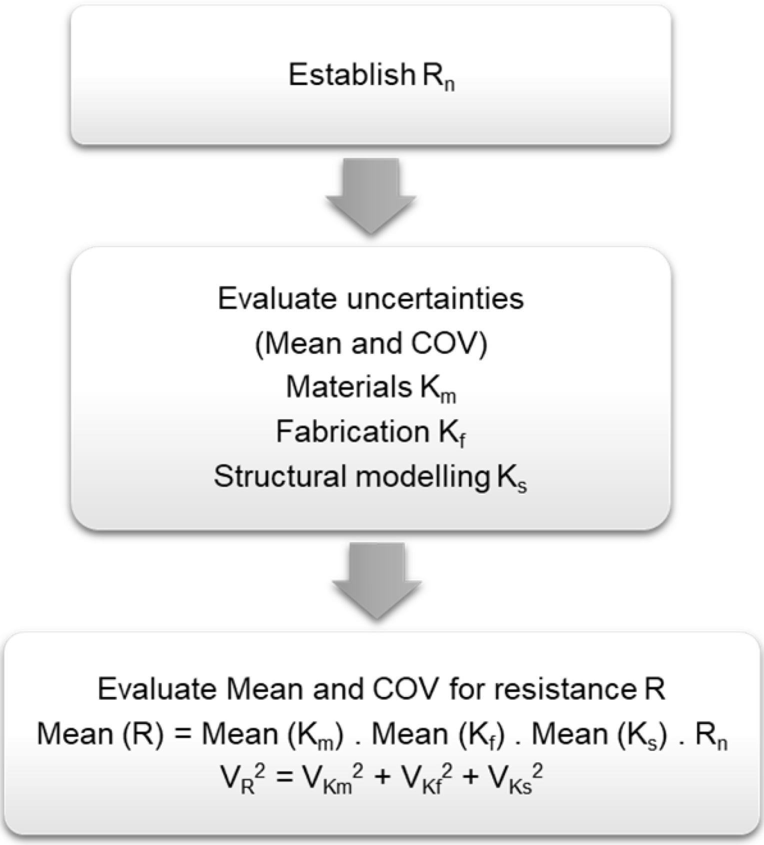
The value of R_N is usually established by identifying the major parameters that affect the behaviour of the component and constructing appropriate structural models to account for their effects. R_N is usually formulated using five percentile characteristic material properties in accordance with **BP1.2** and **P2.1.1(c)** of the NCC (Volumes One and Two, respectively). Examples can be found in structural design standards for steel, concrete and timber. However, this is no longer necessary for the use of the Verification Method. The value to be used in design can be set arbitrarily as the derived ϕ factors will make all the necessary compensation.

The value of K_m to account for variability of the relevant mechanical properties is usually obtained from test data used for quality control of the material manufacturing process.

The value of K_f to account for variability of the manufacturing/construction process is obtained from the established allowable tolerance and measurement of the dimensions of the component.

The value of K_s to account for variability in structural modelling is obtained from the test research data used in the construction of the structural model.

Figure 4.1 Establishing the resistance



Further information on the construction of the resistance model can be found in the references quoted below. In these references, data on resistance can be used in conjunction with BV1. The calculation of the reliability indices, however, were established using different procedures and assumptions and therefore cannot be meaningfully compared with the values using BV1 approach:

1. For estimates of steel, concrete, timber and masonry: ‘Development of a probability based load criterion for American National Standard A58’ by B. Ellingwood, T.V. Galambos, J. G. MacGregor and C.A. Cornell. National Bureau of Standards, US Department of Commerce, June 1980, (nvlpubs.nist.gov).
2. For Australian data on concrete: ‘Calibration of Australian Standard AS 3600 Concrete Structures: Part 1 Statistical Analysis of material properties and model error, Australian Journal of Structural Engineering, 17:4, p 242-253’ by S.J. Foster et.al (2016).
3. For US data on cold-formed steel: ‘North American Cold-Formed Steel Specification 2012 Edition Chapter F Table F1’.

5 Worked examples

The following examples demonstrate various ways to use the Verification Method. These examples are followed by a brief discussion regarding the assumptions made in order to achieve the results. It should be noted that these examples only highlight the key process steps. In a real life situation, the outcomes should be modified by appropriate experience, expert judgement or peer review in analysing a new product or system.

5.1 Worked example 1: Capacity reduction factor ϕ for a new timber product

Problem:

To determine the design bending stress and appropriate ϕ factor for a new timber product to be placed on the Australian market. Other design requirements are as in AS 1720 and load factors are as in AS/NZS 1170.0.

Product Data:

1155 full size samples were measured and tested to a 5 minute duration test. The following statistical parameters were estimated from the test data:

Mean bending stress, f_u = 37.7 MPa

Coefficient of variation, V_{fu} = 0.40

Dimensional variations:

Width b (Mean/Nominal) = 1.0, V_b = 0.02

Depth d (Mean/Nominal) =1.0, V_d = 0.02

Solution:

Establish the resistance model: The moment capacity of the section is the key parameter in assessing bending resistance following the model used in AS 1720.

$R = k_t \,.\, Z \,.\, f_u$

where:

k_t = factor to account for load duration.

Z = section modulus

f_u = failure stress

There are a few other factors that affect bending strength such as moisture, stability etc. as specified in AS 1720. For this example, we will seek to establish basic design stress thus only load duration factor is taken into account.

The nominal design value R_n is therefore:

$R_n = k_{tn} \,.\, Z_n \,.\, f_{un}$

Therefore;

$(R/R_n) = (k_t/k_{tn}) \,.\, (Z/Z_n) \,.\, (f_u/f_{un})$

$$V_R^2 = V_{kt}^2 + V_Z^2 + V_{fu}^2$$

Step 1 Evaluation of (k_t/k_{tn})

It shall be assumed that the provisions of AS 1720 on load duration are correct.

Mean (k_t/k_{tn}) = 1.0/LDF;

V_{kt} = 0.1

where:

LDF = load duration factor as given in AS 1720 as shown in the table below:

Table 5.1 Load duration factors as given in AS 1720

Action	Load duration factor
Permanent action	0.57
Imposed action	0.80
Wind action	1.00

Step 2 Evaluation of (Z/Z_N)

Z is a function of b and d^2, as a first approximation:

Mean (Z/Z_n) = Mean (b/b_n) . Mean $(d/d_n)^2$ = 1.0

$$V_Z^2 = V_b^2 + 4\,V_d^2$$

V_Z = 0.04

Step 3 Evaluation of ($f_u/f_{u,n}$)

The five percentile value is used as the design stress $f_{u,n}$ as required under BP1.2 and P2.1.1(c).

Two methods are available for f_{un} assessment using assumed distribution (with COV = 0.40):

Lognormal: f_{un}= 0.49 x 37.7 = 18.5 MPa; or

Weibull: f_{un}= 0.37 x 37.7 = 13.9 MPa.

Using direct ranking from data, f_{un} = 16.6 MPa.

Thus the 5-percentile ranges from 13.9 to 18.5 MPa. Select $f_{u,n}$ = 17 MPa. It does not matter what the selected value is, as higher f_{un} will require a lower ϕ and vice versa.

Mean ($f_u/f_{u,n}$) = 37.7/17 = 2.21

V_{fu} = 0.40

Step 4 Combining the steps

(R/R_n) = (k_t/kt_n) . (Z/Z_n) . ($f_u/f_{u,n}$) = 2.21/LDF

$$V_R = \sqrt{(V_{kt}^2 + V_Z^2 + V_{fu}^2)} = 0.42$$

Step 5 Compute ϕ

Table 5.2 presents the outcomes of the computation for NCC given targets.

