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Concrete structures — Commentary (Supplement 1 to AS 3600:2018)



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Preface

This Supplement was prepared by Standards Australia Committee BD-002, Concrete Structures, to supersede AS 3600—2009 Supp 1:2014, *Concrete structures—Commentary (Supplement to AS 3600:2009)*.

The objective of this document is to provide commentary to assist in the interpretation of the provisions of AS 3600:2018, *Concrete structures*, by —

- (a) providing background reference material to the requirements in the Standard;
- (b) indicating the origin of particular requirements;
- (c) indicating departures from previous practice due to changes that have occurred in construction practice and theory since the previous edition of the Standard was published; and
- (d) explaining the application of certain clauses.

In this document, AS 3600:2018 is referred to as “the Standard” or “the 2018 edition of the Standard” while this document is referred to as “this document”. The numbers and titles of sections and clauses used in this document align with those in the Standard except that the numbers are prefixed with the letter “C”. The figures in this document are numbered according to the clause in which they are referenced and also carry the “C” prefix. Therefore references to sections, clauses, tables, figures, equations and appendices that are not identified by the prefix “C” refer to the Standard unless indicated otherwise.

Documents referenced in the text and documents recommended for further reading are listed at the end of the applicable section or appendix to which they relate.

While it is intended that this document be read in conjunction with the Standard, it does not form an integral part of the Standard.

The opportunity has also been taken to include improvements suggested since the last edition by users. Like the Standard itself, this document is an ongoing work-in-progress. Suggestions for improvements to both the Standard and to this document, in regard either to content or to clarity of wording, are therefore welcomed.

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NOTES

Supplement

Concrete structures — Commentary (Supplement 1 to AS 3600:2018)

Section C1 Scope and general

C1.1 Scope and application

C1.1.1 Scope

Most concrete structures in Australia are designed and constructed to comply with the National Construction Code (NCC) (Ref. 1) and for such structures the design requirements are set out in the NCC and in AS/NZS 1170.0 (Ref. 2). AS 3600:2018 (the Standard) sets out minimum provisions for the design and construction of concrete structures in Australia. It is called up by the NCC so that compliance with the requirements of the NCC is deemed to be satisfied by following the provisions of the Standard.

The Standard covers reinforced and prestressed concrete structures. It does not provide rules for all plain concrete structures, except for plain concrete footings and pedestals. The exclusion of mass concrete structures recognizes the fact that they are generally outside the range of normal structures. The treatment of plain concrete members in the Standard is thus limited, and other design criteria, not covered therein, will usually need to be considered in the design of plain concrete members and structures.

Clause 1.1.1 Note 1 points out that much of the content of the Standard is based on general principles and therefore may be applicable to design situations not specifically covered by the Standard.

In the preparation of a standard such as the Standard, a certain level of knowledge and competence of the users has to be assumed. As indicated by Clause 1.1.1 Note 2, it is assumed that the users would be professionally qualified civil or structural engineers experienced in the design of concrete structures, or equally qualified but less experienced persons working under their guidance. Therefore it is intended that the Standard be applied and interpreted primarily by such persons. Similarly, it is intended that the construction of the structure be carried out and supervised by suitably qualified persons using appropriate quality control systems.

C1.1.2 Application

Clause 1.1.2 places various restrictions on the materials that can be used in conjunction with the Standard.

A lower limit on concrete compressive strength of 20 MPa is imposed because strength grades less than this are not normally suitable for structures. In the Standard, the upper limit for the concrete compressive strength is 120 MPa. The design procedures in the Standard apply to structures with concrete strengths within these limits. This is not to suggest that concretes with greater strength cannot be produced commercially or cannot be used in the construction of concrete structures. However, when used in such situations, the applicability of the rules given in the Standard needs to be checked.

Concretes made from naturally occurring Australian coarse aggregates have surface-dry densities falling in the range 2100 kg/m³ to 2800 kg/m³. Lightweight structural concretes in Australia generally use naturally occurring sands combined with manufactured lightweight aggregates, for which the surface-dry density is seldom less than 1800 kg/m³. Density limits for structural concretes have been set accordingly.

Chemical, mechanical and dimensional requirements for steel reinforcement having lower characteristic yield/proof stresses (f_{sy}) of 250 MPa or 500 MPa are given in AS/NZS 4671 (Ref. 3). The number corresponding to f_{sy} is also termed the strength grade. AS/NZS 4671 defines three ductility classes distinguished in requirements by the letters “L” (low), “N” (normal) and “E” (earthquake), placed immediately after the strength-grade number. For each ductility class, different minimum values for uniform elongation/strain and maximum stress-to-yield/proof-stress ratio apply. The latter class (E) is

specifically for use in New Zealand in accordance with NZS 3101 (Refs. 4 and 5), and is not referenced in the Standard.

On account of its low ductility, reinforcing steel of Ductility Class L has some design restrictions placed on its use when acting as longitudinal tensile reinforcement compared with Ductility Class N reinforcing steel. These design restrictions are given in later clauses of the Standard, and are intended to prevent the Ductility Class L reinforcing steel from being used in any situation where it is required to undergo large plastic deformation under strength limit state conditions. By large plastic deformation, it is meant reaching longitudinal tensile strains at any point along the bar or wire in excess of the uniform elongation. The uniform elongation corresponds to the onset of local necking when the maximum stress is reached in the bars. Moreover, at strains in excess of the uniform elongation a reinforcing bar would be in a state of incipient failure due to bar fracture. Premature fracture of the reinforcing bars at a critical section of an indeterminate member could prevent the formation of a complete collapse mechanism and thereby unduly weaken the member. This should be avoided.

The former limitation on steel grades available for use (up to 500 MPa) did not allow manufacturers and designers to explore the options offered by higher grades of steel, which may reduce cost or enhance sustainability in addition to carrying higher loads. Since 2009, the Standard has allowed the yield stress for fitments to be up to 800 MPa in Clause 10.7.3.3. In the 2018 revision, the existing grades remain unchanged but limits for other reinforcing elements were extended to 600 MPa to allow industry to explore the benefits of higher grades of steel. AS/NZS 4671 was revised and published in 2019 (Ref. 3) to include these higher strength steels. These changes are designed to encourage innovation and product development by manufacturers and designers but will ultimately benefit the end user of buildings and construction infrastructure. It has the potential to improve the sustainability outcomes for the Australian community by reducing the cost to build a structure and/or reducing the impact on the environment, as less material may be required. For ultimate limit states, the strength of the reinforcement in design models is now limited to 600 MPa, unless noted otherwise, as existing research supports this upper bound for the strength equations. Further research is expected in the future to review these strength models.

In Section 16, minimum requirements are provided for reinforced concrete structures containing steel fibres. The Standard does not provide any rules for any other special types of reinforcement such as polypropylene fibres and carbon-fibre-based reinforcement materials. This is partly due to the limited application of these reinforcement types at the time of preparation of the Standard, because of their cost and/or limited availability, and partly because of the limited data available for the development of general design provisions. Because of their low modulus of elasticity, polypropylene fibres may not be suitable to resist applied loads in concrete structures unless supplemented by other structural reinforcement. However, polypropylene fibres can be useful in controlling early-age shrinkage and thermal cracking. Stainless steel reinforcement may be used in aggressive marine environments to overcome the difficulties associated with providing appropriate concrete quality and cover to protect normal reinforcing steel (Refs. 6 and 7). Designers should ask suppliers for data on the appropriate material properties for the situation in which they propose to use this reinforcement.

C1.1.3 Exclusions

Clause 1.1.3 recognizes the fact that separate standards have been developed for a range of specific structures and that the design requirements and material specifications in these standards may take precedence over the requirements of the Standard.

C1.2 Normative references

All documents referenced in the Standard that form part of the requirements of the Standard are listed in Appendix A. Documents referenced in the Standard for informative purposes are listed in the Bibliography at the end of the Standard.

Conformance with the documents listed in Appendix A is mandatory to satisfy the requirements of the Standard, and by association, comply with the NCC. Designers, specifiers, contractors and supervisors should be aware that alternative specifications are not to be substituted for these normative documents.

The documents referenced in the Standard are subject to revision from time to time. A check should be made as to their currency.

C1.3 Existing structures

The evaluation of an existing structure for strength and serviceability may be made using the general principles of structural mechanics, together with information provided in the Standard and this document, as appropriate. Additional information may be obtained by testing of structures and structural members in accordance with Appendix B, which is normative and deals with the testing of members and structures.

C1.4 Documentation

Clause 1.4 outlines the information that should be included on the drawings and/or specifications for a concrete structure. The information required for Clause 1.4(a) to (j) and (o) to (q) for the structure should be shown on only one of the drawings, usually the first sheet, or cited in the project specification as appropriate (Refs. 8 and 9). The information required for Clause 1.4(k) to (n) and (r) for each structural member should be shown on at least one of the drawings containing plans, elevations or sections of that member.

C1.5 Construction

The extent of supervision and inspection on a project will depend on its importance and complexity. Any structure that is large (i.e. extensive in plan area and/or above five storeys in height) or contains tendons that are stressed, is considered to be complex and should be supervised by a person responsible to a qualified engineer and experienced in the supervision of comparable structures.

In addition, the works should be inspected at specified stages by a suitably qualified person.

If any doubt arises during construction regarding a design matter, or the interpretation of the documents, the contractor, supervisor, inspector or their representative should refer it to the designer for resolution.

Suitable site records of the project should be kept during construction and be available for inspection during the progress of the work, and after completion of the work.

Such records should include, as appropriate —

- (a) quantity, grade, ductility class and type of reinforcement and grade and type of tendons;
- (b) each date on which concrete was placed and the corresponding location of that concrete in the structure;
- (c) the results of all tests on the test samples of concrete together with the locations in the structure of the batches sampled, and copies of the supplier's identification certificates;
- (d) details of all prestressing and grouting operations; and
- (e) as-constructed drawings, specifications and issued instructions.

C1.6 Definitions

No comment is made here on the definitions included in the Standard. However, comment is given in later clauses on some terms and definitions, as appropriate, where they are first used.

C1.7 Notation

Most items of notation in the Standard have been in use for some years, and are not discussed here. However, new notation has been introduced into a number of clauses of the Standard and such notation

is discussed in the relevant clauses of this document. For example, a range of new capacity reduction (ϕ) factors for strength design has been introduced in Section 2 and these factors are discussed in [Clause C2.2](#) of this document.

The principles that have been used in developing the notation are broadly in agreement with those of ISO 3898 (Ref. 10) which specifies only the general terms. The particular terms used in the Standard have been derived using these principles.

C1.8 References

- [1] National Construction Code (NCC), Building Code of Australia, Australian Building Codes Board, Canberra, Vol. 1 and 2, 2019.
- [2] AS/NZS 1170.0, *Structural design actions, Part 0: General principles*, Standards Australia, Sydney, 2002.
- [3] AS/NZS 4671, *Steel for the reinforcement of concrete*, Standards Australia, Sydney, 2019.
- [4] NZS 3101.1, *Concrete structures standard, Part 1: The design of concrete structures*, Standards New Zealand, Wellington, 2006.
- [5] NZS 3101.2, *Concrete structures standard, Part 2: Commentary on the design of concrete structures*, Standards New Zealand, Wellington, 2006.
- [6] BS 6744, *Stainless steel bars — Concrete reinforcement — Requirements and test methods*, British Standards Institution, 2016.
- [7] BS EN 10088-1, *Stainless steels — List of stainless steels*, British Standards Institution, 2014.
- [8] CIA Z06, *Reinforcement Detailing Handbook*, Concrete Institute of Australia, Sydney, 2010.
- [9] AS/NZS 1100.501, *Technical drawing, Part 501: Structural engineering drawing*, Standards Australia, Sydney, 2002.
- [10] ISO 3898, *Bases for design of structures — Names and symbols of physical quantities and generic quantities*, International Organization for Standardization, Switzerland, 2013.

Section C2 Design procedures, actions and loads

C2.1 Design procedures

The aim of structural design is to provide a structure that has adequate strength, is serviceable, robust and durable, and serves its intended function while satisfying various other relevant requirements such as ease of construction and overall economy.

