

Supplement

Masonry structures — Commentary (Supplement 1 to AS 3700:2018)



AS 3700:2018 Sup 1:2020

This Australian Supplement was prepared by BD-004, Masonry Structures. It was approved on behalf of the Council of Standards Australia on 13 January 2020.

This Supplement was published on 24 January 2020.

The following are represented on Committee BD-004:

- Australian Building Codes Board
- Australian Institute of Building Surveyors
- Cement Concrete & Aggregates Australia
- Concrete Masonry Association of Australia
- Consult Australia
- Engineers Australia
- Galvanizers Association of Australia
- Housing Industry Association
- Masonry Contractors Association of NSW
- Master Builders Australia
- Queensland University of Technology
- Steel Reinforcement Institute of Australia
- Think Brick Australia
- University of Adelaide
- University of Newcastle

Keeping Standards up-to-date

Ensure you have the latest versions of our publications and keep up-to-date about Amendments, Rulings, Withdrawals, and new projects by visiting:

www.standards.org.au

ISBN 978 1 76072 721 5

Supplement

Masonry structures — Commentary (Supplement 1 to AS 3700:2018)

Originated as AS 3700 Supp 1—1991.
Previous edition 2012.
Revised and redesignated as AS 3700:2018 Sup 1:2020.

COPYRIGHT

© Standards Australia Limited 2020

All rights are reserved. No part of this work may be reproduced or copied in any form or by any means, electronic or mechanical, including photocopying, without the written permission of the publisher, unless otherwise permitted under the Copyright Act 1968 (Cth).

Preface

This Supplement was prepared by the Standards Australia Committee BD-004, Masonry Structures, to supersede AS 3700 Supp 1—2012, *Masonry structures—Commentary (Supplement 1 to AS 3700—2011)*.

The objective of this Supplement is to provide commentary to assist in the interpretation and correct application of the provisions of AS 3700:2018, *Masonry structures*, by outlining the background to various requirements of the Standard.

This Supplement does not aim to give a comprehensive textbook treatment of the requirements of the Standard. In particular, the commentary seeks to avoid repeating technical content that is already covered in detail in the following publications:

- (a) “*Australian Masonry Manual*” published in 1991 by a joint committee of the Public Works Department, NSW and the Association of Consulting Structural Engineers, NSW.
- (b) Current technical literature produced by —
 - (i) Think Brick Australia; and
 - (ii) Concrete Masonry Association of Australia.

In this Supplement, AS 3700:2018 is referred to as “the Standard” or “the 2018 edition of the Standard” while this document is referred to as “this Supplement”. The numbers and titles of the sections and clauses in this Supplement align with those in AS 3700:2018 except that the numbers are prefixed by the letter “C”. The figures in this Supplement 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 this Supplement are listed at the end of the applicable section or appendix. The referenced documents are numbered. Bibliographies listing documents recommended for additional information are included as the last item of the section or appendix to which they relate.

Contents

Preface	ii
Section C1 Scope and general	1
C1.1 Scope	1
C1.2 Normative references	1
C1.3 Use of alternative materials or methods	1
C1.4 Information to be provided on documents	1
C1.4.1 General	1
C1.4.2 Provision for demolition	2
C1.5 Definitions	2
C1.5.1 Administrative definitions	2
C1.5.2 Technical definitions	2
C1.6 Notation	2
C1.7 Existing structures	2
Section C2 Requirements for design	3
C2.1 Scope of section	3
C2.2 Aim	3
C2.3 General requirements	3
C2.4 Design requirements	3
C2.5 Serviceability, strength and stability	3
C2.5.1 General	3
C2.5.2 Design for serviceability	3
C2.5.3 Design for strength	4
C2.5.4 Design for stability	4
C2.6 Loads and load combinations	4
C2.6.1 Loads, and other forces and actions	4
C2.6.2 Design load combinations	4
C2.6.3 Design loads for lateral supporting members	4
C2.6.4 Design loads for connections to lateral supports	4
C2.7 Lateral support	5
C2.8 Other design requirements	5
C2.8.1 General	5
C2.8.2 Design for water penetration	5
C2.8.3 Design for accidental damage	5
C2.8.4 Masonry under construction	5
C2.9 Bibliography	5
Section C3 Design properties	6
C3.1 Scope of section	6
C3.2 Masonry units	6
C3.3 Masonry	6
C3.3.1 General	6
C3.3.2 Compressive strength	6
C3.3.3 Flexural tensile strength	7
C3.3.4 Shear strength	7
C3.3.5 Shear factor	8
C3.3.6 Elastic properties of masonry	8
C3.4 Ties and accessories	8
C3.5 Grout	9
C3.6 Reinforcement	9
C3.7 Tendons	9
C3.8 References and bibliography	9
C3.8.1 References	9
C3.8.2 Bibliography	10
Section C4 General design aspects	11

C4.1	Scope of section	11
C4.2	Members of mixed construction	11
C4.3	Chases, holes and recesses	11
C4.4	Capacity reduction factors	11
C4.5	Cross-section properties	12
C4.5.1	Bedded thickness	12
C4.5.2	Effective width of compression faces and flanges	12
C4.5.3	Structural end of a masonry member	12
C4.5.4	Bedded area	12
C4.5.5	Combined cross-sectional area	12
C4.5.6	Design cross-sectional area	13
C4.5.7	Grout area	13
C4.5.8	Section modulus and section moment of area	13
C4.5.9	Chases, holes and recesses	13
C4.6	Design for robustness	13
C4.6.1	General	13
C4.6.2	Robustness of walls	13
C4.6.3	Robustness of isolated piers and unreinforced piers	13
C4.7	Prevention of moisture penetration	13
C4.7.1	Cavities	13
C4.7.2	Weepholes	14
C4.7.3	Damp-proof courses (DPCs) and flashings	14
C4.7.4	Single leaf and solid walls	14
C4.8	Control joints	14
C4.8.1	General	14
C4.8.2	Contraction joints	15
C4.8.3	Expansion joints	15
C4.8.4	Articulation joints	15
C4.8.5	Detailing	15
C4.9	Mortar joints	15
C4.9.1	Thickness	15
C4.9.2	Finishing	16
C4.10	Wall ties	16
C4.11	Bonding, tying and supporting	16
C4.11.1	General	16
C4.11.2	Bonding with masonry header units	16
C4.11.3	Tying with connectors	16
C4.11.4	Fixing to supporting structures	17
C4.12	Stack bonded masonry	17
C4.12.1	Solid and cored unit masonry	17
C4.12.2	Hollow unit masonry	17
C4.13	Arches and lintels	17
C4.14	Interaction between masonry members and slabs, beams or columns	18
C4.15	Corbelling	18
C4.16	Attachment to face of walls	18
C4.17	References and bibliography	18
C4.17.1	References	18
C4.17.2	Bibliography	19
Section C5	Design for durability	20
C5.1	Scope of section	20
C5.2	General	20
C5.3	Exposure environments	20
C5.4	Locations	20
C5.5	Masonry units	21
C5.6	Mortar	21
C5.7	Built-in components	21
C5.8	Grout	21
C5.9	Reinforcement and tendons	21