Table 5.2 Outcomes for timber beams with targets from Table BV1.2			
Action	**Target β**	**ϕ**	**Load duration**
Permanent	5.0	0.60	0.57
Imposed action	4.0	0.75	0.8
Non-cyclonic wind action	3.7	0.57	1.0
Cyclonic wind action	3.7	0.65	1.0
	Average ϕ	**0.65**	

5.1.1 Discussion

It is not important for the verifier to know the derivation of the proposed design stress and capacity reduction factor, ϕ. Looking at the data, the proposed design stress is approximately the five percentile value which is a requirement under **BP2** and **P2.1.1(c)**. In this example, we did not question the validity of $M = Z \cdot f_u$ as we ignored the structural modelling factor in this case. For more complex situations, uncertainty in structural analysis could be a significant factor. Uncertainty in structural modelling is high when the structural action is not well understood and empirical factors are introduced to reconcile the structural model with experimental data such as shear strength, or anchors. In this example, the variability in the material strength is the controlling factor, which is normal for materials like timber.

From Table 5.2, it is seen that the adoption of a basic bending design stress of 17 MPa and the average ϕ value of 0.65 is achieved for the given target reliability indices. This is more conservative than actual practice as it was intended to be.

If we are to establish our own target reliability indices by matching it with typical timber beams that have the following properties according to AS 1720 (Leicester, 1986):

$R/R_n = 1.85/LDF$
$V_R = 0.40$
$\phi = 0.85$

The target reliability indices are then-

Table 5.3 Target reliability indices

Load type	Permanent	Imposed	Non-cyclonic wind	Cyclonic wind
Reliability index By calibration	4.0	3.6	2.9	3.2
Target index according to the Verification Method	4.5	3.6	3.2	3.2

Table 5.4 Reliability index calculation outcomes for timber beams – with targets set from a DTS Solution with the Verification Method minimum rule

Action	Target β	ϕ	Load duration
Permanent action	4.5	0.75	0.57
Imposed action	3.6	0.92	0.8
Non-cyclonic wind action	3.2	0.79	1.0
Cyclonic wind action	3.2	0.97	1.0
	Average ϕ	**0.85**	

From Table 5.3, it is seen that the adoption of a basic bending design stress of 17 MPa, the average ϕ value of 0.85 is achieved for the given target reliability indices. This is very close to actual practice.

5.2 Worked example 2: Capacity reduction factor for a concrete product

Problem:

To determine the design value and associated capacity reduction factor, ϕ, for a concrete panel used to resist wind load in bending. The bending capacity of the panel was established with a series of prototype testing.

The data statistics are given in Table 5.5.

Table 5.5 Test data statistics

No of samples	Average	COV	Minimum
20	1.50	0.26	0.89

Solution:

It is assumed the data is a fair representation of the properties of the products and the COV of the samples is taken as COV of the products. The design value is taken as either the mean value (1.50 kN-m) or the minimum value (0.89 kN-m) from the test.

The results of the computation are given in Table 5.6.

Table 5.6 ϕ values for different nominal design value R_n

Action type	Target β	Φ value R_n =1.5 kN-m	Φ value R_n =0.89 kN-m
Non-cyclonic wind action	3.7	0.35	0.60
Cyclonic wind action	3.7	0.38	0.65
	Average φ	**0.37**	**0.63**
	Φ R_n	**0.56**	**0.56**

5.1.2 Discussion

This example is designed to show that the adoption of different nominal design values makes no difference to the final outcomes in terms of (ϕ R_n).

5.3 Worked example 3: Capacity reduction factor for a metal anchor in concrete

This worked example intends to demonstrate the use of the Verification Method in converting overseas design procedures for products for use in Australia. The design of metal anchors in concrete is well developed by the European Organisation for Technical Assessment (EOTA) for use with EuroCode. To adapt the same procedure for use in Australia, one option is to use the Verification Method to determine the required capacity reduction factor ϕ to be used in conjunction with Australian loading specifications.

The strength for a concrete cone failure as shown in Figure 5.1 is given by Eligehausen et al (2006) as follows:

$$N_{Rk,c} = k\sqrt{f'_c}h_{ef}{}^{1.5}$$

where:

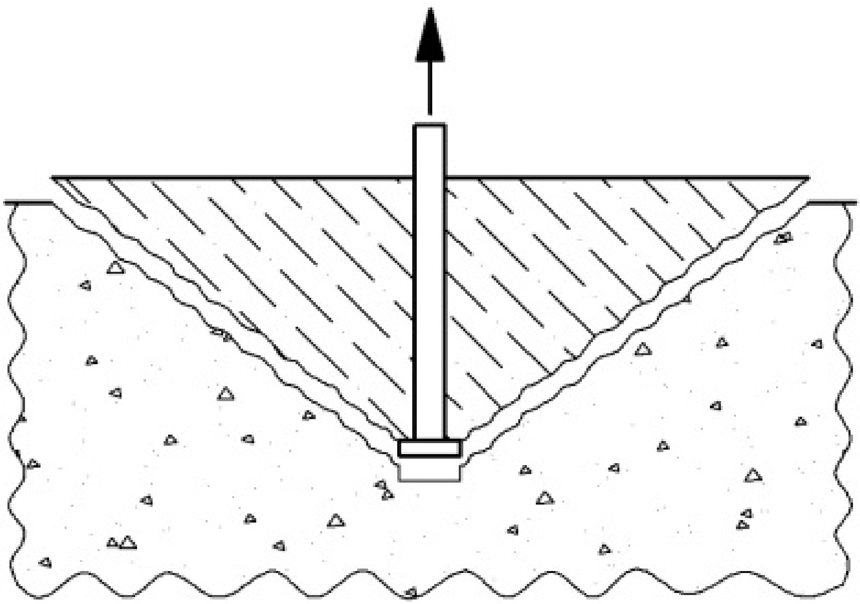
$N_{Rk,c}$ = characteristic strength of fastener to concrete cone failure

k = characteristic strength of fastener to concrete cone failure

f'_c = characteristic compressive strength of concrete

h_{ef} = effective embedment depth of fastener

Figure 5.1 Metal anchor in concrete failure



The above parameters are random variables as there are variabilities associated with the fastener's geometry, dimensional tolerance of the manufacturing process, the installation process and the theory associated with the above equation. The corresponding nominal strength of an anchor, i.e. the analytical expression for strength to be used in design before the application of the capacity factor, may be represented as:

$$N_n = k_n\sqrt{f_n}{h_n}^{1.5}$$

where:

N_n = nominal value of $N_{Rk,c}$

k_n = nominal value of parameter k

h_n = nominal value of the embedment depth, h_{ef}

f_n = nominal value of the compressive strength of concrete, f'_c and

'nominal value' means the value to be used in design calculation

The mean to nominal strength is therefore:

$$\left(\frac{\bar{N}}{N_n}\right) = \left(\frac{\bar{k}}{k_n}\right)\left(\frac{\bar{f}}{f_n}\right)^{0.5}\left(\frac{\bar{h}}{h_n}\right)^{1.5}$$

The corresponding expression for the coefficient of variation of strength (V_N) is approximated as follows:

$${V_N}^2 = {V_k}^2 + 0.25{V_{conc}}^2 + 2.25{V_{hef}}^2$$

where:

V_k = Coefficient of variation of parameter k

V_{conc} = Coefficient of variation of concrete strength

V_{hef} = Coefficient of variation of embedment depth

Estimates of the evaluation of the resistance model variables are given in Table 5.7.

Table 5.7 Estimates of the evaluation of the resistance model

$\bar{k}/k_n$	$\bar{f}/f_n$	$\bar{h}/h_n$	R/R_n	Vk	Vconc	Vhef	VN
1.0	1.20	1.0	1.10	0.10	0.11	0.05	0.14

Calculations of the capacity factors to Table BV1.2 targets are given in Table 5.8.

