COMMITTEE BD-004

DR AS 3700:2017

(Project ID: 104048)

Draft for Public Comment Australian Standard

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BEGINNING DATE FOR COMMENT:	**26 June 2017**
CLOSING DATE FOR COMMENT:	**28 August 2017**

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Masonry structures

(Revision of AS 3700—2011)

STANDARDS Australia

Draft for Public Comment
Australian Standard

The committee responsible for the issue of this draft comprised representatives of organizations interested in the subject matter of the proposed Standard. These organizations are listed on the inside back cover.

Comments are invited on the technical content, wording and general arrangement of the draft.

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Please place relevant clause numbers beside each comment.

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The coordination of the requirements of this draft with those of any related Standards is of particular importance and you are invited to point out any areas where this may be necessary.

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Draft for Public Comment

STANDARDS AUSTRALIA

Committee BD-004—Masonry structures

DRAFT

Australian Standard

Masonry structures

(Revision of AS 3700—2011)

(To be AS 3700:201X)

Comment on the draft is invited from people and organizations concerned with this subject. It would be appreciated if those submitting comment would follow the guidelines given on the inside front cover.

Important: Please read the instructions on the inside cover of this document for the procedure for submitting public comments

This document is a draft Australian Standard only and is liable to alteration in the light of comment received. It is not to be regarded as an Australian Standard until finally issued as such by Standards Australia.

PREFACE

This Standard was prepared by the Standards Australia Committee BD-004, Masonry Structures, to supersede AS 3700—2011.

The objective of this Standard is to provide minimum requirements for the design and construction of unreinforced, reinforced and prestressed masonry, including built-in components.

The principal changes from editions of the Standard prior to 2011 include the following:

(a) Increased capacity reduction factor for compression.

(b) Increased strength factor for M4 mortars.

(c) Added a definition for adequate lateral support.

(d) Added specification for deemed-to-satisfy waterproof coatings.

(e) Added guidance on design of control joint spacings.

(f) Mandatory screw fixing of face-fixed masonry veneer ties in some locations.

(g) Restructured durability requirements.

(h) Improved requirements for fire protection of structural steelwork.

(i) Removed allowance for panel action in masonry walls designed for compression.

(j) Improved ease of design for masonry walls in compression.

(k) Added provisions for design of cavity walls under lateral loading.

(l) Improved strength design provisions for wall ties.

(m) Moved the earthquake detailing provisions from an appendix into the Standard.

(n) Improved and clarified provisions for verification of properties during construction.

(o) Simplified the provisions for tolerances during construction.

(p) Removed former Section 12 on simplified design of masonry for small buildings (refer to AS 4773.1, *Masonry in small buildings,* Part 1: *Design*).

(q) Rearranged and renumbered the appendices.

(r) Inclusion of provisions for stack bonded masonry.

(s) Minor changes to ensure consistency with AS 4773.

(t) New Appendix I on the relationship between ISO 9223 corrosivity categories and durability class. Also includes solutions for wall ties, connectors and accessories, and lintels and shelf angles as provided in AS/NZS 2699.1, AS/NZS 2699.2 and AS/NZS 2699.3 respectively.

Acknowledgment is made of the assistance gained from the publications made available to the Committee by its member organizations, *Think Brick Australia* and the *Concrete Masonry Association of Australia.*

Statements expressed in mandatory terms in notes to tables and figures are deemed to be requirements of this Standard.

The terms ‘normative’ and ‘informative’ have been used in this Standard to define the application of the appendix to which they apply. A ‘normative’ appendix is an integral part of a Standard, whereas an ‘informative’ appendix is only for information and guidance.

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SECTION 1 SCOPE AND GENERAL

1.1 SCOPE

This Standard sets out minimum requirements for the design and construction of masonry of the following types:

(a) Unreinforced, reinforced and prestressed masonry using manufactured units of clay or concrete laid in mortar.

(b) Unreinforced masonry using manufactured calcium silicate units laid in mortar.

(c) Autoclaved aerated concrete (AAC) masonry laid in thin-bed mortar.

(d) Square-dressed natural stone laid in mortar.

This Standard does not provide for the following:

(i) Design values or material properties for the design and construction of masonry incorporating square-dressed natural stone. The wide variations in properties of natural stone require that each case be considered individually for the determination of relevant design values.

(ii) Specific requirements for prefabricated masonry panels or masonry in composite action with steel or concrete structural members; however, the principles of this Standard may be used for such types of construction.

The provisions of this Standard do not include specification for design and construction of AAC laid in other than thin-bed mortar; however, for masonry so constructed the principles of this Standard may be used.

NOTE: This Standard assumes the structural design of masonry is entrusted to experienced structural engineers or similar appropriately qualified persons, and that the execution of such work is carried out under the direction of appropriately qualified persons who are experienced in masonry construction and who understand the structural requirements specified herein.

1.2 NORMATIVE REFERENCES

The following are the normative documents referenced in this Standard:

NOTE: Documents referenced for informative purposes are listed in the Bibliography.

AS	
1170	Structural design actions
1170.4	Part 4: Earthquake actions in Australia
1316	Masonry cement
1391	Metallic materials—Tensile testing at ambient temperature
1478	Chemical admixtures for concrete, mortar and grout
1478.1	Part 1: Admixtures for concrete
1530	Methods for fire tests on building materials, components and structures
1530.4	Part 4: Fire-resistance tests of elements of building construction

AS	
1672	Limes and limestones
1672.1	Part 1: Limes for building
2193	Calibration and classification of force-measuring systems
2701	Methods of sampling and testing mortar for masonry construction
2758	Aggregates and rock for engineering purposes
2758.1	Part 1: Concrete aggregates
2870	Residential slabs and footings
3582	Supplementary cementitious materials
3582.2	Part 2: Slag—Ground granulated blast-furnace
3600	Concrete structures
3972	General purpose and blended cements
4055	Wind loads for housing
4100	Steel structures
5100	Bridge design (all parts)
AS/NZS	
1170	Structural design actions
1170.0	Part 0: General principles
1170.1	Part 1: Permanent, imposed and other actions
1170.2	Part 2: Wind actions
1170.3	Part 3: Snow and ice actions
2699	Built-in components for masonry construction
2699.1	Part 1: Wall ties
2699.2	Part 2: Connectors and accessories
2699.3	Part 3: Lintels and shelf angles (durability requirements)
2904	Damp-proof courses and flashings
3582	Supplementary cementitious materials
3582.1	Part 1 Fly ash
3582.3	Part 3: Amorphous silica
4455	Masonry units, pavers, flags and segmental retaining wall units
4455.1	Part 1: Masonry units
4456	Masonry units, segmental pavers and flags—Methods of test
4456.4	Method 4: Determining compressive strength of masonry units
4456.11	Method 11: Determining coefficients of expansion
4456.12	Method 12: Determining coefficients of contraction
4456.15	Method 15: Determining lateral modulus of rupture
4671	Steel reinforcing materials
4672	Steel prestressing materials
4672.1	Part 1: General requirements
EN	
1996.1 and 2	
10088	Stainless Steel
10088-1	Part 1: List of Stainless Steels
12878	Pigments for the colouring of building materials based on cement and/or lime. Specifications and methods of test

6744 Stainless steel bars—Reinforcement of concrete—Requirements and test methods

1.3 USE OF ALTERNATIVE MATERIALS OR METHODS

Provided the requirements of Section 2 are met, this Standard shall not be interpreted in a way that prevents the use of alternative materials or methods of design or construction not specifically referred to herein.

NOTE: Where the intended use is subject to the control of a building authority, approval for the use of alternative materials or methods will need to be obtained from the relevant authority.

1.4 INFORMATION TO BE PROVIDED ON DOCUMENTS

1.4.1 General

Documents shall show the following information to construct the masonry:

(a) Category, type and work sizes of the masonry units as defined in Clauses 1.5.2.23 and 1.5.2.24. This shall include the face shell width for units that are to be face-shell bedded (see Clause 3.2).

(b) The characteristic unconfined compressive strength (f'_{uc}) required for the masonry units.

(c) Design thickness of the mortar joints, if other than 10 mm.

(d) Mortar class or mix proportions or, for proprietary thin-bed mortar, the relevant specification.

NOTE: If mortar is specified by class, a range of deemed-to-satisfy mortar mixes is given in Table 11.1. If it is desired to use a mix other than those deemed to satisfy, the standard requires that evidence of performance be provided (see Clause 11.4.1(ii)). In all cases, the designer should ensure that the requirements for mortar are adequately communicated to the builder.

(e) Finish and depth of raking (if any) of the mortar joints.

(f) Bond pattern for the masonry.

(g) Salt attack resistance grade of the masonry units (see Clause 5.5).

(h) Duty rating and durability class of wall ties (see Clause 3.4 and Clause 5.7).

(i) The strength and stiffness of accessories (see Clause 3.4).

(j) The characteristic lateral modulus of rupture (f'_{ut}) (see Clause 3.2).

(k) The coefficient of expansion or contraction of the units that has been used in the design.

(l) Values required for any other properties of the masonry or its components, where those values differ from values given in this Standard.

(m) For special masonry, where applicable, the values of the characteristic compressive strength (f'_{m}) and the characteristic flexural tensile strength (f'_{mt}) used in design of the masonry.

(n) For special masonry, the site control testing requirements for the masonry or its components.

(o) For reinforced masonry, the grade and type of reinforcement.

(p) For prestressed masonry, the grade and type of tendons.

(q) For grouted masonry—

(i) cement content;

(ii) the characteristic compressive cylinder strength of grout at 28 days (f'_c);

(iii) maximum aggregate size; and

(iv) workability or site mix proportions.

(r) Principal dimensions of the masonry, including the positions of supports, piers, openings, length of returns and all features affecting the strength and stability of the structure.

(s) Positions and details (including dimensions, durability rating, and material types) of the following:

(i) Subfloor or cavity ventilation.

(ii) Termite barriers.

(iii) Damp-proof courses (DPCs).

(iv) Flashings.

(v) Slip materials.

(vi) Control joints.

(vii) Lintels.

(viii) Roof anchorage.

(ix) Bracing to walls.

(x) Connectors.

1.4.2 Provision for demolition

A note shall be included on the documents for any building or structure that is identified as being one that cannot be demolished without danger of unintended collapse.

NOTE: For information on demolition of structures, see AS 2601.

1.5 DEFINITIONS

For the purpose of this Standard the definitions below apply.

1.5.1 Administrative definitions

1.5.1.1 *Approved*

Except as otherwise stated, approved by the building authority.

1.5.1.2 *Building authority*

Body having statutory powers to control the design and erection of buildings or structures in the area in which the buildings or structures are to be erected.

1.5.1.3 *Documents*

Drawings and specifications that set out the work to be executed.

1.5.1.4 *Supervision*

Site control of work directed towards obtaining compliance with the design and the provisions of this Standard.

1.5.2 Technical definitions

1.5.2.1 *Areas*

Bedded areas, combined cross-sectional areas, design cross-sectional areas and grout areas.

NOTE: See Clauses 4.5.4, 4.5.5, 4.5.6 and 4.5.7 for requirements relating to these areas.

1.5.2.2 *Bed face*

The face of the masonry unit that is in direct contact with bed joint mortar.

1.5.2.3 *Bed joint*

See joints.

1.5.2.4 *Bedding*

One of the following:

(a) *Bedding planes* Those planes in the masonry formed by the horizontal joints between the bed faces of the units.

(b) *Full bedding* Bedding covering the entire plan area of the masonry unit with mortar during laying.

(c) *Face-shell bedding* Bedding covering the plan area of the face shells, but not the webs, of hollow masonry units with mortar during laying.

1.5.2.5 *Bond beam*

Horizontal reinforced masonry member forming part of the masonry.

1.5.2.6 *Buttress*

An engaged pier that provides lateral support.

NOTE: See Clause 1.5.2.22; see also Clause 1.5.2.18 for dimensions.

1.5.2.7 *Cavity*

In members consisting of two leaves of masonry, the space between the leaves.

1.5.2.8 *Cement*

Cement complying with AS 3972 or AS 1316.

1.5.2.9 *Characteristic lateral modulus of rupture* (f'_{ut})

The characteristic value of the flexural tensile strength of the masonry unit obtained when subjected to bending in the direction normal to the plane of the wall.

1.5.2.10 *Characteristic value*

One of the following:

(a) *For strength properties*, the value of the material property that is exceeded by 95% of the material.

(b) *For coefficients of expansion and contraction*, the value that is exceeded by 5% of the material.

1.5.2.11 *Coefficients of expansion and contraction*

The coefficients of expansion and contraction of masonry units as determined in accordance with AS/NZS 4456.11 and AS/NZS 4456.12.

1.5.2.12 *Column*

See pier.

1.5.2.13 *Compressive strength*

One of the following:

(a) *For masonry units*, the characteristic unconfined compressive strength of masonry units (f'_{uc}) when tested in accordance with AS/NZS 4456.4.

(b) *For masonry*, the characteristic unconfined compressive strength of masonry (f'_m) determined in accordance with Clause 3.3.2.

1.5.2.14 *Concrete*

A mixture of portland cement, aggregate and water, with or without the addition of fly ash, ground granulated blast-furnace slag, silica fume and/or chemical admixtures.

1.5.2.15 *Connectors*

Masonry connectors of the following categories:

(a) *Collar joint tie* A tie connecting two masonry leaves through a collar joint.

(b) *Control joint tie* A tie connecting a leaf of masonry across a control joint for the purpose of maintaining alignment.

(c) *Head and column ties* Ties connecting masonry to a structural support.

1.5.2.16 *Control joint*

See joint.

1.5.2.17 *Core*

The void within the cross-section of a cored, hollow or horizontally cored masonry unit.

1.5.2.18 *Dimensions of buttresses and piers*

One of the following:

(a) *Width and thicknesses of an isolated pier* The larger and smaller horizontal dimensions, respectively.

(b) *Width of a buttress or engaged pier* Horizontal dimension parallel to the wall to which it is bonded.

(c) *Thickness of a buttress or engaged pier* Horizontal dimension at right angles to the wall to which it is bonded, including the wall thickness.

1.5.2.19 *Efflorescence*

Salts left on the surface of units after the evaporation of water. The surface deposits can take the form of either loose crystalline salts or amorphous films of non-crystalline salts.

1.5.2.20 *Grout*

Highly workable concrete placed in cores or cavities to produce grouted masonry.

1.5.2.21 *Joint*

One of the following:

(a) *Bed joint* A joint (typically horizontal) formed by the mortar on which the masonry units are laid.

(b) *Perpend joint* A joint (typically vertical) formed between adjacent masonry units laid in the same course.

(c) *Collar joint* A vertical joint filled with mortar between two leaves of masonry.

(d) *Control joint* A joint or gap constructed in masonry to control and absorb movements in that masonry. Control joints include the following:

(i) Contraction joint (or opening control joint), which opens as the masonry contracts or shrinks.

(ii) Expansion joint (or closing control joint), which closes as the masonry expands.

(iii) Articulation joint, which opens or closes to allow structural movements.

1.5.2.22 *Lateral support*

The support (including a footing, buttress, cross-wall, beam, floor, or braced roof structure) that effectively restrains a wall or isolated pier in the direction of its thickness, except that an isolated pier may also have lateral support in the direction of its width.

NOTE: A lateral support is required to satisfy Clause 2.7.

1.5.2.23 *Masonry*

Assemblage of masonry units properly bonded with mortar. The term includes the following:

(a) *Plain masonry* Masonry that is not grouted, reinforced or prestressed.

(b) *Grouted masonry* Masonry in which some or all of the cavities in the masonry, or cores in hollow units, are filled with grout.

(c) *Unreinforced masonry* Masonry that is not reinforced.

(d) *Reinforced masonry* Masonry in which some or all of the grouted cavities or cores are reinforced with steel reinforcement to comply with the provisions of Section 8.

NOTE: Accessories, including bed joint mesh, that do not comply with the requirements of Section 8 are not regarded as reinforcement for the purposes of this definition.

(e) *Prestressed masonry* Masonry in which some or all of the cavities or cores are reinforced with stressed tendons to comply with the requirements of Section 9.

(f) *Member of mixed construction* A member composed of more than one type of masonry system (e.g. unreinforced and reinforced), or more than one type of masonry unit, strength, durability grade or mortar.

(g) *Special masonry* Masonry with specified strength values higher than those specified in this Standard, which is tested during its construction to verify that the specified strength values have been achieved (see Clause 12.7).

NOTE: The methods of testing for compressive and flexural strength of masonry are given in Appendices C and D.

1.5.2.24 *Masonry unit*

A preformed component, intended for use in bonded masonry construction (see Figure 1.1). The term covers the following:

(a) Categories:

(i) *Solid unit* A unit without cores, but which may contain recesses (commonly called frogs) not greater than 10% of the gross volume, intended to be laid with full bed joints.

(ii) *Cored unit* A unit with cores, intended to be laid with its cores vertical and with full bed joints.

(iii) *Hollow unit* A unit with cores, intended to be laid with its cores vertical and with face-shell-bedded joints.

NOTE: Hollow units are tested using the face-shell area.

(iv) *Horizontally cored unit* A unit with cores, intended to be laid with its cores horizontal and with full bed joints.

(v) *Special purpose unit* A unit intended for a special purpose that does not fall within the categories of Items (i) to (iv).

(b) Types:

(i) *Calcium silicate masonry unit* A unit manufactured from calcium silicate and in accordance with AS/NZS 4455.1.

(ii) *Clay masonry unit* A unit manufactured from burnt clay or shale and in accordance with AS/NZS 4455.1.

(iii) *Concrete masonry unit* A unit manufactured from concrete and in accordance with AS/NZS 4455.1.

(iv) *Autoclaved aerated concrete masonry unit (AAC)* A unit manufactured from autoclaved aerated concrete and in accordance with AS/NZS 4455.1.

(v) *Natural stone masonry units* A unit of square-dressed natural stone in accordance with BS 5628-3.

(c) Dimensions:

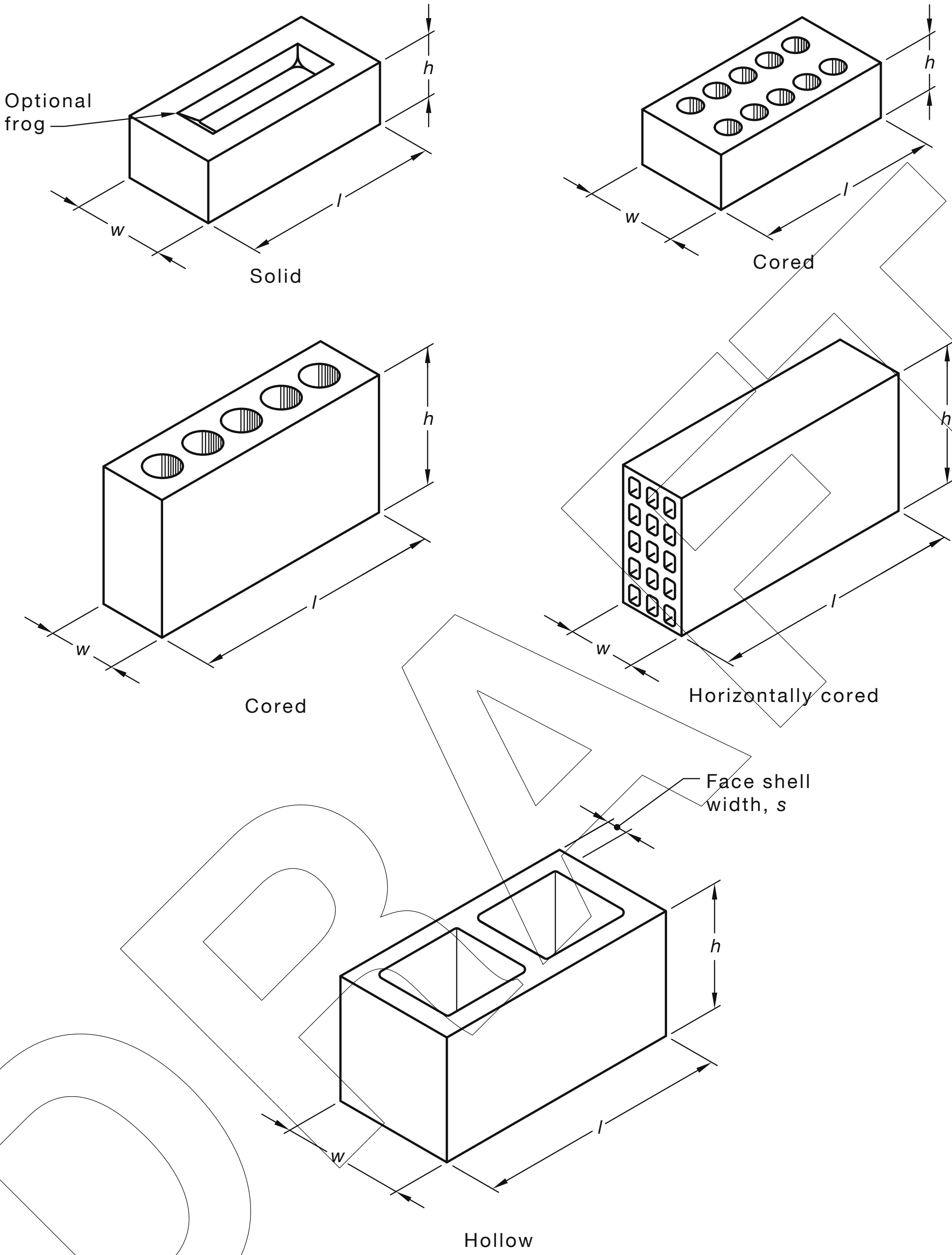
(i) *Height* The height based on work size (see h in Figure 1.1).

(ii) *Length* The length based on work size (see l in Figure 1.1).

(iii) *Width* The width based on work size (see w in Figure 1.1).

(iv) *Face shell width* The width of the face shell based on work size (see s in Figure 1.1).

(v) *Full bed width* The overall width of the unit based on work size (see w in Figure 1.1).



NOTES:

1 The figures above show only the basic shapes of the principal units. There are many commercially available 'fittings' and 'special' units used for particular applications.

2 The figures above are indicative only and are for the purpose of illustrating terms.

FIGURE 1.1 TYPICAL MASONRY UNITS

1.5.2.25 *Member*

A wall or portion thereof (including a pier, buttress, bond beam or lintel), or an isolated pier.

1.5.2.26 *Mortar*

One of the following:

(a) A mixture of cementitious material, sand (fine aggregate) and water, with or without additives or chemical admixtures.

(b) Thin-bed mortar—a proprietary material specifically manufactured to bond masonry units with a design joint thickness of not less than 2 mm and not greater than 4 mm.

1.5.2.27 *Mortar joints*

One of the following:

(a) *Flush joint* A joint that is finished flush with the surface of the masonry units.

(b) *Raked joint* A joint that is raked out to a specified depth behind the face of the masonry.

(c) *Tooled joint* A joint, including flush joint and raked joint, in which the surface is trowelled or ironed to a smooth, dense finish.

NOTE: Mortar joints for other than AAC are classified according to the types of finish given to their exterior surface.

1.5.2.28 *Pier*

One of the following (see also Clause 1.5.2.18 for dimensions):

(a) *Engaged pier* Pier bonded integrally with the wall, which it stiffens locally.

NOTE: An engaged pier has insufficient strength and stiffness to provide lateral support to the wall (see Clauses 2.6.3 and 7.3.4.3).

(b) *Isolated pier* Isolated vertical element whose width and thickness do not exceed one-fifth of the height.

NOTE: If either the width or thickness exceeds one-fifth of the height, the member is a wall.

1.5.2.29 *Prestressing tendon*

Any bar, wire or strand used to prestress masonry.

1.5.2.30 *Rotational restraint*

The restraint against rotation of a wall edge or isolated pier by supports that allow the transfer of moment.

1.5.2.31 *Shear wall*

A wall that carries in-plane lateral forces and transfers them to other parts of the structure.

1.5.2.32 *Structural backing*

One of the following:

(a) *Flexible structural backing* A structural system supporting a veneer and having a stiffness (EI) less than or equal to 0.5 times the uncracked stiffness of the veneer.

(b) *Stiff structural backing* A structural system that is not classed as a flexible structural backing.

1.5.2.33 *Surf coast*

An area of saltwater where breaking waves are a normal occurrence (see Clauses 5.3.1 and 5.3.2).

NOTES:

1 'Breaking waves' applies to beaches where surf breaks regularly and does not apply to wind-driven white caps or choppy water.

2 As an indication of 'normal occurrence' breaking waves on an average of 4 d week or 60% of the time would satisfy this definition.

1.5.2.34 *Ties*

Ties of categories and types as follows:

(a) Categories:

(i) *Veneer tie* A tie connecting a masonry veneer to a frame or wall designed to resist lateral forces.

(ii) *Cavity tie* A tie connecting two leaves of masonry that are separated by a cavity of any width.

(b) Types:

(i) *Type A tie* A tie designed primarily for wind loading.

(ii) *Type B tie* A tie designed primarily for seismic ductility.

NOTE: Type B ties are commonly used in New Zealand and are not generally available in Australia.

1.5.2.35 *Wall*

One of the following:

(a) *Cavity wall* A wall consisting of two leaves of masonry separated by a cavity.

(b) *Diaphragm wall* A wall consisting of at least two leaves of masonry with vertical masonry diaphragms joining the leaves, constructed so as to achieve monolithic structural action.

(c) *Masonry veneer wall* A wall consisting of a leaf of masonry together with a backing system that provides lateral support.

(d) *Single leaf wall* A wall whose thickness is equal to the width of the masonry unit used in that wall.

(e) *Solid wall* A wall whose thickness is greater than the width of one unit and in which the vertical joint between masonry leaves is filled solidly with mortar and which is intersected with header units or wire ties to give monolithic structural action.

(f) *Wall of geometric section* A wall, other than rectangular section, that is constructed so as to achieve monolithic structural action.

1.5.2.36 *Work sizes*

The sizes of the masonry unit specified for manufacture and from which deviations are measured.

NOTE: Work sizes refer to the principal dimensions of the masonry unit and are used for the calculation of all section properties in design and testing (see Figure 1.1).

1.6 NOTATION

Unless a contrary intention is expressed—

(a) the notations used in this Standard shall have the meanings given below with respect to the structure or member or condition to which a clause is applied;

(b) where non-dimensional ratios are involved, both the numerator and denominator are expressed in consistent units; and

(c) dimensional units used in expressions or equations shall be consistent unless otherwise specified.

A_b = the bedded area of a masonry member cross-section (Clause 4.5.4)

A_c = the cross-sectional area of a grouted or reinforced masonry member (Clause 4.5.5)

A_d = the design cross-sectional area of a member (Clause 4.5.6)

= the design cross-sectional area of the shear resisting portion of the member

A_{de} = the effective area of dispersion of the concentrated load on the member at mid-height (Figure 7.3 and Clause 7.3.5.4)

A_{ds} = the bearing or dispersion area of a concentrated load in a member at the design cross-section under consideration (Clauses 7.3.5.2, 7.3.5.3 and 7.3.5.4)

A_g = the design cross-sectional area of grout in a grouted or reinforced masonry member (Clause 4.5.7)

A_p = the cross-sectional area of prestressing tendons (Clauses 9.3.3.2 and 9.5.3)

A_s = the total cross-sectional area of the main reinforcement in a reinforced masonry member (Clauses 8.5, 8.7.2 and 8.8)

A_{sd} = the portion of the cross-sectional area of the main tensile reinforcement used for design purposes in a reinforced masonry member (Clause 8.6)

A_{st} = the cross-sectional area of fully anchored longitudinal reinforcement in the tension zone of the cross-section under consideration (Clauses 8.6)

A_{sv} = the cross-sectional area of shear reinforcement per spacing interval (Clauses 8.7.4 and 8.8)

a_f = an aspect factor (Clause 7.4.4.2 and Table 7.5)

a_v, a_h = slenderness coefficients for assessing slenderness ratio (Clauses 6.3.2.2, 7.3.3.4 and 7.3.4.3)

a_{vf} = a slenderness coefficient for assessing slenderness ratio in design for fire resistance for structural adequacy (Clause 6.3.2.2)

a_1 = the distance from the end of the wall or pier to the nearest end of the bearing area (Clause 7.3.5.4 and Figure 7.3)

B = a height factor used in calculating diagonal bending moment capacity (Clause 7.4.4.3)

b	=	the width of a masonry member of solid rectangular cross-section; or
	=	the effective width of a member in accordance with Clause 4.5.2
b_v, b_h	=	vertical and horizontal bending coefficients for out-of-plane loads on walls (Clause 7.4.4.4)
b_w	=	the width of the web of the shear-resisting area of a member; for a solid rectangular cross-section, $b_w = b$ (Clause 8.8)
C_c	=	the coefficient of creep (Clause 9.3.3.2)
C_i	=	a coefficient evaluated from the test results (Clause 6.5.4)
C_s	=	a coefficient evaluated from the test results (Clause 6.3.3)
C_v	=	robustness coefficient (Clause 4.6.3)
D	=	the overall depth of a cross-section in the plane of bending (Clause 8.9)
d	=	the effective depth of a reinforced masonry member or wall (Clauses 8.3, 8.6 and 8.8)
d_p	=	the effective depth of the tendon (Clauses 9.2.2, 9.5.1, 9.5.2 and 9.5.3)
d_{sc}	=	the diameter of shear connector (Clause 7.5.6)
d_1, d_2	=	the eccentricities of masses used in the bond test (Paragraph D6.4, Appendix D)
E_m	=	the modulus of elasticity of masonry (short-term loading) (Table 3.4)
E_L	=	the modulus of elasticity of masonry (long-term loading) (Table 3.4)
E_p	=	the modulus of elasticity of prestressing tendons (Clause 3.7.2)
E_s	=	the modulus of elasticity of reinforcement (Clause 3.6.2)
E_I	=	stiffness of a structural system (Clause 1.5.2.33)
e	=	the eccentricity of prestress (Clause 9.5.4)
e_1, e_2	=	the eccentricities of vertical force (Clause 7.3.4.5 and Table 7.3)
e_m	=	coefficient of expansion of the masonry unit determined in accordance with AS/NZS 4456.11
F_d	=	the design compressive force acting on the cross-section of a member, assessed from loads complying with Clause 2.6 for strength (in unreinforced masonry, F_d is typically that value of the design compressive force that acts simultaneously with a bending moment, shear force or other load action)
F_{dt}	=	the design tension force acting on the cross-section of a member, assessed from loads complying with Clause 2.6 for strength (Clauses 8.10 and 9.7)
F_o	=	the basic compressive strength capacity of a plain or grouted masonry cross-section (Clause 7.3.2)
F_{sp}	=	the total load at which the specimen fails (Paragraphs C7.2, Appendix C and D6.6, Appendix D)

F_t = the mean strength of the tie in accordance with Tables 3.5 and 3.6, Clauses 3.4, and 7.7.4

F_{td} = the design compressive or tensile tie force (Clauses 7.7.4)

f' = a characteristic strength value (Paragraph B2, Appendix B)

f'_c = the characteristic compressive cylinder strength of grout at 28 days (Clause 3.5)

f'_{cg} = the design characteristic compressive strength of the grout (Clause 3.5)

f'_m = the characteristic compressive strength of masonry (Clause 3.3.2)

f'_{mb} = the characteristic compressive strength of masonry specimens for masonry units whose ratio of height to mortar bed joint thickness is 7.6 (Clause 3.3.2)

f'_{mp} = the characteristic compressive strength of prestressed masonry at the transfer of prestress (Clause 9.5.1)

f'_{mg} = the characteristic compressive strength of grouted masonry specimens manufactured and tested in accordance with Appendix C and determined in accordance with Appendix B (Clause 7.3.2)

f'_{ms} = the characteristic shear strength of masonry (Clause 3.3.4)

f'_{mt} = the characteristic flexural tensile strength of masonry (Clause 3.3.3)

f'_t = the equivalent characteristic torsional strength (Clause 7.4.4.3)

f'_{uc} = the characteristic unconfined compressive strength of masonry units (Clauses 3.2)

f'_{ut} = the characteristic lateral modulus of rupture of masonry units (Clause 1.5.2.9)

f'_{vm} = the characteristic shear strength of reinforced masonry (Clause 8.8)

f_d = the minimum design compressive stress on the bed joint at the cross-section in a member (Clauses 7.4.3.2 and 7.4.3.3)

= F_d/A_d

f_{ksp} = the (lower) 5 percentile value for a set of test results (Paragraph B2, Appendix B)

f_p = the tensile strength of prestressing tendons (Clause 3.7.1 and Table 3.8)

f_{pe} = the effective stress in the tendon after losses (Clause 9.5.2)

f_{pt} = the stress in prestressing tendons at transfer (Clause 9.3.3.2)

f_{py} = the yield strength of prestressing tendons (Clause 3.7.1)

f_{sp} = the unconfined compressive or flexural strength of a masonry test specimen (Paragraphs B2, Appendix B, C7.2, Appendix C and D6.6 and D7.2, Appendix D)

$f_{sp\ell}$ = the least value of a set of specimen test results (Paragraph B2, Appendix B)

$f_{sp}(1), f_{sp}(i), f_{sp}(n)$	=	the group of test results in the assessment of characteristic value (Paragraph B2, Appendix B)
f_{sy}	=	the design yield strength of reinforcement (Clause 3.6.1 and Table 3.7)
f_{vr}	=	effective shear strength (Clause 8.7.2)
f_{vs}	=	the design shear strength of main reinforcement (Clause 8.8)
f_{yc}	=	the characteristic tensile yield strength of the connector (Clause 7.5.6)
G	=	the assumed slope of the crack line (Clause 7.4.4.2)
G_g	=	the gravity load (Clause 7.5.5.2)
H	=	the clear height of a member between horizontal lateral supports (Clause 6.3.2.2)
	=	for a member without top horizontal support, the overall height from the bottom lateral support (Clause 6.3.2.2)
	=	the height of the member perpendicular to the direction of the shear load (Clause 8.7)
H_d	=	the design height (Clause 7.4.4.2)
H_1	=	the height of taller opening (Clause 7.3.4.3)
h_u	=	the height of masonry unit in a wall (Clauses 7.4.3.4 and 7.4.4.2)
j	=	the number of results remaining in assessment of strength values from test results (Paragraph A3.2, Appendix A)
K_{wt}	=	wall tie stiffness coefficient (Clause 7.7.3)
k	=	a reduction factor for slenderness and eccentricity (Clauses 7.3.3.2, 7.3.4.2 and 7.3.4.5)
k_1, k_2	=	coefficients to assess lateral load capacity (Clause 7.4.4.2 and Table 7.5)
k_3, k_4	=	coefficients used for prestressed bending moment (Clause 9.5.2)
k_a	=	the aspect ratio factor for the specimen (Paragraph C7.2 and Table C1, Appendix C)
k_b	=	the concentrated bearing factor (Clause 7.3.5.4)
k_c	=	a strength factor for grout in compression (Clauses 7.3.2 and 8.5)
k_h	=	a factor reflecting the influence of the ratio of the masonry unit height to mortar bed joint thickness, used in assessing characteristic compressive strength for masonry other than special masonry (Table 3.2)
k_k	=	a characteristic value factor used to assess a characteristic strength value of a population from a set of representative test results (Paragraph B2 and Table B1, Appendix B)
k_m	=	a factor used to derive the characteristic compressive strength of masonry, reflecting the influence of masonry unit type and bedding type (see Clause 3.3.2 and Table 3.1)
k_p	=	the perpend spacing factor (Clause 7.4.3.4)

k_s	= a reduction factor used in assessing the capacity of reinforced members in compression (Clause 8.5)
k_{sp}	= masonry pier strength factor (Paragraph D7.2, Appendix D)
k_{sw}	= a reduction factor used in assessing the design overturning capacity of a wall (Clause 8.7.4)
k_t	= a thickness coefficient (Clauses 7.3.3.4, 7.3.4.3 and Table 7.2)
k_u	= the neutral axis parameter, being the ratio, at ultimate strength under any combination of bending and compression of the depth to the neutral axis from the extreme compressive fibre, to d (Clauses 8.3 and 9.2.2)
k_v	= a shear factor (Clause 3.3.5 and Table 3.3)
L	= the clear length of a wall between vertical lateral supports (Clause 7.4.4.4); or
	= for a wall without a vertical support at one end or at a control joint, the length to that unsupported end or control joint (Clause 7.4.4.4)
	= the clear length of the wall or pier (Clause 7.3.5.4)
	= the length of the member in the direction of the shear load (Clause 8.7.4)
L_d	= the design length (Clause 7.4.4.2)
L_e	= the effective dispersion length of the load (Figure 7.3)
L_{es}	= the percentage loss of prestress due to masonry creep and moisture movement (Clause 9.3.3.2)
L_o	= the length of opening (Clause 7.4.4.2)
ln	= the natural logarithm (Clause 6.3.3)
l	= the distance between the anchorages at the ends of the member (Clause 9.5.2)
l'	= the distance from the centroid of the reinforcement under consideration to the tensile end of the member (Clause 8.7.4)
l_u	= the length of masonry unit (Clause 7.4.4.2)
M_{cd}	= the diagonal bending moment capacity per unit length of diagonal crack line (Clause 7.4.4.3)
M_{ch}	= the horizontal bending moment capacity of a wall (Clause 7.4.3.2)
M_{cr}	= the ultimate cracking bending moment (Clause 9.5.4)
M_{cv}	= the vertical bending moment capacity of a member (Clause 7.4.2)
M_d	= the design bending moment acting on the cross-section of a member, assessed from loads complying with Clause 2.6 for strength (Clauses 8.6 and 9.5.1)
M_{dh}	= the design horizontal bending moment resulting from transient out of plane forces acting on a wall in horizontal-spanning action (Clause 7.4.3.2)
M_{dp}	= the bending moment capacity of masonry wall (Clause 9.5.1)

M_{dv}	=	the design vertical bending moment resulting from transient out-of-plane forces acting on a member in vertical spanning action (see Clause 7.4.2)
M_{sp}	=	the bending moment about the centroid of the bedded area of the test joint at failure in a masonry test specimen (Paragraphs D6.6 and D7.2, Appendix D)
M1, M2, M3, M4	=	the classification of mortar types (Clause 11.4 and Table 11.1)
m_1, m_2, m_3	=	the masses of components used in flexural strength testing (Paragraphs D6.6, Appendix D)
n	=	the number of test results under consideration (Paragraph B2, Appendix B)
P	=	the effective prestressing force (after losses) (Clause 9.5.4)
P_v	=	the applied uniform vertical load (Clause 8.7.4)
R1, R2, R3, R4, R5	=	classification of corrosion resistance ratings (Table 5.1)
R_{f1}, R_{f2}	=	restraint factors (Clause 7.4.4.2)
r	=	the width of the connector (Clause 7.5.6)
r_h	=	the proportion of the vertical shear plane that is intersected by masonry header units (Clause 3.3.4)
S_p	=	structural performance factor for earthquake design (Clause 10.2.2)
S_r	=	the slenderness ratio of a member (Clause 7.3.4.3)
S_{rf}	=	the slenderness ratio of a member, used in design for fire resistance for structural adequacy (Clause 6.3.2.2)
S_{rs}	=	the simplified slenderness ratio of a member (Clause 7.3.3.3)
s	=	the spacing of shear reinforcement along the member (Clause 8.8)
s_p	=	the minimum overlap of masonry units in successive courses (Clause 7.4.3.4)
t	=	the overall thickness of a masonry member cross-section perpendicular to the principal axis under consideration (Clauses 6.3.2.2, 7.3.3.4, 7.3.4.3)
t_b	=	the bedded thickness of a member (Clause 4.5.1)
t_c	=	the period to failure in relation to insulation (Clause 6.5.4)
t_f	=	the period to failure in relation to structural adequacy (Clause 6.3.3)
t_{fs}	=	the thickness of the face shell for hollow block masonry or the flange thickness for diaphragm walls [Clause 7.3.4.5(b)]
t_j	=	the mortar joint thickness (Clause 7.4.4.2)
t_m	=	the material thickness of member (Clause 6.5.4)
t_r	=	the minimum thickness of the pier (Clause 4.6.3)
t_s	=	the thickness of face shell of a hollow masonry unit at the thinnest point (Clause 7.4.4.3)
t_u	=	the width of masonry unit (measured through the wall) (Clauses 7.4.3.4, 7.4.4.3 and Paragraph D6.6, Appendix D)

t_w = the overall thickness of a wall or isolated pier (Clause 7.3.4.5)

t_{wp} = the overall thickness of a masonry wall plus an engaged pier or buttress (Table 7.2)

u = the thickness of the connector (Clause 7.5.6)

V = the coefficient of variation in a set of specimen test results (Paragraph B3, Appendix B)

V_c = the design shear strength of a shear connector (Clause 7.5.6)

V_d = the design shear force acting on the cross-section of a member, assessed from loads complying with Clause 2.6 for strength (Clauses 7.5.4 and 8.7.2)

V_o = the shear bond strength of an unreinforced masonry cross-section (Clause 7.5.4)

V_{sc} = the characteristic shear strength of a connector (determined by tests) [Clause 7.5.6(b)]

V_1 = the shear friction strength of the section (Clause 7.5.4)

w_d = the total design wind, earthquake or similar pressure acting on a wall (Clauses 7.4.4.2 and 7.4.4.4)

X_1, X_i, X_n = a group of test results in the assessment of strength values where n is the number of test results (Paragraph A3.2, Appendix A)

X_i = the ith result from a group of test results (Paragraph A3.2, Appendix A)

X_m = the mean of a group of test results (Paragraph A4, Appendix A)

X_1, X_i, X_j = the group of test results in the assessment of strength values following the exclusion of suspect values from the set of results (Paragraph A3.2, Appendix A)

x = the depth of compression zone (Clause 9.5.1)

Y_i = the natural logarithm of the remaining test results (Paragraph A3.2, Appendix A)

Y_m = the mean of the logarithms (Paragraph A3.2, Appendix A)

Y_s = the standard deviation of the logarithms (Paragraph A3.2, Appendix A)

Y_u = the upper rejection limit in the assessment of strength values from test results (Paragraph A3.2, Appendix A)

Y_l = the lower rejection limit in the assessment of strength values from test results (Paragraph A3.2, Appendix A)

Z = the section modulus of the uncracked section, referred to the extreme fibre at which cracking occurs (Clause 9.5.4)

= hazard factor as per AS 1170.4 (Table 10.3)

Z_d = the section modulus of the design cross-sectional area, A_d, of a member (Clause 4.5.8 and Paragraph D6.6, D7.2, Appendix D)

= for plain masonry, the section modulus of the bedded area (Clauses 7.4.2 and 7.4.3.2)

Z_p = the lateral section modulus based on mortar contact area of the perpend joints (Clause 7.4.3.2)

Z_t = the equivalent torsional section modulus measured normal to the diagonal crack line (Clause 7.4.4.3)

Z_u = the lateral section modulus of the masonry units (Clause 7.4.3.2)

α = a slope factor (Clause 7.4.4.2 and Table 7.5)

ε_s = the free moisture shrinkage strain one year after prestressing (Clause 9.3.3.2)

ϕ = the capacity reduction factor for design strength of unreinforced, reinforced, prestressed masonry, wall ties and accessories (Clauses 4.4, Clauses 7.4.3.2 and Table 4.1)

σ_{pu} = the tensile stress in the tendons at the ultimate strength limit state (Clause 9.5.2)

μ = structural ductility factor for earthquake design (Clause 10.2.2)

1.7 EXISTING STRUCTURES

Where the strength or serviceability of an existing structure is to be evaluated, the general principles of this Standard may be used; however, the design values given in Section 3 for the properties of the materials may not be applicable.

NOTE: Appendix B provides a method for determining characteristic values of properties from test results.

SECTION 2 REQUIREMENTS FOR DESIGN

2.1 SCOPE OF SECTION

This Section sets out general requirements for the design of masonry. Particular requirements for the structural design of unreinforced masonry are set out in Section 7, for reinforced masonry in Section 8 and for prestressed masonry in Section 9.

2.2 AIM

The aim of design is to provide a structure that is durable, fire resistant and serviceable, and has adequate strength and stability while serving its intended function and satisfying other relevant requirements such as resistance to water penetration, robustness, ease of construction and economy.

2.3 GENERAL REQUIREMENTS

2.3.1 Durability

A masonry member or structure shall withstand the expected wear and deterioration throughout its design life, taking into account the exposure environment and importance of the structure, without the need for undue maintenance.