C5.10	References and bibliography	21
C5.10.1	References	21
C5.10.2	Bibliography	22
Section C6	Design for fire resistance	23
C6.1	General	23
C6.2	Fire-resistance levels	23
C6.3	Structural adequacy	24
C6.3.1	General	24
C6.3.2	Design of walls using tabulated values	24
C6.3.3	Design of walls based on test results	24
C6.3.4	Isolated piers	25
C6.3.5	Minimum reinforcement	26
C6.3.6	Protection to reinforcement	26
C6.4	Integrity	26
C6.4.1	General	26
C6.4.2	Design from tabulated values	26
C6.4.3	Design based on test results	26
C6.5	Insulation	26
C6.5.1	General	26
C6.5.2	Material thickness of member	26
C6.5.3	Design of walls using tabulated values	27
C6.5.4	Design based on test results	27
C6.6	Recesses for services	27
C6.7	Chases	28
C6.8	Protection of structural steelwork	28
C6.9	References and bibliography	28
C6.9.1	References	28
C6.9.2	Bibliography	29
Section C7	Structural design of unreinforced masonry	30
C7.1	General	30
C7.2	General basis of design	30
C7.3	Design for members in compression	30
C7.3.1	General	30
C7.3.2	Basic compressive capacity	30
C7.3.3	Design by simple rules	31
C7.3.4	Design by refined calculation	32
C7.3.5	Concentrated loads	34
C7.4	Design for members in bending	35
C7.4.1	General	35
C7.4.2	Design for vertical bending	35
C7.4.3	Design for horizontal bending	37
C7.4.4	Design for two-way bending	38
C7.5	Design for members in shear	39
C7.5.1	Shear walls	39
C7.5.2	Two or more shear walls acting together	39
C7.5.3	Design for compression and in-plane lateral forces	39
C7.5.4	Shear capacity	39
C7.5.5	Compressive stress on bed joints	40
C7.5.6	Shear connectors	41
C7.6	Design of masonry veneer walls	41
C7.6.1	General	41
C7.6.2	Wall ties with flexible structural backing	41
C7.6.3	Wall ties with stiff structural backing	41
C7.7	Design of cavity walls	42
C7.7.1	General	42
C7.7.2	Compressive load capacity with both leaves loaded	42
C7.7.3	Lateral bending capacity	42

C7.7.4	Wall ties	42
C7.8	Design of diaphragm walls	43
C7.8.1	General	43
C7.8.2	Lateral bending capacity	43
C7.8.3	Diaphragms	43
C7.9	References and bibliography	43
C7.9.1	References	43
C7.9.2	Bibliography	44
Section C8	Structural design of reinforced masonry	45
C8.1	Scope of section	45
C8.2	Exclusions	45
C8.3	General basis of design	45
C8.4	General reinforcement requirement	46
C8.4.1	General	46
C8.4.2	Main reinforcement	46
C8.4.3	Secondary reinforcement	46
C8.4.4	Reinforcement detailing, cover and protection	46
C8.4.5	Close-spaced reinforcement for increased ductility in earthquakes	47
C8.4.6	Wide-spaced reinforcement	47
C8.5	Design of members in compression	47
C8.5.1	Basic compressive capacity	47
C8.5.2	Concentrated loads	48
C8.6	Design of members in bending	48
C8.7	Design of walls for in-plane shear	49
C8.7.1	General	49
C8.7.2	Long walls	49
C8.7.3	Short walls	49
C8.7.4	Stability	50
C8.8	Design of walls for out-of-plane shear	50
C8.9	Design of beams in shear	50
C8.10	Design of members in tension	52
C8.11	Design for combined loading	52
C8.11.1	Members in combined bending and compression	52
C8.11.2	Members in combined bending and tension	53
C8.12	References and bibliography	53
C8.12.1	References	53
C8.12.2	Bibliography	53
Section C9	Structural design of prestressed masonry	54
C9.1	Scope of section	54
C9.2	General basis of design	54
C9.3	Design criteria for prestressing tendons	54
C9.4	Design of members in compression	54
C9.5	Design of members in bending	55
C9.5.1	General	55
C9.5.2	Ultimate tensile stress in tendons	55
C9.5.3	Upper limit on tendon area	55
C9.5.4	Minimum bending strength	55
C9.6	Design of members in shear	56
C9.7	Design of members in tension	56
C9.8	Design for combined loading	56
C9.9	Design of anchorage zones	56
C9.10	References and bibliography	56
C9.10.1	References	56
C9.10.2	ibliography	56
Section C10	Design for earthquake actions	57
C10.1	Scope of section	57
C10.2	General design criteria	57

C10.2.1	General	57
C10.2.2	Structural ductility factor (μ) and structural performance factor (S_p)	57
C10.2.3	Structural elements	57
C10.2.4	Non-structural components	57
C10.2.5	Connections and wall anchorage	57
C10.3	Detailing masonry structures for earthquake loads	57
C10.3.1	General	57
C10.3.2	Fixing of cavity walls to supports	58
C10.4	Restrictions on the use of loadbearing unreinforced masonry	58
C10.5	References and bibliography	58
C10.5.1	References	58
C10.5.2	Bibliography	58
Section C11	Materials	59
C11.1	Scope of section	59
C11.2	Masonry	59
C11.3	Masonry units	59
C11.4	Mortar	59
C11.4.1	General	59
C11.4.2	Materials	60
C11.4.3	Mortar durability	60
C11.4.4	Structural properties of mortar	60
C11.4.5	Mortar for reinforced or prestressed masonry	61
C11.5	Wall ties, connectors, accessories and lintels	61
C11.5.1	Wall ties	61
C11.5.2	Connectors and accessories	61
C11.5.3	Lintels	61
C11.6	Damp-proof courses (DPCs), flashings and weatherings	61
C11.7	Grout	61
C11.7.1	General	61
C11.7.2	Materials	61
C11.7.3	Strength	61
C11.8	Reinforcement and tendons	61
C11.8.1	Reinforcement	61
C11.8.2	Tendons	61
C11.8.3	Reinforcement embedded in mortar joints	62
C11.9	References and bibliography	62
C11.9.1	References	62
C11.9.2	Bibliography	62
Section C12	Construction	63
C12.1	Scope of section	63
C12.2	General	63
C12.3	Materials	63
C12.3.1	General	63
C12.3.2	Masonry units	63
C12.3.3	Mortar	64
C12.4	Workmanship	65
C12.4.1	Base course	65
C12.4.2	Mortar joints	65
C12.4.3	Movement control joints	65
C12.4.4	Bonding	65
C12.4.5	Cutting of units	65
C12.4.6	Holes and chases	65
C12.4.7	Building in	66
C12.4.8	Bolts and anchors	66
C12.4.9	Rate of construction	66
C12.4.10	Sections of masonry constructed at different rates or times	66

C12.4.11	Construction during adverse weather conditions.....	66
C12.4.12	Disturbance of new masonry	66
C12.4.13	Cavities in walls.....	66
C12.4.14	Weepholes.....	66
C12.4.15	Joint finishing.....	66
C12.4.16	Damp-proof course and flashing (DPC)	67
C12.4.17	Lintels.....	67
C12.5	Tolerances in masonry	67
C12.5.1	General.....	67
C12.5.2	Measurement of bow	67
C12.5.3	Reinforcement and tendons.....	67
C12.6	Site control.....	67
C12.6.1	General.....	67
C12.6.2	Verification of strength properties	68
C12.6.3	Verification of durability resistance.....	68
C12.6.4	Verification of mortar composition.....	68
C12.7	Additional site control of special masonry	68
C12.7.1	General.....	68
C12.7.2	Rate of sampling.....	69
C12.7.3	Target strength.....	69
C12.8	Grouted masonry	69
C12.8.1	Cleaning out.....	69
C12.8.2	Grouting.....	69
C12.8.3	Sampling and testing of grout.....	69
C12.9	Masonry under construction	69
C12.9.1	Temporary bracing.....	69
C12.9.2	Premature loading	70
C12.9.3	Physical damage.....	70
C12.9.4	Weather conditions.....	70
C12.10	Cleaning	70
C12.11	Testing of <i>in situ</i> masonry.....	71
C12.12	References and bibliography	71
C12.12.1	References.....	71
C12.12.2	Bibliography.....	71
Appendix CA	(informative) Assessment of strength values from test results	72
Appendix CB	(informative) Determination of characteristic value.....	73
Appendix CC	(informative) Method of test for compressive strength.....	74
Appendix CD	(informative) Method of test for flexural strength	76
Appendix CE	(informative) Durability testing	79
Appendix CF	(informative) Method of measurement of bow	81
Appendix CG	(informative) Strength testing of <i>in situ</i> masonry	82
Appendix CH	(informative) Guidance on the use of masonry in restoration work and masonry constructed using square-dressed natural stone.....	83
Appendix CI	(informative) ISO 9223 corrosivity categories and relationship to durability class	84