The procedures set out in Section 2 deal with the design of concrete structures for strength and serviceability, earthquake actions, robustness, durability, fire resistance and fatigue, and are in accordance with the requirements of the NCC (Ref. 1) and AS/NZS 1170.0 (Ref. 2). For those structures that fall outside the jurisdiction of the NCC, the general design requirements are set out in AS/NZS 1170.0. Information on design loads and design actions, in addition to the information in Clause 2.4, are to be found in the relevant parts of the AS/NZS 1170 series (Refs. 3 to 5).

No mention is made in the NCC, or in AS/NZS 1170.0, of “ease of construction” or “economy” as design requirements. These are regarded as matters for the building owner, the client, the designer and the building contractor. How a particular structure needs to be detailed to be “easy to construct” and the measures to be taken to ensure that it is “economical” are not therefore covered in the Standard. Nevertheless, designers should be aware of the importance of these matters and address them in each specific design case.

Designers should take account of all relevant design limit states and other criteria that apply to a particular structure by considering it in its own environment and in the context of the use to which it will be put, even when these considerations are not covered in the Standard.

Major changes have been made to Section 2 to align content with AS/NZS 1170.0. Furthermore, Section 2 has been extended to include a range of new, alternative check procedures for the strength design of structures and members.

C2.1.1 Design for strength and serviceability

Clause 2.1.1 refers to AS/NZS 1170.0, which specifies the general principles for design. The procedures specified in Clause 2.1.1 apply to the complete structure and also to its component members.

During construction there may be critical periods for the partially built structure when unusual loads and/or load paths are called into play and the members may not have reached their full design strength. Designers need to consider these situations.

A structure has adequate strength and is serviceable if the probability of structural failure and the probability of loss of serviceability are both acceptably low throughout its intended life. AS/NZS 1170.0 provides only limited requirements for design for serviceability, both for structures covered by the NCC and those outside its jurisdiction. Many limit states for serviceability are seen as not being a matter for regulation within the NCC, and for those structures outside its jurisdiction, the requirements will usually be peculiar to the specific structure. Nevertheless, the Standard provides requirements for specific aspects of serviceability that relate to concrete structures.

The Standard also permits testing of a structure in lieu of calculation to check that the strength and serviceability requirements can be achieved. See also [Appendix CB](#).

C2.1.2 Design for earthquake actions

The provisions for the design for earthquake actions in the Standard are significantly different from the 2009 edition of the Standard. A new section (Section 14) on earthquake actions has been introduced. Section 14 provides values for the structural ductility factor (μ) and structural performance factor (S_p), which can be used for calculating the earthquake design actions in accordance with AS 1170.4. The section also provides corresponding design and detailing requirements for each set of factors.

The requirement in Clause 2.1.2 stating that the “reinforcement shall be detailed to provide the structure with the assumed ductility ...” is there to remind designers that the reduction in earthquake design actions that results from adopting ductility values greater than 1.0 does not come for free, and that a level of detailing consistent with the ductility assumption made in the seismic load analysis is required to ensure the assumed ductility is actually achieved in the structure. Further, earthquake design needs to be treated as a separate loading scenario to gravity and wind ultimate limit state scenarios. As such, the reinforcement detailing for earthquake actions needs to be adopted based on the corresponding structural ductility factor assumed in the design, irrespective of the relative magnitude of the wind and earthquake lateral design actions.

Designers need also to be aware that the intensity and magnitude of the earthquake actions applied to the structure is directly affected by the structure itself (i.e. its mass, stiffness and ductility), as well as by the magnitude of the event, the proximity (distance) of the structure to the event and the site conditions. Design models need to therefore assume effective member stiffnesses that are appropriate for the structural proportions of the respective member and earthquake design actions being considered. This is achieved by adopting the effective section properties given in Clause 6.2.4.2.

Additional commentary regarding design for earthquake actions is provided at the beginning of [Section C14](#).

C2.1.3 Design for robustness and structural integrity

Robustness is one of the prerequisites of structural safety (Ref. 7). A structure needs to be able to withstand local damage caused by accidents or unforeseen events without progressive collapse. The Standard requires that concrete structures are designed to be robust in accordance with AS/NZS 1170.0:2002 Section 6 (Ref. 2), which states that “the structures shall be detailed ... so that the structure can withstand an event without being damaged to an extent disproportionate to that event”. A structure needs to be designed such that should a local accident occur, the damage is contained within an area local to the accident or, should one member be removed, the remainder of the structure would hang together and not precipitate a progressive collapse. This requires that the structural members and the connections between them have adequate ductility (Ref. 7).

The Standard does not specify any quantitative measures of robustness to be used in structural design. Beeby (Ref. 7) has proposed an approach whereby robustness is quantified in terms of a structure’s ability to absorb energy. Examples of the use of Beeby’s approach are available in Ref. 8.

Other sources of design information on the topic are the Designers’ Guide to Eurocode 1 (Ref. 9), and PCA Notes on ACI 318-08 (Ref. 10).

C2.1.4 Design for durability and fire resistance

Durability and fire resistance are critical considerations for many structures and represent limit states that have to be considered by the designer. They need to be satisfied concurrently with the other design requirements and criteria.

A “durable” structure is generally interpreted as meaning one that can withstand the wear and deterioration to be expected during its design life without the need for undue maintenance. In the past, “durability” had been considered to lie outside the scope of the NCC. However, in 2002, the Australian Building Codes Board published a guideline document *Durability in Buildings* (Ref. 11), which provides designers with the various approaches that can be taken to achieve adequate durability for a given structure. Specific guidance for concrete structures with respect to durability is provided in the Standard (refer to Section 4) because it is recognized that, while concrete structures will deteriorate with time and their strength and structural behaviour will be adversely affected, they should continue to function satisfactorily until the end of their “design life”.

The detailed rules for design for fire resistance are given in Section 5.

C2.1.5 Design for fatigue

Design for fatigue is a new requirement in the Standard. Assessment for fatigue of concrete and steel is primarily required only for structures and structural members subjected to a high number of repetitive load cycles, where accumulation of damage over the design life can lead to premature brittle failure at stresses much lower than limited by equivalent static stress. Members subjected to low number of high stress cycles are not covered by the Standard, and the designer should carry out a special study in these circumstances.

As fatigue is the accumulation of strain damage it is important that the stresses in the concrete and steel materials remain within the elastic range to avoid any effects of strain hardening. For this reason, it is recommended that the permanent applied stress in concrete is not more than $0.45f'_c$ and the maximum stress in steel reinforcing bar is not more than $0.8f_{sy}$.

C2.1.6 Material properties

Clause 2.1.6 draws the attention of designers to the necessity of adopting appropriate values of material properties in both analysis and design. In particular, the nonlinear strength design procedures require the use of the mean concrete strength rather than the traditional characteristic strength. The Standard allows a wide range of strength design approaches to be used (refer to Clause 2.2). In Section 3, appropriate material properties to be used in design are specified and these are intended to be used in the absence of more accurate test data.

Concrete is unlike many other materials in that its design properties vary with time. These properties are affected by how, and for how long, the concrete is cured. The material properties that are used in the design should therefore reflect those of the concrete in the structure at the appropriate age.

C2.2 Design for strength

In Clauses 2.2.2 to 2.2.6, details are given of five alternative checking procedures that may be used in the strength design of concrete members and structures. Each alternative checking procedure is paired with one of the five alternative methods of analysis that are covered in Section 6. These strength check clauses acknowledge the existence of sophisticated and accurate methods of analysis that have become available, such as linear and nonlinear finite element analysis, and checking methods are provided in the Standard that allow these powerful methods of analysis to be employed in design in appropriate circumstances.

Clause 2.2.2 deals with the normal “ultimate strength” method that has been in use for many years, and which requires a linear elastic analysis of the structure. The checking procedure in Clause 2.2.3 is to be used when a linear elastic stress analysis is undertaken using linear finite elements. Clause 2.2.4 gives details of the strength checking method to be employed when strut-and-tie methods are used for design. Clauses 2.2.5 and 2.2.6 give the checking procedures to be adopted when nonlinear methods of analysis are used, including, for example, a nonlinear finite element analysis.

It is important to note that the design checks covered in Clauses 2.2.2, 2.2.3 and 2.2.4 use characteristic values of both the concrete compressive strength and the steel yield stress. The first two checking procedures apply to analytic methods that, in effect, assume linear material behaviour in the structure and rely on elastic analysis and limit states design concepts, even when some redistribution of the elastically determined stress resultants is assumed in the design. The situation is similar for the checking method to be used with strut-and-tie methods of analysis. The values for the appropriate capacity reduction factors (ϕ) given in Clauses 2.2.2, 2.2.3 and 2.2.4 are generally quite similar. For example, a value of about 0.65 applies in situations where failure is triggered by concrete in compression, or the failure of the member is non-ductile. A value of 0.85 applies for ductile failures, such as flexural failure of an under-reinforced member constructed with Class N steel in tension.

The check procedures and the values of the reduction factors are rather different in Clauses 2.2.5 and 2.2.6, which apply when nonlinear methods of analysis are used for design and nonlinear structural behaviour is assumed to occur at high overload. In these cases, mean values for the material properties

and strengths have to be used in the design check calculations, and not the characteristic values. This is because a single global safety factor has to be used when nonlinear relationships exist between the overall system behaviour and the individual material properties. In the linear design methods, the use of characteristic values for the material strengths effectively introduces partial safety factors for each material.

If nonlinear behaviour becomes a matter of concern in a special structure, then a sensitivity study should be undertaken, in which analyses are carried out for a range of different input values of the material properties and strengths. In order to achieve comparable global factors of safety against failure for all the available methods, the values of the reduction factors used in Clauses 2.2.5 and 2.2.6 are somewhat lower than the values used for the linear methods.

It is important to note that the nomenclature and notation for the specific reduction factors vary according to the checking method. In particular, the notation used in Clause 2.2 for the various reduction factors is as follows:

- (a) ϕ is the capacity reduction factor to be used in the normal ultimate strength design method specified in Clause 2.2.2.
- (b) ϕ_s is the stress reduction factor to be used for design based on stress analysis, as specified in Clause 2.2.3.
- (c) ϕ_{st} is the strength reduction factor to be applied to the strength of struts and ties in the strut-and-tie design method specified in Clause 2.2.4.
- (d) ϕ_{sys} is the system strength reduction factor to be used in the collapse load design method specified in Clauses 2.2.5 and 2.2.6.

The general terminology and notation of AS/NZS 1170.0 (Ref. 2) are used throughout Clause 2.2, where the term “design action” refers not only to design loads, but also to any other actions that may have to be considered by the designer, such as imposed deformations, temperature gradients and the like. The design action to be used in the design calculations will be the most severe combination of factored actions, as defined in AS/NZS 1170.0 (Ref. 2).

The term “design action effect” refers typically to the internal stress resultants, such as moments, shears, etc., that are induced in critical regions of the structure by the design actions, but may also refer to other relevant effects, such as stresses or strains or deformations.

C2.2.1 General

The first paragraph of Clause 2.2.1 specifies strength design be carried out using the strength checking procedures specified in Clauses 2.2.2 to 2.2.6 and the methods of structural analysis specified in Section 6. The second paragraph permits designers to use the most appropriate strength checking procedure for each member, without having to use the same method for other members within the structure and for the structure as a whole. Nevertheless, the design has to satisfy a general check on equilibrium for the structure and its components, as well as for compatibility.

C2.2.2 Strength check procedure for use with linear elastic methods of analysis, with simplified analysis methods and for statically determinate structures

Clause 2.2.2 is based on checking the strength of the member at critical, nominated cross-sections. This is the normal ultimate strength design method and will be used for most routine design. The method is essentially unchanged from previous editions of the Standard and the terminology and notation conform to AS/NZS 1170.0. The general statement of the strength design requirement is:

$$\phi R_u \geq E_d$$

The symbol R_u is the ultimate strength of the section (e.g. the bending strength of a beam section) and E_d is the design action effect induced by the strength design load condition (e.g. the design ultimate bending moment in the section). The factor ϕ is the capacity reduction factor. In this strength check, the characteristic material strength properties have to be used to determine the strengths of cross-sections and local regions.

The strength check thus consists of comparing the design ultimate strength of the cross-section or local region of the member with the design action effect, calculated from the design ultimate load condition by elastic analysis (or an equivalent simplified method) of the overall structure.