2.3.2 Fire resistance

A masonry member or structure shall have fire resistance so that the member can perform its structural function for the required period and, if necessary, prevent the spread of fire.

2.3.3 Serviceability

A masonry member or structure shall remain serviceable and fit for the purpose for which it was constructed, throughout its design life.

2.3.4 Strength

A masonry member or structure shall have the capacity to resist the design loads.

2.3.5 Stability

A masonry member or structure shall be designed to be stable throughout its design life.

2.4 DESIGN REQUIREMENTS

2.4.1 Design for durability

A masonry member or structure designed for durability in accordance with Section 5 is deemed to meet the requirements of Clause 2.3.1.

2.4.2 Design for fire resistance

A masonry member or structure designed for fire resistance in accordance with Section 6 is deemed to meet the requirements of Clause 2.3.2.

2.4.3 Design for serviceability

A masonry member or structure designed for serviceability as set out in Clauses 2.5.1 and 2.5.2 is deemed to meet the requirements of Clause 2.3.3.

2.4.4 Design for strength

A masonry member or structure designed for strength as set out in Clauses 2.5.1 and 2.5.3 is deemed to meet the requirements of Clause 2.3.4.

2.4.5 Design for stability

A masonry member or structure designed for stability as set out in Clauses 2.5.1 and 2.5.4 is deemed to meet the requirements of Clause 2.3.5.

2.4.6 Design for earthquakes

A masonry member or structure shall be designed for earthquakes as either one of the following:

(a) Considered to be part of the seismic force-resisting system and designed accordingly.

(b) Separated from all structural elements such that no interaction takes place as the structure undergoes deflections, determined in accordance with this Standard, to arise from the earthquake effects.

All masonry members or structures, including those deliberately designed to be independent of the seismic force-resisting system, shall be capable of performing their required function while sustaining the deformation of the structure resulting from the application of the earthquake forces determined for each limit state.

NOTES:

1 For multistorey structures where unreinforced masonry is part of the seismic force-resisting system, particular attention should be given to the effects of structural deformations and different material stiffnesses when designing the masonry (see AS 1170.4).

2 Where the masonry does not form part of the seismic force-resisting system, an adequate independent force-resisting system should be provided.

2.4.7 Design for other requirements

A masonry member or structure designed for other requirements as set out in Clause 2.8, in accordance with Section 4 and Section 7, 8, 9 or 10, is deemed to meet the requirements of Clause 2.3.6.

2.5 SERVICEABILITY, STRENGTH AND STABILITY

2.5.1 General

Calculations related to serviceability, strength and stability shall—

(a) be in accordance with accepted principles of mechanics;

(b) provide for all loads and forces to be transferred through the structure to the foundation;

(c) provide for compatibility between each masonry member and the structural members giving vertical and, where required, lateral support to that masonry member;

(d) be based on the material properties of the masonry in accordance with Section 3; and

(e) be consistent with the assumptions that—

  (i) plane sections remain plane under bending actions;

  (ii) there is a recognized stress-strain relationship for masonry; and

  (iii) the tensile strength of unreinforced masonry is taken as zero except for stresses resulting from wind, earthquake and similar forces of a short-term, transient nature.

2.5.2 Design for serviceability

A masonry member or structure shall be designed to allow movements to be controlled or isolated so that damage to the masonry, the building and its components is avoided and structural and other requirements are satisfied.

The movements to be allowed for shall include the following:

(a) The expansion characteristics of clay masonry and the shrinkage characteristics of concrete masonry, calcium silicate masonry and AAC masonry.

(b) Thermal movements.

(c) Deflections, shortening, shrinkage, creep and similar deformations in adjacent or associated materials.

(d) Foundation movements.

(e) Deformation due to construction loads or construction sequences.

A masonry member or structure shall be designed for serviceability by controlling or limiting deflections and cracking arising from the actions specified in Clause 2.6 and by providing member robustness as specified in Clause 4.6.

2.5.3 Design for strength

A masonry member or structure shall be designed for strength as follows:

(a) The loads and other actions shall be as specified in Clause 2.6.

(b) The design strength shall be determined in accordance with the requirements of Sections 4, 7, 8, 9 and 10.

(c) The member shall be proportioned so that its design strength is not less than the design action effect.

2.5.4 Design for stability

The structure as a whole and its parts shall be designed to prevent instability due to overturning, uplift and sliding under the loads and other actions specified in Clause 2.6.

2.6 LOADS AND LOAD COMBINATIONS

2.6.1 Loads, and other forces and actions

2.6.1.1 *Dead, live, wind, snow and earthquake loads*

The dead loads, live loads, wind loads, snow loads and earthquake loads used in structural design shall be those specified in AS/NZS 1170.0, AS/NZS 1170.1, AS/NZS 1170.2, AS/NZS 1170.3, AS 1170.4 and AS 4055. For earthquake actions, the additional requirements of Section 10 shall apply.

2.6.1.2 *Other forces and actions*

Account shall be taken of other forces and actions that could affect the stability, strength and serviceability of the structure and its component members, including the following:

(a) Earth pressure, with or without groundwater pressure.

(b) Liquid pressure.

(c) Construction loads and procedures.

(d) Foundation movements.

(e) Shrinkage or expansion effects.

(f) Axial shortening effects.

(g) Creep effects.

(h) Temperature effects.

(i) Interactions with other materials and members or between different masonry materials.

(j) Design of masonry members of bridges for loads resulting from floods or collision, which shall be carried out in accordance with the AS 5100 series.

2.6.2 Design load combinations

Structures, as a whole, and component members shall be designed to resist each compatible simultaneous combination of applied forces acting on them, including each of the load-factored combinations for factored loads given in AS/NZS 1170.0 and AS 1170.4.

2.6.3 Design loads for lateral supporting members

The horizontal design load acting on a structural member or system providing lateral support to a masonry member (see Clause 2.7) shall be the greater of the following:

(a) The sum of the simple static reactions to the applied horizontal forces for the appropriate load combination and 2.5% of the vertical load that the masonry member is designed to carry.

(b) 0.5 kPa acting on the tributary area.

Design loads specified in this Clause are considered appropriate for each of the limit states and do not require additional factoring.

2.6.4 Design loads for connections to lateral supports

The horizontal design load acting on a connection providing lateral support to a masonry member shall be the value derived from Clause 2.6.3 increased by a factor of 1.25.

2.7 LATERAL SUPPORT

A structural member or system is considered to provide adequate lateral support to a masonry member provided it can resist the loads specified in Clause 2.6.3 without failure and satisfies the following requirements:

(a) The maximum deflection at the point of support shall not exceed span/500 for unreinforced masonry, under the loads specified in Clause 2.6.3.

(b) The maximum deflection at the point of support shall not exceed span/250 for reinforced masonry, under the loads specified in Clause 2.6.3 unless specified otherwise for architectural purposes.

NOTE: Steel mullions built into masonry panels are not required to satisfy these limits but should be designed appropriately for serviceability and ultimate conditions considering their interaction with the masonry.

2.8 OTHER DESIGN REQUIREMENTS

2.8.1 General

Requirements such as resistance to water penetration, fatigue, progressive collapse and any special performance requirements shall be taken into account in the design of the structure.

2.8.2 Design for water penetration

When moisture penetration is to be prevented, a masonry member or structure shall be designed, constructed and protected against moisture ingress in accordance with Clause 4.7.

2.8.3 Design for accidental damage

The structure and its component members shall be designed to take into account the possibility of abnormal load conditions and to restrict, to a reasonable level, the adverse consequences, including progressive collapse that could result from failure caused by such load conditions.

2.8.4 Masonry under construction

Account shall be taken of the lower stability, stiffness and strength of newly constructed masonry.

NOTE: See also Section 12, particularly the requirements of Clause 12.9.1 for temporary bracing.

2.9 THERMAL PERFORMANCE

Where a wall is designed to contribute to the thermal performance of a building by insulation, thermal mass or both, it shall be of appropriate materials and construction.

2.10 ACOUSTIC INSULATION

Where it is necessary to limit the transmission of sound through a wall, the wall shall be of appropriate materials and construction.

SECTION 3 DESIGN PROPERTIES

3.1 SCOPE OF SECTION

This Section specifies design properties for masonry and its component materials, connectors, reinforcement and prestressing tendons.

NOTE: Requirements for materials are given in Section 11.

3.2 MASONRY UNITS

The work size width, height, length and face shell width (for units that are to be face-shell-bedded) used in the design shall be specified. When used, the design characteristic unconfined compressive strength (f'_{uc}) and the design characteristic lateral modulus of rupture (f'_{ut}) shall also be specified.

In the absence of test data, the design characteristic lateral modulus of rupture (f'_{ut}) shall not exceed 0.8 MPa.

3.3 MASONRY

3.3.1 General

Where the values of characteristic strengths used in structural design are to be based on the results of tests, including, in the case of special masonry, the values of f'_m or f'_{mt}, the results shall be from tests on specimens made from materials having the same properties and strength characteristics as those to be used in the actual construction.

Masonry constructed from square-dressed natural stone shall be designed as special masonry.

3.3.2 Compressive strength

The characteristic compressive strength of masonry (f'_m) shall be as follows:

(a) For masonry other than special masonry, one of the following:

(i) For masonry constructed with clay, concrete or calcium silicate units, the design characteristic compressive strength shall be taken as $f'_m = k_h f'_{mb}$

where

$f'_{mb} = k_m \sqrt{f'_{uc}}$

The values of f'_{mb} in Table 3.1 are deemed to satisfy this formula.

k_m = compressive strength factor (see Table 3.1)

k_h = joint thickness factor = $1.3\,(h_u/19t_j)^{0.29}$ but to not exceed 1.3

Table 3.2 is deemed to satisfy this expression.

f'_{uc} = characteristic unconfined compressive strength of masonry units in megapascals

(ii) For AAC masonry with thin-bed mortar, the design characteristic compressive strength shall be taken as $f'_m = f'_{uc}$.

For concrete and AAC units, f'_{uc} shall be the 28 day strength. If tested at another age, a correction shall be made to obtain the 28 day strength.

(b) For special masonry, the design characteristic compressive strength shall be taken as a value determined in accordance with Appendix B from tests carried out in accordance with Appendix C.

TABLE 3.1

CHARACTERISTIC COMPRESSIVE STRENGTH OF MASONRY (f'_{mb})

Type of masonry unit	Bedding type	Mortar class (see Note 2)	Characteristic unconfined compressive strength of unit (f'_{uc}) MPa								Compressive strength factor (k_m)
			5	10	15	20	25	30	40	≥50	
Clay	Full	M2	2.5	3.5	4.3	4.9	5.5	6.0	7.0	7.8	1.1
Clay	Full	M3	3.1	4.4	5.4	6.3	7.0	7.7	8.8	9.9	1.4
Clay	Full	M4	4.5	6.3	7.7	8.9	10.0	10.9	12.7	14.1	2.0
Clay	Face shell	M3	3.6	5.1	6.2	7.2	8.0	8.8	10.1	11.3	1.6
Concrete	Full	M3	3.1	4.4	5.4	6.3	7.0	7.7	8.8	9.9	1.4
Concrete	Face shell	M3	3.6	5.1	6.2	7.2	8.0	8.8	10.1	11.3	1.6
Calcium silicate	Full	M3	3.1	4.4	5.4	6.3	7.0	7.7	8.8	9.9	1.4
Calcium silicate	Full	M4	4.5	6.3	7.7	8.9	10.0	10.9	12.7	14.1	2.0

NOTES:

1 Linear interpolation may be used.

2 Mortar classification is given in Table 11.1.

TABLE 3.2

JOINT THICKNESS FACTOR (k_h)

Ratio of masonry unit height to mortar bed joint thickness	0.0	3.3	7.6	9.0	11.9	16.2	19.0
Joint thickness factor (k_h)	0.00	0.78	1.00	1.05	1.14	1.24	1.30

3.3.3 Flexural tensile strength

The characteristic flexural tensile strength (f'_{mt}) for unreinforced, reinforced and prestressed masonry shall be as follows:

(a) Under actions resulting from wind, earthquake loads or similar forces of a short-term, transient nature, and where any simultaneous force due to vertical loads (F_d) is not a negative value (that is, not a net tension force):

(i) At mortar joints:

(A) For clay, concrete and calcium silicate masonry (except special masonry), a value not greater than 0.20 MPa.

(B) For AAC masonry with thin-bed joints (except special masonry), a value not greater than 0.26 MPa.

(C) For special masonry, a value obtained from tests in accordance with Appendix D but not greater than 1 MPa.

(ii) At interfaces with other materials:

(A) Where results of tests in accordance with Appendix D are available, the design characteristic flexural tensile strength shall be taken as a value derived from those tests.

(B) In other cases, the design characteristic flexural tensile strength shall be taken as zero.

Live loads shall not be considered to be of a short-term transient nature.

(b) In all other loading cases, the design characteristic flexural tensile strength shall be taken as zero.

3.3.4 Shear strength

The design characteristic shear strength (f'_{ms}) shall be as follows:

(a) For shear in the horizontal direction in continuous horizontal mortar joints:

(i) For masonry constructed with clay, concrete or calcium silicate units, the design characteristic shear strength (f'_{ms}) shall be taken as $1.25 f'_{mt}$ but not greater than 0.35 MPa, nor less than 0.15 MPa.

(ii) For AAC masonry with thin-bed mortar, the design characteristic shear strength (f'_{ms}) shall be taken as $0.67 f'_{ut}$.

(b) For shear in the vertical direction:

(i) Where neither connectors nor masonry header units cross the shear plane, the design characteristic shear strength (f'_{ms}) shall be taken as zero.

(ii) Where connectors in accordance with Clause 4.11 intersect the shear plane, the design characteristic shear strength (f'_{ms}) shall be taken as—

(A) for mortar joints—$1.25 f'_{mt}$ or 0.35 MPa, whichever is less; or

(B) for thin-bed mortar joints—zero.

(iii) For masonry in ordinary running bond, or if there are masonry header units in accordance with Clause 4.11 intersecting the shear plane, the design characteristic shear strength (f'_{ms}) shall be taken as—

(A) for masonry constructed with clay, concrete or calcium silicate units, $1.2 r_h$ MPa; or

(B) for masonry built with AAC units, $0.6 r_h$ MPa

where

r_h = proportion of the vertical shear plane that is intersected by masonry header units

(c) At a joint or interface confined by bonded reinforcement normal to the shear plane, the design characteristic shear strength shall be taken as 0.35 MPa.

(d) At membrane-type damp-proof courses (DPCs), flashings, and at the interface with materials other than covered by Clause 3.3.4(c), the design characteristic shear strength shall be taken as zero, or a value based on the results of tests in accordance with Clause 3.3.1.

3.3.5 Shear factor

The shear factor (k_v) for calculating the frictional component of shear capacity in accordance with Clause 7.5.4 shall be as given in Table 3.3, substantiated by appropriate tests.

TABLE 3.3

SHEAR FACTOR (k_v)

Type of masonry	Location	k_v
Clay, concrete or calcium silicate	At mortar bed joints	0.3
AAC	At thin-bed mortar joints	0.12
All	At membrane-type DPCs, flashings and similar locations where the membrane is in contact with the unit, or concrete or within the mortar and the DPC membrane consists of—	
	(a) bitumen-coated aluminium or embossed polyethylene	0.3
	(b) Polyethylene and bitumen-coated aluminium	0.15
	At interfaces of masonry with concrete	0.3
	At interfaces of masonry with steel	0.2
	At slip joints comprising two layers of membrane-type DPC material	0.1
	At other locations	zero

3.3.6 Elastic properties of masonry

Where masonry is assumed to have linear elastic properties and in the absence of test results, design values for elastic moduli shall be as given in Table 3.4.

TABLE 3.4

ELASTIC MODULI FOR MASONRY

Masonry unit type	Mortar class	Short-term loading E_m	Long-term loading E_L
Clay units with f'_{uc} in the range 5 to 30 MPa	M2 and M3	700 f'_m	450 f'_m
Clay units with f'_{uc} > 30 MPa	M3 and M4	1000 f'_m	660 f'_m
Concrete units with density ≥1800 kg/m^3 and calcium silicate units	M3 and M4	1000 f'_m	500 f'_m
Concrete units with density <1800 kg/m^3	M2 and M3	750 f'_m	500 f'_m
Grouted concrete or clay masonry	Any	1000 f'_c	350 f'_c
AAC	Thin bed	500 f'_m	250 f'_m

NOTES:

1 The above values are suitable for use with uncracked reinforced masonry section properties. For cracked reinforced masonry section properties, other values may be assumed based on reinforced concrete principles.

2 Mortar classification is given in Table 11.1.

3.4 TIES AND ACCESSORIES

The type and duty rating of wall ties shall be specified. Where they are designed to connect structural elements, the design strength and stiffness of accessories shall also be specified.

The mean tie strength (F_t) of Type A veneer wall ties shall be not less than the values given in Table 3.5. The design strength of Type A cavity wall ties shall be not less than the values given in Table 3.6.

TABLE 3.5

STRENGTH OF TYPE A VENEER TIES

Tie duty rating	Mean tie strength (F_t) kN	
	Tension	Compression
Light duty	0.30	0.36
Medium duty	0.60	0.72
Heavy duty	1.5	1.8

NOTE: The values in the Table represent mean values determined from a tie test in accordance with AS/NZS 2699.1.

TABLE 3.6

STRENGTH OF TYPE A CAVITY TIES

Tie duty rating	Mean tie strength (F_t) kN	
	Tension	Compression
Light duty	0.30	0.36
Medium duty	0.60	0.72
Heavy duty	1.5	1.8

NOTE: The values in the Table represent mean values determined from a tie test in accordance with AS/NZS 2699.1.

3.5 GROUT

The design characteristic compressive strength of grout (f'_{cg}) measured in accordance with the requirements for concrete in AS 3600; and (f'_c) measured in accordance with the requirements for concrete in AS 3600, but shall be greater than or equal to the characteristic compressive strength of masonry (f'_m) provided the requirements of Clause 11.7.3 are met.

3.6 REINFORCEMENT

3.6.1 Strength

The design yield strength of reinforcement (f_{sy}) shall be in accordance with Table 3.7.

TABLE 3.7

STRENGTH AND DUCTILITY OF REINFORCEMENT

Reinforcement		Design yield strength (f_{sy}) MPa	Ductility class
Type	**Designation grade**		
Bar plain to AS/NZS 4671	R250N	250	N
Bar deformed to AS/NZS 4671	D500L (fitments only)	500	L
	D500N	500	N
Welded mesh, plain, deformed and indented to AS/NZS 4671	D500L	500	L
	D500N	500	N
Stainless steel plain bar to BS 6744 (see Note 2)	200	200	N
Stainless steel ribbed bar to BS 6744 (see Note 2)	500	650	N

NOTES:

1 Reference should be made to AS/NZS 4671 for explanation of designations.

2 Stainless steel bars to BS 6744 are deemed to satisfy the requirements for Ductility Class N reinforcement as in AS/NZS 4671.

3.6.2 Modulus of elasticity

The modulus of elasticity of reinforcement (E_s), for all stress values not greater than the design yield strength of reinforcement (f_{sy}), shall be either—

(a) taken as equal to 200×10^3 MPa; or

(b) determined by test.

3.6.3 Stress-strain curves

The stress-strain curve for reinforcement shall be either—

(a) assumed to be of a form defined by recognized simplified equations; or

(b) determined from suitable test data.

3.7 TENDONS

3.7.1 Strength

The strength of prestressing tendons shall be as follows:

(a) The tensile strength of prestressing tendons (f_p) shall be taken as the minimum tensile strength specified in Table 3.8.

(b) The yield strength of prestressing tendons (f_{py}) shall be taken as—

   (i) for wire used in the as-drawn condition .... $0.75f_p$;

   (ii) for stress-relieved wire ....$0.85f_p$; and

   (iii) for all grades of strand and bar tendons.. $0.85f_p$.

3.7.2 Modulus of elasticity

The modulus of elasticity of prestressing tendons (E_p), shall be either—

(a) taken as—

   (i) for stress-relieved wire to AS/NZS 4672.1................................ 200×10^3 MPa;

   (ii) for stress-relieved steel strand to AS/NZS 4672.1 .... 195×10^3 MPa; and

(iii) for cold-worked high tensile alloy steel bars to AS/NZS 4672.1 ....170×10^3 MPa;

or

(b) determined by test.

3.7.3 Stress-strain curves

The stress-strain curve for prestressing tendons shall be determined from testing in accordance with AS 1391.

3.7.4 Other properties

Where other properties of prestressing tendons are required, reference shall be made to AS 3600.

TABLE 3.8

TENSILE STRENGTH OF PRESTRESSING TENDONS (f_p)

Material type and Standard	Nominal diameter mm	Area mm^2	Minimum breaking load kN	Minimum tensile strength MPa
Wire, AS/NZS 4672.1	5	19.6	30.4	1550
	5	19.6	33.3	1700
	7	38.5	65.5	1700
7-wire super strand, AS/NZS 4672.1	9.3	54.7	102	1860
	12.7	100	184	1840
	15.2	143	250	1750
7-wire regular strand, AS/NZS 4672.1	12.7	94.3	165	1750
Bars, AS/NZS 4672.1 (Super grade only)	23	415	450	1080
	295	660	710	1080
	32	804	870	1080
	38	1140	1230	1080

SECTION 4 GENERAL DESIGN ASPECTS

4.1 SCOPE OF SECTION

This Section sets out requirements for the general design aspects of masonry members and structures, including prevention of moisture penetration, joints, bonding and tying, connections, arches and lintels, sills and corbelling. It also provides capacity reduction factors and cross-sectional dimensions and properties required for design.

4.2 MEMBERS OF MIXED CONSTRUCTION

Members of mixed construction shall comply with one of the following:

(a) A member constructed of more than one type of masonry unit or mortar shall be designed assuming the least favourable combination of unit and mortar type when determining properties for strength and durability.

(b) A member composed of more than one type of masonry system (e.g. unreinforced and reinforced) shall be designed for composite action between the systems.

In each case, account shall be taken of the different material properties of the parts and the potential for differential movement.

4.3 CHASES, HOLES AND RECESSES

The effect of any chases, holes and recesses required in the masonry shall be taken into account in the design to ensure that the masonry achieves the required performance, including structural, moisture-resistance, fire-resistance, insulation and other properties required of the masonry.

NOTES:

1 The need for chases, holes and recesses to masonry should be minimized.

2 See also Clause 4.5.9.

4.4 CAPACITY REDUCTION FACTORS

The value of the capacity reduction factor (ϕ) used to evaluate the strength capacity of a member shall be taken from Table 4.1.

TABLE 4.1

CAPACITY REDUCTION FACTORS

Type of masonry or accessory and action effect	Capacity reduction factor (ϕ)
(a) Unreinforced masonry:	
(i) Compression	
(A) Solid or cored	0.75
(B) Hollow	0.50
(C) Grouted	0.60
(ii) Flexure	0.60
(iii) Shear	0.60
(iv) Other actions	0.60
(b) Reinforced and prestressed masonry:	
(i) Compression	0.75
(ii) All other actions	0.75
(c) Wall ties, connectors and accessories:	
(i) Wall ties in tension or compression	0.95
(ii) Connectors across a joint in masonry	0.75
(iii) Accessories and other actions	0.75

4.5 CROSS-SECTION PROPERTIES

4.5.1 Bedded thickness

The bedded thickness (t_b) of a masonry member for design shall be taken as follows:

(a) If the mortar bed joints are finished flush or are tooled to a depth not exceeding 3 mm, t_b shall be equal to the overall thickness of the masonry member.

(b) If the mortar bed joints are raked, t_b shall be equal to the overall thickness of the masonry, less the depth of raking.

NOTES:

1 The dimensions used to determine bedded thickness are based on the work size dimensions of the masonry units.

2 The effects of occasional deep raking, such as that required for overflashing set into mortar joints, should also be considered in the structural design.

4.5.2 Effective width of compression faces and flanges

Provided the outstanding portions of the face or flange are built to act in monolithic structural action with the diaphragm or web, the effective width (b) of each outstanding portion of the compression face or flange of an I, T, L or similar cross-sectional form shall be assumed to extend beyond the face of the connecting diaphragm or web portion by an amount not exceeding the least of—

(a) six times the thickness of the compression face or flange;

(b) half the clear distance to the next adjacent diaphragm or web;

(c) the distance to the structural end of the masonry in the outstanding face or flange;

(d) in a member spanning between lateral supports, 0.10 times the span distance between those supports; or

(e) in a shear wall with returns, acting as a cantilever—

(i) where the outstands project from both sides of the web portion of the wall, 0.08 times the total height of the wall above the level being analysed; or

(ii) where the outstand projects from only one side of the web portion of the wall, 0.06 times the total height of the wall above the level being analysed.

The effects of any holes or openings in the flange on the effective flange width shall be taken into account.

NOTE: Clause 8.6 provides criteria for the design width of the compression face for hollow unit reinforced masonry walls in bending.

4.5.3 Structural end of a masonry member

The structural end of a masonry member shall be the vertical cross-section in the member across which vertical shear cannot be transferred, and shall include—

(a) the actual end or face of the member;

(b) a control joint in the member; and

(c) a vertical mortar joint in the masonry (other than perpend joints in normal running bond), unless that joint is intersected by masonry bonding or tying complying with Clause 4.11 or is otherwise constructed to give the required monolithic structural action.

4.5.4 Bedded area

The bedded area of a masonry member cross-section (A_b) shall be as follows:

(a) For solid, cored and horizontally cored unit masonry, the bedded area of the mortar based on the bedded thickness in accordance with Clause 4.5.1.

(b) For hollow unit masonry, the bedded area of the mortar based on the face shell width and not including the area of webs in the units.

4.5.5 Combined cross-sectional area

The combined cross-sectional area of a grouted or reinforced masonry member (A_c) shall be as follows:

(a) For a fully grouted member, the product of the overall width less the depth of raking (if any), and the length.

(b) For a partially grouted member, the product of the overall width less the depth of raking (if any), and the portion of the length represented by the grouted cores, plus the bedded area of the portion of the length represented by the ungrouted cores.

4.5.6 Design cross-sectional area

The design cross-sectional area of a member (A_d) shall be as follows:

(a) For unreinforced and ungrouted masonry, the bedded area (A_b) in accordance with Clause 4.5.4.

(b) For grouted or reinforced masonry, the combined cross-sectional (A_c) area in accordance with Clause 4.5.5.

4.5.7 Grout area

The design cross-sectional area of grout in a grouted or reinforced member (A_g) shall be as follows:

$$A_g = A_d - A_b \qquad \ldots 4.5.7$$

where

A_d = design cross-sectional area of a masonry member in accordance with Clause 4.5.6

A_b = bedded area of a masonry cross-section or member in accordance with Clause 4.5.4

4.5.8 Section modulus and second moment of area

The section modulus of a cross-section shall be derived from the dimensions of the design cross-sectional area at the nearest mortar joint.

In ungrouted hollow unit masonry, the contributions of the webs of the units may be taken into account for determining the stiffness and deflection of masonry members.

4.5.9 Chases, holes and recesses

The design properties of the member shall take into account any chases, holes and recesses in the member.

NOTE: See also Clause 4.3.

4.6 DESIGN FOR ROBUSTNESS

4.6.1 General

Masonry members and their connections shall have an adequate degree of robustness. This requirement shall be deemed to be satisfied provided the members are proportioned in accordance with Clause 4.6.2 or Clause 4.6.3.

This requirement is in addition to other design requirements, including structural design in accordance with Sections 7, 8 or 9.

4.6.2 Robustness of walls

Walls shall be proportioned to resist an ultimate uniformly distributed out-of-plane load of 0.5 kPa in accordance with Clauses 7.4, 8.6 or 9.5. No load factor is required to be applied to this load.

A control joint in a wall, or an edge to an opening in a wall, shall be regarded as an unsupported edge to that wall unless specific measures are taken to provide adequate lateral support at that edge (see Clause 2.7).

The robustness of walls of geometric section may be checked by determining an equivalent thickness of a rectangular section from first principles and applying the provisions of this Clause.

In the case of loadbearing members, vertical loads applied to the top of the member shall be ignored when determining its robustness.

4.6.3 Robustness of isolated reinforced and unreinforced masonry piers

Isolated piers shall be proportioned such that—

$$\frac{H}{t_r} \leq C_v \qquad \text{. . . 4.6.3}$$

where

H = the clear height of the pier between horizontal lateral supports, in metres

t_r = the minimum thickness of the pier, in metres

C_v = robustness coefficient—

(a) for isolated piers unreinforced vertically, 13.5; and

(b) for isolated piers reinforced vertically or prestressed, 30.

NOTE: Reinforced vertically means complying with the reinforcement requirements of Clause 8.6, and prestressed means complying with the prestressing requirements of Clause 9.5, for bending in the vertical direction.

The robustness of isolated piers of geometric section may be checked by determining an equivalent thickness of a rectangular section from first principles and applying the provisions of this Clause.

4.7 PREVENTION OF MOISTURE PENETRATION

4.7.1 Cavities

In cavity walls and masonry veneer walls, cavities with a width of at least 40 mm, which are properly detailed and constructed, shall be regarded as being resistant to the passage of moisture from the exposed face through to the inner, unexposed face of the wall.

Where insulating material is placed in a cavity, the moisture resistance of the wall shall be maintained.

4.7.2 Weepholes

Weepholes shall be provided to drain moisture from or through masonry construction. Where flashings are incorporated in the masonry, weepholes shall be provided in the masonry course immediately above the flashing, at centres not exceeding 1200 mm.

4.7.3 Damp-proof courses (DPCs) and flashings

DPCs or flashings shall be incorporated into masonry construction to—

(a) provide a barrier to the upward or downward passage of moisture through masonry;

(b) prevent moisture from entering into the interior of a building from the exterior;

(c) prevent moisture passing across a cavity to the inner leaf; or

(d) shed moisture through masonry to the outer face.

Bituminous damp-proof course materials without metal strips shall not be used where the superimposed masonry exceeds either two storeys or 8 m in height.

Sheet material used as a DPC shall be at least 20 mm wider than the thickness of the masonry member in which it is placed.

Overflashings shall be designed such that they can be set to a depth of at least 15 mm into the masonry.

NOTE: Clause 12.4.16 gives construction requirements for DPCs and flashings.

4.7.4 Single-leaf and solid walls

Where the prevention of moisture penetration is required, external single-leaf walls and solid walls shall be protected on the outside face by a suitable weather-resistant coating (see Notes below).

Where a coating is to be applied for the purpose of this Clause, all mortar joints shall be tooled and be free of cracks and holes, or the surface to be coated shall be bagged or rendered.

The following weather-resistant coating systems are deemed to be satisfactory:

(a) Three coats of 100% acrylic-based exterior quality paint. The first coat shall be worked thoroughly into the texture of the masonry by brush to ensure complete coverage of all voids and irregularities (see Note 1).

(b) A first coat of waterproof cement paint, worked into the surface and over-coated with two coats of 100% acrylic-based paint (see Note 1).

(c) Where the masonry texture and jointing are to be obscured, rendering with a proprietary cement-based high-build waterproof render, followed by an elastomeric acrylic polymer coating.

(d) Clear water repellent coatings, provided there is a weatherproof overhang of not less than 1500 mm.

NOTES:

1 For guidance on the painting of buildings, see AS/NZS 2311.

2 Regular maintenance of any coating, in accordance with the manufacturer's recommendations, is necessary to maintain the integrity of the weatherproofing.

3 The above coatings may also be used to protect the outer leaf and cavity space of a cavity wall (see Clause 5.4.2).

4.8 CONTROL JOINTS

4.8.1 General

Control joints shall be incorporated in masonry, to control and limit the movements allowed for in Clause 2.5.2. Locations of joints shall be determined in accordance with Clause 4.8.2, 4.8.3 or 4.8.4. Where a control joint is incorporated, the joint is considered to be a structural end (see Clause 4.5.3).

Control joints shall be detailed in accordance with Clause 4.8.5.

4.8.2 Contraction joints

4.8.2.1 *General*

Contraction joints shall be placed in unreinforced concrete masonry or calcium silicate masonry in accordance with the following locations:

(a) In straight, continuous walls having no openings, at centres not more than the values specified in Table 4.2 or calculated in accordance with Clause 4.8.2.2.

(b) Where the height of the wall changes abruptly by more than 20% of its lesser height.

(c) Where walls change thickness.

NOTE: Engaged piers are not considered to be changes of thickness. Chases that have less than 75% of the leaf thickness remaining are considered to be changes of thickness.

(d) At control or construction joints in footings or slabs.

(e) At junctions of walls constructed of different masonry or other materials.

4.8.2.2 *Spacing*

The maximum spacing of contraction joints shall be calculated on the following basis and comply with:

(a) The design contraction shall be taken as the coefficient of contraction of the masonry units, determined in accordance with AS/NZS 4456.12.

(b) The spacing of joints shall be such that, at each joint, the calculated opening movement does not exceed 10 mm.

In straight continuous walls, the spacing shall not exceed the values in Table 4.2.

NOTE: Guidance on design and location of contraction joints is available in masonry industry publications.

TABLE 4.2

SPACING OF CONTRACTION JOINTS FOR UNREINFORCED MASONRY WALLS

Masonry wall construction and surface finish	Maximum joint spacing m
External masonry that is face-finished, rendered and/or painted	7.0
Internal masonry that is face-finished or sheeted	6.0
Internal masonry that is rendered and/or painted	5.0
External masonry with openings more than 900 mm in height	5.0

4.8.3 Expansion joints

4.8.3.1 *General*

Expansion joints shall be placed in unreinforced clay masonry walls at the following locations:

(a) In straight, continuous walls having no openings, at centres not more than the values calculated in accordance with Clause 4.8.3.2.

(b) Where the height of a wall changes abruptly by more than 20% of its lesser height.

(c) Where walls change thickness.

Engaged piers are not considered to be changes of thickness. Chases that have less than 75% of the leaf thickness remaining are considered to be changes of thickness.

(d) At control or construction joints in footings or slabs.

(e) At a distance from all corners not greater than 4500 mm and as close to the corner as practical.

(f) At junctions of walls constructed of different masonry or other materials

Expansion joints shall be vertical, not toothed, free of mortar and shall extend to the full height of the masonry.

NOTE: Expansion joints may be omitted below the DPC if there is not more than 600 mm of masonry below the DPC at the position of the joint.

4.8.3.2 *Spacing*

The maximum spacing of expansion joints shall be calculated on the following basis:

(a) The design expansion shall be taken as the following proportions of the coefficient of expansion of the masonry units (e_m):

  (i) Vertical expansion joints in walls .... 35%.

  (ii) Vertical expansion joints in parapets .... 70%.

  (iii) Horizontal joints in walls .... 70%.

(b) At each joint, the calculated closing movement shall not exceed 15 mm.

(c) After the full design expansion has taken place, the remaining width of each joint shall be not less than 5 mm.

(d) Allowance shall be made for thermal expansion in walls and parapets.

(e) Allowance shall be made for the effect of frame shortening in reinforced concrete frame buildings when calculating the spacing of horizontal joints.

NOTE: Guidance on design and location of expansion joints is available in masonry industry publications.

4.8.4 Articulation joints

Articulation joints shall be incorporated in all masonry walls that are supported on slabs and footings designed in accordance with AS 2870 for articulated masonry, except in the following cases:

(a) Class A and Class S sites.

(b) Reinforced masonry designed in accordance with Section 8.

Where articulation joints are required, they shall be provided in accordance with the following locations:

(i) In straight, continuous walls having no openings, at centres such that cracking due to foundation movements are avoided. The values given in Table 4.3 are deemed to satisfy this requirement for the circumstances described therein.

(ii) Where the height of the wall changes abruptly by more than 20% of its lesser height.

(iii) Where openings more than 900 mm × 900 mm occur, at not more than 5000 mm centres.

(iv) Where walls change thickness.

(v) Engaged piers are not considered to be changes of thickness. Chases that have less than 75% of the leaf thickness remaining are considered to be changes of thickness.

(vi) At control or construction joints in footings or slabs.

(vii) At a distance from all corners not greater than 4500 mm and not less than 470 mm for cavity walls or 230 mm for veneer walls.

TABLE 4.3

SPACING OF ARTICULATION JOINTS FOR UNREINFORCED CAVITY, SINGLE LEAF AND MASONRY VENEER WALLS

Site class (see Note)	Masonry wall construction and surface finish	Joint spacing up to 4 m high for 10 mm joints m	Joint spacing 4 m up to 8.5 m high for 10 mm joints m	Joint spacing 4 m up to 8.5 m high for 15 mm joints m
M, M-D	External face finish masonry	6.0	4.2	6.0
M, M-D	External rendered and/or painted masonry	5.5	3.9	5.5
M, M-D	Internal face finish or sheeted masonry	6.0	4.2	6.0
M, M-D	Internal rendered and/or painted masonry	5.5	3.9	5.5
H, H1-D	External face finished masonry	5.5	3.9	5.5
H, H1-D	External rendered and/or painted masonry	5.0	3.5	5.0
H, H1-D	Internal face finish or sheeted masonry	5.5	3.9	5.5
H, H1-D	Internal rendered and/or painted masonry	5.0	3.5	5.0
H, H2-D	External face finished masonry	5.0	3.5	5.0
H, H2-D	External rendered and/or painted masonry	4.5	3.2	4.5
H, H2-D	Internal face finish or sheeted masonry	5.0	3.5	5.0
H, H2-D	Internal rendered and/or painted masonry	4.5	3.2	4.5

NOTE: Site class is defined in AS 2870.

4.8.5 Detailing

Control joints shall comply with the following:

(a) Expansion joints and articulation joints shall be free of mortar.

(b) Any material used to fill joints shall be of a type that will not inhibit the performance of the joints during the life of the building.

(c) Joints shall be vertical and not toothed, unless this toothing is specifically considered in the design.

(d) Joints shall extend to the full height of the masonry but may be omitted below the damp-proof course (DPC) if there is not more than 600 mm of masonry below the DPC at the position of the joint.

4.9 MORTAR JOINTS

4.9.1 Thickness

The design thickness of mortar joints shall be as follows:

(a) For other than thin-bed mortar, including bed joints and perpends, not greater than 10 mm, unless the effects of greater thickness on compressive and flexural strength are taken into account in the design.

(b) For thin-bed mortar, not less than 2 mm and not greater than 4 mm.

4.9.2 Finishing

With the exception of thin-bed mortar, the surfaces of joints exposed to aggressive soils and those in exposure environments classed as marine, severe marine and special, as given in Table 5.1, shall be tooled to give a dense, water-shedding finish.

For walls constructed with hollow unit, ungrouted masonry joints shall not be raked. In other masonry, the depth of raking, if any, shall be not closer than 5 mm to any perforation in cored unit masonry or 20 mm in hollow unit masonry.

4.10 WALL TIES

Wall ties for cavity walls and masonry veneer walls shall be as follows:

(a) Of Type A and a duty rating appropriate to the structural requirements of the masonry (see Clauses 7.6.2, 7.6.3 and 7.7.4).

(b) Designed to transfer the design loads.

(c) Embedded at least 50 mm into the mortar joint and, where applicable, into the grout, have at least 15 mm cover from any exposed surface of the joint and be positively attached to the structural backing as follows:

   (i) For face-fixed ties in masonry veneer more than 3.0 m above the ground, by screw fixing.

   (ii) For side-fixed ties, by either screw or nail fixing.

   NOTES:

   1 AS 2699 (all parts) requires the manufacturer to supply the fasteners with the ties.

   2 Tests have shown that nail fixing for face-fixed ties in timber veneer construction and clip-on ties in steel stud veneer construction do not provide the required attachment to the structural backing under earthquake loading.

   3 Reduction of embedment within the limits of wall tolerance (see Table 12.1) is assumed not to affect performance.

(d) Spaced to comply with the following:

   (i) Not greater than 600 mm in each direction.

   (ii) Adjacent to horizontal or vertical lateral supports and control joints, and around openings in the masonry, with the first row of ties located within 300 mm from the line of lateral support, the control joint or the perimeter of opening.

   When a masonry veneer connected to a flexible structural backing is continuous past a horizontal floor support, this edge distance applies to the first row of ties immediately above and below the line of the floor.

   Where ties are required to be designed for double the design tie force (see Clauses 7.6.2 and 7.7.4) and this is achieved by doubling the number of ties in the row, all the ties in the row are required to satisfy the edge distance requirement.

NOTE: A row of ties may be in a single bed joint or distributed between up to two adjacent bed joints, provided both bed joints are within 300 mm of the line of the floor or support.

4.11 BONDING, TYING AND SUPPORTING

4.11.1 General

Where monolithic structural action is required across a vertical joint between two masonry members or between two components of a member, the interface shall be intersected by either—

(a) masonry header units that provide engagement across the interface and comply with Clause 4.11.2; or

(b) connectors that tie the two members or components together across the interface and comply with Clause 4.11.3.

The vertical joint at the interface shall be filled with mortar.

4.11.2 Bonding with masonry header units

Masonry header units used to obtain monolithic structural action shall be—

(a) of a masonry material compatible with the masonry in the wall, or detailed so that any incompatibility does not impair the structural integrity of the wall;

(b) spaced—

  (i) between the leaves in solid masonry construction—either in header courses at centres of 600 mm or less, in which at least each alternate unit is a header, or with the equivalent number of header units distributed uniformly throughout the wall; and

  (ii) at other interfaces—either with at least every fourth course of masonry at the interface fully engaged, or with the equivalent number of header units distributed uniformly throughout the interface area;

(c) properly bonded with each section of the masonry in accordance with the selected bond pattern, and providing engagement of at least 90 mm into the masonry on each side of the interface; and

(d) built in as the work progresses.

4.11.3 Tying with connectors

4.11.3.1 *Other than diaphragm walls and walls of geometric section*

For other than diaphragm walls and walls of geometric section, connectors used at an interface to obtain monolithic structural action shall comply with the following:

(a) Connectors shall have a characteristic tensile capacity of 0.4 kN.

Medium duty Type A cavity wall ties are deemed to satisfy this requirement.

(b) Connectors shall be spaced—

  (i) between the leaves in solid masonry construction—not more than 400 mm in each direction; and

  (ii) at other interfaces—at a horizontal spacing of one connector for each 200 mm width (or part thereof) of the interface area, and at a vertical spacing of 300 mm average and 400 mm maximum between connectors, or at such alternative spacing as is substantiated by calculation or tests.

(c) Connectors shall be placed in aligning mortar bed joints to tie the masonry members or components together across the interface, and built into the masonry in accordance with Clause 12.4.7.

4.11.3.2 *Diaphragm walls and walls of geometric section*

For diaphragm walls and walls of geometric section, at the engagement of diaphragms with leaves, it shall be permissible to use shear connectors to obtain monolithic action at the interface. The spacing of the connectors shall be as calculated but not greater than 600 mm.

NOTE: The method for the design of shear connectors is given in Clause 7.5.6.

4.11.4 Fixing to supporting structures

Where a load is to be transferred from a masonry member to a supporting structure, this shall be achieved by one of the following methods:

(a) Bedding the masonry on a mortar bed on the supporting structure, with or without a DPC.

(b) Provision of bonded reinforcement normal to the joint between the masonry member and the support.

(c) Provision of connectors such as column ties or head ties of sufficient strength and stiffness, spaced such that they are capable of transferring the calculated loads.

The shear transfer capacity, as determined in accordance with Clause 7.5 or 8.7, shall be greater than the magnitude of the loads to be transferred to the supporting structure determined in accordance with Clause 2.6.

4.12 STACK BONDED MASONRY

4.12.1 Solid and cored unit masonry

Solid and cored unit masonry laid in a stack bond pattern, including masonry in veneer walls, shall be reinforced using properly anchored bed joint reinforcement, of area not less than 0.00035 × gross vertical cross-sectional area of the wall. The reinforcement shall comply with Clauses 5.9.3 and 11.8, shall be continuous between lateral supports and shall be spaced vertically at centres not exceeding six times the thickness of the stack bonded leaf. Bed joints in the following locations shall be reinforced: the first bed joint above or below an unrestrained horizontal edge of the masonry, at least one bed joint within 300 mm above a horizontal line of lateral support, and at least one bed joint within 300 mm below a horizontal line of lateral support.

Such masonry shall be designed as unreinforced for compression in accordance with Clause 7.3, unreinforced for shear in accordance with Clause 7.5 and unreinforced for vertical bending in accordance with Clause 7.4.2. Such masonry shall be designed as reinforced for one way horizontal bending in accordance with Clause 8.6, except that the requirements of this Clause 4.12 shall take precedence over the requirements of Clause 8.6(a) and 8.6(b).