Table 5.8 Evaluation of the ϕ factor – Table BV1.2 targets

Action type	Target reliability index	Capacity reduction factor
Permanent action	4.3	0.71
Imposed action	4.0	0.54

Action type	Target reliability index	Capacity reduction factor
Non-cyclonic wind action	3.7	0.47
Cyclonic wind action	3.7	0.47
	Average ϕ = 0.55	

Calculations of the capacity factors to Table BV1.2 minimum targets are given in Table 5.9.

Table 5.9 Evaluation of the ϕ factor Table BV1.2 minimum targets

Action type	Target reliability index	Capacity reduction factor
Permanent action	3.8	0.77
Imposed action	3.5	0.67
Non-cyclonic wind action	3.2	0.60
Cyclonic wind action	3.2	0.66
	Average ϕ = 0.67	

The equivalent ϕ factor adopted in EOTA is 0.67. This indicates that the BV1 Verification Method procedure is conservative as intended.

5.4 Worked example 4: Capacity reduction factor ϕ for reinforced concrete (RC) beam in flexure

Problem:

To determine the flexural capacity of ductile RC beam and appropriate φ factor. Other design requirements are as in AS 3600. The beam size is taken as d = 500 mm and b = 250 mm and the reinforcement ratio is taken as ρ = 0.6%, (where ρ = A_{st}/bd, hence, A_{st} = 750 mm^2). The specified concrete strength (F_{c}') is 30 MPa and reinforcement strength (F_{sy}) is 350 MPa.

Mean $(f_c'/F_c') = 1.03$; V_{fc} = 0.18

Mean $(f_{sy}/F_{sy}) = 1.18$; V_{fsy} = 0.12

Dimensional variations:

Width b (Mean/Nominal) = 1.01, V_b = 0.04

Depth d (Mean/Nominal) =0.99, V_d = 0.04

Depth A_{st} (Mean/Nominal) =1.0, V_{st} = 0.015

Solution:

Establish the Resistance Mode

According to AS 3600, the resistance model for ductile members in flexure is,

$$R = A_{st}f_{sy}\left[d - \frac{A_{st}f_{sy}}{2\alpha_2 f_{cm} b}\right]$$

Where:

A_{st} is the area of tensile steel,

f_{sy} the measured yield strength of the steel,

f_{cm} is the measured mean concrete strength,

b is the width of the beam and

α_2 is the rectangular stress block height parameter and is calculated from:

$$\alpha_2 = 1.0 - 0.003 f_{cm} \quad 0.67 \leq \alpha_2 \leq 0.85$$

Mean strength:

$$R_m = A_{st,n}f_{sy,n}\left[d_n - \frac{A_{st,n}f_{sy,n}}{2\alpha_2 f_{cm,n} b}\right] = 750 * 413\left[495 - \frac{750*413}{2*0.85*30.9*252.5}\right] = 146\ kN.m$$

The coefficient of variation of strength (V_R) can be approximately estimated as:

$$V_R = \sqrt{2V_{Ast}^2 + 2V_{fsy}^2 + 2V_d^2 + V_{fcm}^2 + V_b^2}$$

$$= \sqrt{2 * 0.015^2 + 2 * 0.12^2 + 2 * 0.04^2 + 0.18^2 + 0.04^2}$$

$$= 0.257$$

Evaluation of (R/R_n)

The five percentile value is used as the design stress as required under BP1.2 and P2.1.1(c).

Lognormal distribution for resistance with COV=0.26.
Nominal design value as 5 percentile value R_N = 92.8 kN.m
Therefore, Mean (R/Rn) = 146/92.8 = 1.57

Table 5.10 presents the outcomes of the computation for NCC given targets.

Table 5.10 Outcomes for RC beams with targets from Table BV1.2

Action type	Target β	Calculated ϕ
Permanent action	4.3	0.64
Imposed action	4.0	0.63
Non-cyclonic wind action	3.7	0.55
Cyclonic wind action	3.7	0.65
	Average ϕ	**0.62**

From Table 5.4.1, it is seen that the adoption of a nominal design bending strength of 92.8 kN.m, the average ϕ value of 0.62 is achieved for the given target reliability indices. This is conservative comparing with current practice where ϕ = 0.8 is adopted.

Alternatively, if the minimum specified β 's are used (i.e. target - 0.5) then the results are as given in Table 5.4.2. The average ϕ is then 0.79.

Table 5.11 Outcomes for RC beams with minimum targets

Action type	Target β	Calculated ϕ
Permanent action	3.8	0.73
Imposed action	3.5	0.81
Non-cyclonic wind action	3.2	0.74
Cyclonic wind action	3.2	0.87
	Average ϕ	**0.79**

The last two examples demonstrate that the target reliability indices chosen in the Verification Method are conservative. The minimum acceptable targets are much closer to the actual practice.

6 References

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APPENDICES

Appendix A Compliance with the NCC

A.1 Responsibilities for regulation of building and plumbing in Australia

Under the Australian Constitution, State and Territory governments are responsible for regulation of building, plumbing and development / planning in their respective State or Territory.

The NCC is an initiative of the Council of Australian Governments and is produced and maintained by the ABCB on behalf of the Australian Government and each State and Territory government. The NCC provides a uniform set of technical provisions for the design and construction of buildings and other structures, and plumbing and drainage systems throughout Australia. It allows for variations in climate and geological or geographic conditions.

The NCC is given legal effect by building and plumbing regulatory legislation in each State and Territory. This legislation consists of an Act of Parliament and subordinate legislation (e.g. Building Regulations) which empowers the regulation of certain aspects of buildings and structures, and contains the administrative provisions necessary to give effect to the legislation.

Each State's and Territory's legislation adopts the NCC subject to the variation or deletion of some of its provisions, or the addition of extra provisions. These variations, deletions and additions are generally signposted within the relevant section of the NCC, and located within appendices to the NCC. Notwithstanding this, any provision of the NCC may be overridden by, or subject to, State or Territory legislation. The NCC must therefore be read in conjunction with that legislation.

A.2 Demonstrating compliance with the NCC

Compliance with the NCC is achieved by complying with the Governing Requirements of the NCC and relevant Performance Requirements.

The Governing Requirements are a set of governing rules outlining how the NCC must be used and the process that must be followed.

The Performance Requirements prescribe the minimum necessary requirements for buildings, building elements, and plumbing and drainage systems. They must be met to demonstrate compliance with the NCC.

Three options are available to demonstrate compliance with the Performance Requirements:

- a Performance Solution,
- a DTS Solution, or
- a combination of a Performance Solution and a DTS Solution.