In the specific case of a check for moment capacity of a beam section, for example, the design check becomes —

$$\phi M_u \geq M^*$$

where

M^* = moment calculated to act at the section when the structure is subjected to the strength design action combination as specified in AS/NZS 1170.0

The elastic analysis used to evaluate M^* is carried out in accordance with Section 6. The section ultimate strength in bending (M_u) is determined using ultimate strength theory in accordance with Section 8.

The capacity reduction factor (ϕ) takes the following into account:

- (a) Variations in material strength, material properties, position of reinforcing and prestressing steel, dimensions of members and homogeneity.
- (b) Differences between the ultimate strength obtained from tests and the ultimate strength of the member in the structure.
- (c) Inaccuracies in the design equations related to local member behaviour and overall structural behaviour.
- (d) Degree of ductility in relation to the action effects being considered.
- (e) Importance of the member in the structure and the consequences of its failure.

The values of the capacity reduction factors (ϕ) given in Table 2.2.2 have been modified from those in the 2009 edition of the Standard based on recent research (Refs. 12, 13 and 14). In most instances, the factor ϕ is assigned a single value between 0.65 and 0.85, depending on the type of action effect and the Ductility Class (N or L) of the longitudinal tensile reinforcement.

The behaviour of concrete members subjected to axial force without bending [row (a) of Table 2.2.2] is assumed to be ductile if they are subjected to tension and contain Ductility Class N longitudinal tensile reinforcement and/or prestressing tendons ($\phi = 0.85$), but non-ductile if they are subjected to tension and contain Ductility Class L longitudinal tensile reinforcement ($\phi = 0.65$), or if the members are subjected to pure axial compression ($\phi = 0.65$).

Members in bending without axial tension or compression and containing Class N longitudinal reinforcement and/or tendons [row (b)(i)] are assumed to behave in a ductile manner, provided the neutral axis parameter (k_{uo}) does not exceed 0.36. In this case, $\phi = 0.85$. More heavily reinforced sections with larger values of k_{uo} are considered to be less ductile, and ϕ decreases linearly from 0.85 to 0.65 as k_{uo} increases from 0.36 to 0.545. When k_{uo} exceeds 0.545, $\phi = 0.65$. Balanced failure occurs when Grade 500 MPa steel reinforcement is used and the value of $k_{uo} = \varepsilon_{cu}/(\varepsilon_{cu} + \varepsilon_{sy}) = 0.545$, with concrete extreme fibre compressive strain, $\varepsilon_{cu} = 0.003$ and the tensile steel strain at depth d_o is the yield strain $\varepsilon_{sy} = 0.0025$. The corresponding curvature at such a cracked section is $0.0055/d_o$. The ductility limit of $k_{uo} = 0.36$ corresponds to a curvature at the ultimate limit state of $0.00833/d_o$ (i.e. 51.5 % higher than that at balanced failure).

For under-reinforced sections in bending without axial tension or compression and containing Ductility Class L tensile reinforcement [row (b)(ii)], the tensile reinforcement can fracture at an ultimate curvature less than that reached using Ductility Class N reinforcement, and therefore sections are assumed to be non-ductile. The maximum value of ϕ is reduced from 0.85 to 0.65 for cross-sections containing Ductility Class L longitudinal tensile reinforcement.

For sections subjected to bending combined with axial force and containing Ductility Class N reinforcement and/or tendons, the relevant values of ϕ are given in rows (c)(i), d(i) and (d)(ii), where ϕ varies from 0.65 to 0.85. For sections subjected to bending combined with axial force and containing Ductility Class L reinforcement, the relevant values of ϕ is 0.65.

The note to the table addresses the design of members involving mixed construction where Ductility Class L reinforcement (mesh) together with Ductility Class N reinforcement (bars) and/or prestressing tendons are used as longitudinal tensile reinforcement in the design for strength in bending, with or without axial force present. It is recommended that the maximum value of ϕ for calculating the member design strength be limited by the presence of the Ductility Class L reinforcement and taken as 0.65.

Shear and torsion in reinforced members attract an intermediate value of $\phi = 0.75$ for members containing Ductility Class N fitments (and the shear strength is not limited by web crushing), otherwise $\phi = 0.7$. Sections in bearing are treated as non-ductile with a value of $\phi = 0.6$, as are all plain concrete members and fixings subjected to bending, shear and axial force. For singly reinforced walls forming part of a primary lateral load system, $\phi = 0.65$.

C2.2.3 Strength check procedure for use with linear elastic stress analysis

This check procedure allows members or regions to be designed on the basis of critical stresses, calculated by elastic analysis using, for example, linear finite elements. Design situations are thus catered for where it is not possible to use the simple ultimate strength design method detailed in Clause 2.2.2. This can occur, for instance, in the design of a complex three-dimensional region of a structure, such as a joint, where it is not possible to identify simple cross-sections (such as exist in a beam or column) and the design actions acting within the disturbed region.

The maximum stresses are determined from an elastic stress analysis assuming an uncracked structure. The analysis is carried out for the critical combination of the design loads for strength.

The calculated maximum compressive stresses in the concrete are not to exceed the limiting stress $\phi_s \beta 0.9 f'_c$ as specified in Clause 2.2.3(b). The symbol β is a factor to be applied to the compressive strength and is described below while ϕ_s is a stress reduction factor whose values are given in Table 2.2.3. In this strength design check, the stress reduction factor ϕ_s plays a comparable role to that of the strength reduction factor ϕ in design based on elastic analysis, as previously discussed in C2.2.2.

With regard to the calculated tensile stresses, reinforcement or tendons need to be included to carry all of the internal tensile forces, calculated by integrating the tensile stresses. In determining the required steel areas, the value chosen for the steel stress is not to exceed the yield stress (f_{sy} or f_{py}) reduced by the stress reduction factor ϕ_s . Some averaging of the tensile stresses is allowed by Clause 2.2.3(d) over an area “appropriate” to the size of the region being considered. The size of the area over which tensile stresses are to be averaged is a matter of engineering judgement and is dependent on the geometry of the region and the nature of the stress distribution. No hard and fast rules can be provided.

As an elastic analysis does not allow for assessment of strains normal or transverse to the direction of principal compressive stresses, an efficiency factor β is applied to the allowable compressive stress to account for the disturbing effect of transverse tensile strains on the compressive strength of concrete. This effect was documented as early as 1977 by Robinson and Demorieux (Ref. 15) but was first quantified, and is best known, from the publications of Vecchio and Collins (Ref. 16 and 17) who first proposed the modified compression field theory (MCFT).

While there are a number of variants of the efficiency factor, the model proposed by Collins and Mitchell (Ref. 18) has generally withstood the test of time and has been incorporated into the Canadian Code of Practice, *Design of Concrete Structures for Buildings*, since 1984 (Ref. 19). Based on the panel tests of Vecchio and Collins (Ref. 15), Collins and Mitchell proposed that —

$$\beta = \frac{1}{0.8 + 170\varepsilon_1}$$

where

$$\varepsilon_1 = \text{major principal strain normal to the direction of the compression field}$$

A plot of the MCFT against test data are shown in [Figure C2.2.3](#) [labelled Vecchio and Collins 1986 (Ref. 16)], together with other alternative predictions.

When adopting a plastic design procedure on top of an elastic analysis, the transverse strain is required to be sufficiently large for the ductility demands of the structure or member to be met. The factor β takes account of the effect of transverse strain on the strength of the concrete or, if the element is in a tri-axial compressive state, the effect of any confinement on the concrete strength.

The value of $\beta = 0.6$ is calibrated for a maximum angle of crack rotation from the angle at the initial cracking (elastic) condition to that at the ultimate (plastic) condition of $\pm 15^\circ$. If the rotation of the cracks is greater than 15° , the factor $\beta = 0.6$ may be unconservative. A more refined evaluation of the parameter β is given in the *fib* Model Code (Ref. 20) as follows:

- (a) If no reinforcement has yielded and at least one principal stress is tensile —

$$\beta = \frac{1.18}{1.14 + 0.00166\sigma_{si}} \leq 1.0$$

where

$$\sigma_{si} = \text{maximum tensile stress (in MPa) in any layer of reinforcing steel}$$

- (b) If one or more layers of reinforcement has yielded —

$$\beta = 1 - 0.032|\theta_{pl} - \theta_{el}| \pm \frac{1.18}{1.14 + 0.00166f_{sy}}$$

where

$$\theta_{pl} = \text{compression field angle with respect to x-axis at the ultimate limit state}$$

$$\theta_{el} = \text{first cracking angle with respect to the x-axis}$$

In a two dimensional model, if both principal stresses are compressive, β may be taken as 1.0 or determined using a refined model based on the bi-axial stress condition.

Clause 2.2.3(b)(ii) allows for cases where confinement reinforcement is used locally to strengthen the concrete in compression as, for example, the concrete behind a prestressing anchoring plate where local stresses may be high. In this case, the effective confining pressure ($f_{r,eff}$) may be calculated using the model provided in Clause 10.7.3.3 [refer to Equation 10.7.3.3(5)] and the axial stress obtained from a modified form of the equation proposed by Richart et al. (Ref. 21) —

$$f_o = 0.9f'_c + Cf_{r,eff}$$

$$\beta = \frac{f_o}{0.9f'_c} \leq 2 \quad \text{or} \quad \beta = 1 + \frac{Cf_{r,\text{eff}}}{0.9f'_c} \leq 2$$

where

$$C = \begin{cases} 4 & \text{for } f'_c \leq 80 \text{ MPa} \\ 3 & \text{for } f'_c > 80 \text{ MPa (Ref. 21)} \end{cases}$$

Other suitable models for determining the efficiency of confinement and the effective confining pressure provided by confining reinforcement include those of Sheikh and Uzumeri (Ref. 23) and Mander et al. (Ref. 24).

Further information may be obtained from Foster and Marti (Ref. 25), Foster et al. (Ref. 26) and *fib* Bulletin 45 (Ref. 27).

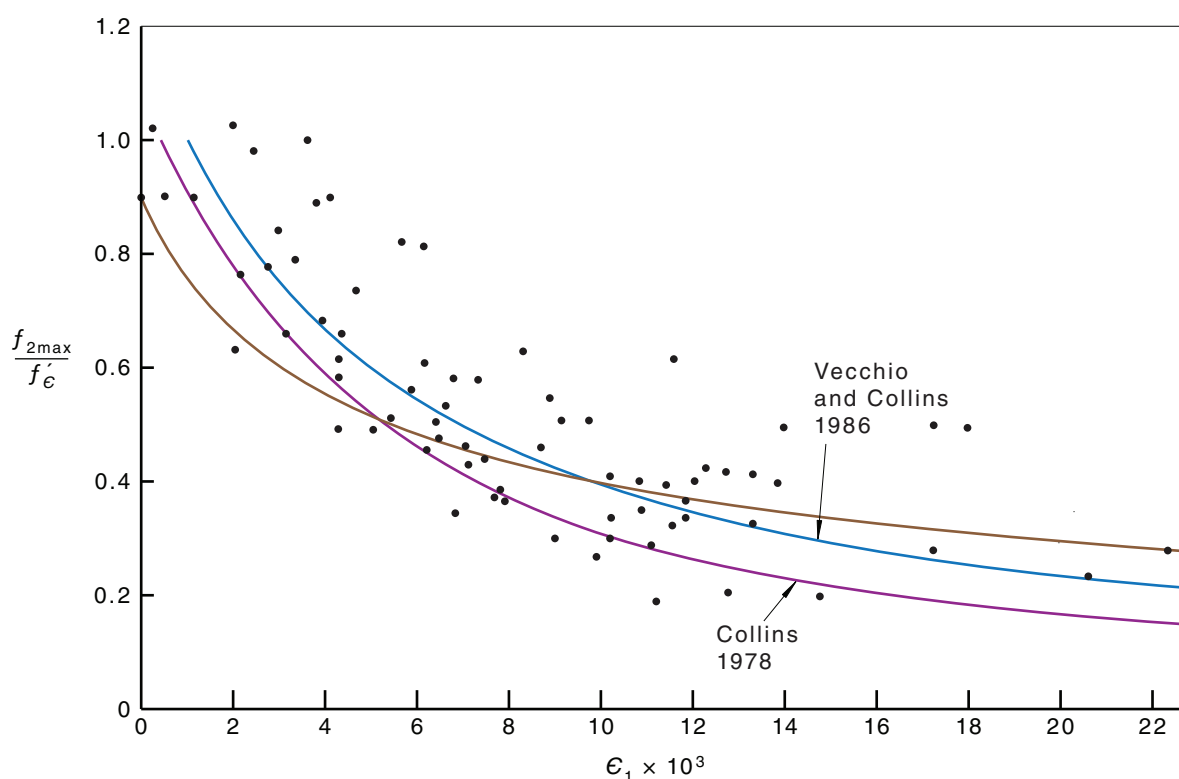


Figure C2.2.3 — Influence of transverse tensile strain on compressive strength (Ref. 16)

C2.2.4 Strength check procedure for use with strut-and-tie analysis

Clause 2.2.4 allows designers to use analysis and design methods based on the strut-and-tie approach, according to the provisions given in Clause 6.8, Section 7 and Clause 12.2. Designers should consult the commentary to these provisions for a detailed discussion on the background to each clause and the methodology. Clause 2.2.4 specifies how the design check is to be carried out when strut-and-tie analysis is used to check the design strength of the steel ties, the concrete struts and the concrete nodes within a member or a region of a structure. The strength reduction factor ϕ_{st} is applied both to the forces in the struts and ties and to the stresses in the nodes.