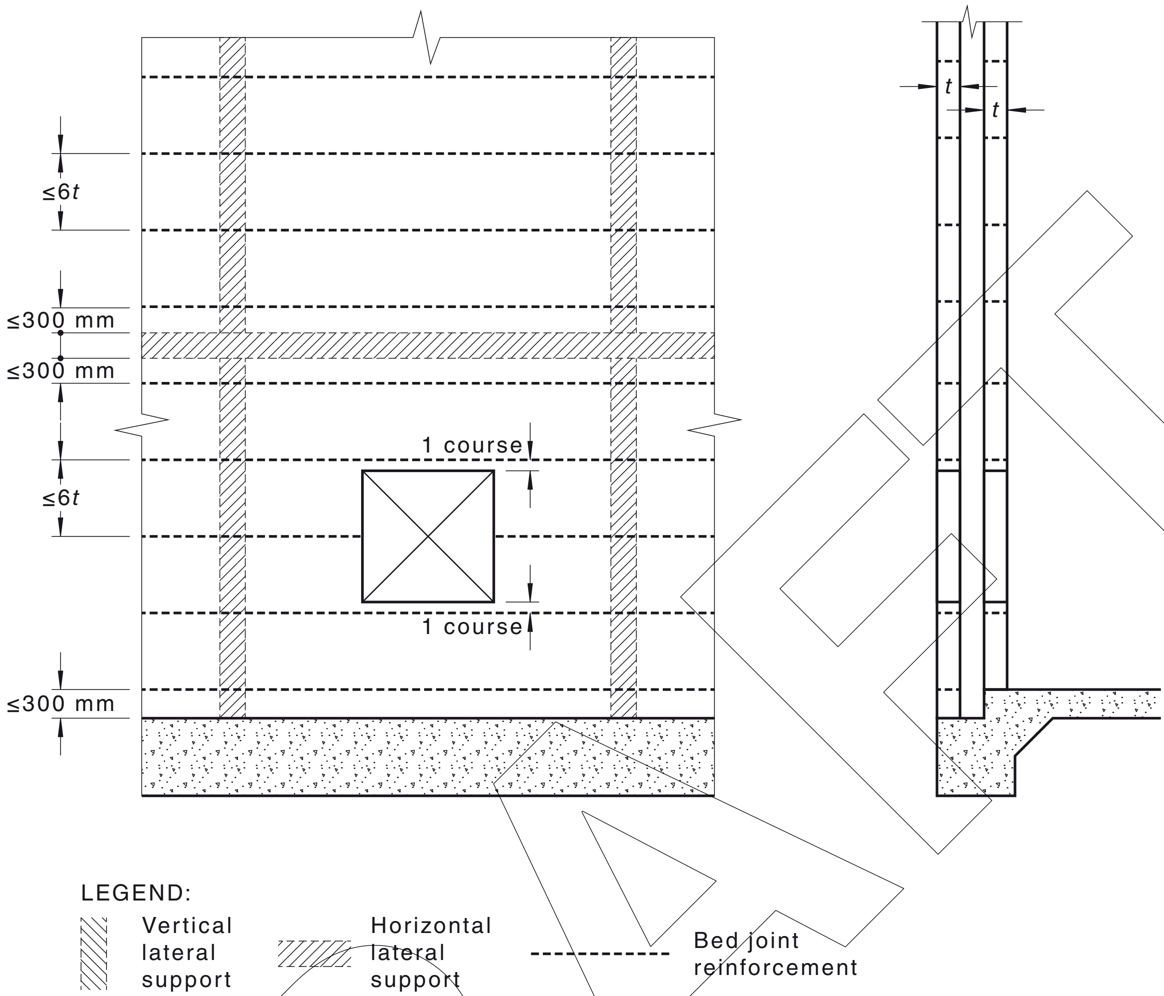


FIGURE 4.1 REINFORCEMENT PLACEMENT FOR STACK BONDING

4.12.2 Hollow unit masonry

Hollow unit masonry laid in a stack bond pattern shall be reinforced or prestressed and designed for the actions it is required to resist in accordance with Section 8 or Section 9.

4.13 ARCHES AND LINTELS

Unreinforced masonry over openings shall be supported by arches, lintels or frames.

Lintels shall comply with the following:

(a) End bearings of lintels on each side of an opening shall be not less than 100 mm in length for openings up to 1000 mm wide and 150 mm for wider openings.

(b) Maximum deflection under serviceability loads shall not exceed span/360 or 10 mm, whichever is the lesser.

Arches shall have sufficient rise to carry the loads and shall be provided with buttresses capable of taking the horizontal thrust from the arch.

4.14 INTERACTION BETWEEN MASONRY MEMBERS AND SLABS, BEAMS OR COLUMNS

Where a masonry member and associated slabs, beams or columns are designed for composite action, the magnitude of the stresses likely to occur, as a result of the composite action, shall be assessed and taken into account in the design.

Where a masonry member and associated slabs, beams or columns are not designed for composite action, regions of compressive stress concentrations shall be designed in accordance with the requirements of Clause 7.3.5 for concentrated loads.

Where a concrete slab is supported by an unreinforced masonry wall, a slip joint, as shown in Figure 4.2, shall be provided to allow for differential movement such as brick growth and concrete shrinkage.

Slip joints shall consist of—

(a) 2 layers of rigid DPC; or

(b) 1 layer of DPC over a flat rigid material; or

(c) 2 layers of flexible DPC over a level surface of solid masonry; or

(d) other proprietary systems that have demonstrated slip performance.

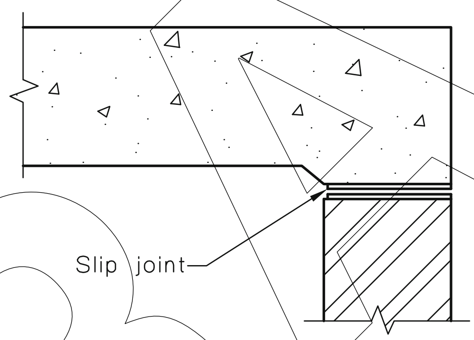


FIGURE 4.2 SLIP JOINT

4.15 CORBELLING

4.15.1 Corbels normal to the plane of the wall

Corbels normal to the plane of the wall shall be constructed using solid masonry units and shall satisfy the structural design requirements of this Standard. The additional loading from the corbel shall be taken into account in the design of the wall.

The projection of the corbel beyond the face of the wall shall not exceed one-half the wall thickness.

The projection of any masonry unit in the corbel shall not exceed the lesser of—

(a) one-third of the unit dimension measured in the direction of the corbel; and

(b) one-half of the height of the unit.

For the application of this Clause to cavity walls, the wall thickness shall be taken as the thickness of the leaf being corbelled.

4.15.2 Corbels in the plane of the wall

Corbels in the plane of the wall shall be designed to satisfy the structural design requirements of this Standard.

4.16 ATTACHMENT TO FACE OF WALLS

Where timber floors or roofs are to be supported by a wall plate, or similar member, attached to the face of the wall, anchors shall be spaced at not more than 600 mm and shall be of sufficient capacity to transfer the vertical shear into the wall.

SECTION 5 DESIGN FOR DURABILITY

5.1 SCOPE OF SECTION

This Section specifies requirements for the design of masonry for durability.

NOTE: The performance requirements for durability are set out in Clause 2.3.1.

5.2 GENERAL

Masonry materials, accessories and built-in items shall be selected and combined to satisfy the durability requirements of Clauses 5.5 to 5.9, for the relevant exposure environment in accordance with Clause 5.3 and the location in accordance with Clause 5.4.

In determining the exposure environment in accordance with Clause 5.3, the most severe exposure environment shall govern, in order as given in Clause 5.3.1 to Clause 5.3.5.

NOTE: For example, an industrial environment within a marine environment will be classed as marine and an industrial environment within a moderate environment will be classed as industrial. Masonry designed in accordance with this Section is deemed to satisfy those performance requirements (see Clause 2.4.1).

5.3 EXPOSURE ENVIRONMENTS

5.3.1 Severe marine

Areas up to 100 m from a non-surf coast and up to 1 km from a surf coast shall be regarded as severe marine environments. The distances specified are from the mean high-water mark.

5.3.2 Marine

Areas from 100 m up to 1 km from a non-surf coast and from 1 km up to 10 km from a surf coast shall be regarded as marine environments. The distances specified are from the mean high-water mark. Sheltered bays such as Port Phillip Bay and Sydney Harbour are considered to be non-surf coast.

5.3.3 Industrial

Industrial environments are those within 1 km of major industrial complexes producing significant acidic pollution.

NOTE: There are only a few such regions in Australia; for example, around Port Pirie.

5.3.4 Moderate

Moderate environments are those with light industrial pollution or very light marine influence, or both. They include built-up areas within 50 km of the coast and more than 1 km from a non-surf coast and more than 10 km from a surf coast, including suburban areas of cities such as Melbourne, Adelaide and Hobart, many areas of Sydney, Perth and Brisbane, and many inland cities.

5.3.5 Mild

5.3.5.1 *General*

Environments more than 50 km from the coast and not classed as industrial shall be regarded as mild environments and subdivided in accordance with Clause 5.3.5.2 to Clause 5.3.5.3.

5.3.5.2 *Mild-tropical*

Environments more than 50 km from the coast and falling within the tropical climatic zone shown in Figure 5.1 shall be regarded as mild-tropical environments.

5.3.5.3 *Mild-temperate*

Environments more than 50 km from the coast and falling within the temperate climatic zone shown in Figure 5.1 shall be regarded as mild-temperate environments.

5.3.5.4 *Mild-arid*

Environments more than 50 km from the coast and falling within the arid climatic zone shown in Figure 5.1 shall be regarded as mild-arid environments.

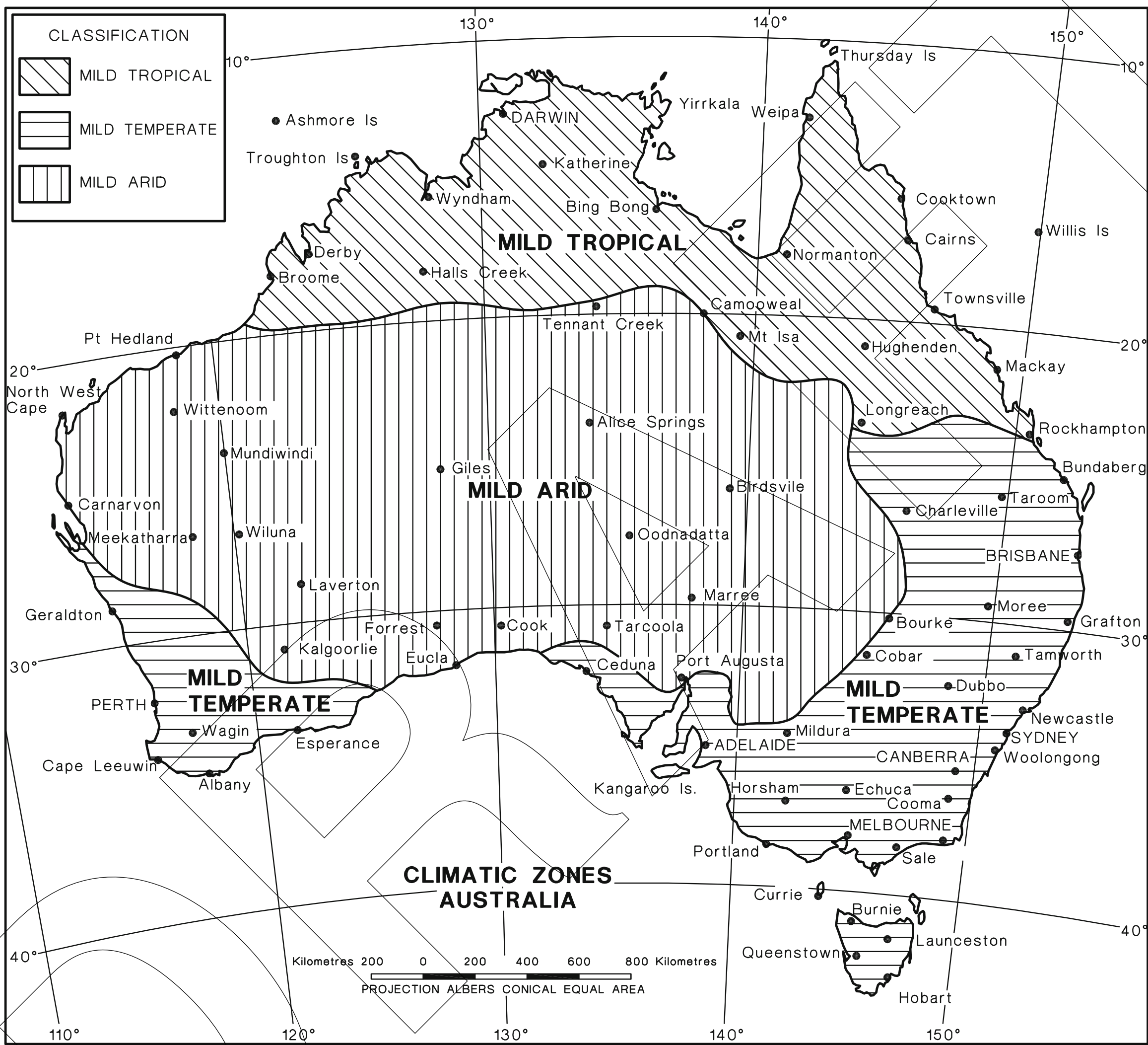


FIGURE 5.1 CLIMATIC ZONES FOR MILD ENVIRONMENT

5.4 LOCATIONS

5.4.1 Exterior

The following shall be regarded as being in exterior locations:

(a) The exposed leaf of an external cavity wall or masonry veneer wall.

(b) The cavity space in an external cavity wall or veneer wall.

(c) Wall ties in an external cavity wall or veneer wall.

(d) Roof tie-down straps in an external cavity wall or veneer wall.

(e) Lintels embedded in either leaf of an external cavity wall.

5.4.2 Exterior-coated

The following shall be regarded as being in exterior-coated locations:

(a) Elements in exterior locations as defined in Clause 5.4.1, but located above a DPC, sheltered by a roof, eave or coping, having any junctions with other building elements properly flashed, and protected by a weather-resistant coating in accordance with Clause 4.7.4.

(b) Elements below a DPC or in contact with the ground, and protected from water ingress by a continuous impermeable membrane such as that used for protection of slabs on ground. Painted water-resistant systems are not adequate for this purpose.

5.4.3 Interior

The following shall be regarded as being in interior locations:

(a) Elements that are above a DPC and enclosed within the building except during construction.

(b) The masonry units and mortar forming the internal leaf of a cavity wall.

5.5 MASONRY UNITS

The minimum salt attack resistance grade of masonry units shall be as given in Table 5.1.

NOTE: The means for demonstrating compliance with the required grade are given in AS/NZS 4455.1.

5.6 MORTAR

The minimum mortar class shall be as given in Table 5.1.

NOTE: Means of providing the required mortar class are given in Clause 11.4.3.

5.7 BUILT-IN COMPONENTS

Built-in components for masonry construction, including (but not limited to) wall ties, masonry anchors, connectors, shelf angles, lintel bars, bed joint mesh, bolts and fixings shall have at least the durability class given in Table 5.1.

NOTE: Guidance on corrosivity categories and their relationship to durability classes is given in Appendix I. Tables I2 to I4 show solutions for wall ties, connectors and accessories, and lintels and shelf angles as provided in AS/NZS 2699.1, AS/NZS 2699.2 and AS/NZS 2699.3 respectively.

For situations not covered in Table 5.1, built-in components shall have a demonstrated satisfactory service history in the application or be evaluated using tests that are applicable to the material and design life of the component and that demonstrate compliance to the design life.

Built-in components shall be in accordance with AS/NZS 2699.1, AS/NZS 2699.2 or AS/NZS 2699.3, with the exception of steel mullions and other structural steel elements not covered in those Standards, which shall meet the requirements of Clause 2.3.1.

5.8 GROUT

Where grout is required to provide protection to reinforcement, tendons or other steel items, it shall have a GB or GP cement content of not less than 300 kg/m^3.

NOTE: Additional requirements for grout are given in Clause 11.7.

TABLE 5.1
DURABILITY REQUIREMENTS

1	2		3	4		5	6
Exposure environment	**Location**		**Salt attack resistance grade of masonry units**	**Mortar class**		**Durability class of built-in components**	**Reinforcement cover (see Clause 5.9.2) mm**
Any	Interior	Normal	Protected	Clay units	M2	R1	5
				Concrete or calcium silicate units	M3	R1	5
		Subject to non-saline wetting and drying	General purpose	M3		R3	15
		Subject to saline wetting and drying	Exposure	M4		R4	25
	Exterior-coated (see Note 1)	Above a DPC	Protected	Clay units	M2	R1	5
				Concrete or calcium silicate units	M3	R1	5
		Below a DPC	Protected	Clay units	M2	R2	15
				Concrete or calcium silicate units	M3	R2	15
	Below a DPC or in contact with the ground	Non-aggressive soils	General purpose	M3		R3	15
		Aggressive soils (see Note 2)	Exposure	M4		R4	25
Mild-arid	Exterior		Protected	Clay units	M2	R1	5
				Concrete or calcium silicate units	M3	R1	5
Mild-temperate	Exterior		Protected	Clay units	M2	R1	5
				Concrete or calcium silicate units	M3	R1	5
Mild-tropical	Exterior		Protected	Clay units	M2	R2	15
				Concrete or calcium silicate units	M3	R2	15
Moderate	Exterior		Protected	Clay units	M2	R1	5
				Concrete or calcium silicate units	M3	R1	5
Industrial	Exterior		Exposure	M4		R4	25
Marine (see Note 3)	Exterior		General purpose	M3		R3	15
Severe marine (see Note 4)	Exterior		Exposure	M4		R4	25
Special (see Note 5)	Exterior		(See Note 5)	(See Note 5)		R5	(See Note 5)

NOTES TO TABLE 5.1

1 Exterior-coated exposure requires protection in accordance with Clause 5.4.2. The coating requirements are different for locations above and below a DPC.

2 Soils in marine or severe marine environments shall be considered as aggressive. Where sulfate attack from groundwater is possible, Type SR cement shall be used. Type SR cement may be either blended or portland cement.

3 All external elements in contact with freshwater or subject to non-saline wetting and drying shall be treated as for a marine environment.

4 All external elements in contact with saline or contaminated water or subject to saline wetting and drying shall be treated as for a severe marine environment.

5 Requirements for especially aggressive environments depend on the nature of the corrosive agents and cannot be defined. Units, mortars, covers or coatings shown by test, or known by experience, to be resistant to the particular corrosive agent shall be used.

6 Joints exposed in marine, severe marine and special environments shall be tooled to give a dense, water-shedding finish (see Clause 4.9.2).

7 Means for determining salt attack resistance grades for masonry units are given in AS/NZS 4455.1.

8 Cover requirements shall be satisfied where grout cover is relied upon to provide durability protection otherwise use the durability class in column five of Table 5.1.

9 M2 mortar is not permitted for concrete and calcium silicate masonry. There are also limitations on the constituents of M3 and M4 mortars for calcium silicate masonry (refer to Table 11.1).

5.9 REINFORCEMENT AND TENDONS

5.9.1 General

Reinforcement and tendons shall have, as a minimum, the durability protection given in Table 5.1.

NOTE: Reinforcement and tendons may have either the appropriate durability class for built-in components given in column 5 of Table 5.1, or be protected by the relevant cover given in column 6 of Table 5.1.

The detailing of reinforcement in masonry shall be in accordance with AS 3600, except as specifically modified in this Clause.

The amount and disposition of reinforcement and the clear distance between parallel bars shall be such as to ensure that the grout can be placed and compacted to fill completely the area to be grouted and provide the required bond and protection to the reinforcement.

Reinforcement in the form of bundled bars, bent-up bars and reinforcing fabric shall not be used without checking that the reinforcement can be properly placed and the grout fully compacted.

5.9.2 Reinforcement and tendons in grouted cavities and cores

Where steel reinforcement and tendons rely on grout to provide protection, the steel shall be surrounded by grout complying with Clauses 5.8 and 11.7 to provide a cover, measured from the face of the grout to the surface of the reinforcement or tendon, in accordance with Table 5.1. When designing for reinforced masonry in compression, as per Clause 8.5, the minimum cover shall be determined from the annulus ring conditions stipulated in Clause 8.5 and shall supersede the nominated values in Table 5.1

Horizontal reinforcement in reinforced hollow unit masonry may be supported on the webs of recessed-web hollow units, provided the reinforcement is completely surrounded by grout, except where it is in contact with the masonry units.

Where recessed-web flush-ended hollow blocks are used, the reinforcement shall be supported clear of the webs to permit the covers required by Table 5.1 to be achieved at all positions along the reinforcement, including within perpend joints.

5.9.3 Reinforcement and tendons embedded in mortar joints

Reinforcement and tendons (including ducts) placed in mortar joints (including bed joints) shall—

(a) have an overall diameter or thickness not more than two-thirds the design thickness of the mortar joint;

(b) have at least 15 mm cover to any exposed surface of the mortar joint; and

(c) be suitably protected to provide the durability class required by Table 5.1 (see Clause 5.9.5).

5.9.4 Unbonded tendons in cavities and cores

Unbonded tendons in cavities and cores shall be suitably coated or protected to provide the durability class required by Table 5.1 (see Clause 5.9.5).

5.9.5 Durability class for steel reinforcement and tendons

Where steel reinforcement and tendons, including those unbonded in cavities and cores and those embedded in mortar joints, rely on their durability class to provide protection, the following shall be deemed to satisfy the requirements of Table 5.1:

(a) For durability classes R1 to R3, steel wire or bar with a galvanized coating mass of at least 470 g/m^2.

(b) For durability class R4, stainless steel reinforcement with physical and mechanical properties in accordance with BS 6744 and the chemical composition conforming to one of designations 1.4301, 1.4162, 1.4436, 1.4429, 1.4362 or 1.4462 to EN 10088-1 (as identified by BS 6744)

SECTION 6 DESIGN FOR FIRE RESISTANCE

6.1 GENERAL

This Section sets out the requirements for the design of masonry to resist the effects of fire and gives methods for determining fire-resistance levels for structural adequacy, integrity and insulation.

To ensure the appropriate fire resistance is achieved, supports and their connections shall be designed in accordance with Clauses 2.6.3 and 2.6.4.

6.2 FIRE-RESISTANCE LEVELS

The fire-resistance level that a member can provide, in terms of structural adequacy, integrity and insulation, shall be determined by one of the following means:

(a) Design from tabulated values or design based on test results in accordance with the following clauses:

(i) For structural adequacy .... Clause 6.3.

(ii) For integrity .... Clause 6.4.

(iii) For insulation .... Clause 6.5.

(b) Testing of a prototype in accordance with AS 1530.4.

(c) A recognized method of calculation, based on the properties of materials at elevated temperatures and using accepted engineering principles to predict the behaviour of the member.

6.3 STRUCTURAL ADEQUACY

6.3.1 General

Members shall be designed for fire resistance for structural adequacy in accordance with Clause 6.3.2, 6.3.3 or 6.3.4.

For reinforced members required to have a fire-resistance period for structural adequacy, in addition to the requirements given in Section 8, the minimum reinforcement shall be in accordance with Clause 6.3.5 and protection to reinforcement shall be in accordance with Clause 6.3.6.

Where a wall contains an opening of a height greater than one-fifth of the height of the wall, it shall be regarded as being divided into two sub-panels, with free edges at both ends of the opening.

6.3.2 Design of walls using tabulated values

6.3.2.1 *General*

Where a wall is designed using tabulated values, the slenderness ratio shall not exceed the relevant value obtained from Table 6.1. For the purpose of this Clause (6.3.2), the slenderness ratio shall be determined in accordance with Clause 6.3.2.2.

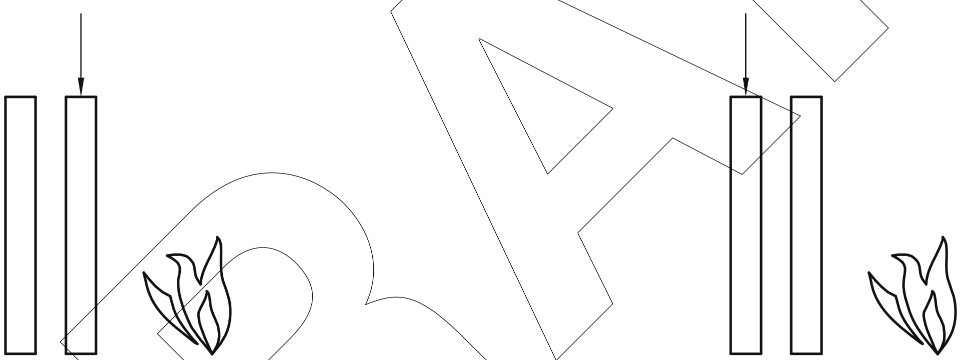
For cavity walls the following shall apply:

(a) Where both leaves have superimposed axial force, with values within 10% of each other, including the case of no superimposed axial force on either leaf, the slenderness ratio shall be based on two-thirds of the sum of the thicknesses of the two leaves and the fixity of the leaf not exposed to the fire [see Figure 6.1(a)].

(b) For all other cases, the slenderness ratio shall be based on the thickness and fixity of the more heavily loaded leaf [see Figure 6.1(b)].

If the two leaves of a cavity wall are constructed of masonry units of different types and the slenderness ratio is determined by Item (a), design for structural adequacy shall be based on the less fire-resistant material.

(a) Both leaves approximately equally loaded



(b) Superimposed load predominantly on one leaf

FIGURE 6.1 CAVITY WALLS EXPOSED TO FIRE

6.3.2.2 *Slenderness ratio*

The slenderness ratio of a member about a given principal axis shall be as follows:

(a) For a wall that has no lateral support on its vertical edges:

$$S_{\mathrm{rf}} = \frac{a_{\mathrm{vf}} H}{t} \qquad \text{. . . 6.3.2.2(1)}$$

(b) For a wall that is laterally supported along one or both of its vertical edges, the least of:

$$S_{\mathrm{rf}} = \frac{a_{\mathrm{vf}} H}{t}\text{; or} \qquad \text{. . . 6.3.2.2(2)}$$

$$S_{\mathrm{rf}} = \frac{0.7}{t}\sqrt{a_{\mathrm{vf}} H a_{\mathrm{h}} L}\text{; or} \qquad \text{. . . 6.3.2.2(3)}$$

$$S_{\mathrm{rf}} = a_{\mathrm{h}} L / t \qquad \text{. . . 6.3.2.2(4)}$$

where

S_{rf} = the slenderness ratio of a member, used in design for fire resistance for structural adequacy

a_{vf} = a slenderness coefficient for assessing slenderness ratio in design for fire resistance for structural adequacy

= 0.75 if the member is laterally supported along its top edge

= 2.0 if the member is not laterally supported along its top edge

H = the clear height of a member between horizontal lateral supports; or

= for a member without top horizontal support, the overall height from the bottom lateral support

t = the overall thickness of the member cross-section perpendicular to the principal axis under consideration

= for members of cavity wall construction, the wall thickness assessed in accordance with Clauses 6.3.2.1(a) and 6.3.2.1(b)

a_h = slenderness coefficients for assessing slenderness ratio

= 1 if the member is laterally supported along both its vertical edges

= 2.5 if the member is laterally supported along one vertical edge

L = the clear length of a wall between vertical lateral supports; or

= for a wall without a vertical support at one end, the length to that unsupported end; or

= for a wall with a control joint, the length to the control joint; or

= for a wall containing an opening, the length to the edge of the opening

A control joint in a wall, or an edge to an opening in a wall, shall be regarded as an unsupported edge to that wall unless specific measures are taken to provide adequate lateral support at that edge.

NOTE: See Clause 2.7 for the requirements of an adequate lateral support.

TABLE 6.1

MAXIMUM SLENDERNESS RATIOS FOR STRUCTURAL ADEQUACY

Type of masonry unit	Fire-resistance period, min					
	30	**60**	**90**	**120**	**180**	**240**
1 Unreinforced masonry:						
(i) Clay units	25.0	22.5	21.0	20.0	18.0	17.0
(ii) Calcium silicate units with basaltic aggregate—						
(A) less than 45% of the total aggregate weight; or	20.5	19.0	18.0	17.5	16.5	15.5
(B) at least 45% of the total aggregate weight	25.0	22.5	21.0	20.0	18.0	17.0
(iii) Concrete units with basaltic aggregate—						
(A) less than 45% of the total aggregate weight; or	19.5	18.0	17.0	16.0	15.5	15.0
(B) at least 45% of the total aggregate weight	25.0	22.5	21.0	20.0	18.0	17.0
2 Reinforced masonry	36.0	36.0	36.0	36.0	36.0	36.0

NOTE: The term 'basaltic' shall be taken to apply to all basic rock types, gabbro, basalt, dolerite, diorite, trachyte, and similar rocks that contain less than 10% quartz.

6.3.3 Design of walls based on test results

Where the design of a wall is based on test results, the following shall apply:

(a) The results shall be from tests in relation to structural adequacy in accordance with AS 1530.4, on a specimen or specimens built using the same type of masonry unit.

For the purpose of this Clause, the masonry units shall be deemed to be of the same type where—

(i) *for clay units*, clay and shales are of the same mineralogy and geological type, blended in the same proportions and manufactured by similar processes; and

(ii) *for calcium-silicate units and concrete units*, aggregates are of the same geological type and grading and cementitious materials are of the same type and grade, blended in the same proportions and manufactured by similar processes.

Where non-loadbearing tests are carried out, the results shall be applied only to non-loadbearing walls.

(b) The fire-resistance period for structural adequacy shall be as follows:

(i) Where only one test result is available, and provided that the slenderness ratio of the member is not greater than that of the tested specimen, the fire resistance period shall be the period to failure or termination of the test.

(ii) Where two or more test results are available, and provided that the values of slenderness ratios of the test specimens cover a minimum range of two and the slenderness ratio of the member is not more than two outside the tested range, the fire resistance period shall be obtained by determining a coefficient C_s as the lowest value from substituting the test results in the following equation:

$$C_s = \frac{(S_{rf} - 13)}{ln(720/t_f)} \qquad \text{. . . 6.3.3}$$

where

S_{rf} = the simplified slenderness ratio of a member

t_f = the period to failure in relation to structural adequacy, in minutes

ln = the natural logarithm

The resultant value of C_s shall be used to calculate the resistance period corresponding to the member slenderness ratio using Equation 6.3.3.

For the purpose of this Clause, the slenderness ratio (S_{rf}) of the test specimen shall be evaluated using a value of a_{vf} = 0.75, in accordance with Clause 6.3.2.2.

6.3.4 Isolated piers

For isolated piers, the slenderness ratio for structural adequacy shall be based on the smaller of the plan dimensions.

For reinforced piers where the reinforcement is near two exposed faces, the cover for protection to the reinforcement shall be at least 1.5 times that required by Clause 6.3.6.

6.3.5 Minimum reinforcement

For reinforced masonry, where the slenderness ratio is obtained from Table 6.1, the reinforcement shall comply with the following:

(a) Where the member spans horizontally, the reinforcement shall be such that the member is capable of withstanding a design lateral load of 0.5 kPa.

(b) Where the member spans vertically, the reinforcement shall be designed such that the member is capable of withstanding the greater of—

  (i) a flexural moment equivalent to the applied vertical compressive load on the wall times the height of the wall divided by 36; or

  (ii) a design lateral load of 0.5 kPa.

NOTES:

1 The minimum reinforcement requirement for structural purposes is given in Section 8.

2 The design loads specified in this Clause do not require additional factoring.

6.3.6 Protection to reinforcement

The protection to the reinforcement in a reinforced masonry member that is required to provide fire resistance for structural adequacy shall be not less than the value given in Table 6.2.

NOTE: See Clause 6.3.4 for requirements for isolated piers.

TABLE 6.2

PROTECTION TO REINFORCEMENT FOR STRUCTURAL ADEQUACY

Fire-resistance period, min	30	60	90	120	180	240
Minimum dimension from the reinforcement to the exposed face of the masonry, mm	30	30	30	40	50	60

6.4 INTEGRITY

6.4.1 General

Members shall be designed for fire resistance for integrity in accordance with Clause 6.4.2 or Clause 6.4.3.

6.4.2 Design from tabulated values

Where test results for integrity are not available, a member shall be deemed to have the required level of fire resistance for integrity if it meets the requirements of Table 6.1 for structural adequacy (at the required fire resistance level for integrity) and the requirements of Table 6.3 for insulation (at the required fire resistance level for integrity).

6.4.3 Design based on test results

Where the design of a wall is based on test results, the following shall apply:

(a) The results shall be from tests for integrity conducted in accordance with AS 1530.4, on a specimen or specimens built using the same type of masonry unit.

For the purpose of this Clause, the masonry units shall be deemed to be of the same type where—

  (i) *for clay units*, clay and shales are of the same mineralogy and geological type, blended in the same proportions and manufactured by similar processes; and

(ii) *for calcium-silicate units and concrete units*, aggregates are of the same geological type and grading and cementitious materials are of the same type and grade, blended in the same proportions and manufactured by similar processes.

Where non-loadbearing tests are carried out, the results shall be applied only to non-loadbearing walls.

(b) The fire resistance period in relation to integrity shall be as follows:

(i) Where only one test result is available, and provided the overall thickness of the member is not less than that of the tested specimen, the fire resistance period shall be the period to failure or termination of the test.

(ii) Where two or more test results are available, and provided the overall thickness of the test specimens covers a minimum range of 20 mm, the fire resistance period shall be obtained by linear interpolation between the test values on the basis of thickness. In cases outside the tested range of thickness, the fire resistance period for integrity shall be taken as the lesser of the values obtained by extrapolation for structural adequacy and insulation in accordance with Clause 6.3.3(b)(ii) or Clause 6.5.4(b)(ii).

6.5 INSULATION

6.5.1 General

Members shall be designed for fire resistance for insulation in accordance with Clause 6.5.3 or Clause 6.5.4.

6.5.2 Material thickness of member

For determining the fire-resistance period in relation to insulation, the material thickness of a wall shall be as follows:

(a) *For a single leaf wall*—

(i) where the wall is built from units with a proportion of cores not greater than 30% or built from fully grouted units, the material thickness shall be the overall thickness of the wall; or

(ii) where the wall is built from units with a proportion of cores greater than 30% and not fully grouted, the material thickness shall be the net material volume of an ungrouted unit divided by the area of the vertical exposed face of the unit.

(b) *For a cavity wall*, the material thickness shall be the sum of the material thicknesses of the separate leaves of the wall.

In determining the material thickness for a cement-rendered wall or leaf, any render applied to a face exposed to fire shall not be considered. Render applied to a face not exposed to fire may be considered for up to a maximum thickness of 20 mm.

6.5.3 Design of walls using tabulated values

When a wall constructed with filled perpend joints is designed using tabulated values, the material thickness shall be not less than the relevant value obtained from Table 6.3.

NOTES:

1 Face-shell-bedded perpend joints are considered to be filled perpends.

2 No tabulated values are available for walls that do not have all perpend joints filled with mortar (e.g. walls with horizontally cored units).

If the two leaves of a cavity wall are of masonry units of different types, design shall be based on the less fire-resistant material (with the greater required thickness given in Table 6.3.)

TABLE 6.3

MATERIAL THICKNESS FOR INSULATION OF WALLS WITH FILLED PERPEND JOINTS

Type of masonry unit	Material thickness of wall, mm					
	Fire-resistance period, min					
	30	60	90	120	180	240
Clay	60	90	110	130	160	180
Calcium silicate	50	70	90	110	135	160
Concrete with density:						
(a) >1800 kg/m^3	55	80	100	120	150	180
(b) ≤1800 kg/m^3	55	75	90	110	135	160

6.5.4 Design based on test results

Where the design of a member is based on test results, the following shall apply:

(a) The results shall be from tests for insulation, conducted in accordance with AS 1530.4, on a specimen or specimens built using the same type of masonry unit.

For the purpose of this Clause, the masonry units shall be deemed to be of the same type where—

(i) *for clay units*, clay and shales are of the same mineralogy and geological type, blended in the same proportions and manufactured by similar processes; and

(ii) *for calcium-silicate units and concrete units*, aggregates are of the same mineralogy, geological type and grading and cementitious materials are of the same type and grade, blended in the same proportions and manufactured by similar processes.

(b) The fire-resistance period in relation to insulation shall be as follows:

(i) Where only one test result is available, and provided the material thickness and the overall thickness of the member are not less than that of the tested specimen, the fire resistance period shall be the period to failure or termination of the test.

(ii) Where two or more test results are available, provided the material thickness of the test specimens covers a minimum range of 20 mm and the member thickness is not more than 20 mm outside the range of the tested thicknesses, the fire resistance period shall be obtained by determining a coefficient C_i as the lowest value from substituting the test results in the following equation:

$$t_c = C_i\, t_m^{1.7} \qquad \text{. . . 6.5.4}$$

where

t_c = the period to failure in relation to insulation, in minutes

t_m = the material thickness of the member, in millimetres

The resultant value of C_i shall be used to calculate the resistance period corresponding to the member thickness using Equation 6.5.4.

6.6 RECESSES FOR SERVICES

Provided the depth of material removed is not greater than half the wall thickness and the total area of recesses is not greater than 10 000 mm^2 total of both faces within any 5 m^2 of wall area, the effect of recesses for services on the fire resistance periods for structural adequacy, integrity and insulation of a wall shall be ignored.

Where these limits are exceeded, the member thickness of the masonry (t) shall be taken as the overall thickness of the wall less the depth of recess.

Where the wall is constructed of cored or hollow units, the recess may extend into the cores. Any such extension shall be considered part of the recess.

6.7 CHASES

6.7.1 General

The chasing of masonry members that are subject to fire shall be kept to a minimum.

6.7.2 The effect of chases on structural adequacy

The effect of chases on the fire-resistance period for structural adequacy shall be dealt with as follows:

(a) For vertically spanning walls—

  (i) where the chase is vertical—ignored;

  (ii) where the chase is horizontal and of length not greater than four times the wall thickness—ignored; and

  (iii) where the chase is horizontal and of length greater than four times the wall thickness—considered, using the slenderness ratio of the wall, based on the wall thickness at the bottom of the chase.

(b) For walls spanning vertically and horizontally (panel action)—

  (i) where the length of chase is not greater than half the wall height (for a vertical chase) or half the wall length (for a horizontal chase)—ignored;

  (ii) where the length of chase is greater than half the wall height (for a vertical chase) or half the wall length (for a horizontal chase)—considered, using the slenderness ratio of the wall, based on the wall thickness at the bottom of the chase.

In the case of a vertical chase, it is acceptable for the chase to be regarded as an unsupported edge and the panel designed as two sub-panels.

6.7.3 The effect of chases on integrity and insulation

The following apply to the effect of chases on the fire-resistance periods for integrity and insulation:

(a) For walls constructed of solid, cored or grouted hollow units—

  (i) the effects shall be ignored where the depth of material removed is not greater than 30 mm, the cross-sectional area (normal to the plane of the wall) of the chase is not greater than 1000 mm^2, and the total face area of the chase is not greater than 100 000 mm^2 on both faces of the wall in any 5 m^2 area; and

  (ii) for other cases, the effects shall be taken into account by calculating the integrity and insulation, based on the thickness of the wall at the base of the chase.

(b) For walls of ungrouted hollow masonry units, the integrity and insulation requirements shall be based on the thickness of the wall at the base of the chase.

6.8 PROTECTION OF STRUCTURAL STEELWORK

Where masonry is used to protect steelwork that requires a fire resistance level, including mullions in cavity walls, the material thickness of the masonry shall be not less than—

(a) the value from Table 6.4; or

(b) a value determined in accordance with AS 4100.

In the application of Table 6.4 the following shall apply:

(i) Material thickness of masonry shall be calculated in accordance with Clause 6.5.2, except that the limit on the proportion of cores in Clause 6.5.2(a) shall be 25%.

(ii) 'Filled column spaces' means that all spaces between the masonry and the steel (including any re-entrant parts of the column itself) shall be filled with grout or masonry.

TABLE 6.4

MATERIAL THICKNESS FOR PROTECTION OF STRUCTURAL STEELWORK

Column spaces	Masonry material	Material thickness of masonry, mm				
		Fire resistance period (insulation/integrity/structural adequacy), min				
Exposed to fire on not more than 3 sides		**60/60/60**	**90/90/90**	**120/120/120**	**180/180/180**	**240/240/240**
Filled	Clay	50	50	50	65	90
	Concrete	50	50	50	65	90
	Calcium silicate	50	50	50	50	65
Not filled	Clay	50	50	65	—	—
	Concrete	50	50	65	—	—
	Calcium silicate	50	50	50	—	—
Exposed to fire on 4 sides		**60/—/—**	**90/—/—**	**120/—/—**	**180/—/—**	**240/—/—**
Filled	Clay	50	50	50	75	100
	Concrete	50	50	50	75	100
	Calcium silicate	50	50	50	65	75
Not filled	Clay	50	50	65	—	—
	Concrete	50	50	65	—	—
	Calcium silicate	50	50	50	—	—

SECTION 7 STRUCTURAL DESIGN OF UNREINFORCED MASONRY

7.1 GENERAL

This Section sets out specific requirements for the structural design of unreinforced masonry for the strength and serviceability limit states. These requirements are in addition to the general requirements of Section 4.

Masonry containing reinforcement that does not comply with the structural design requirements of Section 8 shall be regarded as unreinforced masonry and shall comply with the requirements of this Section.

7.2 GENERAL BASIS OF DESIGN

Each unreinforced masonry member shall be designed to comply with Clauses 7.3 to 7.5, taking into account the strength of the material and the further provisions in Clauses 7.6 to 7.8, as applicable, for the relevant type of member.

The value of the design bending moment shall include the bending moments, if any, resulting from load eccentricities or bending moments applied at the ends of the member.

For a member under combined bending and compression, the following shall apply:

(a) Where—

(i) any bending moments acting on the wall, other than those arising from the laterally applied loads, are small enough to be ignored; and

(ii) the compressive stresses on any bed joint in the wall from any simultaneously acting loads do not exceed $3f'_{mt}$ in magnitude,

the member shall be designed to comply with Clause 7.4 for bending and to comply with Clause 7.3 for all forces and moments other than those resulting from short-term out-of-plane loads.

(b) In all other cases, the member shall be designed to comply with Clause 7.3 for the combined action of all forces and moments.

7.3 DESIGN FOR MEMBERS IN COMPRESSION

7.3.1 General

Unreinforced masonry members resisting compressive forces, with or without simultaneously acting bending moments, shall be designed to comply with Clause 7.3.3 or Clause 7.3.4. Members with concentrated loads shall be designed to comply with Clause 7.3.5.

Any unreinforced masonry member that is designed to resist compressive forces caused by other than its own weight shall have a minimum thickness of at least 90 mm.

In determining the compressive capacity of a masonry member, the following factors shall be considered:

(a) Slenderness.

(b) Effective eccentricity of loading at each end.

(c) Characteristic compressive strength of the masonry.

(d) Cross-sectional area of the masonry.

In a wall or isolated pier subject to compression and bending, the vertical and bending forces shall be considered at top and bottom of the member by regarding the vertical force as acting at statically equivalent effective eccentricities, (e_1) and (e_2) respectively, at each end. In this calculation, the most unfavourable disposition of live loads shall be considered.

NOTE: In designing by simple rules in accordance with Clause 7.3.3, it is not necessary to calculate the end eccentricities explicitly.

Although it is assumed in cavity wall construction that each leaf can give support to the other leaf for robustness purposes (see Clause 4.6) or for withstanding lateral loads (see Clause 7.7.3), no such mutual support shall be assumed or relied upon with respect to buckling actions under compressive forces.