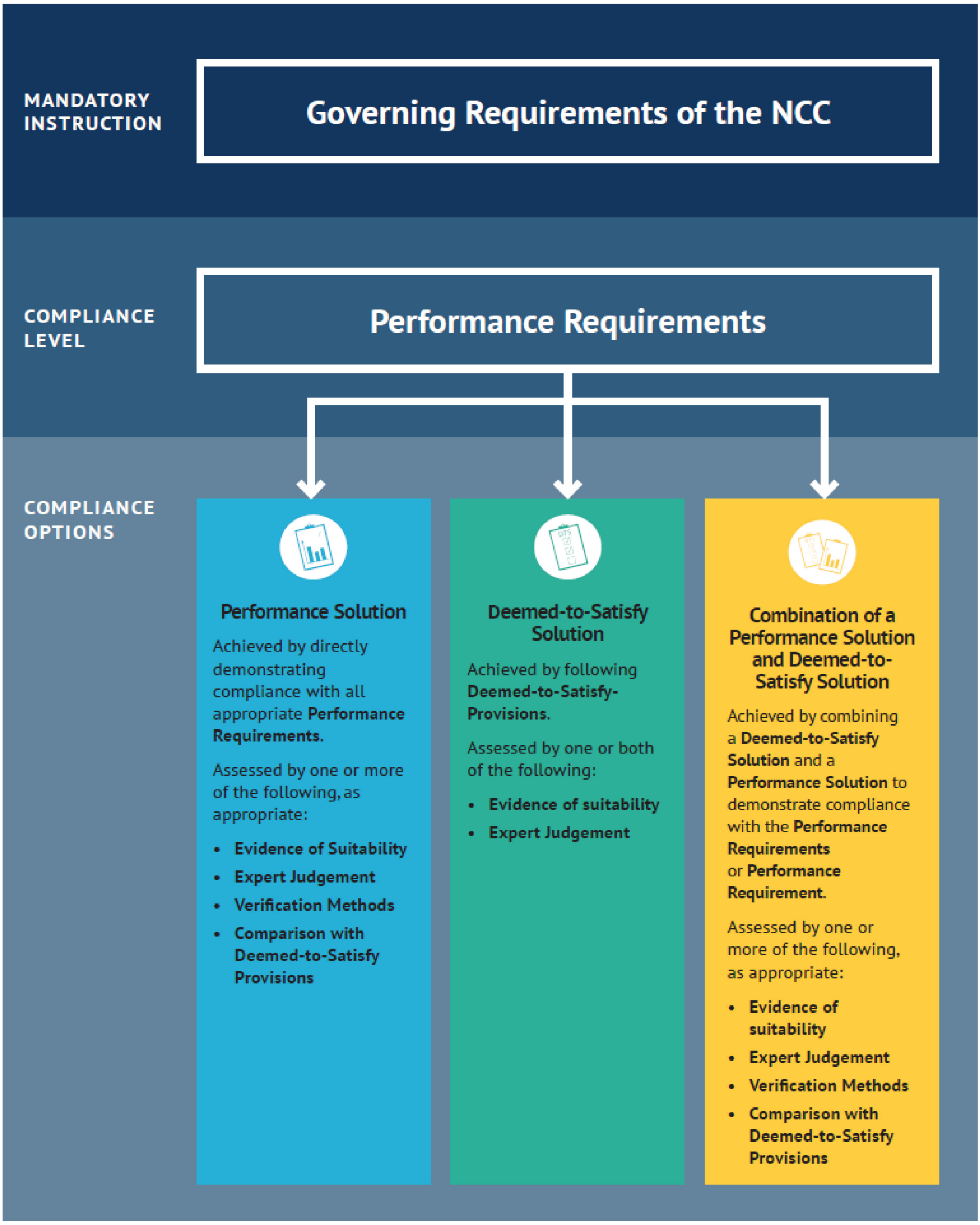
All compliance options must be assessed using one or a combination of the following Assessment Methods, as appropriate:

- Evidence of Suitability
- Expert Judgement
- Verification Methods
- Comparison with DTS Provisions.

A figure showing hierarchy of the NCC and its compliance options is provided in Figure A.1. It should be read in conjunction with the NCC.

To access the NCC or for further general information regarding demonstrating compliance with the NCC visit the ABCB website (abcb.gov.au).

Figure A.1 Demonstrating compliance with the NCC



Appendix B Acronyms and symbols

The following table, Table B.1 contains acronyms used in this document.

Table B.1 Acronyms

Acronym	Meaning
ABCB	Australian Building Codes Board
AEP	Annual probabilities of exceedance
DTS	Deemed-to-Satisfy
IGA	Inter-government agreement
NCC	National Construction Code

The units and notation used in this Handbook are designed specifically for use with the Verification Methods. The symbols used are outlined in Table B.2.

Table B.2 Symbols

Symbol	Meaning
C	Aerodynamic shape factor to convert wind speed to wind pressure
C_D	Factor to cover the effects of the ground condition
C_E	Factor to cover the effects of exposure for snow action
C_F	Factor to cover the geometrical effects such as the roof shape for snow action
COV	Co-efficient of variation
C_Q	Correction factor for action ($= 1+V_Q^2$)
C_R	Correction factor for resistance ($= 1+V_R^2$)
C_W	Factor to cover the effects of mass distribution of a building for an earthquake action
E	Earthquake action effect
g	Permanent action
G	Permanent action effect
H_x	Factor to convert action to action effect for 'x' action
K_i	Uncertainty factors for resistance

Symbol	Meaning
M	Factor to cover all multipliers for wind speed: direction, exposure, shielding and topographic
Q	Imposed action effect
q	Imposed action
Q_m	Mean action
Q_n	Nominal design value of the action
R_m	Mean resistance
R_n	Nominal design value of the resistance
s_G	Ground snow load
V	Basic wind speed
V_Q	Coefficient of variation with respect to action
V_R	Coefficient of variation with respect to resistance
W	Wind action effect
β	Beta, with respect to structural reliability, structural reliability index
γ_i	Gamma i, action factor for action i as defined in AS/NZS 1170.0
ϕ	Phi, capacity factor as used in Limit State Design to account for uncertainties in resistance

Appendix C Construction of the action models

This subsection describes the derivation of the action models which, in general, include two components:

(a) a factor to convert action into action effect; and
(b) the intensity of the action.

The former is largely based on judgment and the latter on statistical data. Lognormal distribution has been assumed for all variables for ease of combination. It is important to note that the models (except for permanent action) were established based on a reference time period of one year. Thus the resulting reliability indices are also for a one-year time reference.

The lognormal probability density function for random variable *X* is expressed as follows:

$$\mathbf{f_x(x) = \frac{1}{\sqrt{2\pi}\zeta_x x} e^{\frac{1}{2}\left(\frac{\ln x - n_x}{\zeta_x}\right)^2}}$$ **Equation C.1**

Where η_x and ζ_x are the scale and shape parameters, respectively.

The Australian Standards typically specify an action effect as the action multiplied by a number of modification factors (e.g. AS/NZS 1170.2:2011; AS/NZS 1170.4:2007). When an action effect Q, is expressed as multiplication of modification factors$k_{i,}i = 1, K, m,$, and the *n*-th power of action q, q^n, i.e.:

$$\mathbf{Q = k_1 k_2 \dots k_m q^n}$$ **Equation C.2**

in which $k_i's$ and q are all lognormally distributed.

The mean μ_Q of Q is determined to be:

$$\boldsymbol{\mu_Q = \mu_{k_1}\mu_{k_2} \dots \mu_{k_m}\mu_q^n\left(1 + V_q{}^2\right)^{\frac{n(n-1}{2}}}$$ **Equation C.3**

where μ_Q, $\mu'_{k_i}s$, and μ_q denote the means of Q, k'_is and q, respectively, and V_q is the coefficient of variation of q. The squared shape parameter $\zeta_Q{}^2$ of Q becomes:

$$\boldsymbol{\zeta_Q{}^2 = \zeta_{k_1}{}^2 + \zeta_{k_2}{}^2 + \dots + \zeta_{k_m}{}^2 + n^2\zeta_q{}^2 = \ln\left(1 + V_{k_1}{}^2\right) + \ln\left(1 + V_{k_2}{}^2\right) + \dots + \ln\left(1 + V_{k_m}{}^2\right) + n^2\ln\left(1 + V_q{}^2\right)}$$ **Equation C.4**

The coefficient of variation V_Q of Q is then:

$$\mathrm{V_Q} = \sqrt{\exp\left(\zeta_Q{}^2\right) - 1} = \sqrt{\left(1 + \mathrm{V_{k_1}}{}^2\right)\left(1 + \mathrm{V_{k_2}}{}^2\right)\dots\left(1 + \mathrm{V_{k_m}}{}^2\right)\left(1 + \mathrm{V_q}{}^2\right)^{n^2} - 1}$$

Equation C.5

When all of the coefficients of variation are not large, say less than 0.3, V_Q may be approximated by:

$$\boldsymbol{V_Q = \sqrt{V_{k_1}{}^2 + V_{k_2}{}^2 + \dots + V_{k_m}{}^2 + n^2V_q{}^2}}$$ **Equation C.6**

In this Handbook, the approximated formula for V_Q is used for computation.