C2.2.5 Strength check procedure for use with nonlinear analysis of framed structures

The strength checking procedure given in Clause 2.2.5 allows design to be based on a collapse load analysis. This is quite different to the ultimate strength design procedure covered in Clause 2.2.2. In ultimate strength design, the structure is assumed to behave elastically in order to evaluate the stress resultants (i.e. the action effects) such as the internal moments, shears, torsions and axial forces at individual cross-sections of beams, columns and slabs. The ultimate strengths of the individual cross-sections are then determined on the assumption of local inelastic behaviour. In collapse load design, a nonlinear analysis is undertaken for the entire structure in order to calculate the load at which the structure will collapse ($R_{u.sys}$). The design load capacity of the entire structure is then compared directly with the critical design ultimate load combination. A system strength reduction factor ϕ_{sys} is applied to the calculated load capacity to obtain the design load capacity. The design check therefore becomes —

$$\phi_{sys} R_{u.sys} \geq E_d$$

where

E_d = design action effect for the critical load combination

It is important to note that in the nonlinear analysis employed to evaluate the load capacity $R_{u.sys}$, mean values (not characteristic values) of the material properties are used. Therefore values for the system reduction factor are somewhat different to the values of the capacity reduction factor specified in Clauses 2.2.2 to 2.2.4, where characteristic values of material properties are used in design.

C2.2.6 Strength check procedure for use with nonlinear stress analysis

The strength checking procedure given in Clause 2.2.6 allows design to be based on a nonlinear stress analysis using, for example, nonlinear finite elements. If the analysis takes full account of the nonlinear behaviour of the materials at collapse, as well as any geometric nonlinear effects, then this analysis, similar to the one dealt with in Clause 2.2.5, provides a detailed treatment of the conditions at failure of the structural system and delivers a value of the collapse load. The design check specified in Clause 2.2.6 is similar to that in Clause 2.2.5. The values for the system (capacity) reduction factor, given in Table 2.2.5, are also applicable.

C2.3 Design for serviceability

C2.3.1 General

In order to satisfy the design requirements relating to serviceability, a concrete structure needs to be serviceable and perform its intended function throughout its working life. Unlike strength, design for serviceability is not covered in the NCC regulations. This places greater responsibility on designers as, together with the owner/user of the facility, they have to identify the appropriate serviceability criteria to be met. Further, at the design/planning stage, it needs to be recognized that meeting the relevant criteria will frequently involve controlling the construction techniques and practices as well as carrying out particular design procedures. In some instances, the serviceability limit states can be the controlling criteria. For example, no cracking permitted in the walls, floors and roofs of a facility used for testing of pathogenic organisms. In other cases, the required performance criteria can be equally demanding. For example, limiting surface flatness in specialist warehouses, limiting vibrations in hospital wards, limiting deflections in the floors of a manufacturing facility and controlling surface defects, such as cracking and crazing, in monumental-type structures.

In general, design for serviceability is covered by AS/NZS 1170.0, which applies to structures constructed from all materials. Additional criteria, such as the criteria provided in Clauses 2.3.2, 2.3.3 and 2.3.4, are applicable to concrete structures or members, although some guidance is also provided on matters that have wider application (e.g. deflections likely to cause problems with the ponding of water). The limits or values quoted have been derived from experience and observation of behaviour of typical concrete building structures in the past. Consequently, whether or not the limits are appropriate for current buildings and practices is a matter of engineering judgement based on experience, and should be evaluated in terms of the particular structure under review.

Excessive deflection and/or excessive cracking should not impair the function of the structure or be aesthetically unacceptable. Excessive deflection should also not cause unintended load paths, such as occurs when a deflecting slab begins to bear on a non-loadbearing partition. Cracks should not be unsightly or wide enough to lead to durability problems. In addition, vibration should not cause distress to the structure or discomfort to its occupants. If required, other serviceability design criteria should be considered.

Design checks for the serviceability limit states involve appropriate consideration of the time-dependent deformation of the structure, including the effects of cracking, tension stiffening, creep and shrinkage of the concrete.

C2.3.2 Deflection

Methods for the calculation of deflection have been subject to frequent refinement over the past 50 years. Nevertheless, the provisions in the Standard have limited precision. It can be expected that evaluation of experience gained from structures built to conform to the Standard will lead to further amendments in the future.

The mechanisms that govern deflection of concrete members need to be understood by designers and contractors. Final deflections consist of an elastic component due to the application of the permanent and applied actions (sustained dead and live loads), a shrinkage component that develops progressively as the concrete loses moisture, and a creep component that develops as the concrete matrix gradually deforms and adjusts to the sustained “actions”. All of these components depend significantly on the extent of cracking, including early age thermal cracking, cracking caused by construction loads, restrained shrinkage cracking and cracking caused by external loads. The situation is further complicated by the fact that the properties of the concrete change over time. For example, strength and elastic modulus tend to increase with time, as do the magnitudes of creep and shrinkage strain, but the rates of shrinkage and creep tend to diminish with time. Information on the changes in concrete properties with time is provided in Section 3 and this document.

In general, the later the actions are applied to a concrete member, the smaller will be its deflections. Mature concrete is stronger and stiffer and the time-dependent deformations are less.

There are three main types of deflection problems that may affect the serviceability of a concrete structure (Ref. 28):

- (a) Type 1 — where excessive deflection causes either aesthetic or functional problems.
- (b) Type 2 — where excessive deflection results in unintended load paths or damage to either structural or non-structural elements attached to the member.
- (c) Type 3 — where dynamic effects due to insufficient stiffness cause discomfort to occupants.

Examples of deflection problems of Type 1 include visually unacceptable sagging (or hogging) of slabs and beams, and ponding of water on roofs. Type 1 problems are generally overcome by limiting the magnitude of the final long-term deflection (referred to as the total deflection in Table 2.3.2) to some appropriately low value. The total deflection of a beam or slab in a building is usually the sum of the short-term and time-dependent deflections caused by the permanent (dead) load (including self-weight), the prestress (if any), the expected in-service live load and the load-independent effects of shrinkage and temperature change.

A total deflection limit that is appropriate for the particular member and its intended function has to be selected by the designer. The selected limit should be appropriate for the structure and its intended use, but in no case is the limit to be greater than $\text{span}/250$ for a span supported at both ends and $\text{span}/125$ for a cantilever (refer to Table 2.3.2). For example, a total deflection limit of $\text{span}/250$ may be appropriate for the floor of a car park, but would be totally inadequate for a gymnasium floor that is required to remain essentially plane under service conditions and where functional problems arise at very small total deflections. In some situations, the limit of $\text{span}/250$ may not be sufficient to meet particular aesthetic or functional requirements such as visual sagging and ponding of water.

Examples of Type 2 problems include deflection-induced damage to ceiling or floor finishes, cracking of masonry walls and other brittle partitions, improper functioning of sliding windows and doors, tilting of storage racking and so on. To avoid these problems, a limit has to be placed on that part of the total deflection that occurs after the attachment of the non-structural elements in question (i.e. the incremental deflection). This incremental deflection is the sum of the long-term deflection due to all the sustained loads and shrinkage, the short-term deflection due to the transitory live load and the short-term deflection due to any dead load applied to the structure after the attachment of the non-structural elements under consideration, together with any temperature-induced deflection.

For roof or floor construction supporting masonry partitions, the incremental deflection that occurs after the attachment of the partitions should not exceed $\text{span}/500$ ($\text{span}/250$ for a cantilever) when provision is made to minimize the effect of deflection on the partition. When no such provision is made, the incremental deflection should be limited to $\text{span}/1000$ ($\text{span}/500$ for a cantilever). For members supporting other brittle finishes (or attached to other non-structural elements that are likely to be damaged by large deflection), the incremental deflection that occurs after the attachment of the brittle finish should not exceed $\text{span}/500$ ($\text{span}/250$ for a cantilever) or the limit specified by the manufacturer whichever is the more severe. For members supporting brittle partitions or finishes, this criterion will usually be more severe than the limit on total deflection, unless provision is made to minimize the effect of the movement by detailing joints and delaying the erection of the brittle elements until a substantial part of the long-term deflection has occurred. This latter precaution may not be practical if speed and flexibility of construction are desired.

Type 3 deflection problems include the perceptible springy vertical motion of floor systems and other vibration-related problems. Very little quantitative information for controlling this type of deflection problem is available. For a member subjected to vehicular or pedestrian traffic, a maximum limit of $\text{span}/800$ ($\text{span}/400$ for a cantilever) is imposed on the immediate deflection caused by imposed loads (including live load and dynamic impact). This limit provides a minimum requirement on the instantaneous stiffness of the member that is deemed to be sufficient to avoid Type 3 problems. Such problems are potentially the most common for prestressed concrete floors, where load balancing is often employed to produce a nearly horizontal floor under the sustained load and the bulk of the final deflection may be due to the transient live load. As such structures are often uncracked at service loads, the total deflection is small and Types 1 and 2 deflection problems are easily avoided.

The total deflection is measured from the as-cast position. The Standard does not provide specific guidance on the treatment of camber that could be used to eliminate the effect of part or all of the total deflection and possibly permit more slender members, particularly for longer spans. However, if camber is used to meet the total deflection limit and the stiffness of the member is thereby reduced, then care should be taken to check the incremental deflection, the imposed action deflection, the support rotations, and the possibility of excessive vibration.

Appropriate methods for deflection control are specified in the Standard as follows:

- | | | |
|-------|---|--------------|
| (i) | Beam deflection by refined calculation | Clause 8.5.2 |
| (ii) | Beam deflection by simplified calculation | Clause 8.5.3 |
| (iii) | Deemed to conform span-to-depth ratios for reinforced beams | Clause 8.5.4 |
| (iv) | Slab deflection by refined calculation | Clause 9.4.2 |

- | | | |
|-------|---|-----------------|
| (v) | Slab deflection by simplified calculation | Clause 9.4.3 |
| (vi) | Deemed to conform span-to-depth ratios for reinforced slabs | Clause 9.4.4 |
| (vii) | For steel fibre reinforced concrete beams and slabs | Clause 16.4.7.3 |

Deflections due to lateral loading may be the critical serviceability consideration for unbraced frames in multistorey buildings. The maximum limit on the inter-storey lateral drift of 1/500 of the storey height is intended to provide appropriate stiffness and serviceability for most multistorey buildings and unbraced framed structures under the design lateral loads for serviceability.

C2.3.3 Cracking

C2.3.3.1 General

The control of cracking under service conditions is required for aesthetics, as well as for durability. Cracks should not adversely affect the appearance of the structure or be wide enough to facilitate the corrosion of reinforcement. In the Standard, crack control may be deemed to be satisfied by conforming to certain minimum reinforcement requirements, appropriate detailing of the reinforcement, and in the case of flexural cracking, by limiting the stress in the reinforcement crossing the crack to some appropriately low value. The limiting steel stress depends on the maximum acceptable crack width for the structure and that, in turn, depends on the structural requirements and the local environment. Alternatively, the Standard permits the control of cracking by ensuring that the calculated maximum crack width under in-service conditions is less than the characteristic maximum design crack width $w'_{\max}$ selected for the particular concrete surface under consideration.