7.3.2 Basic compressive capacity

The basic compressive capacity of the cross-section (F_o) shall be taken as follows:

(a) For ungrouted masonry:

$$F_o = \phi f'_m A_b \qquad \text{. . . 7.3.2(1)}$$

(b) For grouted masonry:

(i) If no testing is done:

$$F_o = \varphi \left[f'_m A_b + k_c \left(\frac{f'_{cg}}{1.3} \right)^{(0.55+0.005 f'_{cg})} A_g \right] \qquad \text{. . . 7.3.2(2)}$$

(ii) If testing is done:

$$F_o = \phi f'_{mg} A_d \qquad \text{. . . 7.3.2(3)}$$

where

ϕ = the capacity reduction factor (see Clause 4.4)

f'_m = the characteristic compressive strength of the masonry (see Clause 3.3.2)

A_b = the bedded area of a masonry member cross-section (see Clause 4.5.4)

k_c = a strength factor for grout in compression

= 1.4 for hollow concrete masonry units of density greater than 2000 kg/m^3

= 1.2 for all other masonry

f'_{cg} = the design characteristic compressive strength of grout, in megapascals (see Clause 3.5)

A_g = the design cross-sectional area of grout (see Clause 4.5.7)

f'_{mg} = the characteristic compressive strength of grouted masonry specimens manufactured and tested in accordance with Appendix C and determined in accordance with Appendix B

A_d = the design cross-sectional area of the member (see Clause 4.5.6)

7.3.3 Design by simple rules

7.3.3.1 *General*

Clauses 7.3.3.1 to 7.3.3.3 apply to unreinforced masonry walls with or without engaged piers, and to isolated piers of rectangular cross-section. The possibility of buckling about each of the two principal axes of the cross-section of the member shall be assessed.

For design under this Clause (7.3.3), it is not necessary to consider the interaction of vertical and lateral loads acting simultaneously; the resistance to lateral loading shall be checked independently in accordance with Clause 7.4.

If the member under consideration has insufficient capacity under the simple rules, its capacity may be determined by refined calculation as specified in Clause 7.3.4.

For members of cavity wall construction, each leaf shall be assessed separately and individually for thickness and for slenderness ratio.

7.3.3.2 *Compression on uniform symmetrical members*

A member shall be designed such that the following relationship is satisfied:

$$F_d \leq k\, F_o \qquad \text{. . . 7.3.3.2}$$

where

F_d = the design compressive force acting on the cross-section of the member simultaneously with a bending moment, shear force or other load action

F_o = the basic compressive strength capacity (see Clause 7.3.2)

k = a reduction factor for slenderness and eccentricity in accordance with Clause 7.3.3.3

Where the load is applied directly to an engaged pier, the capacity of the pier shall be determined in accordance with Clauses 7.3.4 or 7.3.5.

7.3.3.3 *Reduction factor for slenderness and eccentricity (k)*

The reduction factor slenderness and eccentricity (k) shall be determined as follows:

(a) For a wall or pier supporting a concrete slab as defined in Table 7.1, k is the lesser of—

(i) $0.67 - 0.02(S_{rs} - 14)$; or

(ii) 0.67

(b) For a wall or pier supporting other systems as defined in Table 7.1, k is the lesser of—

(i) $0.67 - 0.025(S_{rs} - 10)$; or

(ii) 0.67

(c) For a wall with the load applied to the face as defined in Table 7.1, k is the lesser of—

(i) $0.067 - 0.002(S_{rs} - 14)$; or

(ii) 0.067

The following shall apply:

(A) For masonry not reinforced for vertical bending, the wall shall extend at least one storey above the point of load application.

(B) For single-leaf walls, the minimum thickness shall be 140 mm.

(C) For cavity walls, the thickness of the loaded leaf shall be not less than 100 mm and the sum of the thicknesses of the two leaves not less than 200 mm. (Since the floor line is a line of lateral support, cavity wall ties in this region shall be at 300 mm centres maximum.)

(D) If a load is applied to both faces, the wall shall be designed for the difference between the two loads.

In Items (a)(i), (b)(i) and (c)(i), S_{rs} is the simplified slenderness ratio determined in accordance with Clause 7.3.3.4.

The values of *k* given in Table 7.1 shall be deemed to satisfy the equations in this Clause. In all cases, the capacity of the wall shall be checked for the total load on the wall at any level.

7.3.3.4 *Simplified slenderness ratio (S_{rs})*

The simplified slenderness ratio (S_{rs}) of a member about a given principal axis shall be calculated from the following equation:

$$S_{rs} = \frac{a_v H}{k_t t} \qquad \text{. . .7.3.3.4}$$

where

S_{rs} = the simplified slenderness ratio

a_v = vertical slenderness coefficient

= 1 if the member is laterally supported along its top edge

= 2.5 if the member is not laterally supported along its top edge

H = the clear height of a member between horizontal lateral supports, in millimetres

= for a member without top horizontal support, the overall height from the bottom lateral support

k_t = a thickness coefficient derived from Table 7.2

= 1 if there are no engaged piers

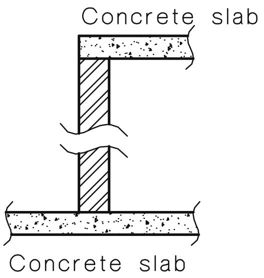
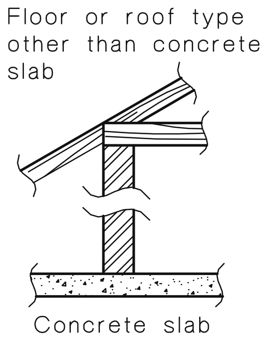
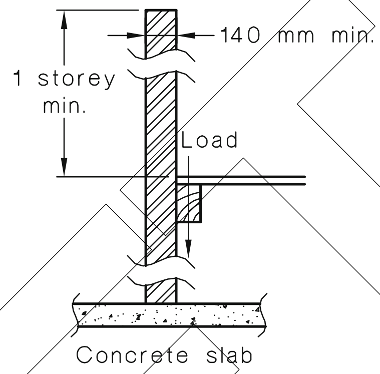
If the engagement of a pier to the wall does not meet the requirements of Clause 4.11 for bonding or tying, the value of k_t shall be taken as 1.0.

t = the overall thickness of the member's cross-section perpendicular to the principal axis under consideration, in millimetres

t_{wp} = the overall thickness of a masonry wall plus an engaged pier or buttress (see Table 7.2), in millimetres

TABLE 7.1

REDUCTION FACTOR (*k*) FOR SLENDERNESS AND ECCENTRICITY FOR DESIGN BY SIMPLE RULES

S_{rs}	Reduction factor (*k*)		
			
≤8	0.67	0.67	0.067
10	0.67	0.67	0.067
12	0.67	0.63	0.067
14	0.67	0.58	0.067
16	0.63	0.53	0.063
18	0.59	0.48	0.059
20	0.55	0.43	0.055
22	0.51	0.38	0.051
24	0.47	0.33	0.047
26	0.43	0.28	0.043
28	0.39	0.23	0.039
30	0.35	0.18	0.035
32	0.31	0.13	0.031
34	0.27	0.08	0.027
36	0.23	0.03	0.023

NOTES:

1 Linear interpolation within each category is permitted.

2 This Table does not cover the case of load applied to one face of the wall at roof level. Because of the low pre-compression at this level, a detail that minimizes the load eccentricity should be used.

7.3.4 Design by refined calculation

7.3.4.1 *General*

The provisions of this Clause (7.3.4) are for uniform symmetrical members in uniaxial bending and compression, including unreinforced masonry walls, with or without engaged piers, and isolated piers. Other cases may be designed using the same general principles with appropriate modification based on the principles of structural mechanics.

NOTE: A simplified approach to the design of non-uniform and non-symmetrical members, such as L- and T-sections, is to design for a rectangular wall of the same overall height and length with an equivalent thickness such that the radius of gyration of the rectangular wall is the same as that of the section being designed (i.e. $r = \sqrt{\frac{I}{A}}$, where I is the moment of inertia and A is the area of the section).

The possibility of buckling about each of the two principal axes of the cross-section of the member shall be assessed.

For members of cavity wall construction, each leaf shall be assessed separately and individually for thickness and for slenderness ratio.

7.3.4.2 *Uniaxial eccentric compression on uniform symmetrical members*

A member shall be designed such that the following relationship is satisfied:

$$F_d \leq kF_o \quad \ldots 7.3.4.2$$

where

F_d = the design compressive force that acts on the cross-section of a member simultaneously with a bending moment, shear force or other load action

k = a reduction factor for slenderness and eccentricity (see Clause 7.3.4.5)

F_o = the basic compressive strength capacity (see Clause 7.3.2)

TABLE 7.2

THICKNESS COEFFICIENTS (k_t) FOR WALLS STIFFENED BY MONOLITHICALLY ENGAGED PIERS

Pier spacing/pier width (see Note 1)	Thickness coefficient (k_t)		
	Pier thickness ratio (t_{wp}/t)		
	1	2	3
6	1.0	1.4	2.0
8	1.0	1.3	1.7
10	1.0	1.2	1.4
15	1.0	1.1	1.2
20 or more	1.0	1.0	1.0

NOTES:

1 Pier spacing is taken as the distance between centre-lines of piers.

2 Linear interpolation may be used.

7.3.4.3 *Slenderness ratio*

The slenderness ratio of a member (S_r) about a given principal axis shall be as follows:

(a) For a wall that is laterally supported along one or both of its vertical edges and subject to design compressive force $F_d \leq 0.20\, F_o$, the lesser of—

$$S_r = \frac{a_v H}{k_t t}; \text{ or} \quad \ldots 7.3.4.3(1)$$

$$S_r = \frac{0.7}{t}\sqrt{a_v H a_h L} \quad \ldots 7.3.4.3(2)$$

where

S_r = the slenderness ratio of a member

a_v = vertical slenderness coefficient

= 0.75 for a wall laterally supported and partially rotationally restrained at both top and bottom

= 0.85 for a wall laterally supported at top and bottom and partially rotationally restrained at one of them

= 1.0 for a wall laterally supported at both top and bottom

= 1.5 for a wall laterally supported and partially rotationally restrained at the bottom and partially laterally supported at the top

= 2.5 for a freestanding wall

The situations shown and described in Figures 7.1(A) to 7.1(E) are deemed to provide the appropriate values.

H = the clear height of the wall between horizontal lateral supports; or

= for a wall without top horizontal support, the overall height from the bottom lateral support

k_t = a thickness coefficient derived from Table 7.2

If the engagement of a pier to the wall does not meet the requirements of Clause 4.11 for bonding or tying, the value of k_t shall be taken as 1.0

t = the overall thickness of the wall's cross-section perpendicular to the principal axis under consideration

NOTE: Each leaf of a cavity wall is considered separately (see Clause 7.3.1).

a_h = horizontal slenderness coefficient

= 1.0 for a wall laterally supported along both vertical edges (regardless of the rotational restraint along these edges); or

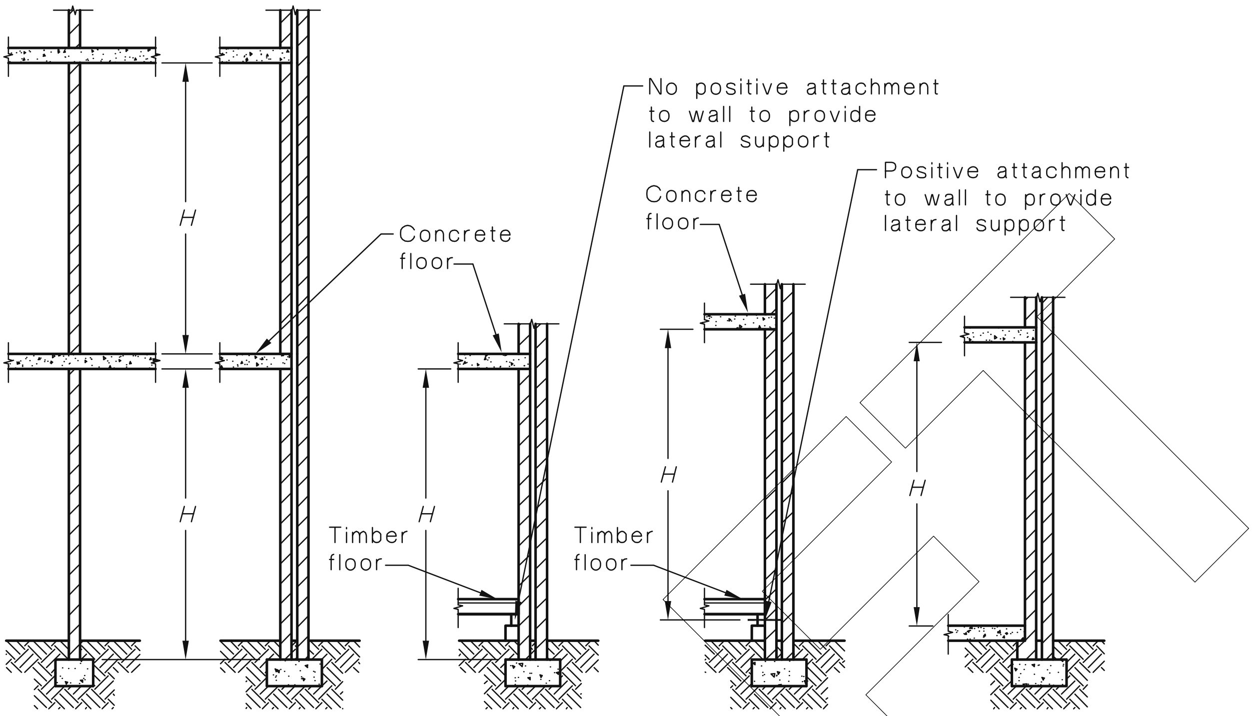
= 2.5 for a wall laterally supported along one vertical edge, and unsupported along its other vertical edge

The situations shown and described in Figure 7.2 are deemed to provide the appropriate values.

L = the clear length of the wall between vertical lateral supports; or

= for a wall without a vertical support at its end or at a control joint, the length to that unsupported end or control joint

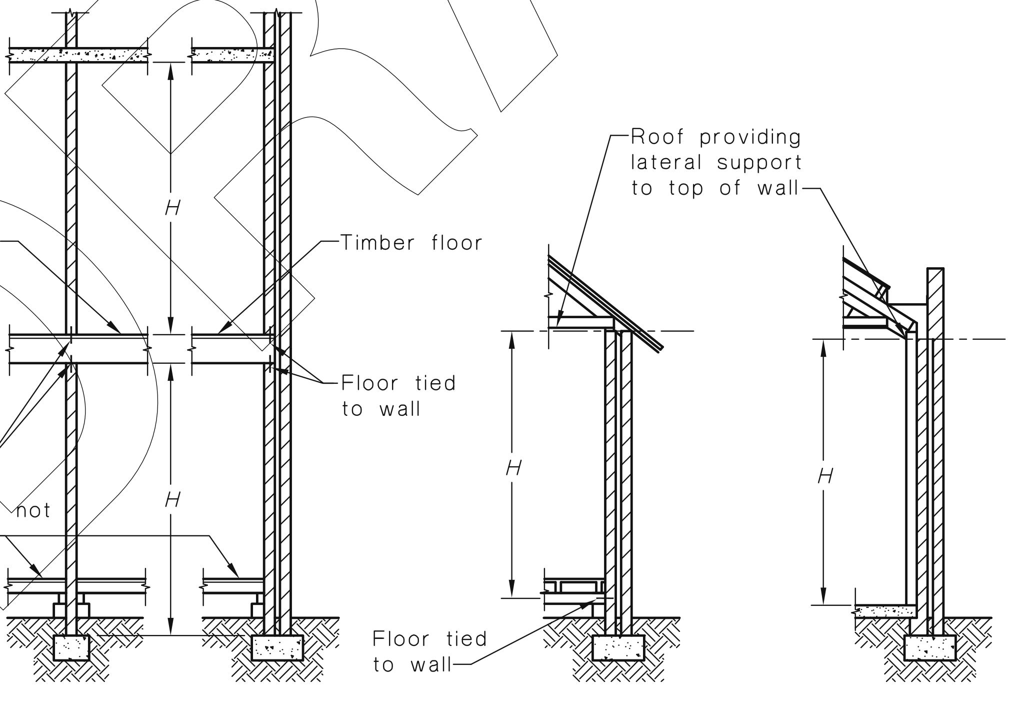
A wall shall not be regarded as being laterally supported at an edge unless that lateral support meets the requirements of Clause 2.7, in the direction perpendicular to the principal axis under consideration.



$a_v = 0.75$

NOTE: Concrete floor systems spanning from one side only shall have a bearing depth of at least two-thirds of the thickness of the supporting wall or leaf and not less than 85 mm.

FIGURE 7.1(A) VALUE a_v—LATERAL SUPPORT AND PARTIAL ROTATIONAL RESTRAINT AT TOP AND BOTTOM (FULLY BRACED CONSTRUCTION)



$a_v = 0.85$

FIGURE 7.1(B) VALUE a_v—LATERAL SUPPORT AT TOP AND BOTTOM AND PARTIAL ROTATIONAL RESTRAINT AT ONE END (FULLY BRACED CONSTRUCTION)

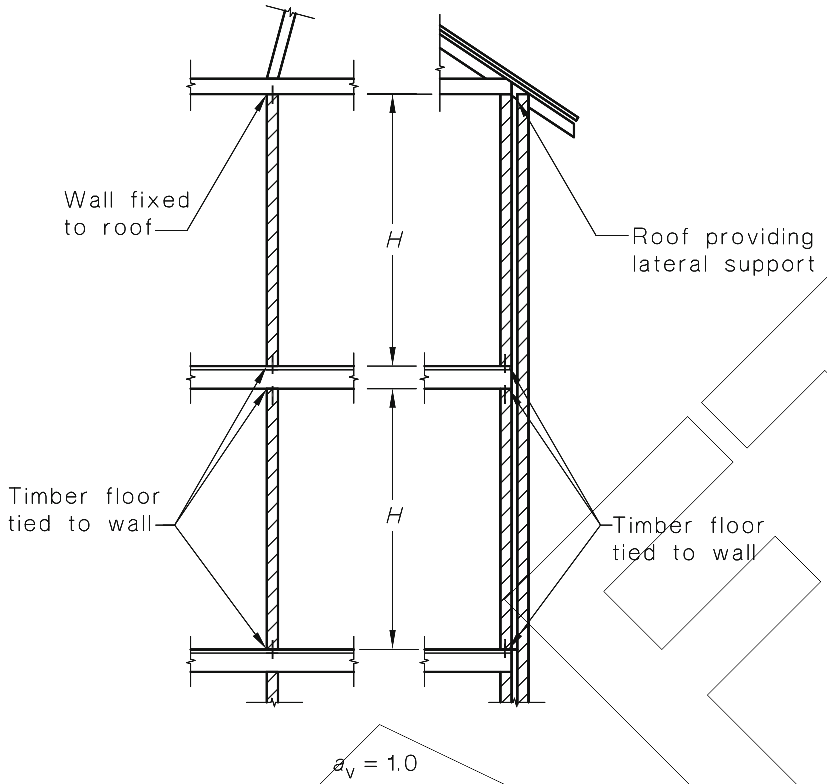


FIGURE 7.1(C) VALUE a_v—LATERAL SUPPORT AT TOP AND BOTTOM (FULLY BRACED CONSTRUCTION)

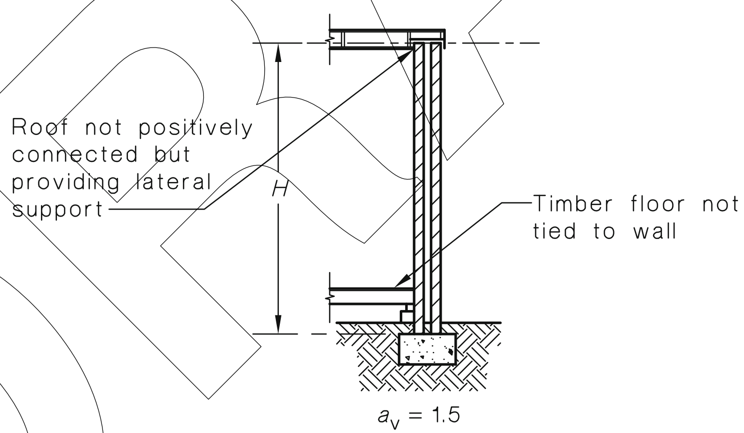


FIGURE 7.1(D) VALUE a_v—LATERAL SUPPORT AND PARTIAL ROTATIONAL RESTRAINT AT THE BOTTOM AND PARTIAL LATERAL SUPPORT AT THE TOP

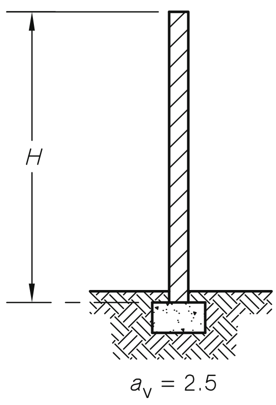


FIGURE 7.1(E) VALUE a_v—MEMBERS FREE AT THE TOP

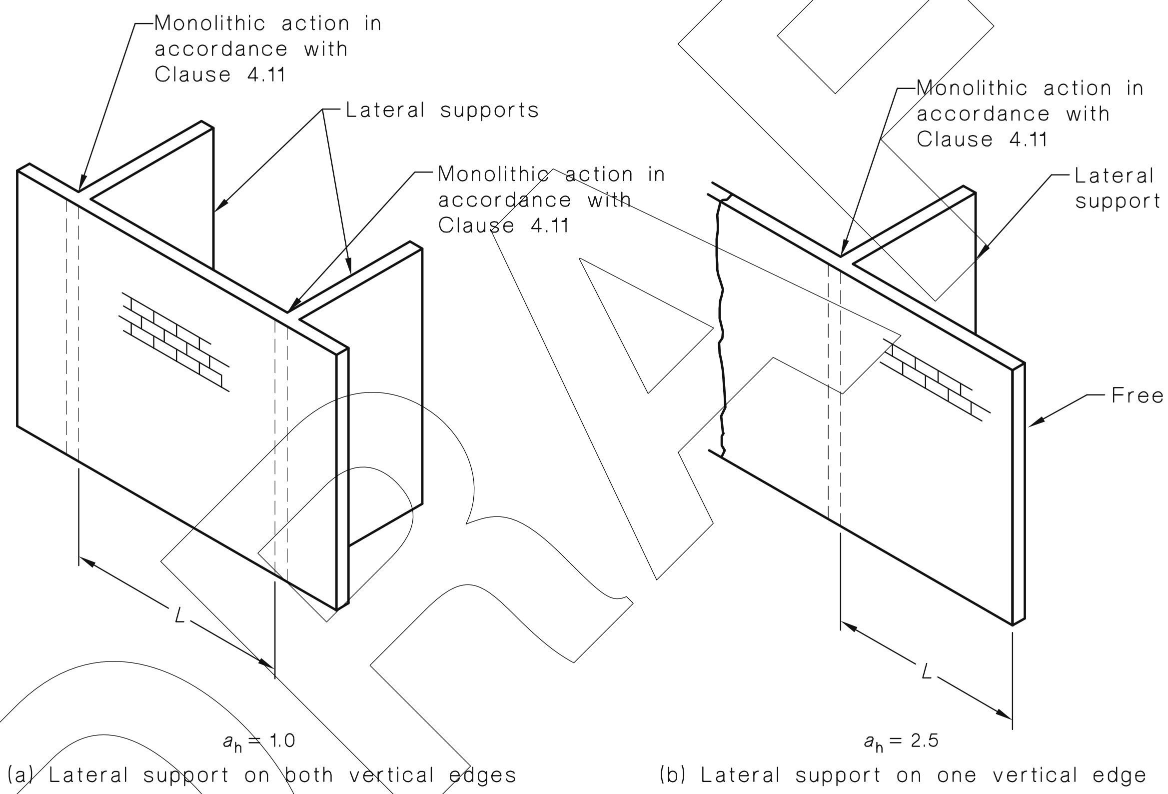


FIGURE 7.2 VALUES a_h—LATERAL SUPPORT

(b) Where openings in a wall are such that the masonry between any two consecutive openings is by definition an isolated pier, the slenderness ratio of that isolated pier shall be taken as the lesser of the following:

(i)
$$S_r = \frac{2H_1}{t} \qquad \ldots 7.3.4.3(3)$$

where

S_r = the slenderness ratio of the member

H_1 = the height of the taller opening

t = the overall thickness or depth of the member's cross-section perpendicular to the principal axis under consideration

(ii) The value for the pier assessed in accordance with this Clause.

(c) For all other cases:

$$S_r = \frac{a_v H}{k_t t} \qquad \ldots 7.3.4.3(4)$$

where

S_r = the slenderness ratio of a member

a_v = vertical slenderness coefficient

= 0.75 for a member laterally supported and partially rotationally restrained at both top and bottom

= 0.85 for a member laterally supported at top and bottom and partially rotationally restrained at one of them

= 1.0 for a member laterally supported at both top and bottom

= 1.5 for a member laterally supported and partially rotationally restrained at the bottom and partially laterally supported at the top

= 2.5 for a freestanding wall or pier

The situations shown and described in Figures 7.1(A) to 7.1(E) are deemed to provide the appropriate values.

H = The clear height of the member between horizontal lateral supports; or

= for a member without top horizontal support, the overall height from the bottom lateral support

k_t = a thickness coefficient derived from Table 7.2

If the engagement of a pier to the wall does not meet the requirements of Clause 4.11 for bonding or tying, the value of k_t shall be taken as 1.0.

t = the overall thickness of the member's cross-section perpendicular to the principal axis under consideration

NOTE: Each leaf of a cavity wall is considered separately (see Clause 7.3.1).

7.3.4.4 *Effective eccentricity*

The effective load eccentricity shall be determined taking into account the relative stiffness of the masonry members (or member) and floor slabs or beams that the masonry supports, the relative rotation of the slab or beams and the supporting masonry member and any other factors that affect the interaction of the components framing into the joint.

The requirement for effective eccentricity shall be deemed to be satisfied if the effective eccentricity is calculated assuming that the load transmitted to a wall by a single floor or roof acts at one-third of the depth of the bearing area from the loaded face of the wall or loadbearing leaf. Where a supported floor or roof is continuous over a wall, each side of the floor or roof shall be taken as being individually supported on one-half of the total bearing area. The resulting eccentricity of load, at any level, shall be calculated on the assumption that the total vertical load on the wall above the plane under consideration is axial immediately above the horizontal plane under consideration.

Alternatively, for walls with a design compressive stress immediately above the plane under consideration greater than 0.25 MPa, a rigid frame analysis may be used with the appropriate allowance for relative slab/wall rotation at the joints.

7.3.4.5 *Reduction factor (k) for slenderness and eccentricity for refined calculation*

The reduction factor for slenderness and eccentricity (k) shall be not less than zero and shall be the lesser of the values calculated in Items (a) and (b), as follows:

(a) For lateral instability:

$$k = 0.5\left(1+\frac{e_2}{e_1}\right)\left[\left(1-2.083\frac{e_1}{t_\mathrm{w}}\right)-\left(0.025-0.037\frac{e_1}{t_\mathrm{w}}\right)(1.33S_\mathrm{r}-8)\right] + 0.5\left(1-0.6\frac{e_1}{t_\mathrm{w}}\right)\left(1-\frac{e_2}{e_1}\right)(1.18-0.03S_\mathrm{r}) \quad \ldots 7.3.4.5(1)$$

(b) For local crushing:

(i) For members of solid cross-section with full bedding or walls fully grouted:

$$k = 1-2\frac{e_1}{t_\mathrm{w}} \quad \ldots 7.3.4.5(2)$$

(ii) For members with face shell bedding or a diaphragm wall, the lesser of:

$$k = \frac{1-\dfrac{t_\mathrm{fs}}{t_\mathrm{w}}}{1-\dfrac{t_\mathrm{fs}}{t_\mathrm{w}}+2\dfrac{e_1}{t_\mathrm{w}}} \quad \ldots 7.3.4.5(3)$$

and

$$k = \frac{1}{2\dfrac{t_\mathrm{fs}}{t_\mathrm{w}}}\left(1-2\frac{e_1}{t_\mathrm{w}}\right) \quad \ldots 7.3.4.5(4)$$

where

e_1 = the larger eccentricity of the vertical force, at either top or bottom of the member, not less than $0.05t_\mathrm{w}$

e_2 = the smaller eccentricity of the vertical force, at the other end of the member, not less than e_1, and negative when the eccentricities are on opposite sides of the member

t_w = the overall thickness of the wall or isolated pier

t_{fs} = the thickness of the face shell for hollow block masonry or the flange thickness for diaphragm walls

The values given in Table 7.3 shall be deemed to satisfy Equations 7.3.4.5(1) to 7.3.4.5(4) for solid and cored unit masonry. Grouted hollow unit masonry shall be treated as for solid masonry.

The values given in Table 7.4 shall be deemed to satisfy Equations 7.3.4.5(1) to 7.3.4.5(4) for ungrouted hollow unit masonry.

TABLE 7.3

REDUCTION FACTOR (k) FOR SLENDERNESS AND ECCENTRICITY—SOLID AND CORED UNIT MASONRY

Slenderness ratio (S_r)	Reduction factor (k)															
	Eccentricity to thickness ratio (e_1/t_w)															
	0.05			0.10			0.20			0.30			0.40			0.50
	Ratio of end eccentricities (e_2/e_1)															
	1	0	–1	1	0	–1	1	0	–1	1	0	–1	1	0	–1	All
6	0.90	0.90	0.90	0.79	0.80	0.80	0.58	0.60	0.60	0.38	0.40	0.40	0.17	0.20	0.20	0.00
8	0.83	0.87	0.90	0.74	0.80	0.80	0.54	0.60	0.60	0.34	0.40	0.40	0.14	0.20	0.20	0.00
10	0.77	0.81	0.85	0.68	0.75	0.80	0.49	0.60	0.60	0.30	0.40	0.40	0.11	0.20	0.20	0.00
12	0.71	0.75	0.80	0.62	0.70	0.77	0.44	0.58	0.60	0.26	0.40	0.40	0.09	0.20	0.20	0.00
14	0.65	0.69	0.74	0.57	0.64	0.71	0.40	0.53	0.60	0.23	0.40	0.40	0.06	0.20	0.20	0.00
16	0.59	0.63	0.68	0.51	0.58	0.66	0.35	0.48	0.60	0.19	0.38	0.40	0.03	0.20	0.20	0.00
18	0.53	0.57	0.62	0.45	0.53	0.60	0.30	0.43	0.56	0.15	0.34	0.40	0.00	0.20	0.20	0.00
20	0.47	0.51	0.56	0.40	0.47	0.55	0.26	0.38	0.51	0.12	0.30	0.40	0.00	0.20	0.20	0.00
22	0.40	0.45	0.50	0.34	0.41	0.49	0.21	0.33	0.46	0.08	0.25	0.40	0.00	0.17	0.20	0.00
24	0.34	0.39	0.45	0.28	0.36	0.43	0.16	0.28	0.40	0.04	0.21	0.38	0.00	0.14	0.20	0.00
26	0.28	0.33	0.39	0.23	0.30	0.38	0.12	0.23	0.35	0.01	0.17	0.33	0.00	0.10	0.20	0.00
27	0.25	0.30	0.36	0.20	0.27	0.35	0.09	0.21	0.33	0.00	0.15	0.30	0.00	0.08	0.20	0.00

NOTES:

1 Linear interpolation may be used between the values given in the Table.

2 Shaded cells in the Table indicate that failure would be by crushing. For other values, failure would be by instability.

TABLE 7.4

REDUCTION FACTOR (k) FOR SLENDERNESS AND ECCENTRICITY—FOR HOLLOW UNIT MASONRY

Slenderness ratio (S_r)	Reduction factor (k)															
	Eccentricity to thickness ratio (e_1/t_w)															
	0.05			0.10			0.20			0.30			0.40			0.50
	Ratio of end eccentricities (e_2/e_1)															
	1	0	–1	1	0	–1	1	0	–1	1	0	–1	1	0	–1	All
6	0.89	0.89	0.89	0.79	0.80	0.80	0.58	0.67	0.67	0.38	0.58	0.58	0.17	0.46	0.51	0.00
8	0.83	0.87	0.89	0.74	0.80	0.80	0.54	0.67	0.67	0.34	0.55	0.58	0.14	0.43	0.51	0.00
10	0.77	0.81	0.85	0.68	0.75	0.80	0.49	0.63	0.67	0.30	0.51	0.58	0.11	0.39	0.51	0.00
12	0.71	0.75	0.80	0.62	0.70	0.77	0.44	0.58	0.67	0.26	0.47	0.58	0.09	0.35	0.51	0.00
14	0.65	0.69	0.74	0.57	0.64	0.71	0.40	0.53	0.67	0.23	0.43	0.58	0.06	0.32	0.51	0.00
16	0.59	0.63	0.68	0.51	0.58	0.66	0.35	0.48	0.62	0.19	0.38	0.57	0.03	0.28	0.51	0.00
18	0.53	0.57	0.62	0.45	0.53	0.60	0.30	0.43	0.56	0.15	0.34	0.52	0.00	0.25	0.49	0.00
20	0.47	0.51	0.56	0.40	0.47	0.55	0.26	0.38	0.51	0.12	0.30	0.48	0.00	0.21	0.44	0.00
22	0.40	0.45	0.50	0.34	0.41	0.49	0.21	0.33	0.46	0.08	0.25	0.43	0.00	0.17	0.40	0.00
24	0.34	0.39	0.45	0.28	0.36	0.43	0.16	0.28	0.40	0.04	0.21	0.38	0.00	0.14	0.35	0.00
26	0.28	0.33	0.39	0.23	0.30	0.38	0.12	0.23	0.35	0.01	0.17	0.33	0.00	0.10	0.30	0.00
27	0.25	0.30	0.36	0.20	0.27	0.35	0.09	0.21	0.33	0.00	0.15	0.30	0.00	0.08	0.28	0.00

NOTES:

1 Linear interpolation may be used between the values given in the Table.

2 Shaded cells in the Table indicate that failure would be by crushing. For other values, failure would be by instability.

7.3.5 Concentrated loads

7.3.5.1 *General*

Each concentrated compression load acting on a masonry member shall be assumed to disperse through the masonry in accordance with Clause 7.3.5.2.

The member shall be designed to comply with Clause 7.3.5.3.

If a concentrated load is applied to an engaged pier, the pier shall be designed to carry the total load.

7.3.5.2 *Dispersion of a concentrated load through the masonry*

A concentrated load acting on a member shall be assumed to disperse through the masonry at an angle of 45° from the horizontal, from the perimeter of the bearing area of the load (see Figure 7.3) but this dispersion shall not extend—

(a) into the dispersion zone of an adjacent concentrated load on the member; or

(b) beyond the structural end of the masonry in the member as defined in Clause 4.5.3.

7.3.5.3 *Load capacity under a concentrated load*

Where a member is of uniform and symmetrical cross-section, it shall be designed as follows:

(a) The member shall be designed such that the following relationship is satisfied for the cross-section immediately below or above the concentrated load:

$$F_d \leq k_b F_o \qquad \text{. . . 7.3.5.3}$$

where

F_d = the design compressive force acting on the cross-section of a member simultaneously with a bending moment, shear force or other load action

F_o = the basic compressive strength capacity of the cross-section (see Clause 7.3.2)

k_b = the concentrated bearing factor, in accordance with Clause 7.3.5.4.

(b) The member shall be designed to comply with Clause 7.3.1 for every cross-section in the zone of dispersion subject to the following:

(i) The design compressive force (F_d) acting on any given cross-section shall include the design concentrated load, plus that portion of the other compressive forces acting on the cross-sectional area (A_{ds}) under consideration; and

(ii) The design bending moment (M_d) acting on the same cross-section under consideration shall include the bending moment, if any, from the design concentrated load, plus that portion of the bending moments from other loads and forces that act on that cross-sectional area (A_{ds}).

(c) The member shall be designed to comply with Clause 7.3.1 subjected to a load equivalent to that present in a zone of dispersion half the height from the point of application of the load. This load shall be considered to have the same eccentricity as the concentrated load.

7.3.5.4 *Concentrated bearing factor (k_b)*

The value of the concentrated bearing factor (k_b), for use in Clause 7.3.5.3, shall be as follows:

(a) Other than in solid or cored unit masonry or grouted masonry:

$k_b = 1.00$

(b) In solid or cored unit masonry, or in grouted masonry—

$$k_b = 0.55 \frac{\left(1 + 0.5 \frac{a_1}{L}\right)}{\left(\frac{A_{ds}}{A_{de}}\right)^{0.33}} \text{; or} \qquad \ldots 7.3.5.4(1)$$

$$k_b = 1.50 + \frac{a_1}{L} \qquad \ldots 7.3.5.4(2)$$

whichever is less, but not less than 1.0.

where

a_1 = the distance from the end of the wall or pier to the nearest end of the bearing area

A_{ds} = the bearing or dispersion area of a concentrated load in a member at the design cross-section under consideration (see Clauses 7.3.5.2 and 7.3.5.3)

A_{de} = the effective area of dispersion of the concentrated load on the member at mid-height

= $L_e t$

Where concentrated loads are applied in close proximity to each other, the value of A_{de} shall be calculated taking into account Clause 7.3.5.2(a)

L_e = the effective dispersion length of the load (see Figure 7.3)

t = the overall thickness of a masonry member cross-section perpendicular to the principal axis under consideration

L = the clear length of the wall or pier (see Figure 7.3)

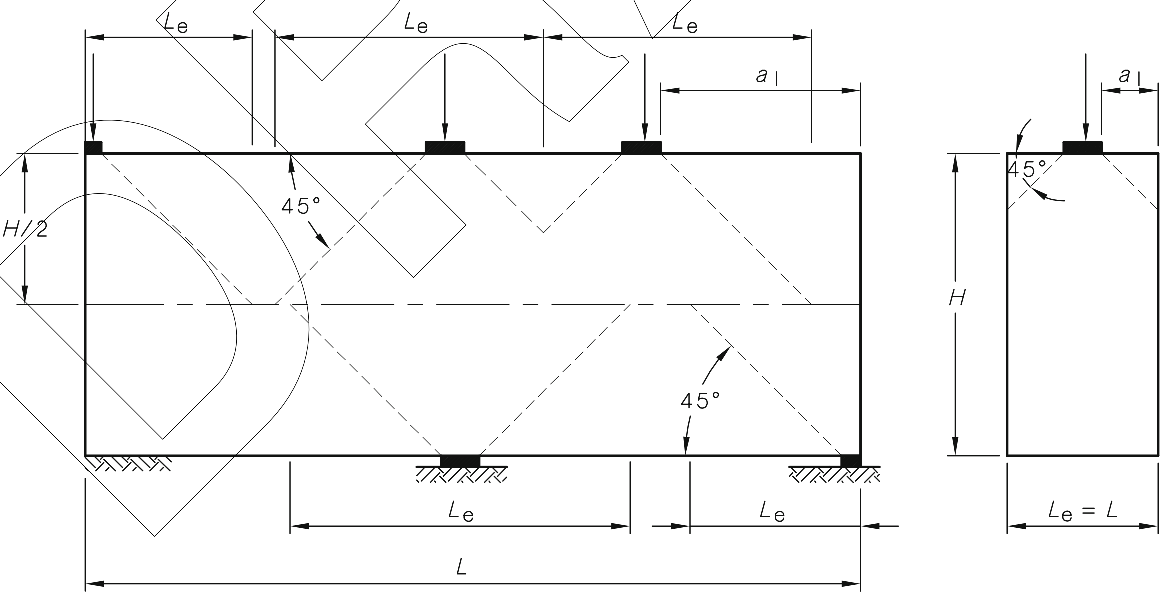


FIGURE 7.3 LOADING POSITIONS AND EFFECTIVE AREAS OF DISPERSION

7.4 DESIGN FOR MEMBERS IN BENDING

7.4.1 General

Unreinforced masonry members resisting bending moments shall be designed for vertical bending, horizontal bending or two-way bending in accordance with Clauses 7.4.2 to 7.4.4, whichever gives the greatest capacity.

7.4.2 Design for vertical bending

The design of an unreinforced masonry wall, or part thereof, to withstand vertical bending from actions of a short-term transient nature, which include out-of-plane wind loads, earthquake loads or similar forces, shall be such that the following relationship is satisfied under each combination of simultaneously acting design vertical bending moment (M_{dv}) and design compressive force acting on the cross-section of a member (F_d) under consideration:

$$M_{dv} \leq M_{cv} \qquad \ldots 7.4.2(1)$$

where

M_{dv} = the design vertical bending moment resulting from transient out-of-plane forces acting on the member in vertical-spanning action

M_{cv} = the vertical bending moment capacity of the member as follows:

(i) Where

$f'_{mt} > 0$

M_{cv} = the lesser of—

(A) $$M_{cv} = \phi f'_{mt} Z_d + f_d Z_d\text{; and} \qquad \ldots 7.4.2(2)$$

(B) $$M_{cv} = 3.0\, \phi f'_{mt} Z_d \qquad \ldots 7.4.2(3)$$

(ii) Where

$f'_{mt} = 0$

$$M_{cv} = f_d Z_d \qquad \ldots 7.4.2(4)$$

where f_d shall not be taken as greater than 0.36 MPa

and where

ϕ = the capacity reduction factor (see Clause 4.4)

f'_{mt} = the characteristic flexural tensile strength of the masonry (see Clause 3.3.3)

Z_d = the section modulus of the bedded area

f_d = the minimum design compressive stress on the bed joint at the cross-section under consideration (see Clause 7.4.3.3)

NOTE: The member should also comply with Clause 7.3 (see Clause 7.2).

7.4.3 Design for horizontal bending

7.4.3.1 *General*

An unreinforced masonry wall, or part thereof, shall be designed to withstand horizontal bending from actions of a short-term transient nature, which include out-of-plane wind loads, earthquake loads or similar forces in accordance with Clause 7.4.3.2, provided it satisfies the following:

(a) The wall has all perpends completely filled, except in the case of hollow unit masonry where only the width of the face shells need be filled.

(b) The wall, or that part being designed for horizontal bending, has at least four continuous courses of masonry acting together in horizontal bending.

Walls not satisfying the requirement of Items (a) and (b) above are deemed to be outside the scope of this Standard.

7.4.3.2 *Horizontal bending with tension stresses permitted*

A wall complying with the requirements of Clause 7.4.3.1 shall be proportioned so that the following relationship is satisfied under each combination of simultaneously acting design horizontal bending moment and design compressive stress acting on the bed joints:

$$M_{dh} \leq M_{ch} \qquad \text{. . . 7.4.3.2(1)}$$

where

M_{dh} = the design horizontal bending moment resulting from transient out-of-plane forces acting on a wall in horizontal-spanning action

M_{ch} = the horizontal bending moment capacity in N.mm per metre length of the wall as follows:

(a) For masonry constructed of other than AAC units, the least of—

(i) $$M_{ch} = 2.0\,\phi\,k_p\left(\sqrt{f'_{mt}}\right)\left(1+\frac{f_d}{f'_{mt}}\right)Z_d\,; \qquad \text{. . . 7.4.3.2(2)}$$

(ii) $$M_{ch} = 4.0\,\phi\,k_p\left(\sqrt{f'_{mt}}\right)Z_d\,;\ \text{or} \qquad \text{. . . 7.4.3.2(3)}$$

(iii) $$M_{ch} = \phi\left(0.44 f'_{ut} Z_u + 0.56 f'_{mt} Z_p\right) \qquad \text{. . . 7.4.3.2(4)}$$

(b) For AAC masonry with thin-bed mortar:

$$M_{ch} = \phi\left(0.22 f'_{ut} + 0.33 f'_{mt}\right)Z_d \qquad \text{. . . 7.4.3.2(5)}$$

where

ϕ = capacity reduction factor (see Clause 4.4)

k_p = a perpend spacing factor assessed in accordance with Clause 7.4.3.4

f'_{mt} = the characteristic flexural tensile strength, in megapascals (see Clause 3.3.3)

f_d = the minimum design compressive stress on the bed joint at the cross-section in a member under consideration, in megapascals (see Clause 7.4.3.3)

f'_{ut} = the characteristic lateral modulus of rupture of the masonry units, in megapascals (see Clause 3.2)

Z_d = the section modulus of the bedded area, in cubic millimetres per metre length

Z_u = the lateral section modulus of the masonry units, in cubic millimetres per metre length, based on—

(i) for solid or cored units, the gross cross-section; and

(ii) for hollow units, the face shell dimensions

Z_p = the lateral section modulus, in cubic millimetres per metre length, based on the mortar contact area of the perpend joints

7.4.3.3 *Compressive stress on bed joints*

The value of the minimum design compressive stress on the bed joints (f_d), for use in Clauses 7.4.2, 7.4.3 and 7.4.4 shall be the compressive stress at the bed joint under consideration, resulting from the (minimum) design compressive force that acts simultaneously with the loads and forces producing the bending in the masonry member, and based on the design cross-sectional area (A_d) of the bed joint.

Where the design for horizontal bending of a wall is based on an equivalent compressive stress applied throughout the whole height of the wall, the value of f_d to be used in Clause 7.4.3.2 shall be as follows:

(a) If the top of the wall is not laterally supported, the design compressive stress at the top of the wall.