The design event for safety depends on the importance level of structures. The importance levels, design annual probabilities of exceedance (AEP) and the corresponding design gust speed values for wind action specified in the NCC are tabulated in Table C.1, in which $F_C = F_D = 1.0$ are used. Similarly, the importance levels, design AEP and the corresponding design probability factor values for snow and earthquake actions are tabulated in Table C.2.

Table C.1 Annual probability of exceedance and design gust speed for wind action

Importance Level	Region A: AEP	Region A: Design speed (m/s)	Region B: AEP	Region B: Design speed (m/s)	Region C: AEP	Region C: Design speed (m/s)	Region D: AEP	Region D: Design speed (m/s)
1	1:100	41	1:100	48	1:200	61	1:200	72
2	1:500	45	1:500	57	1:500	66	1:500	80
3	1:1000	46	1:1000	60	1:1000	70	1:1000	85
4	1:2000	48	1:2000	63	1:2000	73	1:2000	90

Table C.2 Annual probability of exceedance and design probability factor for snow and earthquake action

Importance Level	Snow: AEP	Snow: Design probability factor	Earthquake: AEP	Earthquake: Design probability factor
1	1:100	1.4	1:250	0.75
2	1:150	1.5	1:500	1.0
3	1:200	1.6	1:1000	1.3
4	1:250	1.65	1:1500	1.5

C.1 Permanent action

The source of uncertainties regarding permanent action include:

(a) variability in densities;
(b) variability in dimensions; and
(c) variability due to the designers' estimates.

The uncertainties are generally regarded as small in comparison to other kinds of action. The probability of occurrence is almost certain and the variability with time is small. There is data on (a) and (b) but not (c), which is the larger source of uncertainty. There is little variation reported in literature from those assumed in this Handbook.

For the purpose of the NCC, the following model for permanent action effect is to be used:

$$G = H_G \times g$$

where:

G = permanent action effect
H_G = factor to convert permanent action to action effect
g = permanent action

The corresponding nominal design action effect is: $G_n = H_{G_n} \times g_n$

Therefore: $(G/G_n) = \left(H_G/H_{G_n}\right) \times (g/g_n)$

The MEAN and COV values of the parameters have been assessed as follows:

$MEAN\left(H_G/H_{G_n}\right) = 0.95$ $\quad$ $COV\left(H_G/H_{G_N}\right) = 0.07$
$MEAN(g/g_n) = 1.05$ $\quad$ $COV(g/g_n) = 0.07$

Therefore:

$MEAN(G/G_n) = 1.0$ $\quad$ $COV(G/G_n) = 0.10$

C.2 Imposed action

The peak imposed action model was established for one-year time reference for (a) office, (b) residential, and (c) school. It used the survey data (JCSS 2001), CIB report on actions on structures (CIB 1989), and structural design actions specified in Australian Standard, AS/NZS 1170.1-2002 (Standards Australia 2016).

The imposed actions are composed of sustained loads such as the weights of furniture and equipment, and intermittent loads that include the crowds of people and furniture stacking during renovation. Following the specification in JCSS (2001), the occurrences of both sustained and intermittent loads are assumed to follow Poisson processes with occurrence rates λ and ν, respectively.

The magnitude of sustained load s is assumed to be gamma distributed, with variance contributed from two sources: one is variance $\sigma^2{}_v$ around the mean μ_s, and another is $\sigma^2{}_u$ accounting for spatial load variation within the defined floor area. The variance is further influenced by a 'peak factor', κ. Hence the variance ${\sigma_s}^2$ of sustained load is determined by:

$$\boldsymbol{\sigma_s}^2 = \boldsymbol{\sigma_v}^2 + \boldsymbol{\sigma_u}^2 \frac{\mathbf{A_0}}{\mathbf{A}} \boldsymbol{\kappa}$$ Equation C.7

where A_o is the reference area and A is the floor area. It is assumed that $A_o/A = 1$ and $\kappa = 2$.

Similarly, the variance $\sigma^2{}_p$ of intermittent load is determined as follows:

$$\boldsymbol{\sigma}^2{}_\mathbf{p} = \boldsymbol{\sigma}^2{}_\mathbf{U} \frac{\mathbf{A_0}}{\mathbf{A}} \boldsymbol{\kappa}$$ Equation C. 8

where $\sigma^2{}_U$ is the spatial load variance.

The input parameter values for the imposed load models are tabulated in Table C.3 and Table C.4. Monte Carlo simulation with a sample of size 10,000 is conducted to obtain the annual maxima of the combined sustained and intermittent loads. The simulated annual maximum imposed loads larger than the 90th-percentile value are then fitted to lognormal distribution. The mean value, standard deviation, and the probability of design load estimated by the fitted lognormal distribution are tabulated in Table C.5.

Table C.3 Model parameters used in simulation for sustained imposed action

Type of use	Design load (kPa)	Ref. area	μ_s(kPa)	σ_v (kPa)	σ_u (kPa)	$1/\lambda$
Office	3.0	20	0.5	0.3	0.6	5
Residential	2.0	20	0.3	0.15	0.3	7
School	3.0	100	0.6	0.15	0.4	10

Table C.4 Model parameters used in simulation for intermittent imposed action

Type of use	Design load (kPa)	Ref. area	μ_p(kPa)	σ_U (kPa)	$1/\nu$
Office	3.0	20	0.4	0.4	1.0
Residential	2.0	20	0.3	0.3	1.0
School	3.0	100	0.5	0.4	0.3

Table C.5 Mean value and standard deviation determined by simulation for imposed action

Type of use	Design load (kPa)	Ref. area	Mean (kPa)	COV	Probability of design load
Office	3.0	20	1.33	0.46	0.98
Residential	2.0	20	1.00	0.44	0.97
School	3.0	100	1.87	0.40	0.93

Note that for many types of structural use, the mean duration of sustained load before a magnitude change is often greater than one year and that of intermittent load is often around one year, there is some extent of dependence between annual maxima. As a result, the imposed action models derived based on annual load maxima, which are implicitly assumed to be independent, are only approximate models.

For the purpose of reliability assessment, it is desirable to have only one representative model for imposed action, q. Since $MEAN(q/q_N)$, $COV(q/q_N)$, where q_N stands for imposed design load, and the probability of design values for office, residential, and school are sufficiently close, a common model for imposed action could be taken as the average of the three types of occupancy. The proposed model for imposed action then has the following properties:

$$MEAN(q/q_N) = 0.52 \qquad COV\ q/q_N = 0.43$$

For the purpose of this document, the following model for imposed action effect is to be used:

$$Q = H_Q \times q$$

where:

Q = imposed action effect
H_Q = factor to convert imposed action to action effect

The corresponding nominal design action effect is: $Q_n = H_{Q_N} \times q_n$

Therefore: $(Q/Q_n) = (H_Q/H_{Q_n}) \times (q/q_n)$

The MEAN and COV values of the parameters have been assessed as follows:

$MEAN(H_Q/H_{Q_n}) = 0.95$ $\quad COV(H_Q/H_{Q_n}) = 0.07$

Therefore:

$MEAN(Q/Q_n) = 0.50$ $\quad COV(Q/Q_n) = 0.43$

C.3 Wind action

Wind action on buildings is a function of a number of factors:

(a) the wind climate;
(b) the building exposure; and
(c) the building shape and dimension.

The wind climate in Australia is classified into non-cyclonic and cyclonic regions. The non-cyclonic regions are further divided into Regions A (1 to 7) and B, and the cyclonic regions into Regions C and D, as specified in Australian Standard AS/NZS 1170.2:2011. The variable used to describe the wind action is the 0.2-second regional gust speed at 10 m above ground in terrain category 2 (Standards Australia 2011).