The deemed-to-satisfy provisions for crack control in the Standard are intended to apply when the characteristic maximum design crack width $w'_{\max}$ is greater than or equal to 0.2 mm but less than or equal to 0.7 mm. This range of crack widths is thought to be appropriate for most structures, but smaller limits may be required for structures in aggressive environments (e.g. marine structures), for liquid-retaining structures, and for monumental structures requiring an exceptionally high quality finish. Where a concrete surface is located in a sheltered environment and cracking will not be visible, a characteristic maximum design crack width of up to 0.7 mm may be acceptable. A guide to the control of cracking is available in Ref. 29.

Some recommended upper limits on maximum crack widths for serviceable reinforced concrete structures are given in [Table C2.3.3.1](#).

These recommended crack widths are consistent with a design life of 50 years ± 20 %. For different design lives, these crack widths do not apply.

Table C2.3.3.1 — Recommended maximum final design crack widths (Ref. 28)

Environment	Design requirement	Maximum final design crack width ($w'_{\max}$), mm
Sheltered environment (where crack widths will not adversely affect durability)	Aesthetic requirement (a) where cracking could adversely affect the appearance of the structure (i) distant in buildings (ii) close in buildings (iii) where cracking will not be visible and aesthetics are not important	 0.5 0.3 0.7
Exposed environment	Durability requirement (b) where wide cracks could lead to corrosion of reinforcement Aesthetic requirement (c) monumental structure requiring high quality finish	 0.3 0.2
Aggressive environment	Durability requirement (d) where wide cracks could lead to corrosion of reinforcement	 0.30 (when $c^{\#} \geq 50$ mm) 0.25 (otherwise)
NOTE $c^{\#}$ is the concrete cover to the nearest steel reinforcement.		

C2.3.3.2 Control of cracking

The deemed-to-satisfy design requirements for crack control as outlined elsewhere in the Standard, are referred to here.

The requirements for control of load-induced cracking are as follows:

- | | | |
|-----|--|----------------------------|
| (a) | For reinforced concrete beams in tension and bending | Clause 8.6.2 |
| (b) | For prestressed concrete beams in bending | Clause 8.6.3 |
| (c) | In the side face of beams | Clause 8.6.4 |
| (d) | At openings and discontinuities in beams | Clause 8.6.5 |
| (e) | For flexure in reinforced concrete slabs | Clause 9.5.2.1 and 9.5.2.2 |
| (f) | For flexure in prestressed slabs | Clause 9.5.2.3 |
| (g) | For steel fibre reinforced beams and slabs | Clause 16.4.7.4 |

The requirements for the control of shrinkage and temperature-induced cracking are as follows:

- | | | |
|-------|---|----------------|
| (i) | General requirements for slabs | Clause 9.5.3.1 |
| (ii) | Reinforcement in the primary direction of a slab | Clause 9.5.3.2 |
| (iii) | Reinforcement in the secondary direction of an unrestrained slab | Clause 9.5.3.3 |
| (iv) | Reinforcement in the secondary direction of a restrained slab | Clause 9.5.3.4 |
| (v) | Reinforcement in the secondary direction of a partially restrained slab | Clause 9.5.3.5 |

- | | | |
|--------|---------------------------------|---------------|
| (vi) | In the vicinity of restraints | Clause 9.5.4 |
| (vii) | At openings and discontinuities | Clause 9.5.5 |
| (viii) | In restrained walls | Clause 11.7.2 |

The detailing requirements for crack control in walls are specified in Clauses 11.7.1 to 11.7.4, while the crack control provisions for non-flexural members and in the disturbed regions in the vicinity of concentrated loads are given in Clause 12.7.

The control of plastic shrinkage cracking, settlement cracking and early thermal cracking in the pre-hardened concrete is outside the scope of the Standard. However, it is recommended plastic shrinkage and early thermal cracking be adequately controlled by appropriate specification of the concrete and appropriate measures taken during construction to control the evaporation of bleed water and the heat of hydration. Early thermal cracking is likely in a concrete member if it has a cross-section greater than 400 mm × 400 mm and a thermal gradient of greater than 20 °C occurs between the core and the surface of the member. A series of preventative/controlling measures are possible, ranging from adjusting the materials used, specifying suitable concrete mix proportions, using insulated formwork and using reinforcement/fibres.

C2.3.4 Vibration

Clause 2.3.4 requires that designers consider vibration in concrete structures and members. Generally, vibration is likely to be a controlling consideration where —

- (a) rhythmic loads are generated by people jumping in unison (e.g. in gymnasias or grandstands);
- (b) mechanical plant imparts vibrations to the structure;
- (c) vibrations are generated by people walking in buildings, such as hospitals, where the occupants may be especially sensitive to vibration; or
- (d) in tall structures subjected to dynamic wind loading.

The design of structures subject to dynamic loads, such that the vibrations generated do not exceed acceptable levels, is a complex subject. References 30 to 42 provide an introduction to the specialist literature in this field. Essential to the solution is a detailed understanding of both the magnitude and nature of the applied dynamic loads (harmonic, transient, or random force) and the magnitude of the mass, stiffness and damping characteristics of the structure, as well as the acceptability criteria relevant to the type of structure under consideration.

Among the dynamic loads that may require consideration for the serviceability limit state are —

- (i) pedestrian traffic on suspended floors and footbridges (Refs. 30 to 42);
- (ii) wind loads on structure (Refs. 43 and 44);
- (iii) service loads due to plant and manufacturing processes (e.g. forging hammers, generators and presses) (Refs. 45 to 49);
- (iv) adjacent road or rail traffic (Refs. 50 and 51); and
- (v) blasting (assuming a nearby quarry) (Refs. 52 and 53).

Where a structure supports vibrating machinery (or any other significant dynamic load) or where a structure may be subjected to ground motion caused by earthquake, blast or adjacent road or rail traffic, vibration control becomes an important design requirement. This is particularly so for slender structures, such as tall buildings or long-span beams or slabs.

Vibration is best controlled by isolating the structure from the source of vibration. Where this is not possible, vibration may be controlled by limiting the frequency of the fundamental mode of vibration

of the structure to a value that is significantly different from the frequency of the source of vibration. When a structure is subjected only to pedestrian traffic, 5 Hz is often taken as the minimum frequency of the fundamental mode of vibration of a beam or slab and methods for vibration analysis are described in Refs. 36 to 40.

The designer should use the appropriate acceptability criteria in judging the adequacy of the design. AS 2670.1 (Ref. 54) and AS 2670.2 (Ref. 55) provide some guidance. The difficulties of obtaining precise information on the dynamic loads, and of finding relevant acceptability criteria, mean that the designer is unlikely to obtain solutions with the same degree of confidence that applies to static loading situations.

C2.4 Design for fatigue

Clause 2.4 allows for concrete structures to be designed for fatigue in accordance with the new Section 18. Designers should consult the commentary in [Section C18](#) for a detailed discussion on the background to each clause and the methodology. Table 2.4 specifies strength reduction factors to be used when assessing the fatigue strength of concrete and steel reinforcement and tendons ($\phi_{c.fat}$ and $\phi_{s.fat}$, respectively).

Fatigue design is a strength limit state assessed at operational stress levels. For this reason, the fatigue strength of material needs to be reduced to achieved acceptable structural reliability. The factors in Table 2.4 are consistent with other strength reduction factors for concrete and steel within this section. The factors are equivalent to the inverse of the material reduction factors in other international standards and codes of practice, and compatible with ISO 2394 and ISO 22111.

C2.5 Actions and combinations of actions

C2.5.1 Actions and loads

The nomenclature for loads and loading is consistent with that in AS/NZS 1170.0 (Ref. 2) and follows that adopted by ISO. The actions (loads) to be used for the design of concrete members are set out in AS/NZS 1170.1 (Ref. 3).

Accidental loading includes collisions, explosions, subsidence of subgrades, extreme erosion and cyclonic storms in regions not normally exposed to them.

The deemed-to-satisfy wind loads in AS 4055 (Ref. 55) may be used for housing where the limitations of that standard apply. However, there may be advantages in using the more rigorous wind load procedures from AS/NZS 1170.2 (Ref. 4), especially for larger housing estates. Structural design actions for earthquakes in Australia are specified in AS 1170.4 (Ref. 4).

C2.5.2 Combinations of actions and loads

Generally, the combinations of actions (loads) used for the design of concrete members are set out in AS/NZS 1170.0. However, it does not prescribe specific combinations for serviceability and the designer should consult the commentary in AS/NZS 1170.0 Supp 1 for guidance.

AS/NZS 1170.0 makes reference to the individual material standards for the combinations to be used when a member is prestressed. Therefore these combinations are specified in Clause 2.5.2.2. The load combinations to be used in fatigue design are outlined in Table 2.5.2.3(A). The load factor applied to the prestressing force P in these load combinations, termed the representative value factor γ_p , is given in Table 2.5.2.3(B).

C2.5.3 Construction effects

Designers should be aware that the way a concrete structure, especially a multistorey building, is propped and stripped will have a significant effect on the loads generated in the members during construction and also a major influence on the deflected shape of the structure. Precast members

that are subsequently made composite with an *in situ* slab or topping are frequently required to carry construction loads (including the weight of the wet concrete of the slab or topping) before composite action is established. This may have a significant effect on deflection calculations at service loads, in addition to strength implications. Some guidance on construction loads is provided in AS 3610 (Ref. 57), with a minimum of 1.0 kPa to be used for personnel and equipment and 4.0 kPa for stacked materials. AS 3610 also emphasizes the need for careful assessment of the loads induced by floor-to-floor propping in multistorey construction (see also [Clause C17.6.2](#)).

C2.5.4 Arrangements of vertical loads on continuous beams, frames and floor systems

Clause 2.5.4 applies both to strength design and serviceability design. It draws attention to the difficulty faced by designers because of the many different patterns of vertical loads that may have to be considered in order to determine the peak design action effects (maximum design moments, shears, etc.) in critical sections and regions of a complex structural system.

It is important to understand that the clause gives a list of only the minimum necessary load patterns that have to be considered. Other load patterns may need to be considered depending on the nature of the applied loading and the layout of the structure. The key to interpreting Clause 2.5.4 correctly is contained in the lead-in sentence to Clause 2.5.4(a), (b) and (c), which states clearly and unambiguously: “The loading arrangements to be considered shall include at least the following”. This interpretation is reinforced by the note at the end of the clause, which states: “The load arrangements listed are the minimum to be considered for design ...”

In a three-dimensional structure with multiple spans in both directions and with multiple floors, a consideration of the all-spans-loaded pattern and the alternative-spans-loaded (chequerboard) pattern is always necessary, but these two patterns are not necessarily critical and many others may have to be investigated in order to identify the critical pattern that produces peak action effects in any particular region.

For dead loads, Clause 2.5.4(a) refers only to the case of all spans loaded, but states that this has to be considered as a minimum. Other more critical situations may arise, for example during construction, and will need to be investigated as necessary.

Clause 2.5.4(b) applies to the simple but unusual situation in design where, for live load, the pattern of loaded and unloaded spans is invariable.

Clause 2.5.4(c) deals with the more usual case of variable live loads acting on either two-dimensional or three-dimensional systems. Notwithstanding the minimum requirements specified in Item (c), in complex design situations, many alternative arrangements of the loads will usually have to be evaluated systematically and exhaustively in order to identify the critical design action effects.

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Section C3 Design properties of materials

Throughout Section 3, the designer is given the option of either —

- (a) using the prescribed value of a particular material property (or material characteristic); or
- (b) determining the particular property or characteristic by testing.

The latter is clearly the more reliable way of determining material properties for use in design, but may not always be the most practical or economical option. The alternative adopted should be appropriate to the desired accuracy.

C3.1 Properties of concrete

The Standard specifies prescribed values of the various concrete properties for concrete with characteristic compressive strengths in the range 20 MPa to 120 MPa.