Supplement

Masonry structures — Commentary (Supplement 1 to AS 3700:2018)

Section C1 Scope and general

C1.1 Scope

The scope of the Standard is intended to cover all masonry structures, including masonry retaining walls, masonry water-retaining structures and masonry in bridge structures.

The clauses in the Standard are written with the expectation that their application is to masonry members in the form of walls, beam and piers whose bed joints are horizontal. Further, they have been written with the expectation that those applying and using the provisions of the Standard will be qualified and experienced in their areas of responsibility. It is expected that, in their hands, careful and experienced application of these provisions will result in safe, serviceable and economical masonry construction.

No specific guidance is given on autoclaved aerated concrete (AAC) laid in other than thin-bed mortar, prefabricated masonry or composite action of masonry with other materials. However, the Standard is not intended to prevent the use of these forms of construction.

C1.2 Normative references

All documents referenced within the Standard that form part of the requirements of the Standard are included in this list. Documents referenced in the Standard for informative purposes are listed in the Bibliography at the end of the Standard.

C1.3 Use of alternative materials or methods

The purpose of Clause 1.3 is to spell out that the Standard is not to be used to prevent innovation or the use of new design procedures or construction techniques; however, these would generally mean that the designer will have to find alternative means to demonstrate conformance to the relevant requirements. The soundness of any innovation, or of any departures from the requirements of the Standard, will have to be adequately demonstrated. It will be necessary to obtain the approval of the relevant authority for the use of alternative materials or methods.

Existing structures

Although Clause 1.3 provides guidance on the use of the Standard for the evaluation of strength or serviceability of existing structures, the determination of whether or not an existing structure is to be demolished is not covered by the Standard. Such a decision can only be made giving regard to the degree of deviation from the requirements of the Standard, the resulting level of safety, the consequence of failure in terms of injury and cost, and the cost of demolition and reconstruction.

C1.4 Information to be provided on documents

C1.4.1 General

It cannot be expected that satisfactory masonry will be built if those responsible for building it are not adequately informed on all of the relevant design requirements for that masonry. The Standard requires that this information be set out clearly on the documents, preferably the drawings that are used on site by those carrying out the construction.

It is not necessary for all items of information to be provided for every project; items are only required if they are necessary for construction of the particular project.

The list of information to be shown on the documents includes category, type and work sizes of masonry units. Refer to Clause 1.5, Definitions, for the meaning of these terms.

C1.4.2 Provision for demolition

The Clause requires that notification be given to the potential problem of progressive collapse during demolition and the problems of demolishing prestressed masonry.

C1.5 Definitions

Clause 1.5 contains only those definitions considered necessary for the purposes of the Standard, and is not a comprehensive listing of all terms currently used in connection with masonry construction.

Where it is necessary to provide commentary on a definition, it is located in the commentary to the clause affected by the definition.

C1.5.1 Administrative definitions

(No Commentary)

C1.5.2 Technical definitions

The term “masonry” is a generic term covering any or all of brickwork and blockwork in clay, concrete, calcium silicate, AAC and square-dressed natural stone.

C1.6 Notation

(No Commentary)

C1.7 Existing structures

(No Commentary)

Section C2 Requirements for design

C2.1 Scope of section

(No Commentary)

C2.2 Aim

Most of the provisions in the Standard relate to the structural design of masonry for strength or collapse. Consideration is also required to be given to serviceability requirements.

C2.3 General requirements

All parts of the masonry, including built-in components, are required to have sufficient durability, fire resistance (where required), serviceability (particularly stiffness), strength and stability to serve their intended function throughout the intended life of the structure. All components should have an expected life similar to that of the building.

C2.4 Design requirements

Clause 2.4 sets out the means of satisfying the requirements of Clause 2.3.

C2.5 Serviceability, strength and stability

C2.5.1 General

A number of different stress-strain relationships have been assumed and used in determining the various strength capacities of masonry members for the Standard. These are as follows:

- (a) *For unreinforced members:*
 - (i) For members under the action of out-of-plane bending for transient lateral loads and small compression forces — a linear stress-strain relationship is assumed for both tension and compression stresses, and the cross-section is assumed to be uncracked until failure occurs.
 - (ii) For all other cases — the tensile capacity is assumed to be zero and the cross-section is assumed to be cracked throughout the tension zone.
- (b) *For reinforced and prestressed members:*
 - (i) For flexure — a parabolic stress-strain relationship is assumed for the masonry in compression. This has been simplified to a rectangular stress block, in a way similar to that used for reinforced concrete design.
 - (ii) For compression — a linear stress-strain relationship is assumed.

C2.5.2 Design for serviceability

The Standard requires that the masonry be designed to allow movements to be controlled or isolated so that the damage to the masonry, the building and its components is avoided and the structural and other requirements are satisfied or limited. The upper limit on crack width of 1 mm for masonry required in previous editions of the Standard was removed.

Where unreinforced loadbearing masonry is adequately designed for strength, the deflection and resulting crack widths of the masonry are likely to be insignificant as far as serviceability is concerned.

Where unreinforced masonry is used in conjunction with loadbearing frames or other structures, the effects of interaction have to be considered. For example, where unreinforced masonry is used in

framed construction, the deflection of the frame has to be limited to ensure that no deleterious cracking occurs in the masonry.

Reinforced masonry will tolerate larger deflections than unreinforced masonry before collapse occurs. Therefore, design for strength will not necessarily provide a guarantee of adequate serviceability performance. In particular, centrally reinforced masonry walls may exhibit larger than expected lateral deflections and crack widths because of their small effective depth.

C2.5.3 Design for strength

Clause 2.5.3 provides direction on the determination of loads and resistance against strength failure.

C2.5.4 Design for stability

Clause 2.5.4 provides direction on the determination of loads and resistance against instability.

C2.6 Loads and load combinations

C2.6.1 Loads, and other forces and actions

(No Commentary)

C2.6.2 Design load combinations

Typically, different load combinations and different design loads are associated with different failure modes. Thus, the combination of loads that controls the design of a cross-section for flexure may differ from that which controls the design for shear. For example, the bending capacity of a cross-section is usually dependent upon the magnitude of the simultaneously acting compressive force and, in such cases, the capacity has to be checked for all possible load combinations.

Because a compressive force in an unreinforced masonry member increases its bending and shear capacity, it is important, when evaluating the bending or shear capacity of such a member, that the simultaneously acting compressive force be evaluated conservatively. An overestimate of the magnitude of the compressive force can be as unsafe in these circumstances as is an underestimate in other circumstances.

C2.6.3 Design loads for lateral supporting members

The simple static reaction at supports that resist horizontal loads has to include both the reaction to horizontally applied loads and a component to resist the horizontal buckling of vertically loaded masonry (equal to 2.5 % of the vertical load).