(b) If the wall is laterally supported at its top edge, the design compressive stress at a distance equal to half the height of the wall below its top lateral support.

7.4.3.4 *Perpend spacing factor (k_p)*

The value of the perpend spacing factor, for use in Clause 7.4.3.2, shall be the lesser of—

(a) $$k_p = \frac{s_p}{t_u} \qquad \text{. . . 7.4.3.4(1)}$$

(b) $$k_p = \frac{s_p}{h_u} \qquad \text{. . . 7.4.3.4(2)}$$

(c) $k_p = 1$

where

s_p = the minimum overlap of masonry units in successive courses

t_u = the width of the masonry unit (measured through the wall)

h_u = the height of the masonry unit in the wall

NOTE: For normal stretcher bond, $k_p = 1$; for stack bond, $k_p = 0$.

7.4.4 Design for two-way bending

7.4.4.1 *General*

Unreinforced masonry wall, or part thereof, shall be designed to withstand two-way bending from actions of a short-term transient nature, which include out-of-plane wind loads, earthquake loads or similar forces in accordance with Clause 7.4.4.2 or Clause 7.4.4.4, provided it satisfies the following:

(a) The wall is designed as a rectangular panel with lateral support at least along its base and one of its vertical edges.

(b) The wall is of single leaf or solid construction, or of cavity wall construction with each leaf of the cavity wall being of single leaf or solid construction.

(c) The requirements for robustness in Clause 4.6 are satisfied.

(d) The construction and detailing of the wall are such that the requirements of Clauses 7.4.2 and 7.4.3 for development of bending strength capacity are satisfied within the four edges of the rectangular panel.

(e) If the wall has an opening, the panel on each side of the opening is checked independently.

For solid walls of multi-unit thickness that rely on a filled collar joint and metal ties to connect the leaves [see Clause 4.11.1(b)], in the absence of more precise calculations, the capacity shall be taken as the sum of the capacities of the separate leaves acting independently.

NOTE: For the distribution of load between the leaves of a cavity wall, see Clause 7.7.3.

7.4.4.2 *Lateral load capacity of masonry other than AAC*

For masonry other than AAC unit masonry, a wall complying with the requirements of Clause 7.4.4.1 shall be proportioned such that the following relationship is satisfied:

$$w_d \leq w \qquad \ldots 7.4.4.2(1)$$

where

$$w = \frac{2a_f}{L_d^{\,2}}\left(k_1 M_{ch} + k_2 M_{cd}\right) \qquad \ldots 7.4.4.2(2)$$

where

w = two-way bending capacity of the wall

w_d = the total design wind, earthquake or similar pressure acting on the wall

a_f = an aspect factor (see Table 7.5)

L_d = the design length—

(i) when only one vertical edge of the wall is laterally supported—the actual length of the wall to the unsupported end or to a control joint;

(ii) when both vertical edges are laterally supported—half the actual length of the wall between lateral supports; or

(iii) when an opening is present in the wall—the distance from the laterally supported end of the wall to the nearest edge of the opening

M_{ch} = the horizontal bending moment capacity of a wall as in Clause 7.4.3.2

M_{cd} = the diagonal bending moment capacity per unit length of diagonal crack line (see Clause 7.4.4.3)

k_1, k_2 = coefficients (see Table 7.5) where—

R_{f1} = the restraint factor for the first supported edge of the wall

= 0 if no rotational restraint

= 1 if fully restrained against rotation; or

= an intermediate value if there is partial rotational restraint

R_{f2} = the restraint factor for the second supported edge of the wall

= 0 if no rotational restraint

= 1 if fully restrained against rotation; or

= an intermediate value if there is partial rotational restraint

α = a slope factor

$= \dfrac{GL_d}{H_d}$

NOTE: When the factor $\alpha < 1$, the failure crack pattern for the wall will have a vertical crack; otherwise it will not.

L_o = the length of the opening, if any

H_d = the design height—

(a) when the top edge is not laterally supported—the actual height of the wall (H); or

(b) when the top edge is laterally supported—half the actual height of the wall ($H/2$)

G = the assumed slope of the crack line, taken as

$\dfrac{2(h_u + t_j)}{l_u + t_j}$ for half-overlap stretcher bonding

where

h_u = the height of masonry unit

t_j = the mortar joint thickness

l_u = the length of masonry unit

7.4.4.3 *Diagonal bending moment capacity*

The diagonal bending moment capacity per unit length of diagonal crack line (M_{cd}) for use in Clause 7.4.4.2 shall be calculated from the following equation:

$$M_{cd} = \phi f'_t Z_t \qquad \text{. . . 7.4.4.3(1)}$$

where

ϕ = the capacity reduction factor (see Clause 4.4)

f'_t = the equivalent characteristic torsional strength = $2.25\sqrt{f'_{mt}} + 0.15 f_d$

f_d = the minimum design compressive stress on the bed joints (see Clause 7.4.3.3)

NOTE: f'_t, f'_{mt} and f_d are in megapascals.

Z_t = the equivalent torsional section modulus measured normal to the diagonal crack line, calculated as follows:

(a) For a solid rectangular section $B \times t_u$, where $B \geq t_u$:

$$Z_t = \frac{\dfrac{2B^2 {t_u}^2}{3B + 1.8 t_u}}{\left[(l_u + t_j)\sqrt{1 + G^2}\right]} \qquad \text{. . . 7.4.4.3(2)}$$

(b) For a solid rectangular section $B \times t_u$, where $B < t_u$:

$$Z_t = \frac{\dfrac{2B^2 {t_u}^2}{3t_u + 1.8B}}{\left[(l_u + t_j)\sqrt{1 + G^2}\right]} \qquad \text{. . . 7.4.4.3(3)}$$

(c) For a hollow section:

$$Z_{\mathrm{t}} = 2Bt_{\mathrm{s}} \frac{\left(\dfrac{Bt_{\mathrm{s}}}{1.5B + 0.9t_{\mathrm{s}}} + t_{\mathrm{u}} - t_{\mathrm{s}}\right)}{\left[\left(l_{\mathrm{u}} + t_{\mathrm{j}}\right)\sqrt{1+G^2}\right]} \qquad \ldots 7.4.4.3(4)$$

where

B = a height factor = $\dfrac{h_{\mathrm{u}} + t_{\mathrm{j}}}{\sqrt{1+G^2}}$

t_{u} = the width of the masonry unit (measured through the wall)

l_{u} = the length of masonry unit

h_{u} = the height of masonry unit

t_{j} = the mortar joint thickness

t_{s} = the thickness of face shell of a hollow masonry unit at the thinnest point

G = the assumed slope of the crack line (see Clause 7.4.4.2)

TABLE 7.5

COEFFICIENTS FOR OUT-OF-PLANE LATERAL LOAD RESISTANCE

Opening	Number of vertical edges supported	Slope factor α	Aspect factor (a_{f})	k_1	k_2
No	Both	≤1	$\dfrac{1}{\left(1-\dfrac{\alpha}{3}\right)}$	$\dfrac{R_{\mathrm{f1}} + R_{\mathrm{f2}}}{2} + 1 - \alpha$	$\alpha\left(1+\dfrac{1}{G^2}\right)$
No	Both	>1	$\dfrac{\alpha}{\left(1-\dfrac{1}{3\alpha}\right)}$	$\dfrac{R_{\mathrm{f1}} + R_{\mathrm{f2}}}{2}$	$1+\dfrac{1}{G^2}$
No	One	≤1	$\dfrac{1}{\left(1-\dfrac{\alpha}{3}\right)}$	R_{f1}	$\alpha\left(1+\dfrac{1}{G^2}\right)$
No	One	>1	$\dfrac{\alpha}{\left(1-\dfrac{1}{3\alpha}\right)}$	R_{f1}	$1+\dfrac{1}{G^2}$
Yes	Both	≤1	$\dfrac{1}{\left(1-\dfrac{\alpha}{3}\right)+\dfrac{L_{\mathrm{o}}}{L_{\mathrm{d}}}\left(1-\dfrac{\alpha}{2}\right)}$	R_{f1}	$\alpha\left(1+\dfrac{1}{G^2}\right)$
Yes	Both	>1	$\dfrac{\alpha}{\left(1-\dfrac{1}{3\alpha}\right)+\dfrac{L_{\mathrm{o}}}{2L_{\mathrm{d}}}}$	R_{f1}	$1+\dfrac{1}{G^2}$

7.4.4.4 *Lateral load capacity of AAC masonry*

AAC unit masonry laid in thin-bed mortar, having panel proportions L:H not exceeding 2.5:1 and are supported on at least three edges and do not contain openings, shall be designed such that the following relationship is satisfied:

$$w_{\mathrm{d}} \leq 12\frac{H}{L}\left(b_{\mathrm{v}}M_{\mathrm{cv}}/H^{2}+b_{\mathrm{h}}M_{\mathrm{ch}}/L^{2}\right) \qquad \ldots 7.4.4.4$$

where

w_{d} = the total design wind, earthquake or similar pressure acting on the wall

H = the clear height of a member between horizontal lateral supports; or

= for a member without top horizontal support, the overall height from the bottom lateral support

L = the clear length of a wall between vertical lateral supports; or

= for a wall without a vertical support at one end or at a control joint, the length to that unsupported end or control joint

M_{cv} = the vertical bending moment capacity per unit length, in accordance with Clause 7.4.2

M_{ch} = the horizontal bending moment capacity per unit height, in accordance with Clause 7.4.3.2

b_{v} = the vertical bending coefficient (see Table 7.6)

b_{h} = the horizontal bending coefficient (see Table 7.6)

The bending coefficients (b_{v}) and (b_{h}) for use in this Clause shall be as derived from Table 7.6, taking into account the nature of the support along each of the four edges of the wall.

Walls that are not covered by Table 7.6, or for which the value of b_{v} or b_{h} is less than zero, shall be regarded as being outside the scope of Clause 7.4.4 and shall be designed to comply with other relevant clauses of the Standard.

TABLE 7.6

BENDING COEFFICIENTS FOR WALLS OF AAC MASONRY IN TWO WAY BENDING

Edge restraint to wall panels	Vertical bending coefficient (b_v)	Horizontal bending coefficient (b_h)
1 S or R S S S or R	1.00	1.00
2 S or R S R S or R	1.00	1.25
3 S or R R R S or R	1.00	1.5
4 Free S S S or R	0.25	1.00
5 Free S R S or R	0.25	1.25
6 Free R R S or R	0.25	1.50

LEGEND:

H = the height of the wall panel as defined in Clause 1.6.

L = the length of the wall panel as defined in Clause 1.6.

S = the edge of the wall that is simply supported, that is laterally supported but not rotationally fixed.

R = the edge of the wall that is rotationally restrained as well as laterally supported (note that structural continuity of wall construction at an edge does not necessarily provide rotational restraint at that edge).

Free = the edge of the wall that has neither lateral support nor rotational restraint.

NOTE: A control joint in a wall shall be deemed to be a free edge.

7.5 DESIGN FOR MEMBERS IN SHEAR

7.5.1 Shear walls

Shear walls shall be designed to comply with Clauses 7.2 to 7.4 and in accordance with the provisions of Clauses 7.5.2 to 7.5.4.

7.5.2 Two or more shear walls acting together

Where wind loads and other lateral forces are resisted by two or more shear walls acting together, the load and force actions shall be distributed between the shear walls using recognized principles, taking into account the relative stiffness of the walls under these actions, and the effects of openings, if any, in the walls.

7.5.3 Design for compression and in-plane lateral forces

The strength capacity and stability of a shear wall under compression and flexure in the direction of the length of the wall shall be based on the properties of the whole monolithic cross-section of the wall, with the effective widths of flanges (if any) calculated in accordance with Clause 4.5.2.

Flanges shall be designed in accordance with the requirements of Clauses 7.2 to 7.4 and Clause 7.5.4 to carry their portion of the load action, and shall be bonded or tied to the remainder of the wall in accordance with the requirements of Clause 4.11.

7.5.4 Shear capacity

7.5.4.1 *Horizontal planes*

The shear resistance on a horizontal plane can result from both shear bond (or resistance to rupture) and shear friction. At the base of arches, the shear bond shall be taken as zero or a value substantiated by tests.

The design of an unreinforced masonry member to withstand horizontal shear forces, with or without simultaneous compressive forces acting across the shear plane, shall be such that the following relationship is satisfied under each combination of simultaneously acting design shear force (V_d) and design compressive stress acting at the cross-section under consideration (f_d):

$$V_d \leq V_o + V_1 \qquad \ldots 7.5.4.1$$

where

V_o = the shear bond strength of the section

= $\phi f'_{ms} A_d$

ϕ = the capacity reduction factor (see Clause 4.4)

f'_{ms} = the characteristic shear strength (see Clause 3.3.4)

A_d = the design cross-sectional area of the shear-resisting portion of the member (see Clause 4.5.6)

V_1 = the shear friction strength of the section

= $k_v\ f_d\ A_d$

k_v = the shear factor as given in Table 3.3

f_d = the minimum design compressive stress on the bed joint at the cross-section in a member (see Clause 7.5.5) but not greater than 2 MPa

7.5.4.2 *Vertical planes*

The design of an unreinforced masonry member to withstand vertical shear forces shall be such that the following relationship is satisfied.

$$V_d \leq \phi f'_{ms}\ A_d$$

where

ϕ = the capacity reduction factor (see Clause 4.4)

f'_{ms} = the characteristic shear strength (see Clause 3.3.4)

A_d = the design cross-sectional area (see Clause 4.5.6).

Where it is necessary to transfer longitudinal shear forces across vertical mortar joints in the masonry, the bonding or tying across the shear plane shall comply with Clause 4.11 and be such that it will develop the characteristic shear strength (f'_{ms}) required in the design in accordance with Clause 3.3.4(b).

7.5.5 Compressive stress on bed joints

7.5.5.1 *Loading from other than earthquake action*

For other than earthquake-induced shear, the value of the design compressive stress on the bed joints (f_d), for use in Clause 7.5.4.1, shall be the compressive stress at the bed joint under consideration, resulting from the (minimum) design compressive force that acts simultaneously with the loads and forces producing shear in the masonry member, and based on the design cross-sectional area (A_d) of the bed joint.

7.5.5.2 *Earthquake loading*

For earthquake-induced shear, the value of the design compressive stress on the bed joints (f_d), for use in Clause 7.5.4.1, shall be the compressive stress at the bed joint under consideration, resulting from 0.9 times the gravity load (G_g), and based on the design cross-sectional area (A_d) of the bed joint.

Where the vertical gravity load acting on the masonry member contributes to the resistance but not to the induced lateral earthquake load on the masonry, the live load shall be ignored and the gravity load shall be $0.8G_g$. For other cases, the provisions in AS/NZS 1170.0 shall apply.

NOTE: The above provision allows for the fact that the earthquake-induced shear forces and the shear resistance will both be a function of the gravity load (G_g). Possible vertical acceleration effects are also taken into account.

7.5.6 Shear connectors

The design shear strength of connectors (V_c) shall be calculated as follows:

(a) *Shear connectors across mortar joints* Where shear force is to be transferred across a mortar joint by steel connectors, the connectors shall be embedded at least 50 mm into the masonry at each side of the interface. The design shear strength (V_c) of the connector shall be taken as follows:

(i) For connectors of rectangular cross-section:

$$V_c = \phi \frac{ruf_{yc}}{12} \qquad \ldots 7.5.6(1)$$

(ii) For connectors of circular cross-section:

$$V_c = \phi \frac{(d_{sc})^2 f_{yc}}{18} \qquad \ldots 7.5.6(2)$$

where

ϕ = the capacity reduction factor (see Clause 4.4)

r = the width of the connector

u = the thickness of the connector

f_{yc} = the characteristic tensile yield strength of the connector

d_{sc} = the diameter of the shear connector

(b) *Shear connectors connecting masonry to other structural members* The design shear strength (V_c) of a connector shall be such that—

$$V_c = \phi V_{sc} \qquad \ldots 7.5.6(3)$$

where

ϕ = the capacity reduction factor (see Clause 4.4)

V_{sc} = the characteristic shear strength of the connector (determined by tests)

7.6 DESIGN OF MASONRY VENEER WALLS

7.6.1 General

Masonry veneer walls shall be designed to comply with Clauses 7.2 to 7.5 and in accordance with the provisions of Clause 7.6.2 or 7.6.3.

NOTES:

1 Typical masonry veneer construction on a timber or steel frame is masonry veneer with a flexible structural backing.

2 Masonry veneer attached to a rigid structure is masonry veneer with a stiff structural backing. This may also apply to a cavity wall with only the inner masonry leaf supported (see Clause 7.7.4.)

The wall ties for a masonry veneer wall shall be designed to transfer all out-of-plane lateral loads and forces applied to the face of the masonry veneer to the structural backing that supports that wall.

The wall ties connecting the veneer and its backing shall comply with the provisions of Clause 4.10.

7.6.2 Wall ties with flexible structural backing

For veneer with flexible structural backing, both the ultimate strength and serviceability limit states shall be considered as follows:

(a) *Ultimate strength limit state* For the ultimate strength limit state, each tie shall be designed so that the following relationship is satisfied:

$$F_{td} \leq \phi F_t \qquad \ldots 7.6.2$$

where

F_{td} = the design compressive or tensile tie force

ϕ = the capacity reduction factor (see Clause 4.4)

F_t = the strength of the tie based on its duty rating (see Table 3.5)

In the absence of an analysis considering the stiffness of the ties, the structural backing and the veneer before and after cracking, the design compressive or tensile tie force (F_{td}) shall be taken as 20% of the total tributary lateral load on a vertical line of ties between horizontal supports.

The ties shall be designed to resist $2 \times F_{td}$ in the following locations:

(i) The row at the top of a single storey veneer.

(ii) Where a veneer is continuous past a horizontal floor support, at the first row above and the first row below the floor.

(iii) On lines opposite vertical lateral supports.

NOTES:

1 The requirement for higher resistance may be achieved by using ties of a higher duty or by doubling the number of ties in these locations.

2 Vertical lateral supports are required to be in accordance with Clause 2.7.

(b) *Serviceability limit state* For the serviceability limit state, the deflection of the structural backing under the design serviceability wind load shall be not greater than its span divided by 300.

Earthquake loads are not required to be considered for the serviceability limit state.

7.6.3 Wall ties with stiff structural backing

For veneer with stiff structural backing, each tie shall be designed such that the following relationship is satisfied for the ultimate strength limit state:

$$F_{td} \leq \phi F_t \qquad \ldots 7.6.3$$

where

F_{td} = the design compressive or tensile tie force

ϕ = the capacity reduction factor (see Clause 4.4)

F_t = the strength of the tie based on its duty rating (see Table 3.5)

In the absence of a more exact analysis, the design compressive or tensile tie force (F_{td}) shall be taken as 1.3 times the total tributary design lateral load acting on the tie.

7.7 DESIGN OF CAVITY WALLS

7.7.1 General

Each leaf of a cavity wall shall be designed as an independent member to transmit its share of vertical and lateral loads in accordance with Clauses 7.2 to 7.5, and in accordance with the provisions of Clauses 7.7.2 to 7.7.4.

7.7.2 Compressive load capacity with both leaves loaded

Where both leaves of a cavity wall support the compressive force, then either—

(a) the stronger leaf shall be designed to withstand the whole of the compressive force; or

(b) each leaf shall be designed as an independent compression member to carry its share of the total compressive force, taking into account all factors likely to affect the distribution of load between the leaves, including any different material capacities and properties in the leaves, differential movements and rotational effects from floors and beams.

7.7.3 Lateral bending capacity

The total lateral loads and forces acting on a cavity wall shall be either—

(a) assessed as being shared between the two leaves of the wall in a manner that takes into account the flexural rigidities of the two leaves, the stiffness of the wall ties connecting the two leaves, and the amount of the total load that is applied directly to the face of each leaf; or

(b) assumed to be resisted wholly by one leaf of the wall.

If the two leaves of a cavity wall are tied together with wall ties of medium or heavy duty rating, designed in accordance with Clause 4.10, the requirements of this Clause for the sharing of load between the leaves shall be considered to have been met if the wall as a whole is designed using the following equation:

$$w_d = K_{wt}\left[w_1 + \left(\frac{t_2}{t_1}\right)w_2\right] \qquad \ldots 7.7.3$$

where

K_{wt} = wall tie stiffness coefficient

= 0.9 for wall ties of medium duty rating

= 1.0 for wall ties of heavy duty rating

w_1 = the load capacity of the thicker leaf, determined in accordance with Equation 7.4.4.2(2)

w_2 = the load capacity of the thinner leaf, determined in accordance with Equation 7.4.4.2(2)

t_1 = the thickness of the thicker leaf

t_2 = the thickness of the thinner leaf

In Equation 7.7.3, for the determination of w_1 and w_2, it shall be permissible to assume that both leaves are supported in the same manner as the leaf with the greater support.

7.7.4 Wall ties

Each tie shall be designed such that the following relationship is satisfied:

$$F_{td} \leq \phi F_t \qquad \ldots 7.7.4$$

where

F_{td} = the design compressive or tensile force tie force

ϕ = the capacity reduction factor (see Clause 4.4)

F_t = the strength of the tie based on its duty rating (see Table 3.6).

In the absence of a more exact analysis, the design compressive or tensile tie force (F_{td}) shall be taken as the design load acting on the tributary area of the tie, except that a cavity wall with only the inner masonry leaf supported shall be considered as a veneer wall with stiff structural backing (see Clause 7.6.3).

The ties shall be designed to resist $2 \times F_{td}$ on each line of ties opposite a vertical lateral support.

NOTES:

1 The requirement for higher resistance may be achieved by using ties of a higher duty or by doubling the number of ties in these locations.

2 Vertical lateral supports are required to be in accordance with Clause 2.7.

7.8 DESIGN OF DIAPHRAGM WALLS

7.8.1 General

Diaphragm walls shall be designed to comply with Clauses 7.2 to 7.5, and in accordance with the provisions of Clauses 7.8.2 and 7.8.3.

7.8.2 Lateral bending capacity

A diaphragm wall shall be designed to resist lateral loads and forces by vertical spanning action of the wall, with its strength capacity being based on the section modulus of the overall cross-section of the wall, using an effective flange width determined in accordance with Clause 4.5.2.

Each leaf shall also be designed to withstand, by horizontal spanning action between the diaphragms, the lateral loads and forces that are applied directly to that leaf, without any sharing of loads between the two leaves.

7.8.3 Diaphragms

Diaphragms in diaphragm walls shall be designed to withstand their portion of the load action, including the whole of the lateral shear forces acting on the wall.

SECTION 8 STRUCTURAL DESIGN OF REINFORCED MASONRY

8.1 SCOPE OF SECTION

This Section sets out specific requirements for the structural design of masonry with steel reinforcement to enhance its resistance to applied forces for the strength and serviceability limit states.

NOTE: The requirements of this Section are in addition to the general requirements of Section 4 and the durability requirements of Section 5.

8.2 EXCLUSIONS

Masonry that includes reinforcement not complying with the requirements of this Section shall be treated as unreinforced masonry and designed in accordance with Section 7, and the reinforcement shall be in accordance with the durability requirements of Clause 5.9.

Masonry that includes pre-stressed tendons shall be designed in accordance with Section 9.

8.3 GENERAL BASIS OF DESIGN

The structural design of reinforced masonry shall be in accordance with general principles used for reinforced concrete design, modified to suit the particular characteristics of reinforced masonry.

Calculations for strength of cross-sections in bending, or in bending combined with axial force, shall incorporate equilibrium and strain-compatibility considerations and shall be consistent with the following:

(a) Plane sections remain plane when considering the strain distribution in masonry in compression, or reinforcement in tension or compression.

(b) The strain in the outermost compressive fibre does not exceed 0.0035.

(c) The depth to the neutral axis used in design for bending, from the extreme compressive fibre to the neutral axis ($k_u d$) shall not exceed $0.4d$. The effective depth (d) is from the extreme compressive fibre of the masonry to the resultant tensile force in the steel in the tensile zone.

In masonry with raked joints, the extreme compressive fibre is taken to be at the depth of the raking. In masonry with ironed joints, the extreme compressive fibre is taken to be at the face of the masonry.

(d) A uniform compressive stress of (0.85) (1.3) f'_m acts on the area bounded by—

(i) the compression face of the cross-section; and

(ii) a line parallel to the neutral axis (at the strength limit state for the loading concerned) and located at a distance $0.85k_u d$ from the extreme compressive fibre.

(e) The tensile strength of masonry is zero.

(f) Stresses in steel reinforcement are derived from the stress-strain curve for the type of steel.

8.4 GENERAL REINFORCEMENT REQUIREMENT

8.4.1 General

Reinforcement shall be designed as—

(a) main reinforcement, which is designed to resist axial compression, bending, shear, tension or a combination of these; or

(b) secondary reinforcement, which is designed to resist shrinkage, temperature and the effects of localized loading.

8.4.2 Main reinforcement

The quantity and disposition of main reinforcement shall satisfy the specific requirements of Clauses 8.5 to 8.9.

8.4.3 Secondary reinforcement

Where a wall is designed to resist non-uniform loads or pressures, temperature, or shrinkage, secondary reinforcement shall be provided.

The area of secondary reinforcement shall be not less than $0.00035A_d$.

NOTE: This Clause does not apply to mixed construction.

8.4.4 Reinforcement detailing, cover and protection

8.4.4.1 *General*

The detailing of reinforcement in reinforced masonry shall be in accordance with AS 3600, except as specifically modified by this Clause (8.4.4) or Clause 5.9.

Reinforcement placed in mortar joints in the masonry shall comply with Clause 5.9.3. All other reinforcement shall be surrounded by grout, with cover as given in Clause 5.9.2.

8.4.4.2 *Minimum bar spacing*

The minimum clear distance between parallel bars shall be such that the grout can be properly placed and compacted. Bars with a clear spacing of 20 mm are deemed to comply with this requirement.

8.4.5 Close-spaced reinforcement for increased ductility in earthquakes

For a wall to be classed as one with close-spaced reinforcement it shall be fully grouted and contain reinforcement at a maximum spacing of 800 mm both horizontally and vertically. The minimum area of horizontal and vertical reinforcement shall be $0.0013A_d$.

8.4.6 Wide-spaced reinforcement

For a wall to be classed as one with wide-spaced reinforcement, the reinforcement not complying with Clause 8.4.5 shall nevertheless comply with Clauses 8.5 to 8.8.

8.5 DESIGN OF MEMBERS IN COMPRESSION

Members subjected to compressive forces shall be designed such that the following relationship is satisfied:

$$F_d \le \varphi\, k_{es}\left[f'_m A_b + k_c\left(\left(\frac{f'_{cg}}{1.3}\right)^{\left(0.55+0.005 f'_{cg}\right)}\right)A_g + \alpha_r f_{sy}\, A_s\right] \qquad \ldots 8.5$$

where

F_d = the design compressive force acting on the cross-section

ϕ = the capacity reduction factor (see Clause 4.4)

k_{es} = a reduction factor, taken as

$(1.0 - 0.025S_r)(1.0 - 2.0\, e/t)$

S_r = the slenderness ratio (see Clause 7.3.4.3)

e = the effective eccentricity (see Clause 7.3.4.4)

f'_m = the characteristic compressive strength of the masonry (see Clause 3.3.2)

A_b = the bedded area of the masonry cross-section (see Clause 4.5.4)

k_c = a strength factor for grout in compression

= 1.4 for hollow concrete masonry units of density greater than 2000 kg/m^3

= 1.2 for all other masonry

f'_{cg} = the design characteristic compressive strength of the grout

the value of f'_{cg} shall be greater than or equal to 12 MPa but not less than f'_m $1.3f'_{uc}$

.

A_g = the design cross-sectional area of grout in the reinforced masonry member (see Clause 4.5.7)

f_{sy} = the design yield strength of reinforcement (see Clause 3.6.1)

A_s = the total cross-sectional area of main reinforcement

α_r = the reinforcing contribution factor—

1.0 for piers

0.40 for walls

The main reinforcement in the direction of the axial load shall—

(a) be located symmetrically in the cross-section;

(b) posses sufficient development length in accordance with AS 3600;

(c) not be spliced; where lap splice is required, minimum lap displace of 300 mm shall apply,

(d) be (for piers—isolated or engaged) laterally restrained in both horizontal directions by ties of not less than 6 mm diameter round bars, which shall be spaced at centres not exceeding the least cross-section of the member or 400 mm, whichever is the lesser;

(e) be (for walls) shall be surrounded by an annulus of grout in accordance with Clause 3.5 of thickness not less than twice the radius of the reinforcing bar;

(f) have an area, $A_s \geq 0.002A_d$.

Reinforcement quantities in excess of $A_s = 0.04A_d$ shall be used only if the required minimum bar spacing and grout cover can be achieved, and the grout can be properly placed and compacted around the reinforcement.

If the reinforcement does not comply with these requirements, A_s shall be taken as zero and the member designed as unreinforced masonry in accordance with Section 7.

8.6 DESIGN OF MEMBERS IN BENDING

Reinforced masonry members subjected to bending, and complying with the following equation, are deemed to satisfy the requirements of Clause 8.3:

$$M_{\mathrm{d}} \leq \phi\, f_{\mathrm{sy}} A_{\mathrm{sd}} d \left[1 - \frac{0.6 f_{\mathrm{sy}} A_{\mathrm{sd}}}{(1.3 f'_{\mathrm{m}}) bd}\right] \qquad \ldots 8.6$$

where

M_{d} = the design bending moment acting on the cross-section of the member

ϕ = the capacity reduction factor (see Clause 4.4)

f_{sy} = the design yield strength of reinforcement (see Clause 3.6.1)

A_{sd} = the portion of the cross-sectional area of the main tensile reinforcement used for design purposes in a reinforced masonry member

= the lesser of $\dfrac{(0.29)1.3 f'_{\mathrm{m}} bd}{f_{\mathrm{sy}}}$ and A_{st}

d = the effective depth of the reinforced masonry member

f'_{m} = the characteristic compressive strength of the masonry (see Clause 3.3.2)

b = the width of a masonry member of solid rectangular cross-section; or

= the effective width of a member in accordance with Clause 4.5.2

A_{st} = the cross-sectional area of fully anchored longitudinal reinforcement in the tension zone of the cross-section under consideration

The main reinforcement in the direction of bending shall—

(a) be spaced at centres not exceeding 2000 mm;

(b) include an area of at least 100 mm^2 within 300 mm of the edges of the member; and

(c) be such that $M_{\mathrm{d}} \geq 1.2$ bending capacity of the unreinforced masonry.

For stack bonded solid and cored unit masonry designed in accordance with Clause 4.12.1, the requirements of Clause 4.12.1 for reinforcement spacing and distribution shall take precedence over the requirements of Clause 8.6(a) and 8.6(b) above

In members designed for bending in two directions, the requirements for main tensile reinforcement shall be met for the reinforcement in each direction.

The width of the compression face of reinforced masonry in bending (b) shall be assumed to extend beyond the line of the tensile reinforcement by a distance not exceeding the least of the following:

(i) For vertical reinforcement—

(A) 400 mm;

(B) twice the thickness of the wall; and

(C) the distance to the structural end of the masonry in the element.

(ii) For horizontal reinforcement—

(A) one and one-half times the thickness of the wall; or

(B) the distance to the horizontal edge of the masonry.

8.7 DESIGN OF WALLS FOR IN-PLANE SHEAR

8.7.1 General

For walls subjected to in-plane shear forces, the member shall be designed in accordance with Clauses 8.7.2 or 8.7.3.

In addition, the member shall be checked for bending in accordance with Clause 8.6 and for stability in accordance with Clause 8.7.4.

8.7.2 Long walls

A reinforced wall with $H/L \leq 2.3$ subjected to in-plane shear shall be designed such that the following relationship is satisfied:

$$V_d \leq \phi (f_{vr} A_d + 0.8 f_{sy} A_s) \qquad \text{. . . 8.7.2}$$

where

V_d = the design shear force acting on the cross-section of the masonry wall

ϕ = the capacity reduction factor (see Clause 4.4)

f_{vr} = effective shear strength, in megapascals

= $(1.50 - 0.5\ H/L)$

A_d = the design cross-sectional area of member (see Clause 4.5.6)

f_{sy} = the design yield strength of reinforcement (see Clause 3.6.1)

A_s = the cross-sectional area of reinforcement, as follows:

(a) If $H/L > 1.0$

$A_s = A_{sh}\ L/H$

where

A_{sh} = total area of anchored horizontal reinforcement

(b) If $H/L \leq 1.0$

A_s = the total cross-sectional area of horizontal reinforcement, or total cross-sectional area of vertical reinforcement, whichever is less

The reinforcement shall comply with the following:

(i) The reinforcement shall be located symmetrically in the cross-section.

(ii) Vertical reinforcement shall be spaced at centres not exceeding $0.75H$ and in any case not greater than 2000 mm horizontally. Horizontal reinforcement shall be spaced at centres not exceeding $0.75L$ and in any case not greater than 3000 mm vertically.

(iii) The vertical reinforcement shall be such that $A_s \geq 0.0013A_d$ and the horizontal reinforcement is such that $A_s \geq 0.0007A_d$. If the reinforcement does not meet these requirements then the wall shall be designed in accordance with Clause 7.5.3.

(iv) Reinforcement with an area of at least 100 mm^2 shall be included within 300 mm of edges parallel to the main reinforcement. It shall be permissible to omit the reinforcement at an edge of the wall, provided the member is anchored to an abutting reinforced concrete member.

Anchorage by the provision of starter bars of an area equal to or exceeding the area of the main reinforcement, lapped with the main reinforcement and anchored to the concrete member in accordance with the requirements of AS 3600, shall be deemed to satisfy this requirement.

8.7.3 Short walls

A reinforced wall with $H/L > 2.3$ subjected to in-plane shear shall be designed in accordance with Clause 8.8.

8.7.4 Stability

Where a reinforced wall with $H/L \leq 2.3$ is not externally supported against overturning, it shall be anchored to provide resistance to overturning and shear forces such that the following relationship is satisfied:

$$V_{\mathrm{d}} \leq \phi \left[k_{\mathrm{sw}} P_{\mathrm{v}} L/2 + f_{\mathrm{sy}} A_{\mathrm{sv}} \left(L - 2l'\right)\right] / H \qquad \ldots 8.7.4$$

where

V_{d} = the design shear force acting on the cross-section of the masonry wall

ϕ = the capacity reduction factor (see Clause 4.4)

k_{sw} = a reduction factor used in assessing the design overturning capacity of a wall

$= \left(1 - \dfrac{P_{\mathrm{v}}}{A_d f'_{\mathrm{m}}}\right)$

P_{v} = the applied uniform vertical load

f'_{m} = the characteristic compressive strength of the masonry (see Clause 3.3.2)

A_{d} = the design cross-sectional area of the masonry (see Clause 4.5.6)

A_{sv} = the cross-sectional area of shear reinforcement under consideration perpendicular to the direction of the applied shear and anchored in accordance with Clause 8.7.2

f_{sy} = the design yield strength of the reinforcement (see Clause 3.6.1)

L = the length of the member in the direction of the shear load

l' = the distance from the centroid of the reinforcement under consideration to the tensile end of the member

H = the height of the member.

8.8 DESIGN OF WALLS FOR OUT-OF-PLANE SHEAR

A reinforced wall subjected to out-of-plane shear shall be such that the following relationship is satisfied:

$$V_{\mathrm{d}} \leq \phi\left(f'_{\mathrm{vm}} b_{\mathrm{w}} d + f_{\mathrm{vs}} A_{\mathrm{st}} + f_{\mathrm{sy}} \frac{A_{\mathrm{sv}} d}{s}\right) \qquad \ldots 8.8$$

but not more than $4\ \phi\ f'_{\mathrm{vm}}\ b_{\mathrm{w}}\ d$

where

V_{d} = the design shear force acting on the cross-section of the masonry wall

ϕ = the capacity reduction factor (see Clause 4.4)

f'_{vm} = the characteristic shear strength of reinforced masonry

= 0.35 MPa

b_{w} = the width of the web of the shear-resisting area of the masonry wall (for a solid rectangular cross-section, $b_{\mathrm{w}} = b$)

d = the effective depth of the reinforced masonry wall

f_{vs} = the design shear strength of the main reinforcement

= 17.5 MPa

A_{st} = the cross-sectional area of fully anchored longitudinal reinforcement in the tension zone of the cross-section under consideration, or $0.02 b_w d$, whichever is less

f_{sy} = the design yield strength of the reinforcement (see Clause 3.6.1)

A_{sv} = the cross-sectional area of the shear reinforcement

s = the spacing of the shear reinforcement along the member

8.9 DESIGN OF BEAMS IN SHEAR

A reinforced beam subjected to shear shall be designed in accordance with Clause 8.8.

Where shear reinforcement is required, it shall be—

(a) spaced at centres (in the direction of the span) not exceeding $0.75D$ or 600 mm, whichever is less; and

(b) in accordance with the requirements for shear reinforcement in AS 3600.

8.10 DESIGN OF MEMBERS IN TENSION

A reinforced member subjected to axial tension shall be such that the following relationship is satisfied.

$$F_{dt} \leq \phi f_{sy} A_s \qquad \ldots 8.10$$

where

F_{dt} = the design tension force acting on the cross-section of the member

ϕ = the capacity reduction factor (see Clause 4.4)

f_{sy} = the design yield strength of the reinforcement (see Clause 3.6.1)

A_s = the total cross-sectional area of the main reinforcement.

The main reinforcement in the direction of axial load shall—

(a) be located symmetrically in the cross-section;

(b) include an area of at least 100 mm^2 within 300 mm of the edges of the member; and

(c) be spaced at centres not exceeding 2000 mm.

8.11 DESIGN FOR COMBINED LOADING

8.11.1 Members in combined bending and compression

A reinforced member subjected to the simultaneous action of compression forces and bending moments shall be designed such that the compressive load capacity is given by Clause 8.5 and the bending capacity is in accordance with Clause 8.6.

8.11.2 Members in combined bending and tension

A reinforced member subjected to simultaneous axial tension and bending shall be designed such that the tensile load capacity is in accordance with Clause 8.10 and the bending capacity is in accordance with Clause 8.6.

SECTION 9 STRUCTURAL DESIGN OF PRESTRESSED MASONRY

9.1 SCOPE OF SECTION

This Section sets out specific requirements for structural design of masonry reinforced with prestressed steel tendons to enhance its resistance to applied forces for the strength and serviceability limit states.

NOTE: The requirements of this Section are in addition to the general requirements of Section 4 and the durability requirements of Section 5.

Prestress may be applied in either of two forms, as follows:

(a) Post-tensioning, where the tendons are tensioned against the masonry.

(b) Pre-tensioning, where the tendons are tensioned between independent anchorages and released when the masonry has achieved sufficient strength.

9.2 GENERAL BASIS OF DESIGN

9.2.1 General

The structural design of prestressed masonry shall be in accordance with the general principles used for prestressed concrete design, modified to suit the particular characteristics of prestressed masonry.

The masonry shall be designed to satisfy both the strength and serviceability limit states using the appropriate loading combinations and capacity reduction factors. The level of cracking for the serviceability limit state is controlled by the level of prestress, with the section fully prestressed (uncracked) or partially prestressed (cracked at some lower level of load). In assessing the level of prestress, due allowance shall be made for losses calculated in accordance with Clause 9.3.1.2.

Calculations for the capacities of cross-sections in bending, or in bending combined with axial force, shall incorporate equilibrium and strain compatibility considerations and shall be consistent with the following assumptions:

(a) Plane sections remain plane when considering the strain distribution in the masonry in compression, or bonded tendons or other reinforcement in tension or compression.

(b) The tensile strength of the masonry is zero.

In the absence of a more exact calculation, the dispersion angle of the prestressing force from the anchorage shall be assumed to be 60°, that is 30° each side of the centre-line.

9.2.2 Additional requirements for strength

Additional requirements for the strength limit state are as follows:

(a) Stresses in tendons shall—

  (i) be derived from an appropriate stress-strain curve; or

  (ii) take into account the strain history of the tendon.

(b) The depth to the neutral axis used in design for bending, from the extreme compressive fibre to the neutral axis ($k_u d_p$), shall not exceed $0.4d_p$. The effective depth (d_p) is from the extreme compressive fibre of the masonry to the resultant tensile force in the steel in the tensile zone.

In masonry with raked joints, the extreme compressive fibre shall be taken to be at the depth of the raking. In masonry with ironed joints, the extreme compressive fibre shall be taken at the face of the masonry.

The effective depth (d_p) to unbonded tendons shall be determined by taking full account of the freedom of the tendons to move in the depth of the cross-section.

NOTE: Unbonded tendons may be restrained from moving within the voids of the section by the use of spacers or other forms of restraint.

(c) A uniform compressive stress of (0.85) (1.3) f'_m shall be assumed to act on the area bounded by—

(i) the compression face of the cross-section; and

(ii) a line parallel to the neutral axis and located at a distance of $0.85k_u d_p$ from the extreme compression fibre.

9.2.3 Additional requirements for serviceability

For the serviceability limit state, materials shall be assumed to be elastic with cross-sectional properties as defined in Clauses 3.3 to 3.7.

9.3 DESIGN CRITERIA FOR PRESTRESSING TENDONS

9.3.1 General

9.3.1.1 *Maximum initial prestress*

The jacking force shall not exceed 70% of the characteristic breaking load of the tendon.

9.3.1.2 *Loss of prestress*

When calculating the forces in the tendons at the various stages considered in the design, allowance shall be made for immediate and deferred losses of prestress, calculated in accordance with Clauses 9.3.2 and 9.3.3 respectively.

9.3.2 Immediate loss of prestress

9.3.2.1 *General*

The immediate loss of prestress shall be estimated by adding the calculated losses of prestress due to elastic deformation of masonry, friction, anchorage and other immediate losses, as applicable.

9.3.2.2 *Loss of prestress due to elastic deformation of masonry*

Calculation of the immediate loss of prestress due to elastic deformation of the masonry at transfer shall be based on the value of the modulus of elasticity of the masonry.

9.3.2.3 *Loss of prestress due to friction*

The loss of prestress due to frictional effects shall be calculated using the provisions of AS 3600.

9.3.2.4 *Loss of prestress during anchoring*

In a post-tensioned member, allowance shall be made for loss of prestress when the prestressing force is transferred from the tensioning equipment to the anchorage.

9.3.3 Time-dependent losses of prestress

9.3.3.1 *General*

The total time-dependent loss of prestress shall be estimated by adding the calculated losses of prestress due to moisture movements of the masonry, creep of the masonry, tendon relaxation, and other considerations as may be applicable.

9.3.3.2 *Loss of prestress due to creep and moisture movements of the masonry*

The percentage loss of prestress due to masonry creep and moisture movement (L_{es}) shall be evaluated from the following equation:

$$L_{es} = 100\left[\varepsilon_s + \left(\frac{C_c f_{pt} A_p}{E_m A_d}\right)\right]\frac{E_p}{f_{pt}} \qquad \text{. . . 9.3.3.2}$$

where

ε_s = the free moisture shrinkage strain one year after prestressing

In the absence of known values, for fired clay masonry, ε_s may be assumed to lie in the range −0.0015 to +0.0002; for calcium silicate and concrete masonry, ε_s may be assumed to be up to 0.0007.

C_c = the coefficient of creep

In the absence of known values, 0.7 for fired clay masonry, 1.5 for calcium silicate masonry and 2.5 for concrete masonry.

f_{pt} = the stress in prestressing tendons at transfer, in megapascals

A_p = the cross-sectional area of the prestressing tendons

A_d = the cross-sectional area of the member

E_p = the modulus of elasticity of the prestressing tendons

E_L = the modulus of elasticity of the masonry

9.3.3.3 *Loss of prestress due to tendon relaxation*

The loss of prestress due to tendon relaxation shall be calculated using the provisions of AS 3600.

9.3.3.4 *Loss of prestress due to temperature effects*

The loss of prestress due to differential thermal movements between the masonry and the prestressing tendons shall be considered.

9.3.4 Close-spaced reinforcement for increased ductility in earthquakes

For a wall to be classed as one with close-spaced reinforcement, it shall be fully grouted and contain prestressing tendons and, optionally, reinforcement, at a maximum spacing of 800 mm both horizontally and vertically. The minimum total area of reinforcement and prestressing tendons in the horizontal and vertical directions shall be $0.0015A_d$.