The prescribed values of the concrete properties are approximate. They are typical values, providing a ballpark estimate that is usually sufficient for most applications. However, the prescribed values may overestimate or underestimate the actual values by as much as 30 %, and designers should consider the consequences of such uncertainty. Where accuracy of the design value selected for a particular material property is important and the consequences of an inaccurate estimate are severe, the option of undertaking tests on concrete that is, as far as possible, the same as that intended to be used in the structure should be considered. Alternatively, better estimates of material properties may be available from historical records of concrete at a specific location or from a particular supplier.

The strength and deformation characteristics of concrete are required in most aspects of structural design. The characteristic compressive strength of concrete is a measure of concrete quality (certainly not the only measure) and plays an important role in the design of structures for durability. While not significantly affecting the ultimate strength in flexure of a beam or slab, the magnitude is more important with regard to strength in shear and torsion and has a much greater influence on the strength of columns and walls in compression. The concrete properties greatly affect the in-service behaviour of structures. In particular, the tensile strength of concrete, the elastic modulus, the creep coefficient and the shrinkage strain all have a significant effect on the deformation of a concrete structure at the serviceability limit states and on the extent and severity of cracking.

The elastic modulus is required in the analysis of structures to estimate the stiffness of each member and to determine the internal actions in an indeterminate structure. It is also required to estimate the instantaneous deformations caused by the internal actions and the stresses induced by imposed deformations. The tensile strength of concrete is required to determine the extent of cracking due to applied load, applied deformation and restraint. The creep coefficient associated with a particular time period and a particular loading regime is required to estimate the time-dependent deformation of the structure, and the magnitude and rate of shrinkage strain is required to predict the development of load-independent deformations with time and the onset of time-dependent cracking.

C3.1.1 Strength

C3.1.1.1 Characteristic compressive strength

As in previous versions of the Standard, the strength of concrete is specified in terms of the lower characteristic compressive cylinder strength at 28 days (f'_c). This is the value of compressive strength exceeded by 95 % of all standard cylinders tested at 28 days after casting and cured under standard laboratory conditions.

The standard strength grades are 20 MPa, 25 MPa, 32 MPa, 40 MPa, 50 MPa, 65 MPa, 80 MPa, 100 MPa and 120 MPa. The normal strength grades (20 MPa to 50 MPa) may be considered as Normal Class Concrete specified only in terms of characteristic compressive strength. The higher strength grades (65 MPa to 120 MPa) are Special Class Concretes requiring additional specification (e.g. workability,

water/binder ratios, types and quantities of admixtures, water-reducing agents, aggregate type and shrinkage requirements).

In lieu of adopting a standard strength grade for the characteristic compressive strength (assuming the concrete is adequately cured and is in accordance with AS 1379), f'_c may be determined statistically from compressive cylinder tests carried out in accordance with AS 1012.9.

Amendment 2 to the Standard added Clause 3.1.1.1(b), which covers the characteristic cube strength of grout at 28 days ($f'_{c.cube}$) and Table 3.1.1.1 was added to facilitate the conversion of standard cube strength test results to cylinder strength test results.

C3.1.1.2 Mean *in situ* compressive strength

In the Standard, the ratio of the mean value of cylinder strength f_{cm} to the lower characteristic strength is taken to be 1.25 for a strength grade of 20 MPa reducing linearly to 1.10 for a strength grade of 100 MPa (i.e. $f_{cm} = (1.2875 - 0.001875f'_c)f'_c$). In the absence of more accurate information, the *in situ* compressive strength of concrete (i.e. the compressive strength of the concrete in the structure on-site) is taken to be 90 % of the cylinder strength. The mean value of cylinder strength (f_{cm}) and the mean *in situ* strength (f_{cmi}) corresponding to the standard strength grades are shown in [Table C3.1.1.2](#).

Table C3.1.1.2 — Mean cylinder and *in situ* concrete compressive strengths at 28 days

Characteristic cylinder strength (f'_c)	Mean cylinder strength (f_{cm})	Mean <i>in situ</i> strength (f_{cmi})
20	25	22
25	31	28
32	39	35
40	48	43
50	59	53
65	75	68
80	91	82
100	110	99
120	128	115

C3.1.1.3 Tensile strength

In a concrete specimen subjected to uniaxial tension, the uniaxial tensile strength (f_{ct}) is the maximum stress that the concrete can withstand prior to cracking. Direct uniaxial tensile tests are difficult to perform and tensile strength is usually measured via either flexural tests on prisms (in accordance with AS 1012.11) or indirect splitting tests on cylinders (in accordance with AS 1012.10). In flexure, the apparent tensile stress at the extreme tensile fibre of the critical cross-section under the peak load is calculated assuming linear elastic behaviour and the calculated value is taken to be the flexural tensile strength (or modulus of rupture), $f_{ct.f}$. The flexural tensile strength ($f_{ct.f}$) is significantly higher than f_{ct} due to the strain gradient and the post-peak unloading portion of the stress-strain curve for concrete in tension, and typically, f_{ct} is about 50 % to 60 % of $f_{ct.f}$. The indirect tensile strength measured from a split cylinder test ($f_{ct.sp}$) is also higher than f_{ct} (usually by about 10 %) due to the confining effect of the bearing plate in the standard test.

The Standard permits f_{ct} to be determined from either the measured values of $f_{ct.f}$ or $f_{ct.sp}$ using the relationships $f_{ct} = 0.6 f_{ct.f}$ or $f_{ct} = 0.9 f_{ct.sp}$.

In the absence of more accurate data from testing and where standard curing is specified, the lower characteristic 28 days flexural tensile strength ($f'_{ct.f}$) and the lower characteristic 28 days uniaxial

tensile strength (f'_{ct}) may be taken for design purposes as $f'_{ct,f} = 0.6\sqrt{f'_c}$ and $f'_{ct,f} = 0.36\sqrt{f'_c}$, respectively.

The mean and upper characteristic values may be estimated by multiplying the lower characteristic values by 1.4 and 1.8, respectively. The scatter of test results, together with the prescribed values of flexural tensile strength, is shown in [Figure C3.1.1.3](#) (Ref. 1).

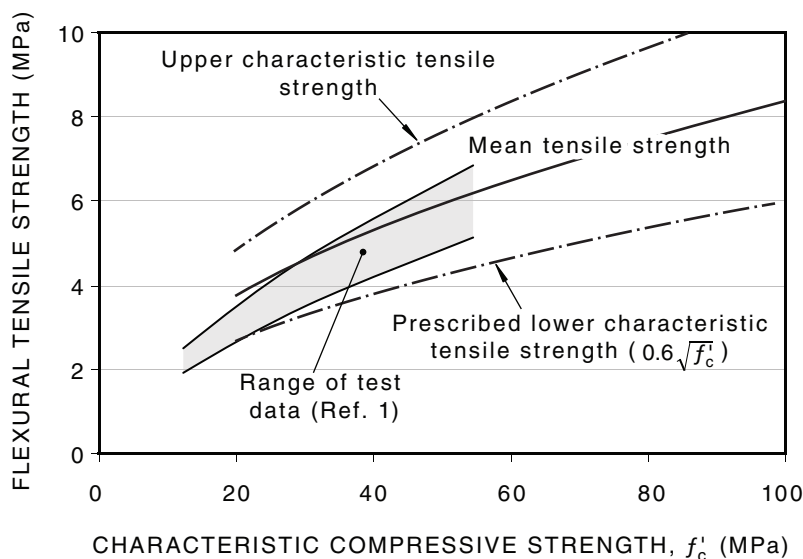


Figure C3.1.1.3 — Relationship between compressive strength and flexural tensile strength (Ref. 1)

The lower characteristic tensile strength is appropriate for use in serviceability calculations when the loss of stiffness due to cracking is to be determined. However, in some situations, it is more appropriate to use either the mean or the upper characteristic tensile strength in design calculations. For example, in the calculation of the minimum ultimate bending strength of a beam in accordance with Clause 8.1.6.1, it may be unconservative to use the lower characteristic tensile strength in meeting the requirement that $(M_{uo})_{min}$ be greater than $1.2 M_{cr}$.

C3.1.2 Modulus of elasticity

The value of the elastic modulus E_c at an appropriate age may be determined by tests in accordance with AS 1012.17. Alternatively, prescribed values may be obtained from one of two specified expressions: one for low to normal strength concrete where $f_{cmi} \leq 40$ MPa and the other for higher strength concrete, where $f_{cmi} > 40$ MPa. The first expression, originally proposed by Pauw (Ref. 2) for normal strength concrete ($f_{cmi} \leq 40$ MPa), has been shown to overestimate the elastic modulus for high strength concrete (Ref. 3). The second expression was proposed by Gilbert (Ref. 3) and provides a better estimate of the elastic modulus for high strength concrete, as shown in [Figure C3.1.2\(A\)](#). The prescribed values for E_c for *in situ* normal weight concrete ($\rho = 2400$ kg/m³) at age 28 days for each of the standard strength grades are given in Table 3.1.2.

The elastic modulus defines the linear relationship between stress and instantaneous strain for stress levels less than about $0.4f_{cm}$ for normal strength concrete ($f'_c \leq 50$ MPa) and less than about $0.6f_{cm}$ for high strength concrete ($50 < f'_c \leq 100$ MPa) and for stresses applied over a relatively short period (say up to 5 min). Although the elastic modulus increases with time as concrete gains strength and stiffness, it is common practice in design to assume that E_c is constant with time and equal to its value calculated at the time of first loading.

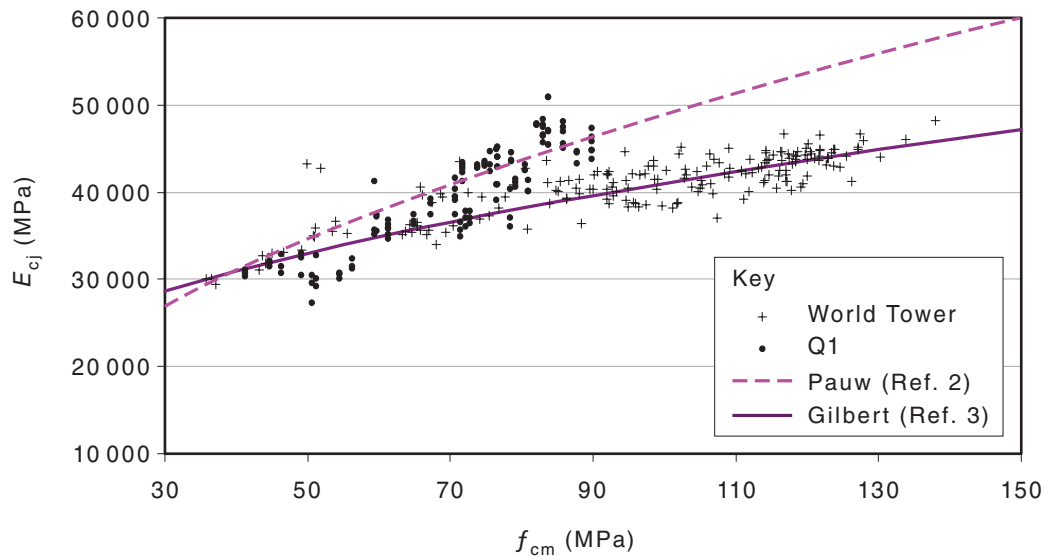


Figure C3.1.2(A) — Prescribed elastic modulus versus test data (Ref. 4)

The prescribed values of E_{cj} specified in the Standard will usually be in the range $\pm 20\%$ of the actual value, depending, among other things, on the aggregate type and quantity and the rate of application of the load. The effect of aggregate type on elastic modulus is illustrated in [Figure C3.1.2\(B\)](#) (Ref. 5).