The supporting members have to also resist a lower bound horizontal pressure of 0.5 kPa.

The 2.5 % of vertical load and the 0.5 kPa are considered to be factored loads and do not need to be further factored to determine the design action effect.

Lateral supports have to have sufficient stiffness to ensure that the masonry remains serviceable.

C2.6.4 Design loads for connections to lateral supports

Designers have to ensure that all possible loads and load combinations acting on connections have been considered (e.g. earthquake loadings on partition walls, accidental loads and similar loads).

The Standard requires that the connections to lateral supports be designed for loads 25 % greater than the factored design load.

C2.7 Lateral support

Clause 2.7 is included to set parameters for strength and stiffness in order for a member or system to be considered as an adequate lateral support.

C2.8 Other design requirements

C2.8.1 General

(No Commentary)

C2.8.2 Design for water penetration

(No Commentary)

C2.8.3 Design for accidental damage

Loadbearing masonry construction, along with large panel precast concrete construction, is commonly held to be more susceptible to progressive collapse than most other forms of construction. However, appropriately designed multistorey masonry structures can withstand severe structural damage without progressive collapse occurring, and adequate resistance against accidental damage can often be provided simply by giving appropriate attention to detailing and without significantly increasing construction costs.

Clause 2.8.3 does not define when it is necessary to design against accidental damage or progressive collapse, nor what criteria have to be used in any such design—the range of possible hazards is too large and too unpredictable. It is left to the designer to assess each case on its individual merits and to implement such preventative measures as the circumstances warrant.

C2.8.4 Masonry under construction

Consideration should be given to the use of temporary supports and the appropriate times for the application of loads and removal of supports. For example, reinforced masonry retaining walls should not be back-filled until the grout has achieved sufficient strength to resist the applied short-term loads. Allowance should be made for the deflection of slabs due to the removal of props, stacked materials and the like, and the possible formation of cracks in the masonry above. Clause 12.9.1 has requirements for temporary bracing.

C2.9 Bibliography

BAKER L.R., LAWRENCE S.J., PAGE A.W. *Australian Masonry Manual. PWD/ASCE (NSW)*. Joint Committee, Sydney, 1991

Design of clay masonry for serviceability. Manual 7. Think Brick Australia, Sydney, 2012

Design of clay masonry for sound insulation, Manual 11. Think Brick Australia, Sydney, 2018

Detailing of Clay masonry. Manual 9. Think Brick Australia, Sydney, 2011

Industry Reference Guide. Manual 19. Think Brick Australia, Sydney, 2018

Section C3 Design properties

C3.1 Scope of section

The Standard uses characteristic values, e.g. f'_{uc} , f'_m , f'_{mt} , f'_{ms} which are 95-percentile values of the properties. (That is, the characteristic value of the property is exceeded by 95 % of the masonry or masonry components under consideration.)

The various tables and equations for strength capacities used in the Standard have been devised and adjusted where necessary to suit the values for the characteristic strengths defined above. Appendix B provides a method of determining from test results estimates of the 95-percentile strengths, with 75 % confidence that the estimates are not too high.

C3.2 Masonry units

The Standard uses the unconfined compressive strength (f'_{uc}) of the masonry unit, which converts the confined strength from a crushing test to the strength of an idealized prism with a height:thickness ratio of 5:1.

Clause 12.4.2 requires that hollow masonry be face-shell bedded and Clause 4.5.4 requires that the bedded area used in design be the face shell area. Therefore, hollow units have to be tested using face-shell bedding.

The characteristic lateral modulus of rupture of masonry units (f'_{ut}) is used in assessing the resistance of walls to out-of-plane horizontal bending. The strength of some units will exceed the default value of 0.8 MPa (given in Clause 3.2). Higher values used in design have to be established by test. AS/NZS 4456.15 (Ref. 1) specifies the test method for establishing this property.

C3.3 Masonry

C3.3.1 General

The most common practice is for the design to be based on values of f'_m , f'_{mt} , f'_{ms} provided in Section 3 of the Standard, rather than on tests.

When design is to be based on test results, tests for a specific project will not be necessary if a sufficient body of relevant test data are already available. For special masonry, designers are advised to seek advice from unit manufacturers and laboratories experienced in the testing of masonry for help in selecting a characteristic strength value for use in the design that can be realistically attained on site.

C3.3.2 Compressive strength

Manufacturers normally test at other than 28 days and apply appropriate correction factors to obtain the equivalent 28 day strength.

For masonry other than AAC, the apparent compressive strength, resulting from a standard compression test on a masonry unit or prism, is critically dependent on the height relative to the cross-sectional area of the specimen. For convenience, this is often taken as the height/least width ratio. As this ratio decreases, the observed strength of the specimen increases due to the confining effects of the testing machine platens. For example, the confined (apparent) strength of a solid clay unit will be almost twice its equivalent unconfined (true) strength. A similar effect occurs in the testing of prisms, although to a lesser extent, since the aspect ratio is much greater than for the units.

The most effective means of comparison of the compressive strengths of specimens with different aspect ratios is either to carry out unconfined tests, or to convert all compressive strengths to an

equivalent unconfined value in which the effects of platen restraint have been eliminated. The Standard assumes that this is achieved at an aspect ratio of 5.0.

In selecting a value of f'_m a designer has the option of deriving a value appropriate to the unit type and strength, and the mortar type, by using equations in Clause 3.3.2(a)(i) or, for special masonry, using a higher value that has to be confirmed by prism tests during construction. The units of f'_{uc} used in Clause 3.3.2(a)(i) are specified because the equation is dimensionally inconsistent.

The values in Table 3.1 have been obtained from test results. In some cases, the values are well supported; others have very little test support and have been inferred from other results or adapted from previous codes. Test results for a given type of unit and mortar usually exhibit a wide degree of scatter. It is evident that there are other sources of variability additional to the parameters taken into account in Table 3.1. The values provided in the Table are therefore, necessarily, at the low end of the range of each set of test results. Thus, the values in Table 3.1 should be considered as a lower bound; prism tests will probably yield substantially higher results.

The value of k_m in Table 3.1 for masonry utilizing M4 mortar was increased from that in earlier editions of the Standard, based on recent research (Ref. 2).

The values for hollow masonry are for prisms built using face-shell bedding and for prisms and units tested in compression using face-shell capping. This reflects current construction practice where, even for walls built with nominal full bedding, the webs of the units do not align because of the geometry of the units.

The values in Table 3.1 have been normalized for a unit height to joint thickness ratio of 7.6 (76 mm high unit and a 10 mm thick mortar joint). This normalization has been necessary because it is well established that for a given height of masonry unit the compressive strength of a masonry specimen decreases as the joint thickness increases. The converse is also true, with masonry strength increasing as the relative size of the unit increases in relation to the joint thickness. For this reason, the values obtained from Table 3.1 are modified by a factor (k_h) given in Table 3.2.

For AAC masonry laid in thin-bed mortar (that is, not conventional 10 mm mortar joints), tests have indicated that the presence of the joints does not reduce the strength of the masonry, and the compressive strength of the masonry is the same as the compressive strength of the units.

C.3.3 Flexural tensile strength

The characteristic flexural tensile strength of masonry f'_{mt} is defined in terms of joint strength in vertical bending (that is, with flexural stresses perpendicular to the bed joint). Where no tests are carried out, a value up to 0.20 MPa may be used in design. Although there is no specific requirement that this value be achieved in any subsequent testing, masonry that fails to achieve this basic level of bond strength might be inadequate. For unreinforced special masonry, test results may be used to justify the use of values in design up to 1.0 MPa, although these higher values are difficult to achieve.