The regional gust speeds V for wind regions A to D, specified in Table 3.1 of AS/NZS 1170.2:2011, at 14 exceedance probability levels ($F_C = F_D = 1.0$) are fitted to lognormal distributions. Table C.6 shows the fitted model parameters (η_v and ζ_v), corresponding to η_X and ζ_X, respectively, in Equation C.1 and the estimated means and COVs from the fitted distributions. The means and COVs of V/V_N are assessed in accordance with the importance level requirement of the NCC (ABCB 2019), as tabulated in Table C.7 for Regions A and B and Table C.8 for Regions C and D.

Table C.6 Fitted regional gust wind model parameters, mean and COV

Region	η_v	ζ_v	Mean (*V*)	COV (*V*)
A	3.34	0.16	28.6	0.16
B	3.15	0.30	24.3	0.31
C	3.34	0.29	29.3	0.30
D	3.39	0.34	31.3	0.34

Table C.7 Mean and COV of (V/VN) for Regions A and B

Importance Level	Region A: Mean (V/V_N)	Region A: COV (V/V_N)	Region B: Mean (V/V_N)	Region B: COV (V/V_N)
1	0.70	0.16	0.51	0.31
2	0.64	0.16	0.43	0.31
3	0.62	0.16	0.41	0.31
4	0.60	0.16	0.39	0.31

Table C.8 Mean and COV of (V/V_N) for Regions C and D

Importance Level	Region C: Mean (V/V_N)	Region C: COV (V/V_N)	Region D: Mean (V/V_N)	Region D: COV (V/V_N)
1	0.48	0.30	0.44	0.34
2	0.44	0.30	0.39	0.34
3	0.42	0.30	0.37	0.34
4	0.40	0.30	0.35	0.34

For the purpose of this document, only buildings not sensitive to wind dynamic effects are considered. The following model for wind action effect is to be used:

$$W = H_W \times C \times (M \times V)^2$$

where:

V = the basic wind speed whose statistics are available and given in AS/NZS 1170.2 in terms of annual probability of exceedance

M = factor to convert all multipliers for the wind speed direction, exposure, shielding and topographic effects

C = the aerodynamic shape factor to convert wind speed to wind pressure

H_w = factor to convert wind pressure to wind action effect

W = wind action

The corresponding nominal design action effect is: $W_n = H_{W_n} \times C_n \times (M_n \times V_n)^2$

Therefore: $(W/W_n) = \left(H_W/H_{W_n}\right) \times (C/C_n) \times (M/M_n)^2 \times (V/V_n)^2$

The Mean and COV values of the parameters have been assessed as follows:

$MEAN\left(H_w/H_{W_n}\right) = 0.8 \qquad COV\left(H_W/H_{W_n}\right) = 0.10$

$MEAN(C/C_n) = 1.0 \qquad COV(C/C_n) = 0.2$

$MEAN(M/M_n) = 1.0 \qquad COV(M/M_n) = 0.15$

The factor H_W for wind action is reduced to 0.8 because the wind action is essentially a dynamic action which has been conservatively transformed into an equivalent static action. The coefficient of variation for other factors is based on the JCSS assessment.

For the purpose of the reliability indices, the MEAN(W/W_n)'s and COV(W/W_n)'s of Regions A and C, as shown in Table C.9, are used respectively for non-cyclonic and cyclonic regions.

Table C.9 Mean and coefficient of variation values for peak annual wind actions for non-cyclonic and cyclonic regions of Australia

Importance Level	Non-cyclonic Mean (W/W_n)	Non-cyclonic COV (W/W_n)	Cyclonic Mean (W/W_n)	Cyclonic COV (W/W_n)
1	0.40	0.49	0.20	0.71
2	0.34	0.49	0.17	0.71
3	0.32	0.49	0.15	0.71
4	0.30	0.49	0.14	0.71

C.4 Snow action

Snow action on buildings is a function of a number of factors:

(a) the snow climate;
(b) the building exposure; and
(c) the building shape and dimension, particularly that of the roof.

Snow load in Australia affects only limited regions generally known as the alpine and the sub-alpine regions located in NSW, Victoria and Tasmania, as defined in AS/NZS 1170.3:2003. The variable used to describe the snow action is the ground snow load. The ground snow load at each of the specified probability of exceedance levels is normalised by the ground load at 1/20 exceedance level and is termed probability factor.

The probability factors s_G for ground snow load specified in AS/NZS 1170.3:2003 are fitted to a lognormal distribution. Table C.10 shows the fitted model parameters (η_{s_G} and ζ_{s_G}) and the estimated means and COVs from the fitted distributions. The means and COVs of s_G/s_{GN} are assessed in accordance with the importance level requirement of the NCC, as tabulated in Table C.11.

Table C.10 Fitted model parameters, mean and coefficient of variation for probability factor of snow action

η_{s_G}	ζ_{s_G}	Mean (s_G)	COV (s_G)
–0.83	0.50	0.49	0.53

Table C.11 Mean and coefficient of variation of sG/sGN

Importance Level	Mean (s_G/s_{GN})	COV (s_G/s_{GN})
1	0.35	0.53
2	0.33	0.53
3	0.31	0.53
4	0.30	0.53

For the purpose of this document, the following model for snow action effect is to be used:

$$S = H_S \times C_E \times C_F \times s_G$$

where:

s_G = the ground snow load whose statistics are available and given in AS/NZS 1170.3 in terms of annual probability of exceedance
C_E = factor to cover the effects of exposure
C_F = factor to cover the geometrical effects such as the roof shape
H_S = factor to convert snow action to snow action effect

The corresponding nominal design snow action effect is: $S_n = H_{S_n} \times C_{E_n} \times C_{F_n} \times s_{G_n}$

Therefore: $S/S_n = \left(H_S/H_{S_n}\right) \times \left(C_E/C_{E_n}\right) \times \left(C_F/C_{F_n}\right) \times \left(s_G/s_{G_n}\right)$

The MEAN and COV values of the parameters have been assessed as follows:

$MEAN\left(H_S/H_{S_n}\right) = 0.9$ $\quad COV\left(H_s/H_{S_n}\right) = 0.10$

$MEAN\left(C_E/C_{E_n}\right) = 1.0$ $\quad COV\left(C_E/C_{E_n}\right) = 0.15$

$MEAN\left(C_F/C_{F_n}\right) = 1.0$ $\quad COV\left(C_F/C_{F_n}\right) = 0.10$

The factor H_s was kept the same as for other gravity loads and the factors C_E and C_F were as given in JCSS. The MEAN and COV values for peak annual snow action are given in Table C.12.

Table C.12 Mean and coefficient of variation values for peak annual snow actions

Importance Level	Mean(S/S_n)	COV(S/S_n)
1	0.32	0.57
2	0.30	0.57
3	0.28	0.57
4	0.37	0.57

C.5 Earthquake actions

The variable used to describe the earthquake action is the acceleration coefficient for limit state, as defined in AS/NZS 1170.4–2007. The acceleration coefficient at each of the specified probability of exceedance levels is normalised by the acceleration coefficient at 1/500 exceedance level and is termed probability factor.

The probability factors a for acceleration coefficient specified in AS/NZS 1170.4–2007 are fitted to a lognormal distribution. Table C.13 shows the fitted model parameters (η_a and ζ_a) and the estimated means and COVs from the fitted distributions. The means and COVs of a/a_N are assessed in accordance with the Importance Level requirement of the NCC, as tabulated in Table C.14.