9.4 DESIGN OF MEMBERS IN COMPRESSION

Prestressed masonry members subjected to compression shall be designed in accordance with Clause 8.5 with the yield strength of the prestressing steel (f_{py}) being substituted for the design yield strength of the reinforcement (f_{sy}). The prestressing force in the member shall be treated as an additional axial load at the appropriate eccentricity.

Values of f_{py} are given in Clause 3.7(b).

9.5 DESIGN OF MEMBERS IN BENDING

9.5.1 General

The design strength in bending shall be as follows:

(a) *At transfer* The following shall apply:

(i) No tensile stress shall be induced in the cross-section. For members where self-weight does not cause bending, this may be achieved by locating the tendons within the middle third of the cross-section for a cored or grouted section, or between the face shells for a hollow section.

(ii) The compressive capacity of the masonry in bending shall be checked using the appropriate loading combinations and capacity reduction factor.

This requirement shall be deemed to be satisfied if the maximum compressive stress in the masonry under the (unfactored) loads does not exceed—

(A) 0.4 (1.3 f'_{mp}), for approximately uniform distribution of prestress; and

(B) 0.5 (1.3 f'_{mp}), for approximately triangular distribution of prestress

where

f'_{mp} = the characteristic compressive strength of the prestressed masonry at the transfer of prestress

(b) *At the strength limit state* Prestressed masonry members subject to bending shall be proportioned such that the following relationship shall be satisfied:

$$M_d \le M_{dp} \quad \text{. . . 9.5.1(1)}$$

where

M_d = the design bending moment acting on the cross-section of a member

M_{dp} = the bending moment capacity of the masonry wall, determined as follows:

(i) For members of other than constant width (b)

$$M_{dp} = \phi\sigma_{pu} A_p\left(d_p - \frac{x}{2}\right) \quad \text{. . . 9.5.1(2)}$$

(ii) For members of constant width (b)

$$M_{dp} = \phi\sigma_{pu} A_p d_p\left[1 - \frac{0.6\sigma_{pu} A_p}{(1.3 f'_m) b d_p}\right] \quad \text{. . . 9.5.1(3)}$$

where

ϕ = the capacity reduction factor (see Clause 4.4)

σ_{pu} = the tensile stress in the tendon at the ultimate strength limit state calculated in accordance with Clause 9.5.2

A_p = the cross-sectional area of the prestressing tendons (see Clause 9.5.3)

d_p = the effective depth of the tendons

x = the depth of the compression zone

f'_m = the characteristic compressive strength of the masonry (see Clause 3.3.2)

b = the width of a masonry member of solid rectangular cross-section; or

= the effective width of a member in accordance with Clause 4.5.2

9.5.2 Ultimate tensile stress in tendons

The ultimate tensile stress in tendons shall be calculated as follows:

(a) *For members with bonded tendons* In the absence of more precise calculations, and provided that the minimum effective stress in the tendons is not less than $0.5f_p$, the tensile stress in the tendons at the ultimate strength limit state (σ_{pu}) shall taken as—

$$\sigma_{pu} = f_p\left(1 - \frac{k_3 k_4}{0.85}\right) \qquad \text{. . . 9.5.2(1)}$$

where

k_3 = 0.28 where $\dfrac{f_{py}}{f_p} \geq 0.9$, otherwise 0.4

$$k_4 = \frac{A_p f_p}{bd_p(1.3 f'_m)}$$

where

f_p = the tensile strength of prestressing tendons (see Clause 3.7.1)

A_p = the cross-sectional area of prestressing tendons (see Clause 9.5.3)

f_{py} = the yield strength of prestressing tendons [see Clause 3.7.1(b)]

f'_m = the characteristic compressive strength of the masonry (see Clause 3.3.2)

b = the width of a masonry member of solid rectangular cross-section; or

= the effective width of a member in accordance with Clause 4.5.2

d_p = the effective depth of the tendons

(b) *For members with unbonded tendons*:

$$\sigma_{pu} = f_{pe} + \frac{700 d_p}{l}\left[1 - \frac{0.7 f_p A_p}{f'_m b d_p}\right] \qquad \text{. . . 9.5.2(2)}$$

where

f_{pe} = the effective stress in the tendon after losses

l = the distance between the anchorages at each end of the member

For unbonded tendons, allowance shall be made for the possible movement of ungrouted tendons in the void of the cross-section. In the absence of fixing that holds the tendon in place, the effective depth shall be calculated assuming the tendon has moved in the cavity by the maximum possible amount in the most unfavourable direction.

9.5.3 Upper limit on tendon area

The total cross-sectional area of tendons (A_p) shall be not greater than the following:

$$A_p = \frac{0.29(1.3 f'_m) b d_p}{\sigma_{pu}} \qquad \text{. . . 9.5.3}$$

where

f'_m = the characteristic compressive strength of the masonry (see Clause 3.3.2)

b = the width of a masonry member of solid rectangular cross-section; or

= the effective width of a member in accordance with Clause 4.5.2

d_p = the effective depth of the tendons

σ_{pu} = the tensile stress in the tendons at the ultimate strength limit state (see Clause 9.5.2)

9.5.4 Minimum bending strength

The bending moment capacity of the wall (M_{dp}) at critical sections shall be not less than 1.2 times the ultimate cracking moment (M_{cr}) given by the following equation:

$$M_{cr} = Z\left(\frac{P}{A_d} + \frac{Pe}{Z}\right) \quad \text{. . . 9.5.4}$$

where

Z = the section modulus of the uncracked section, referred to the extreme fibre at which cracking occurs

P = the effective prestressing force (after losses)

e = the eccentricity of prestress

A_d = the design cross-sectional area of the member (see Clause 4.5.6)

9.6 DESIGN OF MEMBERS IN SHEAR

Prestressed masonry members subjected to in-plane or out-of-plane shear shall be designed in accordance with Clause 8.7 or Clause 8.8, and the yield strength of the prestressing tendons (f_{py}) shall be substituted for the yield strength of the reinforcement (f_{sy}).

NOTE: Values of f_{py} are given in Clause 3.7.1.

9.7 DESIGN OF MEMBERS IN TENSION

Prestressed masonry members subjected to axial tension shall be designed such that the following relationship is satisfied:

$$F_{dt} \leq \phi \sigma_{pu} A_p \quad \text{. . . 9.7}$$

where

F_{dt} = the design tension force acting on the cross-section of a member

ϕ = the capacity reduction factor (see Clause 4.4)

σ_{pu} = the tensile stress in the tendons at the ultimate strength limit state (see Clause 9.5.2)

A_p = the cross-sectional area of prestressing tendons (see Clause 9.5.3)

The main prestressing steel in the direction of the axial load shall—

(a) be located symmetrically in the cross-section;

(b) have at least one tendon within 300 mm of the edge of the member; and

(c) be spaced at centres not exceeding 2000 mm.

9.8 DESIGN FOR COMBINED LOADING

9.8.1 Members in combined bending and compression

A prestressed member subjected to the simultaneous action of compression forces and bending moments shall be designed such that the compressive load capacity is 0.85 times that given by Clause 9.4 for a concentrically loaded member and the bending capacity is in accordance with Clause 9.5.

For members containing unbonded prestressing steel that is free to move in the cross-section under flexure, account shall be taken of this movement when assessing the applied bending moments.

9.8.2 Members in combined bending and tension

A prestressed member subjected to simultaneous axial tension and bending shall be designed so that the tensile load capacity is in accordance with Clause 9.7 for a concentrically loaded member and the bending capacity is in accordance with Clause 9.5.

9.9 DESIGN OF ANCHORAGE ZONES

The capacity of masonry immediately beneath the point of application of the prestress shall be assessed using the provisions of Clause 7.3.5.

Consideration shall be given to the possibility of bursting forces or bending, or shear stresses being produced where anchorages, end blocks or bearing plates have a cross-section different in shape from the general cross-section of the member.

SECTION 10 DESIGN FOR EARTHQUAKE ACTIONS

10.1 SCOPE OF SECTION

This Section sets out additional requirements for masonry structures and structural members that are required to be designed and detailed for earthquake actions.

This Section applies only to masonry structures and structural members that form whole, or part of, structures or buildings to which AS 1170.4 applies.

10.2 GENERAL DESIGN CRITERIA

10.2.1 General

General design and detailing requirements shall be in accordance with this Standard and with AS 1170.4, as applicable.

10.2.2 Structural ductility factor (μ) and structural performance factor (S_p)

The structural ductility factor (μ) and structural performance factor (S_p) for masonry structures with members constructed in accordance with this Standard shall be as given in Table 10.1.

A lower μ value than is specified in Table 10.1 may be used.

In all cases, the structure shall be detailed to achieve the level of ductility assumed in the design and in accordance with this Standard.

TABLE 10.1

STRUCTURAL DUCTILITY FACTOR (μ) AND STRUCTURAL PERFORMANCE FACTOR (S_p)

Description	μ	S_p
Close-spaced reinforced masonry in accordance with Section 8	2	0.77
Wide-spaced reinforced masonry in accordance with Clause 8.4.6	1.5	0.77
Pre-stressed masonry in accordance with Section 9	1.5	0.77
Unreinforced masonry in accordance with Section 7	1.25	0.77

10.2.3 Structural elements

The in-plane and out-of-plane earthquake forces and the inter-storey drift induced in a masonry structure and its elements shall be determined from the provisions of AS 1170.4, Sections 5 and 6 or 7, or from AS 1170.4, Section 8. The in-plane and out-of-plane capacities of the structural elements and their connections shall be determined from Sections 7, 8 or 9 of this Standard for unreinforced, reinforced and prestressed masonry.

10.2.4 Non-structural components

The earthquake forces induced in non-structural masonry components shall be determined from the provisions of AS 1170.4 for the design of parts and components. The in-plane and out-of-plane capacities of non-structural components and their connections shall be determined from Sections 7, 8 or 9 of this Standard for unreinforced, reinforced and prestressed masonry.

10.2.5 Connections and wall anchorage

The connections and wall anchorages of all masonry structural and non-structural elements shall be designed and detailed such that 1.25 times the earthquake forces on the wall, calculated from AS 1170.4, induced in those elements can be transferred to their supporting systems (see Clause 2.6.4).

The capacities of shear planes or shear connections to transmit earthquake forces to the relevant support shall be determined from Clause 7.5, 8.7 or 9.6.

Where face loads are transmitted to the structural supporting system by wall ties, the ties shall comply with Clause 4.10 and their capacity shall be determined from Clause 7.6 or 7.7.

10.3 DETAILING MASONRY STRUCTURES FOR EARTHQUAKE LOADS

10.3.1 General

Detailing of masonry elements in buildings up to 15 m high, which have stiffness such that the inter-storey drift is less than 10 mm under the design loads, shall be in accordance with Table 10.2 unless their conformance with the requirements of AS 1170.4 and Sections 7, 8 or 9 of this Standard can be demonstrated by either calculation or test.

Buildings up to 15 m high that satisfy the following criteria are deemed to have an inter-storey drift less than 10 mm:

(a) Having a regular wall arrangement in plan.

(b) Not incorporating soft storeys.

(c) Incorporating either loadbearing masonry walls or concrete frames with concrete or masonry shear walls.

For masonry elements in buildings outside the above criteria, either—

(i) the details (including gaps between masonry and supports) shall be such as to isolate the masonry from the structure for the inter-storey drifts associated with the seismic force-resisting system of the particular building; or

(ii) the building and its masonry components shall be designed and detailed with the masonry considered as part of the seismic force-resisting system.

In either case, the building and its masonry components shall comply with the provisions for component deformation of AS 1170.4.

NOTE: In addition to the detailing requirements, design requirements for the masonry elements in accordance with Clause 10.2.3 or Clause 10.2.4 apply.

10.3.2 Fixing of cavity walls to supports

In external cavity walls more than one storey high, where the internal leaf is discontinuous and supported between concrete floor slabs and the external leaf is continuous past the floor slabs, the external leaf shall be fixed or restrained with additional ties across the cavity at each floor and roof support (see Note 1). The ties shall have sufficient strength to transmit the earthquake load on the exterior wall, in excess of the interior leaf wall/slab shear capacity, directly to the support, rather than via the internal leaf. The factor of 1.25 required by Clause 2.6.4 shall be applied only to the difference between the unfactored earthquake load and the shear capacity of the wall/slab junction.

NOTES:

1 The specified ties are in addition to those required by Clauses 4.10 and 7.7.

2 The shear capacity of the wall/slab junction is given by Clause 7.5.4 for unreinforced masonry, and Clause 8.7.3 for reinforced masonry.

TABLE 10.2

DETAILING OF MASONRY FOR EARTHQUAKE LOADS—BUILDINGS UP TO 15 m HIGH

Application	Minimum thickness of masonry (see Note 2)	Is masonry designed as part of the seismic force-resisting system (see Note 3)	Acceptable details (see Figures 10.1 to 10.12 and Note 4)	Structural ductility and structural performance factors (see Note 5)
Non-loadbearing unreinforced masonry	90 mm	No	Top: T1, T2 Sides: S1, S2, S3 Bottom: B1, B2	Structural ductility factor (μ) and structural performance factor (S_p) based on the building seismic force-resisting system, as determined in accordance with AS 1170.4
Non-loadbearing wide-spaced reinforced masonry (see Note 6)	140 mm	Yes	Top: T1, T2 Sides: S1, S2, S3, S4 Bottom: B3	Structural ductility factor (μ) 1.5 Structural performance factor (S_p) 0.77
		No	Top: T1, T2 Sides: S1, S2, S3, S4 Bottom: B1, B2	Structural ductility factor (μ) and structural performance factor (S_p) based on the building seismic force-resisting system, as determined in accordance with AS 1170.4
Non-loadbearing close-spaced reinforced masonry (see Note 6)	190 mm	Yes	Top: T1, T2 Sides: S1, S2, S3, S4 Bottom: B3	Structural ductility factor (μ) 2.0 Structural performance factor (S_p) 0.77
		No	Top: T1, T2 Sides: S1, S2, S3, S4 Bottom: B1, B2	Structural ductility factor (μ) and structural performance factor (S_p) based on the building seismic force-resisting system, as determined in accordance with AS 1170.4
Loadbearing unreinforced masonry	90 mm	Yes	Top: T3, T5 Sides: S1, S2, S3 Bottom: B1, B2	Structural ductility factor (μ) 1.25 structural performance factor (S_p) 0.77
Loadbearing wide-spaced reinforced masonry (see Note 6)	140 mm	Yes	Top: T4 Sides: S1, S2, S3, S4 Bottom: B3	Structural ductility factor (μ) 1.5 Structural performance factor (S_p) 0.77
Loadbearing close-spaced reinforced masonry (see Note 6)	190 mm	Yes	Top: T4 Sides: S1, S2, S3, S4 Bottom: B3	Structural ductility factor (μ) 2.0 Structural performance factor (S_p) 0.77
Masonry veneer (see Note 7)	90 mm	No	Top: T1, T2 Sides: S1, S2, S3 Bottom: B1, B2	Structural ductility factor (μ) and structural performance factor (S_p) same as those for the building seismic force-resisting system, as determined in accordance with AS 1170.4

NOTES TO TABLE 10.2:

1 The details given in this Table are applicable to buildings up to 15 m high that have stiffness such that the inter-storey drift is less than 10 mm.

2 The thickness of masonry walls shall also comply with the other requirements of this Standard, including Clause 4.6 (robustness) and strength capacities determined in accordance with Sections 7, 8 and 9. The thickness limits given in this Table have been determined giving consideration to the following points:

(a) Unreinforced masonry walls with thickness greater than 90 mm have sufficient robustness for most practical applications.

(b) It is difficult to grout hollow masonry less than 140 mm thick that incorporates either horizontal or vertical reinforcement (but not both), as is usually the case for wide-spaced reinforcement.

(c) It is difficult to grout hollow masonry less than 190 mm thick that incorporates both horizontal and vertical reinforcement, as is the requirement for close-spaced reinforcement.

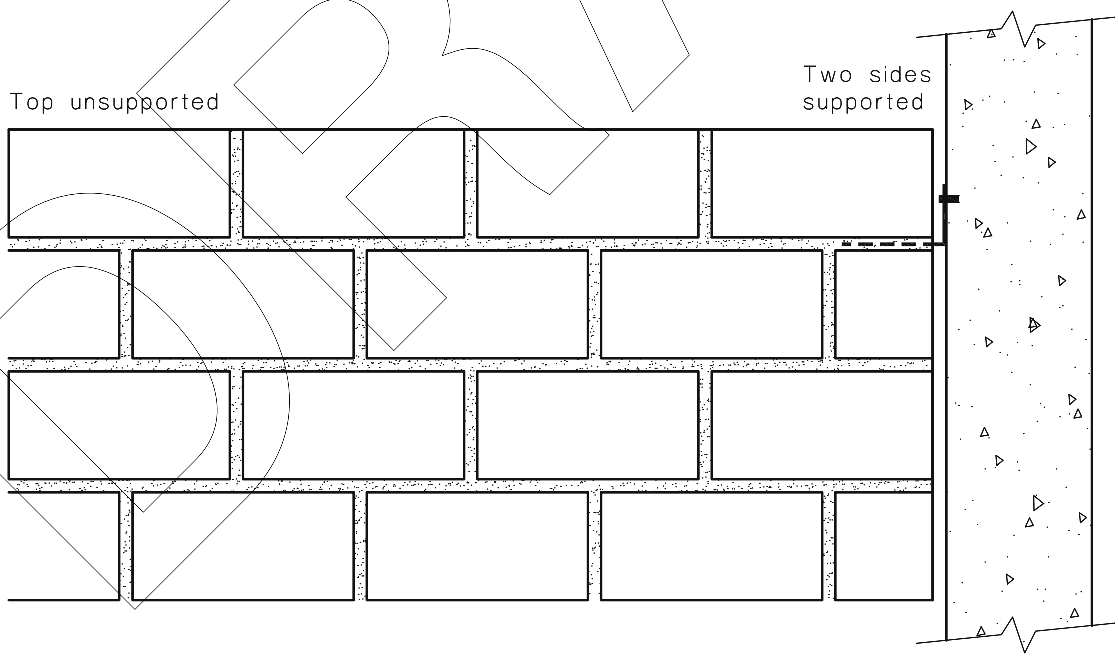
3 Where the masonry is not designed as part of the seismic force-resisting system, an adequate independent force-resisting system is required and the masonry shall be designed to transfer loads within its own height.

4 Refer to Notes to Figures 10.1 to 10.12 for limitations of use.

5 The structural ductility factor (μ) and structural performance (S_p) factors are appropriate for the form and material of the building (refer to AS 1170.4), based on the level of isolation provided by the selected details. These factors are used to determine design loads for the masonry.

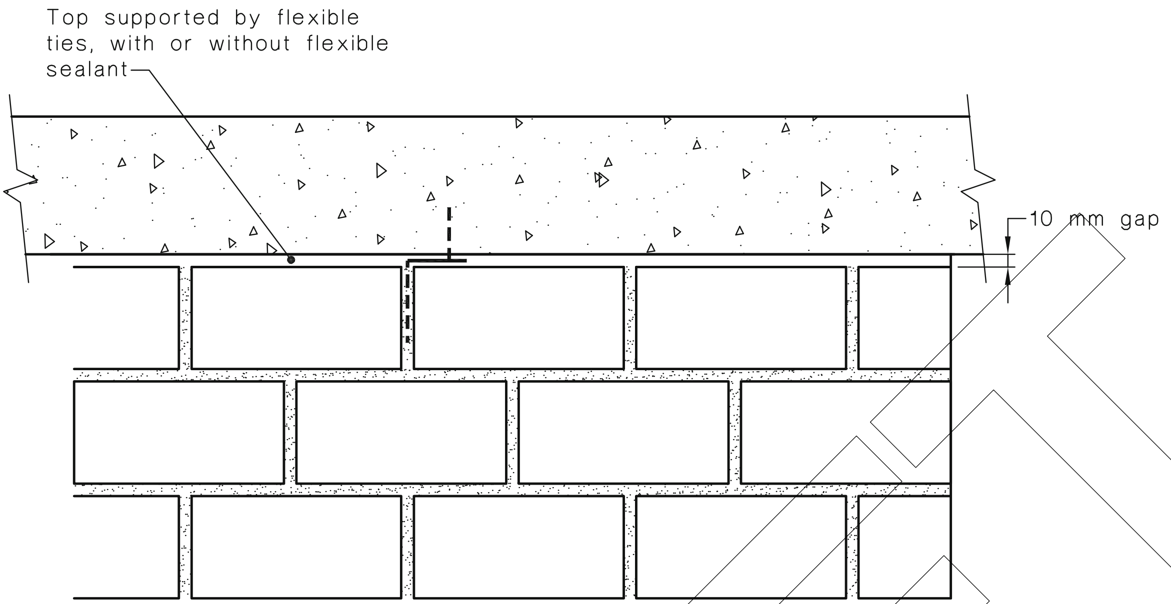
6 If significant reinforced masonry walls are tied rigidly into the supporting structure by starter bars or similar (as is common), and thus become part of the seismic force-resisting system, the structure will most likely assume the ductility factor (μ) and structural performance factor (S_p) of the particular reinforced masonry system (i.e., for close-spaced reinforcement, ductility factor (μ) = 2.0 and structural performance factor (S_p) = 0.77; for wide-spaced reinforcement, ductility factor (μ) = 1.5 and structural performance factor (S_p) = 0.77). If reinforced masonry walls are isolated from the supporting structure by the details shown, and thus are not part of the seismic force-resisting system, the structure ductility factor (μ) and structural performance factor (S_p) will remain unaffected by the reinforced masonry.

7 Location and selection of masonry veneer ties shall comply with Clauses 4.10 and 7.6.



NOTE: Type T1 (no support top) shall be used only if two side supports are provided.

FIGURE 10.1 DETAILS OF TOP SUPPORT T1—NO TOP SUPPORT

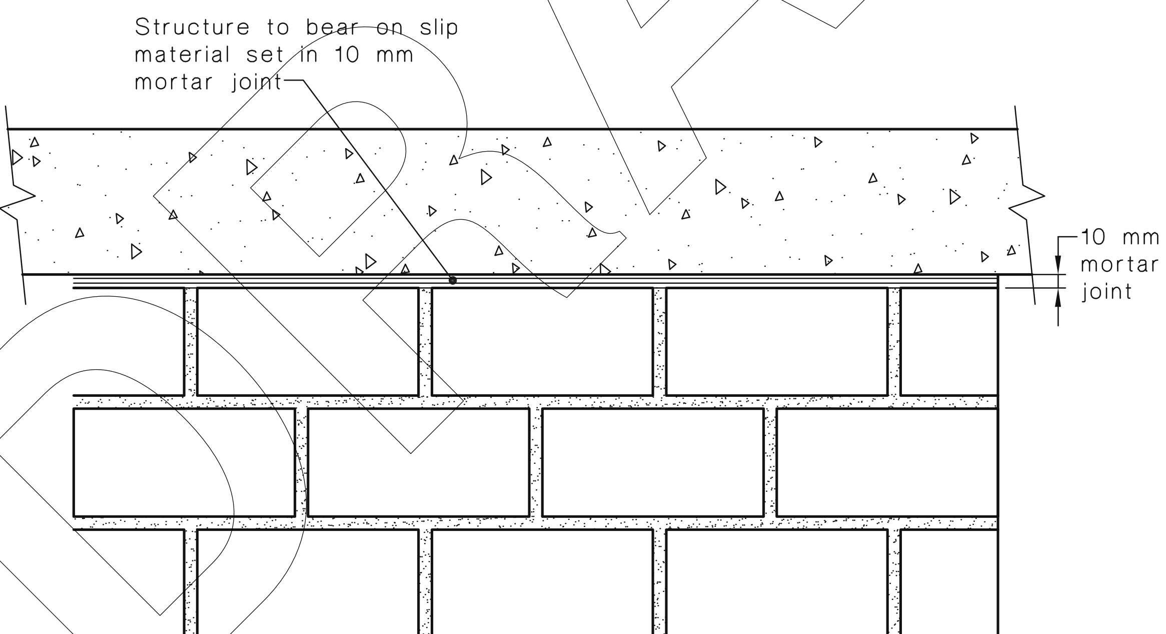


NOTES:

1 Type T2 requires at least 10 mm gap, with or without flexible sealant, and with flexible ties to a rigid support.

2 Flexible ties shall provide out-of-plane support for the wall, while permitting in-plane movement.

FIGURE 10.2 DETAILS OF TOP SUPPORT T2—FLEXIBLE TIES WITH GAP TO A RIGID SUPPORT

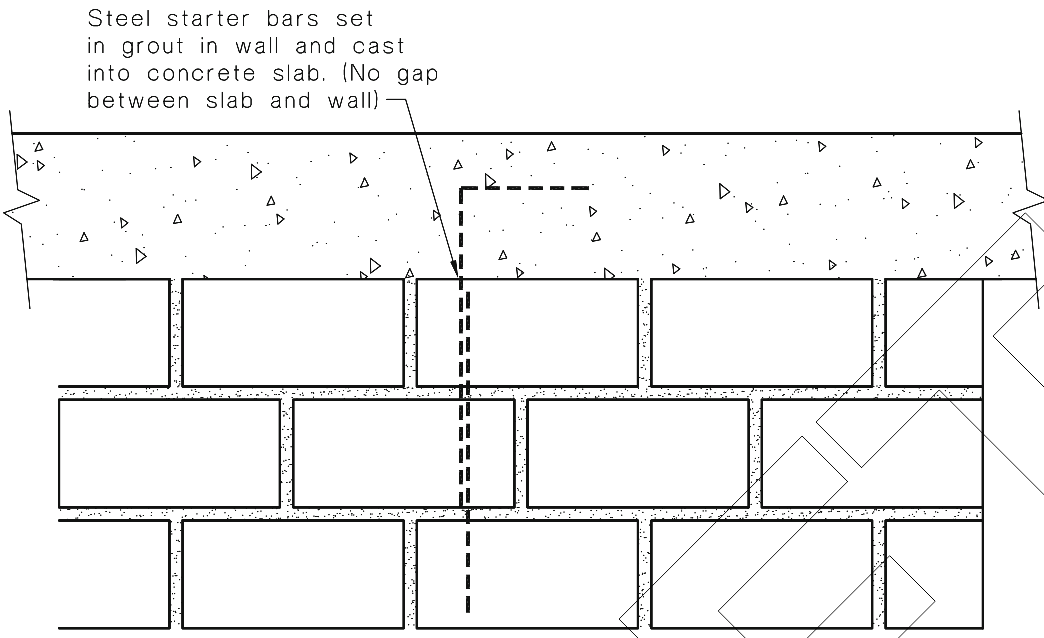


NOTES:

1 Type T3 (bed with slip joint or DPC) shall comply with Clauses 4.9 and 4.7.3 of this Standard.

2 Slip joint material, DPC or similar, shall have a shear factor (k_v) not less than 0.15 and not more than 0.3 determined in accordance with Clause 3.3.5.

FIGURE 10.3 DETAILS OF TOP SUPPORT T3—MORTAR BED WITH SLIP JOINT OR DPC



NOTES:

1 Type T4 (steel starter bars) shall be such that the interface is confined by bonded reinforcement.

2 The shear strength of the joint shall be determined in accordance with Clauses 8.7 and 3.3.4(d).

FIGURE 10.4 DETAILS OF TOP SUPPORT T4—STEEL STARTER BARS

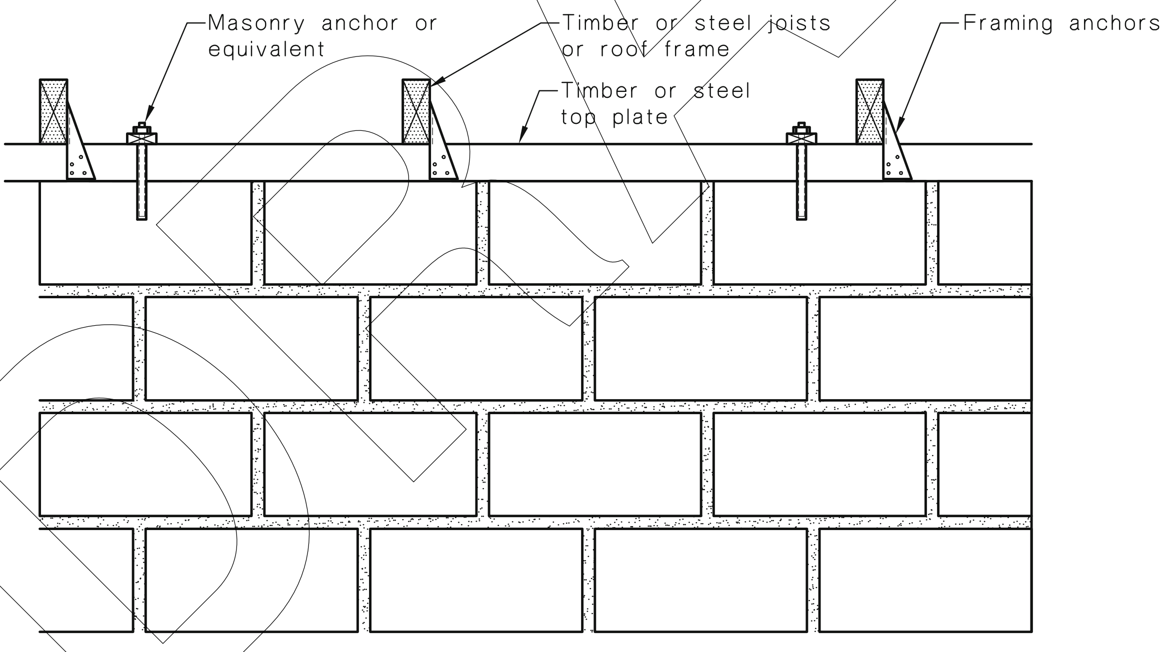
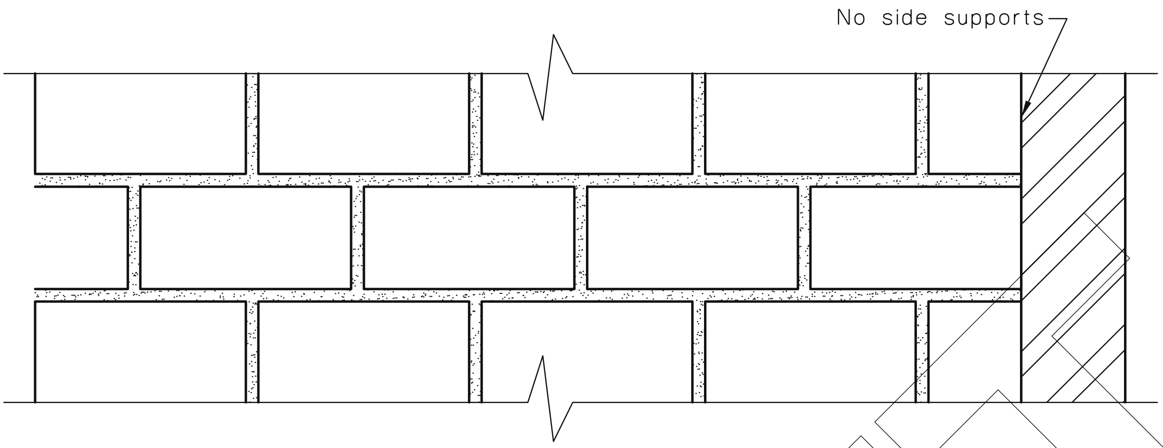
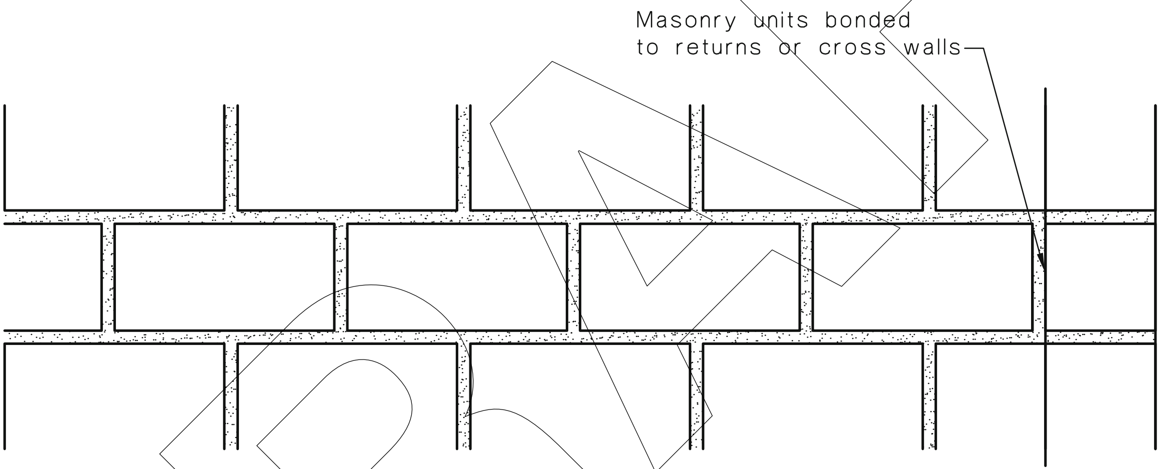


FIGURE 10.5 DETAILS OF TOP SUPPORT T5—TIMBER OR STEEL FRAMING



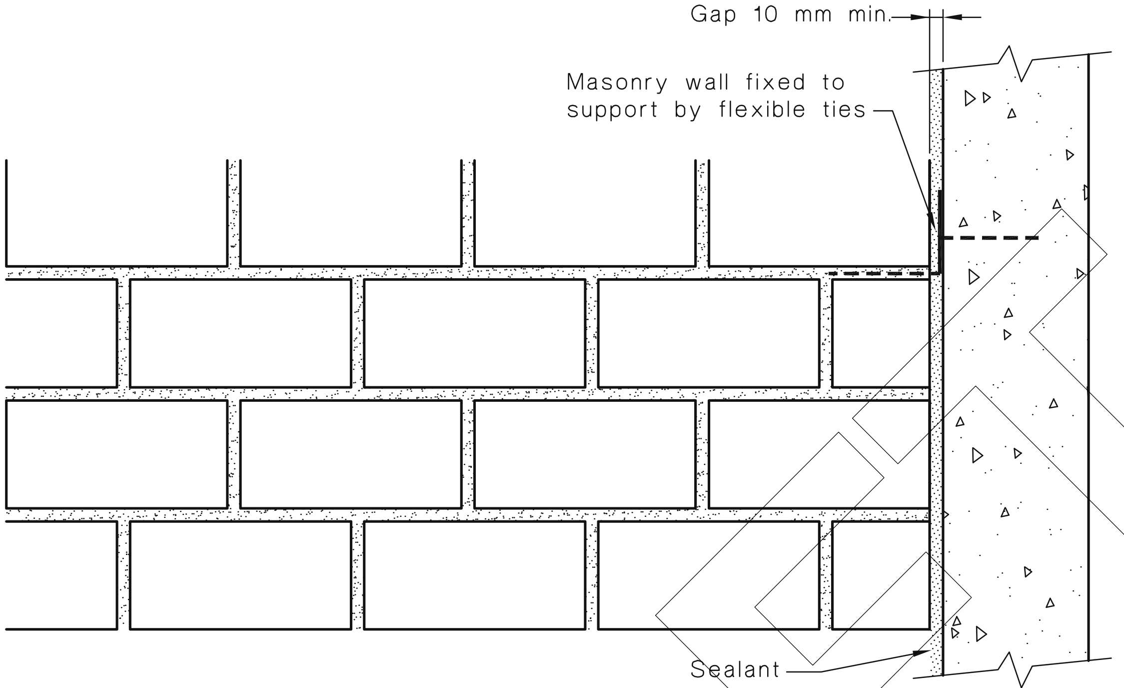
NOTE: Type S1 (no side support) shall be used only if both the top and bottom of the masonry wall are supported by the structure.

FIGURE 10.6 DETAILS OF SIDE SUPPORT S1—NO SIDE SUPPORT



NOTE: Type S2 (built into returns or cross-walls)—bonding shall comply with Clauses 4.11.2 or 4.11.3 of this Standard.

FIGURE 10.7 DETAILS OF SIDE SUPPORT S2—BUILT INTO RETURNS OR CROSS-WALLS

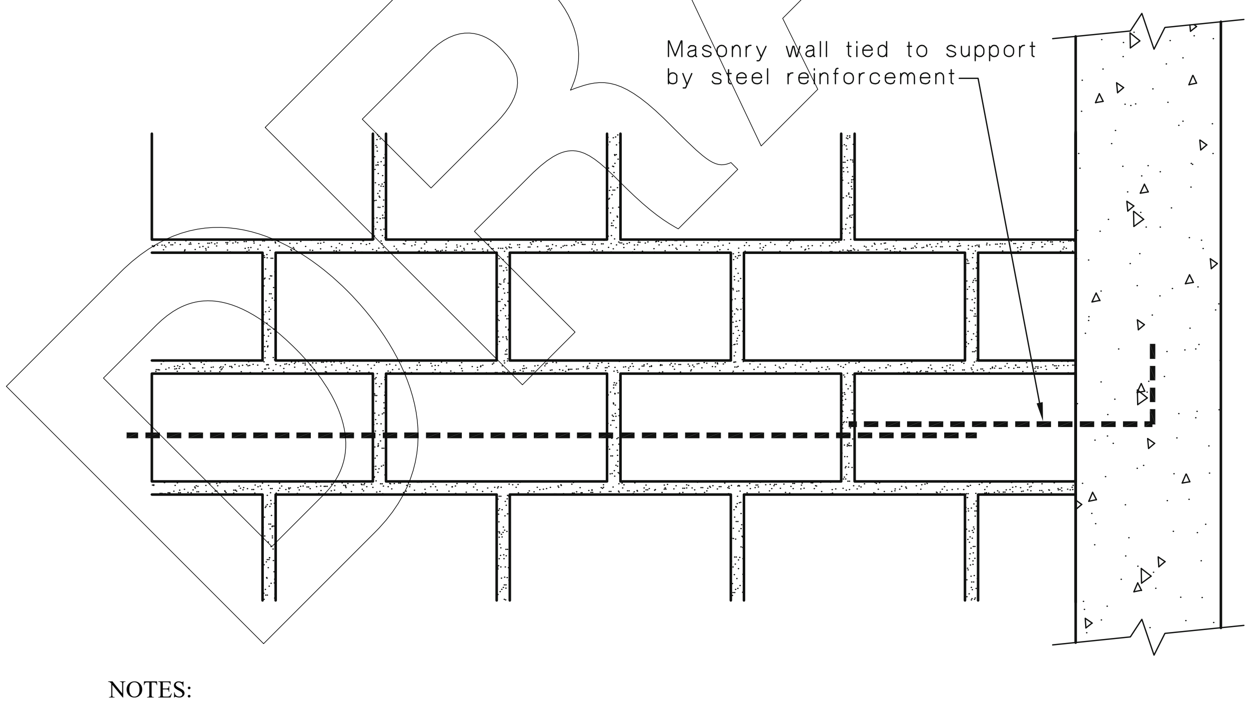


NOTES:

1 Type S3 (tied to piers, mullions or cross-walls)—ties shall comply with Clause 4.11.3 of this Standard.

2 Flexible ties shall provide out-of-plane support for the wall, while permitting in-plane movement.

FIGURE 10.8 DETAILS OF SIDE SUPPORT S3—TIED TO PIERS, MULLIONS OR CROSS-WALLS

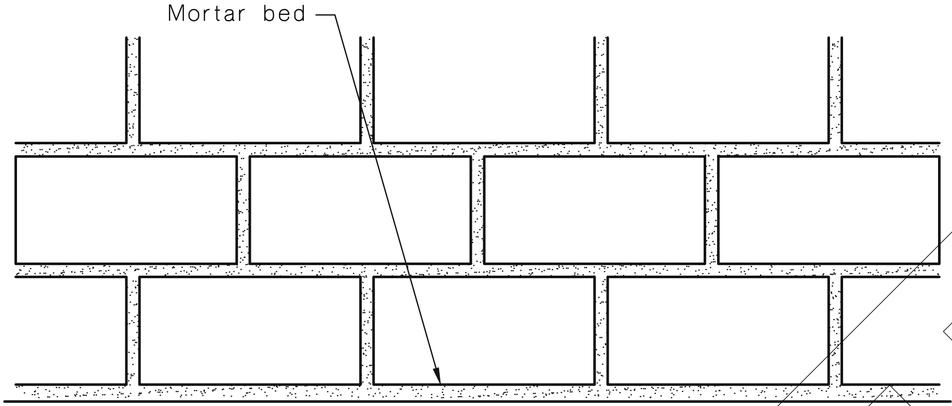


NOTES:

1 Type S4 (steel reinforcement) shall be such that the interface is confined by bonded reinforcement.

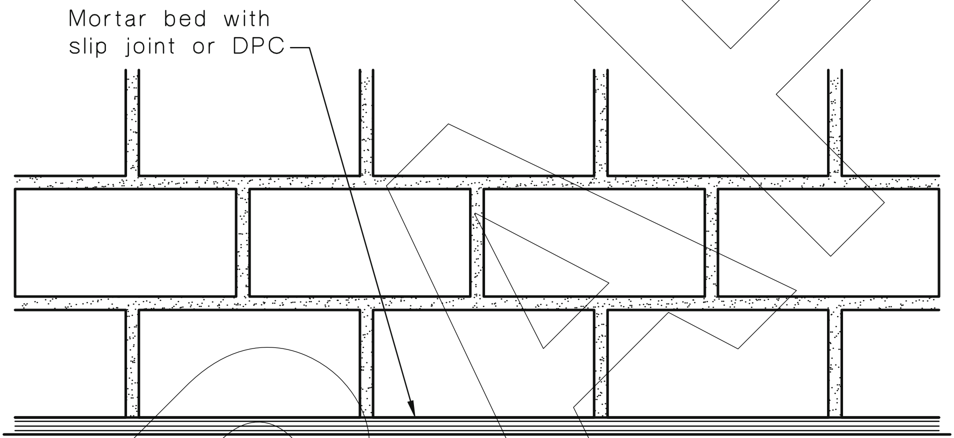
2 The shear strength of the joint shall be determined in accordance with Clauses 8.7 and 3.3.4(d).

FIGURE 10.9 DETAILS OF SIDE SUPPORT S4—STEEL REINFORCEMENT



NOTE: Type B1 (mortar bed) shall comply with Clause 4.9 of this Standard.

FIGURE 10.10 DETAILS OF BOTTOM SUPPORT B1—MORTAR BED



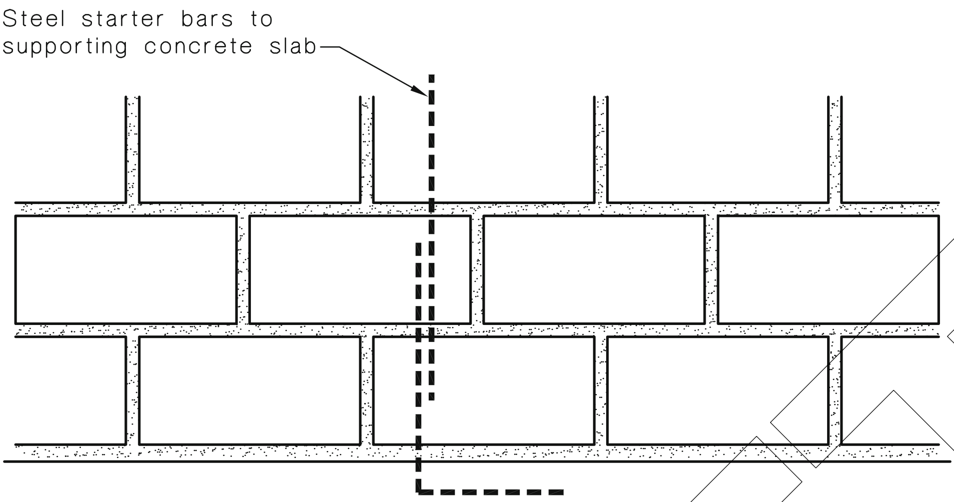
NOTES:

1 Type B2 (bed with slip joint or DPC) shall comply with Clauses 4.9 and 4.7.3 of this Standard.

2 The shear strength of the joint shall be determined in accordance with Clauses 7.5, 3.3.4(a) and 3.5.

3 Slip joint material, DPC or similar shall have a shear factor (k_v) not less than 0.15 and not more than 0.3, determined in accordance with Clause 3.3.5.