The Standard gives a value applicable to flexural tensile strength of AAC masonry.

The Standard requires characteristic flexural tensile strength to be taken as zero for loads other than of a transient nature. The Standard specifies that live loads are not of a transient nature, e.g. tensile strength, should not be used in resisting live loading on balustrades or similar.

C3.3.4 Shear strength

The shear strength of an unreinforced masonry member is made up of two parts, the ability of the mortar to join the units together due to bond and the ability to resist relative movement due to friction between the components (refer to Clause 7.5.4). Clause 3.3.4 deals with the first of these two parts.

For AAC in thin-bed mortar, the shear strength is directly related to the tensile strength indicated by the lateral modulus of rupture of the units (Ref. 3).

The application of shear force to a joint usually produces a non-uniform shear stress distribution, as well as normal stresses from the rotational couple produced by the non-symmetric application of shear force (as, for example, in a couplet test). For this reason, no test for shear strength is given in the Standard, and an empirical relationship in terms of the flexural tensile strength is specified. Research has indicated that the shear strength mainly depends upon the bond between mortar and unit, and a relationship in terms of flexural bond strength is considered appropriate (Ref. 4).

The expressions for the shear strength on vertical shear planes include allowance for the contribution of any connectors or bonded masonry header units that project across the shear plane.

At membrane-type damp-proof courses and flashings, at interfaces between masonry and other materials, and in square-dressed natural stone, the bond contribution to shear strength is to be taken as zero unless appropriate tests are carried out to justify higher figures.

C3.3.5 Shear factor

Clause 3.3.5 deals with the second component of shear strength.

A contribution to shear capacity of a masonry member can be obtained by utilizing the frictional capacity created by the superimposed compressive forces. This is particularly useful where f'_{ms} is zero, such as at membrane-type damp-proof courses and flashings, at interfaces between masonry and other materials, and in square-dressed natural stone. Table 3.3 (Ref. 5) gives the appropriate values for shear factor.

Of particular concern to designers is the requirement that certain built-in membranes be capable of permitting both —

- (a) long-term relative movements between the wall and supporting structure (e.g. concrete floor slab) due to concrete shrinkage, clay expansion or footing movement; and
- (b) lateral out-of-plane and in-plane support for the wall under both wind and earthquake loading.

This problem has been the subject of research into the static, cyclic and dynamic behaviour of unreinforced masonry with membrane-type damp-proof courses and flashings and the results have been used to develop the values of shear factor given in Table 3.3.

C3.3.6 Elastic properties of masonry

Masonry is a complex material, which will often have nonlinear deformation characteristics and properties that vary with the direction of loading because of the influence of the mortar joints. Where the masonry is assumed to have linear elastic properties, suggested approximate values of short-term and long-term elastic moduli, (E_m and E_L , respectively), are given in Table 3.4. The long-term modulus of elasticity (E_L) is less than the short-term modulus (E_m) because of the effects of creep, most of which occurs in the mortar.

Although it is not provided in the Standard, it is suggested that the shear modulus be taken as 0.4 times E_L or E_m as appropriate, and that Poisson's ratio be taken as 0.2.

C3.4 Ties and accessories

Values of strength for Type A ties in tension and compression are given in Tables 3.5 and 3.6 to enable design calculations for tie spacing under specific loading conditions to be carried out. Rating of the ties in accordance with AS/NZS 2699.1 (Ref. 6) ensures that these strengths are applicable.

Design of ties in the 2018 edition of the Standard is based on mean (not 95-percentile) values of strength, based on recent research (Ref. 7). The values in Table 3.5 and Table 3.6 have been adjusted accordingly.

C3.5 Grout

The Standard requires that the characteristic compressive strength of the grout be at least 12 MPa (refer to Clause 11.7.3), but is often supplied at a much higher strength. Recent research has shown that the previous upper limit for the design strength of $1.3f'_{uc}$ is inappropriate and should no longer apply. This requirement has therefore been deleted from the 2018 edition of the Standard. Provided the requirements of Clause 11.7.3 are met, the design compressive strength of the grout is required to be greater than or equal to the characteristic compressive strength of the masonry (f'_m). These requirements for grout strength are consistent with the empirical relationship for the compressive strength of grouted masonry in Clause 7.3.2(b) and Clause 8.5.

Grout may be delivered by trucks to the site by a concrete supplier or prepared using a concrete drum mixer at site. Each pour should be quality ensured using slump measurement and sampling of 100 mm diameter $\times$ 200 mm high cylindrical specimens for 28-day compression strength testing. The characteristic compressive strength of the grout (f'_{cg}) used in the design stage should be greater than 12 MPa and should be checked for conformance of quality of the pouring.

Although higher strength grout can potentially be prescribed to achieve higher compression capacity of grouted unreinforced and reinforced masonry walls, as the research experiments (Ref. 8 and 9) considered only a maximum of 50 MPa grout, it is suggested that the grout strength be limited to 50 MPa irrespective of the strength of blocks used.

C3.6 Reinforcement

The grades specified and the yield strengths given in Table 3.7 for reinforcement are subject to change as the relevant Australian Standards are revised and other grades of reinforcement become commercially available.

Stainless steel bars with the physical and mechanical properties shown in Table 3.7 and otherwise in accordance with BS 6744 or EN 10088-5 and with the chemical composition conforming to one of designations 1.4311, 1.4162, 1.4362, 1.4462, 1.4404 or 1.4429 to EN 10088-1 can be assumed to conform to the Standard. Stainless steel bars to BS 6744 and EN 10088-5 are deemed to meet the requirements for Ductility Class N reinforcement as in AS/NZS 4671.

Although Clause 3.6 specifies a value for elastic modulus (E_s) of 200×10^3 MPa, the elastic modulus of stainless steel can vary between 180×10^3 MPa and 230×10^3 MPa. Significant deviations from the specified value will affect Equation 8.6 for bending of reinforced masonry. (Refer to the development of Equation 8.6.)

C3.7 Tendons

The modulus of elasticity of tendons can vary by 5 % and could be different when a number of tendons are combined into a single cable. Prestressed wire and tendons should not be galvanized.

C3.8 References and bibliography

C3.8.1 References

- [1] AS/NZS 4456.15, *Masonry units, segmental pavers and flags — Methods of test, Method 15: Determining lateral modulus of rupture*, Standards Australia, Sydney, 2003
- [2] LAWRENCE S.J., STEWARD M. Reliability-based calibration of the capacity reduction factor for design of masonry in compression to AS 3700. *Australian Journal of Structural Engineering*. 2009, 9(2) pp. 97–110

- [3] LAWRENCE S.J., SAMARASINGHE W., MALOUKIS W. *Autoclaved Aerated Concrete Masonry in Australia*. Proceedings of the 11th International Brick/Block Masonry Conference, Shanghai, 1997, pp. 125–134
- [4] MASIA M., SIMUNDIC G. and PAGE A.W. *Assessment of the AS 3700 Relationship Between the Bond Strength and Flexural Tensile Bond Strength in Unreinforced Masonry*, Proc. 15th IBMAC, Florianopolis, Brazil, 2012
- [5] GRIFFITH M.C., PAGE A.W. On the Seismic Capacity of Typical DPC and Slip Joints in Unreinforced Masonry Buildings. *Australian Journal of Structural Engineering*. 1998, **1**(2) pp. 133–140
- [6] AS/NZS 2699.1, *Built-in components for masonry construction, Part 1: Wall ties*, Standards Australia, Sydney, 2000
- [7] LAWRENCE S.J. The use of characteristic strengths in masonry design. *Australian Journal of Structural Engineering*. 2007, **7**(3) pp. 225–237
- [8] DHANASEKAR M., RING J., SONG M. *Effectiveness of Vertical Steel Reinforcing Bars to Reinforced Concrete Masonry Walls Under Compression*, Concrete 2017, Adelaide
- [9] DHANASEKAR M., SONG M., RING J. *Response of Reinforced Concrete Masonry Walls to Eccentric Compression*, Proc. 13th Canadian Masonry Symposium, Halifax, Canada, 2017