Table C.13 Fitted model parameters, mean and coefficient of variation for probability factor of earthquake action

η_α	ζ_α	Mean (a)	COV (a)
–3.59	1.25	0.06	1.94

Table C.14 Mean and coefficient of variation of a/a_N

Importance Level	Mean (a/a_N)	COV (a/a_N)
1	0.08	1.94
2	0.06	1.94
3	0.05	1.94
4	0.04	1.94

For the purpose of this document, the following model for earthquake action effect is to be used:

$$E = H_E \times C_R \times C_S \times C_W \times a$$

where:

a = the ground acceleration coefficient whose statistics are available and given in AS/NZS 1170.4 in terms of annual probability of exceedance
C_W = factor to cover the effects of mass distribution of the building

C_s = factor to cover the effects of the ground condition
C_R = factor to cover the dynamic response of the building
H_S = factor to convert earthquake action to earthquake action effect

The corresponding nominal design earthquake action effect is:

$$E_n = H_{E_n} \times C_{R_n} \times C_{S_n} \times C_{W_n} \times a_n$$

Therefore: $(E/E_n) = \left(H_E/H_{E_n}\right) \times \left(C_R/C_{R_n}\right) \times \left(C_S/C_{S_n}\right) \times \left(C_w/C_{W_n}\right) \times (a/a_n)$

The MEAN and COV values of the parameters have been assessed as follows:

$$MEAN\left(E_S/E_{S_n}\right) = 0.9 \qquad COV\left(E_S/E_{S_N}\right) = 0.10$$
$$MEAN\left(C_R/C_{R_n}\right) = 1.0 \qquad COV\left(C_R/C_{R_n}\right) = 0.10$$
$$MEAN\left(C_S/C_{S_n}\right) = 1.0 \qquad COV\left(C_S/C_{S_n}\right) = 0.10$$
$$MEAN\left(C_W/C_{W_n}\right) = 1.0 \qquad COV\left(C_W/C_{W_n}\right) = 0.10$$

Therefore the MEAN and COV values for peak annual earthquake action are given in Table C.15.

Table C.15 Mean and coefficient of variation values for peak annual earthquake actions

Importance Level	Mean(S/Sn)	COV(S/Sn)
1	0.072	1.98
2	0.054	1.98
3	0.045	1.98
4	0.036	1.98

Appendix D Derivation of reliability index

Assume that a structure with resistance R is subjected to action Q. The safety of structures requires the performance function $G(R, Q)$ to be:

$$\mathbf{G(R, Q) = R - Q > 0} \qquad \text{Equation D.1}$$

or equivalently:

$$\mathbf{G(R, Q) = R/Q > 1} \qquad \text{Equation D.2}$$

The failure probability p_f is often evaluated through a reliability index β and normal distribution as follows:

$$\mathbf{p_f = 1 - \Phi^{-1}(\beta) = \Phi^{-1}(-\beta)} \qquad \text{Equation D.3}$$

When both action Q and resistance R follow lognormal distributions, a function for reliability index that gives exact failure probability through Equation D.3 exists. Let η_R and ζ_R be the probability model parameters of R, and η_Q and ζ_Q the model parameters of Q, then:

$$\mathbf{p_f = Pr\left\{\frac{R}{Q} \leq 1\right\} = Pr\left\{\ln\frac{R}{Q} \leq 0\right\} = \Phi^{-1}\left(-\frac{\eta_R - \eta_Q}{\sqrt{{\zeta_R}^{w} + {\zeta_Q}^{2}}}\right)} \qquad \text{Equation D.4}$$

Comparing Equation D.3 and Equation D.4, we have:

$$\boldsymbol{\beta = \frac{\eta_R - \eta_Q}{\sqrt{{\zeta_R}^2 + {\zeta_Q}^2}}} \qquad \text{Equation D.5}$$

Let Q_m and V_Q denote the mean and COV, respectively, of action, and R_m and V_R the mean and COV, respectively, of resistance. Thus:

$$\mathbf{H_R = \ln\frac{R_m}{\sqrt{1+{V_R}^2}};\ \eta_Q = \ln\frac{Q_m}{\sqrt{1+{V_Q}^2}}} \qquad \text{Equation D.6}$$

and

$$\boldsymbol{\zeta_R}^2 = \ln(1 + \mathbf{V_R}^2)\,;\,\boldsymbol{\zeta_Q}^2 = \ln(1 + \mathbf{V_Q}^2)$$ **Equation D.7**

Let $C_R = 1 + V_R{}^2$ and $C_Q = 1 + V_Q{}^2$. Then the reliability index β is derived to be:

$$\boldsymbol{\beta} = \frac{\ln\left(\frac{R_m}{Q_m}\sqrt{\frac{C_Q}{C_R}}\right)}{\sqrt{\ln(C_Q C_R)}}$$ **Equation D.8**

The meaning of β may be explained geometrically, as shown in Figure D.1, in which the probability density function of R/Q is schematically depicted. It shows that β represents the number of standard deviations by which the mean of R/Q is away from the origin. It shows also that the failure probability p_f (shaded area) is the total area of probability density on the left of the origin.

For Q and R following lognormal distributions, Equation D.8 is an exact formula for reliability index. When the coefficient of variation of Q or R, or both, is small, say < 0.30, some practices use an approximate equation of as follows,

$$\boldsymbol{\beta} = \frac{\ln(R_m/Q_m)}{\sqrt{V_R{}^2 + V_Q{}^2}} = \frac{\ln(R_m/Q_m)}{\sqrt{(C_R - 1)(C_Q - 1)}}$$ **Equation D.9**

The comparison for β values versus V_Q using exact (Equation D.8) and approximate (Equation D.9) formulae is shown in Figure D.1, in which the means of R and Q are assumed to be 2.5 and 1, respectively, and V_R to be 0.1. It shows that for $V_Q > 0.3$, the computed β value by Equation D.9 may become unacceptably inaccurate; for $V_Q = 2$, which is roughly equal to the variance of earthquake load, the β value is 1.35 by Equation D.8 and is 0.46 by Equation D.9.

Figure D.1 Probability density of $\ln(R/Q)$ and geometrical meaning of β.