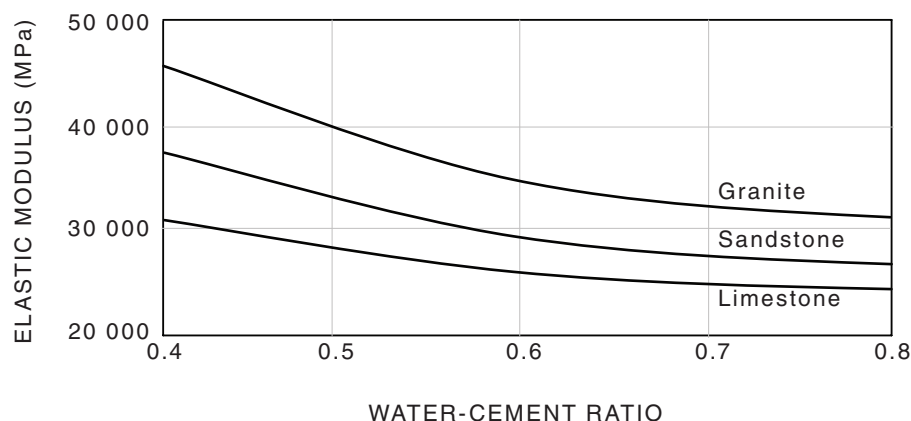


Figure C3.1.2(B) — Effect of aggregate type on elastic modulus (Ref. 5)

In general, the faster the load is applied, the larger is the value of E_{cj} . For stresses applied over a longer time period (say up to one day's duration), significant increases in deformation occur due to the rapid early development of creep. Yet, in a broad sense, loads of one day's duration are usually considered to be short-term and the effects of creep are often ignored. This may lead to significant error. If short-term deformation is required after one day of loading, it is suggested that E_{cj} be reduced by about 20 % to account for early creep (Ref. 6). Typical variations in E_{ci} with time are shown in [Table C3.1.2](#).

Carse and Behan (Ref. 7) showed that the formula proposed by Pauw (Ref. 2) may be used to determine the elastic modulus at any age, provided the average compressive strength at that age is used. The same is true for the expression for high strength concrete. Carse and Behan also showed that a better prediction can be made if the aggregate type, aggregate-cement ratio, and the curing regime are all known.

Table C3.1.2 — Increase in elastic modulus with age of concrete — $E_{cj}(\tau)/E_{cj}(28)$

Cement type	Age of concrete in days (τ)				
	3	7	28	90	360
Ordinary Portland cement	0.70	0.84	1.0	1.11	1.15
High early strength cement	0.77	0.87	1.0	1.06	1.10

C3.1.3 Density

The prescribed value for the density of plain normal-weight concrete is 2400 kg/m³. Alternatively, the density may be determined by tests in accordance with either AS 1012.12.1 or AS 1012.12.2. For the calculation of loads, the mass of concrete members should include an allowance for reinforcement and tendons and a unit weight of 25 kN/m³ is usually taken for both reinforced and prestressed concrete.

C3.1.4 Stress-strain curves

The Standard specifies that, if required, the stress-strain curve for concrete may be determined from test data or may be assumed to be “of curvilinear form defined by recognized simplified equations”. The Standard further specifies that, for design purposes, the shape of the uniaxial stress-strain curve used to model *in situ* concrete be adjusted so that the maximum stress is $0.9f'_c$. This is in recognition that on-site conditions of compaction, curing and exposure may not be as favourable as those for a cylinder prepared, cured and tested in a laboratory environment. Where mean values rather than characteristic values are required, the shape of the stress-strain curve used to model *in situ* concrete should be adjusted so that the maximum stress is f_{cmi} .

Numerous equations describing the curvilinear stress-strain relationship for concrete in compression are available in the literature, including Refs. 8 to 18. Thorenfeldt et al. (Ref. 13) showed that the stress-strain curve for conventional and high strength concretes can be represented by —

$$\sigma = \sigma_{cp} \frac{m\eta}{n - 1 + \eta^{nk}} \quad \text{C3.1.4}$$

where

$$\eta = \varepsilon_c / \varepsilon_{cp}$$

$$\varepsilon_c = \text{concrete strain}$$

$$\varepsilon_{cp} = \text{strain corresponding to the peak } in \text{ situ stress } (\sigma_{cp})$$

$$n = \text{curve fitting factor given by } n = E_c / (E_c - E_{cp})$$

$$E_c = \text{modulus of elasticity of the concrete (specified in Clause 3.1.2)}$$

$$E_{cp} = \sigma_{cp} / \varepsilon_{cp}$$

$$k = \text{decay factor for the post peak response and increases with concrete strength}$$

When $\varepsilon \leq \varepsilon_{cp}$, $k = 1$, and when $\varepsilon > \varepsilon_{cp}$, Collins and Porasz (Ref. 14) proposed that $k = 0.67 + \sigma_{cp}/62 \geq 1.0$, where σ_{cp} is in MPa.

Based on data by Setunge (Ref. 17) for Australian concretes, Attard and Stewart (Ref. 18) recommended that the strain at peak stress is taken as $\varepsilon_{cp} = 4.11\sigma_{cp}^{0.75} / E_c$ where σ_{cp} and E_c are in MPa. A family of stress strain curves (one curve for each standard strength grade) obtained from [Equation C3.1.4](#) (with $\sigma_{cp} = 0.9f'_c$) is plotted in [Figure C3.1.4](#). When modelling the mean stress-strain relationship of *in situ* concrete, σ_{cp} should be taken as f_{cmi} .

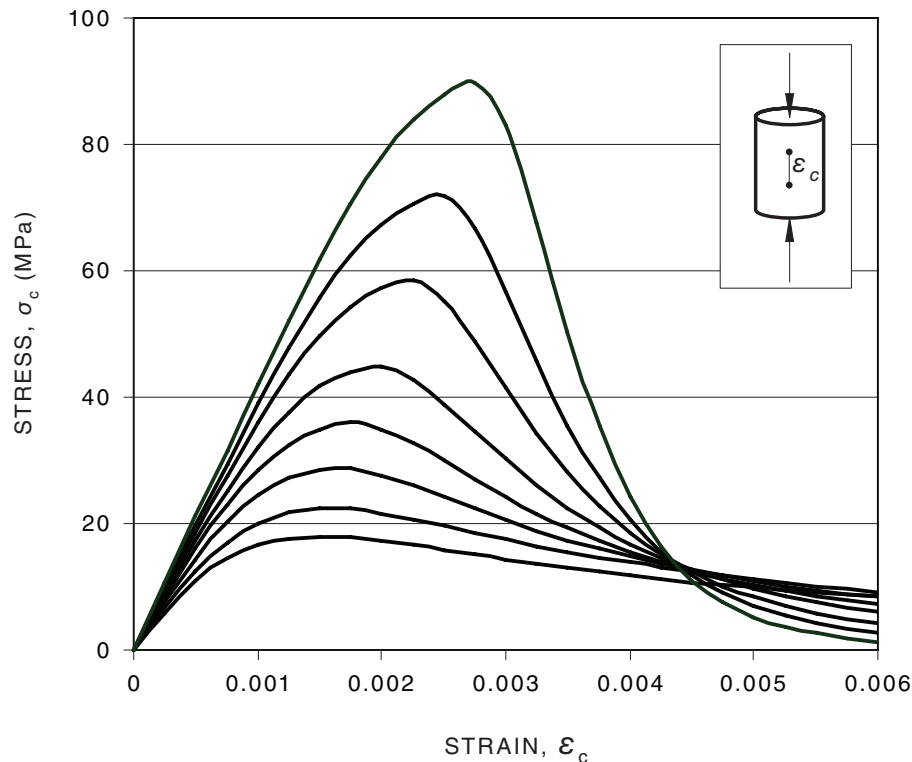


Figure C3.1.4 — Compressive stress-strain curves for concrete

C3.1.5 Poisson's ratio

The specified value for Poisson's ratio for uncracked concrete is $\nu = 0.2$. Alternatively, Poisson's ratio may be determined by tests in accordance with AS 1012.17. The value of 0.2 is a typical value specified in many codes and standards. Poisson's ratio is required as input when designing using either linear or nonlinear stress analysis.

C3.1.6 Coefficient of thermal expansion

The specified value for the coefficient of thermal expansion of concrete is $10 \times 10^{-6}/^{\circ}\text{C}$ within a range of $\pm 20\%$. Alternatively, the coefficient of thermal expansion may be determined from suitable test data.

The value of $10 \times 10^{-6}/^{\circ}\text{C}$ is that specified in the *fib* Model Code (Ref. 19) and should be satisfactory for most structural calculations. However, the coefficient varies over a wide range, depending on the aggregate type, the volume of the cement paste and the degree of saturation of the concrete. An indication of how the type of aggregate affects the coefficient is given in [Table C3.1.6](#). These are average values for the aggregate types quoted, all other factors being constant.

The coefficient of thermal expansion for saturated concrete is about $2 \times 10^{-6}/^{\circ}\text{C}$ less than that for partially dry concrete.

Table C3.1.6 — Coefficient of thermal expansion of concrete^a

Type of aggregate (fine and coarse)	Coefficient of thermal expansion of concrete 10 ⁻⁶ per °C
Quartzite	12.8
Sandstone	11.7
Sand and gravel	10.8
Granite	9.5
Basalt	8.6
Expanded clay and shale	7.6
Limestone	6.8
Clinker	5.9
^a Refer to Table 30.2 in Ref. 20.	

C3.1.7 Shrinkage

Shrinkage of concrete is the time-dependent strain in an unloaded and unrestrained specimen at constant temperature. It is important to distinguish between plastic shrinkage, chemical shrinkage, thermal shrinkage and drying shrinkage. Plastic shrinkage occurs in the wet concrete before setting, whereas chemical, thermal and drying shrinkage all occur in the hardened concrete after setting.

Drying shrinkage is the reduction in volume caused principally by the loss of water during the drying process. It increases with time at a gradually decreasing rate and takes place in the months and years after casting. The magnitude and rate of development of drying shrinkage depend on all the factors that affect the drying of concrete, including the relative humidity, the mix characteristics (in particular, the type and quantity of the binder, the water content and water-to-cement ratio, the ratio of fine-to-coarse aggregate, and the type of aggregate), and the size and shape of the member (Refs. 21 to 23).

Chemical (or autogenous) shrinkage results from various chemical reactions within the cement paste and includes hydration shrinkage that is related to the degree of hydration of the binder in a sealed specimen with no moisture exchange. Chemical shrinkage occurs rapidly in the days and weeks after casting and is less dependent on the environment and the size of the specimen than drying shrinkage.

Thermal shrinkage is the contraction that results in the first few hours (or days) after setting as the heat of hydration gradually dissipates. The term *endogenous shrinkage* is used to refer to that part of the shrinkage of the hardened concrete that is not associated with drying (i.e. the sum of autogenous and thermal shrinkage).

Drying and autogenous shrinkage both increase with time at a decreasing rate. Drying shrinkage may continue for many years depending on the size and shape of the specimen, but most of the autogenous shrinkage takes place in the first 50 days after setting.

Drying shrinkage is dependent on the rate and magnitude of water loss from the hardened concrete. All else being equal, drying shrinkage increases when the water-cement ratio increases, when the relative humidity decreases and when the ratio of the exposed surface area to volume increases. Temperature rises accelerate drying and therefore increase shrinkage. By contrast, endogenous shrinkage increases as the cement content increases and the water-cement ratio decreases. In addition, endogenous shrinkage is not affected significantly by the ambient relative humidity.

The effect of a member's size on drying shrinkage should be emphasized. For a thin member, such as a slab, the drying process may be complete after several years, but for the interior of a larger member, the drying process may continue throughout its lifetime. For uncracked mass concrete structures there is no significant drying (shrinkage) except for about 300 mm from each exposed surface. By contrast, the chemical shrinkage is not affected by the size and shape of the specimen.

Shrinkage is also affected by the volume and type of aggregate. Aggregate provides restraint to shrinkage of the cement paste, so that an increase in the aggregate content reduces shrinkage.

Shrinkage is also smaller when stiffer aggregates are used (i.e. aggregates with higher elastic moduli). Thus shrinkage is considerably higher in lightweight concrete than in normal weight concrete (often by more than 50 %).

C3.1.7.1 Calculation of design shrinkage strain

For estimating the design shrinkage strain of concrete (ϵ_{cs}), one of three approaches may be used. The first approach is to determine ϵ_{cs} from measurements on similar local concrete. The second is to determine ϵ_{cs} by measuring the shrinkage strain that develops in a standard prism after eight weeks of drying in accordance with AS 1012.13 and then determining the final long-term value. The third approach is to calculate ϵ_{cs} using the procedure outlined in Clause 3.1.7.2.