C3.8.2 Bibliography

- BAKER L.R., LAWRENCE S.J., PAGE A.W. *Australian Masonry Manual*. PWD/ASCE (NSW). Joint Committee, Sydney, 1991
- PAGE A.W., MARSHALL R. *The Influence of Brick and Brickwork Prism Aspect Ratio on the Evaluation of Compressive Strength*. Proceedings of the 7th International Brick Masonry Conference, Melbourne, 1985, pp. 653–664
- PAGE A.W., SHRIVE N.G. A Critical Assessment of Compression Tests for Hollow Block Masonry. *Masonry International*. 1998, **2**(2) pp. 64–70
- SHRIVE N.G. *Compressive Strength and Strength Testing of Masonry*. Proceedings of the 7th International Masonry Conference, Melbourne, 1985, pp. 699–710

Section C4 General design aspects

C4.1 Scope of section

(No Commentary)

C4.2 Members of mixed construction

Cavity walls are often built with inner and outer leaves of different units and only one leaf supporting floor loads. Such construction can be quite acceptable, but provision needs to be made in the coursing for wall ties to be placed so that they can transmit the required loads between the leaves, and so that moisture does not flow to the inner leaf. Provision has also to be made for differing elastic properties, differential expansion/shrinkage of the two materials (both moisture related and thermal) and for any contraction of the loaded leaf.

The Standard allows for construction that is a combination of reinforced and unreinforced masonry acting compositely. Because of their greater strength capacity, reinforced masonry members can support or withstand higher loads and can, therefore, experience greater strains and deflections than unreinforced members of the same size. The design needs to ensure that the deflection of the reinforced masonry does not lead to unacceptable cracking of the unreinforced masonry.

C4.3 Chases, holes and recesses

Chases, holes and recesses, including those cut after completion of the masonry, will affect the properties of the wall. For example, horizontal or vertical chases might lessen the resistance to horizontal forces and loads from any structure above. Generally, when chases are used, the thickness of the wall should be measured from the base of the chase to the opposite face (that is, at the thinnest section), except where specific rules apply, such as those issued by Clauses 6.6 and 6.7.

C4.4 Capacity reduction factors

The capacity reduction factor allows for the following:

- (a) Simplifications and inaccuracies in analysis.
- (b) Differences between the strength of members and control specimens constructed in the laboratory, and those constructed under practical on-site conditions.
- (c) The greater or lesser perceived variability of different materials or combinations, additional to those already accounted for in the assessment of characteristic values.
- (d) The structural nature of the member and the mode of failure under consideration.

The values of the capacity reduction factor for unreinforced masonry have been derived principally by calibration with previous working stress codes. In the 2018 edition of the Standard, the capacity reduction factor for unreinforced masonry in compression is given separately for solid or cored, hollow and grouted masonry, and increased based on recent research (Ref. 1 and 2).

Values are included for reinforced masonry, prestressed masonry, wall ties, connectors and accessories.

The Standard does not distinguish between structures on the basis of their importance or consequences of failure. The values for the capacity reduction factors are based on a level of workmanship and site control specified in Section 12.

A size reduction factor required in earlier standards is not considered necessary. Although some weak masonry units or workmanship defects could be present in compression members of small cross-sectional area, such as isolated piers, an additional factor was considered unnecessary, given that reduced robustness limits also apply to these members of small cross-sectional area.

In the Standard, the capacity reduction factor (ϕ) for the compression design of grouted unreinforced masonry and grouted reinforced masonry is validated as 0.6 and 0.75 respectively, based on recent reliability calculations using the compression test data from 112 reinforced masonry walls and 12 unreinforced masonry walls of varied slenderness and eccentricities (Ref. 14 and 15). The reliability calculations were carried out using a target reliability coefficient of 3.6. Variabilities in the dimensions of the blocks, strength of the grout and workmanship were considered from a combination of the available experimental data sets and those reported in the literature (Ref. 15 and 16). The mean model error and its variability was determined from the experimental data sets as 1.40 and 0.13 respectively. Using the standard procedure of first order second moment method, for the target reliability of 3.6, a capacity reduction factor of 0.6 for grouted unreinforced masonry walls and 0.75 for grouted reinforced masonry wall was appropriate for structures whose imposed to permanent load ratios varied from 0.2 (lightly loaded structure) to 1.0 (heavily loaded structure) — a range considered acceptable for masonry structures in Ref. 1 and 2.

C4.5 Cross-section properties

The cross-section properties of a prism, wall or pier are based on the area of the unit in contact with the bed joint mortar, the bedded area (A_b). For walls constructed from solid or cored units that are intended to be laid with full beds, the bedded area is calculated on the gross area of the units with an appropriate reduction for any raking out of the bed joints.

Hollow units are defined as those units intended to be laid on face-shell bedding. The bedded area and section modulus are calculated from the face-shell width, ignoring the presence of the webs. The face shell width is determined from the work size dimensions nominated by the manufacturer and is the same dimension that was used to test the units in accordance with AS/NZS 4456.4 (Ref. 6). Although some hollow units are laid with full bed joints, they have to be designed on the basis of face-shell bedding. However, when calculating an average moment of inertia for the determination of deflections, some allowance for the presence of webs may be made.

The mechanism of failure of face-shell-bedded masonry loaded in compression is different from that for the fully bedded case. Face-shell-bedded masonry fails by tensile splitting of the transverse webs of the units, induced by deep-beam bending of the webs spanning between the loaded face shells. In contrast, fully bedded masonry fails in a vertical splitting mechanism.

C4.5.1 Bedded thickness

(No Commentary)

C4.5.2 Effective width of compression faces and flanges

(No Commentary)

C4.5.3 Structural end of a masonry member

(No Commentary)

C4.5.4 Bedded area

(No Commentary)

C4.5.5 Combined cross-sectional area

In hollow unit masonry that is partially grouted (that is, with some cores grouted and others not), the “portion of the length represented by the grouted cores” extends to the centre-lines of the webs on either side of the grouted cores, even though there may be some vertical discontinuities down the line of these webs over the height of the member.

C4.5.6 Design cross-sectional area

(No Commentary)

C4.5.7 Grout area

(No Commentary)

C4.5.8 Section modulus and section moment of area

(No Commentary)

C4.5.9 Chases, holes and recesses

Attention is required for cored and hollow masonry that are to be chased. Typically, chasing in cored and hollow units can result in the removal of a significant section of the unit, which can encroach on and in some cases result in the cores of units being exposed. This matter should be considered and pre-emptively addressed during the planning and designing stage.

C4.6 Design for robustness**C4.6.1 General**

The robustness provisions are a practical means of providing an upper limit on the size of walls and isolated piers, to ensure that unreasonably slender members are not built. The robustness limits are not a substitute for rational analysis and design for calculated loads, but rather a global limit beyond which even the most lightly loaded walls and piers cannot be built.

The robustness provisions have been derived by giving consideration to the current forms of construction in use (e.g. the increasing use of 90 mm masonry units in partition walls and cavity walls), and a lateral pressure of 0.5 kPa. The likelihood of chasing and its possible effect in reducing support needs to be taken into account.