FIGURE 10.11 DETAILS OF BOTTOM SUPPORT B2—MORTAR BED WITH SLIP JOINT OR DPC



NOTES:

1 Type B3 (steel starter bars) shall be such that the interface is confined by bonded reinforcement.

2 The shear strength of the joint shall be determined in accordance with Clauses 8.7 and 3.3.4(d).

FIGURE 10.12 DETAILS OF BOTTOM SUPPORT B3—STEEL STARTER BARS

10.4 RESTRICTIONS ON THE USE OF LOADBEARING UNREINFORCED MASONRY

Buildings with heights greater than the values shown in Table 10.3 shall not incorporate loadbearing unreinforced masonry elements, except where the masonry complies with the following:

(a) The loadbearing masonry supports only a trafficable or non-trafficable roof or mezzanine floor that exerts a maximum of 10 kN/m permanent load to the masonry.

(b) All isolated masonry piers are constructed of reinforced masonry and designed accordingly.

(c) The area supported by the unreinforced masonry is less than 25% of the plan area of the structure on which it is supported.

(d) The unreinforced masonry is not within 3.0 m of the edge of the structure on which it is supported.

(e) The loadbearing masonry components are designed for loads derived from AS 1170.4.

TABLE 10.3

HEIGHT LIMITS FOR BUILDINGS WITH LOADBEARING UNREINFORCED MASONRY

Hazard factor (Z) (as per AS 1170.4)	Height limits, m				
	Subsoil classifications (as per AS 1170.4)				
	A_e	B_e	C_e	D_e	E_e
≥0.11	12	12	12	10	10
0.10	15	12	12	12	10
0.09	15	15	12	12	12
0.08	15	15	15	12	12
0.07	15	15	15	15	12
≤0.06	15	15	15	15	12

NOTE: These limits are not intended to apply to small loadbearing unreinforced masonry structures (e.g., plant rooms and mezzanine floors) contained within larger buildings, which may be over the prescribed heights.

SECTION 11 MATERIALS

11.1 SCOPE OF SECTION

This Section sets out the requirements for the materials for masonry construction.

NOTE: For design properties, see Section 3.

11.2 MASONRY

Masonry shall be constructed from compatible combinations of units and mortar to achieve—

(a) the strength in accordance with the requirements of the design; and

(b) durability in the proposed exposure environment in accordance with the requirements of the design.

11.3 MASONRY UNITS

Masonry units shall be in accordance with AS/NZS 4455.1.

11.4 MORTAR

11.4.1 General

Mortar (except for thin-bed mortar) shall consist of a mixture of cementitious material and sand (fine aggregate) to which water and any specified additives have been added. The ingredients shall be proportioned to produce a mortar that will have the following characteristics:

(a) Adequate workability to permit the masonry units to be properly placed.

(b) The ability, in conjunction with the masonry units, to provide the structural properties and durability required for the given situation.

Mortars shall be classed as M1 to M4 as follows:

(i) In accordance with Table 11.1; or

(ii) By demonstrated performance meeting the requirements of Table 3.1, Clause 3.3.3 and Table 11.2.

Class M1 mortars shall be used only for restoration of existing buildings that have been originally constructed using this type of mortar.

NOTE: Appendix H provides guidance on the use of masonry in restoration work.

Thin-bed mortar for AAC masonry is a proprietary material and shall be deemed to be Class M3.

11.4.2 Materials

11.4.2.1 *Cement and building lime*

Cement and building lime shall be in accordance with the following Standards:

(a) Portland (Type GP) and blended (Type GB) cements .... AS 3972.

(b) Limes for building .... AS 1672.1.

(c) Masonry cement .... AS 1316.

11.4.2.2 *Sand*

Sand shall be free from materials deleterious to the mortar and to embedded items, and shall be chosen to produce mortar that meets the requirements of this Standard.

11.4.2.3 *Water*

Water shall be free from harmful quantities of materials that are deleterious to the masonry, the reinforcement or any embedded items.

Water that is potable is deemed to satisfy this requirement.

11.4.2.4 *Admixtures*

Admixtures used in mortar shall comply with the following:

(a) Admixtures other than the following types shall not be used unless tests have demonstrated that masonry incorporating the admixtures comply with the requirements of this Standard for compressive strength, flexural strength and durability of the masonry:

    (i) Plasticizers or workability agents specifically designed for use in masonry, including air-entraining agents in accordance with AS 1478.1.

    (ii) Cellulose-type chemical water thickeners.

    (iii) Colouring pigments in accordance with EN 12878.

    (iv) Set-retarding chemical agents in accordance with AS 1478.1.

    (v) Bonding polymers.

(b) Any admixtures shall be stored, handled and used in accordance with the manufacturer's instructions.

(c) Mortars containing set-retarding chemical agents, with or without bonding polymers, shall be factory-batched and delivered without any further additions. These mortars shall set after the designated retardation period.

11.4.3 Mortar durability

Mortars complying with the proportions given in Table 11.1 are deemed to satisfy the durability requirements of Clause 11.4.1(b) for the situations set out in Table 5.1 (see Clause 5.6).

For mortar compositions not covered in Table 11.1, a scratch index shall be determined in accordance with Appendix E. The results shall be interpreted using the values given in Table 11.2. The age at testing shall be not less than 7 d.

NOTE: In colder climates, it may be advisable to allow up to 21 d for the mortar to harden sufficiently before testing.

11.4.4 Structural properties of mortar

Mortars complying with the proportions given in Table 11.1 are deemed to satisfy the structural requirements of Clause 11.4.1(b).

For mortar compositions not covered in Table 11.1, testing shall be carried out in accordance with Appendices C or D, to confirm that the masonry achieves the required strength as follows:

(a) Characteristic compressive strength (f'_{m}) in accordance with Clause 3.3.2(a) or the value specified by the designer.

(b) The flexural tensile strength (f'_{mt}) of 0.2 MPa or the value specified by the designer.

NOTE: Appendices C and D require the materials and methods of construction for test specimens to be the same as those used for work under construction.

11.4.5 Mortar for reinforced or prestressed masonry

The mortar used in masonry that is to be reinforced or prestressed shall be of class M3 or M4.

TABLE 11.1

DEEMED-TO-SATISFY MORTAR COMPOSITIONS

Mortar class	Mix proportions by volume					Units for which mortar is suitable			
	Cement (GB/GP)	Masonry cement	Building lime	Sand	Water thickener (see Note 3)	Fired clay	Concrete	Calcium silicate	AAC
M1 (see Note 4)	0 1	0 0	1 3	3 12	No No	✓ ✓	× ×	× ×	× ×
M2	1	0	2	9	No	✓	×	×	×
M3	1 1 0	0 0 1	1 0 0	6 5 4	Optional Yes Yes	✓ ✓ ✓	✓ ✓ ✓	× ✓ ×	× × ×
M3	Thin-bed mortar for use with AAC (see Clause 11.4.1)								
M4	1 1 1 0	0 0 0 1	0.5 0 0 to 0.25 0	4.5 4 3 3	Optional Yes Optional Yes	✓ ✓ ✓ ✓	✓ ✓ ✓ ×	× ✓ × ×	× × × ×

✓ satisfactory

× unsatisfactory

NOTES:

1 Mortar mixes are designated by the proportions of their ingredients following an initial letter, the chief cementing agent being given as unity (e.g. C 1:L 0.5:S 4.5 or L 1:S3).

2 Volumes refer to materials in the dense-packed condition.

3 The water thickener referred to in this Table is cellulose based. The particular cellulose-based product used shall be suitable for this application, and used in accordance with the manufacturer's or supplier's instructions.

4 Refer to Clause 11.4.1 for restriction on the use of Class M1 mortar.

TABLE 11.2

CRITERIA FOR MORTAR DURABILITY

Mortar Class	Scratch Index (mm)
M2	0.5
M3	0.3
M4	0.1

11.5 WALL TIES, CONNECTORS, ACCESSORIES AND LINTELS

11.5.1 Wall ties

Wall ties shall comply with AS/NZS 2699.1.

11.5.2 Connectors and accessories

Connectors and accessories, including, but not limited to, masonry anchors, connectors, ties (other than wall ties), bed joint mesh, bolts and fixings, shall be in accordance with AS/NZS 2699.2.

Masonry anchors, ties and similar connection accessories (including bed joint mesh) that extend across a cavity in masonry construction shall have a resistance to water transfer similar to that required for wall ties in AS/NZS 2699.1.

11.5.3 Lintels

Steel lintels and shelf angles shall be in accordance with AS/NZS 2699.3.

11.6 DAMP-PROOF COURSES (DPCs), FLASHINGS AND WEATHERINGS

11.6.1 Membrane damp-proof course and flashing (DPC)

Membrane DPC and flashing materials shall be in accordance with AS/NZS 2904.

11.6.2 Mortar weatherings

Mortar used to weather the tops of walls or parapets, and for the laying of masonry units to form copings or sills shall be of Class M3 or M4.

11.7 GROUT

11.7.1 General

Grout shall consist of cement and aggregate, with or without the addition of fly ash, ground granulated iron blast-furnace slag, silica fume or chemical admixtures, and shall be proportioned to produce a pouring consistency that will enable the cores or cavities to be completely filled and reinforcement, if any, to be completely surrounded without segregation of the constituents.

Cement content of grout for durability purposes is given in Clause 5.8.

11.7.2 Materials

11.7.2.1 *Cement*

Cement shall be in accordance with AS 3972.

11.7.2.2 *Fly ash*

Fly ash shall be in accordance with AS/NZS 3582.1.

11.7.2.3 *Slag—Ground granulated blast-furnace*

Ground granulated blast-furnace slag shall be in accordance with AS 3582.2.

11.7.2.4 *Amorphous silica*

Amorphous silica shall be in accordance with AS/NZS 3582.3.

11.7.2.5 *Aggregate*

Aggregate shall be assessed using the test methods in AS 2758.1, and shall be free from materials deleterious to the masonry or to embedded items.

The maximum size of coarse aggregate, if any, shall be chosen with due regard to the spaces into which the grout is to be placed and shall be the least of—

(a) the cover to the bar;

(b) cover to fitments; or

(c) 12 mm.

11.7.2.6 *Water*

Water shall be free from harmful quantities of materials that are deleterious to the masonry, the reinforcement or any embedded items.

Water that is potable is deemed to satisfy this requirement.

11.7.2.7 *Chemical admixtures*

Chemical admixtures shall be in accordance with AS 1478.1.

11.7.3 Strength

Grout shall be proportioned to have a characteristic compressive strength of not less than 12 MPa.

NOTES:

1 For testing of grout, see Clause 12.8.3.

2 For the minimum cement content when grout is required to provide protection to reinforcement, see Clause 5.8.

3 Site-mixed grout should consist of 1 part cement: 2 parts sand: 4 parts aggregate.

11.8 REINFORCEMENT AND TENDONS

11.8.1 Reinforcement

Steel reinforcement shall be in accordance with AS/NZS 4671. The material used in stainless steel reinforcement shall comply with Clause 3.6 and 5.9.

NOTE: The selection of the incorrect grade of stainless steel alloy may result in corrosion of the stainless steel.

11.8.2 Tendons

Tendons shall be in accordance with AS/NZS 4672.1.

11.8.3 Reinforcement embedded in mortar joints

Reinforcement placed in mortar joints shall—

(a) have an overall diameter or thickness not less than 3 mm and not greater than two-thirds of the thickness of the mortar joint; and

(b) be such that the joints containing the reinforcement do not suffer premature cracking under load, or have an unacceptable reduction in flexural strength (see also Clause 5.9.3).

SECTION 12 CONSTRUCTION

12.1 SCOPE OF SECTION

This Section sets out the requirements for masonry construction including workmanship, site control and the additional construction and testing requirements for special masonry.

12.2 GENERAL

Masonry shall be constructed with site control and supervision so that the strength and other requirements of the design and of this Standard are achieved.

12.3 MATERIALS

12.3.1 General

Materials on site, both before and after incorporation in the structure, shall be protected so that their performance in the structure is not impaired.

Materials that have been damaged or have deteriorated such that the performance of the masonry built from them would be adversely affected shall not be used.

12.3.2 Masonry units

12.3.2.1 *Moisture content*

The moisture content of the units at the time of laying shall be such that adverse effects, including reduced bond, occurrence of efflorescence and increased shrinkage, are kept within acceptable limits.

NOTE: Adjusting the composition of the mortar mix to suit the suction of the units, or appropriate use of water thickener, is preferable to wetting of units before laying, which can increase the risk of the adverse effects listed above.

12.3.2.2 *Properties of units*

Where the documents require that masonry units have particular properties, suitability of the units shall be confirmed by the methods set out in AS/NZS 4455.1.

12.3.3 Mortar

12.3.3.1 *General*

The constituents and mix proportions of the mortar shall comply with those specified in the documents.

12.3.3.2 *Measurement of materials*

The method used for measuring materials used in mortar shall ensure that the specified proportions are maintained.

NOTE: It is recommended that volume batching be used.

12.3.3.3 *Mixing*

Mortar shall be mixed by a method and for a period of time that ensure all ingredients are evenly distributed throughout the mixture.

NOTE: When air-entraining admixtures are used, excessive mixing time could lead to a reduction in bond strength and should be avoided.

12.3.3.4 *Age of mortar when used*

Mortar in which initial set has occurred shall not be used.

Retarded mortar shall be used only within the period of retardation guaranteed by the supplier of the mortar.

Prior to its initial set, mortar may be retempered by adding water and thoroughly remixed, ensuring that there is no washing away of cementitious materials.

12.4 WORKMANSHIP

12.4.1 Base course

The surface on which the base course is laid shall be clean.

12.4.2 Mortar joints

Solid and cored units shall be laid on a full bed of mortar. Hollow units shall be face-shell bedded.

Vertical joints in fully bedded masonry shall be filled with mortar unless otherwise specified.

12.4.3 Movement control joints

Expansion joints (closing control joints) and articulation joints shall be clean and free from any hard or incompressible material for the full width and depth of the joint before joint-filling material (if any) is inserted.

Expansion joints shall be completely filled with joint filler—

(a) where specified in the documents; or

(b) where they are located below ground level or where they can be accidentally filled with debris.

12.4.4 Bonding

Masonry shall be bonded in accordance with the specified bond pattern and Clause 4.11.

The bond pattern shall provide for—

(a) header units across vertical joints to achieve such monolithic structural action as required by the design (see Clause 4.11); and

(b) the overlap of the units in successive courses to achieve such horizontal bending strength as required by the design (see Clause 7.4.3).

12.4.5 Cutting of units

The cutting of units shall be avoided as far as possible by setting out to avoid irregular or broken bond, particularly adjacent to openings and in piers. If required, units shall be cut—

(a) so that there is no shattering of the units;

(b) accurately to the leaf thickness; and

(c) so as not to impair the effectiveness of any cavity or reduce the net area of masonry.

For masonry that is special masonry in respect of its compressive strength, units shall not be cut horizontally except by means of a masonry saw.

12.4.6 Holes and chases

Holes and chases shall not be made in masonry, except at positions specified in the design.

During the erection of masonry, the cutting of completed walls shall be kept to a minimum by utilizing holes, sleeves and chases.

12.4.7 Building in

Wall ties and accessories embedded in mortar joints shall be built in as the construction proceeds.

Veneer wall ties shall be attached to the structural backing with the fastener supplied by the tie manufacturer.

NOTE: AS/NZS 2699.1 requires the fastener to be supplied with the tie.

Wall ties shall be embedded at least 50 mm into the mortar joint, have at least 15 mm cover from any exposed surface of the joint and be attached to the structural backing to achieve their rated load capacity.

NOTE: Reduction of embedment within the limits of wall tolerance (see Table 12.1) is assumed not to affect performance.

In hollow unit masonry, where it is not practicable to obtain the required embedment of the ties wholly in the mortar joint, the embedment shall be obtained by filling appropriate cores with grout or mortar.

Wall ties, connectors and accessories that extend across a cavity in the masonry shall be installed in a manner that prevents water from passing across the cavity via those ties, connectors or accessories. The change in height of wall ties across the wall cavity shall not exceed 10 mm.

NOTES:

1 The testing procedures of AS/NZS 2699.1 and AS/NZS 2699.2 are designed to ensure that water will not transfer across a cavity when the ties and connectors are installed without any slope across the cavity.

2 Excessive slope of ties and connectors across a cavity will reduce their strength below the assigned duty rating.

Provision for all partitions, straps, beams, trusses, plates, and similar members that are to be built or keyed into masonry shall be made as work proceeds so as to minimize subsequent cutting or chasing of the masonry.

12.4.8 Bolts and anchors

Bolts and anchors may be set or drilled into the masonry, provided their installation does not adversely affect the performance of the masonry.

12.4.9 Rate of construction

The rate of construction shall be regulated so as to prevent joint deformation, slumping or instability affecting the compliance of the masonry with this Standard.

12.4.10 Sections of masonry constructed at different rates or times

Where two or more adjoining sections of masonry, including intersecting walls, are constructed at different rates or times, the intersections between those sections shall be either raked back or tied with metal ties so that monolithic structural action is obtained in the completed work.

New masonry shall not be toothed into existing work unless so specified.

12.4.11 Construction during adverse weather conditions

Masonry shall not be constructed during weather conditions that might adversely affect the masonry, unless measures are taken to protect the work.

12.4.12 Disturbance of new masonry

Freshly laid masonry shall not be disturbed by scaffolding, formwork or other equipment supported from it.

Individual units that are moved for any reason after initial placement shall be re-laid in fresh mortar.

12.4.13 Cavities in walls

Cavities shall be free from mortar droppings or other materials that might bridge the cavity and allow transmission of moisture. Where ducts, sleeves or pipes are laid along or across a cavity, construction shall be such that transmission of moisture is prevented.

12.4.14 Weepholes

Weepholes shall be free from any mortar or other material that will prevent their proper functioning. Weepholes shall be formed either by the inclusion of a pipe or duct at the given location or by omission of mortar (partially or fully) in the perpend joint.

12.4.15 Joint finishing

Mortar joints shall be finished as specified. The depth of raking shall not exceed the amount specified in the documents (see Clause 1.4.1).

NOTE: See Clause 4.9.2 for limitations on raking.

12.4.16 Damp-proof course and flashing (DPC)

A course upon which a sheet of damp-proof or flashing material is to be laid shall be flushed up with mortar over the full width to form an even bed beneath the damp-proof or flashing material, as necessary to prevent puncturing.

NOTES:

1 This should not be interpreted as requiring core holes to be filled or that the damp-proof or flashing material is to be sandwiched in the mortar joint.

2 This requirement is not intended to prevent the use of joint finishes such as ironed or raked joints.

Where joints in the sheet of damp-proof or flashing material cannot be avoided, the material shall be lapped or sealed against moisture penetration. The length of lapping shall be not less than the thickness of the leaf upon which the sheet is laid. Joints shall not be located at weepholes.

Damp-proofing and flashing materials shall not be breached or punctured during construction, except that they may be pierced where steel starter bars are required to pass through.

Sheet damp-proof materials shall be built in to project from both faces of the wall. On completion of the construction, the projections shall be either cut off flush with the face of the wall or turned down.

Flashings, including overflashings, shall be built-in with projections that are of sufficient size and orientation to direct the moisture from the masonry in the required manner. Overflashings shall be set to a depth of at least 15 mm into the masonry (see Clause 4.7.3).

Any render finish subsequently applied to the surface shall not be allowed to bridge a DPC or make ineffective any other moisture-protection measures.

12.4.17 Lintels

Lintels shall be installed with at least the specified bearing at each end.

NOTE: Proprietary lintels should be installed and propped in accordance with the manufacturer's instructions.

Where steel angle lintels leave a gap between the vertical leg of the angle and the supported masonry, the gap shall be packed with mortar to prevent rotation of the lintel under load.

12.5 TOLERANCES IN MASONRY

12.5.1 General

All masonry shall be built to the specified dimensions within the structural tolerances given in Table 12.1.

NOTE: Particular architectural specifications may require more stringent tolerances.

12.5.2 Measurement of bow

Where required, bow in masonry members shall be measured in accordance with Appendix F.

12.5.3 Reinforcement and tendons

Unless otherwise specified, the reinforcement and tendons shall be held during the grouting operation within the following tolerances:

(a) Across the thickness of a wall...±5 mm.

(b) Along the length of a wall, for vertical bars, or up the height of a wall, for horizontal bars...±50 mm.

(c) In a column or pier ....±5 mm.

These tolerances shall not be applied to reduce the distance between any reinforcement or tendon and the nearest bar or grout surface below the values specified in Clauses 5.9.2, 5.9.3 and 8.4.4.

TABLE 12.1

TOLERANCES IN MASONRY CONSTRUCTION

	Item	Tolerance
(a)	Horizontal position of any masonry element specified or shown in plan at its base or at each storey level	±15 mm
(b)	Relative displacement between loadbearing walls in adjacent storeys intended to be in vertical alignment	±10 mm
(c)	Maximum deviation from plumb within a storey from a vertical line through the base of the member	The lesser of ±10 mm per 3 m of height or 0.05 times the thickness of the leaf
(d)	Maximum deviation from plumb in the total height of the building (from the base)	±25 mm
(e)	Maximum horizontal or vertical deviation of a surface from a plane surface (bow) when measured as described in Clause 12.5.2	±5 mm
(f)	Deviation of bed joint from horizontal, or from the level specified or shown in elevation	±10 mm in any 10 m length, ±15 mm in total
(g)	Deviation from specified thickness of bed joint	±3 mm
(h)	Minimum perpend thickness	5 mm
(i)	Deviation from specified thickness of perpend	±10 mm max.
(j)	Deviation from specified width of cavity	±15 mm

12.6 SITE CONTROL

12.6.1 General

Site control shall be carried out to ensure compliance with the relevant provisions of this Standard.

When there is reasonable doubt that the required properties of the masonry are being achieved, verification of properties shall be carried out as follows:

(a) For masonry that is specifically designed for compressive strength, verification of compressive strength.

(b) For masonry that is specifically designed for flexural strength, verification of flexural strength.

(c) For exterior masonry in any exposure environment (see Table 5.1), verification of durability resistance.

(d) For other cases, verification of mortar composition.

When verification of properties is required, it shall be carried out in accordance with Clauses 12.6.2, 12.6.3 or 12.6.4.

NOTES:

1 Verification of properties is not a routine matter and would only be required if specified or if doubt exists that the required properties were being achieved. In the latter case, it would be by agreement between the parties.

2 The Standard does not provide specific action in the event of non-compliance; this is a contractual matter between the parties.

12.6.2 Verification of strength properties

12.6.2.1 *General*

Where verification of strength properties is required, specimens representative of the masonry shall be made up and tested for compressive strength in accordance with Appendix C or flexural strength in accordance with Appendix D.

NOTE: The type of tests and number of samples should be by agreement between the parties.

The results of tests shall be assessed for compliance with the strength requirements in accordance with Clause 12.6.2.2.

12.6.2.2 *Assessment for compliance with strength requirements*

The whole of the masonry represented by a sample shall be deemed not to comply with the strength requirements of this Standard if the test strength (mean of the sample) is less than f'_{m} or f'_{mt}.

12.6.3 Verification of durability resistance

12.6.3.1 *General*

Where verification of durability resistance is required, the masonry shall be tested for scratch index in accordance with Appendix E.

The age at testing shall be not less than 7 d, plus the following periods, as applicable:

(a) When chemically retarded mortar has been used, the unexpired period for which the mortar was retarded.

(b) The full amount of any time during which the air temperature remains below 4°C.

(c) Half the amount of any time during which the air temperature is between 4°C and 10°C.

Any cleaning of the masonry, as required for the job, shall be carried out before testing.

The resulting scratch indices shall be assessed for compliance with the durability requirements in accordance with Clause 12.6.3.2.

12.6.3.2 *Assessment for compliance with durability requirements*

The whole of the masonry represented by a sample shall be deemed not to comply with the durability requirements of this Standard if the scratch index is greater than 120% of the value for the relevant mortar class given in Table 11.2.

12.6.4 Verification of mortar composition

12.6.4.1 *General*

Where verification of mortar mix proportions is required, the mortar shall be sampled and tested in accordance with AS 2701.

The resulting proportions of total calcium oxide and soluble silica shall be assessed for compliance with the mortar composition specified in Clause 12.6.4.2.

12.6.4.2 *Assessment for compliance with mix proportions*

The whole of the masonry represented by a sample shall be deemed not to comply with the mortar requirements of this Standard if the proportions by mass of total calcium oxide and soluble silica are less than 80% of that which would be contained in a mortar of exactly the specified mix proportions or if specified by class not less than 80% of one of the mortar mix proportions of Table 11.1 for that class.

12.7 ADDITIONAL SITE CONTROL OF SPECIAL MASONRY

12.7.1 General

Special masonry shall be—

(a) sampled in accordance with Clause 12.7.2;

(b) tested for compressive strength or flexural strength, or both, in accordance with Appendix C or D, or both;

(c) controlled for target strength in accordance with Clause 12.7.3; and

(d) assessed for compliance with strength requirements in accordance with Clause 12.6.2.2.

12.7.2 Rate of sampling

For each individual type of masonry to be tested for strength during construction, the number of samples shall be not less than the greatest of—

(a) one sample per storey-height of masonry;

(b) one sample per 400 m^2 of wall area; and

(c) two samples.

Different types of masonry occur when different masonry units, or different mortars, are used. For the purposes of this Clause, accessory or supplementary masonry units that are used in small quantities shall not be considered in themselves to constitute a different type of masonry.

12.7.3 Target strength

The target values for average strength in special masonry shall be as follows:

(a) Compressive strength .... $1.4 f'_{m}$.

(b) Flexural tensile strength .... $1.7 f'_{m}$.

Where during construction of the masonry the mean strength of the last four samples (the moving average), determined in accordance with Appendix A, is less than 90% of the target average strength value, action shall be taken to investigate and rectify deficiencies, and to return the strength of the subsequently built masonry to not less than the target strength value.

12.8 GROUTED MASONRY

12.8.1 Cleaning out

Precautions shall be taken to prevent mortar from falling down cores and cavities that are to be grouted, and any mortar protruding into such cores and cavities shall be removed. Appropriate measures should be taken to inspect and satisfactorily clean out all cores and cavities that are to be grouted.

12.8.2 Grouting

Grouting shall not commence until grout spaces have been cleaned out and the mortar joints have attained sufficient strength to resist blow-outs.

The height of individual lifts in any pour shall be limited in accordance with the fluidity of grout and the suction of units to ensure that the grout can be thoroughly compacted to fill all voids and ensure bond between grout and masonry. Compaction shall be by vibration or by rodding.

On the completion of the last lift, the grout shall be topped up after a waiting period of between 10 min and 30 min and the topping vibrated or rodded so as to merge with the previous pour.

12.8.3 Sampling and testing of grout

When required, grout shall be sampled, tested and evaluated for compressive strength in accordance with the requirements for concrete given in AS 3600.

12.9 MASONRY UNDER CONSTRUCTION

12.9.1 Temporary bracing

Masonry under construction shall be braced or otherwise stabilized as necessary to resist wind and other lateral forces, in such a manner that the structural integrity of the member or structure is not impaired.

NOTE: Guidance on bracing and safe heights for construction is available from workplace safety authorities and masonry industry publications.

12.9.2 Premature loading

Masonry shall not be subjected to any load until it has gained sufficient strength to carry that load safely.

12.9.3 Physical damage

Throughout the construction period, masonry and built-in elements shall be protected to avoid damage and surface contamination.

12.9.4 Weather conditions

During construction, the top surface of the masonry shall be covered to prevent the entry of rainwater.

12.10 CLEANING

Masonry shall be cleaned and stains removed, in such a manner that the work is not damaged. Particular care shall be taken to protect adjacent work from the effects of any acid used in cleaning.

Cleaning with high-pressure water equipment shall be carried out such that mortar joints and masonry units are not damaged.

12.11 TESTING OF IN SITU MASONRY

Where it is required to test the strength properties of in situ masonry, it shall be sampled and tested for compressive or flexural strength.

NOTE: For guidance on testing of in situ masonry, see Appendix G.

APPENDIX A

ASSESSMENT OF STRENGTH VALUES FROM TEST RESULTS

(Normative)

A1 SCOPE

This Appendix sets out the method for evaluating the mean values for a given type of masonry from the results of tests carried out on specimens that are representative of that masonry.

A2 GENERAL

When the mean of a set of test results is to be determined, it shall be calculated in accordance with this Appendix. It shall be permissible to reject abnormal test results in accordance with Paragraph A3 before further calculation. For this purpose, each suspect result shall be individually assessed for rejection, beginning with the extreme value.

A3 ABNORMAL TEST RESULTS

A3.1 General

A result shall be assessed as being abnormal only if—

(a) a specific reason for its abnormality is clearly evident; or

(b) the specimen has been constructed in accordance with Paragraph C3, Appendix C, or Paragraph D3, Appendix D, and the result is outside the limits calculated in accordance with Paragraph A3.2.

A3.2 Rejection limits

The following procedure shall be used to assess rejection limits for a group of test results $X_1 \ldots X_i \ldots X_n$ where n is the number of results under consideration:

(a) Temporarily exclude the suspect value from the set of results and consider only the remaining results $X_1 \ldots X_i \ldots X_j$.

where

X_i = the ith result

j = the number of results remaining

(b) Calculate the natural logarithm, $Y_i = ln\,(X_i)$ for each remaining result.

(c) Calculate the mean of the logarithms (Y_m) and the standard deviation of the logarithms (Y_s) as follows:

$$Y_m = \frac{Y_1 + \ldots Y_i + \ldots Y_{j_r}}{j} \quad \ldots \text{A3.2(1)}$$

$$Y_s = \sqrt{\frac{\sum Y_i^2 - jY^2{}_m}{j-1}} \quad \ldots \text{A3.2(2)}$$

Calculate the limits $Y_l = Y_m - 3Y_s$ and $Y_u = Y_m + 3Y_s$

where

Y_l = the lower rejection limit

Y_u = the upper rejection limit

A3.3 Rejection

The suspect value may be rejected if the logarithm of that value, $Y = \ln(X)$, is less than Y_1 or greater than Y_u. When a value has been rejected, it shall not be considered in any subsequent calculation.

A4 MEAN

When required, the mean (X_m) of a group of test results $X_1 \ldots X_i \ldots X_n$, where n is the number of results under consideration after the elimination of any abnormal results, shall be calculated as follows:

$$X_m = \frac{X_1 + \ldots X_i + \ldots X_n}{n} \qquad \ldots \text{A4}$$

APPENDIX B

DETERMINATION OF CHARACTERISTIC VALUE

(Normative)

B1 SCOPE

This Appendix sets out the method for evaluating the characteristic value (when required) of a group of test results.

NOTE: For the purposes of site control testing, average values are used (see Appendix A).

B2 EVALUATION OF CHARACTERISTIC VALUE

The characteristic value for the property being evaluated shall be as follows:

(a) If $n < 10$, $f' = k_k f_{sp\ell}$ . . . B2(1)

(b) If $n \geq 10$, $f' = k_k f_{ksp}$ . . . B2(2)

where

n = the number of test results in the set used to evaluate f'

f' = a characteristic strength value for the type of masonry represented by the set of specimens

k_k = a characteristic value factor derived from Table B1, in which the coefficient of variation (V) is determined in accordance with Paragraph B3

$f_{sp\ell}$ = the least of the individual results in the set

f_{ksp} = the (lower) 5 percentile value for the set of test results, measured or assessed from a relative cumulative frequency distribution of that test data

The lower 5 percentile value for a given set of test results shall be found as follows:

Rank the test results $f_{sp}(1) \ldots . f_{sp}(i) \ldots . f_{sp}(n)$

$$f_{ksp} = f_{sp}\left(\frac{n+10}{20}\right) \quad \text{. . . B2(3)}$$

where

f_{sp} = the unconfined compressive or flexural strength of a masonry test specimen

For example, if $n = 25$, then $f_{ksp} = f_{sp}(1.75)$, which is obtained by interpolating between f_{sp} (1) and f_{sp}(2).

B3 COEFFICIENT OF VARIATION

For the purpose of calculating k_k the value of the coefficient of variation (V) shall be assessed as follows:

(a) Where $n < 30$

V = a value estimated for that particular type of masonry or obtained from a sufficiently large supplementary body of data (containing not less than 30 relevant test results) that is representative of that type of masonry

Unless otherwise substantiated by analysis of actual test data, the value estimated for V shall be not less than the following:

(i) For characteristic compressive strength .... 0.15.

(ii) For characteristic flexural tensile strength .... 0.30.

(c) Where $n \geq 30$

V = the coefficient of variation of the set of test results being evaluated

TABLE B1

CHARACTERISTIC VALUE FACTOR

Number of test results (n)	**Characteristic value factor (k_k)**					
	Coefficient of variation (V)					
	0	**0.10**	**0.20**	**0.30**	**0.40**	**0.50**
1	1.00	0.72	0.52	0.37	0.27	0.19
2	1.00	0.77	0.59	0.46	0.35	0.27
5	1.00	0.84	0.71	0.60	0.51	0.43
10	1.00	0.91	0.82	0.74	0.67	0.61
20	1.00	0.94	0.87	0.81	0.75	0.69
50	1.00	0.96	0.92	0.88	0.84	0.80
100	1.00	0.97	0.94	0.92	0.89	0.86
200	1.00	0.98	0.96	0.94	0.92	0.90
500	1.00	0.99	0.97	0.96	0.95	0.94
1000	1.00	1.00	1.00	1.00	1.00	1.00

NOTES:

1 Linear interpolation may be used.

2 k_k estimates the lower 5 percentile value of the population with a confidence of 0.75.

APPENDIX C

METHOD OF TEST FOR COMPRESSIVE STRENGTH

(Normative)

C1 SCOPE

This Appendix sets out the method for determining the compressive strength of masonry by test.

C2 NUMBER OF SPECIMENS PER SAMPLE

For each sample of masonry to be tested, the number of specimens for compressive strength shall be three or more.

C3 PREPARATION OF SPECIMENS

C3.1 General

Specimens for compression testing shall be obtained by one of the following methods:

(a) Specimens constructed as piers in accordance with Paragraphs C3.3 to C3.5.

(b) Undamaged portions remaining from piers that have been tested as beams in accordance with Paragraph D7, Appendix D.

(c) Specimens sampled from existing walls in accordance with Paragraph G2. Appendix G.

Specimens shall be clearly identified with the masonry they represent.

Test specimens in accordance with Items (a) and (b) shall be—

(i) built adjacent to the masonry that they represent but in a location where they will be protected from damage during the curing period; and

(ii) made from the materials used in the construction that the sample represents and, as far as practicable, using similar techniques and under similar conditions to those applicable during construction.

The mason chosen to construct the test specimens shall be representative, in terms of quality of work, of all those who are engaged in constructing the masonry.

C3.2 Number of courses in specimens

The number of courses in each specimen shall be such that—

(a) the height-to-width ratio of the pier is between two and five; and

(b) there are not less than three courses, except when ‘three courses’ violates condition (a), in which case it shall be permissible to use two courses in the specimen.

C3.3 Stack-bonded piers

Stack-bonded piers for specimens shall be constructed as follows:

(a) Set out the first course of units on a firm, flat supporting surface, leaving 5 mm to 10 mm between heads.

(b) Place a mortar bed over all units, using full bedding for solid or cored units and face-shell bedding for hollow units.

(c) Wait 30 s before placing the next course of units on the mortar bed in stack bond. Units having frogs in the bed face shall be laid with the frogs up. Cut off surplus mortar but do not strike or tool the joints.

(d) Repeat Steps (b) and (c) until the required number of courses have been laid.

(e) On completion of laying, strike, tool or rake the joints in a manner identical with that used on the masonry represented by the piers. Where specimens are made from units having frogs, fill the uppermost frogs with mortar, and trowel to a smooth finish flush with the top of the units.

(f) Cut the perpends with a wire or trowel without disturbing the piers.

AAC units shall be cut into halves and both halves of each unit shall be used in preparing test specimens.

C3.4 Additional requirements for grouted masonry specimens

For specimens representing grouted hollow unit masonry, the procedure, additional to that in Paragraph C3.3, shall be as follows:

(a) Clean out the cores so that no mortar extrusions remain on the internal surfaces and the cores are free of mortar droppings.

(b) Fill the cores with grout and compact in layers during filling by rodding, finishing to a height 25 mm above the top surface of the unit.

(c) One hour after filling, strike the surplus grout off level with the top surface of the unit.

C3.5 Curing

The piers forming the sample, except those sampled from an existing wall in accordance with Paragraph C3.1(c), shall be fully wrapped in a single vapour-proof sheet and left undisturbed until transported for testing.

C4 AGE AT TEST

Testing of specimens of plain masonry shall be carried out when the age of the specimen after construction is 7 d or, in constructed masonry, as soon as practicable after 7 d, plus, for both cases, the following periods, as applicable:

(a) When chemically retarded mortar has been used, the unexpired period for which the mortar was retarded.

(b) The full amount of any time during which the air temperature remains below 4°C.

(c) Half the amount of any time during which the air temperature is between 4°C and 10°C.

For specimens of grouted masonry, the age at test shall be 28 d after filling the cavities with grout, extended as for Items (b) and (c) only.

C5 TRANSPORTATION

Specimens shall be transported to the testing place within 24 h before the due testing time.

C6 TEST APPARATUS

The apparatus for compression testing of prisms shall be a compression testing machine whose accuracy meets the requirements for Grade A or Grade B machines as given in AS 2193. The upper platen of the machine shall be spherically seated.

C7 TEST PROCEDURE

C7.1 General

The procedure for testing each specimen shall be as follows:

(a) Place the specimen between strips of plywood, 4 mm to 6 mm thick, whose length and width exceed the bed face dimensions of the specimen by 15 mm to 25 mm, or, for hollow units loaded on the face shells, strips whose width is within 1 mm of the thickness of the thinnest part of the face shells. Use fresh strips for each test.

(b) Distribute the load evenly over the whole top and bottom surfaces of the specimen and apply the load steadily and without shock at a rate between 350 kN/min and 700 kN/min until the specimen fails.

(c) Record the load at failure to the nearest 5 kN.

C7.2 Calculation of compressive strength of specimen

The compressive strength of the specimen shall be—

$$f_{sp} = k_a \left(\frac{F_{sp}}{A_d} \right) \qquad \text{. . . C7.2}$$

where

f_{sp} = the unconfined compressive strength of the specimen, in megapascals

k_a = the aspect ratio factor for the specimen, derived from Table C1.

The height shall be the overall height of the prism specimen, and the thickness as defined in Note 2 to Table C1.

F_{sp} = the total load at which the specimen fails, in newtons

A_d = the design cross-sectional area of the specimen, in square millimetres (see Clause 4.5.6)

A_d is based on work size dimensions except that in the case of in situ testing, the actual length shall be taken

TABLE C1

ASPECT RATIO FACTORS (k_a)

Height-to-thickness ratio	0	0.40	1.00	5.0 or more
Aspect ratio factor (k_a)	0	0.50	0.70	1.00

NOTES:

1 Linear interpolation may be used.

2 The thickness used for evaluating the height-to-thickness ratio shall be—

(a) for prisms made from solid or cored masonry units or grouted hollow units—the overall width of the unit; and

(b) for prisms made from hollow masonry units—the minimum thickness of the face shell of the unit.

C8 CALCULATION OF TEST STRENGTH OF THE SAMPLE

The test strength of the sample shall be the mean, calculated in accordance with Appendix A, of the test values obtained from the testing of all the specimens accepted as constituting that sample.

C9 REPORTING OF RESULTS

The test report shall include the following information:

(a) Date of construction of test specimens.

(b) Date of test.

(c) Identification of work represented by the sample.

(d) A statement that the compression testing has been carried out in accordance with the specifications in this Appendix. In the case of masonry sampled in accordance with Paragraph C3.1(c), which have not been wrapped, this shall be noted.

(e) Number of specimens tested.

(f) Type and number of units forming each prism and, for hollow unit specimens, whether grouted or not.

(g) Composition of mortar in the specimens.

(h) For grouted specimens, the mix and strength of the grout.

(i) Individual results determined from each specimen tested, including—

  (i) the work-size dimensions and design cross-section area of the specimen;

  (ii) the magnitude of the failure load; and

  (iii) the compressive strength of the specimen.

(j) Test strength of the sample to the nearest 0.1 MPa.

(k) Detailed information about any result that is considered to be abnormal, including the reasons for the non-inclusion of any abnormal result in the assessment of the strength of the sample.

(l) Reference to this test method, i.e. Appendix C of AS 3700.

NOTE: For tests on in situ specimens, the length of the specimen should be recorded and reported.

APPENDIX D

METHOD OF TEST FOR FLEXURAL STRENGTH

(Normative)

D1 SCOPE

This Appendix sets out the method for determining the flexural strength of masonry perpendicular to the bed joints by test.

Two means of testing are given, of either—

(a) the bond wrench test method (Paragraph D6); or

(b) the beam test method (Paragraph D7).

D2 NUMBER OF SPECIMENS PER SAMPLE

For each sample of masonry to be tested, the number of specimens tested for flexural strength shall be six or more.

D3 PREPARATION OF SPECIMENS

D3.1 General

Specimens for flexural testing by the bond wrench method shall be obtained by one of the following methods:

(a) Specimens constructed as piers in accordance with Paragraphs D3.3 to D3.5.

(b) Portions remaining undamaged from piers constructed in accordance with Paragraphs D3.3 to D3.5, and tested as beams in accordance with Paragraph D7.

(c) Specimens sampled from existing walls in accordance with Paragraph G2, Appendix G.

Specimens for flexural testing by the beam test method shall be constructed as piers in accordance with Paragraphs D3.3 to D3.5.

Specimens shall be clearly identified with the masonry they represent.

Test specimens in accordance with Items(a) and (b) above shall be—

(i) built adjacent to the masonry that they represent but in a location where they will be protected from damage during the curing period; and

(ii) made from the materials used in the construction that the sample represents and, as far as practicable, using similar techniques and under similar conditions to those applicable during construction.

The mason chosen to construct the test specimens shall be representative, in terms of quality of work, of all those who are engaged in constructing the masonry.

D3.2 Number of courses in a specimen

The number of courses in a specimen shall be as follows:

(a) For flexural testing by the bond wrench method, a number between two and seven.

(b) For flexural testing by the beam test method, either nine or seven; except that if the height of the masonry unit is equal to or greater than its width (that is, the thickness of the leaf when the unit is laid in the wall), four courses may be used.

D3.3 Stack-bonded piers

Stack-bonded piers for specimens shall be constructed as follows:

(a) Set out the first course of units on a firm, flat supporting surface, leaving 5 mm to 10 mm between heads.

(b) Place a mortar bed over all units, using full bedding for solid or cored units and face-shell bedding for hollow units.

(c) Wait 30 s before placing the next course of units on the mortar bed in stack bond. Units having frogs in the bed face shall be laid with the frogs up. Cut off surplus mortar but do not strike or tool the joints.

(d) Repeat Steps (b) and (c) until the required number of courses have been laid.

(e) On completion of laying, strike, tool or rake the joints in a manner identical with that used on the masonry represented by the piers.

(f) Cut the perpends with a wire or trowel without disturbing the piers.

AAC units shall be cut into halves and both halves of each unit shall be used in preparing test specimens.

D3.4 Additional preparation requirements for grouted masonry specimens

For specimens representing grouted hollow unit masonry, the procedure, additional to that in Paragraph D3.3, shall be as follows:

(a) Clean out the cores so that no mortar extrusions remain on the internal surfaces and the cores are free of mortar droppings.

(b) Fill the cores with grout and compact in layers during filling by rodding, finishing to a height 25 mm above the top surface of the unit.

(c) One hour after filling, strike the surplus grout off level with the top surface.

D3.5 Curing

The piers forming the sample, except those sampled from an existing wall in accordance with Paragraph D3.1(c), shall be fully wrapped in a single vapour-proof sheet and left undisturbed until transported for testing.

D4 AGE AT TEST

Testing of specimens of plain masonry shall be carried out when the age of the specimen after construction is 7 d or, in constructed masonry, as soon as practicable after 7 d, plus, for both cases, the following periods, as applicable:

(a) When chemically retarded mortar has been used, the unexpired period for which the mortar was retarded.

(b) The full amount of any time during which the air temperature remains below 4˚C.

(c) Half the amount of any time during which the air temperature is between 4˚C and 10˚C.

For specimens of grouted masonry, the age at test shall be 28 d after filling the cavities with grout, extended as for Items (b) and (c) only.

D5 TRANSPORTATION

Specimens shall be transported to the testing place within 24 h of the due testing time.