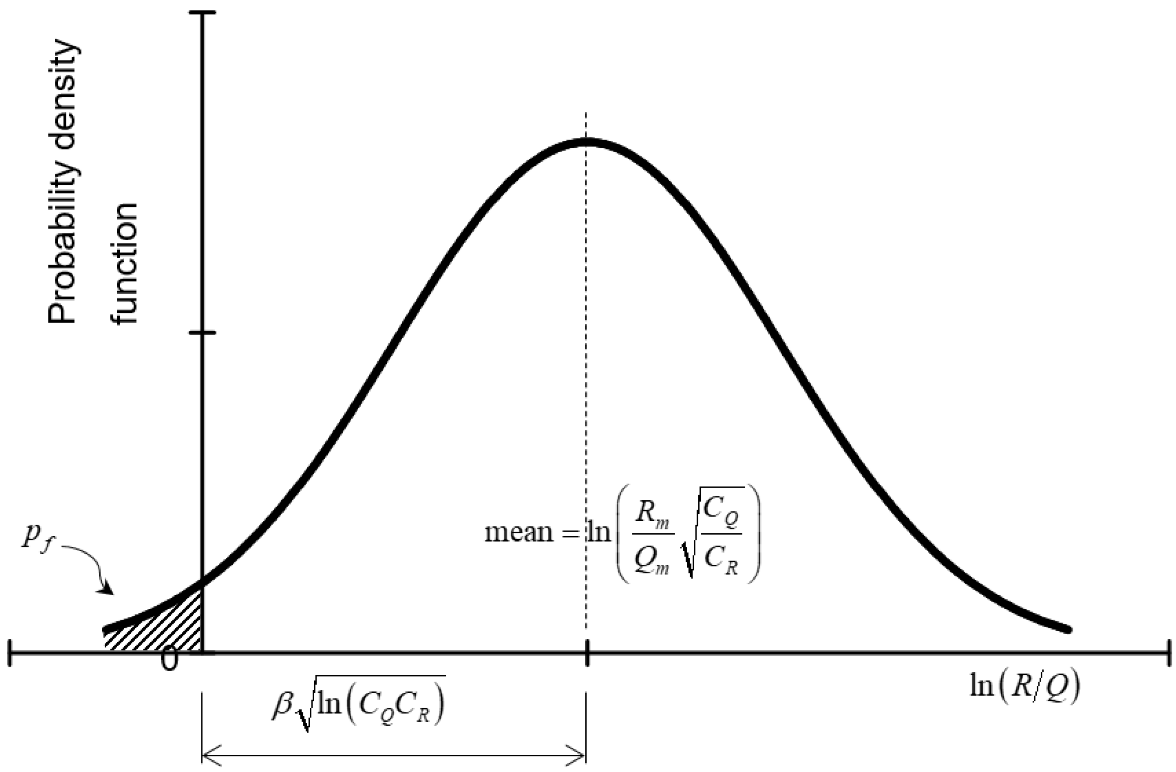
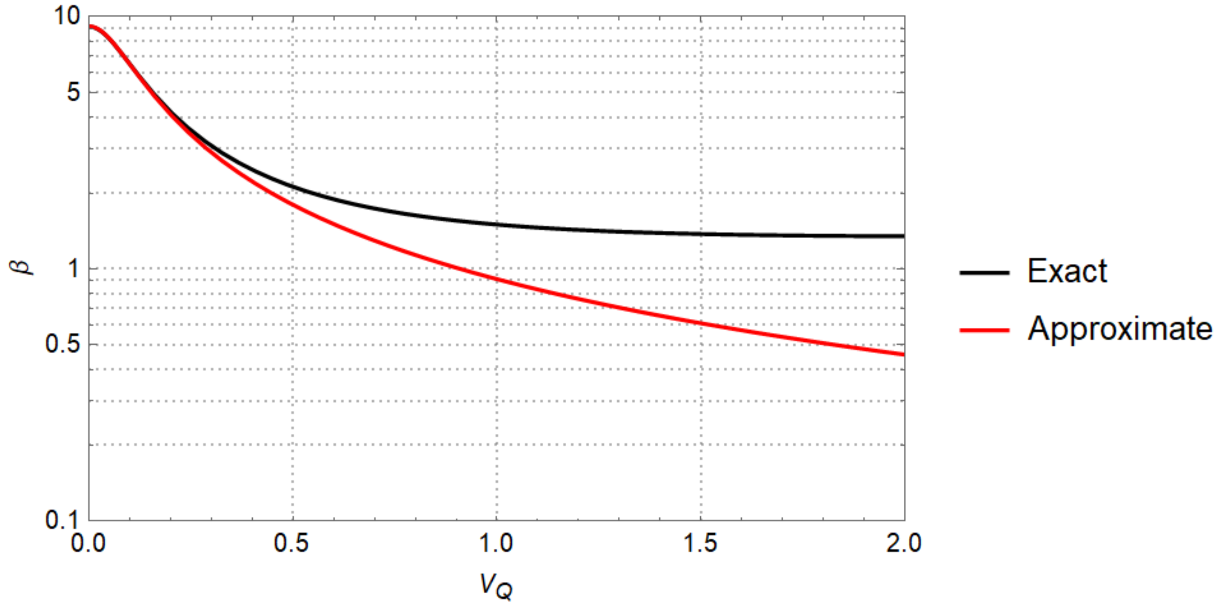


Figure D.2 Comparison of β versus V_Q computed by exact (Equation D.8) and approximate (Equation D.9) formulae.



Appendix E Frequently asked questions

E.1 Why is a one year reference period used?

The reference period affects the load models for time-varying actions such as imposed actions and wind actions. The load models for these actions assumed a statistically stationary process with peak yearly action as a statistically independent random variable. Under these assumptions, it is a simple matter to convert the models from 1 year to 50 years or any other period. While the earlier reliability works[6] used 50 years as the reference period as it was the fashion of the time, the use of one year is more realistic as the resistance models used do not include a time deterioration factor and the load models used peak yearly action as the independent random variable. ISO 2394:2015 also used one year time reference for its recommendations on target reliability indices. Its recommended reliability indices, on average, are fairly close to that of the Verification Method.

E.2 How were the action models derived?

A full derivation of the action models is given in Appendix C of this Handbook. The load models were based on data from the Joint Committee on Structural Safety (JCSS) probabilistic load models. The upper tails of the generated data from the JCSS models were fitted to lognormal distributions for use in the computation of the reliability indices. For imposed and wind actions, these models are only typical representative models:

- A wide range of imposed action models are possible as they are dependent on the type of occupancy, the referenced areas and the type of components. The imposed action model adopted is a hybrid model involving three types of occupancy with one referenced area.
- Wind actions vary with wind regimes and importance level, eight wind models were created corresponding to four levels of importance and two wind regimes (non-cyclonic and cyclonic). The revision concluded that this could be simplified to two representative models for the two wind regimes without loss of accuracy in the determination of capacity reduction factor φ.

E.3 How are load combinations taken into account?

For the development of the load combinations of AS/NZS 1170.0, reliability indices were used to ensure consistency of the combinations for all ranges of load ratios. The Verification Method was developed with an entirely new objective, namely to determine the capacity reduction factor φ for any new product. The load combinations to be used in design are assumed to be those provided in AS/NZS 1170.0. Considering that only one φ factor is going to be used for all combinations of loads, some weighting or averaging process must be used to achieve this purpose. Averaging of the φ (or β) for individual permanent, imposed and wind actions is one way, not the only way, but the simplest way of achieving this objective. It can also be shown that this process is conservative for additive load combinations. The case of subtractive load combination (dead load counteracting uplift wind) was also examined, the resulting minimum reliability indices were controlled not by the φ factor but the load factor on dead load.

E.4 What is the relationship of the Verification Method to ISO 2394?

The Verification Method is a self-contained method without reference to any other document. The Verification Method was developed as a product development tool designed to fit with current Australian codes and standards.

ISO 2394 'General principles on reliability for structures' sets out the principles for reliability-based design. Its principles are applicable to all structures and structural elements, construction, use of structures, maintenance, rehabilitation and decommissioning. It is extremely difficult to make reference to such a wide ranging document in regulation. Therefore reference to ISO 2394 is made in guidance documents to the Verification Method but not within the Verification Method itself.

E.5 Could the Verification Method be used for calibration of Australian Standards?

As stated earlier, the Verification Method was designed as an option to be used for demonstrating compliance with NCC Performance Requirements BP1.1 and BP1.2 in NCC Volume One and P2.1.1(a), (b) and (c) in NCC Volume Two. Its primary purpose is for product development. The advantage is that its use is a straightforward means of demonstrating compliance compared to other methods. However, as is the case for all methods of demonstrating compliance with the NCC Performance Requirements, the NCC evidence of suitability requirements and any State or Territory requirements would also need to be satisfied. Whether it is suitable for use in ‘code calibration’ is a matter of judgement for the relevant committee. It must be emphasised that the Verification Method is only one option amongst other ‘acceptable’ options when demonstrating compliance under a performance pathway. Initial opposition to the Verification Method in 2015 is thought to be the result of a misunderstanding that the Verification Method was mandatory. However, it should be noted that comparison of reliability indices from the Verification Method and other reliability analysis methods is nonsensical unless the many assumptions behind each method are carefully examined.

E.6 Why are there two methods of deriving the target reliability indices?

The Verification Method offers two methods of deriving the target reliability indices:

1. A fixed set of targets in NCC Volume One Table BV1.2: This set of targets is to be used when it is not appropriate to compare the product with an equivalent DTS product. This set of target reliability indices represents conservative estimates of the average indices from common structural components and materials; or

2. A set of target reliability indices can be derived by the user by establishing a resistance model for an appropriate equivalent DTS product and establishing the φ value for the new product accordingly. This set of targets will generally be lower than that of Table BV1.2. This is acceptable as long as it is within 0.5 of Table BV1.2.