C4.6.2 Robustness of walls

(No Commentary)

C4.6.3 Robustness of isolated piers and unreinforced piers

The transition from an isolated pier to a wall occurs at a height to length ratio of 5:1 (refer to Clause 1.5.2.29).

C4.7 Prevention of moisture penetration**C4.7.1 Cavities**

Where a wall or other masonry member is constructed with a cavity to limit moisture penetration, it is recognized that moisture can penetrate through the outer or exposed leaf of the construction, and may flow down the inner (cavity) surface of that outer leaf. The cavity and its detailing, including drainage holes, flashing, wall ties, and other items that cross or bridge the cavity have to be such that this moisture will drain adequately from the cavity and will not permeate, flow or be blown across to the dry side of the cavity. When so constructed, cavities provide a fully effective barrier to the further penetration of moisture horizontally through the masonry.

Wiring, service pipes, insulation or other material may be accommodated within a cavity, provided the moisture resistance of the cavity is maintained.

The Standard does not place limits on the width of cavities. If narrow cavities (less than about 40 mm in width) are used, additional measures need to be taken to ensure that moisture cannot pass across the cavity.

The use of cavities smaller than 40 mm in width incurs the risk of mortar squeezed out from the bed joints bridging the cavity. If narrow cavities are required for architectural or other reasons, consideration should be given to providing a waterproof membrane between the leaves, but this has to allow for placement of the wall ties. When detailing such an arrangement, penetration of the membrane by the ties cannot allow moisture transfer across the cavity.

Certain kinds of thermal insulation can be placed in cavities, even completely filling the cavity, without the moisture resistance of the masonry being impaired. Other kinds of insulation might allow moisture to cross the cavity. Cavity insulation needs to be appropriately selected, in both material and form, and placed so that the moisture resistance of the masonry is maintained. Note that the applicable building regulations might also place limits on cavity width and the placement of insulation in cavities.

C4.7.2 Weepholes

Weepholes are used to drain cavities in cavity wall construction, and to provide a passage for water that may otherwise build up behind retaining walls and the like. Weepholes have to be of such location and size as will allow free passage of all the water to be discharged.

Weepholes are usually formed by omitting the mortar in selected perpend joints; they should be detailed so that they cannot become blocked. Weepholes at the bottom of a wall have to be located above the level of likely soil build-up. Consideration may also need to be given to vermin-proofing.

Where strong wind forces might prevent the escape of water from weepholes, e.g. in tall or exposed structures, special attention should be given to the detailing of the weepholes and the adjacent cavity flashing to prevent build-up of water from crossing the cavity.

C4.7.3 Damp-proof courses (DPCs) and flashings

Flashings and damp-proof courses typically have a weakening effect on the structural capacity of masonry, particularly on the flexural tensile capacity of the bed joints into which the flashing is set. At these locations, the flexural tensile strength have to be taken as that applicable to interfaces between masonry and other materials (refer to Clause 3.3.3).

C4.7.4 Single leaf and solid walls

Single leaf masonry and solid masonry walls might not be impervious to moisture from rain on the exposed face, although many older solid walls with no cavity continue to provide a satisfactory waterproof enclosure. In the Standard, single leaf masonry walls are not regarded as being impervious to moisture from rain on the exposed face, and are required to be further protected by a suitable coating. In the 2018 edition of the Standard, several acceptable coating systems are specified. Additional protection may be provided by an overhanging shelter.

C4.8 Control joints

C4.8.1 General

Consideration of control joints is an integral part of masonry design. For every job it should be decided whether control joints are necessary and, if so, where they are to be located. The locations of control joints have to be shown clearly on the drawings.

Control joints can have a substantial impact on other design requirements for a masonry member and for the associated structure. Because vertical control joints create a free end of the masonry, which may increase slenderness and decrease support, the location of joints have to be determined early in the design process and in conjunction with structural design requirements. The detailing of the joints should also take into account aesthetic requirements and any requirements for resistance to moisture.

penetration, fire, heat and sound. Since joint fillers usually deteriorate more rapidly than the masonry, the economy and ease of ongoing maintenance of control joints should also be considered.

The Standard has provisions for spacing and location of contraction joints, expansion joints and articulation joints. Further information on design procedures and detailing is available in masonry industry publications [see Commentary C4.8.1(a) and (b) below].

Control joints include the following types:

- (a) Contraction joints are opening joints, usually built into concrete, calcium silicate or AAC masonry to cater for shrinkage of the wall. In a wall exposed to the weather, contraction joints have to be sealed with a flexible sealant at the surface. They may also be filled with a compressible material if they are required to serve also as expansion joints (for thermal movement) or as articulation joints (for footing or support movement).

Recommendations are available for spacing and detailing of contraction joints in concrete masonry (Ref. 7). Refer to manufacturer's literature for joints in AAC masonry.

- (b) Expansion joints are closing joints, usually built into clay masonry to cater for brick growth and/or thermal movement. Expansion joints incorporate a compressible material of sufficient thickness to cater for the expansion in the walls adjacent to the joint.

Recommendations are available for procedures to calculate maximum spacing of control joints in clay masonry for various situations (Ref. 8). The procedures take into account the maximum movement allowed to occur at the joints (as specified in Clause 4.8.1), the expansion characteristics of masonry, differential thermal expansion and, where appropriate, the shrinkage or shortening of adjacent structures or materials.

- (c) Articulation joints are opening and closing joints, built into all types of masonry to cater for movement of the footings or supports. The strategic positioning of articulation joints at points of weakness (e.g. door or window openings) will minimize cracking as the supports move due to foundation movement and similar actions. In some cases, contraction and expansion joints may be used as articulation joints. Recommendations are available for the spacing and locations of articulation joints (Ref. 9).

C4.8.2 Contraction joints

(No Commentary)

C4.8.3 Expansion joints

(No Commentary)

C4.8.4 Articulation joints

(No Commentary)

C4.8.5 Detailing

(No Commentary)

C4.9 Mortar joints

C4.9.1 Thickness

Clause 4.9.1 specifies the "design thickness" of mortar joints, which has to be used for dimensioning and setting out of masonry, and has to be the average thickness of the mortar joints in the constructed masonry.

The actual thickness of any individual joint may differ from the design thickness, because of the need to accommodate variability in the dimensions of the masonry units, and for other constructional tolerance requirements. The tolerances specified in Table 12.1 take this into account.

Both the compressive strength and the flexural tensile strength of masonry are reduced as the thickness of the mortar bed joints in the masonry is increased.

C4.9.2 Finishing

Mortar joints that are not tooled are more likely to absorb moisture together with salts and other airborne or waterborne chemicals, and to collect dirt and degrading organisms, with consequential adverse effects on moisture resistance, durability and appearance.

C4.10 Wall ties

The Standard does not provide deemed-to-satisfy values for spacing of wall ties. Emphasis is placed on designing ties and tie spacing to suit the loads that they are presumed to carry.

The Standard contains a requirement for screw fixing of face-fixed veneer ties in masonry veneer more than 3 m above ground. This is in response to research which shows that veneer ties fixed with nails can become loose under repeated loading (Ref. 10). The fixing screw is required to be supplied by the manufacturer.

C4.11 Bonding, tying and supporting

C4.11.1 General

The Standard sets out details relating to two of the various ways in which monolithic structural action can be achieved across an interface between two members or between two parts of a member. The two ways are —

- (a) by masonry header units built across the interface, which themselves develop the required monolithic action (or shear strength) by virtue of their own shear strength capacity; and
- (b) by metal ties, which enable monolithic action to be achieved by holding or tying together the masonry on each side of the interface, with the longitudinal shear capacity being developed by shear or mechanical interlock in the vertical mortar joint, which has to be completely filled.