D6 FLEXURAL STRENGTH BY BOND WRENCH TEST METHOD

D6.1 General

The apparatus used in the bond wrench test shall comply with Paragraph D6.2. The flexural strength shall be calculated in accordance with Paragraph D6.6.

The apparatus shall be calibrated in accordance with Paragraph D6.4. The test shall be carried out in accordance with Paragraph D6.5.

D6.2 Principle

A flexural moment is applied to the test joint by means of four gripping blocks attached to the wrench. The blocks are positioned at the quarter points along the length of the masonry unit on the tension face, and on the compression face as dimensioned in Figure D1.

D6.3 Apparatus

The apparatus for the bond wrench test shall comprise the following:

(a) A bond wrench that applies a bending moment to the joint to be tested in the specimen.

(b) A retaining frame into which the test specimen is clamped.

(c) A means of applying and measuring load to determine the flexural stress at failure to an accuracy within 0.01 MPa.

The gripping blocks shall satisfy the following:

(i) The length of the gripping blocks shall be 30 mm.

(ii) The thickness of the gripping blocks normal to the face of the masonry unit shall be at least 20 mm.

(iii) The height of the gripping blocks in contact with the masonry unit on the tension side shall be at least 25 mm.

(iv) The height of the gripping blocks on the tension side shall be such that a gap of at least 15 mm is maintained between the top surface of the top unit and the loading arm.

NOTE: This is to ensure that no part of the bond wrench other than the gripping blocks comes into contact with the specimen during the test.

(v) The gripping blocks shall be located as far away as possible from the top interface of the joint under test. At least a distance of 20 mm shall be maintained between the compression gripping block and the top interface of the joint under test.

The unit below the mortar joint under test shall be clamped at least 25 mm below the bottom interface. Care shall be taken to clamp only the bottom unit but not any joints below it.

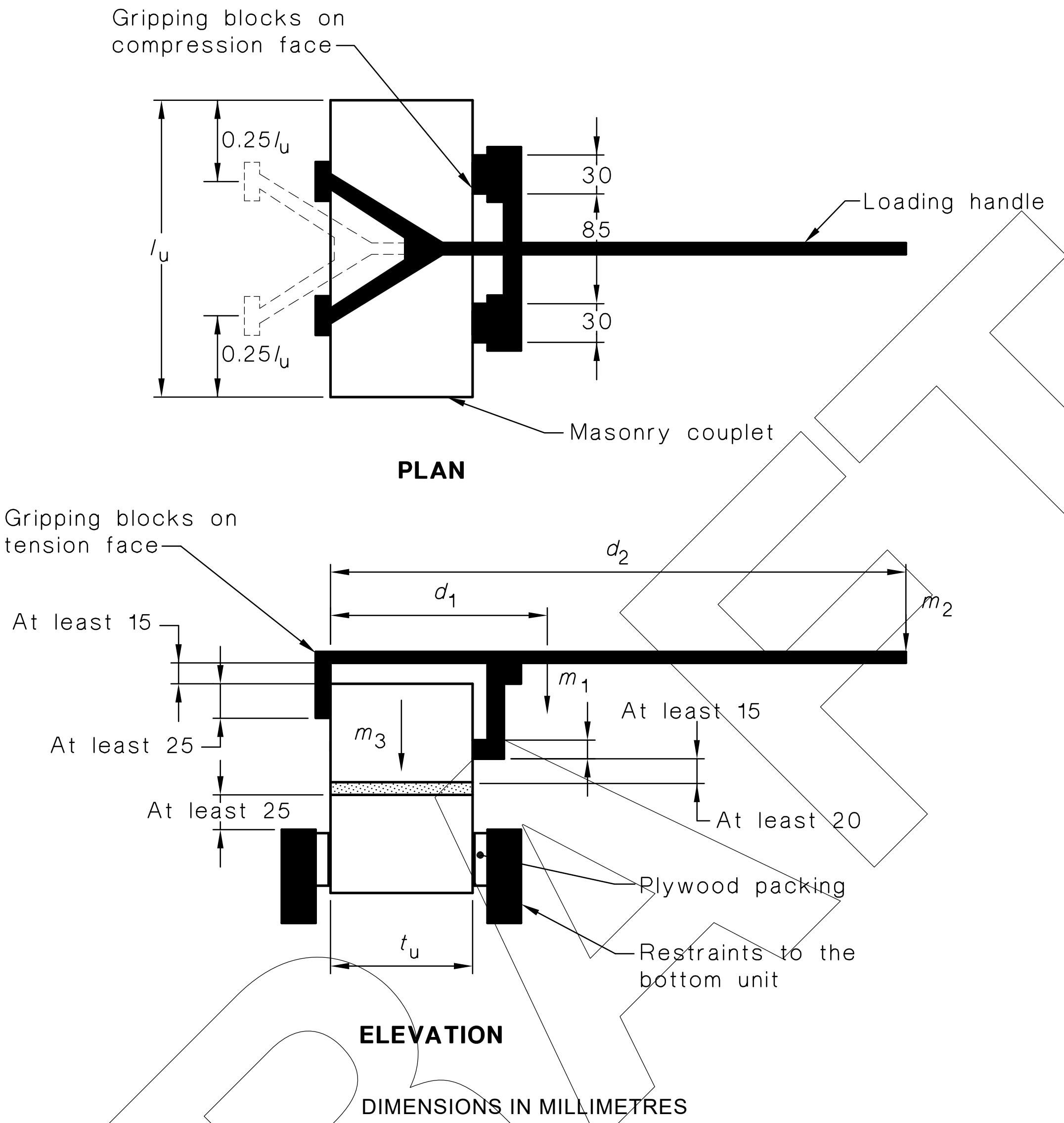


FIGURE D1 SCHEMATIC DIAGRAM OF THE BOND WRENCH SET-UP

D6.4 Calibration of apparatus

The bond wrench (see Figure D1) shall have the following parameters determined:

(a) m_1 = the mass of the bond wrench, to within ±25 g

(b) d_1 = the distance, when the clamp faces are separated by the work-size width of the masonry units, from the inside face of the tension gripping block to the centre of gravity of the bond wrench, to within ±2 mm

(c) d_2 = the distance, when the clamp faces are separated by the work-size width of the masonry units, from the inside face to the tension gripping block to the loading handle, to within ±2 mm

D6.5 Test procedure

D6.5.1 *Suspended container method*

The procedure for testing each specimen using the bond wrench shall be as follows:

(a) Securely retain the lower portion of the specimen in a retaining frame or, for specimens in constructed masonry, by other suitable means. If the joint to be tested is raked on one or both faces of the specimen, locate the face that contains the raking, or the greater depth of raking, on the compression side. Use thin strips of plywood on the clamp faces where crushing might otherwise occur. Add counterbalancing weights to the retaining frame if required, to ensure the stability of the whole apparatus when fully loaded.

(b) Clamp the bond wrench to the top masonry unit of the specimen in a suitable manner (as shown in Figure D1) to prevent crushing of the clamped unit, and adjust the wrench so that its arm is horizontal.

(c) Suspend the empty loading container on the loading notch of the wrench, with the bottom of the container not more than 100 mm above the floor or other support level. Load the container at an even rate of between 10 kg/min and 15 kg/min, using dry sand or other appropriate material, until the top portion of the specimen separates from the restrained lower portion.

(d) Determine the following (see Figure D1):

(i) Mass of the container and contents together (m_2), to within ±100 g.

(ii) Mass of the top masonry unit of the specimen and any mortar adhering thereto (m_3), to within ±100 g.

(iii) Width (t_u) of the top masonry unit, to within ±2 mm.

D6.5.2 *Testing machine method*

Provided it is capable of measuring the load to the accuracy described in Paragraph D6.2(c), a load-measuring machine may be substituted for the suspended container for the determination of m_2.

D6.6 Calculation of flexural strength of specimen (f_{sp})

The flexural strength of each test joint of the specimen shall be calculated from the following equation:

$$f_{sp} = \left(\frac{M_{sp}}{Z_d}\right) - \left(\frac{F_{sp}}{A_d}\right) \quad \text{. . . D6.6}$$

where

f_{sp} = the flexural strength of the specimen, in megapascals

M_{sp} = the bending moment about the centroid of the bedded area of the test joint at failure, in newton millimetres

$$= 9.81 m_2\left(d_2 - \frac{t_u}{2}\right) + 9.81 m_1\left(d_1 - \frac{t_u}{2}\right)$$

Z_d = the section modulus of the design cross-sectional area (A_d) of the member, in cubic millimetres

F_{sp} = the total compressive force on the bedded area of the tested joint, in newtons

$= 9.81\ (m_1 + m_2 + m_3)$

A_d = the design cross-sectional area of the member (see Clause 4.5.6), in millimetres squared

m_1, m_2, m_3 = the masses of components used in flexural strength testing, in kilograms

d_1 = the distance from the inside edge of the tension gripping block to the centre of gravity, in millimetres

d_2 = the distance from the inside edge of the tension gripping block to the loading handle, in millimetres

t_u = the width of the masonry unit, in millimetres

NOTE: The calculation of A_d and Z_d are based on work-size dimensions except for the length in the case of in situ testing.

D7 FLEXURAL STRENGTH BY BEAM TEST

D7.1 Test procedure

The procedure for testing each specimen using the beam test method shall be as follows:

(a) Determine the mass of the specimen to within ±1 kg.

(b) Lay the specimen on its side as a beam with the faces of the units uppermost, and support the beam symmetrically on two straight steel bars placed so as to provide bearing under the centre of each end masonry unit in the beam. The length of the steel supporting bars shall be not less than the length of the masonry unit, and the width of their support to the beam shall be not more than 15 mm.

(c) Measure the span (L) between the centres of the supporting bars to within ±2 mm.

(d) Place two steel loading bars on the top surface of the beam with their centres one-third of the span apart and located symmetrically about the centre of the span to an accuracy of ±2 mm. The length of the steel loading bars shall be not less than the length of the masonry units in the beam, and the width of their bearing on the beam shall be not more than 15 mm.

If required to prevent the beam and the applied load from rocking, a wedge may be placed under one end of one supporting bar and over one end of one loading bar.

(e) Load the specimen through the loading bars, applying the load symmetrically and without shock, and increasing it steadily until the specimen fails.

(f) If the applied load is in the form of masonry units or similar materials, precautions shall be taken to prevent the applied load from falling and causing damage when failure occurs. Alternatively, a calibrated testing machine may be used to apply the load to the beam.

(g) Determine the mass of the applied load at failure to within ±1 kg.

D7.2 Calculation of flexural strength of specimen

The flexural strength of the specimen shall be calculated from the following equation:

$$f_{sp} = \frac{k_{sp} M_{sp}}{Z_d} \quad \text{. . . D7.2}$$

where

f_{sp} = the flexural strength of the specimen, in megapascals

k_{sp} = masonry pier strength factor

= 1.00 for a 4-high pier

= 1.20 for a 7-high pier

= 1.25 for a 9-high pier

M_{sp} = the mid-span bending moment at failure, in newton millimetres

Z_d = the section modulus of the design cross-sectional area (A_d) of the member, in cubic millimetres

NOTE: The calculation of Z_d is based on work-size dimensions.

D8 CALCULATION OF TEST STRENGTH OF THE SAMPLE

The test strength of the sample shall be the mean, calculated in accordance with Appendix A, of the test values obtained from the testing of all the specimens accepted as constituting that sample.

D9 REPORTING OF RESULTS

The test report shall include the following information:

(a) Date of construction of test specimens.

(b) Date of test.

(c) Identification of work represented by the sample.

(d) A statement that flexural strength testing has been carried out in accordance with the specifications in this Appendix. In the case of masonry sampled in accordance with Paragraph D3.1(c), which have not been wrapped, this shall be noted.

(e) Number of specimens tested or number of joints tested.

(f) Type and number of units forming each prism and, for hollow unit specimens, whether grouted or not.

(g) Composition of mortar in the specimens.

(h) For grouted specimens, the mix and strength of the grout.

(i) Individual results determined from each specimen tested, including—

  (i) the work-size dimensions and design section modulus of the specimen;

  (ii) the magnitude of the failure load; and

  (iii) the test strength of the specimen.

(j) Test strength of the sample to the nearest 0.01 MPa.

(k) Detailed information about any result that is considered to be abnormal, including the reasons for non-inclusion of any abnormal result in the assessment of the strength of the sample.

(l) Reference to this test method, i.e. Appendix D of AS 3700.

NOTE: For tests on in situ specimens, the length of the specimen should be recorded and reported.

APPENDIX E

DURABILITY TESTING

(Normative)

E1 SCOPE

This Appendix sets out a method for testing the durability of masonry mortar intended for in situ use. The mechanical test is based on a controlled scratching of the mortar surface. The penetration into the mortar is then measured and is named the scratch index.

NOTES:

1 Durability of masonry mortar is primarily determined by the mortar ingredients and the mixing process, but it is also influenced by the masonry units and workmanship factors. The measurement of a durability index in situ, as provided by the mechanical test, is therefore suitable when the durability of a mortar in particular circumstances is to be assessed.

2 The test simulates and accelerates the physical forces that can cause mortar degradation in service in typical Australian environments. It is not intended to simulate deterioration that might occur under freeze-thaw conditions.

3 The method of test for durability is designed to be used for assessment of compliance of mortar less than 12 months old. If used on mortar more than 12 months old it can only provide an assessment of potential durability from that time onward.

E2 GENERAL

The apparatus used in the mechanical durability test shall comply with Paragraph E4. The test shall be carried out in accordance with Paragraph E5.

The scratch index shall be calculated from the test results in accordance with Paragraph E6, and reported in accordance with Paragraph E7.

E3 SAMPLING

Where mortars are being assessed in accordance with Clause 11.4.3—

(a) a minimum of three results shall be obtained; and

(b) results shall be obtained from joints constructed using the same mortar composition, masonry units and joint finishing as those intended for the finished construction.

NOTES:

1 Joints for testing may be either in existing construction or in test prisms.

2 The term 'result' used in this Clause refers to a scratch index obtained as the average of five separate measurements in accordance with Paragraphs E4 and E5.

Where testing is carried out because masonry quality is in doubt (see Clause 12.6), each distinct area of masonry, such as an individual wall or a specific number of masonry courses, for which the masonry characteristics are identifiably different, shall be tested separately.

NOTE: The number of scratch index results for each identified area should be agreed by the parties concerned.

For each identified area where multiple results of scratch index are required, these results shall be obtained at locations selected at random throughout the area. The results shall apply only to the identified area.

The area identified as applicable to a single scratch index result shall be—

(i) not less than 0.5 m^2; and

(ii) not greater than 10 m^2.

E4 APPARATUS

The apparatus used in the mechanical durability test shall consist of a scratch tool with a rotating handle to apply a controlled scratch force through a probe. It shall incorporate the following:

(a) An adjustment to accommodate recessed, tooled and protruding joints over the range of 15 mm below to 5 mm above the adjacent brick surface.

(b) A means of measuring the penetration of the probe into the mortar surface, relative to the adjacent brick surfaces, to an accuracy of within 0.025 mm.

NOTE: A schematic diagram of the scratch tool is shown in Figure E1.

The probe shall have the following characteristics:

(i) Diameter 6.1 ±0.2 mm.

(ii) Blade thickness 1.0 ±0.1 mm.

(iii) The end ground flat with the cross-section as shown in Figure E2.

(iv) A range of movement, from the point of initial contact, of 10 mm into the mortar joint.

The spring driving the probe shall have a stiffness of 0.8 ±0.1 N/mm.

A record shall be kept of the number of scratch index measurements made with the apparatus and the tip shall be replaced after every 200 index measurements (maximum).

NOTE: Research has shown that variability in measurement increases with tip wear. This could result in the acceptance of mortar that does not comply with the Standard.

E5 PROCEDURE

Each determination of scratch index shall be made from five separate measurements distributed approximately uniformly along the mortar joint to be tested. No measurement shall be made within 10 mm of the location of another measurement.

The procedure for each measurement shall be as follows:

(a) Place the scratch tool at the measurement location and adjust the probe for the required pre-load when in contact with the mortar surface. The initial compression of the spring when the probe contacts the mortar surface shall result in a force of 15.6 ±1.0 N.

(b) With the tool held firmly in contact with the adjacent brick surfaces, turn the probe through two full turns using light rotational pressure on the handle without exerting axial force on the probe. Zero the tool's measuring device.

NOTE: This initial procedure is to dislodge any loose surface grains.

(c) With the scratch tool held firmly in contact with the adjacent brick surfaces, turn the probe through five full turns using light rotational pressure on the handle without exerting axial force on the probe.

(d) Record the approximate location of the measurement and the penetration of the scratch tool in millimetres, to the nearest 0.025 mm.

NOTE: Recording to the nearest 0.001 inches and subsequent conversion to millimetres is acceptable.

E6 CALCULATION OF SCRATCH INDEX

The scratch index shall be calculated as the average of five separate measurements with the scratch tool.

E7 REPORTING OF RESULTS

The test report shall include the following information:

(a) Location and date of construction of the masonry.

(b) Date of test.

(c) Identification and location of each mortar joint for which a scratch index has been determined and for each mortar joint—

  (i) the approximate location of each measurement within the joint;

  (ii) the individual scratch tool measurements; and

  (iii) the calculated scratch index for the joint.

(d) Reference to this test method, i.e. Appendix E of AS 3700.

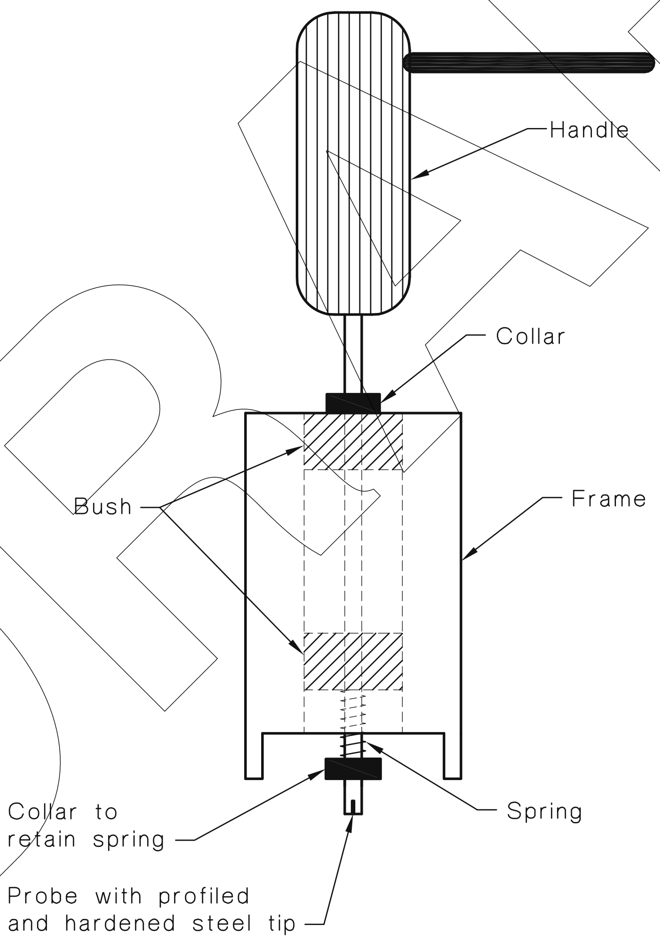
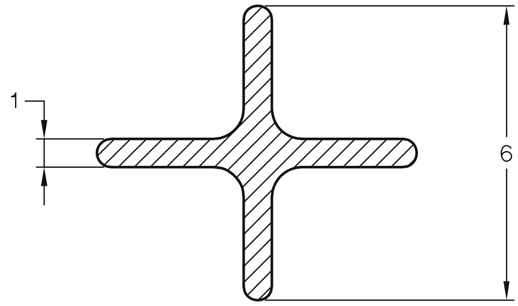


FIGURE E1 SCHEMATIC DRAWING OF THE SCRATCH TOOL



DIMENSIONS IN MILLIMETRES

FIGURE E2 SCHEMATIC CROSS-SECTION OF THE SCRATCH TOOL PROBE

APPENDIX F

METHOD OF MEASUREMENT OF BOW

(Normative)

F1 SCOPE

This Appendix sets out the method of measuring the bow of a masonry member.

F2 PROCEDURE

Bow shall be measured using a modified straightedge and measuring rule, as shown in Figure F1. Each surface shall be assessed separately by placing the straightedge horizontally and vertically to obtain the maximum deviation.

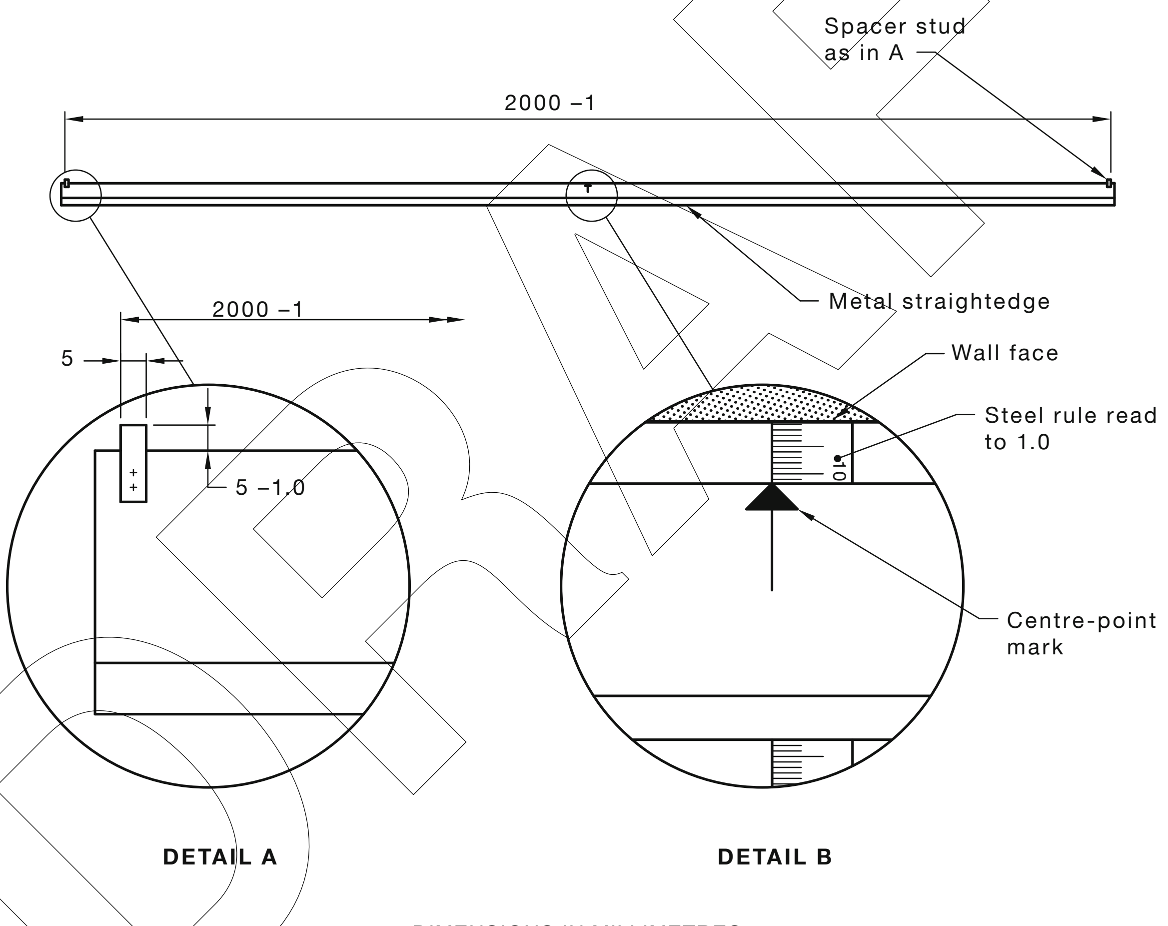


FIGURE F1 BOW-MEASURING DEVICE

APPENDIX G

STRENGTH TESTING OF IN SITU MASONRY

(Informative)

G1 GENERAL

Where it is required to test strength properties of in situ masonry, it may be tested for either compressive or flexural strength.

G2 PREPARATION FOR TESTING

Select part of a wall that is representative of the masonry under test.

For compression strength testing, prisms containing at least three courses and having an aspect ratio (height-to-width ratio) of between two and five should be cut from the wall by methods that ensure no damage to the samples.

NOTE: If a specimen with three courses exceeds a height to width ratio of five, two courses may be used.

For flexural strength testing, masonry above the test area should be carefully removed, as necessary, to provide access for the bond wrench, and perpends should be cut through so that the only resistance to flexure is provided by the bed joint to be tested. These operations should use methods that ensure no reduction in strength to the joints to be tested.

G3 TESTING

The methods for compressive and flexural strength testing are set out in Appendices C and D respectively.

APPENDIX H

GUIDANCE ON THE USE OF MASONRY IN RESTORATION WORK AND MASONRY CONSTRUCTED USING SQUARE-DRESSED NATURAL STONE

(Informative)

H1 SCOPE

This Appendix provides some guidance on the design and construction of masonry incorporating lime–sand mortar (Type M1) and/or square-dressed natural stone.

H2 GENERAL

For masonry used in restoration work (i.e. masonry constructed using Type M1 mortar), and masonry constructed using square-dressed natural stone, the performance requirements set out in Section 2 apply, except for durability requirements. Where there is a need to use M1 mortar for additions to a heritage building, the design should allow for the possibly increased rate of weathering or degradation that might occur in exposed mortar joints.

H3 RESTRICTION ON THE USE OF TYPE M1 MORTARS

Type M1 mortars (i.e. sand–lime mortars), do not possess suitable durability properties and, therefore, cannot generally be used to construct masonry in accordance with this Standard. They are permitted to be used only in masonry being constructed to restore existing buildings that were initially built using these mortars (see Clause 11.4.1).

Special approval or certification should be obtained to construct a new building using Type M1 mortars for cases where this is deemed desirable; for example, the construction of a new building as part of a reconstruction of a complex of period buildings.

H4 MASONRY CONSTRUCTED OF SQUARE-DRESSED NATURAL STONE UNITS

The scope of the Standard includes masonry constructed with square-dressed natural stone, but no material properties are given.

Guidance on the specification of square-dressed natural stone units and the design and construction of masonry using them is given in BS 5390.

H5 DESIGN PROPERTIES

In general, the design properties of the masonry should be based on full bedding at the mortar joints.

H6 CONSTRUCTION

In restoration projects, the sands used in the mortar and the joint tooling techniques should be chosen to match those used in the existing construction so that the appearance of the mortar joints and finished masonry are similar.

APPENDIX I

ISO 9223 CORROSIVITY CATEGORIES AND RELATIONSHIP TO DURABILITY CLASS

(Informative)

I1 GENERAL

The corrosivity of the atmospheric environment affects the durability of components used in masonry. Estimation of the corrosivity zone for atmospheric exposure can be linked to the durability class for built-in components described in this Standard. This Appendix describes the methods which can be used to develop a reasonable estimation of the atmospheric exposure for those locations where actual corrosivity measurements have not been carried out and includes deemed to comply solutions from AS/NZS 2699 (series) (wall ties, connectors and accessories and lintels and shelf angles).

Estimates of the corrosivity of the atmospheric environment can be developed from historical data, by observation of long term performance of similar structures in the local environment, or by direct determination. However, other than for locations exposed to severe salt spray or strong chemical pollution, estimation of the location corrosivity using the techniques described in this Appendix will be satisfactory. If a more comprehensive determination of the corrosivity category is required, the method described in ISO 9223 can be used.

I2 THE RELATIONSHIP BETWEEN ATMOSPHERIC CORROSIVITY CATEGORIES AND DURIBILITY CLASS OF MASONRY COMPONANTS

I2.1 General

External atmospheric environments are classified in ISO 9223 into six categories based on corrosion rates of metals. The following relates these categories to conditions in Australia. These descriptions are developed from AS 4312. If there is any doubt as the corrosivity category, professional advice should be sought.

I2.2 Category C1 (very low)

The only external environments in Australia are some alpine regions although generally these environments will extend into Category C2. Category C1 is normally associated with internal spaces with air conditioning such as offices and public buildings.

I2.3 Category C2 (low)

External environments in this category include dry, rural areas as well as other regions remote from the coast or sources of pollution. Most areas of Australia and New Zealand beyond at least 50 km from the sea are in this category, which can however extend as close as 1 km from seas that are relatively sheltered and quiet. Typical areas occur in arid and rural inland regions, most inland cities and towns such as Canberra, Ballarat, Toowoomba, Alice Springs, and suburbs of cities on sheltered bays, such as Melbourne and Hobart. Proximity to the coast is an important factor.

I2.4 Category C3 (medium)

This category mainly covers coastal areas with low salinity. The extent of the affected area varies significantly with factors such as winds, topography and vegetation. Around sheltered seas, such as Port Philip Bay, Category C3 extends beyond about 50 m from the shoreline to a distance of about 1 km inland. For a less sheltered bay or gulf, such as near Adelaide, this category extends from 100 m from the shoreline to about 3 km to 6 km inland. Along ocean front areas with breaking surf and significant salt spray, it extends from about 1 km inland to between 10 km to 50 km inland, depending on the strength of prevailing winds and topography. Much of the metropolitan areas of Wollongong, Sydney, Newcastle, and the Gold Coast are in this category. In South Australia, the whole of the Yorke Peninsula falls within this or a more severe category, and in the south-east of the state, from Victor Harbour to the Victorian border, this category extends between 30 km and 70 km inland. Such regions are also found in urban and industrial areas with low pollution levels and although uncommon in Australia and New Zealand, exist for several kilometres around major industries, such as smelters and steelworks. Micro-environmental effects, such as result from proximity to airports and sewage treatment works, may also place a site into this category.

I2.5 Category C4 (high)

This category occurs mainly on the coast. Around sheltered bays, Category C4 extends up to 50 m inland from the shoreline. In areas with rough seas and surf, it extends from about 200 m inland to about 1 km inland. As with Categories C2 and C3, the extent depends on winds, wave action and topography. Industrial regions may also be in this category, but in Australia these are only likely to be found within 1.5 km of the plant.

I2.6 Category C5 (very high)

This category is common on the beachfront in regions of rough seas and surf beaches. The region can extend inland for about 200 m. (In some areas of Newcastle, for example, it extends more than half a kilometre from the coast.) This category may also be found in aggressive industrial areas, where the environment may be acidic with a pH of less than 5.5.

I2.7 Category CX (extreme)

These regions are found at some surf beach shoreline regions with very high salt deposition. Such corrosion rates would also be found in severe acidic industrial environments. This category is included in C5 category for the purposes of this standard.

NOTE: CX is not covered in the 2008 edition of AS 4312 and should be considered special (R5) for the purposes of AS 3700.

If a site is considered to be in more than one category, for example an industry on the coast in a tropical region, then the material or coating should be capable of resisting the most severe of the environments involved.

The exposure locations for the durability class R1 to R5 given in this standard are similar to the ISO corrosivity categories but there are slight differences. Table I1 and Figure I1 shows the relationship between the two as they relate to distance from the coast.

If the location of the structure has been defined according to AS 4312, then the required durability class for fixtures can be determined using Table I1.

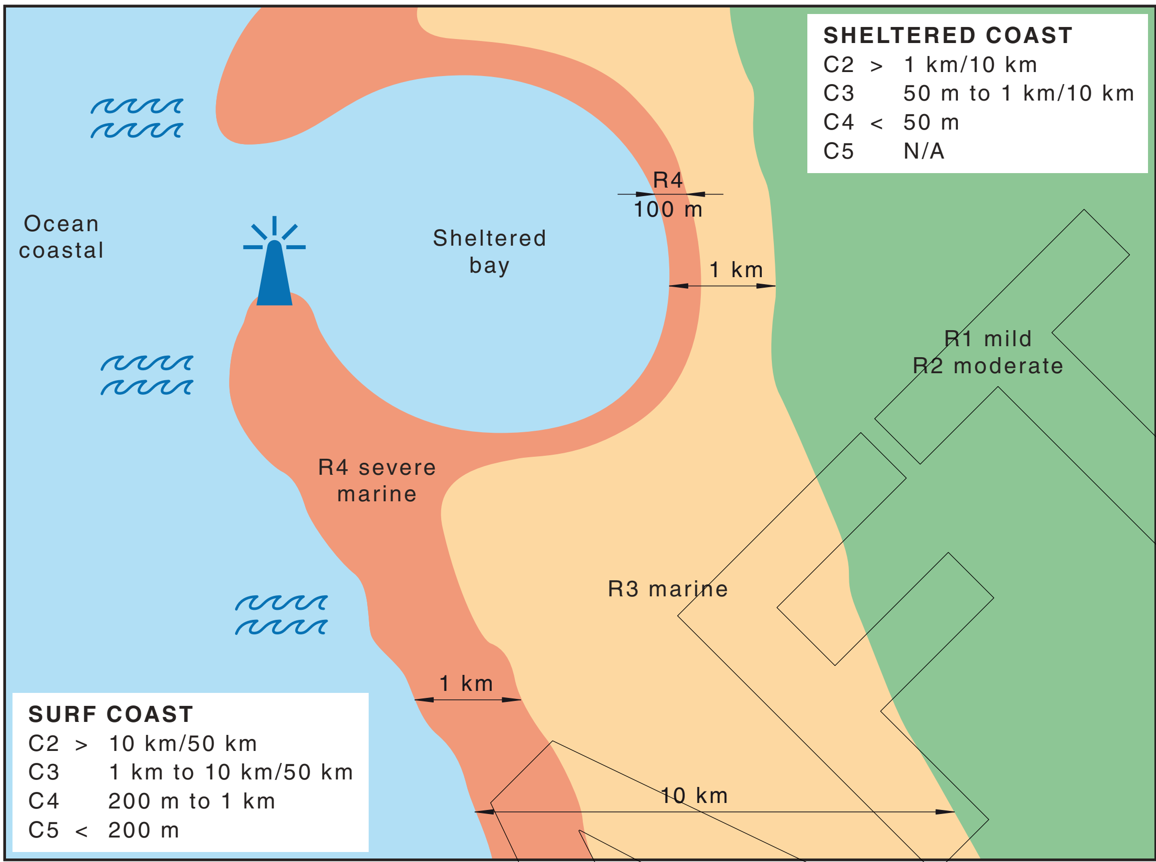


FIGURE I1 SIMPLIFIED CORROSIVITY CATEGORY LOCATIONS IN AUSTRALIA

TABLE I1

TYPICAL RELATIONSHIP OF DURABILITY CLASS OF MASONRY (AS 3700) AND CORROSIVITY CATEGORY FOR EXPOSED STEEL BUILDING ELEMENTS (AS 4312) BASED ON DISTANCE FROM A SURF COAST (SIMPLIFIED)

AS 3700			AS 4312		
Durability class	Typical distance from		Corrosivity category	Typical distance from	
	Surf coast	Sheltered coast		Surf coast	Sheltered coast
R1 R2	>10 km	>1 km	C2	>50 km	>10 km
				10 to 50 km	1 to 10 km
R3	1 km to 10 km	100 m to 1 km	C3	1 km to 10 km	50 m to 1 km
R4	<1 km	<100 m	C4	200 m to 1 km	<50 m
			C5	< 200 m	N/A

NOTES:

1 Durability class R0 is applicable only for interior locations which are not affected by the external environment and is not covered here. Similarly, Category C1 is typically found in internal spaces with air conditioning such as offices and public buildings.

2 A region of 10 to 50 kilometres from a surf coast (or 1 km to 10 km from a sheltered coast) can be in AS 4312 C2 or C3 category, but R3 durability fixtures should normally be selected.

3 Durability class R4 covers both C4 and C5 corrosivity categories for marine exposure (see Table 2). Category C4 and C5 can also be categorised by increasing levels of pollution. There are very few polluted sites in Australia and specialist advice should be sought in these cases.

4 Durability class R5 will need specialist advice for design solutions.

TABLE I2

SOLUTIONS FOR WALL TIES SHOWN IN AS/NZS 2699.1 FOR VARIOUS DURABILITY CLASSES

Durability class of built-in components to AS 3700		Solution for material or protective coating specification for masonry wall ties from AS/NZS 2699.1
R1 R2	Mild moderate	Continuous hot dip galvanized steel sheet to AS 1397 Z600 (minimum average coating thickness ≥42 μm)
		Hot dip galvanized after manufacture from bare steel sheet, wire or bar to AS/NZS 4680 HDG300 (minimum average coating thickness ≥42 μm)
R1, R2, R3	Mild moderate, Marine	Hot dip galvanized steel wire to AS/NZS 4534/SW470Z (minimum average coating thickness ≥65 μm)
R3	Marine	Hot dip galvanized after manufacture from bare steel sheet, wire or bar to AS/NZS 4680 HDG470 (minimum average coating thickness ≥65μm)
		Stainless steel ASTM A240 304L (UNS S30403)
R4	Severe marine industrial	Stainless steel ASTM A240 316L (UNS S31603)
R5	Specific	See Note 2

NOTES:

1 Durability means the capability of a wall tie to perform its function over a specified period.

2 The design solution for R5 (Specific) will depend on the environment to which the wall tie is exposed and the specifier should consult an expert to develop a solution.

3 AS/NZS 2699.1 provides a method for determining the coating thickness for galvanized ties.

4 For all durability classes from R0 through to R3 AS/NZS 2699.1 deems satisfactory only one galvanised wire tie material, carrying a minimum 470 g/m^2 of zinc. Wall ties made from this material must also be coloured red.

5 Stainless grades 304 and 316 can be substituted for 304 L and 316 L respectively.

6 A wall tie deemed to comply with a Durability class is also deemed to comply with lower Durability classes.

7 Alternative solutions to this Table can be developed to meet the requirements of AS/NZS 2699.1 and should have a demonstrated satisfactory service history in the application or be evaluated using tests which are applicable to the material and design life of the component and that demonstrate compliance to the design life.

TABLE I3

SOLUTIONS FOR CONNECTORS AND ACCESSORIES SHOWN IN AS/NZS 2699.2 FOR VARIOUS DURABILITY CLASSES

<table>
<tr><th colspan="2">Durability class of built-in components to AS 3700</th><th>Solution for material or protective coating specification for connectors and accessories from AS/NZS 2699.2</th></tr>
<tr><td rowspan="3">R1
R2</td><td rowspan="3">Mild moderate</td><td>Continuous hot dip galvanized steel sheet to AS 1397 Z600 (minimum average coating thickness ≥42 μm)</td></tr>
<tr><td>Hot dip galvanized after manufacture from bare steel sheet, wire or bar to AS/NZS 4680 HDG300 (minimum average coating thickness ≥42 μm)</td></tr>
<tr><td rowspan="2">Hot dip galvanized steel wire to AS/NZS 4534/SW470Z (minimum average coating thickness ≥65 μm)</td></tr>
<tr><td rowspan="3">R3</td><td rowspan="3">Marine</td></tr>
<tr><td>Hot dip galvanized after manufacture from bare steel sheet, wire or bar to AS/NZS 4680 HDG470 (minimum average coating thickness ≥65 μm)</td></tr>
<tr><td>Stainless steel ASTM A240 304L (UNS S30403)</td></tr>
<tr><td>R4</td><td>Severe marine industrial</td><td>Stainless steel ASTM A240 316L (UNS S31603)</td></tr>
<tr><td>R5</td><td>Specific</td><td>See Note 2</td></tr>
</table>

NOTES:

1 Durability means the capability of a connector or accessory to perform its function over a specified period.

2 The design solution for R5 (Specific) category will depend on the environment to which the connector or accessory is exposed and the specifier should consult an expert to develop a solution.

3 AS/NZS 2699.2 provides a method for determining the coating thickness of galvanized connectors and accessories.

4 For all durability classes from R0 through to R3 AS/NZS 2699.2 deems satisfactory only one galvanised wire material, carrying a minimum 470 g/m^2 of zinc. Wire made from this material must also be coloured red.

5 Stainless grades 304 and 316 can be substituted for 304L and 316L respectively.

6 A connector or accessory deemed to comply with a Durability class is also deemed to comply with lower Durability classes.

7 Alternative solutions to this Table can be developed to meet the requirements of AS/NZS 2699.2 and should have a demonstrated satisfactory service history in the application or be evaluated using tests which are applicable to the material and design life of the component and that demonstrate compliance to the design life.

TABLE I4

SOLUTIONS FOR LINTELS AND SHELF ANGLES FROM AS/NZS 2699.3 FOR VARIOUS DURABILITY CLASSES

Durability class of built-in components to AS 3700		Solution for material or protective coating specification for lintels and shelf angles from AS/NZS 2699.3
R1 R2	**Mild moderate**	Hot dip galvanized to AS/NZS 4791 ILG300 (minimum average coating thickness ≥42 µm)
		Hot dip galvanized to AS/NZS 4680 HDG300 (minimum average coating thickness ≥42 µm)
		Painted to AS/NZS 2312.1/IZS1 or IZS2 (minimum average coating thickness ≥75 µm)
R3	**Marine**	Hot dip galvanized to AS/NZS 4680 HDG600 (minimum average coating thickness ≥85 µm)
		Painted to AS/NZS 2312.1/IZS3 or IZS4 (minimum average coating thickness ≥125 µm)
		Stainless steel ASTM A240 304L (UNS S30403)
R4	**Severe marine industrial**	Duplex coating (hot dip galvanized plus paint coating) consisting of AS/NZS 4680 HDG 600 (minimum average coating thickness ≥85µm) plus AS/NZS 2312.2 System 4D or 4I (minimum average coating thickness ≥350µm)
		Stainless steel ASTM A240 316L (UNS S31603)
R5	**Specific**	See Note 2

NOTES:

1 Durability means the capability of a lintel or shelf angle to perform its function over a specified period. All protective coatings are likely to require minor maintenance to maintain aesthetic appearance and/or corrosion protection during this time.

2 The design solution for R5 (Specific) category will depend on the environment to which the lintel or shelf angle is exposed and the specifier should consult an expert to develop a solution.

3 AS/NZS 2699.3 provides a method for determining the coating thickness of galvanized, painted and duplex lintels and shelf angles.

4 Additional decorative coats can be applied to a lintel or shelf angle, but the coating should not be considered for durability calculations.

5 Any lintel or shelf angle with a coating that is modified by cutting, welding or where the coating is otherwise damaged must have the coating restored to at least equivalent protection provided by the original coating to maintain the durability. Refer to the manufacturer or supplier for details of the appropriate repair method.

6 Stainless grades 304 and 316 can be substituted for 304L and 316L respectively.

7 A lintel or shelf angle deemed to comply with a Durability class is also deemed to comply with lower Durability classes.

8 Alternative solutions to this Table can be developed to meet the requirements of AS/NZS 2699.3 and should have a demonstrated satisfactory service history in the application or be evaluated using tests which are applicable to the material and design life of the component and that demonstrate compliance to the design life.

BIBLIOGRAPHY

AS

1397 Continuous hot-dip metallic coated steel sheet and strip—Coatings of zinc and zinc alloyed with aluminium and magnesium

2601 Demolition of structures

4312 Atmospheric corrosivity zones in Australia

AS/NZS

2311 Guide to the painting of buildings

4534 Zinc and zinc/aluminium-alloy coatings on steel wire

4680 Hot-dip galvanized (zinc) coatings on fabricated ferrous articles

ISO

9223 Corrosion of metals and alloys—Corrosivity of atmospheres—Classification, determination and estimation

ASTM

A240 Standard Specification for Chromium and Chromium-Nickel Stainless Steel Plate, Sheet, and Strip for Pressure Vessels and for General Applications

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PREPARATION OF AUSTRALIAN STANDARDS

Australian Standards are prepared by a consensus process involving representatives nominated by organizations drawn from all major interests associated with the subject. Australian Standards may be derived from existing industry Standards, from established international Standards and practices or may be developed within a Standards Australia technical committee.

During the development process, Australian Standards are made available in draft form at all sales offices and through affiliated overseas bodies in order that all interests concerned with the application of a proposed Standard are given the opportunity to submit views on the requirements to be included.

The following interests are represented on the committee responsible for this draft Australian Standard:

Australasian (Iron and Steel) Slag Association

Australian Building Codes Board

Australian Institute of Building Surveyors

Building Designers Association of Australia

Cement Concrete and Aggregates Australia—Cement

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

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Australian Standards are prepared by committees of experts from industry, governments, consumers and other relevant sectors. The requirements or recommendations contained in published Standards are a consensus of the views of representative interests and also take account of comments received from other sources. They reflect the latest scientific and industry experience. Australian Standards are kept under continuous review after publication and are updated regularly to take account of changing technology.

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