The first and second options will give a more reliable value than the third. The second option involves the standard shrinkage test, where the shrinkage of a 280 mm long prism, with cross-section 75 mm × 75 mm, is measured over a period of 56 days at a relative humidity of 50 %. The results of such a test may be used to estimate the long-term design shrinkage (the 30-year value) of a particular element in a structure using the model outlined in Clause 3.1.7.2. An example of this calculation is provided in [Clause C3.1.7.2](#).

C3.1.7.2 Design shrinkage strain

For high strength concrete, drying shrinkage is less than for normal strength concrete, but autogenous (or chemical) shrinkage is significantly more. The predictive model adopted in the Standard was originally developed by Gilbert (Ref. 3) for estimating the shrinkage strain in normal and high strength concrete. The model has been recalibrated (Ref. 24) based on more recently obtained data from the Australian concrete industry (Ref. 25). The model divides the total shrinkage strain (ϵ_{cs}) into two components, the autogenous shrinkage strain (ϵ_{cse}) and the drying shrinkage strain (ϵ_{csd}) (i.e. $\epsilon_{cs} = \epsilon_{cse} + \epsilon_{csd}$).

The autogenous shrinkage is assumed to develop exponentially [Equation 3.1.7.2(2)], rapidly approaching a final value ϵ_{cse}^* , which varies with the concrete strength according to Equation 3.1.7.2(3), ranging from $\epsilon_{cse}^* = 45 \times 10^{-6}$ when $f'_c = 20$ MPa to $\epsilon_{cse}^* = 350 \times 10^{-6}$ when $f'_c = 100$ MPa. In the first month after setting, 88 % of the autogenous shrinkage is assumed to have occurred. Unlike drying shrinkage, the autogenous shrinkage is assumed to be independent of both the environmental conditions and the size and shape of the concrete member.

At any time (t) after the commencement of drying, the drying shrinkage strain ϵ_{csd} is the product of k_1 , k_4 and $\epsilon_{csd.b}$. The basic drying shrinkage strain $\epsilon_{csd.b}$ varies with concrete strength and is given by —

$$\epsilon_{csd.b} = (0.9 - 0.005f'_c) \times \epsilon_{csd.b}^* \quad \text{C3.1.7.2(A)}$$

The quantity $\epsilon_{csd.b}^*$ depends on the quality of the local aggregates, and in the absence of test data, may be taken to be 800×10^{-6} . If testing is undertaken to determine the 56 day shrinkage strain in accordance with AS 1012.13, a more accurate estimate of $\epsilon_{csd.b}^*$ may be calculated as illustrated subsequently.

The expression in brackets in [Equation C3.1.7.2\(A\)](#) accounts for the effect of concrete strength (or water to cement ratio) on drying shrinkage, ranging from 0.80 when $f'_c = 20$ MPa to 0.40 when $f'_c = 100$ MPa.

The factor k_1 describes the development of drying shrinkage with time and accounts for the dependence of drying shrinkage on the size and shape of the member. It is calibrated in terms of the hypothetical thickness (t_h) in mm and the time (t) after the commencement of drying, in days. The hypothetical thickness of a concrete member is the ratio of the cross-sectional area of the member to half the perimeter of the cross-section exposed to the atmosphere. The rate of development of drying

shrinkage, as well as the final magnitude, increases as the hypothetical thickness reduces. The factor k_1 is given by —

$$k_1 = \frac{\alpha_1 t^{0.8}}{t^{0.8} + 0.15t_h} \quad \text{C3.1.7.2(B)}$$

where

$$\alpha_1 = 0.8 + 1.2e^{-0.005t_h}$$

The factor k_4 depends on the environment, ranging from 0.7 for an arid environment (with average relative humidity 50 %) to 0.5 for a tropical or near-coastal environment (with average relative humidity 70 %).

For a structural element with $t_h = 200$ mm, the shrinkage strain components predicted by the above model at 28 days after the commencement of drying and after 30 years ($t = 10\,950$ days) are given in [Table C3.1.7.2](#). Note that the design shrinkage strain increases with increasing concrete strength, both in the short term and after 30 years.

It is well known that, in addition to the environment, the water-cement ratio and the size and shape of the specimen, shrinkage is also highly dependent on the amount and type of aggregate and the proportions of the binder. The influence of aggregate type on shrinkage is shown in [Figure C3.1.7.2](#) (taken from Ref. 21) for concretes of fixed mix proportions but with different aggregates, and stored in air at 23 °C at 50 % relative humidity.

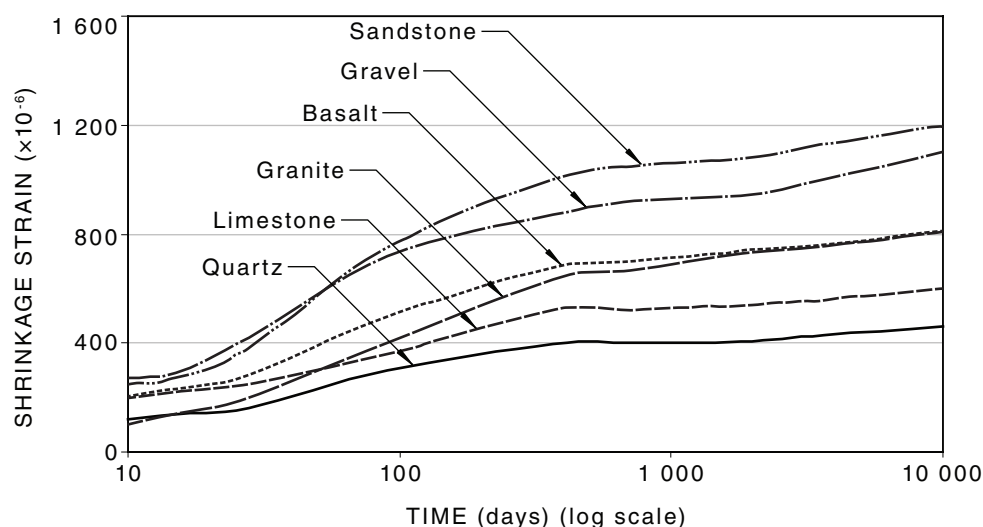


Figure C3.1.7.2 — Effect of aggregate type on shrinkage (Ref. 21)

Table C3.1.7.2 — Design shrinkage strain components ($t_h = 200$ mm and $\epsilon_{csd,b}^* = 800 \times 10^{-6}$)

Environment	f'_c MPa	Shrinkage strain components					
		$t = 28$ days			$t = 10\ 950$ days (30 years)		
		ϵ_{cse}	ϵ_{csd}	ϵ_{cs}	ϵ_{cse}^*	ϵ_{csd}^*	ϵ_{cs}^*
Arid	25	54	175	228	63	529	592
	32	75	167	241	87	506	593
	40	99	158	256	115	478	593
	50	129	146	275	150	444	594
	65	180	130	310	210	393	603
	80	232	113	345	270	342	612
	100	301	90	391	350	273	623
Interior	25	54	162	216	63	492	554
	32	75	155	230	87	469	556
	40	99	146	245	115	444	559
	50	129	136	265	150	412	562
	65	180	120	301	210	365	575
	80	232	105	337	270	317	587
	100	301	84	384	350	254	604
Temperate	25	54	150	203	63	454	516
	32	75	143	218	87	433	520
	40	99	135	234	115	410	525
	50	129	125	254	150	381	531
	65	180	111	291	210	337	547
	80	232	97	329	270	293	563
	100	301	77	378	350	234	584
Tropical	25	54	125	178	63	378	441
	32	75	119	194	87	361	448
	40	99	113	211	115	342	457
	50	129	105	233	150	317	467
	65	180	93	273	210	281	491
	80	232	80	312	270	244	514
	100	301	64	365	350	195	545

The method outlined in the Standard requires only a short calculation, but designers should be aware that the estimate of ϵ_{cs} is within a range of $\pm 30\%$. Because details of the mix proportions and aggregate type are not included in the prediction model for shrinkage, a better estimate of ϵ_{cs} will usually be determined from shrinkage tests on samples kept in the same environmental conditions as the concrete in the structure and made from concrete that is similar to that intended to be used in the structure.

A good estimate of the final design shrinkage strain in a particular member can be obtained from the results of a standard 56 day shrinkage test. In the standard test, shrinkage is measured on a 75 mm $\times$ 75 mm $\times$ 280 mm prism kept at a relative humidity of 50 % and at a temperature of 23 °C after being immersed in water for the first 7 days after casting. For the standard test specimen, the hypothetical thickness is $t_h = 37.5$ mm, $k_4 = 0.7$, and from [Equation C3.1.7.2\(B\)](#) at 56 days, $k_1 = 1.466$.

The relationship between 56 day shrinkage measured on a standard prism in accordance with AS 1012.13 and the design shrinkage strain (ϵ_{cs}) is considered below. The procedure for determining value $\epsilon_{csd,b}^*$ by testing is also considered in the following example calculation.

For a standard prism of concrete with $f'_c = 40$ MPa, assume that the shrinkage strain measured after 56 days is 650×10^{-6} , then —

- determine the value of $\epsilon_{csd,b}^*$ based on the measured 56 day shrinkage; and
- determine the final design shrinkage strain (ϵ_{cs}) after 30 years in a 200 mm thick slab using the same concrete in a temperate inland environment and drying from both the top and bottom surfaces.

For Item (a), almost all the autogenous shrinkage will have occurred at 56 days and for 40 MPa concrete, Equation 3.1.7.2(3) gives —

$$\epsilon_{cse}^* = (0.07f'_c - 0.5) \times 50 \times 10^{-6} = 115 \times 10^{-6}$$

and from Equation 3.1.7.2(2) —

$$\epsilon_{cse}(56) = \epsilon_{cse}^* \times (1.0 - e^{-0.07 \times 56}) = 113 \times 10^{-6}$$

The autogenous shrinkage that would have occurred within the first 7 days of wet curing is $\epsilon_{cse}(7) = \epsilon_{cse}^* \times (1.0 - e^{-0.07 \times 7}) = 45 \times 10^{-6}$. The autogenous shrinkage that occurred after 7 days, and is therefore included in the 56 day shrinkage measurement, is —

$$\epsilon_{cse}(56) - \epsilon_{cse}(7) = 68 \times 10^{-6}$$

The drying shrinkage strain at 56 days is therefore —

$$\epsilon_{csd}(56) = \epsilon_{cs}(56) - \epsilon_{cse}(56) = 650 \times 10^{-6} - 68 \times 10^{-6} = 582 \times 10^{-6}$$

and with $k_1 = 1.466$ and $k_4 = 0.7$, the basic drying shrinkage $\epsilon_{csd,b}$ can now be determined from Equations 3.1.7.2(4) and (5) —

$$\begin{aligned} \epsilon_{csd}(56) &= k_1 k_4 (0.9 - 0.005f'_c) \times \epsilon_{csd,b}^* \\ &= 1.466 \times 0.7 \times (0.9 - 0.005 \times 40) \times \epsilon_{csd,b}^* = 582 \times 10^{-6} \end{aligned}$$

Therefore $\epsilon_{csd,b}^* = 810 \times 10^{-6}$

For Item (b), for a 200 mm thick slab in a temperate inland environment: $t_h = 200$ mm, $k_4 = 0.6$, and from [Equation C3.1.7.2\(B\)](#) at 30 years ($t = 10\,950$ days), $k_1 = 1.22$. With $\epsilon_{csd,b}^* = 810 \times 10^{-6}$, the final drying shrinkage (at age 30 years) (ϵ_{csd}^*) is obtained from —

$$\begin{aligned} \epsilon_{csd}^* &= k_1 k_4 (0.9 - 0.005f'_c) \times \epsilon_{csd,b}^* \\ &= 1.22 \times 0.6 \times (0.9 - 0.005 \times 40) \times 810 \times 10^{-6} = 415 \times 10^{-6} \end{aligned}$$

With the final autogenous shrinkage (ϵ_{cse}^*) equal to 115×10^{-6} , the final design shrinkage strain is —

$$\epsilon_{cs}^* = \epsilon_{cse}^* + \epsilon_{csd}^* = 530 \times 10^{-6}$$

C3.1.8 Creep

When concrete is subjected to a sustained stress, creep strain develops gradually with time. Creep increases with time at a decreasing rate. In the period immediately after first loading, creep develops rapidly, but the rate of increase slows appreciably with time. About 50 % of the final creep develops