Monolithic structural action is typically required across vertical joints as follows:

- (i) Between the leaves in solid masonry construction.
- (ii) At engaged piers.
- (iii) At the intersections of shear walls with returns or cross-walls.
- (iv) At the engagement of diaphragms with the leaves in diaphragm walls.

C4.11.2 Bonding with masonry header units

(No Commentary)

C4.11.3 Tying with connectors

(No Commentary)

C4.11.4 Fixing to supporting structures

Reference should be made to suppliers or manufacturers of fixings and anchors to obtain the characteristic strength capacities of their fixings in the relevant type of masonry or masonry unit.

Fixings and anchors have to be positioned and anchored in a masonry member so that no part of the masonry is subject to loads or load effects in excess of those permitted by the Standard. The requirements of Clause 2.6.4 apply to the design of such anchorages. Where an uplift force is to be resisted by hold-down bolts or anchors, these have to extend to such a depth into the masonry that no uplift tension stresses develop in that masonry. Particular attention should be paid to the sizing (and therefore spacing) of head ties and column ties.

C4.12 Stack bonded masonry

C4.12.1 Solid and cored unit masonry

Stack bonded masonry is a form of construction in which the masonry units in adjacent courses are aligned vertically above one another. Specifically, the Standard has introduced a definition for stack bonded masonry being masonry in which the overlap of masonry units in successive courses is less than $\frac{1}{4}$ the unit length or 50 mm, whichever is the greater. This bonding pattern leads to continuous vertical joints running the full height of the wall resulting in a weak form of construction particularly when subjected to lateral out-of-plane loading. The use of stack bonded masonry in Australia is not encouraged and consequently, in previous editions of AS 3700, no specific design provisions were included. However, with its increasing use either as a veneer or a structural element, specific design provisions have now been included in the Standard.

Research at The University of Newcastle has established that, provided the correct design and construction procedures are followed, stack bonding can be used provided it is accompanied by appropriate bed joint reinforcement (Ref. 11, 12 and 13). The new design provisions can be summarized as follows:

- (a) The bed joint reinforcement needs to be properly anchored and of the appropriate durability class with an area not less than 0.00035 times the gross area of the wall.
- (b) The reinforcement is required to be continuous between lateral supports and spaced vertically at centres not exceeding six times the thickness of the stack bonded leaf.
- (c) The reinforcement is required to have an overall diameter or thickness not less than 3 mm and not greater than two-thirds of the thickness of the mortar joint.
- (d) The masonry is designed as reinforced for horizontal bending and unreinforced for compression, shear and vertical bending.

C4.12.2 Hollow unit masonry

Hollow unit stack bonded masonry is required to be either reinforced or prestressed in accordance with Section 8 or 9.

C4.13 Arches and lintels

When lintels are placed beneath several courses of masonry, the applied vertical load may be calculated assuming a triangular area of masonry acting vertically on the lintel, provided that there is sufficient depth of masonry over the opening and there is sufficient buttressing and a substantial load transferred by arching action back to the supports.

While the strength capacity of lintels should always be checked, the ability of a lintel to support the masonry above is most often governed by deflection, not strength. The 2018 edition of the Standard introduced deflection limits for lintels under serviceability loads.

Recommendations for the design of lintels in clay masonry are given in Ref. 8.

C4.14 Interaction between masonry members and slabs, beams or columns

Given that reinforced concrete and reinforced concrete masonry have similar properties, composite action can be achieved using starter bars set in the concrete.

The 2018 edition of the Standard introduced a requirement for slip joints under concrete slabs supported by unreinforced masonry, with a choice of alternative specifications that are deemed to provide the necessary capacity for differential movement.

C4.15 Corbelling

(No Commentary)

C4.16 Attachment to face of walls

(No Commentary)

C4.17 References and bibliography

C4.17.1 References

- [1] LAWRENCE, S.J. and STEWART, M., Reliability-based calibration of the capacity reduction factor for design of masonry in compression to AS 3700, *Australian Journal of Structural Engineering*, 2009, 9(2), pp. 97–110
- [2] STEWART, m.G., and LAWRENCE, S.J. Model error, structural reliability and partial safety factors for structural masonry in compression, *Masonry International*, 2007, 20(3), pp. 107–116
- [3] ELLINGWOOD, B. Analysis of reliability for masonry structures. *Journal of the Structural Division, ASCE*, 1981, 107, pp. 757–773
- [4] MOOSAVI, H. and KORANEY, Y. Assessment of the structural reliability of loadbearing concrete masonry designed to the Canadian Standard S304.1, *Canadian Journal of Civil Engineering*, 2014, 41, pp. 1046–1053
- [5] KAZEMI, S., MAHOUTIAN, H., MOOSAVI, H. and KORANI, Y. *Reliability analysis of masonry members under compression*, Structures Congress, ASCE, 2011, pp. 605–615
- [6] AS/NZS 4456.4, *Masonry units and segmental pavers and flags—Methods of test*, Part 4: *Determining compressive strength of masonry units*, Standards Australia, Sydney, 2009
- [7] *Design and Construction of Concrete Masonry Buildings*. MA55_E8, Concrete Masonry Association of Australia, Sydney
- [8] *Detailing of Clay Masonry*. Manual 9, Think Brick Australia, Sydney, 2019
- [9] *Articulated Walling*. Technical Note 61, Cement Concrete and Aggregates Australia, Sydney, 2008
- [10] SIMUNDIC, G., PAGE, A.W. and NEVILLE, T.L., *The behaviour of wall ties under cyclic loading*. Proceedings of the 8th North American Masonry Conference, The Masonry Society, Austin, Texas, 1999, pp. 736–764
- [11] MASIA M.J., SIMUNDIC G., PAGE A.W. *The Flexural Strength of Stack Bonded Masonry, Stage 1 – a Preliminary Study*, Proc. 9th International Masonry Conference, Guimaraes, Portugal, 2014
- [12] MASIA M.J., SIMUNDIC G., Page A.W. *The Flexural Strength of Stack Bonded Masonry – an On-going Study*, Proc. 12th North American Masonry Conf., Denver, Colorado, May, Paper 0057, 2015

- [13] MASIA M.J., SIMUNDIC G. and PAGE A.W. *Flexural strength of stack bonded masonry in one way horizontal bending: influence of bed joint reinforcement*, 16th International Brick and Block Masonry Conference, Padua, Italy, 2016
- [14] DHANASEKAR M., RING J., SONG M., *Effectiveness of Vertical Steel Reinforcing Bars to Reinforced Concrete Masonry Walls Under Compression*. Concrete, Adelaide, 2017
- [15] DHANASEKAR M., SONG M., RING J., *Response of Reinforced Concrete Masonry Walls to Eccentric Compression*, Proc. 13th Canadian Masonry Symposium, Halifax, Canada, 2017
- [16] ZHAI, X.M. and STEWART, M.G. *Structural Reliability Analysis of Reinforced Grouted Concrete Block Masonry Walls in Compression Designed to Chinese Code GB 50003*, Research Report 268.08.08, 2008

C4.17.2 Bibliography

BAKER, L.R., LAWRENCE, S.J. and PAGE, A.W. *Australian Masonry Manual*. PWD/ASCE (NSW) Joint Committee, Sydney, 1991

Structural Reliability Verification Method, Australian Building Codes Board, 2015

<https://www.abcb.gov.au/Resources/Publications/Education-Training/Structural-Reliability>